PRELIMINARY INVESTIGATION

OF

SAN TAN BRIDGE

WBC

Wheeler Brooks Coffeen

PRELIMINARY INVESTIGATION

OF

SAN TAN BRIDGE

Prepared for

Maricopa County
Project No. 71108

Registered Professional Engineer (Civil)
CERTIFICATE NO.
11051
DINESH J.
DOSHI
Date Signed 10.31.78
ARIZONA, U.S.A.

Dinesh J. Doshi

Prepared by

WBC Consultants, Inc.
2444 E. University Drive, Ste. 100
Phoenix, Arizona 85034

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WBC Consultants, Inc.
Engineering • Surveying • Planning

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Phoenix, Arizona 85034 • (602) 275-5400

Tucson • Denver • Albuquerque • Los Angeles

October 31, 1978

Mr. Francis Lathrop, P.E.
Maricopa County, Highway Dept.
3333 W. Durango
Phoenix, Arizona 85009

Re: Project No. 71108
San Tan Bridge

Gentlemen:

Submitted herewith is our report which summarizes the feasibility study for a proposed bridge on RWCD floodway @ San Tan Road crossing in accordance with the subject contract.

Please do not hesitate to contact the undersigned if you have any questions regarding this report.

We look forward to serving you with the detailed design of this bridge.

Very truly yours,

WBC CONSULTANTS, INC.

By: D. J. Doshi.
Dinesh J. Doshi, P.E.
Project Manager

DJD/ke

Registered Professional Engineer (Civil)
CERTIFICATE NO. 9628
ARTHUR N. BROOKS
DATE SIGNED 10/31/78
Arizona U. S. A.

Reviewed by:

A N Brooks

Arthur N. Brooks, P.E.
Sr. Vice President

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Registered Professional Engineer (Civil)
CERTIFICATE NO.
11051
DINESH J.
DOSHI
Date Signed [illegible]
ARIZONA, U.S.A.

Dinesh J. Doshi

SCOPE OF WORK

The preliminary design, to establish engineering feasibility, and economic selection of sub-structure and superstructure for a bridge at San Tan Road over RWCD floodway channel is presented herewith.

Basic design criteria were provided by the owners as follows:

STRUCTURAL DATA

1. Loading: HS-20
2. Bridge Lanes = 2
3. Width of the roadway = 28'
4. Sidewalk on one side = 4'
5. Fence along one side
6. Curb along both sides

HYDRAULICS DATA

1. Channel bottom width = 200'
2. Design flow = 8700 cfs
3. Channel side slope = 3:1
4. Mean water depth = 5.95'
5. Mean water velocity = 6.71 cfs
6. Flow line of channel at bridge centerline = 1227.84 ft.
7. High water elevation at bridge centerline = 1233.84 ft.
8. Side slope at bridge = 2:1
9. Side slope transition in each direction = 150' (from 2:1 to 3:1)

10. Minimum pier spacing = 30 ft.

11. Minimum clearance between the bottom of bridge superstructure and maximum water surface = 1 ft.

12. Maximum head loss through the bridge = 3.5 ft.

13. To assure maximum flexibility in the final design of the floodway, the vertical location of the footings should make allowances for the channel bottom to be lowered if needed by 2.0 ft.

The portion of Phase I,A; Design Services of the contract between the County and WBC Consultants completed here includes:

1. Preliminary survey

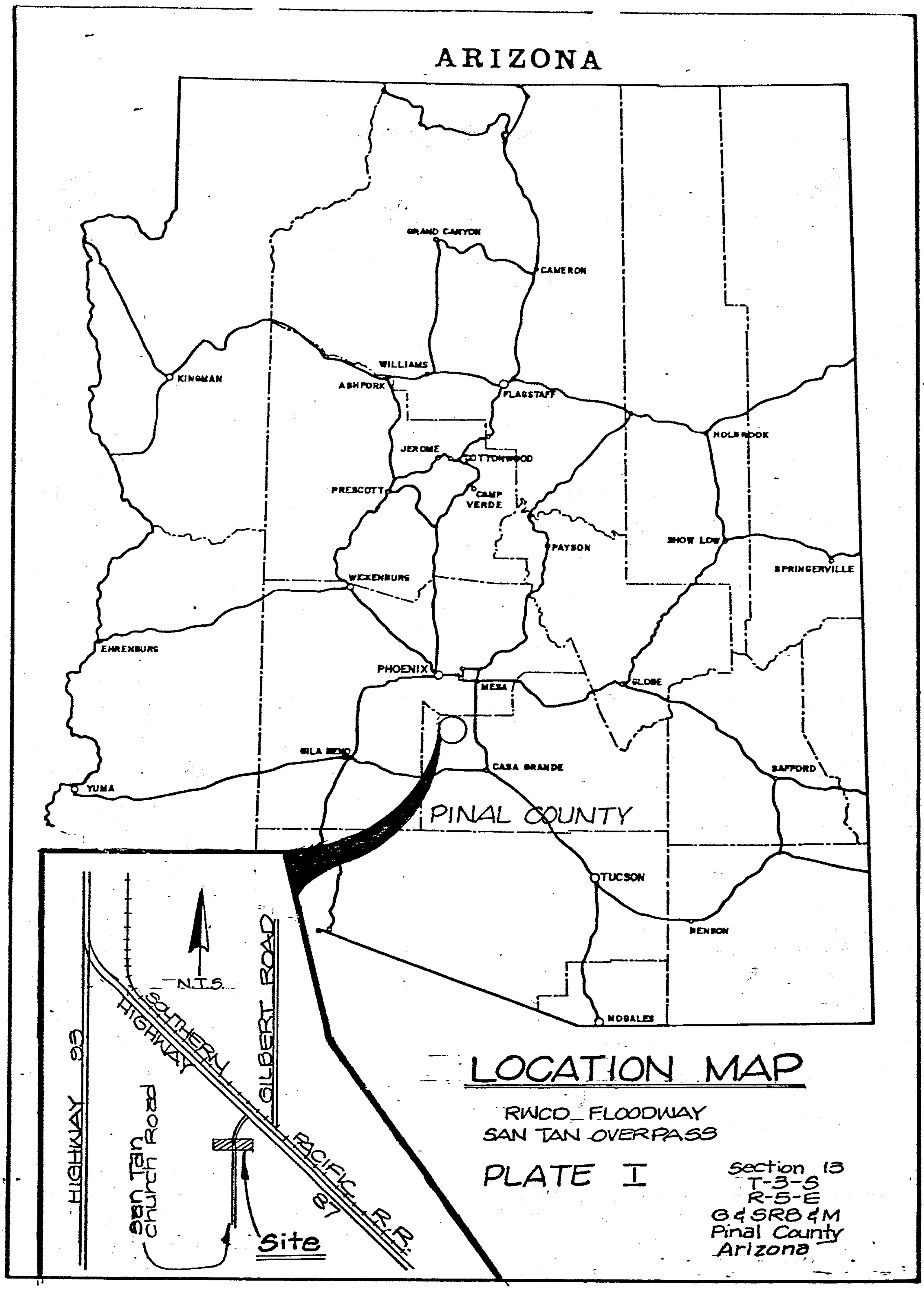
2. Preliminary soils investigation

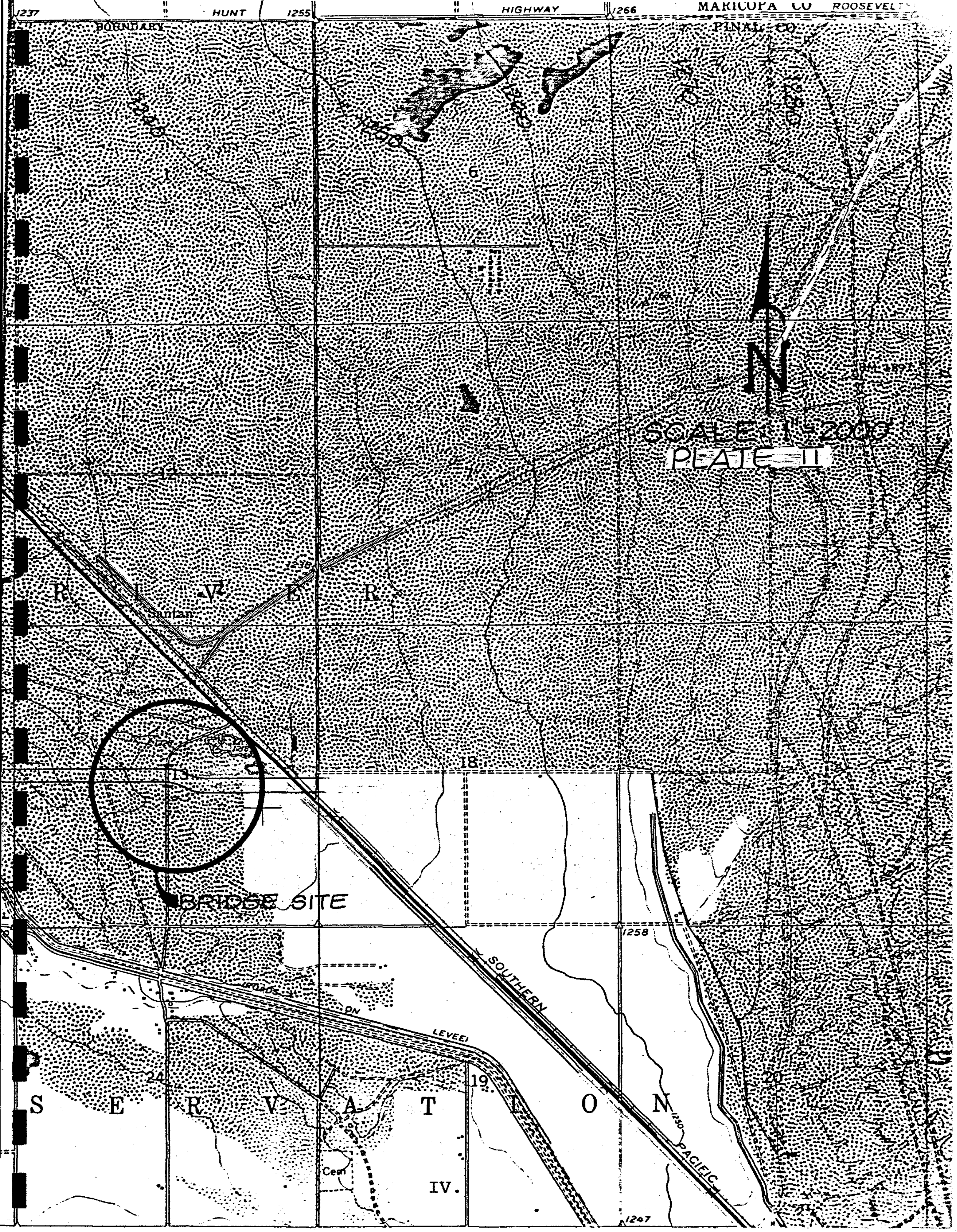
3. Structural investigation report

4. Bridge hydraulics

Preliminary soils investigation prepared by ETL, Inc. was forwarded previously and is included here along with the Addendum No. 1.

Up on the County's written approval of this report, work will proceed on remaining portion of Phase I, A; Design Services.


ARIZONA
GRAND CANYON
CAMERON
KINGMAN
WILLIAMS
ASHFORK
FLAGSTAFF
HOLBROOK
JEROME
COTTONWOOD
PRESCOTT
CAMP VERDE
PAYSON
SHOW LOW
SPRINGERVILLE
WICKENBURG
EHRENBURG
PHOENIX
MESA
GLOBE
GILA BEND
CASA GRANDE
SAFFORD
YUMA
PINAL COUNTY
TUCSON
BENSON
NOGALES
N.T.S.
HIGHWAY 93
SOUTHERN PACIFIC R.R.
HIGHWAY 87
GILBERT ROAD
San Tan Church Road
Site
LOCATION MAP
RWCD FLOODWAY
SAN TAN OVERPASS
PLATE I
Section 13
T-3-S
R-5-E
G & SRB & M
Pinal County
Arizona



1237
HUNT
1255
HIGHWAY
1266
MARICOPA CO
ROOSEVELT
BOUNDARY
PINAL CO
N
SCALE: 1"=2000'
PLATE II
R I V E R
Santan
BRIDGE SITE
1258
SOUTHERN
LEVEE
S E R V A T I O N
PACIFIC
Cem
IV.
1247

DISCUSSION

A location map, indicating the general vicinity of the proposed San Tan Baptist Church Road Bridge, is shown in Plate I.

The bridge is at the crossing of San Tan Baptist Church Road (GRIR 127) over the proposed Williams-Chandler WPP, RWCD floodway, Reach I at centerline station 1235 + 39.27 $\pm$; located within the Gila River Indian Reservation in Section 13, T3S, R5E, G. & S.R.B. & M., Pinal County, Arizona

The general topography is undeveloped desert, and is shown on U.S.G.S. Topographic Map included as Plate II.

Soil and Foundation Investigation

From the surface to approximately 10 ft. in depth the soil is moderately dense, silty clays. Underlying soils, generally extending to a depth of 20 to 50 feet are silty sands of medium density. Excavation of the proposed channel can generally be accomplished with conventional earth moving equipment. Proposed channel slopes of 3:1 appear to provide adequate safety against failure. However, scour and erosion could have adverse effects on long term stability. In the proximity of the bridge the protection of the channel banks is required.

Shallow foundation did not prove to be feasible because of potential structural settlement. The other foundation types considered were driven H piles and drilled caissons.

The preliminary soils report is included in Appendix A along with the Addendum No. 1 as prepared by ETL, Inc.

Structural Investigation

Steel and timber were not considered to be feasible. Concrete was considered suitable along with the grade 40 and 60 steel for reinforcement.

Drilled caisson foundation for substructure was found less expensive when compared with the other alternative of H piles.

Several combinations of spans were reviewed and the structural analysis was prepared for 30, 35, 40, 45 and 60 foot interior spans.

Four deck types considered for super structure included the following:

1. Cast in place slab

2. Precast prestressed voided slab

3. Precast prestressed box girders

4. Rolled steel beam with composite or noncomposite deck slab

Comparison of the above deck types when combined with the circular column bent and drilled caisson foundation indicated essentially the same cost.

Based on the inspection needs and the reduction in constriction time a precast prestressed voided slab deck for the super structure with six spans of 39' center to center was recommended.

The preliminary investigation of feasible structural systems for super structure and sub-structure is included in Appendix B, prepared by Magadini-Alagia Associates.

Hydraulics and Hydrology Investigation

The design flow for the bridge is 8700 cfs based on the flood of 100 year frequency. The hydrology and verification of design flow is beyond the scope of work of this contract. However, the channel design data were verified prior to proceeding with the hydraulic analysis of the proposed bridge. Hydraulic analysis was performed for three different combinations of interior span of 30', 40', and 60'. The analysis included here in Appendix "C" is for only the selected 40foot span alternative.

Bridge backwater analysis was performed in accordance with "Hydraulics of Bridge Waterways: HYDRAULIC DESIGN SERIES NO.1, by Joseph N. Bradley, published by U.S. Department of Transportation/Federal Highway Administration.

The flow is termed as Type I or subcritical flow.

The crossing is considered normal because of its alignment at approximately 90° to the general direction of the flow during high water. The backwater expression can be obtained by applying the conservation of energy principle between Section 1 at the point of maximum backwater upstream from the bridge and Section 4 at a point down stream from the bridge at which normal stage has been reestablished.

The length of the bridge is set at 247 feet, the same as the channel bottom width plus slopes. The waterway is restricted only in the side slope from 3:1 at channel to 2:1 in 150 foot transition on each side of the bridge. This constriction will cause backwater and raise the water level by approximately 6 inches.

Head losses because of constriction due to the piers and the side slopes will be less than the maximum allowable of 3.5 feet as determined by the momentum method.

The subsoil at the channel bottom will be fine silty sand which will have a high scour potential at the design flow of 8700 cfs. It will be necessary to protect the channel bed plus the side slopes with grouted rock riprap.

CONCLUSIONS & RECOMMENDATIONS

A concrete bridge at San Tan Road can be constructed over RWCD floodway channel.

The desired channel bottom elevation of 1227.85 can be achieved at the bridge. The channel embankment slope of 3:1 will be stable but some sort of protection for erosion and scour for long term stability should be considered. Excavation of the channel can generally be accomplished with conventional earth moving equipment.

The recommended superstructure for the bridge is precast prestressed voided slab deck with six spans of 39'0" center to center of abutments and piers. For substructure, three column bents at each pier location with drilled in caissons are recommended. The desired side slope of 2:1 at bridge can be attained with a gradual transition to 3:1 in 150 feet on each side of the bridge.

The rise in water surface due to backwater will be in the range of 0.5 feet. The head losses through the bridge will be less than the maximum allowable 3.5 feet. Scour protection of the channel bed and erosion protection of side slope is recommended. Cemented rock riprap is recommended based on preliminary soils investigation. For the purpose of cost estimation 2 foot thickness of grouted rock riprap is assumed for about 100 feet on each side of the bridge with cut off walls. The actual thickness will be determined after the final detailed soils investigation.

To assure maximum flexibility in the final design of the floodway the vertical location of the footings and/or entire bridge can be designed to make allowances for the channel bottom to be lowered if needed by 2.0 feet.

The total cost of the bridge for budget purpose is estimated to be $574,000.

PRELIMINARY BUDGET COST ESTIMATE

FOR
SAN TAN BRIDGE

1. Earthwork

1.1 Clearing & Grubing

$\frac{(350x300+200x300)}{43560} \times \$500 =$ $ 1,900

1.2 Excavation

$(200 \times 12 + 30 \times 12)350 \times \frac{1.50}{27} =$ 53,600

1.3 Finish Grading

4 acres x $1,000 = 4,000

1.4 Onsite Surplus Disposal

36,000 c.yrds x $1.00 = 36,000

Sub-total Earthwork $ 95,500

2. Roadway Construction

2.1 Existing Pavement Removal

$\frac{450 \times 24}{9.0} \times \$2.00 =$ $ 2,400

2.2 Subgrade Preparation

$2 \times 100 \times 30 \ \frac{\$1.00}{9} =$ 700

2.3 Pavement of Bridge Approaches

$2 \times 100 \times 30 \times \frac{\$6.80}{9} =$ 4,600

2.4 Pavement On the Bridge

$240 \times 30 \times \frac{\$1.60}{9} =$ 1,300

2.5 Catch Basins

$4 \times \$950 =$ $3,800

Sub-total Construction $ 12,800

3. Bridge Structure

3.1 Superstructure

$240 \times 36 \times \$18 =$ $156,000

3.2 Substructure

$240 \times 36 \times \$50 =$ 44,000

3.3 Curb

$3.0 \times 1.0 \times 240 \times 2 \ \frac{240}{27} =$ 12,800

3.4 Handrail & Chain Link Fence

$240 \times \$10$ 2,400

Sub-total Structure $215,200

4. Embankment Protection

4.1 Side Slopes & Channel Bottom Riprap

$350(2 \times 22.5 + 200) \times 2.0$ ft. $\frac{\$30}{27} =$ $130,000

Sub-total Embankment Protection $130,000

TOTAL COST	$453,500
CONTINGENCY 10%	45,500
OVERHEAD & PROFIT 15%	75,000
	$574,000

APPENDIX "A"

ENGINEERS TESTING LABORATORIES, INC.
3737 East Broadway Road
P. O. Box 21387
Phoenix, Arizona 85036
(602) 268-1381

W.C.B. Consultants, Inc.
2444 E. University Dr., Suite 100
Phoenix, Arizona

24 October 1978

Attention: Dinesh Doshi, P.E.

Project: Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan Road Crossing
Pinal County, Arizona

Job No. 812-467
Addendum No. 1
Inv. No. 112-1138-2

In accordance with your request and/or the request of Magadini-Alagia, this firm has accomplished a review of the preliminary soils report relative to the following:

1. Feasibility of the site for development of the proposed bridge structure.

2. Feasibility of increasing the slope configuration under the bridge from 3:1 to 2:1 (horizontal to vertical).

3. Preliminary design recommendations regarding the use of driven H-piles as an alternate foundation system.

4. Preliminary design recommendations regarding drilled caisson imbedment depth necessary to resist a 20 kip lateral load applied 13 feet above channel grade line.

5. General comments relative to slope and bed protection against scour.

Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan
Road Crossing
Pinal County, Arizona
Job No. 812-467
Addendum No. 1

The following comments are presented in answer to the preceding:

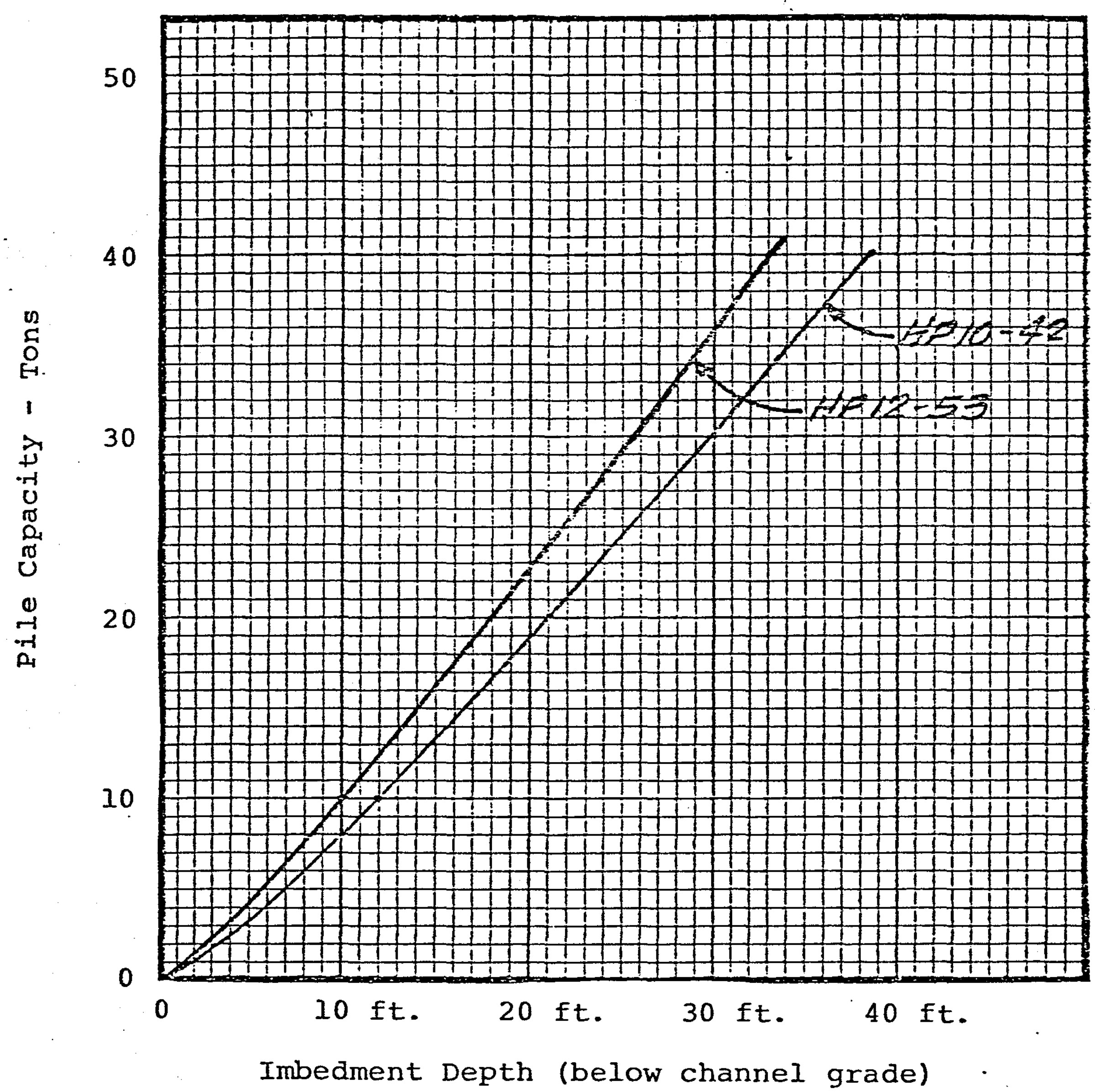
1. The proposed site and soil conditions are adequate for providing support for foundation elements and as such the site is considered suitable for the proposed bridge development.

2. Based upon preliminary shear test data, the proposed slope of 2:1 (horizontal to vertical) will be adequate for the channel slopes underlying the bridge structure. However, final design should include a review of abutment foundation elements as they would pertain to slope stability.

3. The following design charts present the relationship between driven H-piling imbedment depth and preliminary pile capacity for both pier and abutment conditions.

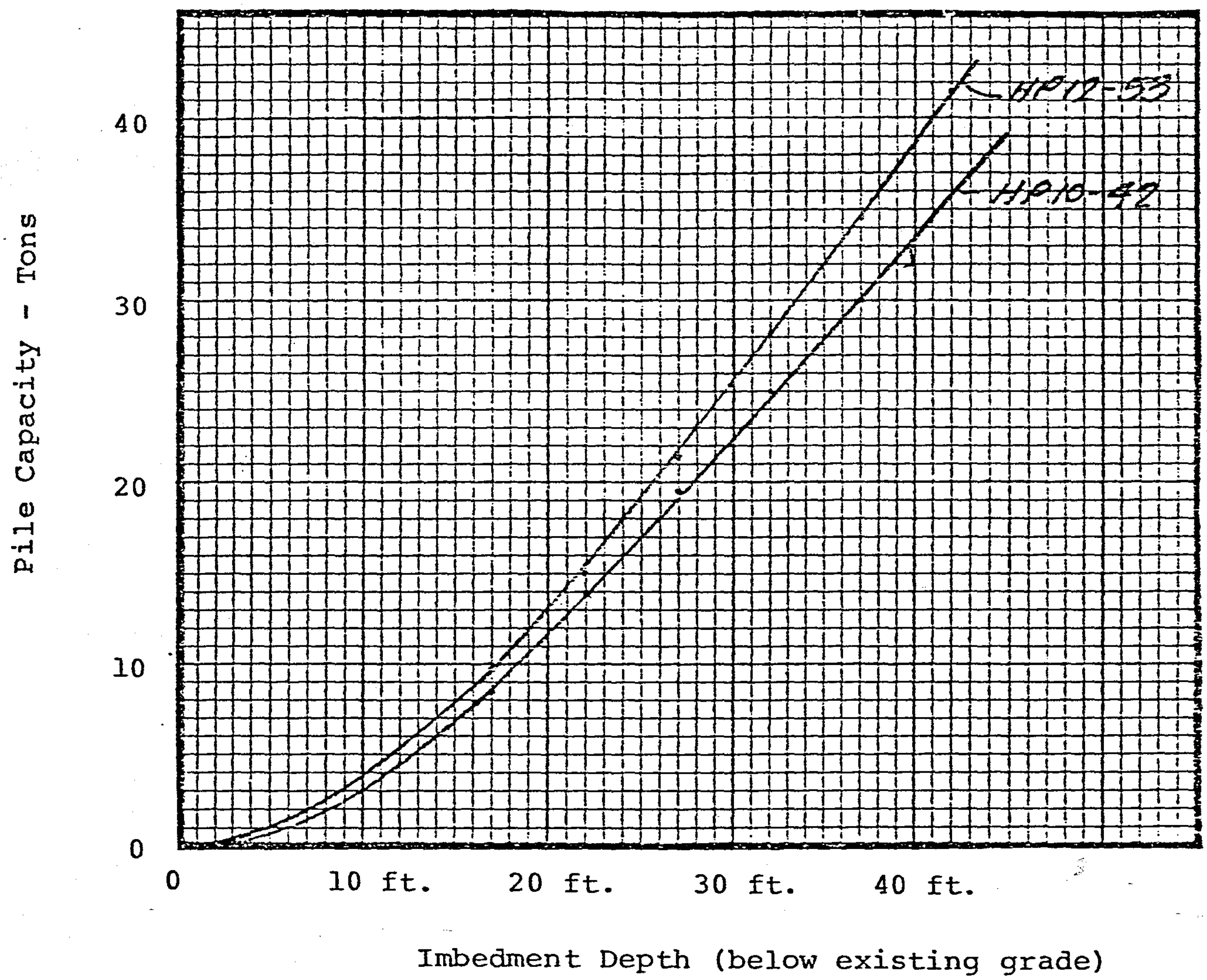
Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan
Road Crossing
Pinal County, Arizona
Job No. 812-467
Addendum No. 1

Driven Piling Below Finished
Channel Grades - Pier Support



Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan
Road Crossing
Pinal County, Arizona
Job No. 812-467
Addendum No. 1

Driven Piling Below Existing Ground Grade
Abutment Support



Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan
Road Crossing
Pinal County, Arizona
Job No. 812-467
Addendum No. 1

4. Based upon preliminary test data, an imbedment depth of approximately 15 feet below scour line would be required for lateral resistance to a 2 foot diameter caisson element subjected to a 20 kip lateral load applied 13 feet above finished channel grade.

5. Based upon present design concepts with a channel elevation at 1227.7 feet (approximately 10 feet below grade), the channel bottom may be founded partly or totally within the silty fine sand subsoils. Therefore, bottom lining would be necessary to eliminate the potential for scour due to a design velocity of 6.7 fps. Additionally, side slopes should be protected to minimize moisture increase in clay soils which could lead to erosion, surface slumping due to reduced shear strength and/or undermining by loss of support from underlying sands.

This addendum shall be attached to the original report and become a part thereof. Please do not hesitate to contact us if we can be of further service to you.

Respectfully submitted,
ENGINEERS TESTING LABORATORIES, INC.
Geotechnical Services

By: Glen K. Copeland — Reviewed by: Kenneth L. Ricker
Glen K. Copeland, P.E. — Kenneth L. Ricker, P.E.
/jm
copies to: Addressee (5)

5-24-79

E

ENGINEERS TESTING LABORATORIES, INC.
3737 East Broadway Road
P. O. Box 21387
Phoenix, Arizona 85036
(602) 268-1381

Wheeler, Brooks, Coffeen, Inc.
2538 E. University Dr., Suite 140
Phoenix, Arizona 85017

2 August 1978

Attention: Art Brooks, P.E.

Project: Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan Road Crossing
Pinal County, Arizona

Job No. 812-467
Inv. No. 1121138

In accordance with our agreement, ETL No. 112-2454, this firm has performed preliminary soil engineering services relative to the subject project.

The accompanying report contains the results of the subsurface exploration, laboratory testing and the conclusions and preliminary recommendations derived from our engineering evaluation. This evaluation has been formulated in accordance with generally accepted professional engineering principles and practices.

To assist implementation of the preliminary recommendations presented, this firm provides complementary services. These services include discussion relative to compliance of final design concepts, development of scope of geotechnical services for final foundation exploration; and submission of addenda where report content clarifications are required. You are encouraged to avail yourself of these services.

It has been a pleasure participating with your firm on this project. Should any questions arise regarding the report, please do not hesitate to contact us.

Respectfully submitted,

ENGINEERS TESTING LABORATORIES, INC.
Geotechnical Services

Registered Professional Engineer
Certificate
Glen K. Copeland
Arizona, U.S.A.

By: [signature]
Glen K. Copeland, P.E.
8-2-78

Reviewed by: [signature]
Kenneth L. Ricker, P.E.

/jm

copies to: Addressee (5)

[illegible] 10/24/78

Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan Rd. Crossing
Job No. 812-467

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Professional Engineer
CERTIFICATE
ARIZONA, U.S.A.

PART I

REPORT

Preliminary Foundation Evaluation
RWCD Floodway Bridge @ San Tan Rd. Crossing
Job No. 812-467

SCOPE

This report presents the results of preliminary soil engineering services conducted for the proposed San Tan Road crossing of the RWCD Floodway Canal located in Section 13, T.3S., R.5E., Pinal County, Arizona. The purpose of this preliminary report is to provide general recommendations for use in the economic evaluation of various foundation systems and to provide comments relative to earthwork requirements. The field and laboratory testing has been performed and report prepared based upon the following information:

1. The proposed bridge will be approximately 300 feet in total length and approximately 35 feet in width. Span lengths have been approximated at 45 feet. It is assumed that the bridge will be constructed utilizing either a pre-cast or cast-in-place concrete system.

2. Maximum foundation loadings are estimated to be:

Support Member	Total Estimated Load
Pier	850 kips
Abutments	550 kips

3. Channel will be constructed with side slopes of 3 to 1 (horizontal to vertical) and an invert elevation of approximately 1227.7 feet (approximately 10 feet below existing grade). A gunite surfacing within the channel may be utilized to minimize erosion and scour.

4. Final design grades for the bridge deck and approach fills have not been established.

This office should be contacted for review and provision of required supplemental recommendations should preliminary design concepts or grades vary substantially or if structural loadings are heavier than estimated.

SOIL EXPLORATION AND TESTING

Three test borings were drilled with a rotary auger (CME-75) drilling rig, utilizing continuous flight augers, at locations shown on the accompanying site plan. During test drilling, encountered subsoils were visually examined, classified and sampled at appropriate intervals.

Representative surface and subsoil samples were subjected to the following laboratory analyses:

Test	Sample(s)	Purpose
Gradation and Atterberg Limits	Subsoil (1)	Classification and evaluation of scour potential
Consolidation	Undisturbed Subsoil (3)	Settlement analysis
Direct Shear	Undisturbed Subsoil (2)	Bearing capacity and shearing strength
Dry Density & Moisture Content*	Undisturbed Subsoil (14)	Physical properties of subsoils in-situ

*Reported on Soil Boring Data sheets.

The consolidation tests were performed on Casagrande "fixed ring" consolidometers. The samples were loaded to selected pressures at in-situ moisture contents and then immersed to show the effect of nearly complete saturation. The shear test was conducted in a "Direct Shear" apparatus with the samples immersed to approximate saturation.

SITE AND SOIL CONDITIONS

The proposed bridge site is located within the present San Tan Road alignment as shown on the attached site plan. Existing development consists of an asphalt paved roadway. Adjacent lands are undeveloped desert property. Vegetative cover consisted of grass, weeds and cacti with sparse trees. Ground surface was described as smooth and moderately firm.

As disclosed by the test borings and illustrated on the "Soil Boring Data" sheets, Part III, the subsoil profile across the site is relatively uniform. New surface soils to an approximate 10.5 foot depth are silty clays of moderately firm to firm consistencies and exhibit moderate densities. These soils also contain lightly to moderately cemented zones below approximately 7 to 8 feet. Underlying soils, generally extending to depths of 20 to 25 feet, are silty sands of medium density. Below this silty sand stratum, a stratum of caliche (well cemented clayey sand) was encountered and was present throughout the remaining depth of test boring penetration (30 feet).

Moisture contents were described as being slightly damp for sandy soils and as being near to below the respective plastic limits for the clay soils. No free groundwater table was encountered in any of the test borings.

DISCUSSION AND RECOMMENDATIONS

General: It is assumed that the structure will bear upon undisturbed soils and that the subsurface conditions within the entire construction site do not vary substantially from soil profiles established at boring locations. Due to the preliminary nature of the exploration services, additional borings should be accomplished at each pier and abutment location after design concepts have been finalized.

The following generalized conclusions are drawn:

1. Near surface soils existent to depths of approximately 10 to 11 feet are predominatnly moderately dense silty clays which possess moderate bearing capacity and low to moderate settlement potential. Shallow spread footing elements supporting abutments could be utilized but will require that they be positioned (horiziontally and vertically) to minimize the effects on the adjacent canal slopes.

2. Subsoils encountered below an approximate 10 to 11 foot depth are predominantly medium dense silty sands. Foundations based at or below this level could be constructed as either circular drilled pier elements in abutment areas or as shallow foundation elements at the pier area, and would afford a moderately high load capacity.

3. An alternate foundation system for structural support could consist of the installation of driven pile foundations. A minimum displacement driven pile, such as an "H" pile, would appear most feasible.

4. Excavation to anticipated channel invert level may be accomplished with conventional earth-moving equipment. However, confined excavations such as for foundation elements, utility trenches, etc. may prove difficult below an approximate 11 foot depth due to caving potential of non-cemented silty sands.

5. Scour depths have been considered in foundation design since the proposed channel may not be lined. Should channel lining be included in final design, a review of pier footing depths should be accomplished.

Foundation Design Recommendations: Spread foundations and circular drilled pier elements bearing upon undisturbed subsoils and driven pile foundations were analyzed for the anticipated loading conditions. Preliminary design recommendations for the various foundation systems are presented below and are based on the assumption of uniform subsoil conditions across the site. Recommendations for other systems or loading conditions will be analyzed upon request.

Unit bearing pressures and load capacities as presented herein should be considered allowable maximums for dead plus design live load conditions.

A one-third increase is permissible when considering total loads including wind or seismic forces, or to determine the maximum toe pressure of eccentrically loaded retaining wall footings provided the resultant of all forces acts within the central third of the footing section. Two (2.0) feet is recommended as the minimum diameter and width of circular pier and continuous footings. Finished grade references should be considered as lowest adjacent grade for abutment footings and as channel invert elevation for central pier element.

Footing System	Minimum Footing Depth (1)	Allowable Soil Bearing Capacity
Shallow Spread:		
Abutment	3.0 ft. (min.)	3.5 ksf
Abutment	4.0 ft.	4.0 ksf
Abutment	5.0 ft.	4.5 ksf
Pier	5.0 ft. (min.)	2.5 ksf (2)
Pier	6.0 ft.	3.0 ksf (2)
Circular Drilled Pier or Caisson:		
Abutment	20.0 ft.	15.0 ksf (3)
Pier	10.0 ft.	15.0 ksf (3)
		Pile Capacity
Driven Piles:		
Abutment	20.0 ft.	20.0 kips (4)
Abutment	25.0 ft.	25.0 kips (4)
Abutment	30.0 ft.	35.0 kips (4)
Pier	10.0 ft.	15.0 kips (4)
Pier	15.0 ft.	20.0 kips (4)
Pier	20.0 ft.	30.0 kips (4)

(1) Minimum depth to footing base referenced below slope face at edge of abutment footings or depth to base of footing below finished channel grade for pier footings.

(2) Bearing capacity reduced to account for partial or total submergence of silty sand bearing soils.

(3) Bearing upon the moderate to heavy cemented subsoils.

(4) Total single pile capacity including dead, live and seismic loads based on HP-10-42.

E

Drilling and belling within the near surface and subsoils in the abutment areas should be readily accomplished with conventional rotary or bucket augers. However, harder drilling conditions should be anticipated at central pier elements due to cemented subsoil conditions. Caving or raveling is anticipated where the caissons penetrate the relatively clean sands from 11 feet to 20 feet. It is not expected that stabilizing techniques will be required to maintain open shafts; however, foundation concrete quantities will probably somewhat exceed ideal geometric volumes.

Disturbed soils (drilling spoil) must be removed from the bearing surface of drilled foundation elements designed on an end-bearing basis. Adequacy of cleaning and verification of proper configuration should be established by inspection of drilled elements. Applicable safety codes will require safety casing for protection of personnel entering shafts for cleaning or inspection. Two (2.0) feet is recommended as the minimum diameter of circular drilled foundations to be entered.

Piles should be spaced at least three pile widths on center. If closer spacing is required, a reduction, due to group action, should be applied to the pile capacity.

The H-pile should be driven with a hammer which develops at least 15,000 foot-pounds of energy per blow. Acceptable criteria should be based on driving the piles to the minimum recommended penetration for the desired load. If hard driving is encountered below this level the final driving resistance may be determined by a dynamic driving criterion such as the Engineering News Formula.

Settlement calculations have not been accomplished due to the variable load conditions and/or footing systems which could be adopted. However, settlements for near surface footings bearing on clays will probably be less than one inch. Settlements, for footings on medium dense sands or cemented subsoils will be less than those developed in the clay soils and will primarily occur during construction phases.

Excavated Channel Slope Stability: Proposed channel slopes of 3 to 1 (horizontal to vertical) appear to provide adequate safety against failure based upon the existing soil conditions, anticipated construction procedures and proposed loading conditions. However, other variables such as embankment slope erosion and scour depth, etc. could have adverse effects on long term stability and should be considered in initial design and future maintenance. In addition, should the channel slopes become saturated or partially or completely submerged the factor of safety could be substantially reduced.

Scour Potential: Based on the results of gradation test, the materials exposed at the channel bottom are predominantly silty medium to fine sands. These materials would be subject to erosion and scour if the velocity is greater than 1.0 foot/sec. Therefore, consideration should be given to channel protection, at least in the bridge crossing area.

Based on the assumption that pier form will have rounded noses and will be founded upon a continuous footing element, the depth of scour appears to be in the range of 3 to 4 feet when using an effective sand grain size of 0.20 mm. Scour depth is dependent upon pier footing configuration, shape and orientation of pier axis to direction of flow. Therefore, scour depth should be checked once final design concepts have been determined.

ALLOWABLE SOIL BEARING CAPACITY ALLOWABLE FOUNDATION PRESSURE	The recommended maximum contact stress developed at the interface of the foundation element and the supporting material.
BACKFILL	A specified material placed and compacted in a confined area.
BASE COURSE	A layer of specified material placed on a subgrade or subbase.
BASE COURSE GRADE	Top of base course.
BENCH	A horizontal surface in a sloped deposit.
CAISSON	A concrete foundation element cast in a circular excavation which mav have an enlarged base. Sometimes referred to as a cast-in-place pier.
CONCRETE SLABS-ON-GRADE	A concrete surface layer cast directly upon a base, subbase or subgrade.
CRUSHED ROCK BASE COURSE	A base course composed of crushed rock of a specified gradation.
DIFFERENTIAL SETTLEMENT	Unequal settlement between or within foundation elements of a structure.
ENGINEERED FILL	Specified material placed and compacted to specified density and/or moisture conditions under observation of a representative of a soil engineer.
EXISTING FILL	Materials deposited through the action of man prior to exploration of the site.
EXISTING GRADE	The ground surface at the time of field exploration.
EXPANSIVE POTENTIAL	The potential of a soil to expand (increase in volume) due to the absorption of moisture.
FILL	Materials deposited by the action of man.
FINISHED GRADE	The final grade created as a part of the project.
GRAVEL BASE COURSE	A base course composed of naturally occurring gravel with a specified gradation.
HEAVE	Upward movement.
NATIVE GRADE	The naturally occurring ground surface.
NATIVE SOIL	Naturally occurring on-site soil.
ROCK	A natural aggregate of mineral grains connected by strong and permanent cohesive forces. Usually requires drilling, wedging, blasting or other methods of extraordinary force for excavation.
SAND AND GRAVEL BASE	A base course of sand and gravel of a specified gradation.
SAND BASE COURSE	A base course composed primarily of sand of a specified gradation.
SCARIFY	To mechanically loosen soil or break down existing soil structure.
SETTLEMENT	Downward movement.
SOIL	Any unconsolidated material composed of discrete solid particles, derived from the physical and/or chemical disintegration of vegetable or mineral matter, which can be separated by gentle mechanical means such as agitation in water.
STRIP	To remove from present location.

Continued on Reverse Side

SUBBASE — A layer of specified material placed to form a layer between the subgrade and base concrete.

SUBBASE GRADE — Top of subbase.

SUBGRADE — Prepared native soil surface.

PART II

RESULTS OF LABORATORY ANALYSES

Job No. 812-467

Boring No.	Depth, ft.	Soil Class.	Particle Size Distribution, %				Atterberg Limits			Moisture - Density Rel.			Specific Gravity	Permeability		'R' Value		
			Cobbles 3" to 12"	Gravel 3" to #4	Sand #4 to 200	Fines -200	PL	LL	PI	Dry Density pcf	Optimum Moisture %	Meth.		Dry Density pcf	K Ft./Yr.	Corrected 'R'	Expansion Pressure psf	
1	12-14			2	79	19												

Boring No.	Depth, ft.	Remarks

LEGEND

Moisture Density Relationship
1. Tested D-1556/AASHTO T-217
2. Tested ASTMD-2922/D-3017
3. Tested ASTMD-2922/AASHTO T-217
4. Rock correction applied to maximum dry density. AASHTO T-224
5. Other

Specific Gravity
6. Minus # 4
7. Plus # 4

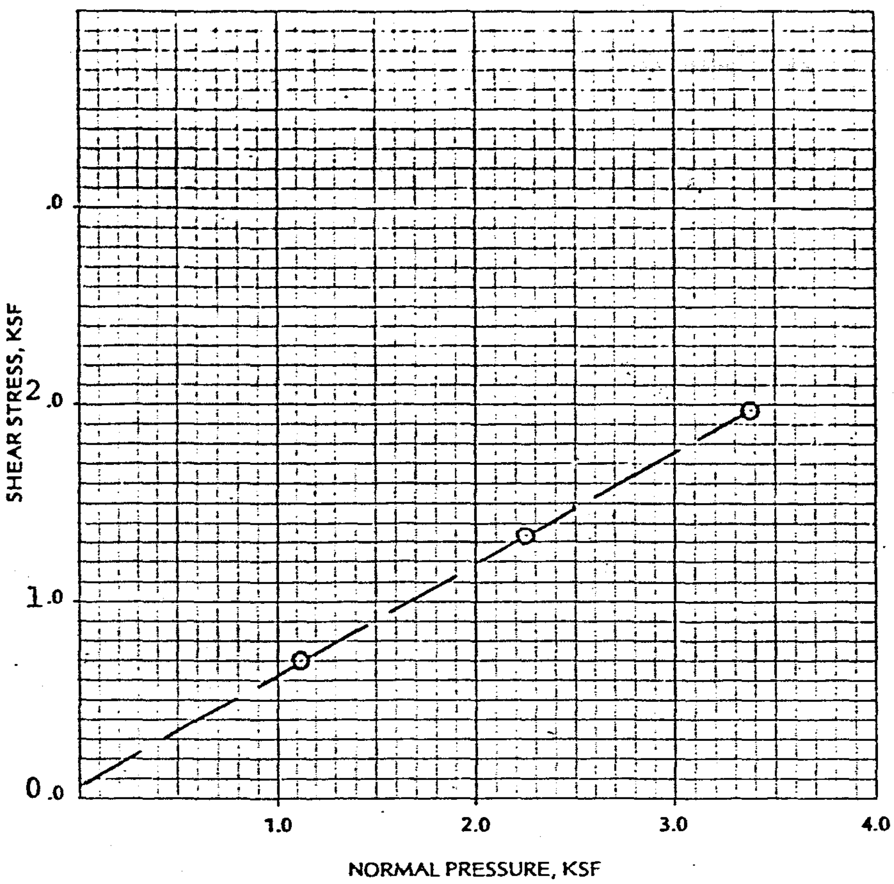
Classification/Particle Size
8. Visual
9. Laboratory Tested

SOIL PROPERTIES

Part 11 of 18 Page 9

Job No. 812-467

DIRECT SHEAR TEST



Lab./Invoice No. ____________

Type of Material ____________

Source of Material TB#2; 5-6'

Sampled By GMD Date 7-10-78

Submitted By GMD Date 7-10-78

Reviewed By GKC Date 7-31-78

Test Procedure ASTM D3080- Single Shear

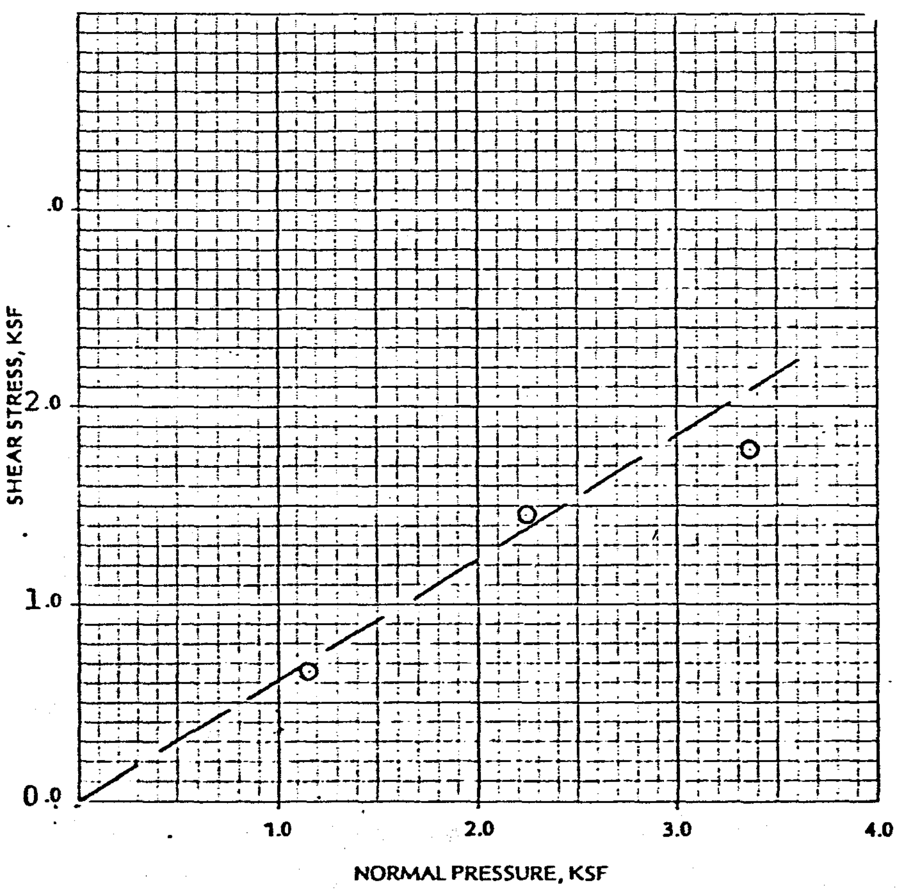
Test Condition: — — InSitu ——— Saturated

Sample Condition: ☒ Undisturbed ☐ Remolded

Initial Dry Density, pcf 88.9

Initial Moisture Content, % 21.2

ϕ = 29 ° C = 0.08 Kips/Sq. Ft.



Lab./Invoice No. ____________

Type of Material ____________

Source of Material TB#1; 15-16'

Sampled by GMD Date 7-10-78

Submitted By GMD Date 7-10-78

Reviewed By GKC Date 7-31-78

Test Procedure ASTM D3080- Single Shear

Test Condition: — — InSitu ——— Saturated

Sample Condition: ☒ Undisturbed ☐ Remolded

Initial Dry Density, pcf 100.9

Initial Moisture Content, % 2.5

ϕ = 31 ° C = 0 Kips/Sq. Ft.

E

Job No. 812-467

Lab No.

CONSOLIDATION PROPERTIES OF SOIL

Type of Material: Silty Sand ☒Undisturbed ☐Remolded ☐Compacted

Source of Material: RWCD Bridge Crossing Boring 1 Depth 10-11'

Test Procedure: ASTM D2435- Reviewed By GKC Date 7-31-78

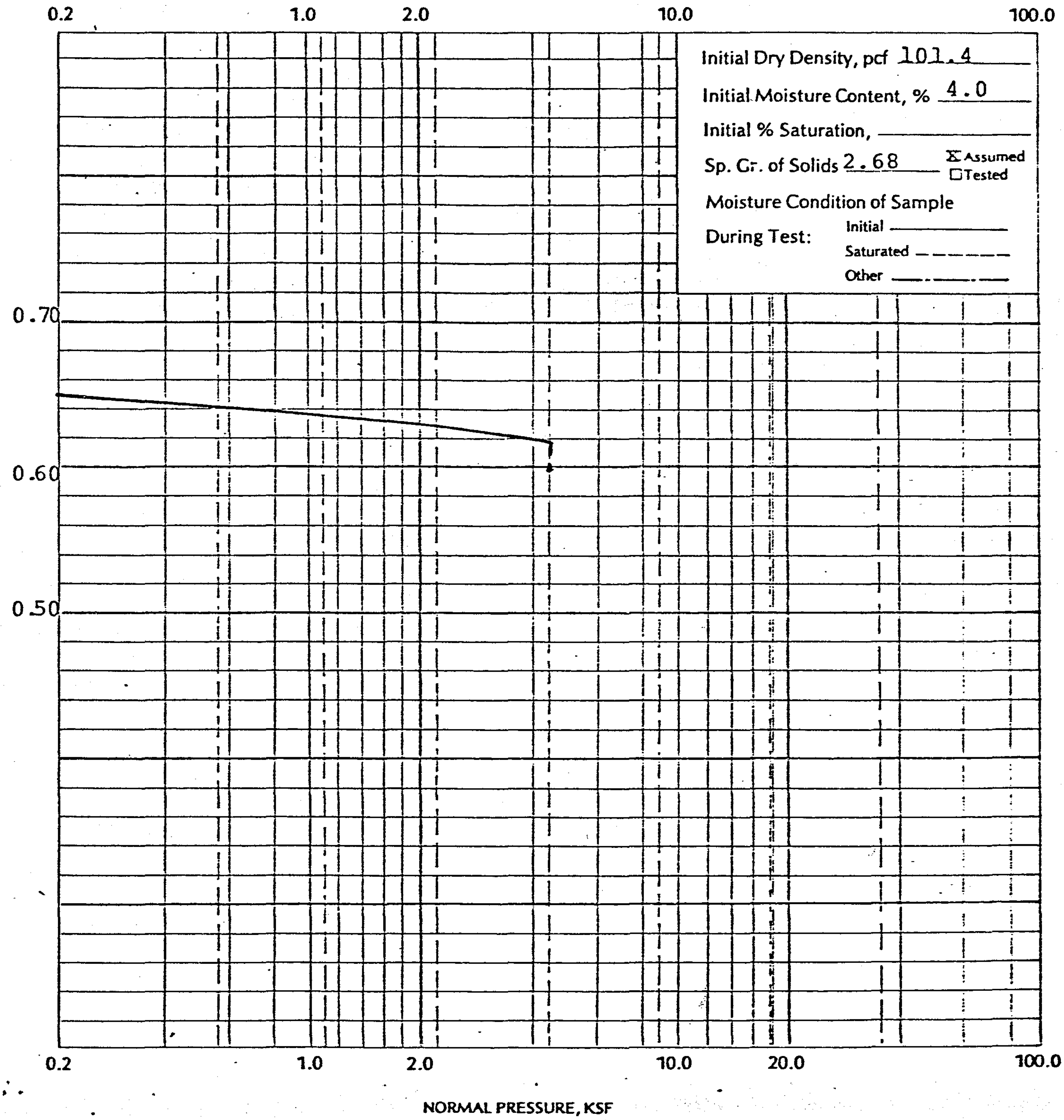
Initial Dry Density, pcf 101.4

Initial Moisture Content, % 4.0

Initial % Saturation,

Sp. Gr. of Solids 2.68 ☒Assumed ☐Tested

Moisture Condition of Sample During Test: Initial — Saturated — Other —



Job No. 812-467

Lab No.

CONSOLIDATION PROPERTIES OF SOIL

Type of Material: Silty Clay — ☒ Undisturbed ☐ Remolded ☐ Compacted

Source of Material: RWCD Bridge Crossing — Boring: 2 — Depth: 5-6'

Test Procedure: ASTM D2435- — Reviewed By: GKC — Date: 7-31-78

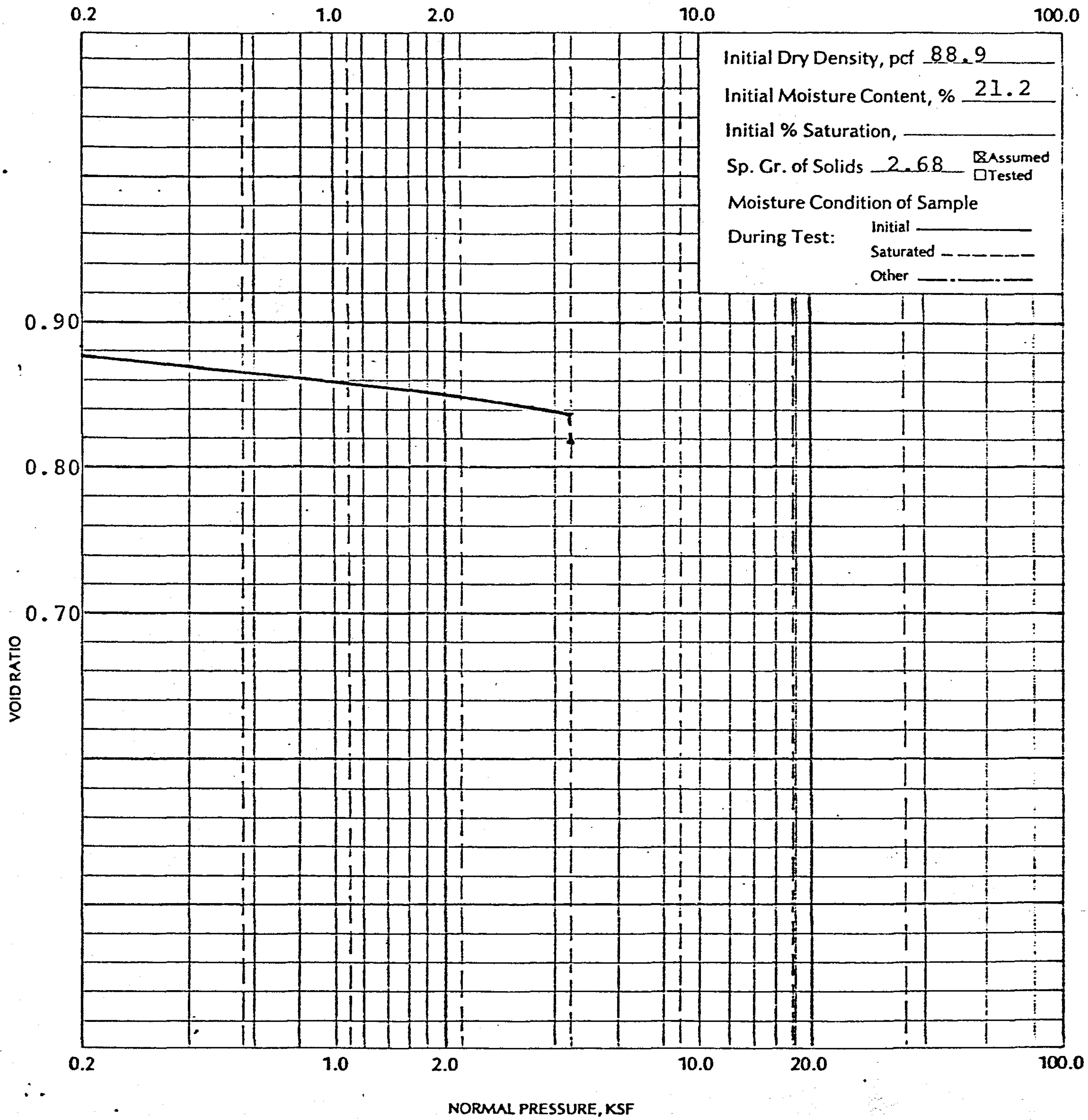
Initial Dry Density, pcf: 88.9

Initial Moisture Content, %: 21.2

Initial % Saturation,

Sp. Gr. of Solids: 2.68 ☒ Assumed ☐ Tested

Moisture Condition of Sample During Test:
- Initial ————
- Saturated – – – –
- Other —·—·—



E

Job No. 812-467

Lab No.

CONSOLIDATION PROPERTIES OF SOIL

Type of Material Sand ☒Undisturbed ☐Remolded ☐Compacted

Source of Material RWCD Bridge Crossing Boring 3 Depth 15-16

Test Procedure ASTM D2435- Reviewed By GKC Date 7-31-78

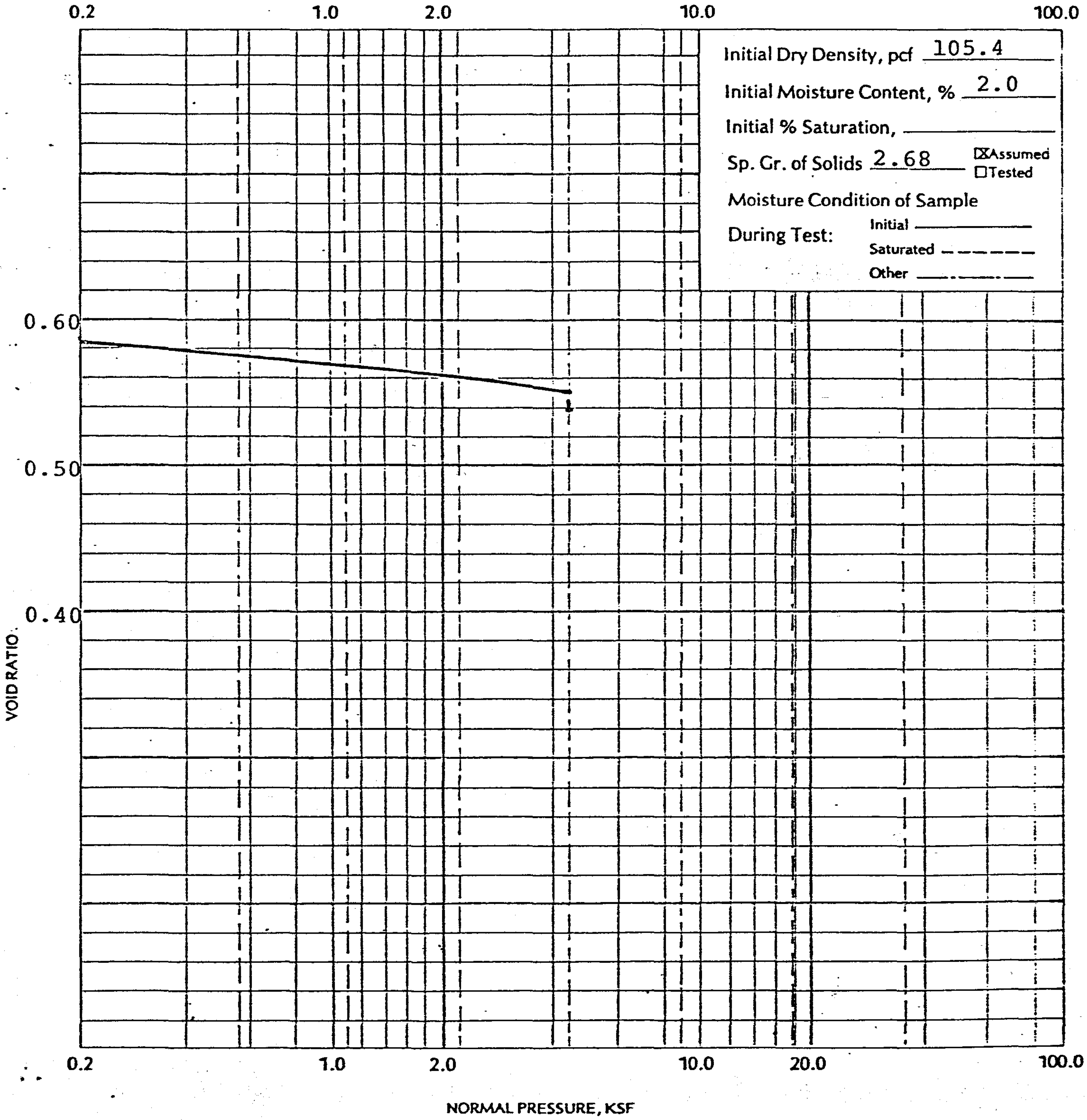
Initial Dry Density, pcf 105.4

Initial Moisture Content, % 2.0

Initial % Saturation,

Sp. Gr. of Solids 2.68 ☒Assumed ☐Tested

Moisture Condition of Sample During Test: Initial ——— Saturated - - - - Other —·—·—



E

PART III

RESULTS OF FIELD EXPLORATION

COARSE-GRAINED SOIL

MORE THAN 50% LARGER THAN 200 SIEVE SIZE

Symbol	Letter	DESCRIPTION	MAJOR DIVISONS
	CW	WELL-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% - 200 FINES	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size.
	GP	POORLY-GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LESS THAN 5% - 200 FINES	
	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, MORE THAN 12% - 200 FINES	
	GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, MORE THAN 12% - 200 FINES	
	SW	WELL-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% - 200 FINES	SANDS More than half of coarse fraction is smaller than No. 4 sieve size.
	SP	POORLY-GRADED SANDS OR GRAVELLY SANDS, LESS THAN 5% - 200 FINES	
	SM	SILTY SANDS, SAND-SILT MIXTURES MORE THAN 12% - 200 FINES	
	SC	CLAYEY SANDS, SAND-CLAY MIXTURES MORE THAN 12% - 200 FINES	

FINE-GRAINED SOIL

MORE THAN 50% SMALLER THAN 200 SIEVE SIZE

Symbol	Letter	DESCRIPTION	MAJOR DIVISIONS
	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	SILTS AND CLAYS Liquid limit less than 50
	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
	OL	ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY	
	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	SILTS AND CLAYS Liquid limit greater than 50
	CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	
	PT	PEAT AND OTHER HIGHLY ORGANIC SOILS	

NOTE — Soils with 5 to 12 percent minus 200 fines should be classified with dual symbols

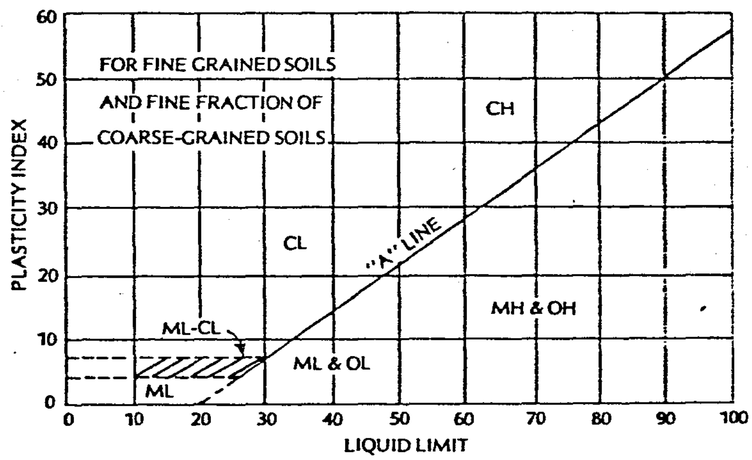
SOIL FRACTIONS

Component	Size Range
Boulders	Above 12 in.
Cobbles	3 in to 12 in.
Gravel	3 in. to No. 4 sieve
Coarse Gravel	3 in. to ¾ in.
Fine Gravel	¾ in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

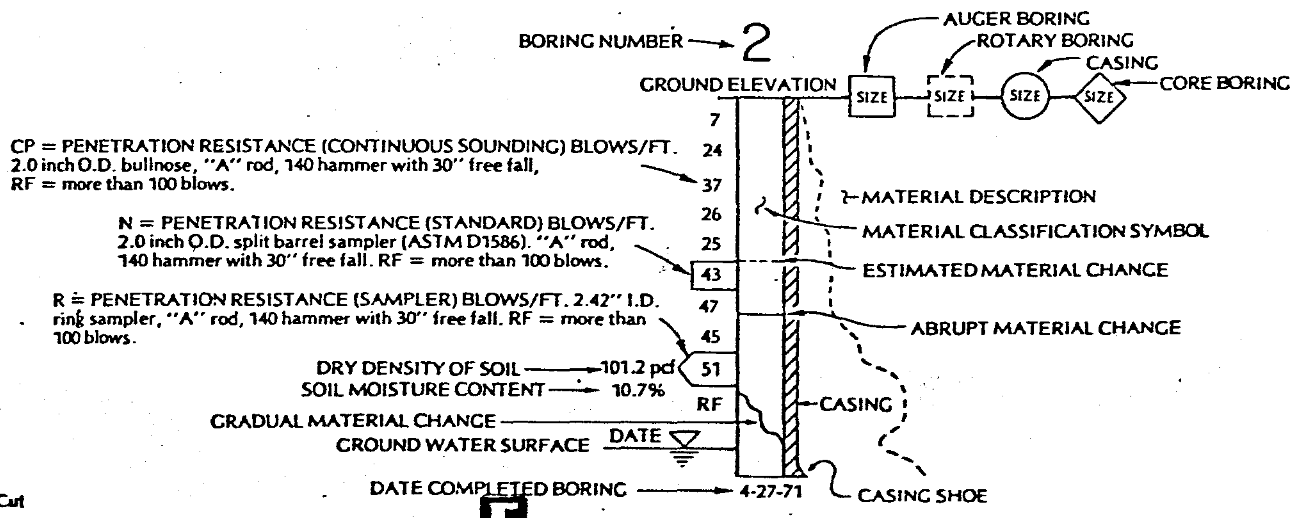
Soil Classification: ASTM D2487

Classification is visual unless accompanied by mechanical analysis and Atterberg limits. Percentage shown on log denotes visual approximation ± 5%.

PLASTICITY CHART



LEGEND OF BORING OPERATIONS



Sample Type

- R Ring
- T Shelby Tube
- S Standard Split Barrel
- B Block
- G Grab
- C Cutting
- V Vertical Face Cut

SOIL BORING DATA

Location of Boring: RWCD Bridge Crossing — No. 1 — Job No. 812-467

Elev. Top of Hole: 99.0 — Datum: See Site Plan — Prepared By: GMD — Date: 7-10-78

Type/Size of Boring: 8" — Date: 7-10-78 — Driller: Pete/Frank — Reviewed By: KLR — Date: 7-31-78

Depth Ft.	Penetration Resistance Blows/Ft. C	Penetration Resistance Blows/Ft. N	Sample Type	Dry Density pcf	Moisture Content	Graphical Log	Description	Soil Classification	Max. Size, In.	Boulders	Cobbles	Gravel	Sand	Silt & Clay	Gradation (Well, Med., Poor)	Grain Shape (Angular, Subangular, Rounded, Subrounded)	Relative Density (Low, Med, High)	Dry Strength (None, Very Low, Low, Medium, High, Very High)	Plasticity (None, Low, Medium, High)	Consistency (Soft, Firm, Stiff, Very Stiff, Hard)	Cementation (None, Light, Moderate, Heavy)
1							SILTY CLAY; Light Brown	CL	1/2"				0	100	X	XX		X	XX	X	X
2		21	R	102.8	10.1		Trace fine sand, moderate cementa-						Tr	–							
3					near or		tion below approximately 5 feet.														
4					below PL																
5																					
6		37	R	94.0	21.9																
7																					
8																					
9																					
10																					
1		17	R	101.4	4.0																
2																					
3					slightly		SILTY FINE SAND; Brown; Trace Clay;	SM	1/2"			Tr	55	35	X	XX	X		X		X
4					damp		Trace medium to coarse sand below						65	45							
5							17.0'.														
6		15	R	100.9	2.5																
7																					
8																					
9																					
20							Moderate to heavy calcite cementation														
1		51	R	102.2	2.2		below 20'.														
2																					
3																					
4																					
5																					
6		50/2"	R	No Recovery																	
7					slightly		CALICHE; Tan; Difficult drilling														
8					damp		below 27'.														
9																					
30																					

Stopped @ 30'
No groundwater encountered

NOTE: THE DATA PRESENTED ON THESE BORING LOGS REPRESENTS SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC TEST LOCATIONS AND AT THE TIME DESIGNATED. THIS DATA MAY NOT REPRESENT CONDITIONS AT OTHER LOCATIONS AND/OR TIMES. THIS BORING DATA WAS COMPILED PRIMARILY FOR DESIGN PURPOSES. THIS DATA SHOULD NOT BE CONSTRUED AS PART OF THE PLANS GOVERNING CONSTRUCTION OR AS DEFINING CONSTRUCTION TECHNIQUES.

SOIL BORING DATA

Location of Boring: RWCD Bridge Crossing No. 2 Job No. 812-467

Elev. Top of Hole: 99.0 Datum: See Site Plan Prepared By: GMD Date: 7-10-78

Type/Size of Boring: 8" Date: 7-10-78 Driller: Pete/Frank Reviewed By: KLR Date: 7-31-78

Depth Ft.	Penetration Resistance Blows/Ft. C	N	Sample Type	Dry Density pcf	Moisture Content	Graphical Log	Description	Soil Classification	Max. Size, In.	Boulders	Cobbles	Gravel	Sand	Silt & Clay	Gradation (Well/Med./Poor)	Grain Shape (Angular/Subangular/Rounded/Subrounded)	Relative Density (Low/Med/High)	Dry Strength (None/Very Low/Low/Medium/High/Very High)	Plasticity (None/Low/Medium/High)	Consistency (Soft/Firm/Stiff/Very Stiff/Hard)	Cementation (None/Light/Moderate/Heavy)
1					near		SILTY CLAY; Light Brown	CL	1/2"				0	100	x	xx		x	xx	x	x
2		21	R	103.7	9.4		Trace fine sand, moderate cementa-						Tr	-							
3					or		tion below approximately 5 feet.														
4					below PL																
5																					
6		20	R	88.9	21.2																
7																					
8																					
9																					
10																					
1		20	R	97.9	5.2																
2																					
3					slightly		SILTY FINE SAND; Brown; Trace Clay;	SM	1/2"			Tr	55	35	x	xx	x		x		x
4					damp		Trace medium to coarse sand below						65	45							
5							15'.														
6		50	R	*	6.1		Moderate to heavy calcite cementation														
7							below 15 ft.														
8																					
9																					
20																					
1		50/10"	R	No Recovery																	
2					slightly		CALICHE; Tan; Difficult drilling														
3					damp		below 27'.														
4																					
5																					
6		50/2"	R	No Recovery																	
7																					
8																					
9																					
30																					

Stopped @ 30'
No groundwater encountered

NOTE: THE DATA PRESENTED ON THESE BORING LOGS REPRESENTS SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC TEST LOCATIONS AND AT THE TIME DESIGNATED. THIS DATA MAY NOT REPRESENT CONDITIONS AT OTHER LOCATIONS AND/OR TIMES. THIS BORING DATA WAS COMPILED PRIMARILY FOR DESIGN PURPOSES. THIS DATA SHOULD NOT BE CONSTRUED AS PART OF THE PLANS GOVERNING CONSTRUCTION OR AS DEFINING CONSTRUCTION TECHNIQUES.

SOIL BORING DATA

Location of Boring RWCD Bridge Crossing No. 3 Job No. 812-467

Elev. Top of Hole 98.8 Datum See Site Plan Prepared By GMD Date 7-10-78

Type/Size of Boring 8" Date 7-10-78 Driller Pete/Frank Reviewed By KLR Date 7-31-78

Depth Ft.	Penetration Resistance Blows/Ft. C	Penetration Resistance Blows/Ft. N	Sample Type	Dry Density pcf	Moisture Content	Graphical Log	Description	Soil Classification	Max. Size, In.	Particle Size Distribution % Boulders	Cobbles	Gravel	Sand	Silt & Clay	Gradation (Well / Med. / Poor)	Grain Shape (Angular / Subangular / Rounded / Subrounded)	Relative Density (Low / Med / High)	Dry Strength (None / Very Low / Low / Medium / High / Very High)	Plasticity (None / Low / Medium / High)	Consistency (Soft / Firm / Stiff / Very Stiff / Hard)	Cementation (None / Light / Moderate / Heavy)
1							SILTY CLAY; Light Brown	CL	1/2"				0	100	x	xx		x	xx	x	x
2		19	R	95.7	11.5		Trace fine sand						Tr	–							
3					near or																
4					below PL																
5																					
6		48	R	94.6	22.1																
7																					
8							Moderate cementation below 8'														
9																					
10																					
1		25	R	94.7	5.0																
2																					
3					slightly		SILTY FINE SAND; Brown; Trace Clay;	SM	1/2"			Tr	55	35	x	xx	x		x		x
4					damp		Trace medium to coarse sand below						65	45							
5							15'.														
6		16	R	105.4	2.0																
7																					
8																					
9																					
20																					
1		49	R	101.4	15.3																
2																					
3																					
4					slightly		CALICHE; Tan; Difficult drilling														
5					damp		below 27'.														
6		75/5"	S																		
7																					
8																					
9																					
30																					

Stopped @ 30'
No groundwater encountered

NOTE: THE DATA PRESENTED ON THESE BORING LOGS REPRESENTS SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC TEST LOCATIONS AND AT THE TIME DESIGNATED. THIS DATA MAY NOT REPRESENT CONDITIONS AT OTHER LOCATIONS AND/OR TIMES. THIS BORING DATA WAS COMPILED PRIMARILY FOR DESIGN PURPOSES. THIS DATA SHOULD NOT BE CONSTRUED AS PART OF THE PLANS GOVERNING CONSTRUCTION OR AS DEFINING CONSTRUCTION TECHNIQUES.

PART IV

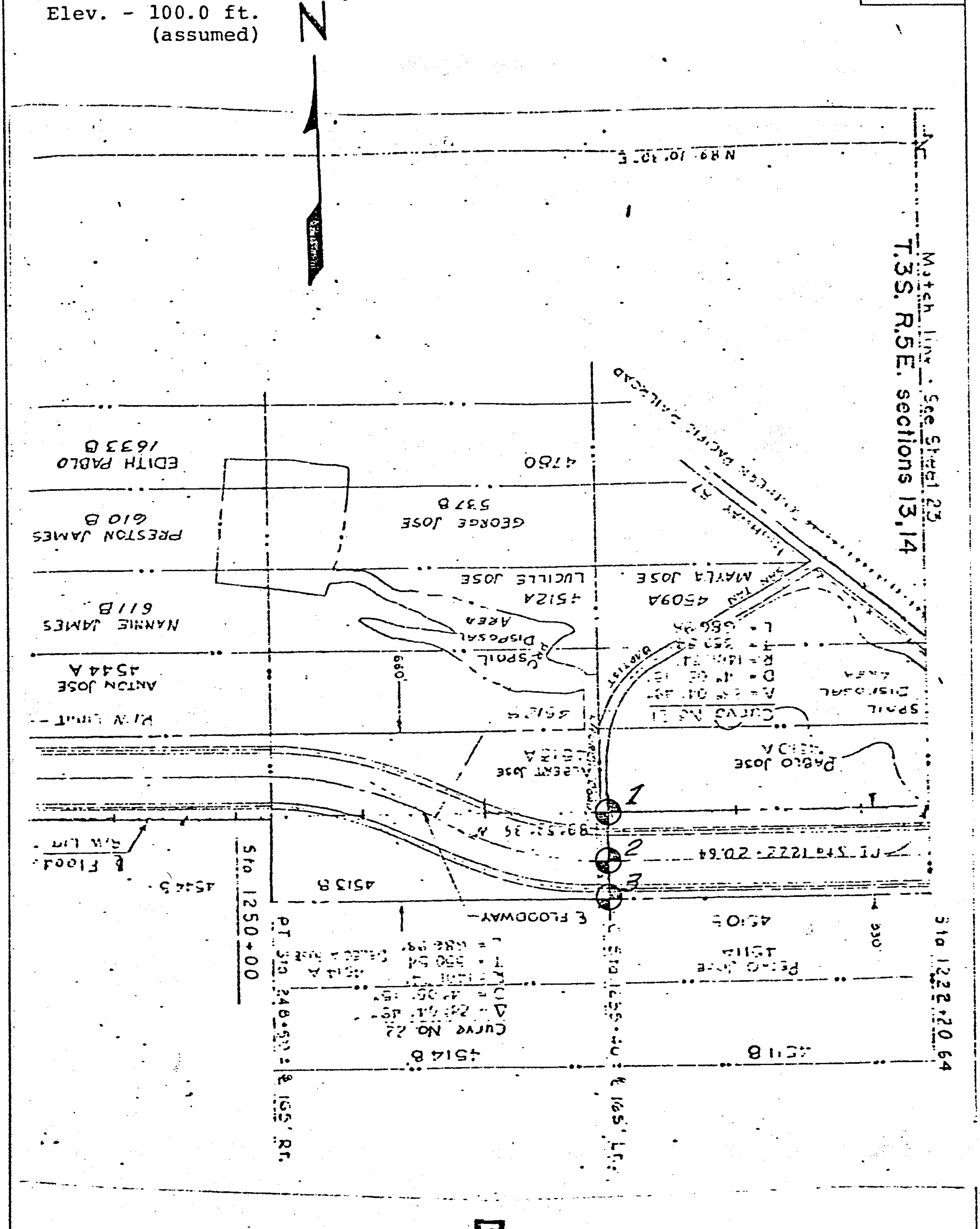
SITE PLAN

Job No. 812-467

SITE PLAN/TEST BORING LOCATION

18 OF 18
PART IV

BENCH MARK: Brass cap in San Tan Road east of Test Boring No. 2
Elev. - 100.0 ft.
(assumed)



E

APPENDIX "B"

RWCD FLOODWAY BRIDGE

SAN TAN ROAD, SECTION 13

T3S, R5E, PINAL COUNTY

ARIZONA

PRELIMINARY INVESTIGATION

ON

FEASIBLE STRUCTURAL SYSTEMS

FOR

SUPERSTRUCTURE AND SUBSTRUCTURE

MAGADINI-ALAGIA ASSOCIATES
STRUCTURAL ENGINEERS, INC.

Registered Professional Engineer (Structural)
CERTIFICATE NO.
5530
S. ALAGIA
Date Signed 12/6/78
ARIZONA, U.S.A.

TABLE OF CONTENTS

RWCD Floodway Bridge, Santan Road

Preliminary Investigation of Structural Systems

Objective:

This report presents the results of preliminary investigation on the feasible superstructure and substructure systems for the proprosed San Tan Road crossing of the RWCD Floodway Canal located in Section 13, T.35, R.5E, Pinal County, Arizona. The objective of this study is to make recommendations as to the type of super and substructure systems that may be adopted to span the floodway.

1.10 Review of Design Criteria:

The structural systems were studied with reference to the following design criteria, as required by Maricopa County Flood Control District, U.S. Bureau of Indian Affairs and Soil Conservation Service, U.S. Department of Agriculture. The structure will be designed according to AASHTO Specifications for Highway Bridges.

1.11 Loads:

The live load the bridge should carry is HS-20 with the necessary impact factor as specified by AASHTO. Necessary provision of 25 lbs per square foot for future wearing surface will be made in the dead load of the deck slab.

1.12 Serviceability:

Apart from the strength requirements, the serviceability of the deck will be maintained by controlling the live load deflections and crack widths in the concrete under ultimate design conditions. The deck will be cambered for dead load deflection.

1.13 Channel Section Requirements:

The channel requirements are stipulated by the Soil Conservation Service of U.S. Department of Agriculture in their letter dated May 5, 1978, addressed to Flood Control District of Maricopa County and by the Flood Control District in their letter dated September 22, 1978 to Wheeler, Brooks Cofeen, Inc. The proposed channel cross section at the bridge site has accordingly been established by Wheeler, Brooks, Coffeen, Inc. The profile of the bridge arrived at based on this cross section is shown in the longitudinal section of the roadway on page 10. In adopting this cross section of channel for the bridge, the cost of the spill abutment with 2:1 slope and the additional length of the bridge was compared with the cost of vertical abutment with wing walls and the former was found to be less expensive.

The bridge length center to center of bearings is estimated to be 234 feet or end to end of abutments 237 feet. Accordingly the following combination of number of spans and span lengths provide a guide to make a selection of the number of spans and the corresponding span lengths.

Interior Spans	End Spans
*6 x 30'-0	2 x 27'-0
*5 x 35'-0	2 x 29'-6
*4 x 40'-0	2 x 37'-0
**4 x 39'-0	2 x 39'-0
**2 x 58'-6	2 x 58'-6

*Continuous deck
**Simple span deck

1.20 Feasibility and Material Selection:

A bridge structure conforming to the criteria stated above is feasible at this site.

The structure, being a permanent one with a channel underneath the roadway, timber is not considered for any part of the structure. Reinforced concrete with fc' = 3,000 p.s.i. to 4,000 p.s.i. with grade 40 and 60 steel for reinforcement, prestressed concrete up to fc' = 5,000 p.s.i. and structural steel (A-36) are considered for the material of the structure.

1.30 TYPE OF FOUNDATION SYSTEM:

Three types of foundation systems and the corresponding design criteria have been mentioned in the Soil Engineer's report which along with the additional information regarding H piles obtained from them provide adequate information in our opinion. Spread footings were proposed at 3' to 5' depth for abutments and at 5' to 6' depth for piers. The boring logs indicated silty clay up to 12'+ in depth. It is felt, for reasons of settlement problems in this type of soil, spread footings to support the structure are not prefereable. The other types of foundations, circular drilled pier or caisson and driven H piles are considered suitable. To achieve the required capacity of driven H piles to support the anticipated loads, the Soil Engineers report that the piles are to be driven to depths of 30' and more below future channel bed. Comparing this with the caisson depth of 10 to 15 feet below scour depth at the pier locations, drilled caisson foundation is found to be less expensive. The final depths of caissons will depend also on the reponse of the soil due to lateral loads.

1.40 Pier Types:

The feasibility of wall piers, column bents and continuation of H-piles to underneath the superstructure deck were examined. Our inquiries in this regard show that it has been the experience of Arizona Department of Transportation, that the present day seismic design requirements impose use of heavy reinforcement in the wall piers and for that reason the circular column bents are economical and have been found to resist seismic and transverse lateral forces better.

Further, scouring around the piers is minimized with the open piers. Continuation of H piles requires curtain wall around the piles for the width of the bridge and this arrangement along with driving the piles to about 30 feet below grade is found to be more expensive than the circular column bents with drilled in caissons. Approximate cost of piers supported by caissons, ranges from $5 to $8 per square foot. Approximate cost of piers supported by piles is $13 per square foot.

1.50 Deck Types

It is learnt from Wheeler, Brooks, Coffeen, Inc. that the existing roadway profile in relation to the cross section of channel allows the flexibility of considering decks of reasonabl depths for the superstructure. The following types are considered.

1) Cast in place slab.
2) Precast prestressed voided slab.
3) Precast prestressed box girders.
4) Rolled steel beam with composite or non-composite deck slab.

An approximte cost comparison of these systems along with the overall depth of deck based on our preliminary study is shown below. The prices reflect only the cost of the deck and the substructure based on circular column bents and drilled caisson foundation.

Type	Approximate Cost in $ Per Sq. Ft. Span (ft) 30	35	40	45	60	Depth of Deck (in.)
Cast in Place Slab	22 (14+8)	22 (15+7)	23 (17+6)	24 (19+5)	N.R.	15.5 to 21.4 4 to 6 in. haunch
Voided Slab	22 (14+8)	22 (15+7)	22 (16+6)	23 (18+5)	N.R.	20 to 26
Box Girder	N.R.	N.R.	N.R.	N.R.	23 (19+4)	29
Rolled Steel Beam	N.R.	N.R.	N.R.	22 (17 +5)	24 (20+4)	35 to 41

N.R. = Not recommended

First figure within peranthesis = Cost of supersturcture

Second figure within peranthesis = Cost of substructure

Support Data:

Unit Costs: Cast In place formed Concrete: $200 to $240/cyd.

Rebar: 35¢/lb. STructural Steel: 60¢/lb. Caissons: $70/ft.

Piles: $25/ft. Prestressed slabs: $15 to $18 per sq. ft.

Preliminary Sizes: See Sketches attached.

1.60 Concrete Barrier and Hand Rail:

The standard concrete barrier for the roadway sides adopted by the Arizona Department of Transportation is shown in the cross sections on Pages 8 and 9. On the walkway side, chain link fence 6'-0 high over a concrete curb of 10" x 9", similar to the type adopted by the Arizona Highway Department is propose.

1.70 Conclusion and Recommendations:

This preliminary study was undertaken to examine the feasible sturcutral systems for the superstructure and substructure for the San Tan Road Bridge with reference to the design criteria regarding loads and channel requirements and the foundation types recommended by the Soils Engineer. As seen in Section 1.50, the systems considred cost about the same assuming the same type of foundation. The cost per square foot of deck area is considered reasonable and hence a suitable bridge structure is feasible at the site.

Considering the existing roadway profile, the required elevation of bottom of superstructure and the required depth of superstructure, solid slab deck or voided slab deck would be a suitable structure resulting in reasonable approach gradients.

Apart from the competitive cost (Section 1.50), the inspection needs and the construction time are expected to be less for the precast prestressed voided slab deck. The lesser amount of concrete required for the voided slab deck as compared to the solid slab, is also an added advantage in view of cement shortage.

Accordingly, voided slab deck with six spans of 39'-0 center to center of abutments and piers, is recommended for the superstructure.

For substructure, 3 column bents at each pier location with drilled in caissons are recommended.

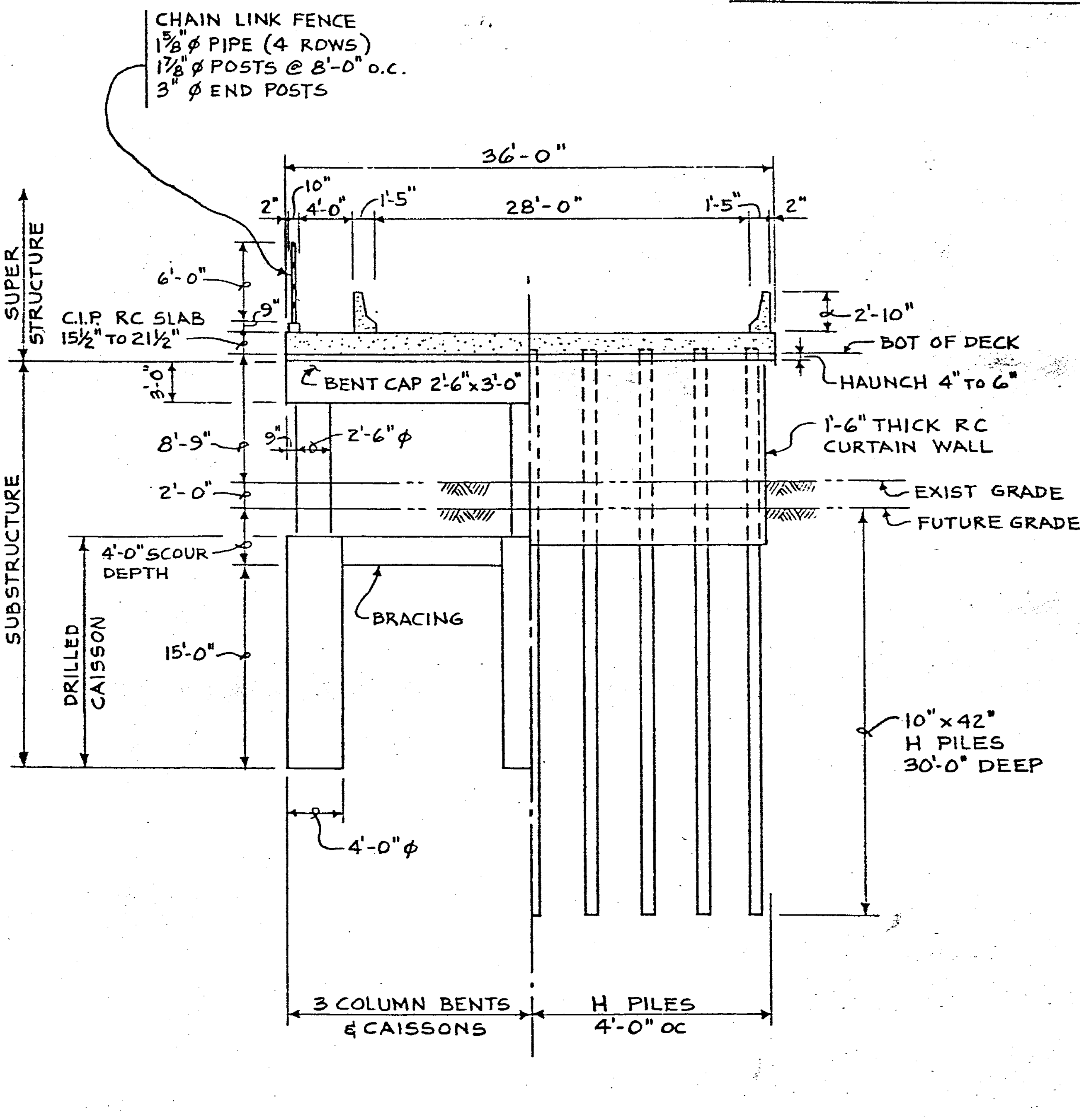
Based on the above recommendations, the cost of super and substructure including barrier and handrail is estimated to be $205,000. This does not include canal lining, riprap for the canal bed, formation of canal banks, pavement, approaches and over-head charges.

DATE	JOB NO.	SHEET NO. 1
8-22-78	7847	OF 3

JOB: SAN TAN ROAD CROSSING

BY: HJR

MAGADINI-ALAGIA ASSOCIATES STRUCTURAL ENGINEERS, INC.



CROSS SECTION OF FEASIBLE STRUCTURAL SYSTEM

DATE	JOB NO.	SHEET NO. 2 OF 3
8-25-78	7847	
JOB: SAN TAN ROAD CROSSING		
BY: HJR		
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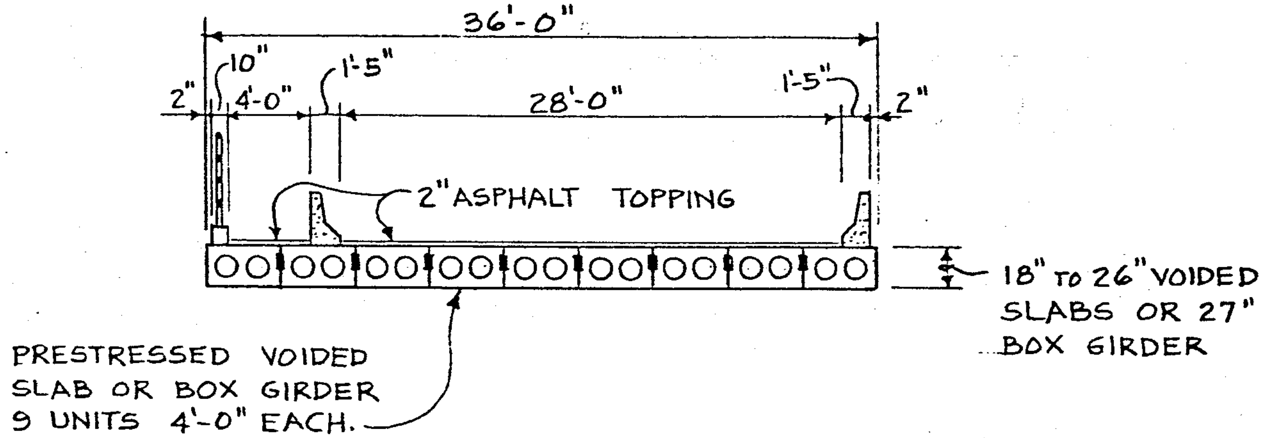
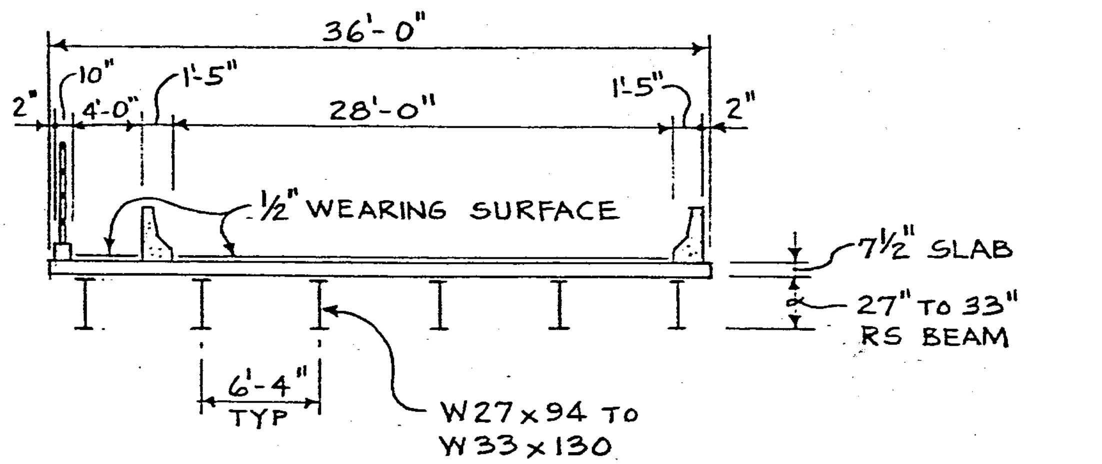


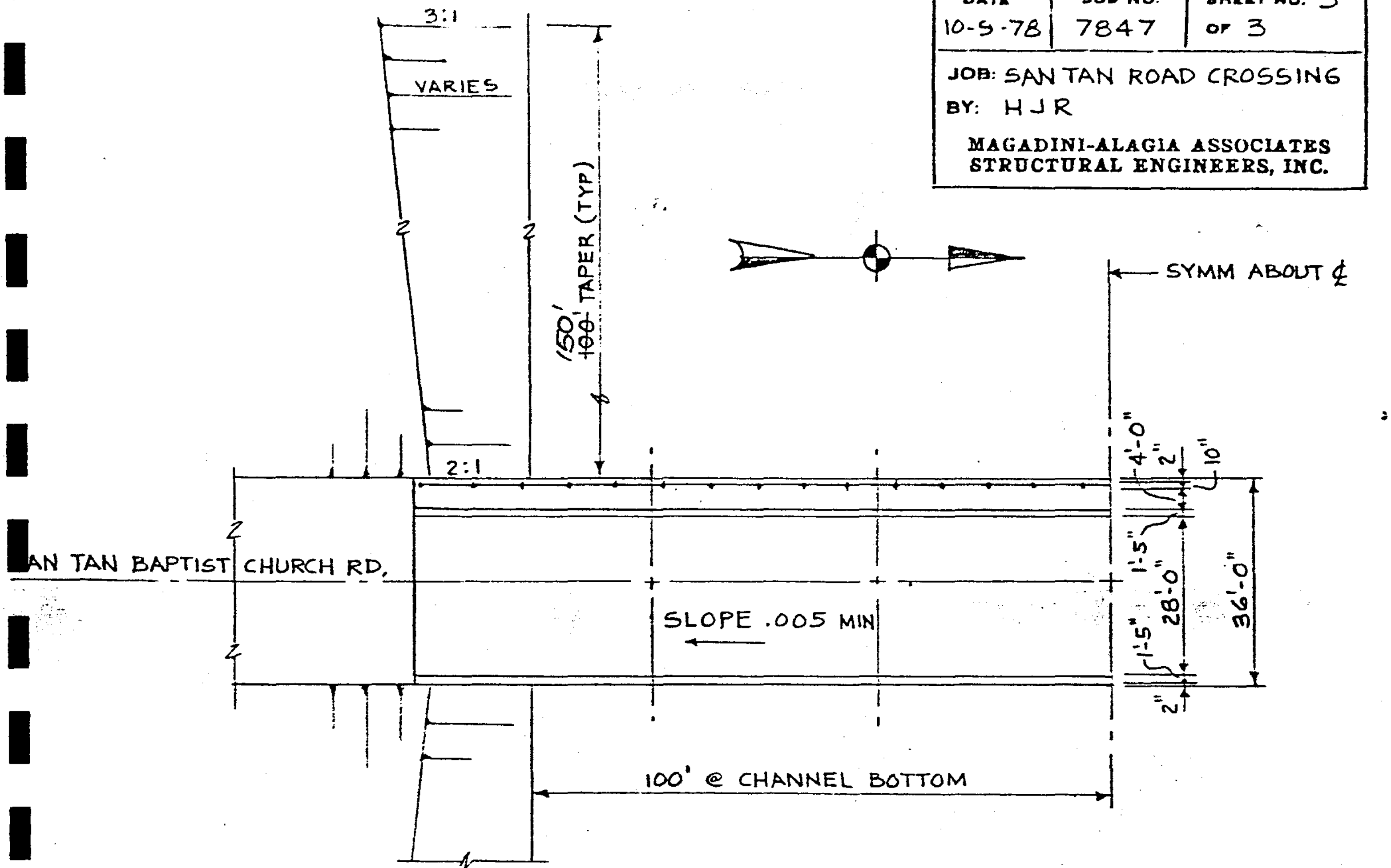


CROSS SECTION OF FEASIBLE STRUCTURAL SYSTEM

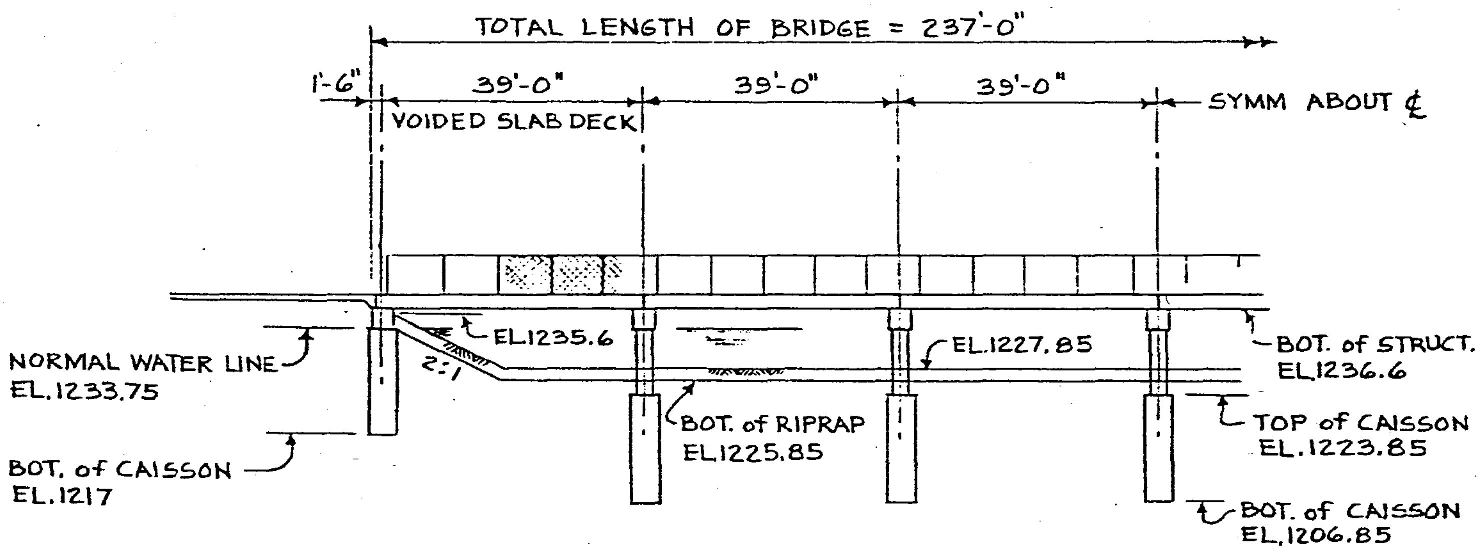
DATE	JOB NO.	SHEET NO. 3
10-5-78	7847	OF 3

JOB: SAN TAN ROAD CROSSING
BY: HJR

MAGADINI-ALAGIA ASSOCIATES
STRUCTURAL ENGINEERS, INC.



LOCATION PLAN



LONGITUDINAL SECTION ON ℄ ROADWAY

<u>Personnel contacted during the preparation of this report:</u>

1. Dinesh Doshi, Wheeler, Brooks, Coffeen, Inc.
2. Eng Tan, Maricopa County Highway Department, Phoenix.
3. Whitaker, Bridge Division, BIA, Phoenix
4. Glen Copeland, ETL, Phoenix.
5. Structures Section, A.D.O.T.

References:

1. AASHTO Specifications for Highway Bridges.
2. A.D.O.T. Standard Drawings for Slabs.
3. ReynoRail II Pipe Railing System, Reynolds Aluminum Co.
4. A.D.O.T. Drawings for RWCD Flood Channel Bridge at Station 190+.
5. Precast Prestressed Concrete Short Span Bridges, Prestressed Concrete Institute.
6. Preliminary Foundation Evaluation on the Bridge by ETL, Inc.
7. Preliminary Channel Cross section and roadway profile by Wheeler, Brooks, Coffeen, Inc.

APPENDIX "C"

SAN TAN BAPTIST CHURCH ROAD BRIDGE

HYDRAULIC & HYDROLOGY REPORT

TABLE OF CONTENTS

1. The Design Criteria and Basic Information

The hydraulic design of the bridge is based on criteria, data and information furnished by the U.S. Department of Agriculture - Soil Conservation Service which can be summarized as follows:

1.1 <u>The Channel Bottom Elevation</u> (Based on Table 3A, SCS, dated August 1977)

- <u>Going Down Station</u>

$$1193.3 + (145823.00 - 123539.27)\ 0.00155 = 1227.84\text{ft.}$$

- <u>Going Up Station</u>

$$1239.5 - (123539.27 - 116029.00)\ 0.00155 = 1227.86\text{ft.}$$

- <u>Design Channel Bottom Elevation</u>

$$EL = \frac{1227.84 + 1227.86}{2} = \underline{1227.85\text{ft}}.$$

1.2 <u>Water Surface Elevation</u> (Based on Table 3A, SCS)

- <u>Going Down Station</u>

$$1199.3 + (145823.00 - 123539.27)\ 0.00155 = 1233.84\text{ft.}$$

- <u>Going Up Station</u>

$$1245.4 - (123539.27 - 116029.00)\ 0.00155 = 1233.76\text{ft.}$$

- <u>Design Water Surface Elevation</u>

$$EL = \frac{1233.84 + 1233.76}{2} = \underline{1233.80\text{ft}}.$$

1.3 <u>Mean Normal Water Depth</u>

$$d_n = 1233.80 - 1227.85 = \underline{5.95\text{ft}}.$$

1.4 <u>Water Cross Section Area</u>

Assuming channel side slopes at 3:1

$$A = \frac{(b + 6d_n + b) \times d_n}{2} = (b + 3d_n)\ d_n$$

Where

b - Channel bottom width = 200'

d_n - Mean normal water depth = 5.95'

A =(200+3x5.95)=<u>1296.21</u> sq. ft.

1.5 <u>Mean Water Velocity</u>

Assuming Q = 8700 cfs

$$v = \frac{Q}{A} = \frac{8700}{1296.21} = \underline{6.71 \text{ cfs}}$$

1.6 <u>Verify Critical Depth</u> (for unconstricted channel)

$$d_c = \frac{4zH_o - 3b + \sqrt{16z^2H_o^2 + 16zH_ob + gb^2}}{10z}$$

Where

z - Channel side slope = 3:1 = 3

b - Channel bottom width = 600'

H_o - Specific head $H_o = d + \frac{v^2}{2g} = 6.65'$

$$d_c = \frac{4x3x6.65 - 3x200 + \sqrt{16x3^2x6.65^2 + 16x3x6.65x200 + 9x200^2}}{10x3}$$

$d_c = 4.52 < d_n = 5.95$ Therefore the flow in the channel <u>remains subcritical</u>

1.7 <u>The Channel Bottom Roughness</u>

Based on the preceding data, we can re-create the Mannings "n" factor used by the SCS Design.

$$n = \frac{1.486}{8700} xAxR^{2/3}xS^{1/2} =$$

$$= \frac{1.486}{8700} x1296.21x5.455^{2/3}x0.00155^{1/2} = 00.027$$

Where

$$R = \frac{A}{P};\quad p = 200+2\sqrt{3x5.95^2+5.95^2} = 237.63 \text{ ft.}$$

2. <u>The Water Depth Analysis</u>

The flow of water in wide and relatively shallow channels is highly influenced by the roughness of its wetted perimiter.

The actual value of Manning's "n" will depend upon two factors:

a. Maintenance of the channel

b. The rate of vegetative growth

The flat channel bottom will encourage vegetative growth. therefore the maintenance of the channel to the extent that the average roughness of its bottom will remain below assumed value of Manning's "n" = 0.027 may prove to be costly and difficult. Assuming substantial vegetative growth from limited maintenance conditions, the roughness "n" may tend to reach the values shown on sketch #1. Under this likely condition, the water depth in the channel was calculated by trial and error. For the assumed peak flow of 8700 cfs, the water depth would reach 6.9 feet or 1.0 feet above the normal depth of 5.95 feet.

Water depth was determined by trial and error.

- <u>Check of the Flow Capacity</u>

$$Q=2Q_1+2Q_2+2Q_3+2Q_4+2Q_5$$

$$Q_1=\frac{1.486}{0.045}\left(\frac{3x6.9x6.9}{2}\right)\left[\frac{3x6.9x6.9}{2x\sqrt{(3x6.9)^2x6.9^2}}\right]0.00155^{1/2}=204.9 \text{ cfs}$$

$$Q_2=\frac{1.486}{0.045}(10x6.9)\left(\frac{10x6.9}{10}\right)^{2/3}0.00155^{1/2}=325.1 \text{ cfs}$$

$$Q_3=\frac{1.486}{0.040}=(30x6.9)\left(\frac{30x6.9}{30}\right)^{2/3}0.00155^{1/2}=1097.2 \text{ cfs}$$

$$Q_4=\frac{1.486}{0.035}(30x6.9)\left(\frac{30x6.9}{30}\right)^{2/3}0.00155^{1/2}=1254.0 \text{ cfs}$$

$$Q_5=\frac{1.486}{0.035}(30x6.9)\left(\frac{30x6.9}{30}\right)^{2/3}0.00155^{1/2}=1462.8 \text{ cfs}$$

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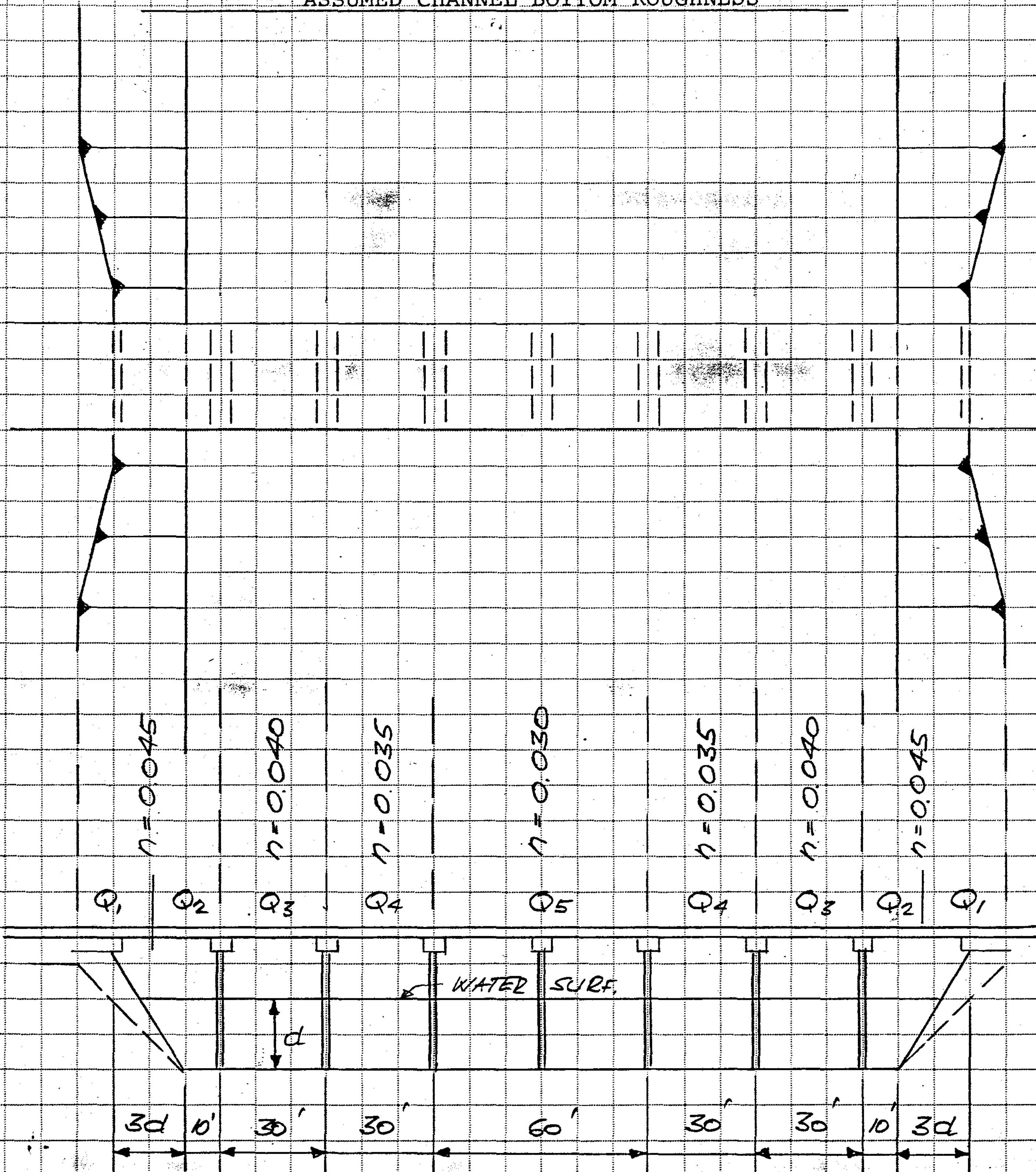
ASSUMED CHANNEL BOTTOM ROUGHNESS



SKETCH # 1

Q=2x204.6+2x325.1+2x1097.2+2x1254.0+2x1462.8=8687.4

Q=8687.4cfs ~ 8700cfs

3. Hydraulic Analysis of the Selected Structural Alternative

The selected bridge structure is as follows:

- Span ± 40 feet
- Deck - precast, prestressed voided slab
- Substructure - drilled caisson foundation

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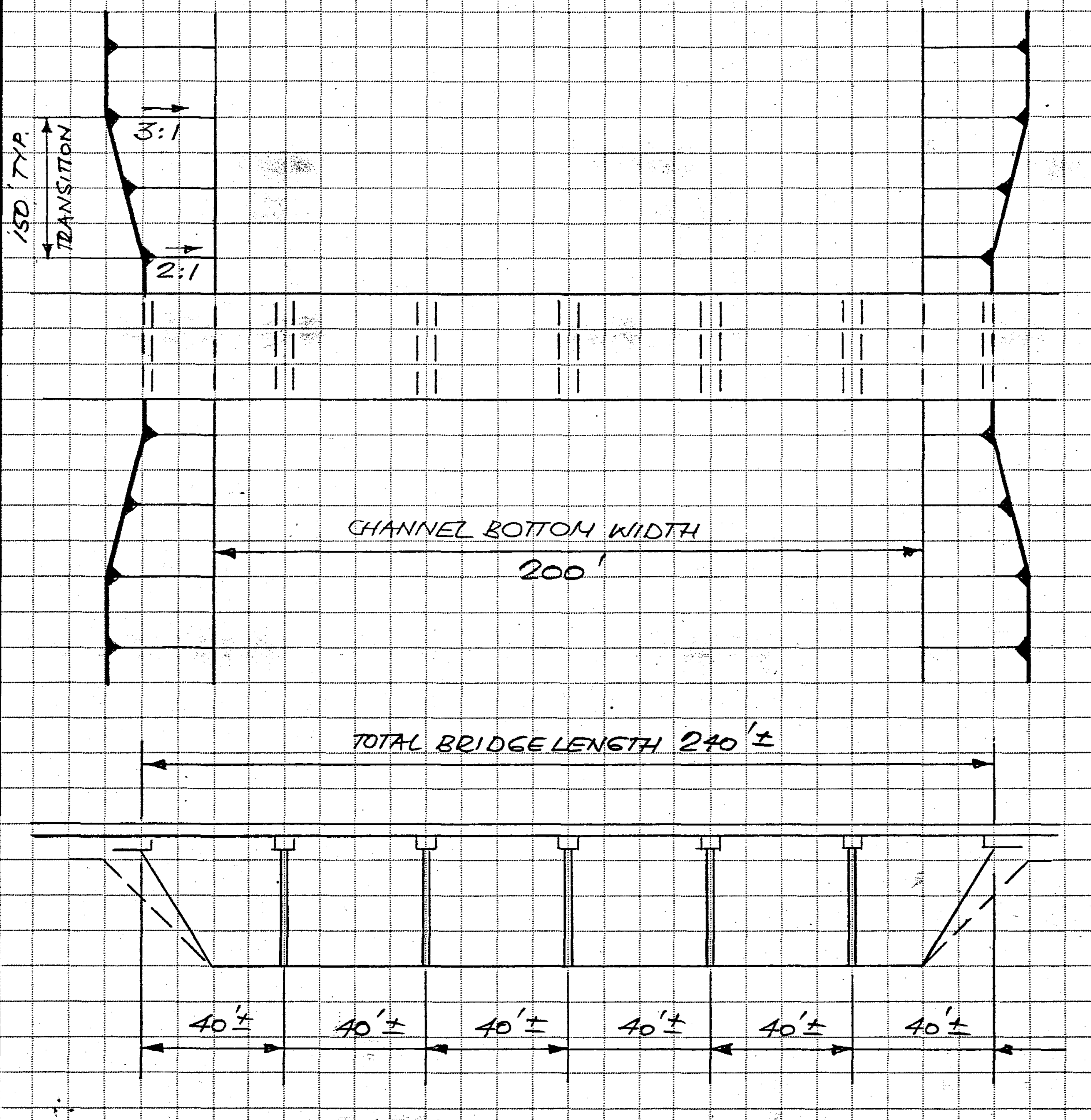

SKETCH #2

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	SUB SECTION	n	$\frac{1.486}{n}$	a	p	r	$r^{2/3}$	k	q	V	qV^2
				SQ.FT.	FT.	FT.			C.F.S	FPS	
Q_A	0-11	0.045	33.02	19.08	11.28	1.69	1.420	894	35.2	1.85	119
Q_B	11-31	0.045	33.02	121.34	20.54	5.91	3.268	13092	515.4	4.25	9300
	31-61	0.040	37.15	207.0	30.00	6.90	3.624	27871	1097.3	5.30	30834
	61-91	0.035	42.46	207.0	30.00	6.90	3.624	31855	1254.1	6.06	46035
	91-121	0.030	49.53	207.0	30.00	6.90	3.624	37159	1462.9	7.07	73070
	121-151	0.030	49.53	207.0	30.00	6.90	3.624	37159	1462.9	7.07	73070
	151-181	0.035	42.46	207.0	30.00	6.90	3.624	31855	1254.1	6.06	46035
	181-211	0.040	37.15	207.0	30.00	6.90	3.624	27871	1097.3	5.30	30834
	211-231	0.045	33.02	121.34	20.54	5.91	3.268	13092	515.4	4.25	9300
Q_C	231-242	0.045	33.02	19.08	11.28	1.69	1.420	894	35.2	1.85	119

$A_{n_1} = 1523.0$

$A_{n_2} = 1485.0$

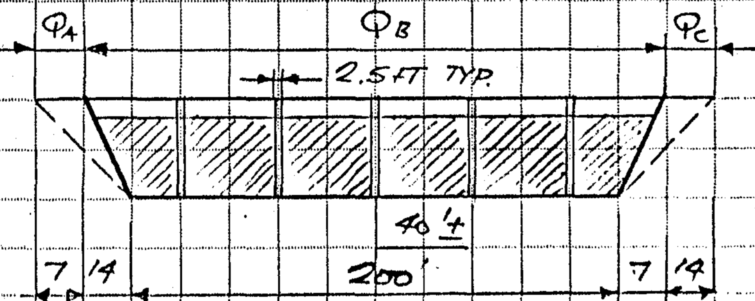
$K_1 = \Sigma k = 221742$ $\quad \Sigma\ 8730$ $\quad \Sigma qV^2 = 318716$

WHERE:

$$r = \frac{a}{p}$$

$$k = \frac{1.486}{n}\, a\, r^{2/3}$$

$$q = k \times S_o^{1/2} \; ; \quad V = \frac{q}{a}$$



FORM 204 Available from NEBS Inc. Townsend, Mass 01470

3.1 Velocity Head Coefficient at Section 1 (α_1)

$$\alpha_1 = \frac{qv^2}{Q_{Vn}{}^2}$$

Where

$V_{n1} = \frac{Q}{A_{n1}} = \frac{8730}{1523} = 5.73$fps. ; $qv^2 = 318,716$ (Page 9)

$$\alpha_1 = \frac{318,716}{8730 \times 5.73^2} = 1.11$$

3.2 Bridge Opening Ratio (M)

$$M = \frac{Q_B}{Q}$$

$$M = \frac{Q_B}{Q_A + Q_B + Q_C} = \frac{8659.6}{8730} = 0.99$$

3.3 Velocity Head Coefficient for Constriction (α_2)

For $\alpha_1 = 1.11$ and $M = 0.99$

$\alpha_2 = 1.10$ (See Figure 1)

3.4 Backwater Coefficient (K_b)

For M=0.99 and spillthrough abutments

$K_b = 0.01$ (See figure 2)

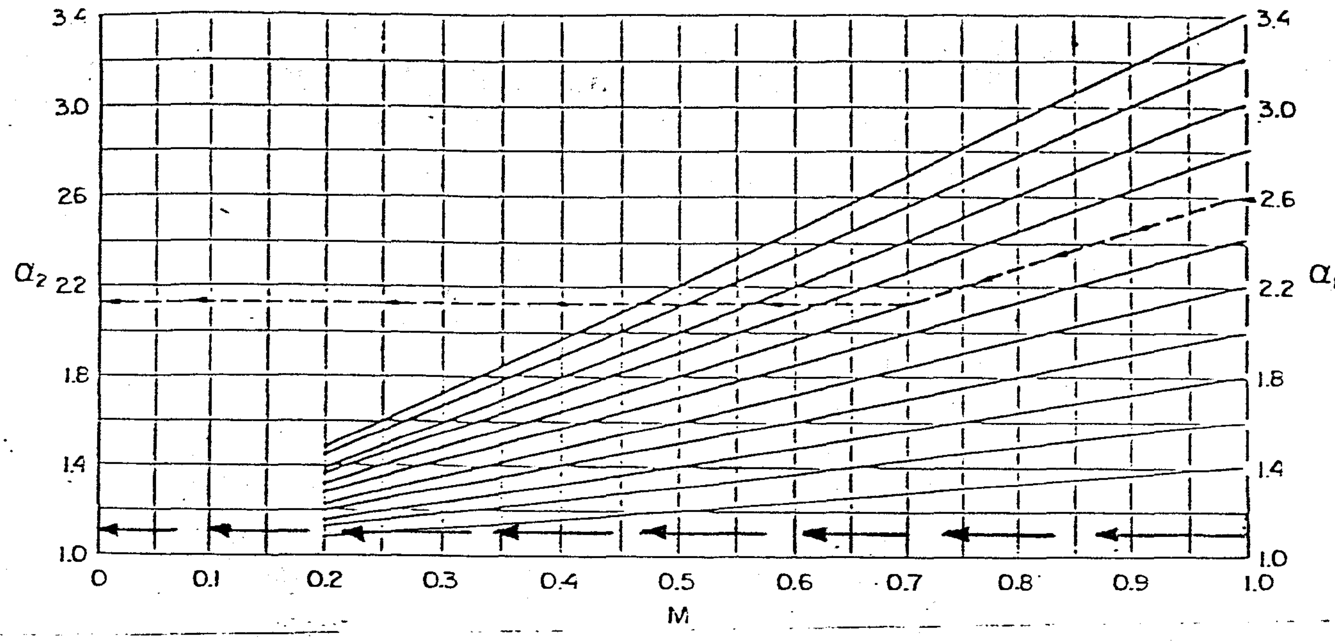


Figure 1 - Aid for Estimating α_2

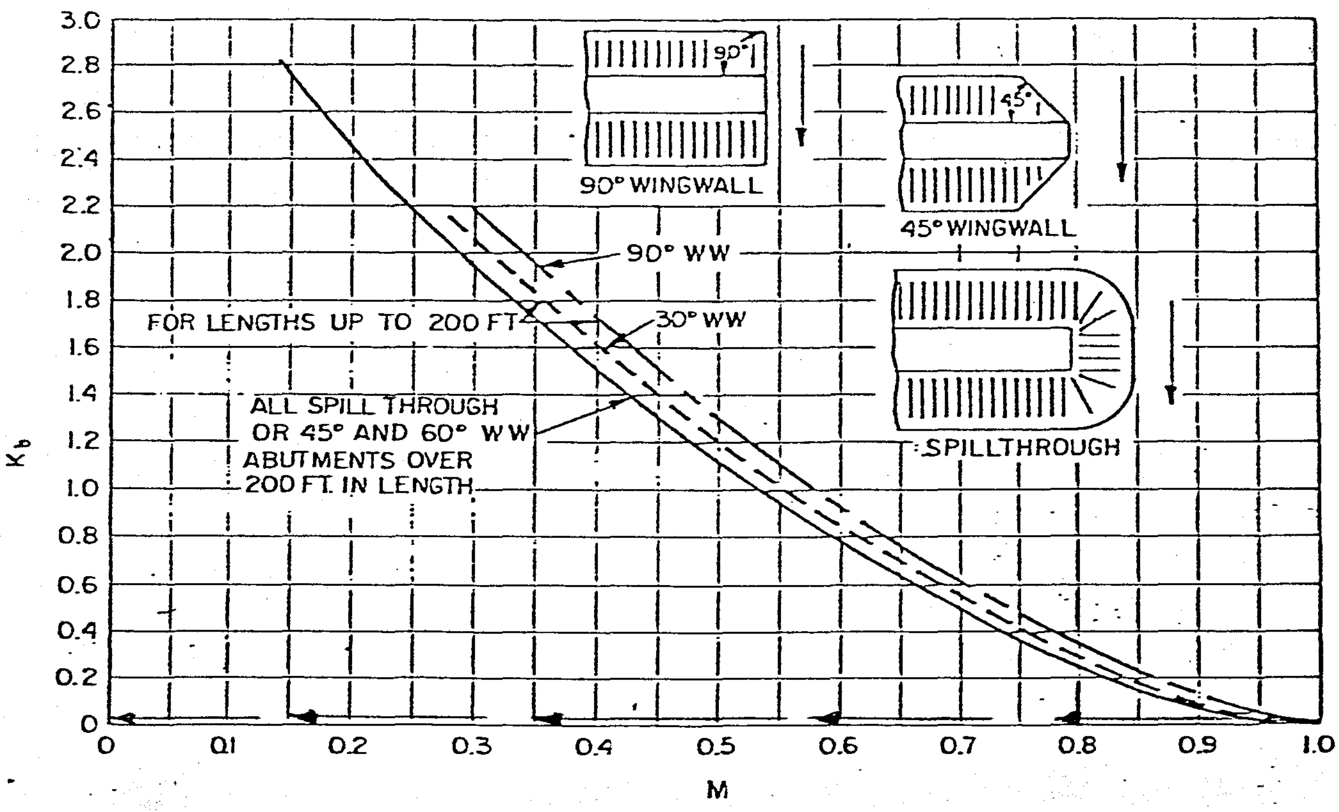


Figure 2 - Backwater Coefficient Base
Curves (Subcritical Flow)

3.5 Projected Area Obstructed by Piers Normal to Flow (A_p)

A_p = 5x2.5x1.5*x6.9=129.38 sq. ft.

3.6 Ratio of Area Obstructed by Piers to Gross Area of Bridge Waterways at Section 2 (J)

$$J = \frac{A_p}{A_{n2}} = \frac{129.38}{1485} = 0.087$$

3.7 Incremental Backwater Coefficient for Piers (ΔK_p)

For J=0.087 & solid piers ΔK=0.351 (See figure 3A)

For M=0.99 & solid piers, the correction factor σ = 0.99 (See figure 3B)

$$\Delta K_p = \Delta K \times \sigma = 0.351x0.99=0.35$$

3.8 The Total Backwater Coefficient (K*)

$$K^* = K_b + \Delta K_p = 0.01+0.35=0.36$$

3.9 Average Velocity in Constriction for Flow at Normal Stage (V_{n2})

$$V_{n2} = \frac{Q}{A_{n2}-A_p} = \frac{8730}{1485-129.38} = 6.44\text{fps}$$

3.10 First Approximation of Backwater

$$h_1 = K^* \; 2 \; \frac{V_{n2}}{2g}$$

$$h_1 = 0.36x1.10x\frac{6.44^2}{2x32.2} = 0.248 \text{ ft.}$$

* Debris Factor =1.5 (assumed)

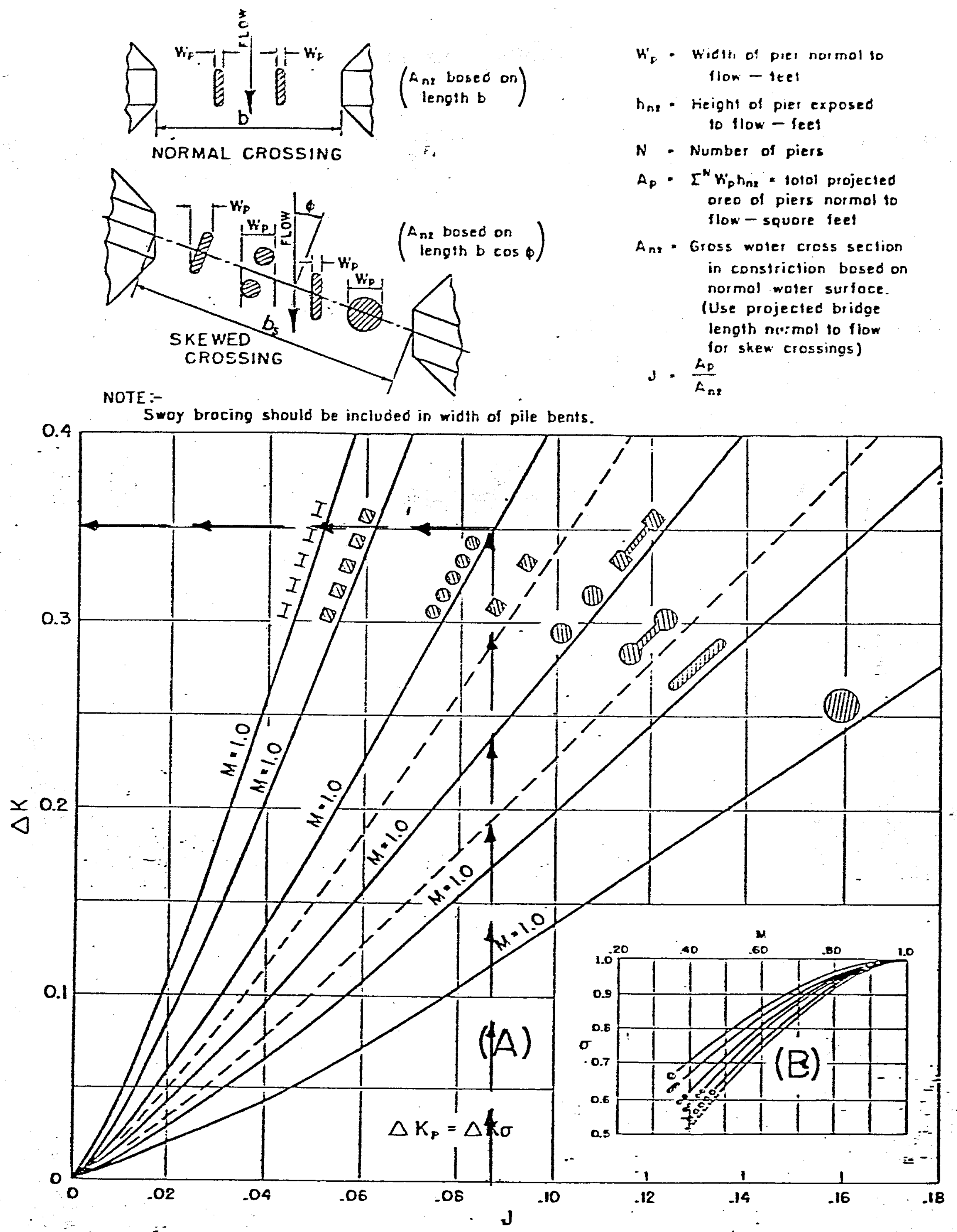


Figure 3 - Incremental Backwater Coefficient for Piers

3.11 <u>Area of Flow at Section 4</u> (Down Stream from the Bridge) (A_4)

$$A_4 = A_{n1} = 1523 \text{ sq. ft.}$$

3.12 <u>Area of Flow Including Backwater at Section 1</u> (A_1)

$$A_1 = A_{n1} + 0.248[200+2x3(6.9+0.248)] = 1523+60.2 = \underline{1583.2 \text{sq ft}}$$

3.13 <u>Total Backwater Produced by the Bridge at Section 1</u> (h^*_1)

$$h^*_1 = K^* \alpha_2 \frac{V_{n2}^2}{2g} + \alpha_1 \left[\left(\frac{A_{n2}}{A_4}\right)^2 - \left(\frac{A_{n2}}{A_1}\right)^2 \right] \frac{V_{n2}^2}{2g}$$

$$h^*_1 = 0.248 + 1.11\left[\left(\frac{1485}{1523}\right)^2 - \left(\frac{1485}{1583.2}\right)^2 \right] x \frac{6.44^2}{2x32.2}$$

$$= 0.248 + 0.050 = \underline{0.30 \text{ ft}}.$$

3.14 <u>Distance from Point of Maximum Backwater to the Water Surface on the Down Stream Side of Roadway Embankment</u> (L)

$$b = \frac{200+200+2x2(6.90+0.30)}{2} = 214.4 \text{ ft.}$$

Where

b = width of constriction

$$\bar{y} = \frac{A_{n2}}{b} = \frac{1485}{214.4} = 6.93 \text{ ft.}$$

Assuming $\Delta h = 0.5$ ft.

$$\frac{\Delta h}{\bar{y}} = \frac{0.50}{6.93} = 0.072$$

For $\frac{\Delta h}{\bar{y}} = 0.072$ and $\bar{y} = 6.93$

$\frac{L}{b} = 0.41$ (See Figure 4)

$L = 0.41 x 214.4 = 87.9$ ft.

3.15 <u>Vertical Difference in Water Surface Elevation Across Roadway Embankment</u> (Δh)

$$h^*_b = K_b x \alpha_2 \frac{V_{n2}^2}{2g} = 0.01 x 1.1 \frac{6.44^2}{2x32.2} = 0.0071 \text{ ft.}$$

For M=0.99

$D_b = 0.030$ (See Figure 5)

$$\Delta h = h^*_1 + h^*_3 + S_0 L_{1-3}$$

$$h^*_3 = h^*_b \left(\frac{1}{D_b} - 1\right) = 0.0071\left(\frac{1}{0.030} - 1\right) = 0.23 \text{ ft.}$$

$$\Delta h = 0.248 + 0.23 + 36 x 0.00155 = \underline{0.53 \text{ ft}}.$$

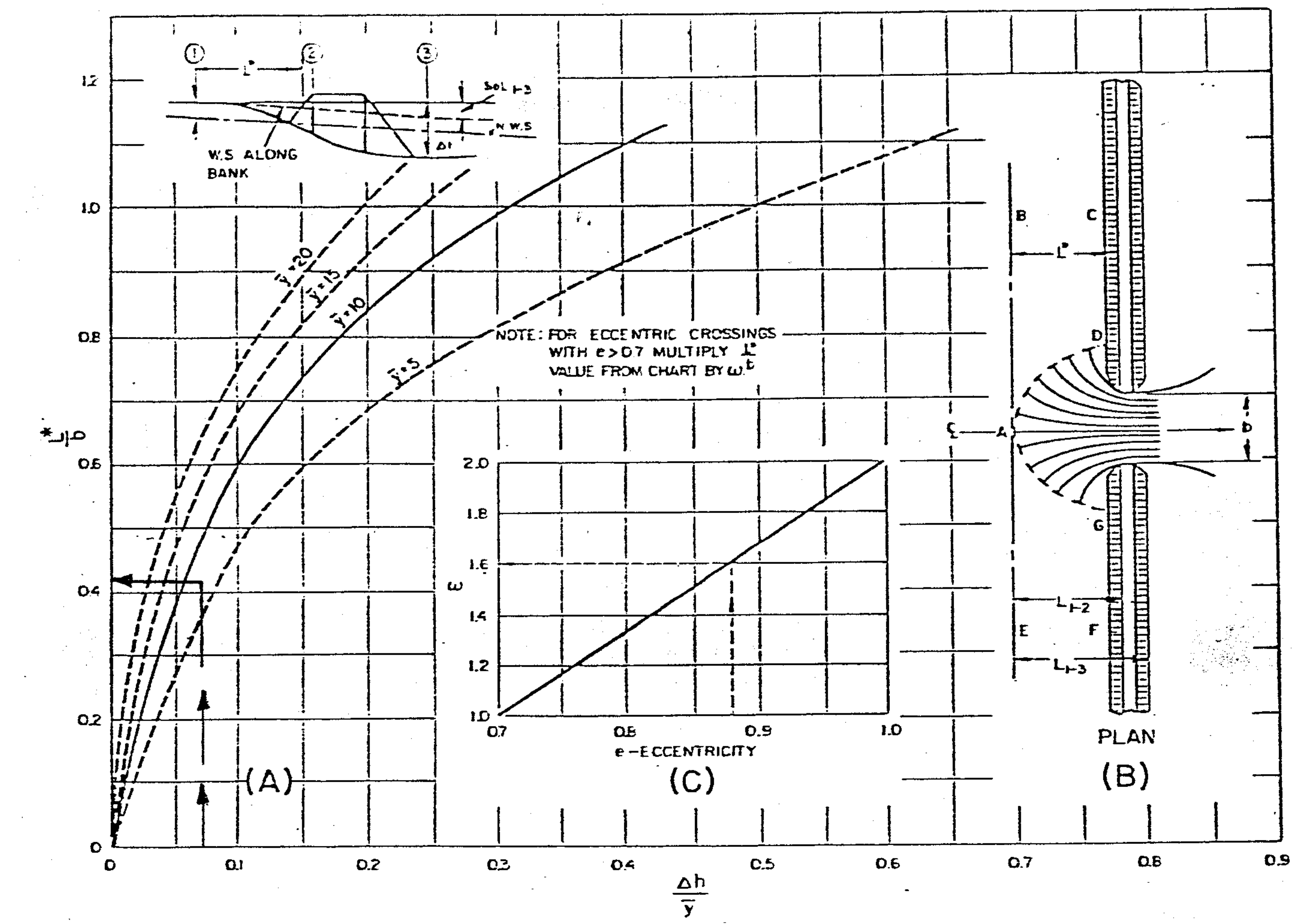


Figure 4 - Distance to Maximum Backwater

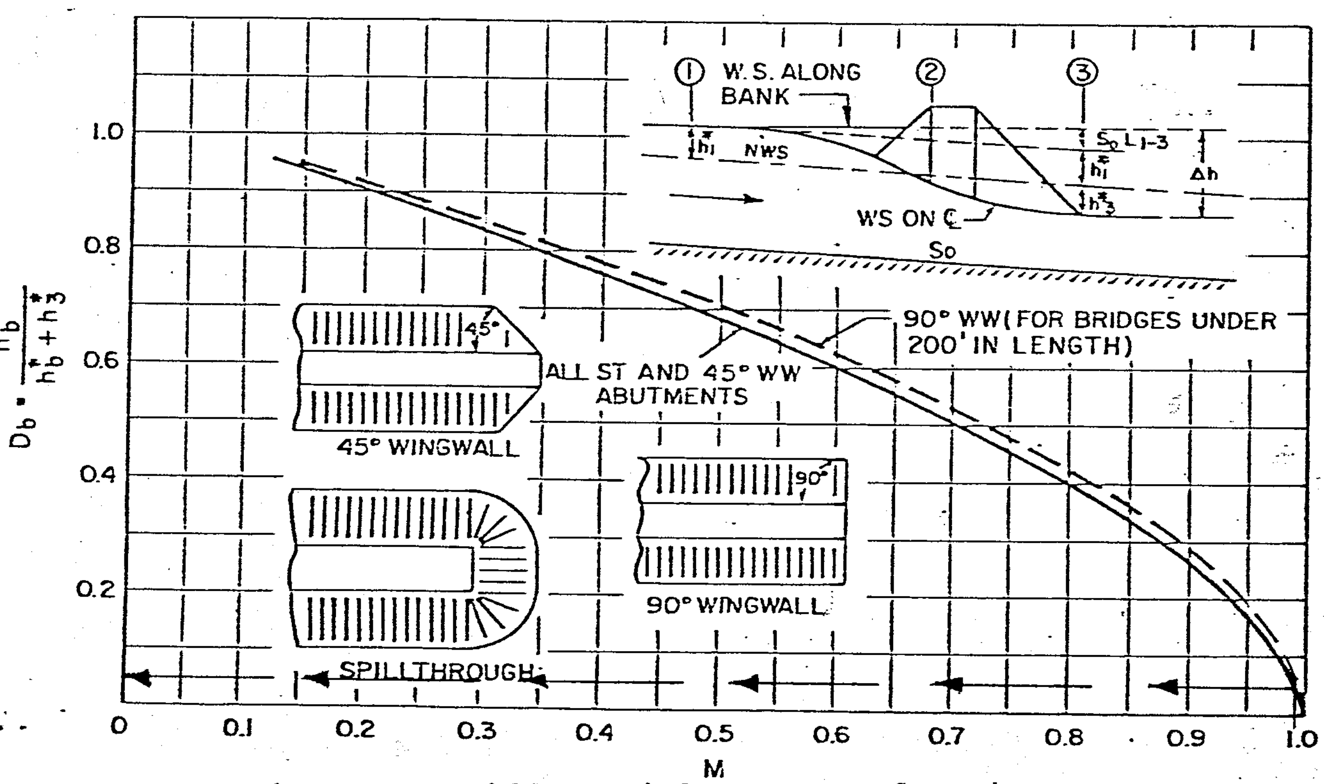


Figure 5 - Differential Water Land Ratio Base Curves

4. The Bridge Pier Loss

The pier loss at the bridge was determined by the momentum method. A curve for each section of the channel was plotted using the depth as the ordinate and values from Column I, II, and III as the abscissa. A vertical line passed through the curves represents a line of equal momentum for the corresponding depths of the flow at the bridge. Therefore, by locating the normal depth of flow (d=5.95'), and the hypothetical high water flow (d=6.90') on the curve #II, the solution was found for the pier loss at the bridge. (Graph. #1) which is less than the desired 3.5' maximum allowable head loss.

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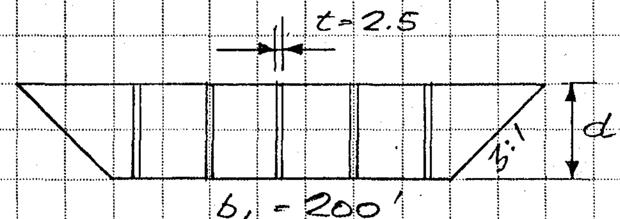
d	A_1	Y_1	M_1	A_0	Y_0	M_0	A_2	M_2	$\frac{Q^2}{gA_1^2}$	$\frac{Q^2A_2}{gA_1^2}$	$\frac{Q^2}{gA_2}$	$\frac{Q^2}{gA_3}$	I	II	III
			2×3			5×6	2-5	4-7					9+11	9+12	9+13
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
4	898	1.962	1664	91.00	1.883	171.3	757.0	1492.7	3.271	2476.5	3107.7	2774.2	3969.2	4600.4	4266.9
5	1075	2.442	2625	118.75	2.325	276.0	956.3	2349.0	2.036	1946.8	2468.0	2188.4	4295.8	4817.0	4537.4
6	1308	2.917	3816	148.50	2.758	409.5	1159.5	3406.5	1.375	1594.4	2028.9	1798.6	5000.9	5435.4	5205.1
7	1547	3.389	5243	180.25	3.183	573.7	1366.8	4669.3	0.983	1343.6	1721.2	1520.7	6012.9	6390.5	6190.0
8	1792	3.857	6912	214.00	3.601	770.7	1578.0	6141.3	0.733	1156.0	1490.8	1312.8	7297.3	7632.1	7454.1
9	2043	4.322	8829	249.75	4.014	1002.4	1793.3	7826.6	0.564	1010.8	1311.8	1151.5	8837.4	9138.4	8978.1
10	2300	4.783	11000	287.50	4.420	1270.8	2012.5	9729.2	0.445	895.0	1169.0	1022.8	10624.2	10898.2	10752

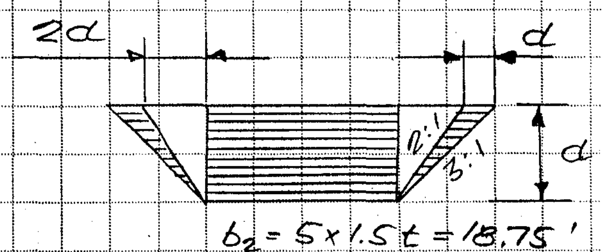
$$A_1 = \frac{(b_1 + b_1 + 6d)d}{2} = (b + 3d)d$$



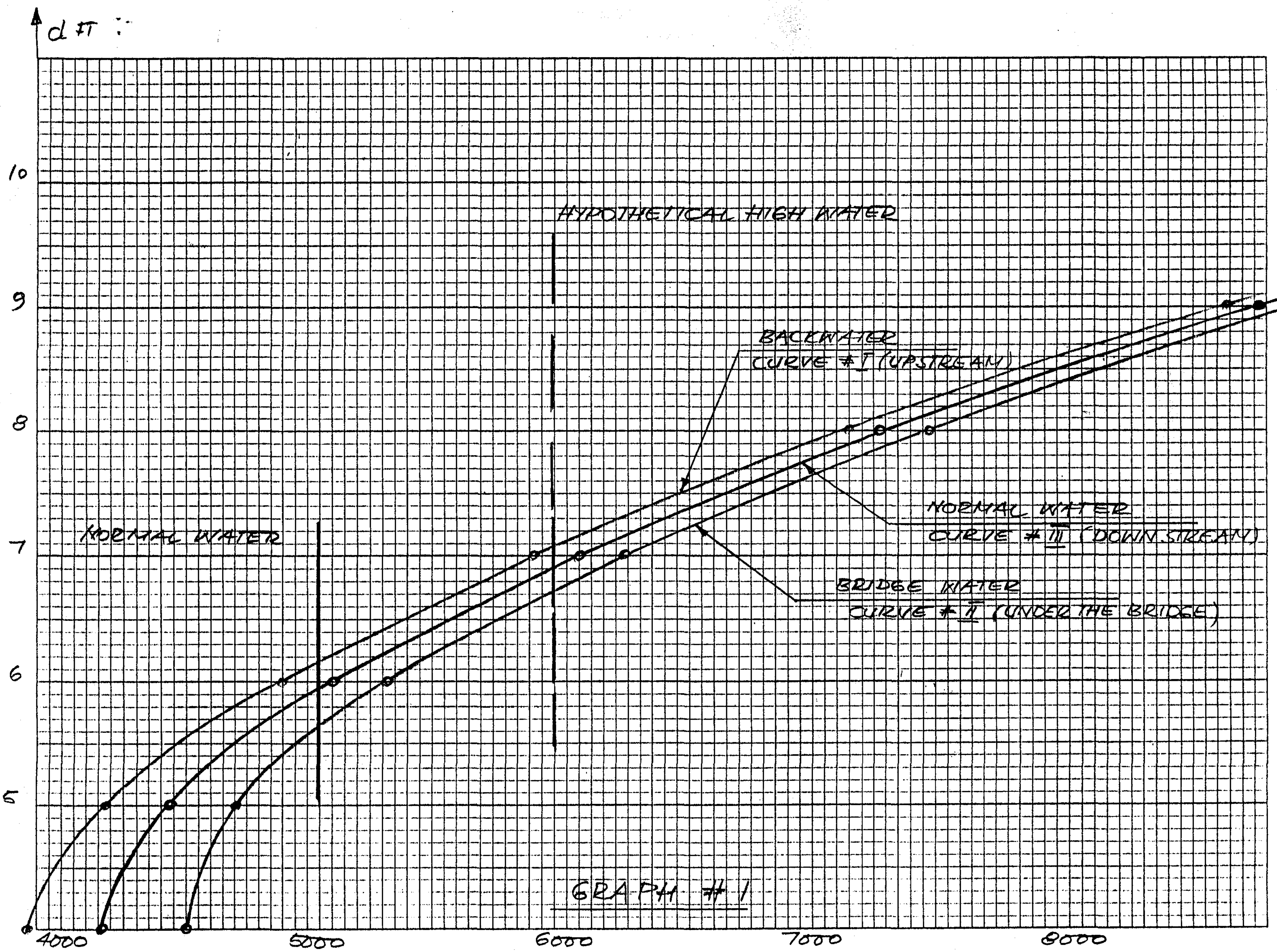
$$Y_1 = \frac{d(b_1 + 6d + 2b_1)}{3(b_1 + 6d + b_1)} = \frac{d(b_1 + 2d)}{2(b_1 + 3d)}$$

$$A_0 = \left(\frac{d \times 3d}{2} - \frac{d \times 2d}{2}\right)2 + 5d \times 1.5t = d^2 + 18.75d$$

$$Y_0 = \frac{d(b_2 + 2d + 2b_2)}{3(b_2 + 2d)} = \frac{d(1.5b_2 + d)}{3(b_2 + d)}$$



$M_1 = A_1 \times Y_1$; $M_0 = A_0 \times Y_0$; $A_2 = A_1 - A_0$; $M_2 = M_1 - M_0$



d FT
10
9
8
7
6
5
4000
5000
6000
7000
8000
HYPOTHETICAL HIGH WATER
NORMAL WATER
BACKWATER
CURVE # I (UPSTREAM)
NORMAL WATER
CURVE # III (DOWN STREAM)
BRIDGE WATER
CURVE # II (UNDER THE BRIDGE)
GRAPH # 1

5. The Scour and Channel Protection

According to the preliminary soils report by ETL - Job # 812-467, the materials to be exposed at the channel bottom (silty medium and fine sands) will be subjected to erosion and scour if the water velocity is greater than 1 FPS.

The mean velocity of the water in the proximity of the bridge will be in the range of 6 - 7 FPS. This indicates that the scour could be extensive and therefore the channel bottom and side slopes should be protected.

5.1 Stone Size Selection

For the velocity of 7 FPS

$D_{50} = 0.43$ ft (See Figure 6) Use 6" min.

The maximum stone diameter

$D_{100} = 1.0$ ft.

5.2 Thickness of Riprap

Preliminary investigation indicates that the necessary thinkness of riprap for embankment protection should be within the range of 1'-6" to 2'-0"

5.3 Length of Riprap

Based on preliminary investigation it is recommended that the riprap should be placed from about 100 ft. upstream to 100 ft. downstream of the bridge.

5.4 Type of Riprap

Based on the design flow of 8600 CFS and the velocity of 7 FPS, it is felt that the grouted riprap with cut-off walls would be better for the long term stability and to minimize maintenance.

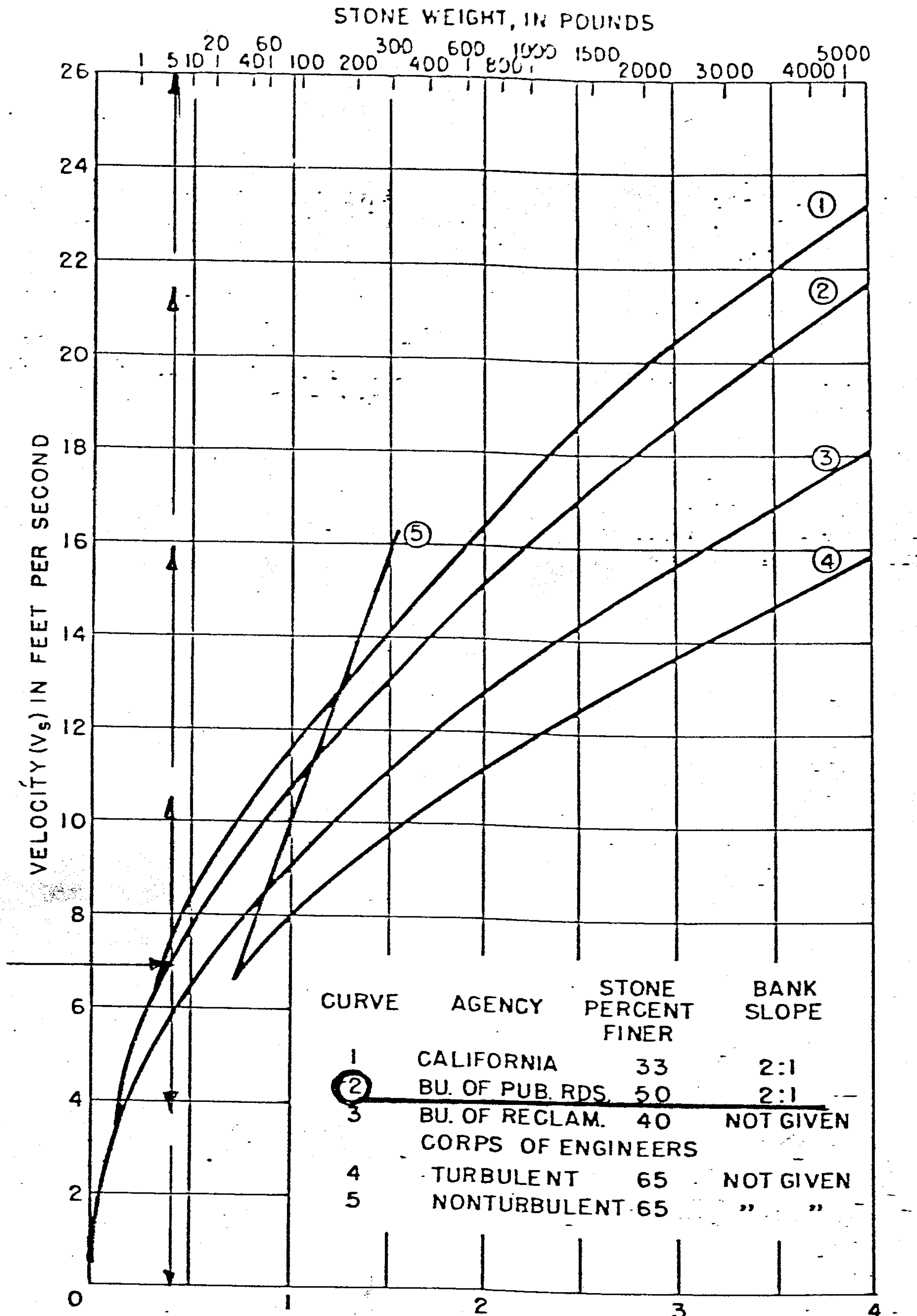


FIG. 9—COMPARISON OF SIZE BY VARIOUS METHODS

Figure 6

5.5 Cost of Riprap

For an assumed thickness of 2'-0", the cost of riprap will be

$$250 \times 236 \times 2.0 \times \frac{\$30.00}{27} = \$131,000$$

Where

250' - width

100' - length one each side of the proposed bridge = 236' total length of riprap

2' - thickness (assumed)

$30 - unit cost per c. yard

References

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