PLAN PREPARATION

BOOK III

Prepared By The Bridge Division

Of

The Texas Highway Department

Austin, Texas December 1952

FOREWORD

This pamphlet is Book III of a five book manual on Plan Preparation that has been prepared jointly by the Road Design, Bridge, and Land Service Roads Divisions of the Texas Highway Department. The primary purpose of the manual is to present material covering plan preparation to selected District Representatives who in turn will become instructors for the entire District Personnel. Book III and Book IV of the Manual, which have been prepared by the Bridge Division, contain not only material always applicable to structure planning and plans, but also several of the subjects presented have been selected because of their frequent reoccurrence in planning improvements to the highway system in its present state of development. A careful study of the entire manual is particularly necessary in order that the Department may uphold established standards in plan preparation and continue to advance in that important phase of highway improvement.

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DEPARTMENTAL USE ONLY

LOCATION OF BRIDGES

1. INTRODUCTORY

Under this subject we will discuss some of the general aspects of site selection for highway bridges. The location of grade separation structures will be covered in a later part of this instruction course.

The selection of the proper site for a bridge with proper fit to topography, is a complex operation. It is an art in which long experience, a high order of engineering skill, and exercise of common sense and judgment must be blended. Problems and their solutions in different situations will appear to be totally dissimilar, and wide differences of opinion between various engineers and interested agencies will exist and must be compromised. Social and economic as well as technical considerations will affect the solutions. The problems do not readily yield to specific analysis but it is possible to discern broad fundamental principles upon which to base critical discussion.

It must be always kept in mind that a modern highway bridge is a very costly facility and of very long life. The greatest of care taken in its location will always be justified. From the purely selfish point of view, the engineers responsible for the new facility will want a monument to their sagacity and foresight and not just another structure that will become inadequate and inappropriate in a few years.

In the short space of time allotted, it is not possible to undertake a detailed treatment of the subject. The best we can do is to point out some of the more general considerations and principles of site selection for bridges, and to present some examples for discussion, questions and answers. From this we hope that you will be able to derive some benefit and guidance for further investigation and study.

2. THE RESIDENT ENGINEER AND THE BRIDGE DIVISION

The Resident Engineer operates under the guidance and authority of the District Engineer and in most jurisdictions will have the primary responsibility for the selection of bridge sites, as a part of his activity in highway location, and will make the first proposed solutions. This is logical, since the Resident Engineer will have the best knowledge of local conditions, and resources for local field investigation and surveys. As the interest of other agencies will be involved, these solutions will be thoroughly reviewed by the District Engineer, the Bridge Division, Bureau of Public Roads, and municipal, county, and governmental agencies affected.

It is the particular charge of Bridge Division field representatives to cooperate with the Resident Engineer in reviewing his proposed solutions, in suggesting alternative studies, and in furnishing maps, recorded hydraulic data and similar information that are not readily available to the Resident Engineer. These representatives will provide guidance for negotiations with the Bureau of Public Roads, the railroad companies, the U. S. Corps of Engineers, and other interested State and Governmental agencies. These negotiations are the particular responsibility of the Bridge Division but the District and Resident Engineers are given opportunity to participate and to express their opinions and recommendations freely at any time.

The local engineers generally are urged to request the services of the Bridge Division early in the development of a project and then to cooperate by securing all field information that may be requested, in order to obtain the very best solution to the given problem at the time that it is needed.

3. LOCATION AND PROGRAMMING

A particularly important aspect of the location problem is its relation to the programming procedure for construction. Location studies and submissions of data should be made and decisions reached well in advance of the programming stage so that realistic estimates can be made for finance consideration.

A second advantage of early location of major structures is in the opportunity for leisurely and thorough design studies to determine the most suitable and economical structure arrangements. This will in turn enable the making of still more accurate estimates for financing. When location and layout studies have not been begun until after the project has been programmed, the urgent demand for construction and completion may cause these most important operations to be hurried or slighted, resulting in defective solutions and excessive costs. No highway locations should be fixed or right-of-way bought, until the sites of major stream crossings have been selected and approved by all interested agencies.

4. BRIDGE LOCATION IN GENERAL

Site selection for bridges, as for any facility, is the first step in the physical development of the structures, before they can be designed, constructed, and placed in the service for which they are intended. Site selection will be based upon considerations of service, safety, fit to local conditions such as topography and culture, economy and appearance. Its importance cannot be overemphasized. Improper site selection may in many cases have more effect upon the usefulness and probable cost of the facility than divergences from the best practice in the later phases of design and construction.

Highway bridges are provided for the purpose of carrying highway traffic, which is the reason for the highway itself, over water courses. They must have a relation to the connecting highway with respect to grade, width, surface, and hazard protection, which is consistent with the traffic service to be rendered by the highway.

Culverts and small bridges individually will be subordinated to highway routing, location, and design restrictions and hence are not considered further here. Of course, in a group they may affect the highway alignment such as in the case of a ridge location. On the other hand, major bridge structures will generally influence the highway location.

5. LOCATION OF MAJOR BRIDGES

The location of major bridges is dependent upon topographic, hydraulic, navigation, lateral road, railway, foundation, structural and economic requirements. Present day traffic demands and advances in the technique of structure design have reduced some of the early economic limitations of structure location. In many cases it may be possible to accept the points of origin and destination of traffic as the primary controls. There are, however, many major streams that will not admit of an economical crossing at any random point. These crossing sites will still remain control points in the highway location.

Satisfactory locations for major bridges will have good foundation material, approach alignment on tangent or easy curvature, permanent narrow straight reaches of channel, right-angled crossings, and grade lines that will permit a minimum length of structure and height of approach fill. Elements of alignment and grade-line will of course comply with the road design requirements for the given class of road.

The grade line and length of bridges will be further controlled by hydraulic requirements for highwater and run-off, as discussed in another paper. A square crossing is also desirable from the hydraulic design point of view, for if the crossing is skewed any appreciable amount, the water surface will slope along the embankment, making the correct distribution of floodwaters across the overflow area difficult to estimate and provide for. Further, the height and length of the main span over navigable streams will be determined by negotiation with the U. S. Corps of Engineers, by acceptance of a suitable application and issue of a permit, based on results of public hearings.

While structures can be built on skew and on horizontal curves, this practice should be avoided as much as possible, due to construction difficulties, increase in cost, and the extended time required to make the special plans that are necessary. In rural work curvature is only very rarely admissable and skews should be used only on small trestles, where adjustment of the alignment or channel changes cannot be made to eliminate the need for such skews.

Increasingly as time goes on, the opportunity for making bridge locations on new routes will become less and less, and the problem will be more and more one of relocation and reconstruction of existing bridges. If the connecting roadway has been correctly located and highly developed and there are important controls, it may be necessary to reconstruct the existing bridge on the present line or very close to it. The principles of location are then not directly involved. However, all possible alternative locations should be investigated, comparative estimates made upon rough preliminary designs, and all data in regard to traffic and land service noted.

In case the existing facility is to be rebuilt, the cost of widening or replacement of the structure involving detours and the cost of revamping approaches if an offset structure is used, must all be taken into account. If a relocation is considered the value of the existing structure as an addition to the secondary system must be included.

6. RECONNAISSANCE, SURVEYS, AERIAL PHOTOGRAPHY AND MAPS

We cannot at this time go into the details of reconnaissance, surveys, preparation of site layouts, reports and so on, but it must be emphasized that good location will be based upon thorough field investigation and accurate and complete representation of all factual data upon the site layout, plan-profile and other exhibits. Contour topography should be secured as may be necessary to supplement available maps. Subsoil investigation and representation will be discussed in the following paper.

During the stage of general reconnaissance the stream to be crossed should be viewed on the ground within the limits of possible crossings, and photographs and notes of desirable crossing sites and existing structures should be secured to supplement map studies, and for reference when the later more detailed surveys are made. At this time also, all available maps and aerial photographs should be collected and examined. In important work, actual aerial reconnaissance may have value.

The use of aerial strip maps and mosaics is well understood. They will be particularly useful where the relief is bold, the terrain is open, or culture is well developed. In cases of heavily wooded flat terrain, such as the lower Sabine valley, their value may be limited. The maps of the Federal Production and Marketing Administration (the old AAA) may often be consulted in their local offices in the principal cities and county seats. When the cost would be justified by the magnitude and complexity of the project, the maps may be purchased or special flights made. These flights will be arranged for through the Road Design Division. Sometimes oblique single pictures made on a chartered flight will be helpful. Such pictures made at times of extraordinary floods have been valuable in later studies for new crossings.

In a previous paper by the Road Design Division the use of Highway Planning Survey data in road design was discussed. Many of these data will be applicable to the problems of bridge location, since generally the road connections will have considerable influence upon the solution of the problem. In particular the aerial map files of the Highway Planning Survey can be consulted. Available contact prints will be loaned to the field or purchased for the account of the District. When the inquiry for such material for bridge investigation is made directly to Road Design or Planning Survey, the Bridge Division should receive a copy of the inquiry, so that all preliminary investigation may be properly coordinated.

For many streams there are now available in the Bridge Division files excellent topographic maps prepared by the U. S. Geological Survey, the State Reclamation Department, and the U. S. Corps of Engineers. Outstanding examples of the available material are the 1915 and 1939 U. S. Engineer Department Surveys of the Trinity River. In general, it is preferable to secure copies of these maps through the Bridge Division, in order to avoid duplication of effort and to enable the Bridge Division to extend the scope of its map files as opportunity will permit. If maps are obtained locally in the field, opportunity should be extended to the Bridge Division to make or secure copies for its files.

In the course of usual field inspections and visits by Bridge Division representatives, such material as described may be suggested. However, in the early stages of any site selection problem, the Resident Engineer should request from the Bridge Division any maps or other information that it may have available, that may be of assistance in the solution of the problem.

7. PLATE I

Plate I illustrates a bridge location problem in the flat coastal country near a sizeable community. The drawing was prepared from aerial photographs. Location "C" was the original bridge crossing, built in 1926. Location "B" will replace this crossing some time in 1953. Location "D" was considered as an alternate to Location "C", and Location "A" as an alternate to Location "B". Assume that one-half of the traffic of 6,000 v. p. d. will wish to by-pass the community. The old crossing was a through truss swing span with timber spans in the approaches and relief structures. Pavement on the high ground on the east was concrete in poor condition. Foundation conditions in all cases are similar. The advantages and disadvantages of each location shown may be listed.

The crossing on Location "D" is square and of minimum length. Distribution of high water flow is definitely indicated, permitting accurate determination of design highwater and bridge lengths. Since the west connection must pass through or near the community, an excessive amount of road construction and travel mileage is involved.

The crossing on Location "C" is longer than that on Location "D" and is skewed at nearly 45 degrees to the direction of high water flow. The over-all distance is shorter and the city is directly served. The construction of the Naval berthing area and great growth of city traffic seriously impede the free flow of through and suburban traffic.

The crossing on Location "A" is about the same length as on Location "C", but is much more nearly at right angles to the direction of flood flows. All traffic would be relieved of interference from openings of the draw span, which would partially offset the slightly increased distance for traffic from the east to the downtown area. If the highway to the east were relocated on a more direct line, this advantage would not exist. Through traffic would be benefited in every respect.

The crossing on Location "B" is apparently longer than in the other cases, and at a skew of 45 degrees to flood waters. Since the velocities at high stages are not great, deflecting levees made from dredged over-burden can be placed at the lower end of each bridge to equalize distribution of flow. The crossing is superior to the others in regard to length of road construction and travel mileage, if the highway to the east is not relocated.

Economic studies of construction and vehicle operating cost were made for lines A and B, with the advantage for line B. As it was determined that the highway to the east would not be relocated, Location "B" was selected as the best balance of all tangible and intangible factors.

8. PLATES II AND III

Plates II and III illustrate a bridge location problem in the rolling country of East Texas. Plate II is a section of the Highway Planning Survey maps for Houston and Leon Counties. Plate III is a section of the U. S. Engineer Department Trinity River Survey 1939. Aerial photographic contact prints are also being used in making this study. Location A is the present bridge crossing. The bridge is a series of light low one way trusses with timber floor supported on an abandoned lock and dam in the river. Location "B" is a variant to improve the curvature at the east end of the bridge. The approach roadways cross the overflow areas on each side of the channel at grades very slightly above the adjacent ground. Construction on high ground on each side is well graded with intermediate type base and surface. Locations "C" and "D" are alternate proposals. Better potential crossings exist within ten miles upstream and down stream from the present crossing, but they lie outside of the area determined by the existing located highway. The advantages and disadvantages of each location shown may be listed.

Line A crosses the main channel and the west overflow area at right angles but is deflected in the east overflow downstream two miles to reach high ground south of Hurricane Bayou. Overflow waters in the east bottom will be deflected along an embankment on this line necessitating a large relief structure at the southeast end. Distribution of flood waters will be uncertain, necessitating larger structures than might otherwise be necessary. The curvature at the east end of the bridge is objectionable. A detour for traffic would be necessary if the present dam structure is to be used.

Line B lies slightly east of Line A and improves the curvature at the bridge end. It is otherwise similar to Line A.

Line C crosses the overflow area at right angles. The crossing is considerably shorter than on Lines A and B. It, however, would necessitate the abandonment and reconstruction of five miles of permanent highway.

The crossing on Line D is intermediate in length, crosses the channel and overflow area at right angles, and does not require the relocation of any permanent highway.

In all cases the proposed waterway would remain in the existing channel. No field studies or comparative estimates of cost have been made as yet for the various alternates.

9. PLATES IV AND V

Plates IV and V illustrate a bridge location problem in the rolling country of East Central Texas. Plate IV is a section of the Highway Survey maps for Brazos, Grimes, and Leon Counties. Plate V was prepared from the U. S. Geological Survey - Texas Reclamation Department Navasota Quadrangle. Location "A" is the original bridge crossing, one of the oldest on the State Highway System, the main bridge having been constructed after 1870. Various substructure and superstructure units have been replaced from time to time. Location "D" is the crossing now being completed. The west approach grading and Coles Creek Bridge, the Navasota River Bridge, and the piers for the main bridge have been completed. The superstructure for the main bridge is under contract. The advantages and disadvantages of each location shown may be listed.

The main channel crossing on Location "A" is square, but the road connections are unsatisfactory. In the west approach the present Jordan Creek crossing is below extreme high water. In the east approach the delta between the Brazos and Navasota Rivers is traversed for a distance of two miles upstream. The existing pavement is 16 foot and 18 foot concrete pavement from Brenham to Washington, 18 foot concrete pavement west to the junction of Location "A" and "D" east of the Navasota River, with flexible base between.

The proposed Location "B" would preserve the concrete pavement sections but has similar disadvantages as to conditions in the overflow area as Location "A". The main channel crossing would be about the same. Relief bridges below the eastward bend of the Navasota River might induce an eventual break-through of the river. An eastward extension of this location to State Highway 105 in Navasota would entail a large embankment cost.

The proposed Location "C" would preserve some of the advantages of the existing main river crossing and provides a definite opening for the Navasota River. Construction of five miles of new highway in the west approach would be necessary. The same disadvantages for embankments in the overflow area exist as for Locations "A" and "B".

Location "D" provides a square crossing of minimum length, but entails construction of six miles of new highway in the west approach. A satisfactory connection with Washington can be made across the peninsula between Coles and Jordan Creeks.

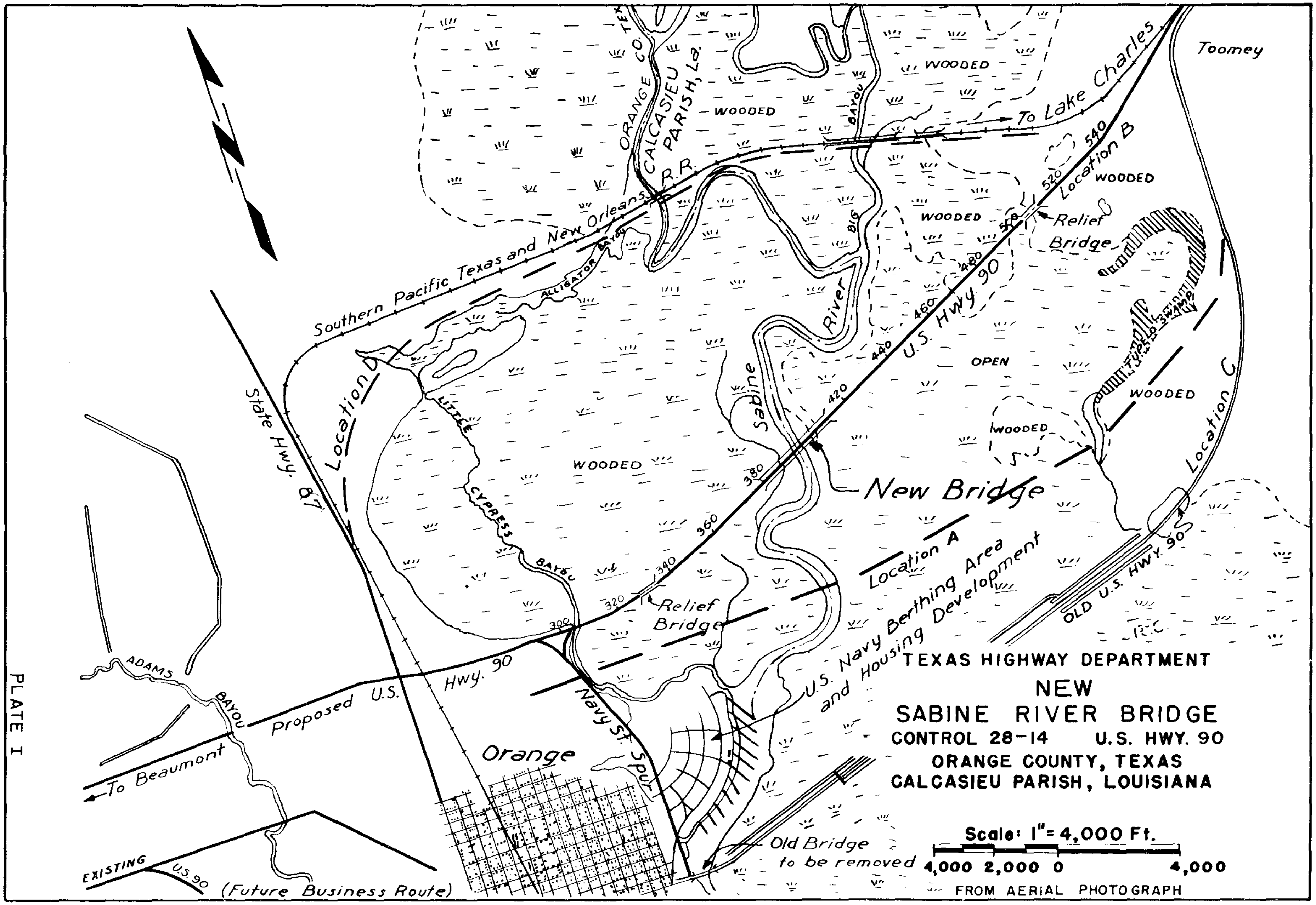
Field surveys and economic studies were made for Locations "B" and "D". The cost of rehabilitating State Highway 90 west of Washington was taken into account. Washington was considered not to be of sufficient importance to be a primary control in the main line, but would remain on a loop along the old road which would not require further improvement. In consideration of comparative costs, reduced crossing length and improved alignment, Location "D" was selected and is being constructed.

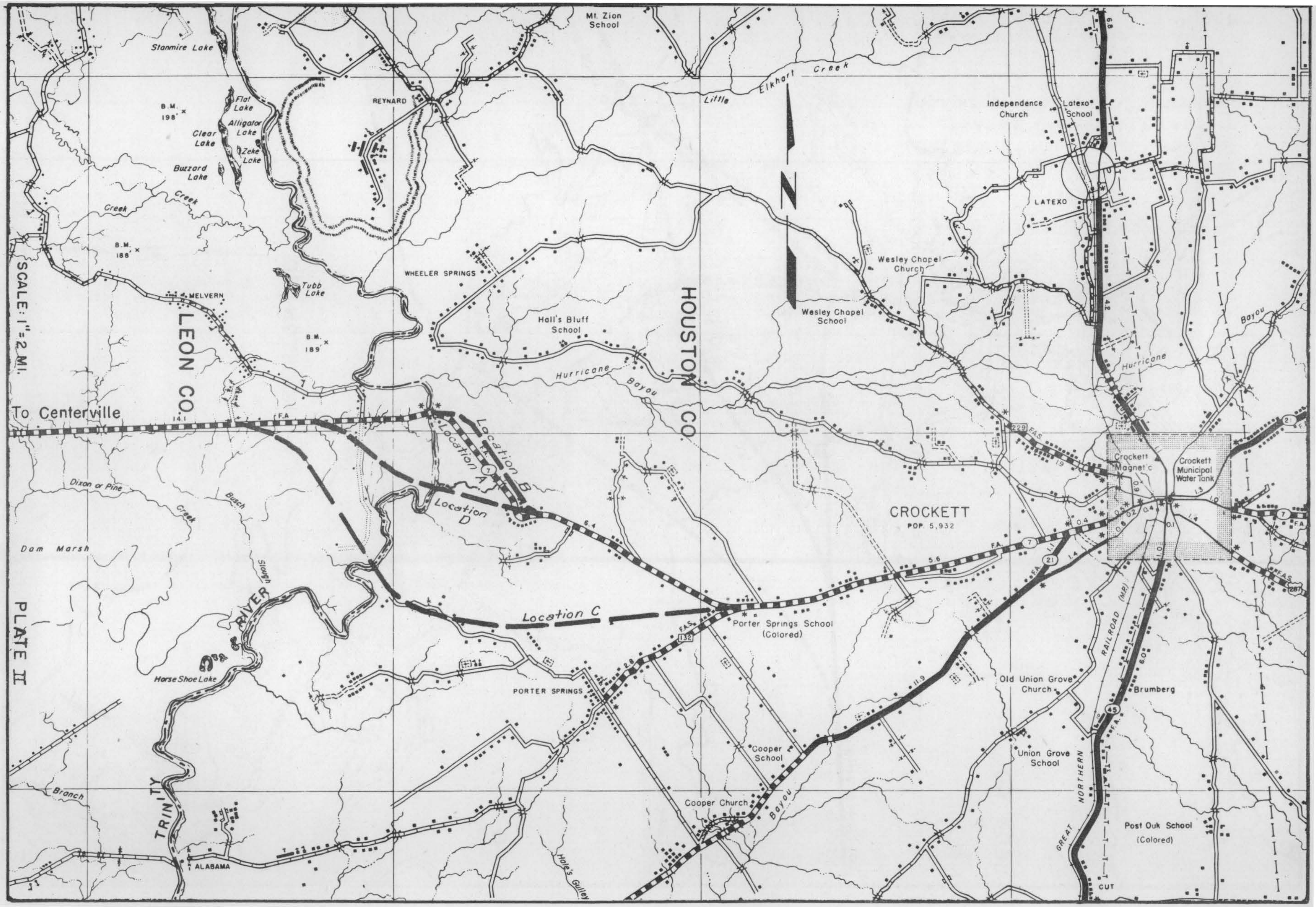
10. CONCLUSION

The limits of location for a major bridge will be defined by the general routing requirements. All possible crossings within these limits should be investigated and comparative estimates made for each possibility, including roadway items within the limits of the alternative locations. The advantages and disadvantages of each alternate should be listed in detail, and an over-all recommendation made for the information of the reviewing authorities.

The collaboration of the Austin office Design Divisions should be enlisted at an early stage, and all available documentary information obtained from them. Preliminary submissions of location data should be made from time to time as may appear to be desirable in order to expedite the review and approval of the location finally selected.

PLATE I


TEXAS HIGHWAY DEPARTMENT
NEW
SABINE RIVER BRIDGE
CONTROL 28-14 U.S. HWY. 90
ORANGE COUNTY, TEXAS
CALCASIEU PARISH, LOUISIANA
Scale: 1" = 4,000 Ft.
4,000 2,000 0 4,000
FROM AERIAL PHOTOGRAPH
Toomey
To Lake Charles
Southern Pacific Texas and New Orleans R.R.
ORANGE CO. TEX.
CALCASIEU PARISH, La.
WOODED
BAYOU
BIG
Sabine River
ALLIGATOR BAYOU
LITTLE CYPRESS BAYOU
Location D
Location B
Location A
Location C
Relief Bridge
New Bridge
U.S. Hwy. 90
OPEN
TUPELO SWAMP
OLD U.S. HWY. 90
State Hwy. 87
Proposed U.S. Hwy. 90
To Beaumont
ADAMS BAYOU
Orange
Navy St. Spur
U.S. Navy Berthing Area and Housing Development
Old Bridge to be removed
EXISTING U.S. 90 (Future Business Route)
300
320
340
360
380
420
440
460
480
500
520
540



SCALE: 1" = 2 MI.
PLATE II
To Centerville
LEON CO
HOUSTON CO
CROCKETT
POP. 5,932
Crockett Magnetic
Crockett Municipal Water Tank
Location A
Location B
Location C
Location D
Mt. Zion School
Stanmire Lake
Flat Lake
Alligator Lake
Clear Lake
Zeke Lake
Buzzard Lake
Creek
Tubb Lake
B.M. 198'
B.M. 188'
B.M. 189'
MELVERN
REYNARD
WHEELER SPRINGS
Hall's Bluff School
Hurricane
Bayou
Little Elkhart Creek
Independence Church
Latexo School
LATEXO
Wesley Chapel Church
Wesley Chapel School
Dixon or Pine
Busch Creek
Dam Marsh
Slough
TRINITY RIVER
Horse Shoe Lake
Branch
ALABAMA
PORTER SPRINGS
Porter Springs School (Colored)
Cooper School
Cooper Church
Hale's Gulley
Old Union Grove Church
Union Grove School
Brumberg
Post Oak School (Colored)
GREAT NORTHERN RAILROAD (MP)
CUT
F.A.
F.A.S.

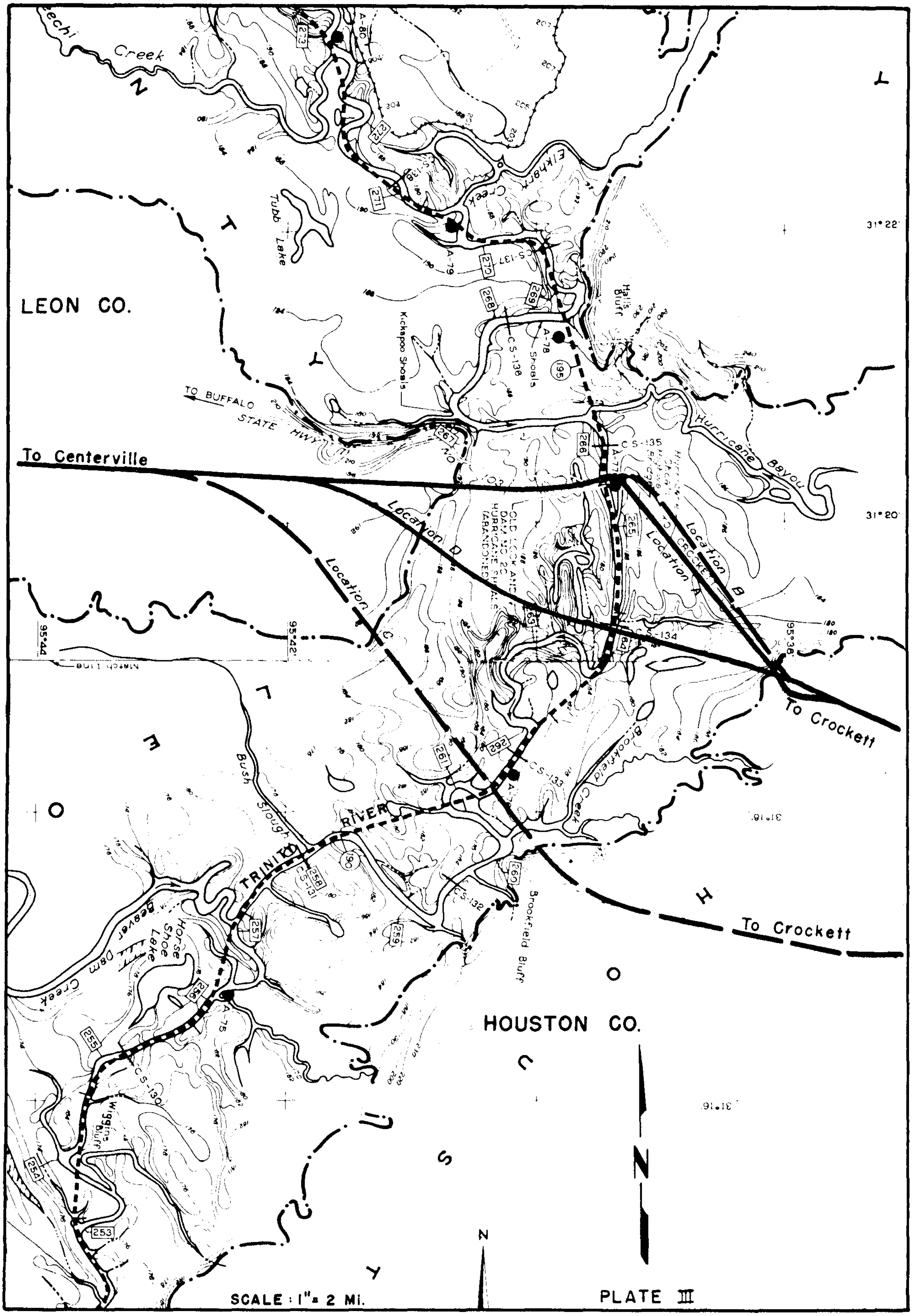
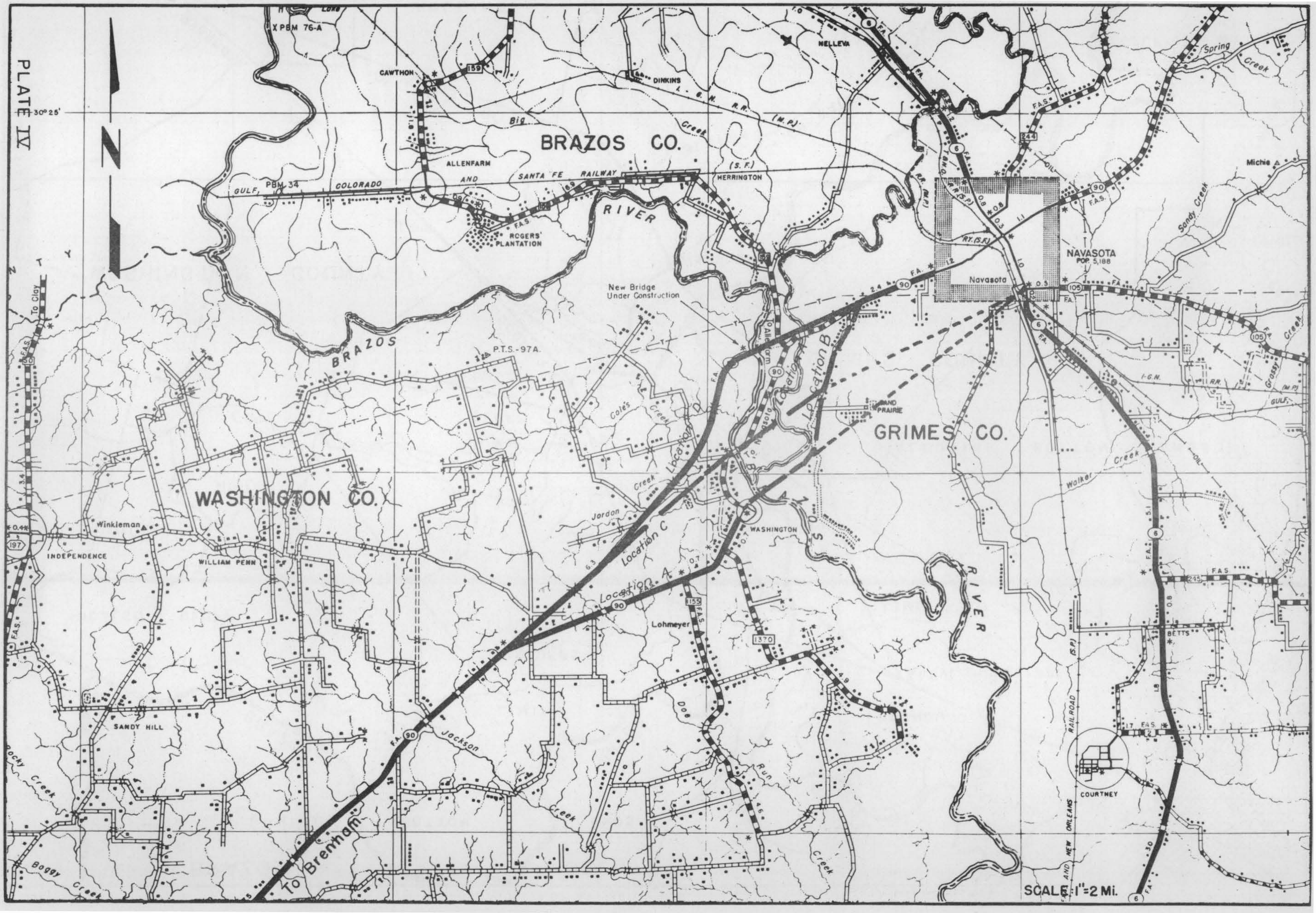

LEON CO.
HOUSTON CO.
To Centerville
TO BUFFALO
STATE HWY
To Crockett
To Crockett
Location D
Location C
Location A
Location B
TRINITY RIVER
Elkhart Creek
Tubb Lake
Kickapoo Shoals
Shoals
Halls Bluff
Hurricane Bayou
Brookfield Creek
Brookfield Bluff
Bush Slough
Horse Shoe Lake
Beaver Dam Creek
Wiggins Bluff
OLD LOCK AND DAM NO. 2 C HURRICANE SHOALS (ABANDONED)
Match Line
31°22'
31°20'
31°18'
31°16'
95°44'
95°42'
95°38'
N
SCALE : 1" = 2 Mi.
PLATE III

PLATE IV


BRAZOS CO.
WASHINGTON CO.
GRIMES CO.
NAVASOTA
POP. 5,188
Navasota
CAWTHON
DINKINS
NELLEVA
ALLENFARM
HERRINGTON
ROGERS' PLANTATION
GULF, COLORADO AND SANTA FE RAILWAY
BRAZOS RIVER
Big Creek
New Bridge Under Construction
P.T.S.-97A
SAND PRAIRIE
WASHINGTON
Winkleman
INDEPENDENCE
WILLIAM PENN
Lohmeyer
SANDY HILL
BETTS
COURTNEY
Location A
Location B
Location C
Location D
Coles Creek
Jordon Creek
Walker Creek
Sandy Creek
Spring Creek
Grassy Creek
Doe Run
Jackson Creek
Rocky Creek
Boggy Creek
Michie
To Clay
To Brenham
To Navasota
RAILROAD AND NEW ORLEANS (S.P.)
SCALE 1"=2 Mi.
30°25'

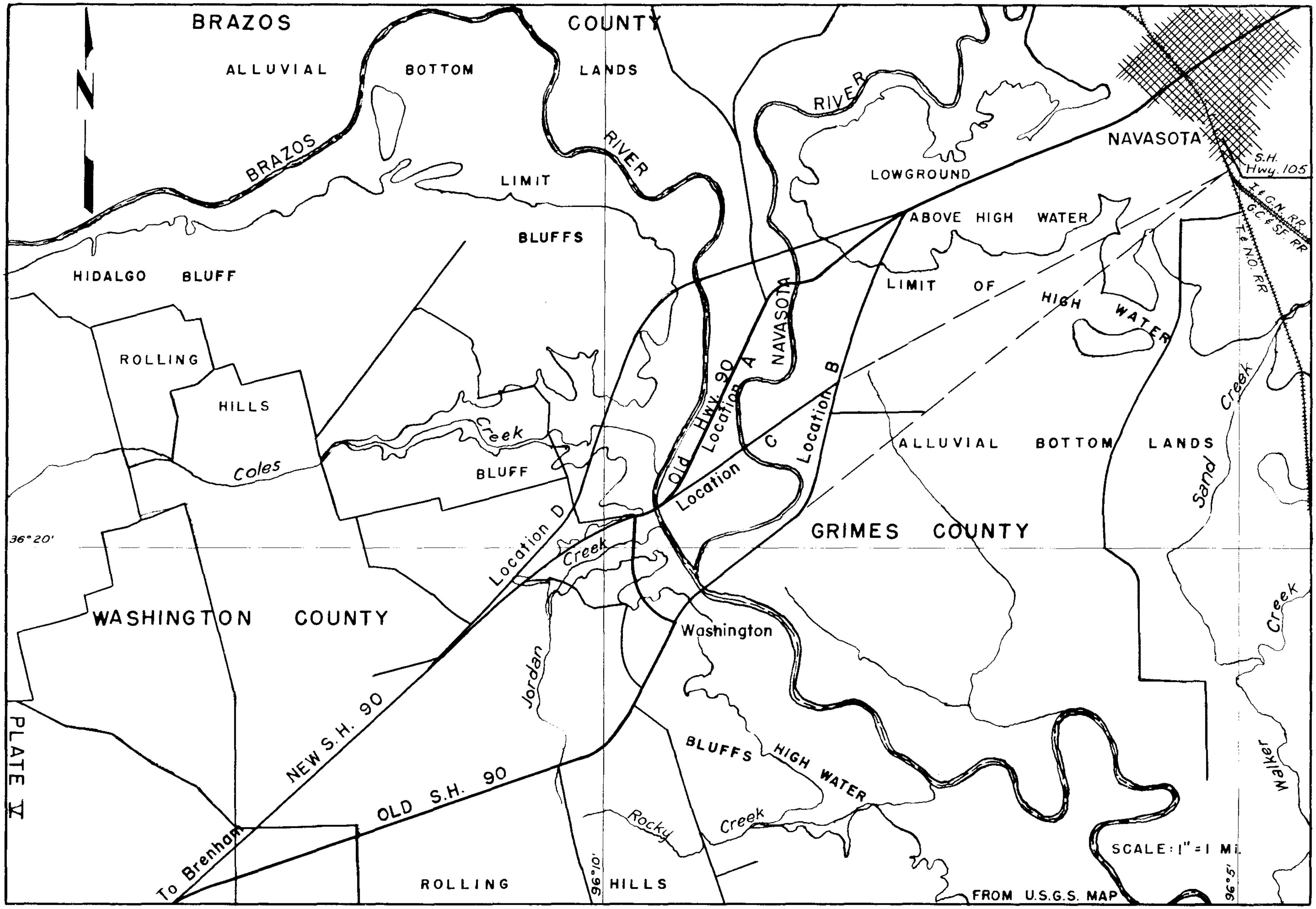

BRAZOS COUNTY
ALLUVIAL BOTTOM LANDS
BRAZOS RIVER
RIVER
LIMIT
BLUFFS
HIDALGO BLUFF
ROLLING HILLS
Coles
Creek
BLUFF
Location D
Creek
WASHINGTON COUNTY
Jordan
NEW S.H. 90
OLD S.H. 90
To Brenham
ROLLING HILLS
96°10'
Rocky
Creek
BLUFFS HIGH WATER
Washington
Old Hwy. 90
Location A
NAVASOTA
Location B
Location C
Location
GRIMES COUNTY
RIVER
LOWGROUND
ABOVE HIGH WATER
LIMIT OF HIGH WATER
NAVASOTA
S.H. Hwy. 105
I.&G.N. RR.
G.C.&S.F. RR.
T.&N.O. RR.
ALLUVIAL BOTTOM LANDS
Sand Creek
Creek
Walker Creek
36°20'
96°5'
SCALE: 1" = 1 Mi.
FROM U.S.G.S. MAP

PLATE V

FOUNDATION EXPLORATION
FOR STRUCTURES WITH INTERPRETATION

GENERAL. As a general rule, the type, exact span lengths, cost, and to some extent, the appearance of a highway structure, are determined by a single factor: the natural foundation material available. The care used in foundation exploration should be commensurate, therefore, with the value of the information to be obtained. Foundation data should be sufficiently complete and accurate to provide the designer with a dependable basis for making a choice of structure type and an economic comparison of layouts, and to permit planning on which construction may proceed with confidence of encountering a minimum of delays.

Many explorations have been made where a large number of test holes were drilled, but the overall factual information about the existing natural formations was very meager. The true evaluation of a foundation exploration should be made on the basis of the cost per foot of reliable information and not on the cost per foot of hole.

The science of Soil Mechanics and Foundation Engineering has made great advancement in the past decade. The ability of engineers to explore, sample, test and evaluate most earth formations in relation to substructure design has increased rapidly in the past few years. The natural result of this development has been the introduction of more economical substructure designs that better fit the existing conditions. It has made possible the wider use of the "Drilled Shaft" type of substructure both with and without "Underreaming."

Research and investigational work has shown that the shear strength of a soil is a reasonable measure of the load carrying capacity of a friction pile driven in that soil. This fact has made it possible to evaluate the accuracy of the generally accepted dynamic methods of determining pile capacities. The results of these evaluations indicate that we need a more realistic method of measuring pile capacities and on large projects it generally will be found economically feasible to make pile load tests to establish the true pile capacity or to make complete soil strength tests as a basis for determining pile capacity. The actual testing of samples of the formations has resulted in more realistic allowable unit design loads being used when founding on shales, various rock formations and clays.

In summary it might be stated that the need for adequate explorations can be justified solely from an economic standpoint. Each project should be studied individually and the extent of the investigation based upon the magnitude of the project and the nature of the existing earth formation as related to the economics of the possible substructure designs.

OBJECTIVE:

The objective in structure foundation exploration is to determine, within the limits of the proposed structure, the elevation at which various earth strata exist, which information, together with the character, strength and description of the formations, will materially affect decisions on design.

Putting it plainly - the objective is to find out what is existing in order that the designing engineer can make a complete study and determine the most economical design. Simple as this sounds, it is amazing how often exploration work fails to accomplish this objective.

METHOD:

It is quite impossible to set forth a methodical rule to be followed making foundation explorations due to the widely divergent job conditions encountered in the various parts of Texas. There are many instances where good foundation material is encountered at shallow depths and adequate investigations can be made by digging open pits. Then, those border line cases will be encountered where a good clay is available near the surface and rock also is available within easy reach. In these cases the engineer in charge of the exploration must be careful not to lose sight of the objective by making decisions for the designer and fail to furnish complete data upon which to make an impartial study of all possible design types. The size of the proposed structure will, of course, be a dominant factor of influence in deciding the method as well as the extent of the exploration.

Aside from the very shallow exploration work where the open pit method is adaptable, the majority of Highway Department exploration work is done with one of the four rotary core drill rigs which operate out of the Camp Hubbard Shops. This equipment is routed by the Bridge Division, and when operating in the field works under the District Engineer or his duly authorized representative. The equipment is constantly being improved as new problems are encountered. Constructive criticism of any part of the operation is always welcome and should be directed to the Bridge Engineer, File D-5, Austin.

Rotary core drill rigs operating out of Austin are mounted on 5 ton trucks with tandem rear axles. (Fig. 1) The rigs are powered by the truck engine through a ''Power-Take-off'' mechanism which utilizes the truck transmission and gives a wide range of power and speed at the drill head. Other features of the rigs include a reciprocating type of power mud pump, hydraulically powered pull down or ''crowd'', hydraulically retracting drill head and many other minor items that assist in obtaining core samples under very difficult conditions.

Exploration methods now in use on these rotary core drills can be divided into five main groups, namely; Wash Boring or Fish Tail Drilling, Dry Barrel or Single Wall Barrel Core Sampling, Wet Barrel or Double Wall Barrel Core Sampling, Push Barrel Sampling, and Cohesionless Sand Sampling.

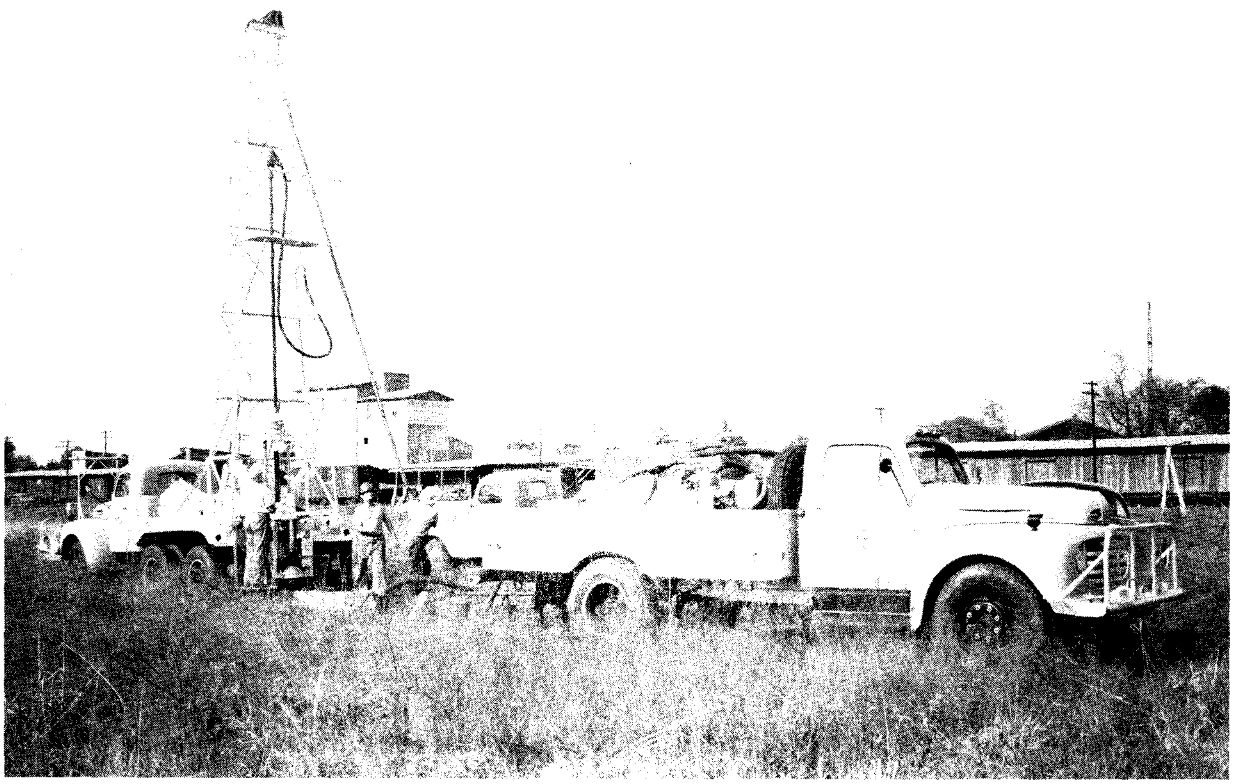


FIG. 1. ROTARY CORE DRILL RIG AND SUPPLY TRUCK - IN OPERATION

Wash boring or Fish Tail drilling should not be permitted until the classification of the strata have been definitely established and it is desired to drill a hole rapidly to establish the elevation at which a hard stratum exists below. The core drillers have been instructed not to use the wash boring method except when specifically directed by the district representative. Attempts to classify by watching the wash water leads to very erroneous conclusions and is to be avoided at all times. The wash boring method is a rapid way to make a hole through most all formations except rocks and hard shales but when you are through, a hole in the ground is about all you have to show for your work.

Dry Barrel or Single Wall sampling is the method most generally used (Fig.2-A). The core sample obtained is generally in a disturbed condition due to the pressure applied to cut the core and pack it in the barrel so that it can be recovered. However, the core can be extracted from the barrel either by water pressure or by hydraulically powered piston extractor and a visual classification made. When used for sampling in practically all materials encountered except very soft mucks and cohesionless sands, the dry barrel sampler will give a sample containing all components in the original formation and the amount of disturbance will depend upon the softness of the formation. Although this method is called the dry barrel method, it should be pointed out that some cooling water is often used with this method and in the hard formations a small amount of water is circulated during the cutting of the core.

FIG. 2A DRY BARREL SAMPLER

Wet Barrel or Double Wall Barrel sampling is used in a wide range of formations when undisturbed core samples are desired. (Figs. 2-B, 2-C) The sampler used consists of an inner and outer barrel. The outer barrel is a thick wall tube with saw tooth cutter. The inner barrel is a thin wall tube connected to the head of the sampler on a free running bearing. The outer barrel is rotated and cuts an annular ring around the core as the sample is received into the inner barrel. The inner barrel remains stationary due to friction between the core sample and the barrel wall. Water is circulated down the drill stem, thence between the inner and outer barrel picking up the cuttings from the annular ring, carrying them up around the outside of the outer barrel to the ground surface where they are deposited in a sump. A viscous mud slurry can be added to the circulating water to lift cuttings consisting of sands and gravels. There are several versions of the double wall barrel samplers. For formations other than rock we use a type that has a thin sheet metal liner that fits the inner core barrel and furnishes a handy method of removing the core as well as a protection to the undisturbed core while transporting same to the laboratory. For rock and hard shales the liner is omitted, as this type material has ample strength for handling without the protection of the liner. The relative projection of the inner and outer core barrel cutting bits can be varied by adding or subtracting collar washers. For rock and hard cutting materials it is necessary that the outer barrel cutter lead the inner cutter as the hard material cannot be penetrated by the knife edge cutter on the inner barrel. However, when taking a core in clays, sand clays, etc., the inner cutter is adjusted to lead the outer cutter and thereby protect the core from erosion by the circulating water. When the proper length core has been cut and received in the inner core barrel the circulating pump is shut off and the outer barrel rotated at a relatively high speed. This generates enough heat to cause the lower end of the core to expand and bind itself in the barrel while the sampler is withdrawn from the hole. This particular operation calls for a driller with skill, experience, and patience.

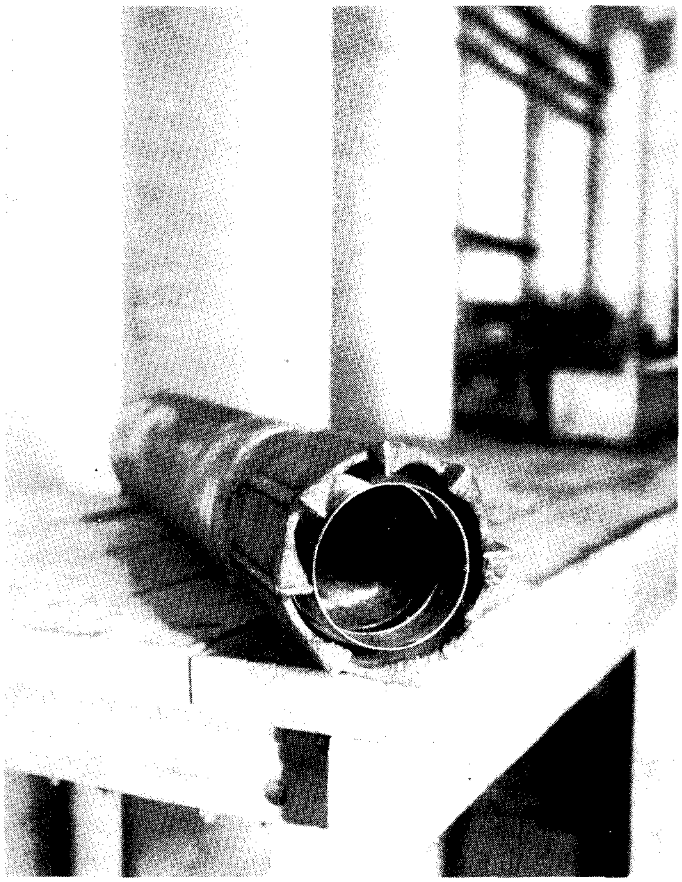


FIG. 2-B WET BARREL SAMPLER

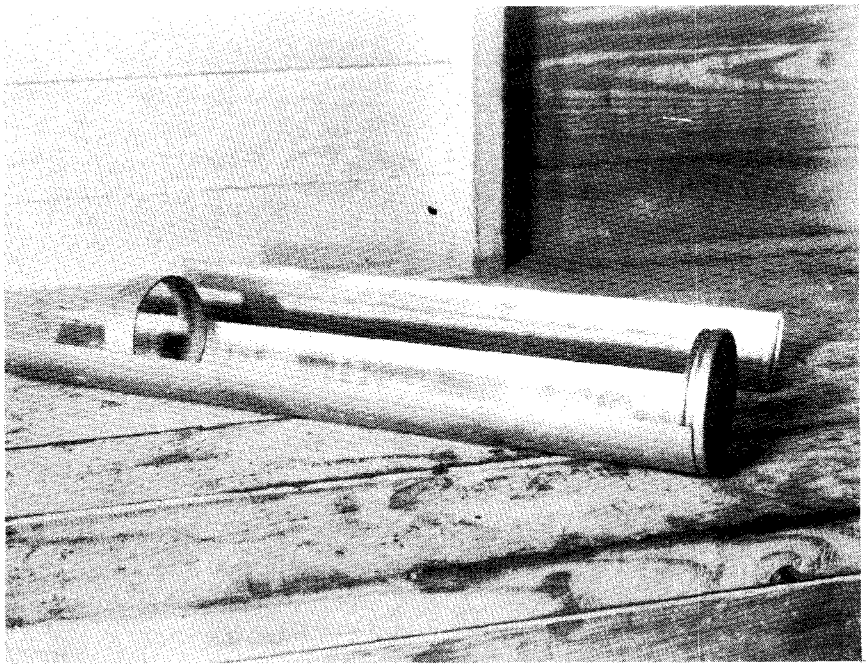


FIG. 2-C SPLIT TYPE LINER

The Push Barrel Sampler (Fig. 2-D) as the name implies, employs the simple principle of pushing a thin walled tube with a sharp cutting edge into the formation with the hydraulic push down on the drill rig. This type sampler recovers very good "Undisturbed" samples where it is adaptable but its usefulness is limited to materials into which it can be forced and which have sufficient cohesion to remain in the barrel while the sampler is withdrawn from the hole. The usual procedure is to force the sampler into the formation with a slow steady push and rotate it about two turns to break off the core before beginning the withdrawal. The push barrel sampler is faster than the double barrel sampler and is to be preferred where it is adaptable.

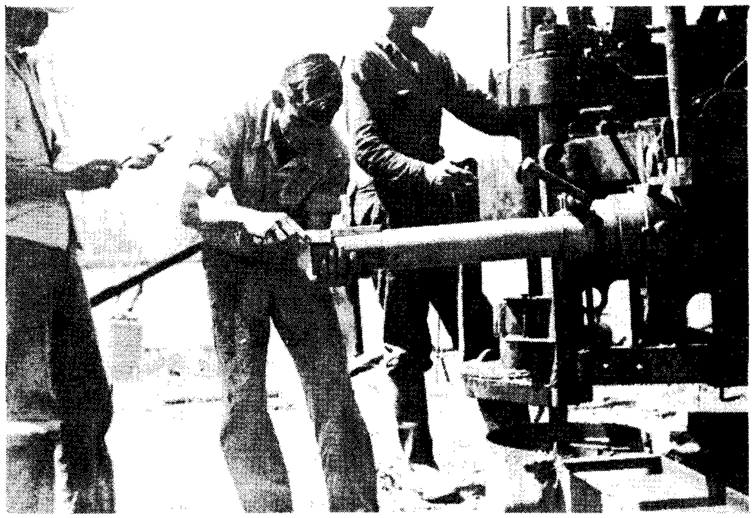


FIG. 2-D PUSH BARREL SAMPLER SHOWING CORE BEING EXTRACTED

The last and least used sampling tool is the cohesionless sand sampler. (Fig. 2-E) It is to some extent a combination of the last two named samplers. It consists of an outer barrel or air bell and inner barrel or sample tube. The use of this sampler is limited to very large projects where loose cohesionless sand exists and it is important that the density and nature of the sand be determined. Due to the limited use of this tool, it is not considered desirable to spend the time describing its use. Complete description and details of the sand sampler can be obtained on request to the Bridge Division, File D-5, Austin.

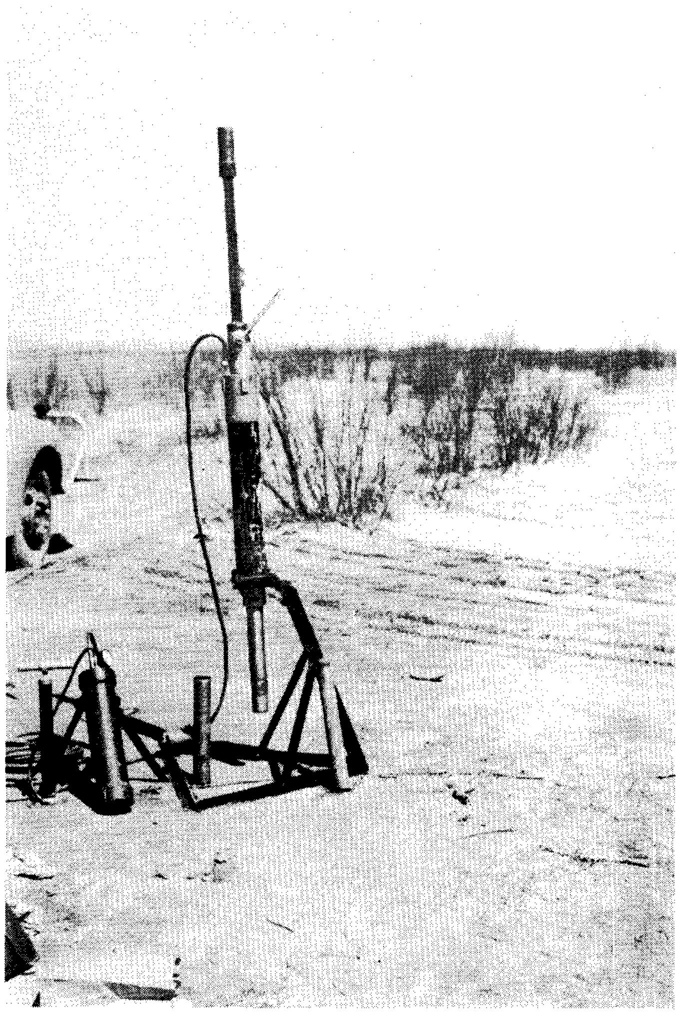


FIG. 2-E. COHESIONLESS SAND SAMPLER

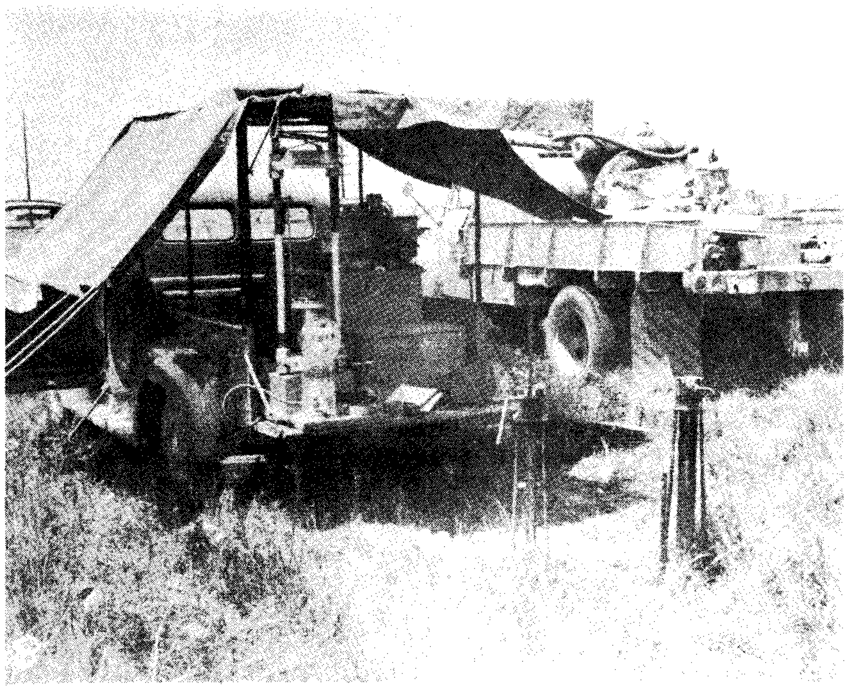


FIG. 3. FIELD LABORATORY FOR TRIAXIAL TESTING

In addition to the above mentioned drilling tools each of the rotary core drills operating out of Austin are equipped to. make the standard Penetrometer Test. (Figs. 4 & 5) This test consists of recording the number of blows of a 170 pound hammer dropping 24 inches that is required to force a 3 inch diameter steel cone 12 inches into a formation. In cases where hard formations are encountered, including rock, the instructions are to hit the pin 100 blows and accurately record the resulting penetration. This test has been in use the past 3 years and it is now standard procedure, in all our exploration work, to make the test at least each 10 feet of hole and more often if necessary in order that each significant formation is tested. Experience to date with the Penetrometer Test indicates that the number of blows of the hammer for the first 6 inches and the second 6 inches of penetration should be recorded separately as it is indicative of a granular material if the number of blows for the second 6 inches is significantly more than that for the first six inches. Curves based upon our experience to date with the Standard Penetrometer Test are attached to and supplement this paper. These curves show the relation between the test results and the shear strength of the soil as measured in the laboratory as well as the relation between the test results and measured dynamic pile resistance. The use of these charts will be discussed later under interpretation of results of sub-surface explorations.

HAMMER UNIT

CLOSE-UP
TRIPPING MECHANISM

FIG. 4

AUTOMATIC TRIP HAMMER
USED IN MAKING THE STANDARD PENETROMETER TEST

STANDARD PENETROMETER TEST
CONICAL DRIVING POINT

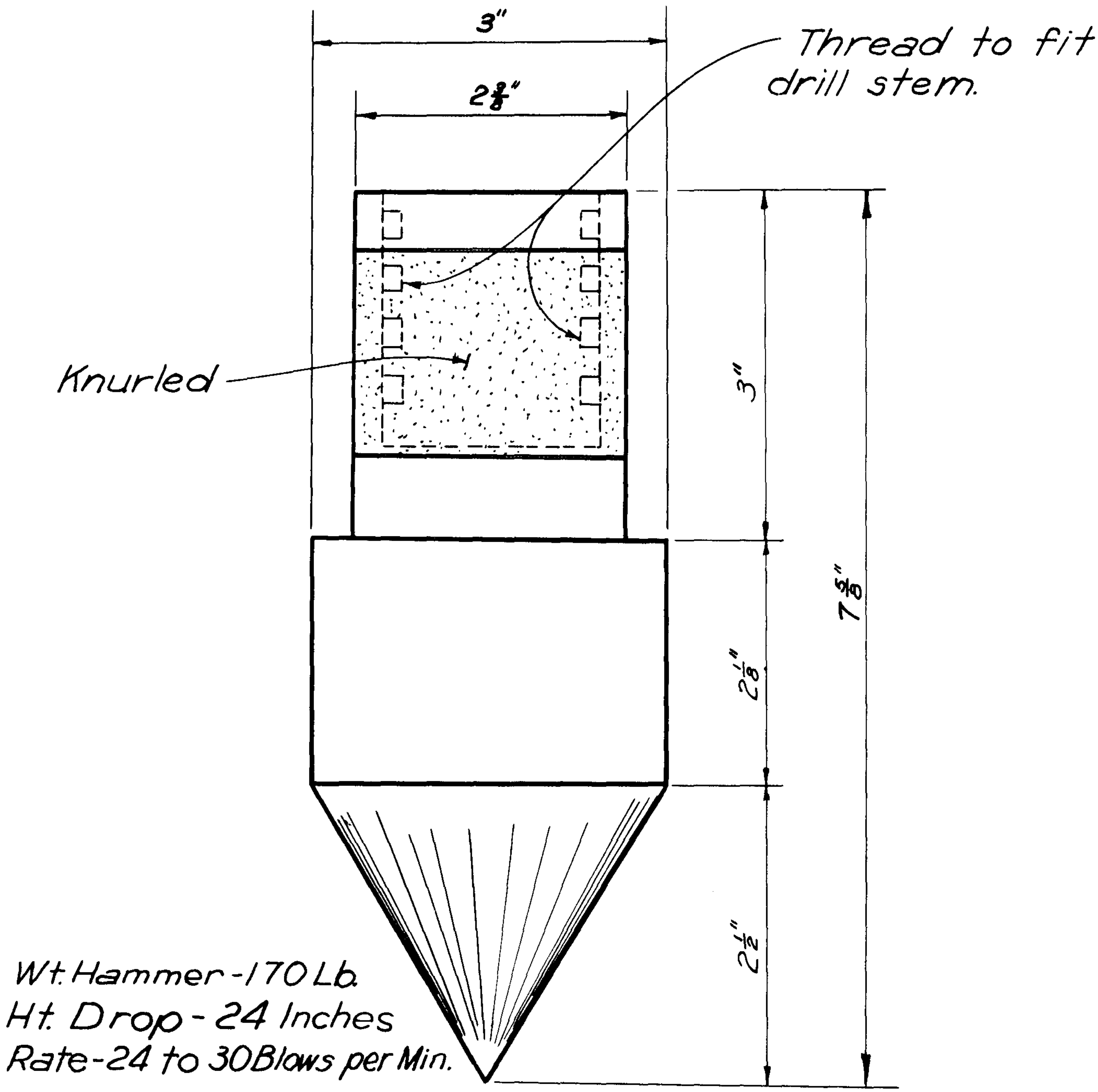


FIG. 5

BRIDGE DIVISION
JUNE 10, 1949

The first test hole at any Structure is generally made with very little advance knowledge of what will be encountered. It should be made carefully and with 100% core recovery with the Dry Barrel Sampler if at all possible. Penetrometer tests should be made each 10 feet and a good descriptive log of each significant stratum should be recorded. This first hole should be carried well below the probable founding depth of the substructure. If so called bed rock or a shale is encountered and is considered to be the probable formation on which the structure will be founded, it is recommended that this first hole be carried ten feet into the formation. For all formations other than rock or shale the hole should be carried to a depth below the probable founding elevation of approximately 50 feet. In applying this rule, where friction piles will probably be used, the founding elevation should be considered as the elevation corresponding to the center of resistance which may be assumed to be at the mid-point of pile penetration in the supporting earth strata. A rough determination of the founding elevation can be made from the results of Penetrometer Tests. A major structure is an exception to this rule in which case a more complete analysis should be made.

As a general rule it is suggested that no undisturbed sampling be attempted on the first hole. Upon completion of this first hole it should be possible to formulate a tentative plan of procedure for the remainder of the exploration. In formulating the tentative plan bear in mind that the completed exploration should contain the following:

1. Test holes at each end of the proposed structure plus a sufficient number of intermediate test holes to determine the location of all significant earth formations well below the probable founding elevation. The recommended maximum spacing of holes is 250 feet where the significant formations appear to be uniformly bedded.
2. An adequate number of Penetrometer tests to represent all significant formations.
3. Undisturbed samples for strength tests where large structures are involved and also for small structures if the formations indicate that the final design may include any of the following:
   (a) Friction piles to be driven in a formation showing less than 30 blows per foot with the Penetrometer.
   (b) Underreamed drilled shafts to be founded in a material showing less than 30 blows per foot with the Penetrometer.
   (c) Drilled shaft type of foundation without underreaming if there is doubt about the safe allowable unit load to be used.
4. A complete log record for each test hole on the Departmental Form 513, including the information called for at the top of the form. (See recommended logging terminology at end of this chapter.)

On large jobs and on jobs where the formations are non-uniform, it is suggested that a pencil profile be plotted showing the test data. A study of this profile will help in attaining the overall objective previously mentioned.

In addition to the rotary drill rig exploration, it is sometimes desirable to drill one or more large auger test holes to determine the feasibility of the drilled shaft type design. At the present time this is the only sure method for determining the presence of water bearing strata that may affect the design and should be resorted to where the information can be justified economically. The large auger hole exploration is also useful on underpass structures as it affords a convenient means for the engineers representing the railroad to make an inspection which may result in a more economical substructure design.

INTERPRETATION:

The interpretation of the data obtained from subsurface explorations presents problems as complex as any encountered in the highway engineering field. The development of the technique of substructure design has lagged behind that in the field of superstructure design due to the difficulty in evaluating the strength and service characteristics of the subsurface materials.

However, significant progress has been made in the field of Soil Mechanics as it pertains to substructure design and much of the guess work, design changes during construction, and waste in overdesign can be avoided by application of recently proven techniques.

The interpretation of the test data is invariably tied in with the extent of the exploration and type of formation. For convenience the explorations are divided into four classes.

I. Plastic clay and sand-clay explorations where the taking of undisturbed samples appear unnecessary and the design is to be based upon observations and Penetrometer Tests.

II. Plastic Clay and Sand-Clay explorations where the use of undisturbed sampling and triaxial testing is indicated in addition to observed soil conditions and Penetrometer Tests.

III. Cohesionless Sand explorations both with and without undisturbed sampling but including visual classifications and Penetrometer Tests.

IV. Hard Clay, Shale and Rock explorations both with and without undisturbed sampling but including visual classifications and Penetrometer Tests.

Examples - Class I Explorations:

(a) <u>Pile Foundation Design</u>

Assume 14 inch square concrete Piles to be driven with No. 1 Vulcan Hammer. Required design load 28.0 tons per pile.

Test Hole Data:

0 feet - 15 feet Soft Gray Clay 9/88 (Pen Test)

15 feet - 38 feet Med. F. Tan Clay 20/132 (Pen Test)

38 feet - 75 feet Firm Tan Sandy Clay 45/176 (Pen Test)

Low water table at 8 feet depth.

Assume 8 feet Alignment hole. Using Correlation curve in Fig. 6, for dynamic resistance and correlation curve in Fig. 7 for estimating the static capacity of the pile the following table can be completed.

Depth Ft.	Estimated Dynamic Res. Tons (Fig. 6)	Frict. Area Pile Sq. Ft.	Static Resistance in Tons/Sq. Ft. (Fig. 7)	Estimated True Capacity Tons
8 -15 (7 Ft.)	4	32.7	0.13	4.
15-38 (23 Ft.)	18	107.4	0.24	25.
38-42 (4 Ft.)	7	18.7	0.56	10.
Total	29			39

This shows that we could expect to obtain the design capacity by the hammer formula with pile penetrations of about 42 feet, whereas if complete soil tests or a pile load test were made we probably would need only about 38 feet of penetration. The final decision is an economic one but ordinarily a saving of only 4 ft. per pile would not justify the time or expense of the more extensive investigation.

(b) Spread Footing or Drilled Shaft Design.

Assume same foundation condition as above example.

Past experience has shown that it is not safe practice to land a footing in a material, with a penetrometer test of less than 30 blows per foot, without making strength tests. We will assume for our example that the proposed structure is small and the soil strength tests cannot be justified. Therefore, we will not consider landing above the 38 ft. depth and our problem is to determine the safe allowable unit load in the material showing 45 blows per foot with the penetrometer. From the correlation curves in Fig. 8, we find that 45 blows per foot on the lower curve shows an allowable bearing of 1.94 tons per square foot and the upper curve shows an allowable bearing of 2.8 tons per square foot. A visual inspection of the material indicates that a value of 2.5 tons per square foot of bearing would be conservative. The accuracy of this step in the solution is naturally dependent upon one's experience with soils. However, the lower curve represents what has proven to be a very conservative minimum value and the upper curve represents a fairly typical clay or sand-clay. The use of the values from these curves will always give conservative design and where the proposed structure is of considerable size, sound engineering will dictate that soil strength tests be run.

The 2.5 tons per square foot obtained from the curves can then be used as a basis for making an economic study of this design as compared with the pile foundation design determined in the first example.

Examples - Class II Explorations

It is not considered within the scope of this manual to cover the details of triaxial testing. Reference is made to a paper entitled "Triaxial Testing: Its Adaption and Application to Highway Materials with Addenda No. 1" which was developed and reported by the Soils Section of the Materials and Test Laboratory and distributed by Administrative Letter 43-50 together with several good texts on the subject such as "Fundamentals of Soil Mechanics" by Donald W. Taylor, "Soil Engineering" by M. G. Spangler, and "Soil Mechanics in Engineering" by Terzaghi and Peck. In the following examples, it is assumed that adequate triaxial tests are complete and the "Rupture" or "Strength" line has been determined on the Mohr's diagram for each significant formation involved.

(a) Pile Foundation Design

For an example in estimating pile lengths based upon soil strengths refer to Fig. 9, which shows a complete study for 14 inch precast concrete pile lengths on a grade separation structure on U. S. 75 in Galveston County.

Fig. 10, shows the tabulated data for each significant strata based upon triaxial test results. Fig. 11, shows the rupture or strength line for the "Firm Silty Clay" stratum at 36 to 40 ft. depth. This Rupture or Strength line was the result of drawing a line tangent to the Mohr's strength circles which were obtained from a series of triaxial tests of undisturbed samples of this particular stratum. Following thru the computations for this 38 to 40 ft. depth material on Fig. 10, the submerged density is shown as 57.2 which was obtained by subtracting 62.5 from the average wet density of 119.7, all in lbs, per cubic foot. The average depth of 38 ft. is the midpoint of the 36 to 40 ft. depth. The overburden pressure is assumed to act hydrostatically and is calculated by the equation:

$$U = \frac{WD}{144} = \frac{57.2\ (38)}{144} = 15.1 \text{ p.s.i.}$$

in which
U = Overburden pressure in lbs. per sq. inch.
W = Submerged Density in lbs. per cu. ft.
D = Average Depth in feet.

The average shearing strength of the stratum can then be taken graphically from the diagram Fig. 11, which is found to be 10.1 p.s.i. or 1454 p.s.f. This value can be calculated if preferred by scaling the value of cohesion, c = 4.5 p.s.i. and the angle of internal friction, ϕ = 20 degrees, from the same diagram and using the following equation:

$$S = c + U \tan \phi = 4.5 + 15.1\ (0.364).$$

$$S = 10.1 \text{ p.s.i.}$$

The pile surface area within the 4 ft. stratum is 4.67(4) = 18.7 sq. ft. The ultimate capacity of the pile within the 38 to 40 ft. stratum is calculated with the following equation:

$$P' = Sa = 1454(18.7) = 27{,}200 \text{ lbs.}$$

$$\text{or } P' = 13.6 \text{ tons.}$$

A factor of safety of 2 is applied to this ultimate capacity for each of the strata and the accumulated static capacity curve using submerged densities is plotted as shown in Fig. 9. This curve shows that the design load of 31.4 tons will require that the pile tip be driven to 40 ft. depth. As an interesting follow-up of this problem, it will be noted that the dynamic driving resistance actually obtained was 17.2 tons as shown by the short dash curve in Fig. 9. This pile was load tested and proven adequate for a design load in excess of 45 tons. Time did not permit running the test to theoretical pile failure but the net settlement obtained indicated the pile could have been proven safe for a design load of over 52 tons which was the indicated capacity based upon using the wet density of the soil in computing the overburden pressure.

(b) Underreamed Drilled Shaft and Spread Footing Design

For this example reference is made to Fig. 11, showing graphically the results of triaxial tests on the 36.0 to 40.0 ft. depth stratum used in the above example. It is assumed that the footing is to be landed so that the point of maximum shear will occur at 38 ft. depth and it is desired to calculate the maximum safe unit design load using a factor of safety of 2. This is an approximate graphic solution which is based upon stress equations and assumptions which will give conservative results when used within the limitations noted.

Assumption and Stress Equations:

$$Z = 0.707\ r \qquad (1)$$

$$U = W(d+Z)12 \qquad (2)$$

$$P' = \frac{V-U}{0.808} \qquad (3 - A)$$

$$P' = \frac{H-U}{0.23} \qquad (3 - B)$$

$$P = \frac{P'}{F.S.} \qquad (4)$$

Where:

Z = depth, in feet, from bottom of footing to point of maximum shear stress in soil.

r = radius of footing or radius of equivalent circular area for non-circular footings, in feet.

U = overburden soil pressure, assumed to act hydrostatically, in pounds per sq. inch.

W = average density of soil overburden in pounds per cu. inch. (Conservative practice requires that we use submerged density for substructures in stream beds, or where surface drainage is poor, and where the overburden soil is sandy).

d = depth, in feet, from surface of ground to bottom of footing. Where material is subject to scour, take d as distance from point of maximum scour to bottom of footing.

V = vertical unit stress or major principal stress expressed in pounds per sq. inch.

H = lateral unit stress or minor principal stress expressed in pounds per sq. inch.

P' = unit load on soil at footing elevation that will result in theoretical failure of soil in shear.

P = maximum safe unit design load in soil at footing elevation based upon a given Factor of Safety (F. S.) usually taken as 2.

P' and P can be expressed either in pounds per sq. inch or per sq. ft. Ultimately, P is usually expressed in Tons per sq. ft.

The angle of 33^{o} - 50' which the "Stress Line" makes with the horizontal axis and the influence values in the above stress equations are based upon an assumed Poissons Ratio of 0.5.

Solution:

Fig. 11 shows the Rupture or Strength Line of the 36.0 to 40.0 ft. stratum as plotted from the Triaxial test data.

The "Stress Line" is drawn in making an angle of 33^{o} -50' with the horizontal axis and passes thru the value of U = 15.1 p. s. i. on the horizontal axis which, as determed by the preceding example, is the overburden soil pressure for this example.

The maximum stress circle is then drawn in tangent to both the "Stress Line" and the "Rupture Line." Where the right side of this maximum stress circle cuts the horizontal axis, the value of V = 88.2 p. s. i. is obtained and where the left side of the maximum stress circle cuts the horizontal axis the value of H = 35.9 p. s. i. is obtained.

P' can then be obtained by substituting the value of V in equation (3 - A) or the value of H in equation (3 - B) as shown in Fig. 11, giving a value of P' = 90.4 p. s. i. by either equation.

Using a factor of safety of 2, compute the value of P = 45.2 p. s. i. or ---- P = 3.2 tons per sq. ft.

With this information, the designer can make an economic comparison of the pile and underreamed drilled shaft designs. On this particular project the pile foundation was chosen because of the reasonable doubt that existed as to the feasibility of underreaming due to the water bearing characteristics of the stratum. Also, the cost differential was small. If the cost differential had been significant, an auger test hole could have been justified to verify the feasibility of underreaming.

In addition to the assumptions stated above, this method of estimating the safe allowable design load on a soil is applicable only when:

1. The depth of the footing below the point of maximum scour is greater than the footing diameter.

2. The foundation soil is a plastic or semi-plastic type of material.

3. The foundation soil is of uniform or of increasing strength for a considerable depth below footing.

4. In case of rectangular footings, the length is not greater than 1 1/2 times the width.

5. The triaxial test results are based upon reasonably undisturbed samples of the strata involved and a sufficient number of tests were made to obtain representative soil strengths.

III. Cohesionless Sand Explorations

This type of formation does not lend itself to undisturbed sampling for Triaxial Testing. Undisturbed samples for density tests can be obtained with the Sand Sampler previously mentioned but the operation is slow, tedious and costly and is not justified except on large projects. The usual procedure is to make an adequate number of Penetrometer tests upon which to base an interpretation. If the sand is known to be cohesionless and the Penetrometer shows less than about 30 to 45 blows per foot without much increase in the number of blows for the second 6 inches of penetration, then the sand is in a very loose state and will be a very poor foundation.

If the sand shows a marked increase in the number of blows for the second 6 inches of penetration under the Penetrometer and the number of blows per foot is above about 45, then the sand is reasonably dense and will become more dense when loaded. A conservative estimate of the static capacity of a friction pile, in such a sand, can be made by assuming an angle of internal friction of 30° and applying the basic coulomb equation:

$$R = \frac{(C = Wh \tan \phi) A}{F.S.} \qquad (5)$$

Where:

C = Cohesion = 0 (in case of sand)

W = Submerged density of sand (Use 50#/c.f.)

h = distance (in feet) below maximum scour depth to center of area of pile.

$\tan \phi$ = 0.577 (Assuming $\phi = 30^{o}$)

A = Surface area (sq. ft.) of pile in friction below the point of maximum scour.

R = design pile capacity in pounds

F.S. = Factor of Safety (Use 2)

Example

Find required penetration of a 15 inch square precast concrete pile to carry a design load of 35 tons in a deep cohesionless sand showing a Penetrometer Test value of 21 blows for first 6 inches of penetration and 48 blows for the second 6 inches of penetration. Assume maximum scour depth to be 15 feet.

Area of 15 inch sq. pile

= 5 sq. ft. per foot of penetration

Then $A = 5p$

where p = effective penetration of pile below point of maximum scour.

Substituting in equation (5)

$$35 \times 2000 = \frac{\left(0 + 50\frac{(p)}{2}\ 0.5777\right)5p}{2}$$

$p = 44$ ft.

Required penetration would then be 44 ft. plus 15 ft. (scour depth) = 59 ft. total.

It is quite evident that the maximum scour depth is very important and good judgment must be exercised in its determination.

IV. Hard Clay, Shale and Rock Explorations.

Materials in this class of exploration will show less than 12 inch penetration with 100 blows under the Penetrometer Test.

(a) Pile Foundation Design

As shown in Fig. 6, a pile driven into materials of this class will reach refusal with a foot or two penetration and the maximum safe design load will be governed by the capacity of the pile as a structural member. Under this condition the pile will be a point bearing pile and sufficient penetration must be required to give adequate lateral support.

(b) Drilled Shaft and Spread Footing Design

Where undisturbed samples are taken, the maximum safe allowable unit pressure is taken as one-half the ultimate crushing strength in tons per square foot as obtained from unconfined compression tests of the samples. This results in an ultra-conservative use of the material strength in its confined state and occasionally a more complete analysis is justified.

On projects where undisturbed samples cannot be justified, a conservative estimate of the maximum safe allowable unit design pressure can be made from the results of Penetrometer Tests and the use of the correlation curve shown in Fig. 12.

Precautions to be observed in the interpretation of results. Make sure that:

(1) The landing elevation of the footing is below the point of maximum scour.

(2) The earth stratum on which the footing is to be founded is of uniform or of increasing strength for at least five feet below the proposed founding elevation and is free from soft or yielding formations for at least 15 feet below the founding elevation.

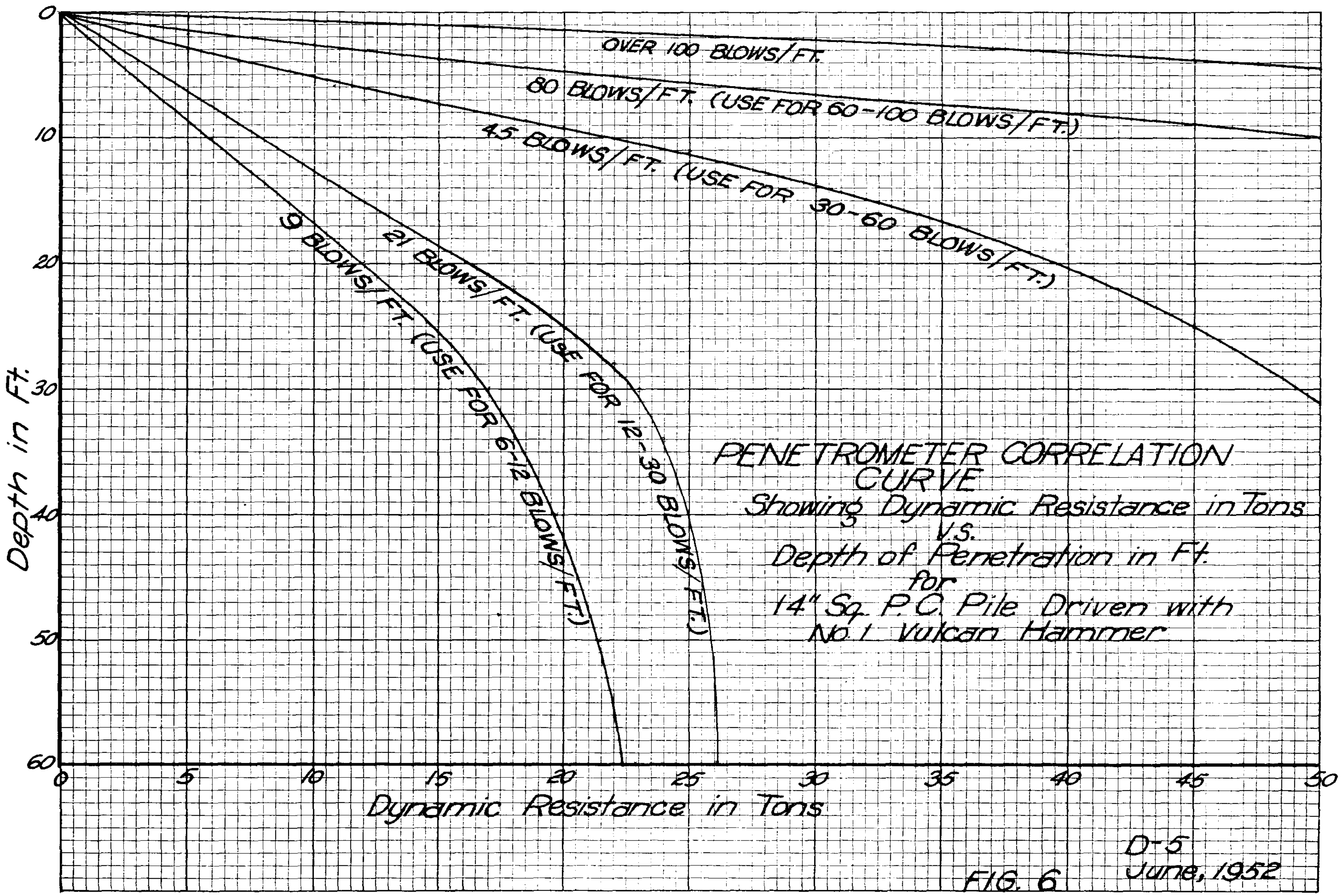
(3) An adequate number of tests have been run to be certain they represent the actual condition of the material.

Conclusion:

The interpretative procedures outlined herein are believed to represent sound conservative engineering practice. However, the correlation curves are based upon experience of the Department to date and are subject to revision as more information is obtained. The use of the curve as well as the graphic method of interpreting the triaxial test data should be accompanied by good sound engineering judgment.

The true cost of an adequate foundation exploration is measured not by the preliminary cost but by the preliminary cost less the saving in construction cost as a result of the adequate exploration.

In addition to this saving in the design of the structure, reliable exploration data will result in better relations between the Contractors and the State, which will ultimately result in lower bid prices.


PENETROMETER CORRELATION CURVE
Showing Dynamic Resistance in Tons
V.S.
Depth of Penetration in Ft.
for
14" Sq. P.C. Pile Driven with
No. 1 Vulcan Hammer
OVER 100 BLOWS/FT.
80 BLOWS/FT. (USE FOR 60-100 BLOWS/FT.)
45 BLOWS/FT. (USE FOR 30-60 BLOWS/FT.)
21 BLOWS/FT. (USE FOR 12-30 BLOWS/FT.)
9 BLOWS/FT. (USE FOR 6-12 BLOWS/FT.)
Depth in Ft.
0
10
20
30
40
50
60
0
5
10
15
20
25
30
35
40
45
50
Dynamic Resistance in Tons
D-5
June, 1952
FIG. 6

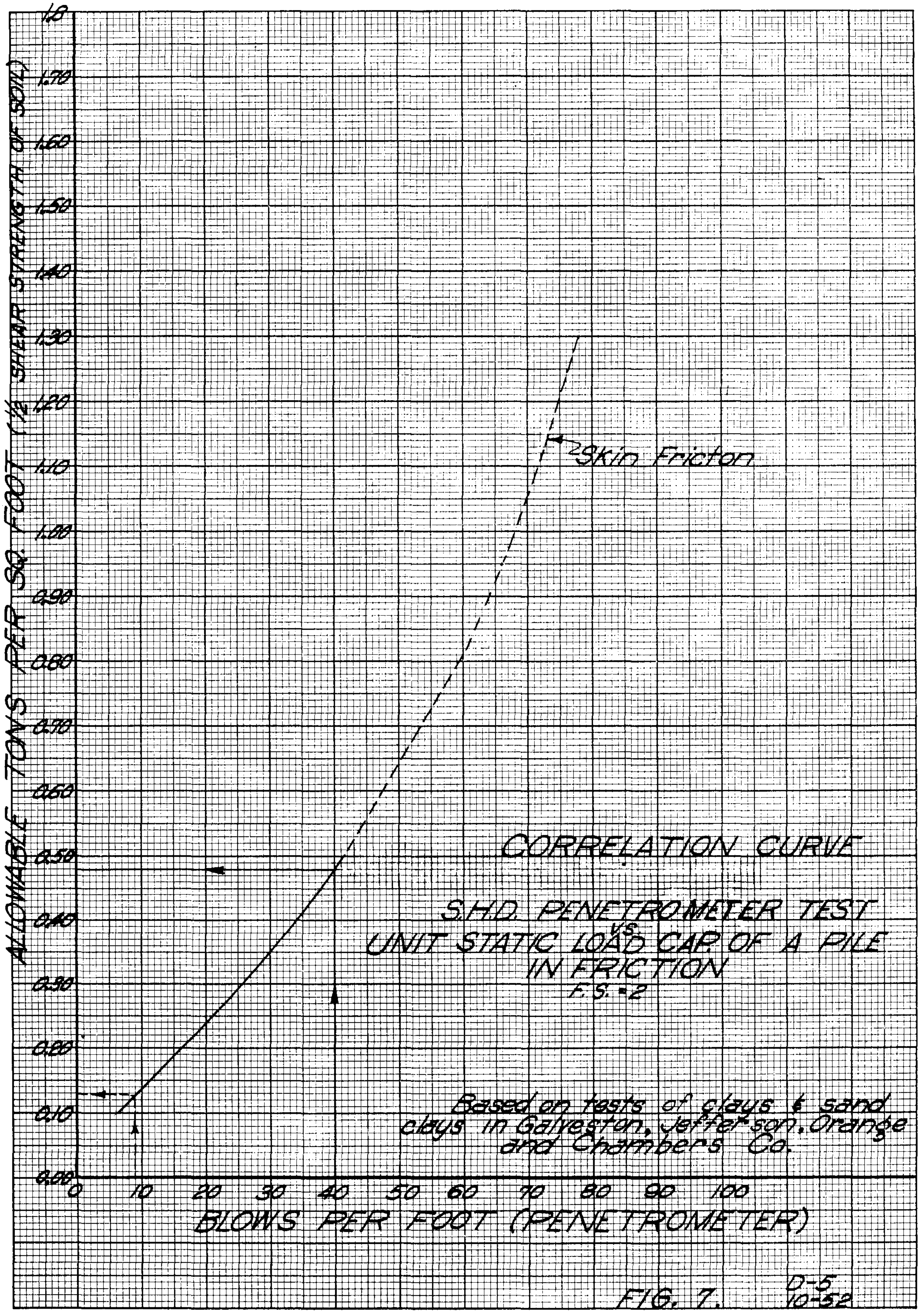
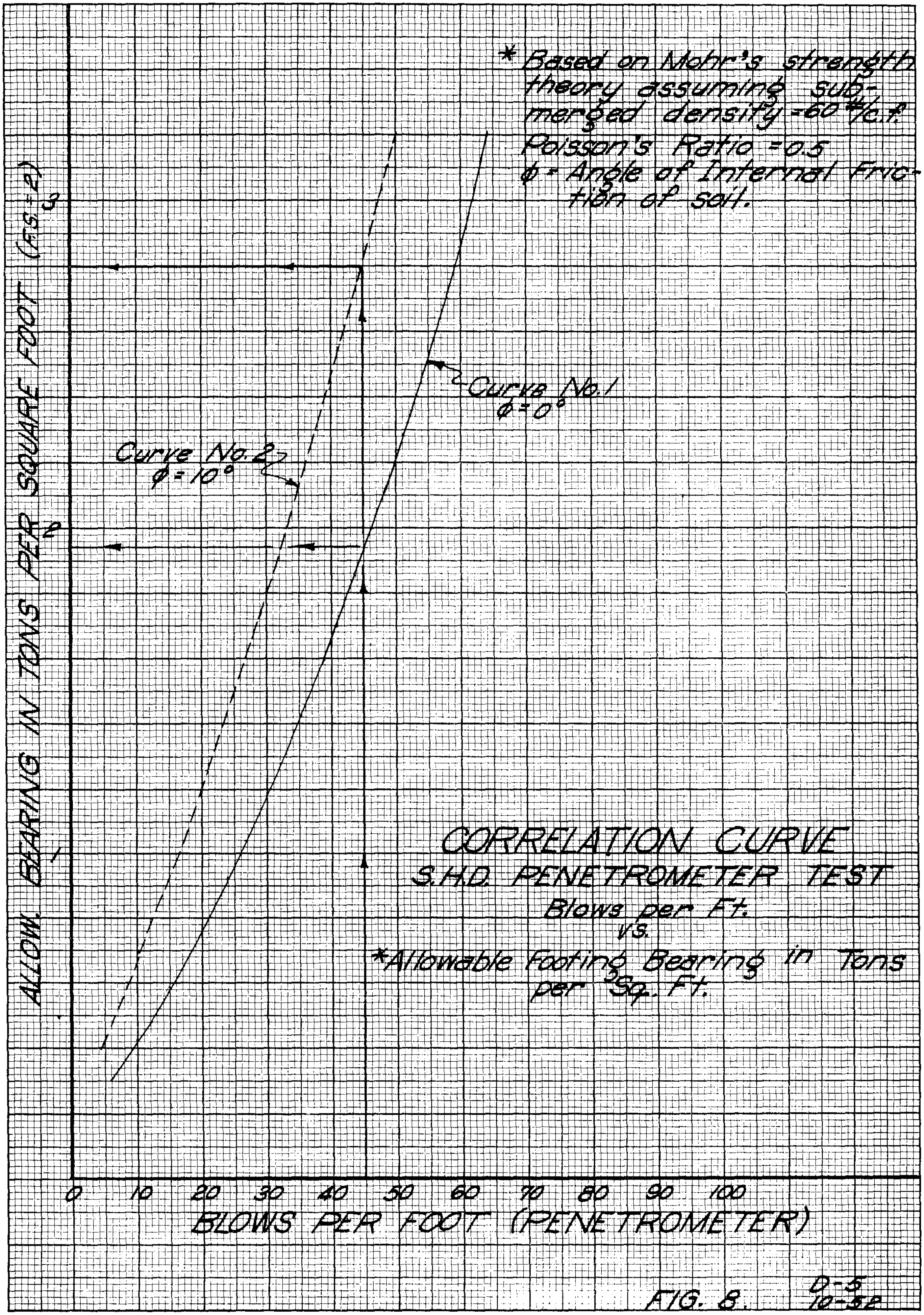
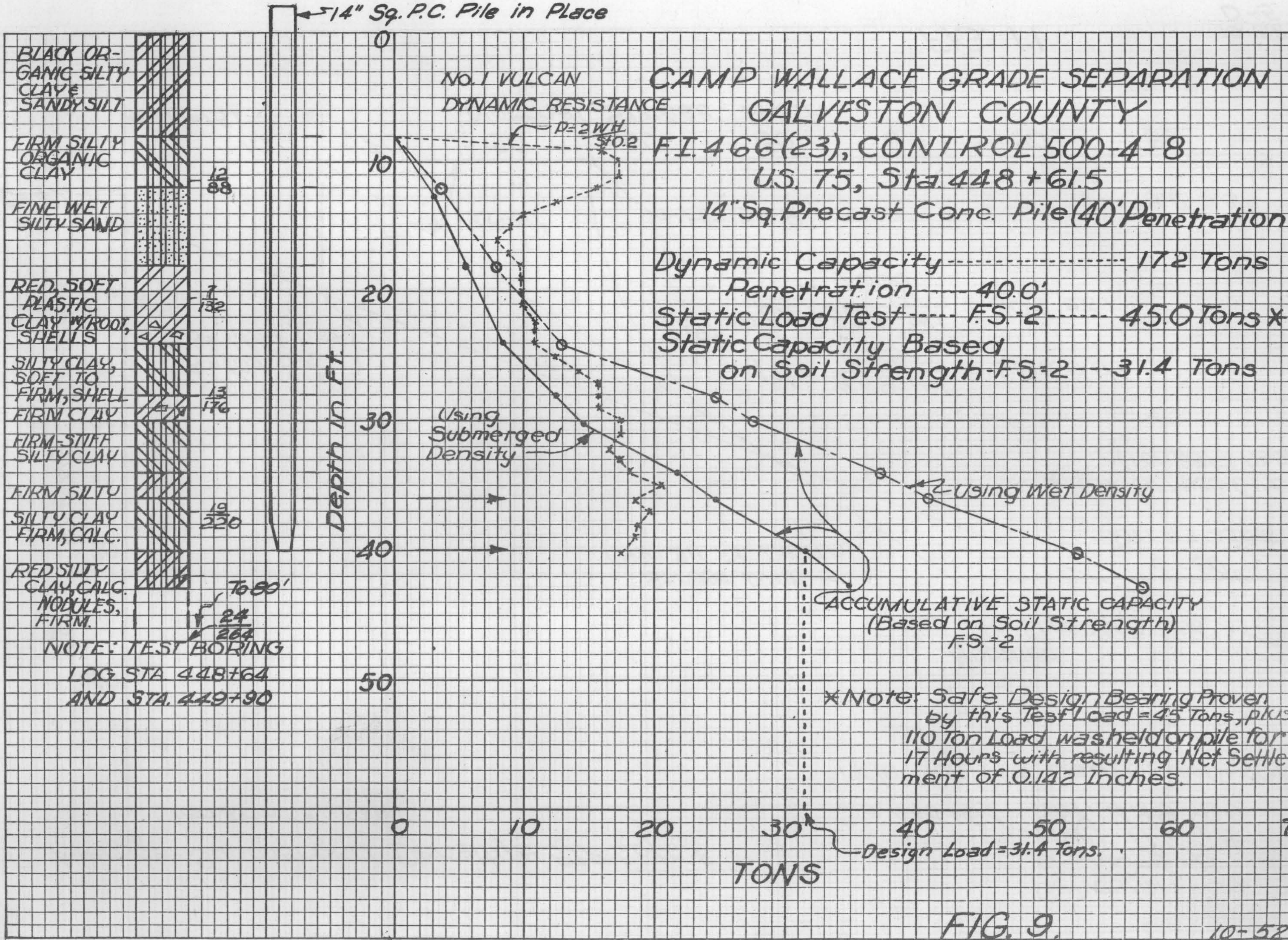

ALLOWABLE TONS PER SQ. FOOT (1/2 SHEAR STRENGTH OF SOIL)
1.8
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0
10
20
30
40
50
60
70
80
90
100
BLOWS PER FOOT (PENETROMETER)
Skin Friction
CORRELATION CURVE
S.H.D. PENETROMETER TEST
VS.
UNIT STATIC LOAD CAP. OF A PILE
IN FRICTION
F.S. = 2
Based on tests of clays & sand clays in Galveston, Jefferson, Orange and Chambers Co.
D-5
10-52

FIG. 7.


* Based on Mohr's strength theory assuming submerged density = 60 #/c.f.
Poisson's Ratio = 0.5
φ = Angle of Internal Friction of soil.
Curve No. 1
φ = 0°
Curve No. 2
φ = 10°
CORRELATION CURVE
S.H.D. PENETROMETER TEST
Blows per Ft.
vs.
*Allowable Footing Bearing in Tons per Sq. Ft.
ALLOW. BEARING IN TONS PER SQUARE FOOT (F.S. = 2)
1
2
3
0
10
20
30
40
50
60
70
80
90
100
BLOWS PER FOOT (PENETROMETER)
FIG. 8.
D-5
10-52



14" Sq. P.C. Pile in Place
BLACK ORGANIC SILTY CLAY & SANDY SILT
FIRM SILTY ORGANIC CLAY
12/88
FINE WET SILTY SAND
RED, SOFT PLASTIC CLAY W/ROOT, SHELLS
7/132
SILTY CLAY, SOFT TO FIRM, SHELL
13/176
FIRM CLAY
FIRM-STIFF SILTY CLAY
FIRM SILTY
19/220
SILTY CLAY FIRM, CALC.
RED SILTY CLAY, CALC. NODULES, FIRM.
To 80'
24/264
NOTE: TEST BORING LOG STA. 448+64 AND STA. 449+90
Depth in Ft.
0
10
20
30
40
50
No. 1 VULCAN DYNAMIC RESISTANCE
D=2WH/S+0.2
Using Submerged Density
Using Wet Density
ACCUMULATIVE STATIC CAPACITY (Based on Soil Strength) F.S.=2
Design Load = 31.4 Tons.
0
10
20
30
40
50
60
70
TONS
CAMP WALLACE GRADE SEPARATION
GALVESTON COUNTY
F.I. 466(23), CONTROL 500-4-8
U.S. 75, Sta. 448+61.5
14" Sq. Precast Conc. Pile (40' Penetration)
Dynamic Capacity ------ 17.2 Tons
Penetration ---- 40.0'
Static Load Test ---- F.S.=2 ---- 45.0 Tons *
Static Capacity Based on Soil Strength - F.S.=2 --- 31.4 Tons
*Note: Safe Design Bearing Proven by this Test Load = 45 Tons, plus 110 Ton Load was held on pile for 17 Hours with resulting Net Settlement of 0.142 Inches.

FIG. 9.

10-52
D-5

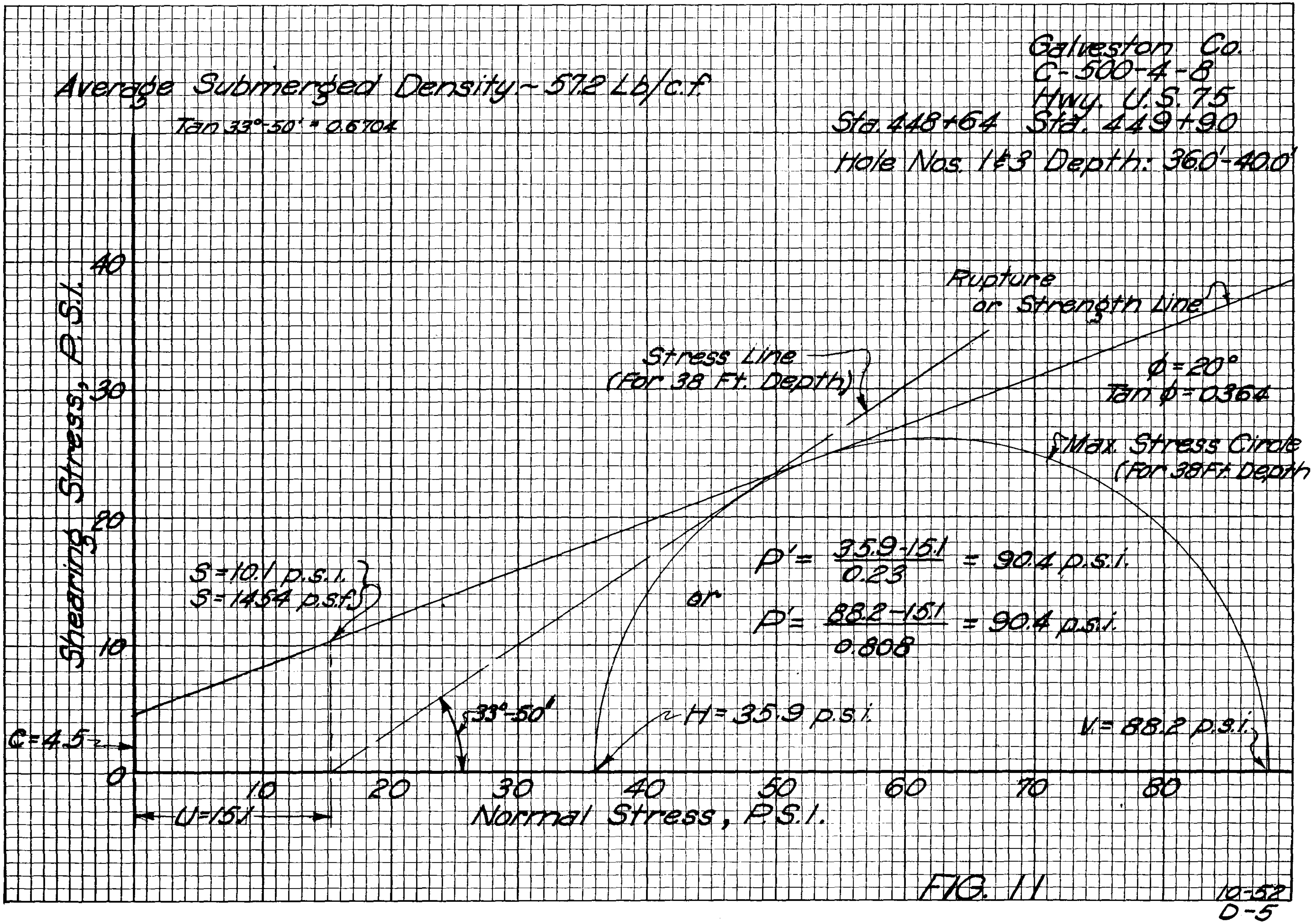

Average Submerged Density ~ 57.2 Lb/c.f.
Tan 33°-50' = 0.6704
Galveston Co.
C-500-4-8
Hwy. U.S. 75
Sta. 448+64 Sta. 449+90
Hole Nos. 1 & 3 Depth: 36.0'-40.0'
Rupture or Strength Line
Stress Line (For 38 Ft. Depth)
φ = 20°
Tan φ = 0.364
Max. Stress Circle (For 38 Ft. Depth
S = 10.1 p.s.i.
S = 1454 p.s.f.
P' = 35.9-15.1 / 0.23 = 90.4 p.s.i.
or
P' = 88.2-15.1 / 0.808 = 90.4 p.s.i.
33°-50'
H = 35.9 p.s.i.
V = 88.2 p.s.i.
C = 4.5
U = 15.1
Shearing Stress, P.S.I.
Normal Stress, P.S.I.
0
10
20
30
40
50
60
70
80
10-52
D-5

FIG. 11

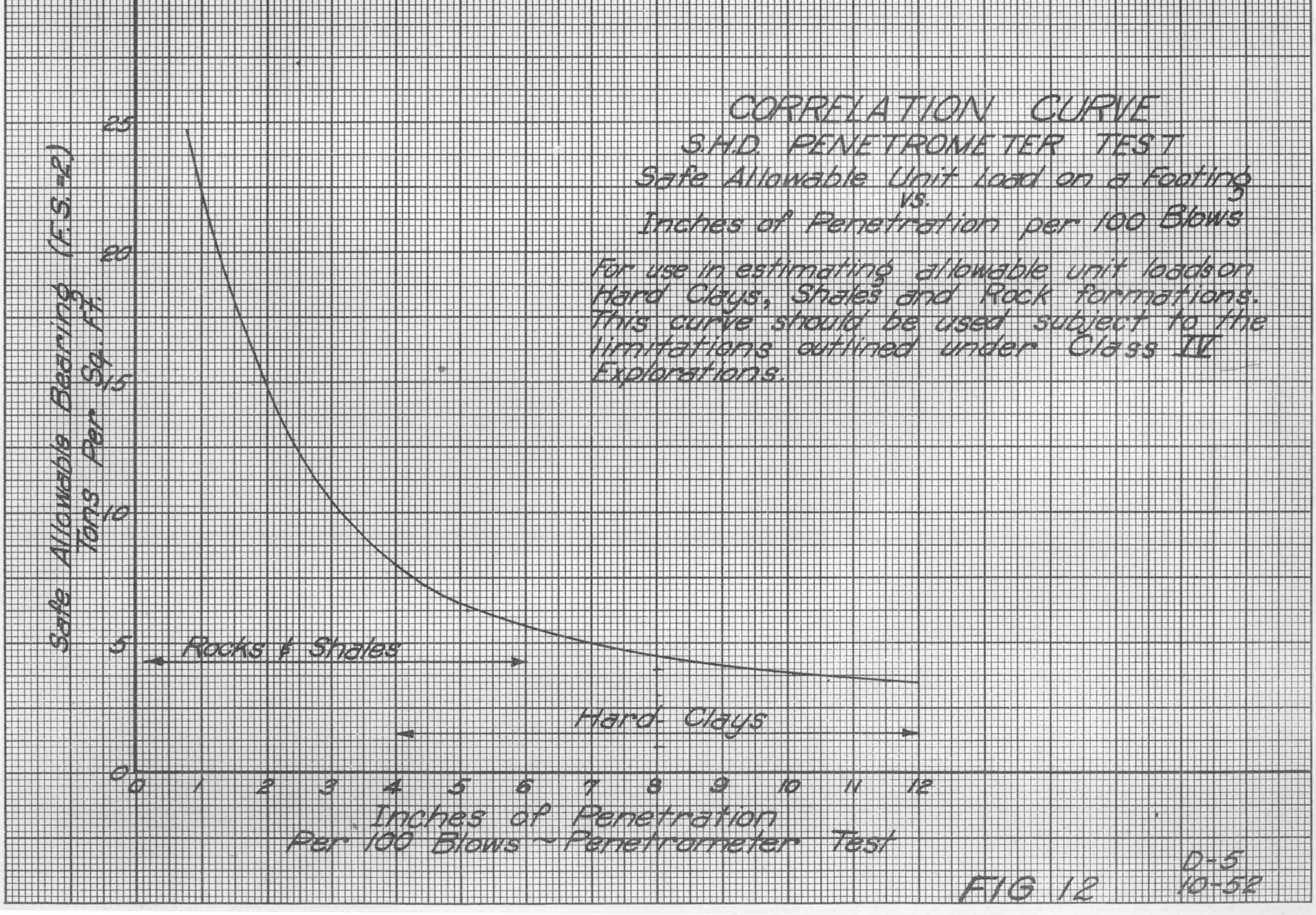

CORRELATION CURVE
S.H.D. PENETROMETER TEST
Safe Allowable Unit Load on a Footing
vs.
Inches of Penetration per 100 Blows
For use in estimating allowable unit loads on Hard Clays, Shales and Rock formations. This curve should be used subject to the limitations outlined under Class IV Explorations.
Safe Allowable Bearing (F.S.=2)
Tons Per Sq. Ft.
0
5
10
15
20
25
0
1
2
3
4
5
6
7
8
9
10
11
12
Rocks & Shales
Hard Clays
Inches of Penetration
Per 100 Blows ~ Penetrometer Test
D-5
10-52

FIG 12

CALCULATED DATA FOR PILE BEARING
(Using Submerged Density)

Control 500-4-8   Project FI 466(23)   Camp Wallace Grade Separation   Sta. 448+61.5   Hwy. US 75

14 inch concrete pile (precast)

STRATA** ft.	DENSITY* "W" #/cf	AV. DEPTH "D" ft.	HYDRO. PRESS. "U" psi	SHEARING STRENGTH "S" psi	SHEARING STRENGTH "S" psf	PILE SURFACE "A" AREA. ft.	ULTIMATE CAPACITY "P" #	ULTIMATE CAPACITY "P" tons
8-12	61.0	10.5	4.45	3.6	519	14.0	7,260	3.63
12-18	61.0	15.0	6.35	3.0	432	28.0	12,100	6.05
18-24	42.0	21.0	6.12	2.8	403	28.0	11,300	5.65
24-28	45.9	26.0	8.29	6.1	880	18.7	16,450	8.23
28-30	64.1	29.0	12.92	5.7	820	9.33	7,650	3.87
30-34	63.4	32.0	14.1	11.0	1585	18.7	29,600	14.80
34-36	58.0	35.0	14.1	8.7	1250	9.33	11,650	5.82
36-40	57.2	38.0	15.1	10.1	1454	18.7	27,200	13.60
40-42.8	55.0	41.3	15.7	7.6	1095	12.5	13,700	6.85
							TOTAL FRICTIONAL RESISTANCE	68.50

*Submerged density calculated by subtracting 62.5 from Av. Wet Density

**Strata measured from original ground elevation of 22.0 ft. Bottom of pile is 42.8 ft. below this datum.

FIG. 10

RECOMMENDED LOGGING TERMINOLOGY

7 BASIC GROUPS OF MATERIAL WITH DEFINITIONS

1. ROCK is a solidified, unyielding material which is not subject to change of form, volume or supporting value under wide changes in moisture content.

2. GRAVEL is a non-plastic, cohesionless, granular material composed of fine to coarse fragments of one or more kinds of rock. (Particle size: 100% retained on No. 10 sieve.)

3. SAND is a non-plastic, cohesionless, granular material composed of fine rock particles. (Particle size: 100% Passing No. 10 sieve and 100% retained on the No. 270 sieve.)

4. CLAY is an earthy material composed of the smallest particles of land waste. Its stability and plasticity varies widely with moisture changes. Particle sizes are all smaller than 0.005 millimeters.

5. SHALE is a fine grained material of highly compressed layers of clay, or silt and has a characteristic laminated structure such that it can be split into thin layers which usually run horizontal. Shale is highly affected by changes in moisture and loses much of its strength when not supported laterally.

6. ORGANIC MATERIAL covers a wide range of materials which cannot be suitably classified under the other 6 groups. It is composed of decayed vegetable, animal, or marine life. Characteristic formations in this group are mucks, peat and lignite.

7. SILT is a fine grained material (Particle size: 100% passing No. 270 sieve and minimum size of 0.005 millimeter) with little or no plasticity except when organic or clayey fractions are present. For the purpose of logging, loess is placed in the Silt Group on account of particle size. Loess is a wind borne deposit while silt, in the strict sense, is deposited by water action.

It is suggested that all formations be classified under one of the 7 basic groups. However, in addition there should be as many descriptive terms used as necessary to clearly cover the KIND and CONDITION of the formation. Also, all logs on a project should be reviewed collectively to be sure that similar materials are described similarly and that unnecessarily detailed logging is avoided.

The suggested descriptive terms are only a few of the commonly used ones. Additional descriptive terms should be used freely in actual practice.

7 BASIC GROUPS WITH SUGGESTED DESCRIPTIVE TERMS

BASIC GROUP	KIND OF FORMATION		CONDITION OF FORMATION		
Rock	Sandstone	Slate	Soft	Firmly Cemented	
	Limestone	Granite	Medium Hard	Laminated	
	Chalk	Flint	Hard	Nodular	
	Conglomerate	Gypsum	Loosely Cemented		
Gravel	Limestone	Clayey	Fine	Dense	
	Flint	Silty	Coarse	Well graded	
	Caliche		Loose	Water Bearing	
	Sandy		Compact	Clean	
Sand	Clayey (Loam)		Fine	Compact (Pack)	
	With Clay Lenses		Coarse	Dense	
	Gravelly		Well Graded	Cohesionless	
	With Sandstone Lenses		Water Bearing		
	Silty		Loose		
Clay	Sandy	Silty	Very Soft	Mucky	Varved
	Gravelly	Organic	Soft	Slickensided	Marly
	Shaley	Calcareous	Plastic	Friable	Marbelized
	Joint	Loamy	Stiff	Fissured	
	With Sand Lenses		Hard	Crumbly	
Shale	Sandy		Soft		
	Silty		Medium Hard		
	With Clay Lenses		Hard		
	With Sandstone Lenses		Fissured		
Organic Material	Lignite		Odorous		
	Peat				
	Muck				
	Silty				
	With Clay Lenses				
	With Sand Lenses				
Silt	Organic		Loose		
	Inorganic		Dense		
	Clayey		Water Bearing		
	Sandy				
	Gravelly				
	Loess				

NOTE: Log observed moisture condition of material in natural state by terms of Dry, Moist or Saturated. Whenever necessary, supply additional appropriate descriptive or classifying terms.
In addition to the description of the KIND and CONDITION of a formation the log should include an accurate color description based upon the appearance of the formation with its natural moisture content.

RAINFALL AND RUNOFF

1. INTRODUCTION.

In the United States the construction cost of highway drainage structures amounts to about $400,000,000.00 annually. Of this amount $240,000,000.00 is for large structures over 20 feet in length and $160,000,000.00 is for small structures. If by rational and balanced procedures of hydraulic design of these structures, a saving of five percent (5%) could be realized this saving would amount to $20,000,000.00 annually which is a sum more than half of the total Federal Aid Highway appropriation to the State of Texas for the year 1953. The problem of hydraulic design is, therefore, one that is a challenge to the exercise of the best engineering talent and judgment.

Before the waterway area and the grade line requirements of a drainage structure can be determined it is necessary to ascertain the flood flow characteristics of the stream to be bridged. Sources of data and methods of analysis used in these determinations will be discussed in this paper. Other papers in this school will demonstrate the use of the information derived in this paper in the actual design of highway drainage structures.

2. FLOOD FLOWS.

For a specific crossing flood flow characteristics that it is desired to know are maximum probable discharge, the percentage of maximum probable discharge that may be expected to occur on the average in any given period of years and the flood stages that correspond with these discharges. This information may be summarized for the use of the hydraulic designing engineer in the stage-discharge curve or the rating curve and the stage-frequency curve. Obviously, the discharge-frequency curve can be derived from the stage-discharge and the stage-frequency curves. Figure 14 is an example of stage-discharges or rating curve.

All flood discharges are derived from runoff from rainfall on the area of watershed tributary to the stream. Flood flow characteristics may, therefore, be broken down into characteristics of rainfall, of drainage area or watershed, and of the channel. In the general case, therefore, data must be collected for all these factors.

3. WATERSHED.

In the determination of flood discharge one of the first steps to be taken is that of evaluating the size, shape, culture, slope and land use of the drainage area.

There are several methods for determining the size and shape of the drainage area. For small areas, the area may be determined by direct field surveys making use of the stadia method. From the direct survey method the culture, land use, slope and land types may be observed, noted in the field book, and subsequently shown on the plans.

Where the area is too large to make direct surveys economical, other methods must be employed. Among these are the use of published contour maps, the use of Texas Highway Planning Survey maps and the use of aerial maps which may be purchased from the U. S. Government or from special aerial surveying companies. In some cases reference can be made to aerial maps in the County office of the Production and Marketing Administration of the Department of Agriculture.

All rainfall does not run off. The amount of runoff is dependent upon the slope of the land, the type of the soil, the cover crop on the land and man made obstructions. Over long periods, fine soils like clay will absorb and hold more rainfall than large particle soils like sand and gravel. Water, however, will enter fine soils more slowly than it will sand and thus fine soils will contribute more to runoff in a large storm. The cover crop has a decided influence on the ability of any soil to permit absorption by providing channels for water to enter the soil, by retarding runoff and by preventing soil compaction by rain drops.

Runoff is also affected by man made obstructions such as terraces, diversion channels and check dams. Air pictures afford excellent opportunity to study the extent and type of man made obstructions so that proper allowances can be made in runoff computations.

4. RAINFALL.

After the drainage pattern has been determined, the next problem is that of determining the rainfall which may be expected on the area.

Rainfall is water evaporated from the sea, land or bodies of water and as vapor is carried by the wind until there is a drop in temperature which causes the vapor to precipitate.

In the ''Handbook of Applied Hydraulics'' by Davis is found the following: ''If a cubic foot of air near sea level, at 100F, has six grains of water vapor intermixed (a medium drop of water would have equal weight), it would produce 30 per cent saturation. If the temperature of the air mass falls to 70 degrees, the percentage of saturation arises to 77 per cent; at 60 degrees, complete saturation is reached, or the dew point is attained and condensation is produced, and at 40F, slightly more than half the moisture will have fallen.''

As you can see the amount of rainfall in a given area depends upon the amount of vapor (clouds), the velocity of the wind and the rate of vapor condensation. Later in this paper the importance of distribution of rainfall will be demostrated.

Rainfall measurements have been taken over long periods at many stations in Texas by the U. S. Weather Bureau and other agencies. The Highway Department has taken the rainfall data from these reports and has compiled general rainfall curves for each district. These data usually provide adquate accuracy for the design of our structure.

From rainfall records it is possible to determine intensities for various intervals of time. For small drainage areas, the time it requires water to flow from farthest reach to the structure site is called the time of concentration (inlet time in storm sewer design) and usually the intensity of rainfall for this time interval is used to determine the discharge.

Another factor to be considered is the direction of the prevailing winds over the drainage area. If the drainage area pattern is crossed by the prevailing winds one rate of runoff may be expected but a different rate of runoff may be anticipated from an area where the prevailing winds pass up or down the main channel.

The importance of distribution can be illustrated by reference to September 1952 floods in West Central Texas. From the information collected during this flood, it is noted that the rainfall covered a rather small area of West Central Texas but affected the Guadalupe, San Marcos, Blanco, Pedernales, Llano and the Colorado Rivers. The rainfall for this storm varied up to 23 inches with this amount found in the Blanco area. As distributed this flood caused the following discharges:

Pedernales River at Johnson City	450,000 c.f.s.
Llano River at Llano	250,000 c.f.s.
Blanco River at U. S. Highway 81-San Marcos	96,400 c.f.s.
Blanco River at Wimberly	99,000 c.f.s.
Colorado River at U. S. Highway 190	80,000 c.f.s.
Colorado River into Lake Travis (2-15-min. Periods)	840,000 c.f.s.

Guadalupe River at Comfort	38,600 c.f.s.
Guadalupe River at New Braunfels	84,000 c.f.s.
Guadalupe River at Victoria	28,000 c.f.s.
San Marcos River at Luling	60,000 c.f.s.

A fifteen or twenty mile shift to the Northwest would have caused the Pedernales and Llano Rivers to have had much larger peak discharges but would have reduced the peak discharges in the Blanco, San Marcos and the Guadalupe Rivers.

5. FLOOD FLOW FROM SMALL WATERSHEDS.

The runoff from small drainage areas, where information on stage-frequency-discharge is not directly available, may be computed by the formula

$$Q = C\ I\ A$$

as explained in the Manuel "Rational Design of Culvert and Bridges." This Manuel has been distributed to the field and is commonly used. In the formula Q = discharge in cubic feet per second (c.f.s.); c = coefficient of runoff or ratio of runoff to rainfall; and I = intensity in inches per hour. One cubic foot per second per acre equals very closely one inch per hour of rainfall as shown:

$$1 \text{ c.f.s.} = 3600 \text{ cubic feet per hour}$$

$$1 \text{ acre inch} = \frac{43560}{12} = 3630 \text{ cubic feet}$$

The value of ''C'' for various watershed conditions is given in the attached Table 3. The conditions set up in Table 3 and the determination of the area A for the watershed have been discussed under Section 3. The value of ''I'' for various locations, frequencies and time of concentrations is given by the Texas Reclamation Department formula

$$I = \frac{b}{(t + d)^e}$$

This formula was derived from an extended study of actual rainfall records. Values of the constants b, d and e are given in the attached Table 2 (6 pages). The value of t is derived from calculations for overland flow using values of runoff velocities found in Table 1. The solution of the formula for I, time of concentrations of 240 minutes or less, and for frequencies from 2 to 100 years averaged for each of the twenty five Highway Districts is given in the attached Figures 2 to 7 inclusive.

6. FLOOD FLOW FROM LARGE WATERSHEDS.

On large watersheds the variation in topography, land use, and rainfall characteristics are so great that the direct application of the formula Q = C I A will give results which are unreliable. The U. S. Geological Survey in copperation with the Texas State Board of Water Engineers, the U. S. Army Engineers and other agencies maintain numerous stream flow gaging stations on the rivers and larger creeks of the State. From these records the State Board of Water Engineers has prepared a series of discharge curves for various frequencies and watershed areas. These curves are reproduced and attached as Figures

9 - 12 inclusive. These curves have been classified in four general areas for the State as shown in Figure 13.

Due to the great variations in rainfall and watershed characteristics within the various regions, the curves will only be approximately correct. Wherever gaging stations are nearby on the same or neighboring streams, studies should be made directly from records of the gaging stations. Actual stage-discharge curves are usually available for all gaging stations. Frequency relations can be determined by methods used by the U. S. Geological Survey and the Texas State Board of Water Engineers. This data can be obtained by application to the Bridge Division.

7. FLOOD FLOW BY MANNING'S FORMULA.

In cases where actual flood flow data are not available, an approximation of the flood flow characteristics can be obtained by the application of the Manning slope area formula to a representative cross section of the stream for various stages of flow. This formula is as follows:

$$Q = A\ V = \frac{A\ (1.486\ R^{2/3}\ S^{1/2}\ )}{n}$$

where R = hydraulic radius or the average depth in the section under consideration; S = the slope of the water surface; and n the coefficient of roughness.

In computing the Manning Formula, it is believed that time can be saved by using the slide rule prepared by District Engineer Gilbert A. Youngs of Atlanta. These slide rules have been distributed to the field through each District Engineer.

As an example of the calculation by Manning formula consider the Pedernales River at Johnson City, Blanco County, U. S. Highway 281. The old Pedernales River Bridge had a U. S. Geological Survey stream gage on one pier but this gage like the bridge was lost because of high-water and drift. Therefore, for this flood the U. S. G. S. took a profile of highwater at several reaches of the river and after subsidence took a typical channel section for each highwater profile from which the area could be determined. A value of "n" was set up for each part of the channel based upon experience, experiment and study. With this basic information the peak discharge was calculated using the formula shown above. In this formula V is the average velocity in feet per second, A is the area in square feet and Q is the discharge in cubic feet per second.

From the former records of this gaging station for less severe floods where the stage height was measured and the velocities obtained by current meter readings, the U. S. G. S. has built up a rating curve for this station. The rating curve shows stage height and discharge so that a structure can be designed for any frequency. The rating curve for the Johnson City gaging station is shown on Figure 14.

In cases where the differential head at an existing crossing can be measured, the discharges can be estimated by the use the head velocity formula:

$$Q = A V = A \sqrt{2 g h}$$

where A = the Waterway area in the structure in square feet, g = 32.2 and h = the differential head in feet.

8. FLOOD FLOW FROM INTERMEDIATE WATERSHEDS.

The limits of application of the here-in-before described methods for determining the flood flow characteristics of runoff from small and large watershed areas are not well defined. Usually the C I A method will be applicable to areas of ten (10) square miles or less, but it can occasionally be extended to areas of two hundred to three hundred (200-300) square miles. Charts of the Texas Board of Water Engineers do not extend below ten (10) square miles and due to the relatively small number of gaging stations on small watersheds, their applicability to areas less than one hundred (100) square miles is doubtful. In the range between ten and two hundred (10-200) square miles, it would be desirable to apply the criteria for both large and small watersheds in order to arrive at a reasonable compromise.

9. EXAMPLE FOR SMALL WATERSHED.

For the small watershed area a site in Harris County has been selected and is shown as Figure 1. From the map the slope, area, shape, culture and land classification is obtained. The slope is Elevation 82.2 - Elevation 67.5 = 14.7 or $\frac{14.7}{4200}$ = 0.0035 feet per foot. On the basis of information gathered in the field and plotted on the drainage map the time of concentration is computed using Table I as follows for overland flow:

1360 feet, slope 0.0035, timber cover	$= \frac{1360}{1 \times 60}$	= 22.7 minutes
1250 feet, slope 0.0035, grass pasture	$= \frac{1250}{1.5 \times 60}$	= 13.9 minutes
1590 feet, slope 0.0035, cultivated	$= \frac{1590}{2 \times 60}$	= 13.2 minutes
Total time		= 49.8 minutes
say 50 minutes		

The design frequency for this structure will be ten (10) years which is usual practice for small structures and the time of concentration is fifty (50) minutes. In order to get the intensity of rainfall refer to Figure 5 or calculate it from information given in Table 2. By use of Figure 5, the value of "I" is 3.75 inches per hour.

In order to determine the value of "C" refer to Table 3 and to Figure 1.

$C = (0.40 \times 0.35 + 0.30 \times 0.25 + 0.30 \times 0.20) = 0.275$

A = 230 acres from the drainage area map Figure 1. with this information

$Q = C\ I\ A = 0.275 \times 3.75 \times 230 = 238$ c.f.s.

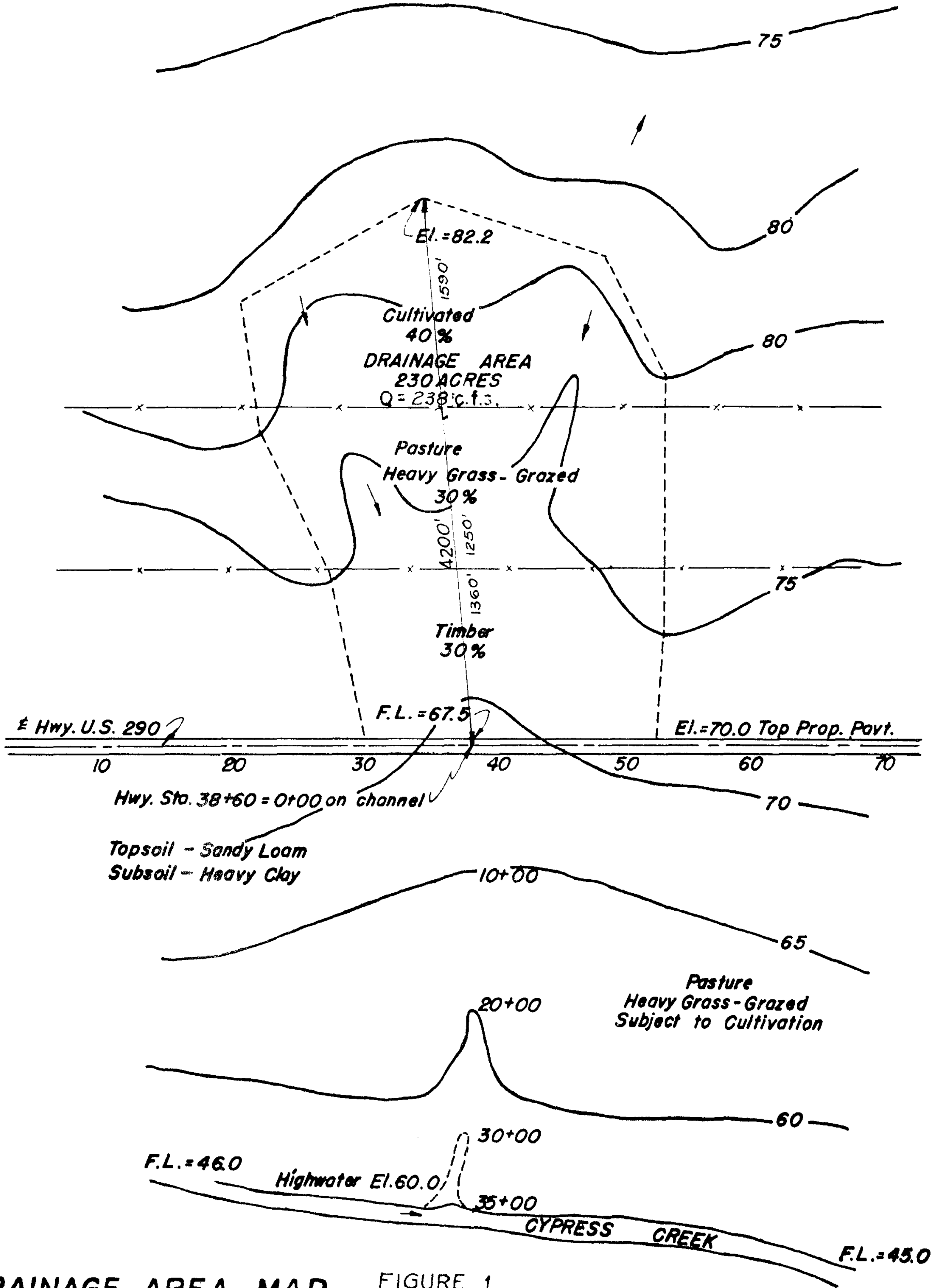
10. EXAMPLE OF LARGE WATERSHED.

For the large watershed the Pedernales River Crossing on U. S. Highway 281 at Johnson City in Blanco County has been selected. The drainage area is 947 square miles as given in the records for the gaging station located on the bridge lost in September 10, 1952 flood. If the drainage area had not been determined by the U. S. G. S. for this gaging station, it would have been determined by reference to U. S. G. S. contour maps for this area. This site was selected because it is covered by gaging records, because it has been subjected to extreme highwater in the September flood, because several other floods permitted the U. S. G. S. to develop a rating curve, and because the highwater slope was well established. Also because our discharge quantity would be checked by the U. S. G. S. for the 1952 flood.

In order to determine the discharge of the 1952 flood a typical cross section about 910 feet upstream from the highway crossing is shown as Figure 8. The highwater slope was established from stakes driven at the Highwater elevation on both sides of the river. At the typical section the North bank had highwater 2.4 feet higher than the south bank because of disturbance caused by the entrance of a creek slightly upstream. This disturbance likewise affected the value of the coefficient of roughness in the Manning Formula. Usual values of ''n'' are shown in Table 4. On Figure 8 are shown typical section for the river below 1952 highwater, the assumed ''n'' values together with the corresponding area ''A'' and wetted perimeter ''P'' from which the average velocity ''V'' was determined by the formula $V = \frac{1.486}{n} \cdot r^{2/3} \cdot s^{1/2}$

The discharge was calculated to be 456000 c.f.s. and the discharge calculated by the U. S. G. S. on a slightly different reach of river was 450000 c.f.s. which is reasonable agreement.

By reference to Figure 9 for the Balcones Area, it is noted that the Maximum flood for an area of 947 square miles is 1,010,000 c.f.s. Therefore, the 1952 flood is not the maximum flood but is estimated to be a 500 year flood. By reference to Figure 10 the 50 year flood has a discharge of between 235000 and 265000 c.f.s. and the 100 year flood has a discharge of between 310000 and 350000 c.f.s. Figure 13 shows areas for use with the Texas Board of Water Engineers curves. Figure 14 is a rating curve for the Pedernales River at Johnson City including the 1952 flood. From this curve the elevation of the 50 and 100 year floods or any other flood can be determined for use in designing the bridge as will be demonstrated by other papers at this school.



DRAINAGE AREA MAP FIGURE 1

Scale 1"=1000'

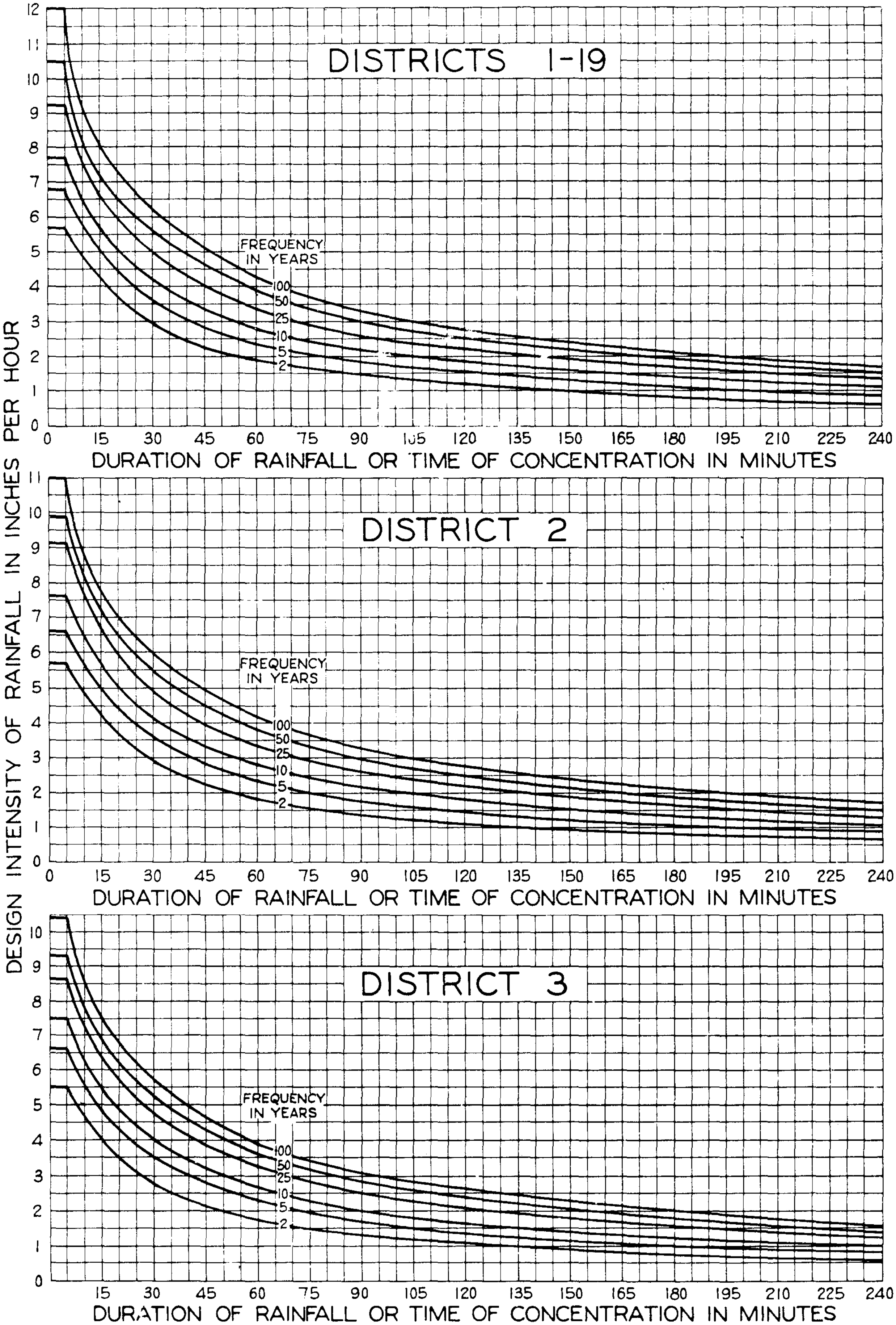

DISTRICTS 1-19
DISTRICT 2
DISTRICT 3
FREQUENCY IN YEARS
100
50
25
10
5
2
DESIGN INTENSITY OF RAINFALL IN INCHES PER HOUR
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240

FIGURE 2

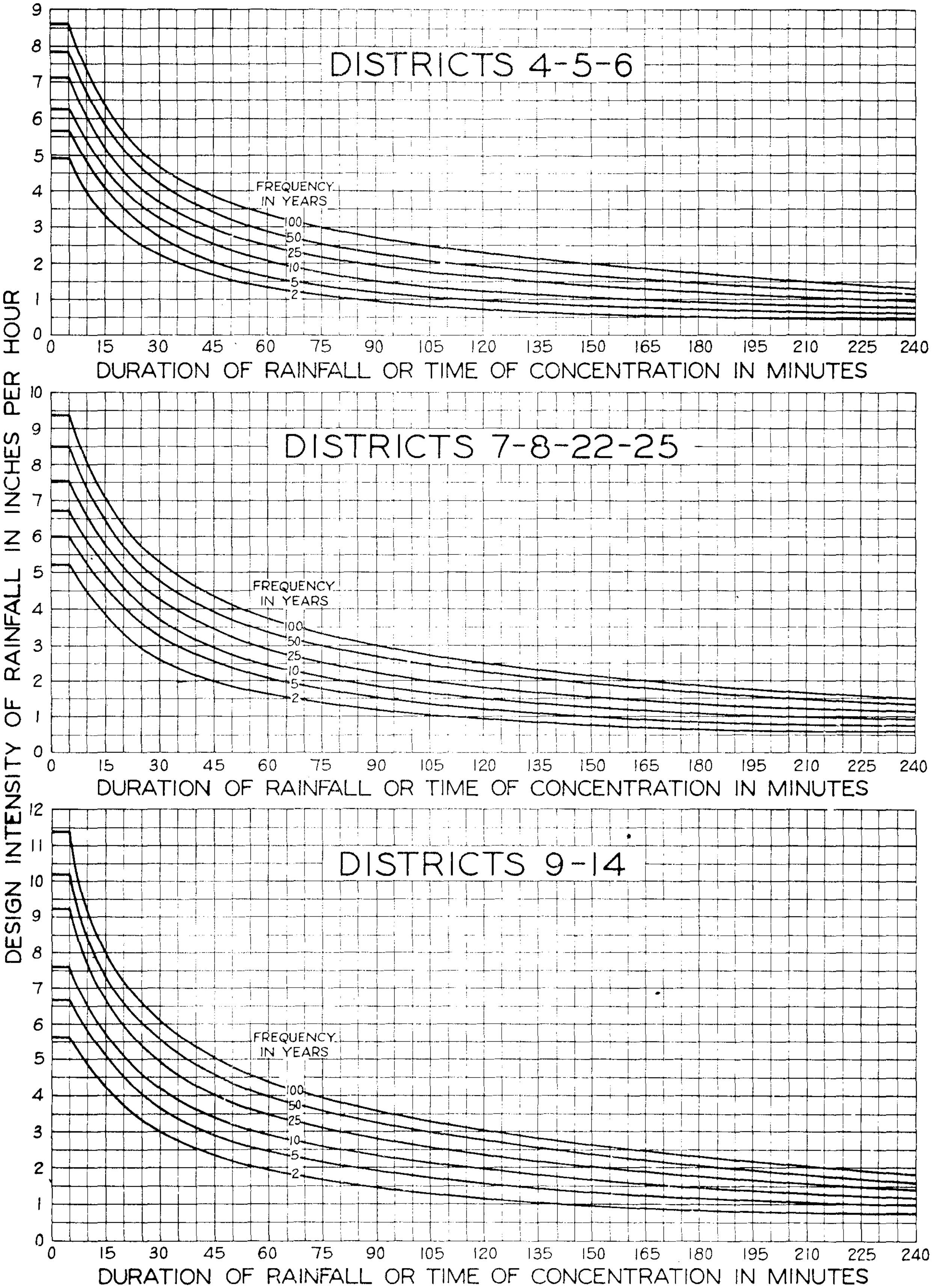

DESIGN INTENSITY OF RAINFALL IN INCHES PER HOUR
DISTRICTS 4-5-6
FREQUENCY IN YEARS
100
50
25
10
5
2
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICTS 7-8-22-25
FREQUENCY IN YEARS
100
50
25
10
5
2
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICTS 9-14
FREQUENCY IN YEARS
100
50
25
10
5
2
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES

FIGURE 3

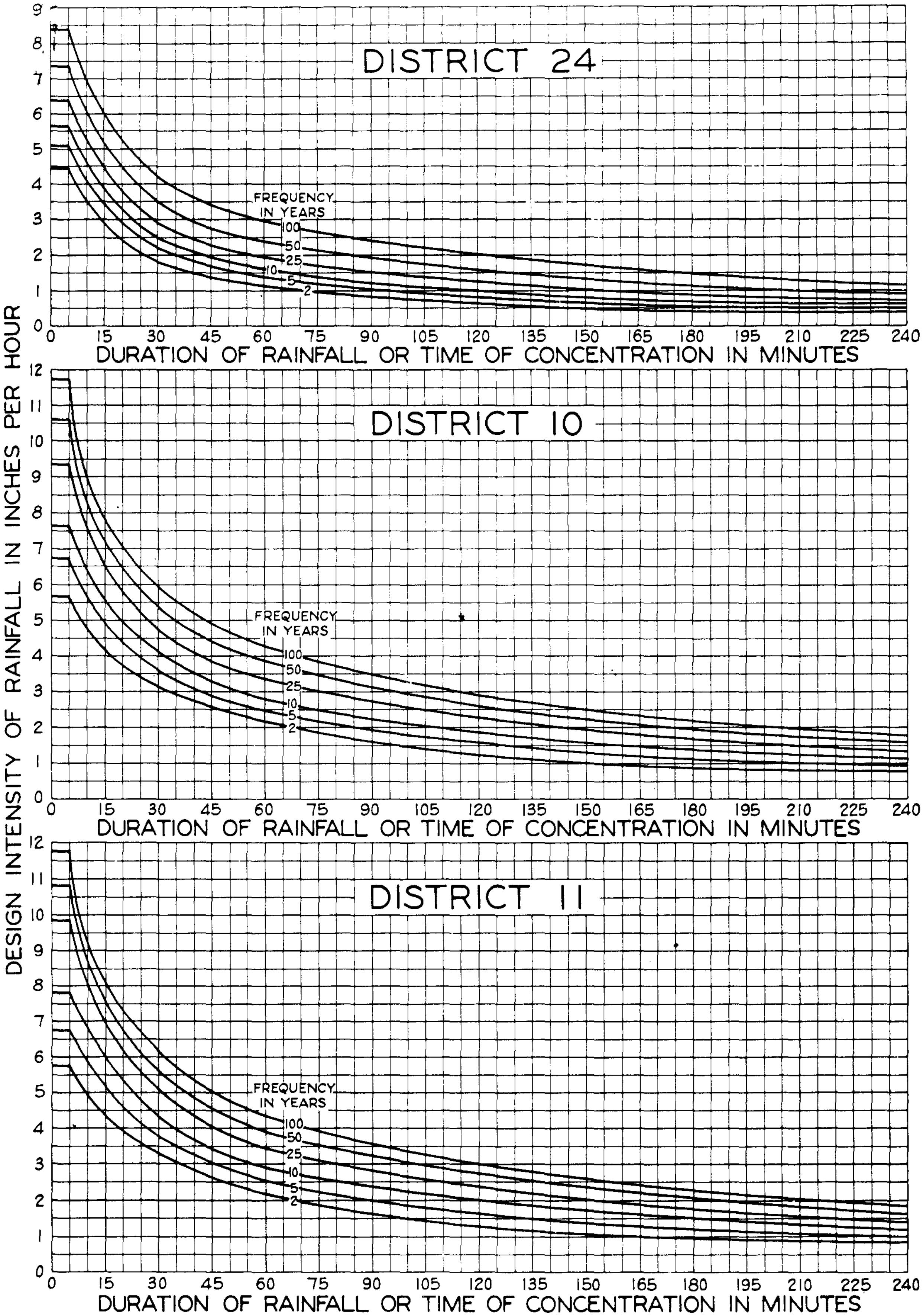
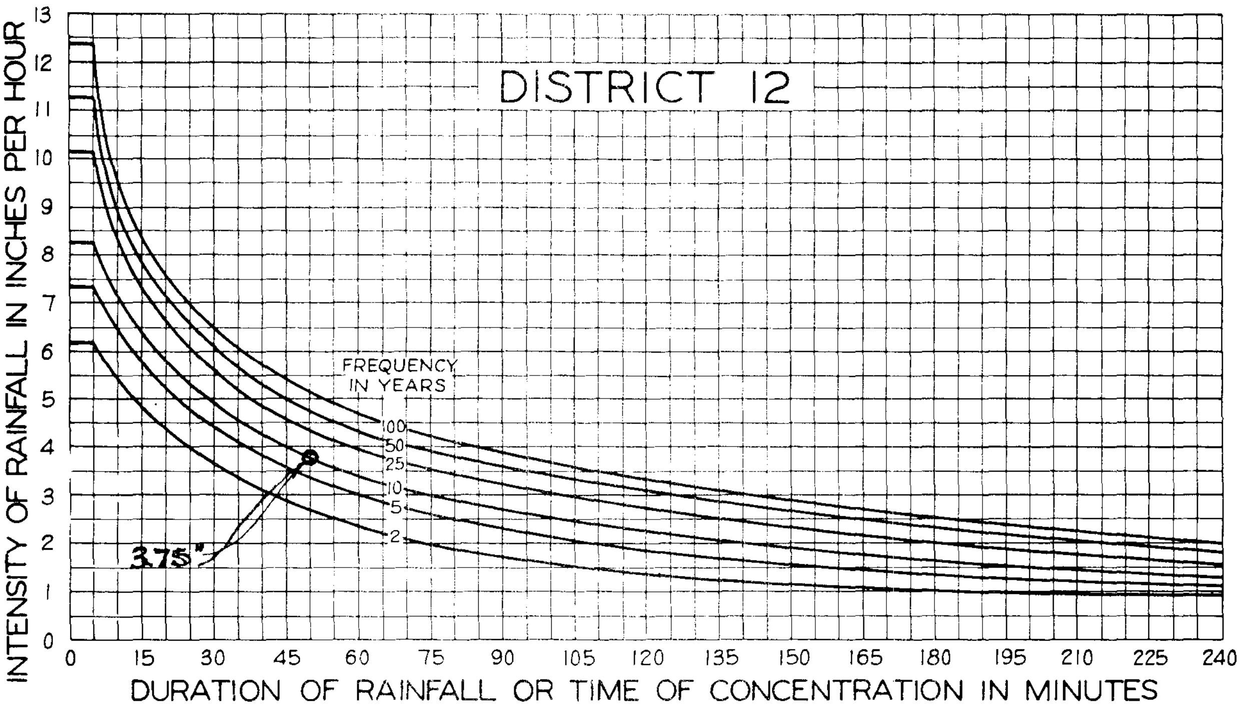

DISTRICT 24
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICT 10
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICT 11
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DESIGN INTENSITY OF RAINFALL IN INCHES PER HOUR

FIGURE 4


DISTRICT 12
FREQUENCY IN YEARS
100
50
25
10
5
2
3.75"
INTENSITY OF RAINFALL IN INCHES PER HOUR
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
0 1 2 3 4 5 6 7 8 9 10 11 12 13
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240

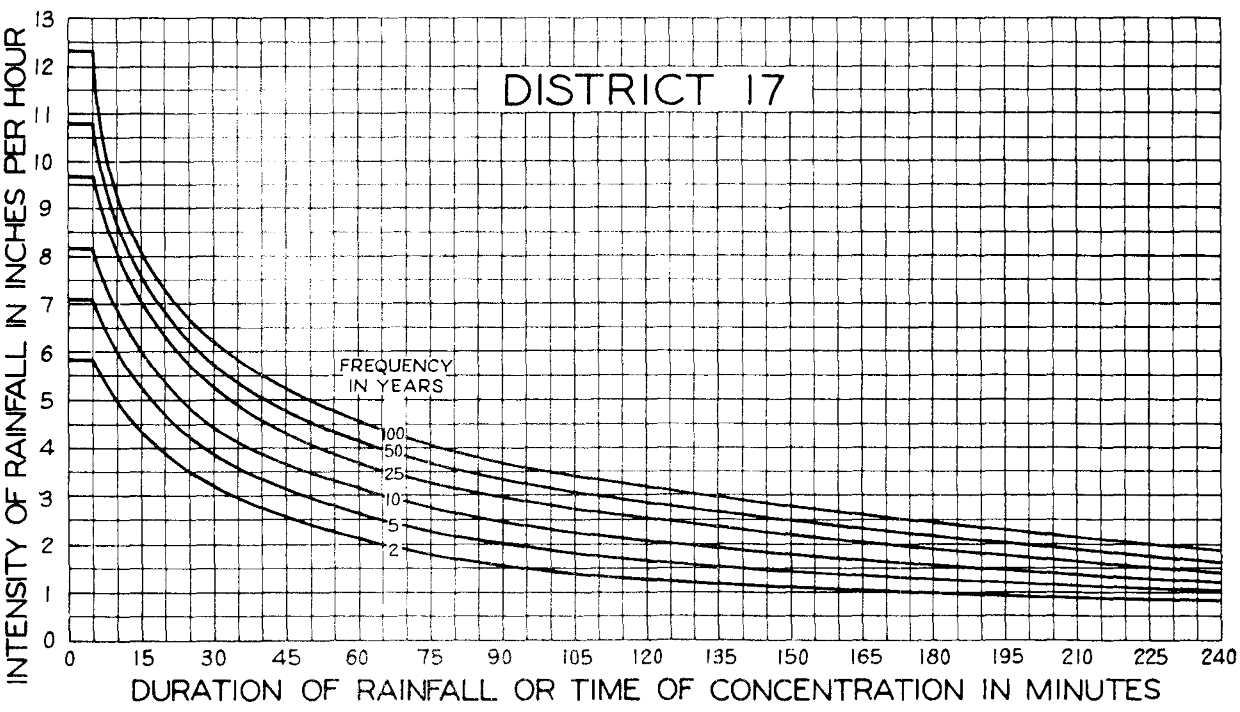

DISTRICT 17
FREQUENCY IN YEARS
100
50
25
10
5
2
INTENSITY OF RAINFALL IN INCHES PER HOUR
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
0 1 2 3 4 5 6 7 8 9 10 11 12 13
0 15 30 45 60 75 90 105 120 135 150 165 180 195 210 225 240

FIGURE 5

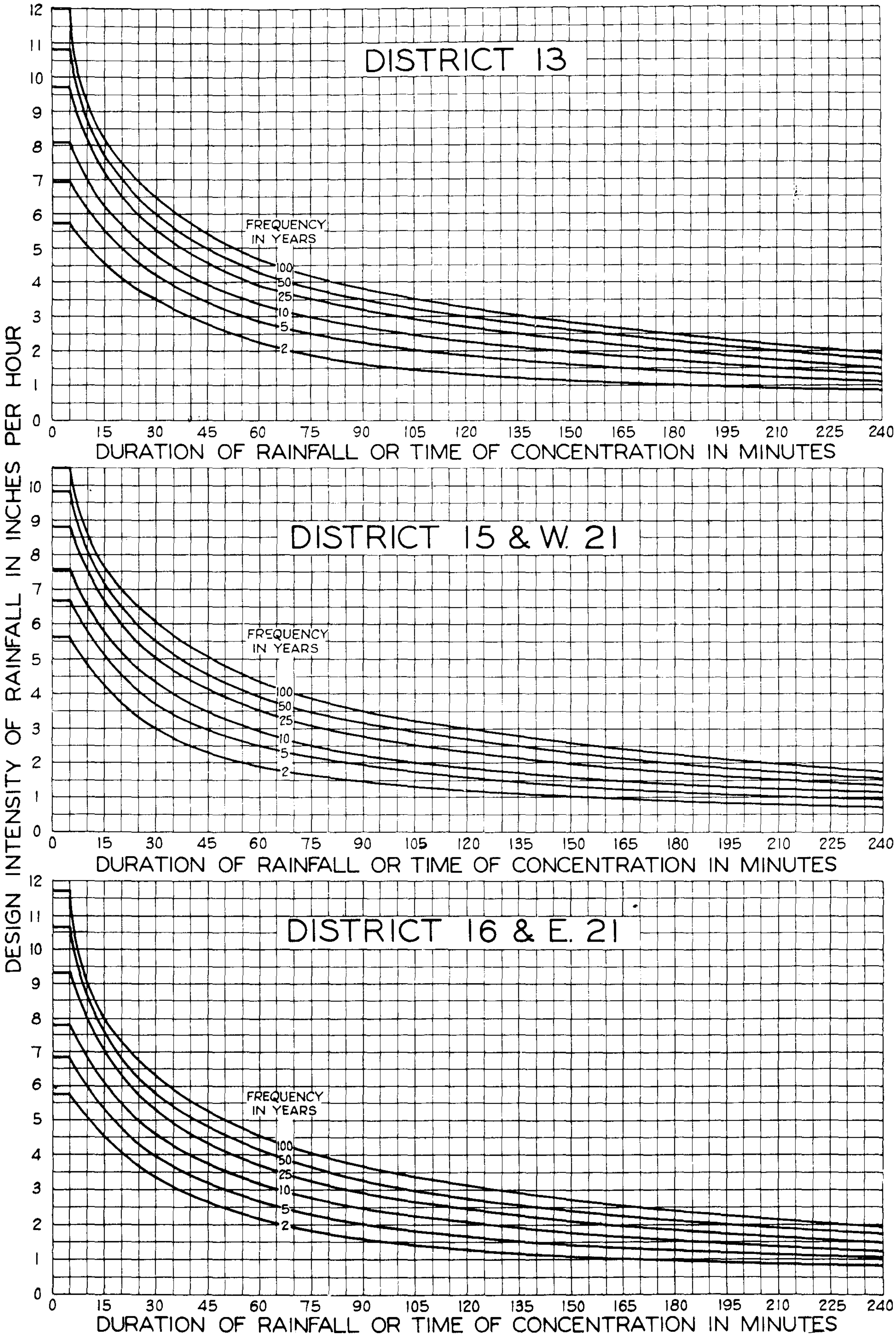

DISTRICT 13
DISTRICT 15 & W. 21
DISTRICT 16 & E. 21
FREQUENCY IN YEARS
100
50
25
10
5
2
DESIGN INTENSITY OF RAINFALL IN INCHES PER HOUR
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
0
15
30
45
60
75
90
105
120
135
150
165
180
195
210
225
240

FIGURE 6

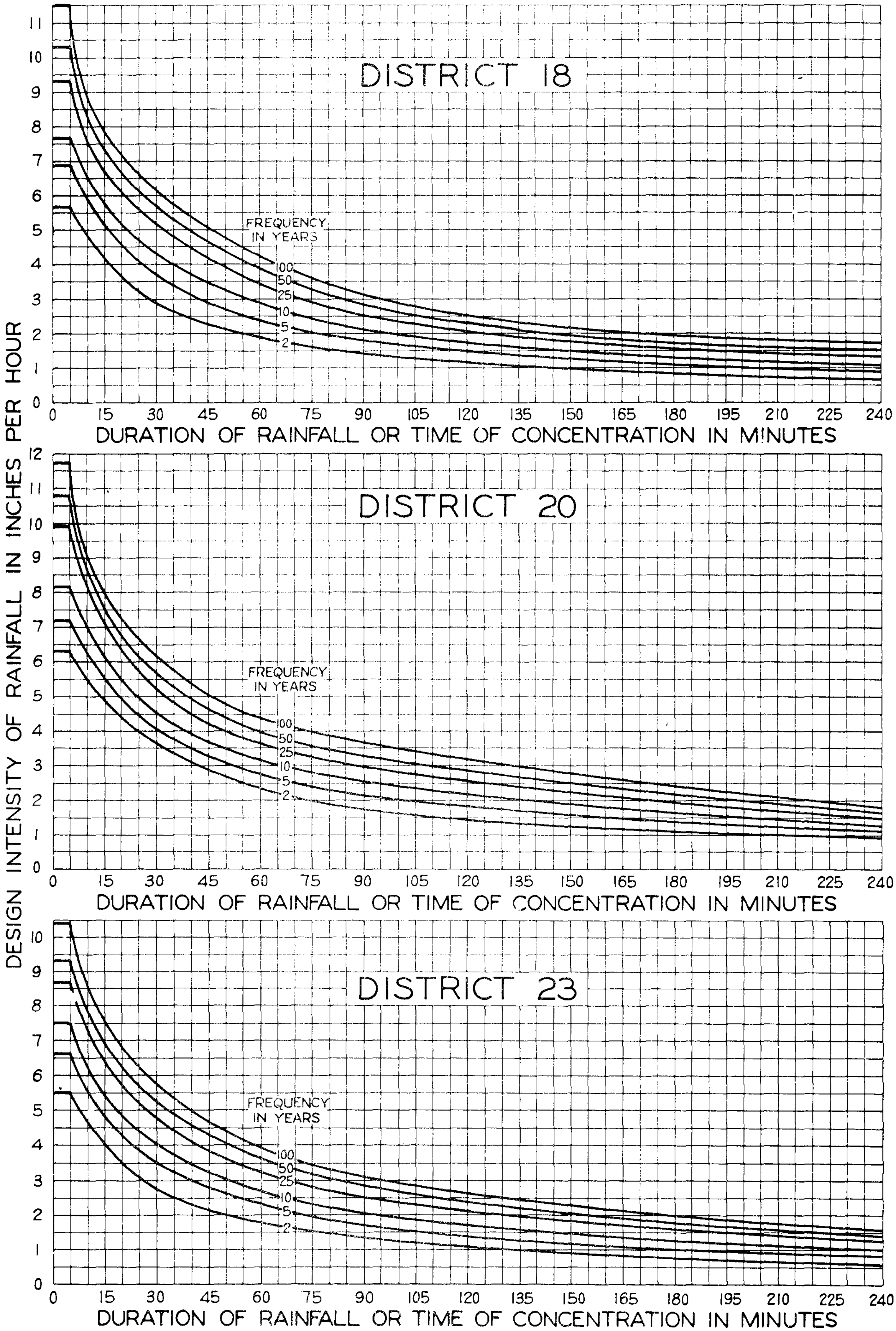

DESIGN INTENSITY OF RAINFALL IN INCHES PER HOUR
DISTRICT 18
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICT 20
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES
DISTRICT 23
FREQUENCY IN YEARS
100
50
25
10
5
2
DURATION OF RAINFALL OR TIME OF CONCENTRATION IN MINUTES

FIGURE 7

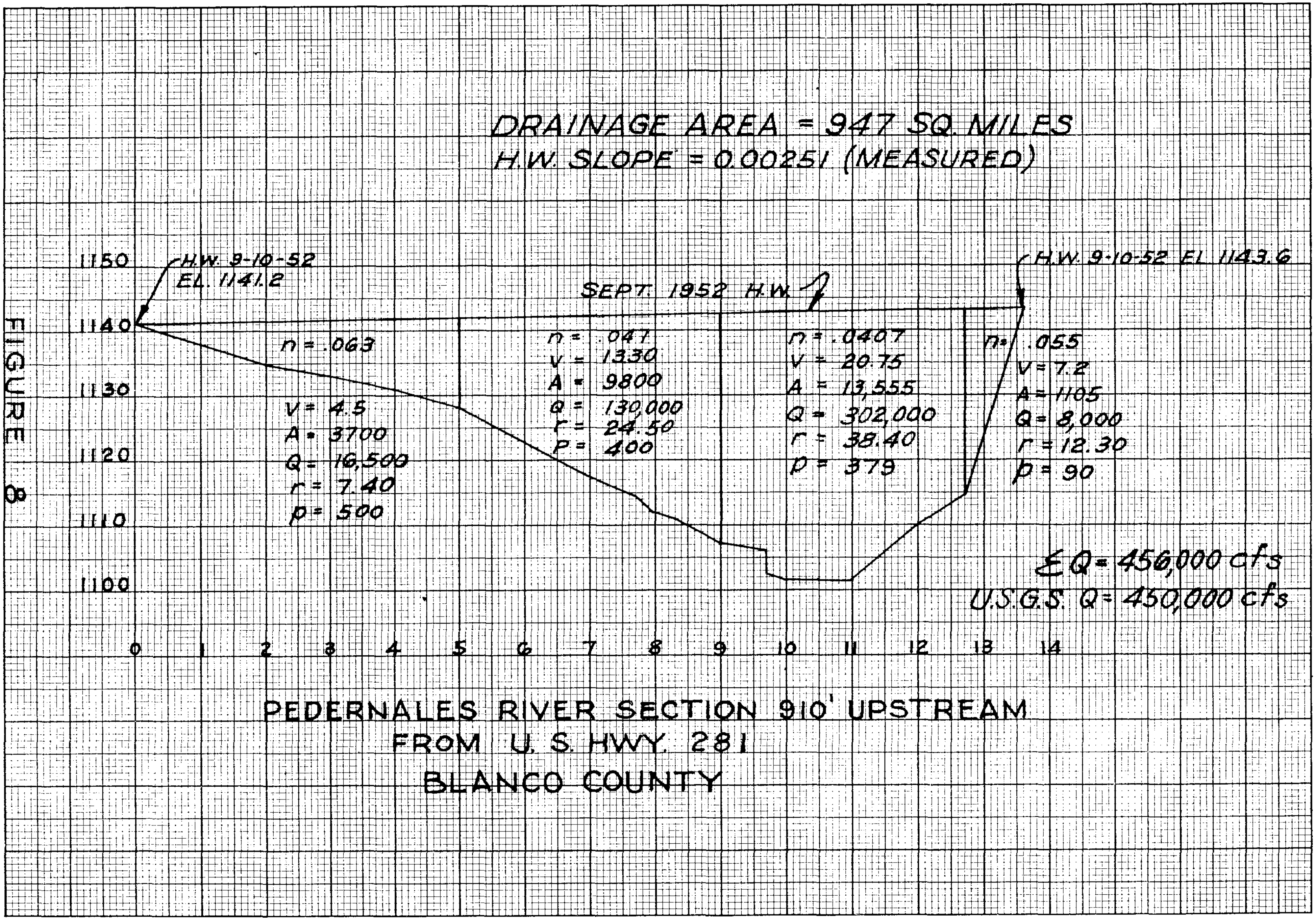

DRAINAGE AREA = 947 SQ. MILES
H.W. SLOPE = 0.00251 (MEASURED)
H.W. 9-10-52
EL. 1141.2
SEPT. 1952 H.W.
H.W. 9-10-52 EL. 1143.6
1150
1140
1130
1120
1110
1100
n = .063
V = 4.5
A = 3700
Q = 16,500
r = 7.40
p = 500
n = .047
V = 13.30
A = 9800
Q = 130,000
r = 24.50
P = 400
n = .0407
V = 20.75
A = 13,555
Q = 302,000
r = 38.40
p = 379
n = .055
V = 7.2
A = 1105
Q = 8,000
r = 12.30
p = 90
ΣQ = 456,000 cfs
U.S.G.S. Q = 450,000 cfs
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
PEDERNALES RIVER SECTION 910' UPSTREAM
FROM U. S. HWY. 281
BLANCO COUNTY

FIGURE 8

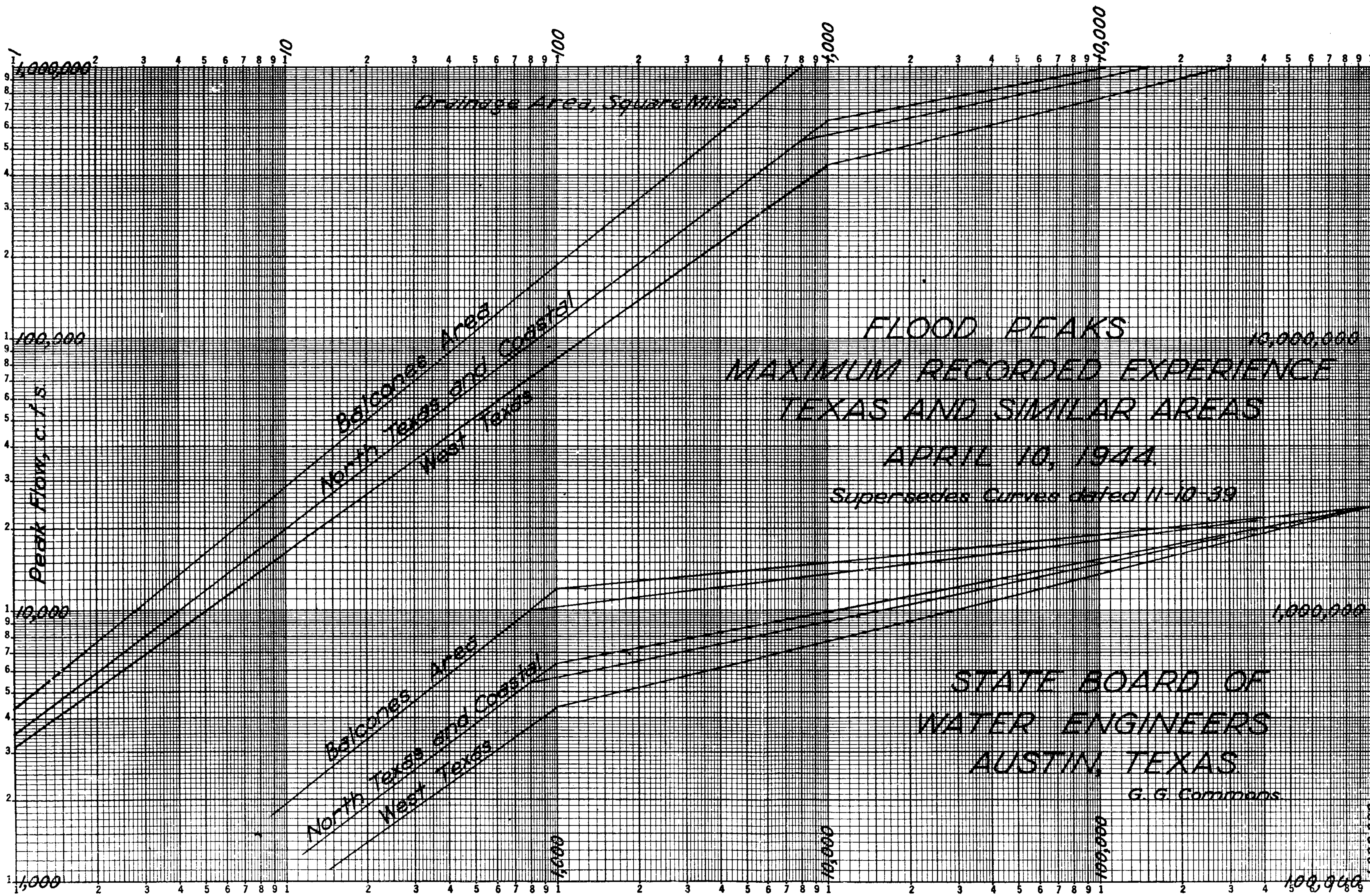

Drainage Area, Square Miles
Peak Flow, c.f.s.
Balcones Area
North Texas and Coastal
West Texas
FLOOD PEAKS
MAXIMUM RECORDED EXPERIENCE
TEXAS AND SIMILAR AREAS
APRIL 10, 1944
Supersedes Curves dated 11-10-39
STATE BOARD OF
WATER ENGINEERS
AUSTIN, TEXAS
G. G. Commons

FIGURE 9

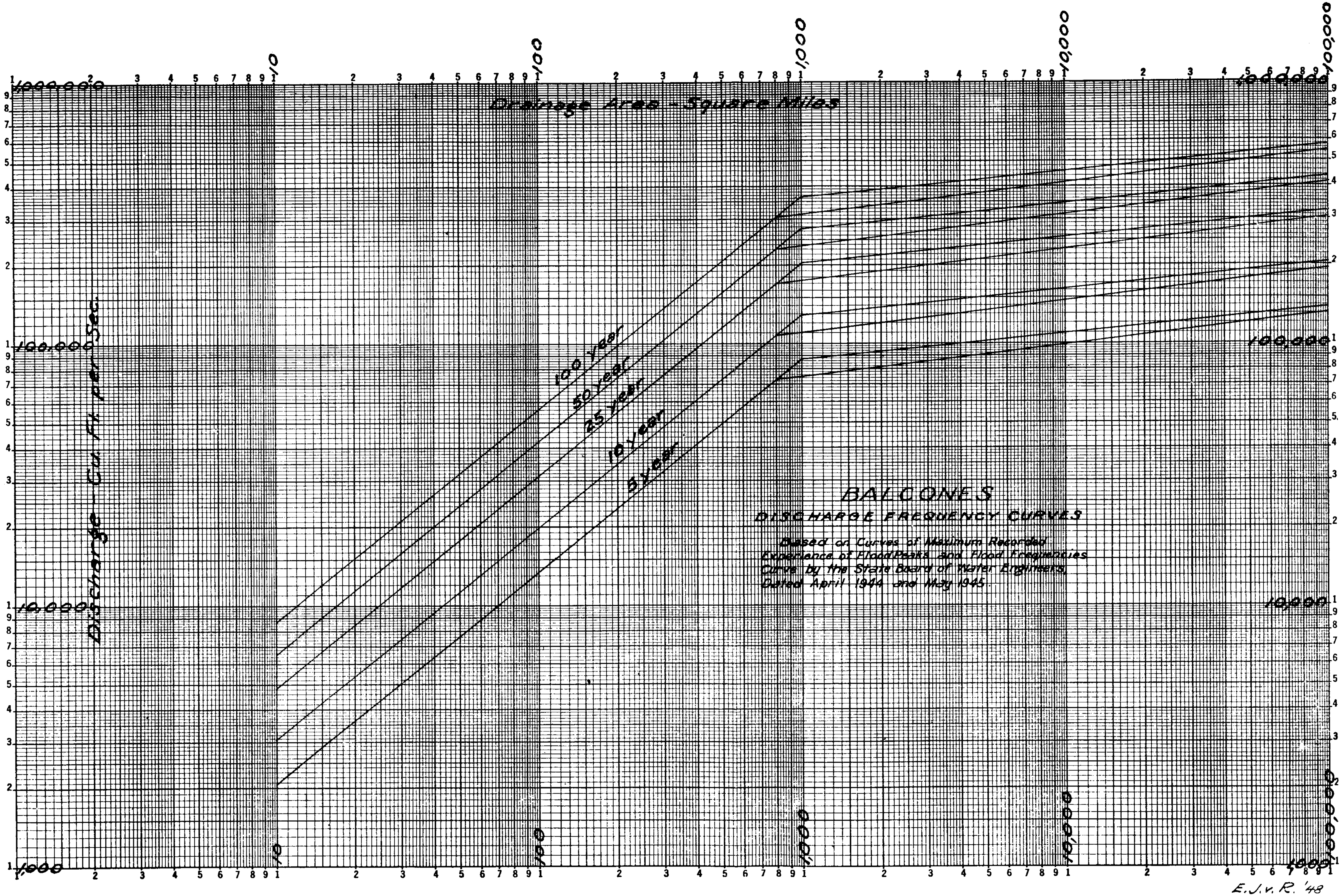

Drainage Area - Square Miles
Discharge - Cu. Ft. per Sec.
100 year
50 year
25 year
10 year
5 year
BALCONES
DISCHARGE FREQUENCY CURVES
Based on Curves of Maximum Recorded
Experience of Flood Peaks and Flood Frequencies
Curve by the State Board of Water Engineers,
Dated April 1944 and May 1945.
E.J.v.R. '48

FIGURE 10

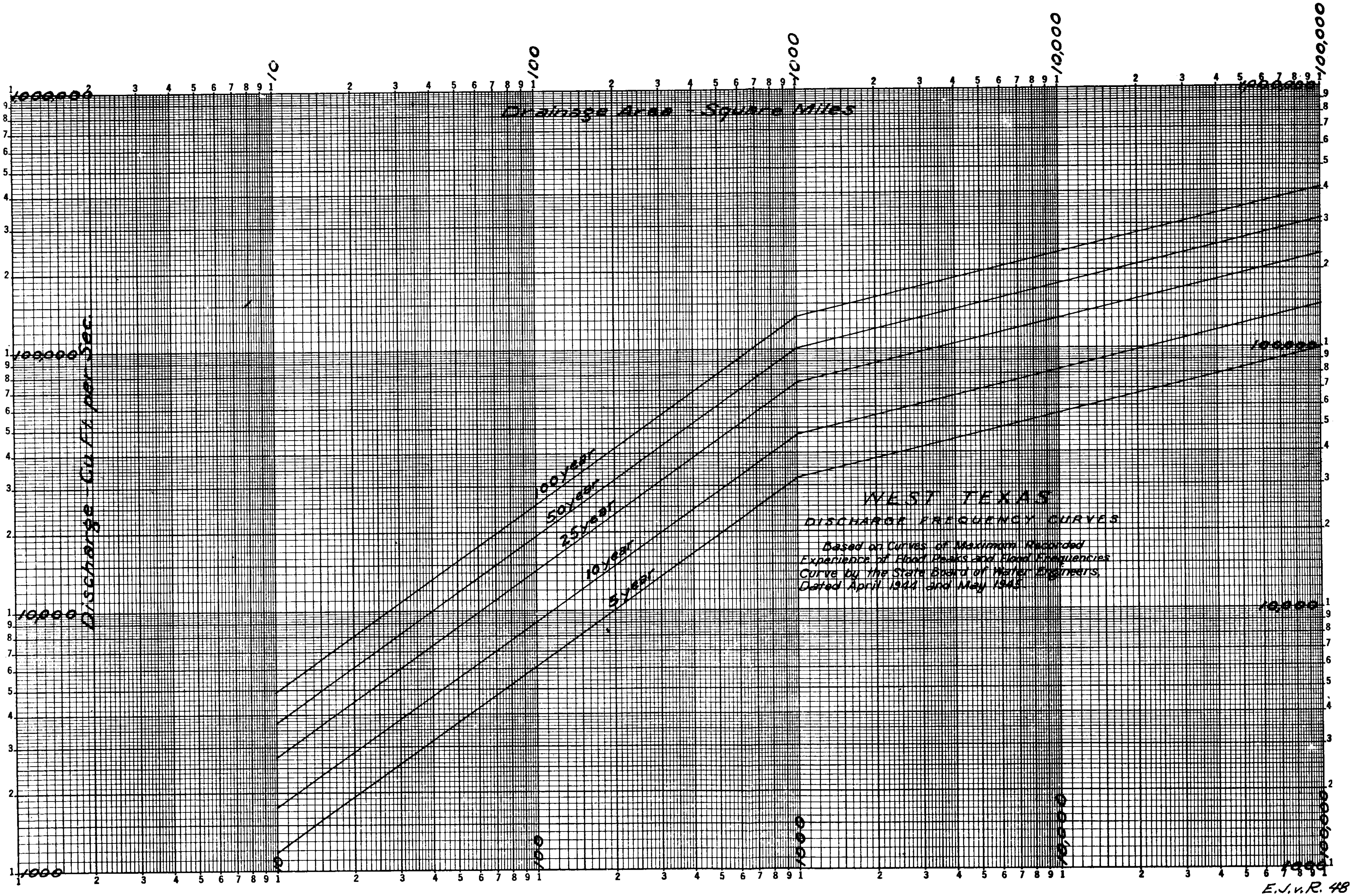

Drainage Area - Square Miles
Discharge - Cu. Ft. per Sec.
WEST TEXAS
DISCHARGE FREQUENCY CURVES
Based on Curves of Maximum Recorded
Experience of Flood Peaks and Flood Frequencies
Curve by the State Board of Water Engineers,
Dated April 1944 and May 1945
100 year
50 year
25 year
10 year
5 year
1,000,000
100,000
10,000
1000
10
100
1000
10,000
100,000
E.J.v.R. 48

FIGURE 11

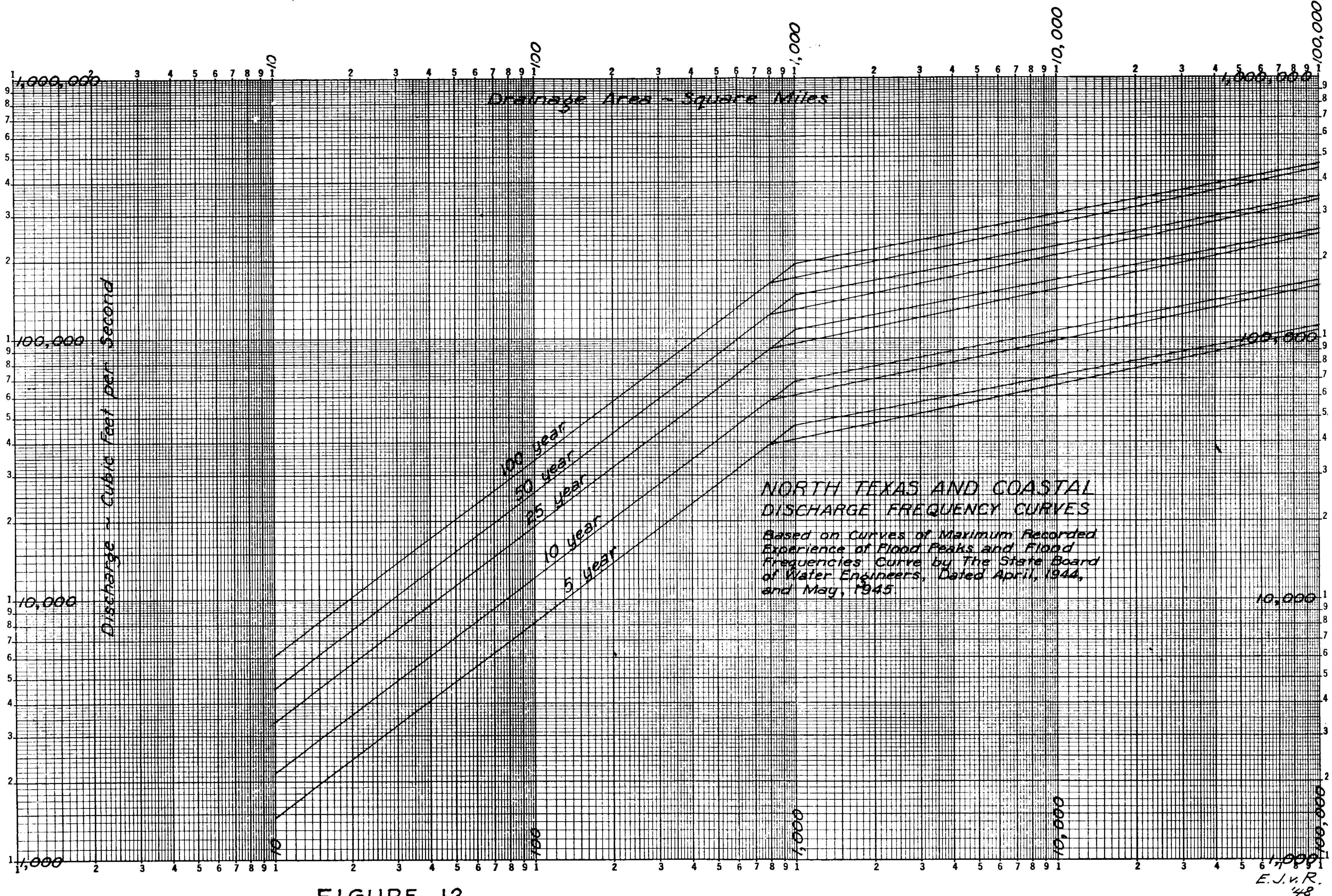

Drainage Area - Square Miles
Discharge - Cubic Feet per Second
1,000,000
100,000
10,000
1,000
10
100
1,000
10,000
100,000
100 year
50 year
25 year
10 year
5 year
NORTH TEXAS AND COASTAL
DISCHARGE FREQUENCY CURVES
Based on Curves of Maximum Recorded Experience of Flood Peaks and Flood Frequencies Curve by The State Board of Water Engineers, Dated April, 1944, and May, 1945.
E.J.v.R. '48

FIGURE 12

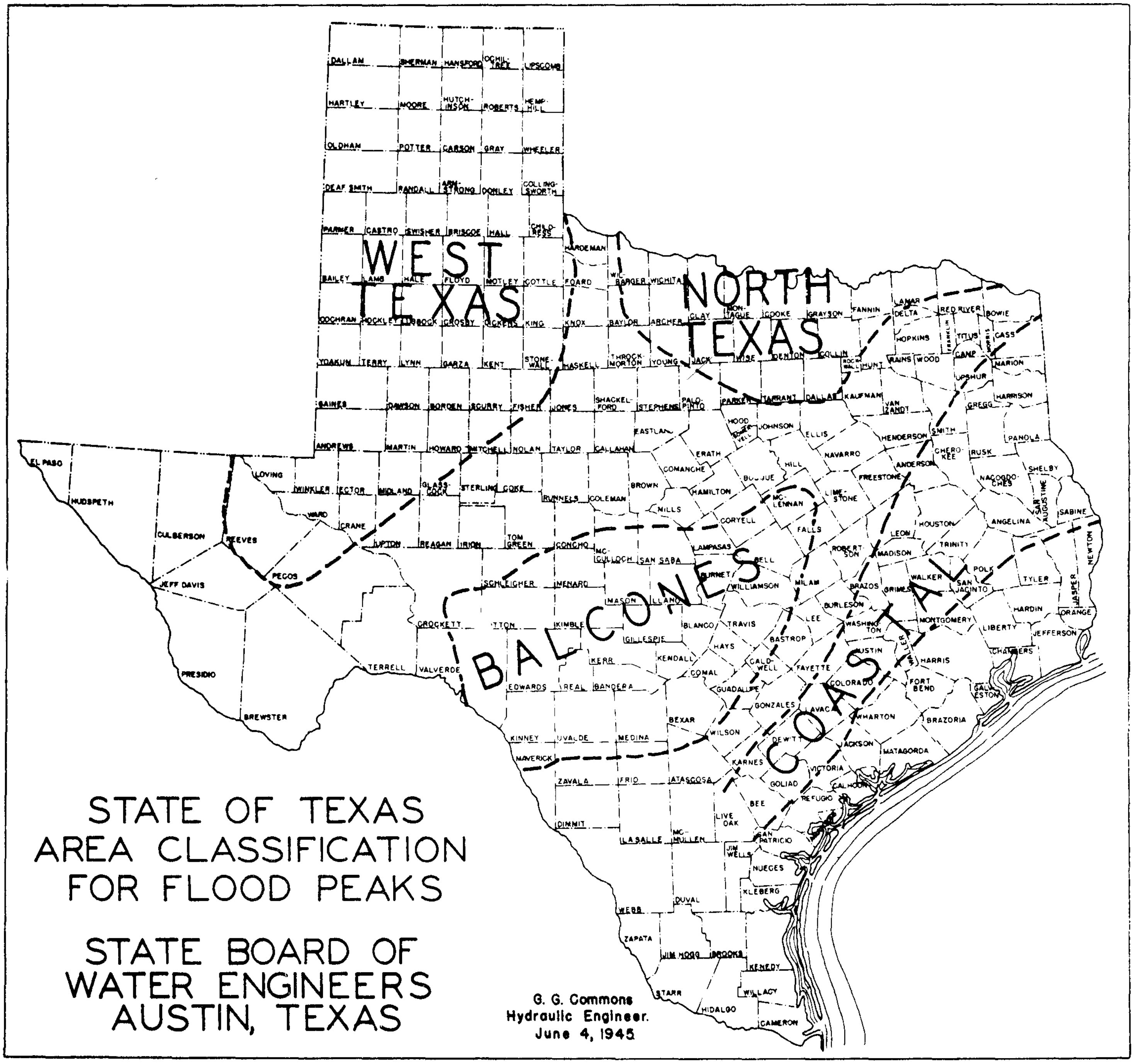

WEST TEXAS
NORTH TEXAS
BALCONES
COASTAL
STATE OF TEXAS
AREA CLASSIFICATION
FOR FLOOD PEAKS
STATE BOARD OF
WATER ENGINEERS
AUSTIN, TEXAS
G. G. Commons
Hydraulic Engineer.
June 4, 1945

FIGURE 13

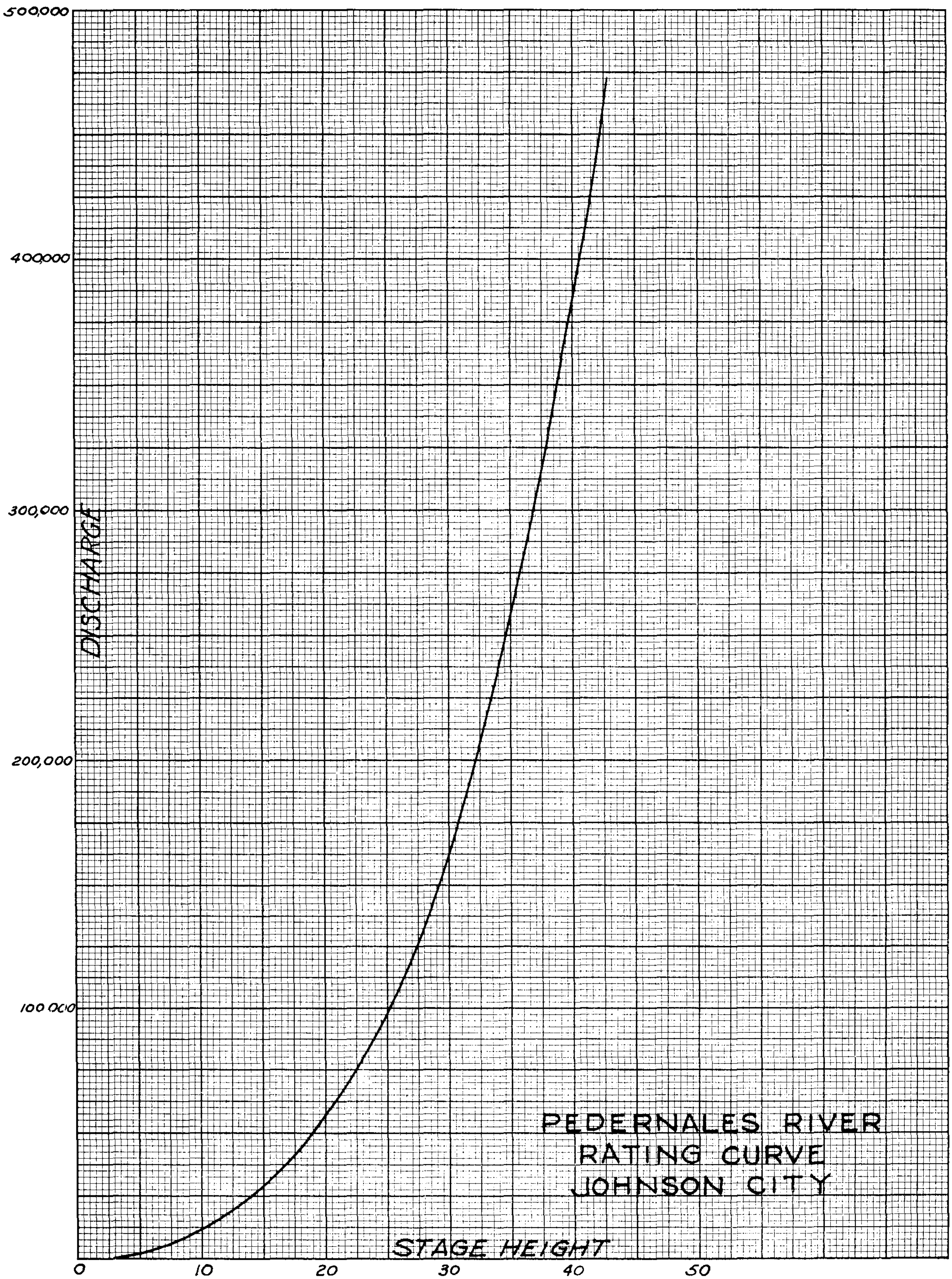

500,000
400,000
300,000
200,000
100,000
DISCHARGE
0
10
20
30
40
50
STAGE HEIGHT
PEDERNALES RIVER
RATING CURVE
JOHNSON CITY

FIGURE 14

TABLE 1

Approximate Average Velocities of Runoff Flow for Calculating Time of Concentration

(Adapted from "Rainfall and Runoff" - Region 2)

Description of Course of Runoff Water	Slope in Per Cent			
	0 - 3	4 - 7	8 - 11	12-15
	Ft/Sec.	Ft/Sec.	Ft/Sec.	Ft/Sec.
Unconcentrated*				
Woodlands	1.0	2.0	3.0	3.5
Pastures	1.5	3.0	4.0	4.5
Cultivated Land (Row Crops)	2.0	4.0	5.0	6.0
Pavements	5.0	12.0	15.5	18.0
Concentrated**				
Vegetative Outlet Channel	Use designed velocities			
Outlet Channel Containing Drop Structures	4.0	5.0	6.0	7.0
Natural Channel Not Well Defined	1.0	3.0	5.0	8.0
Natural Channel Well Defined	Calculate velocities by Manning's Formula			

Note: - Average velocity of flow in variable grade terrace channels may be considered as 1.0 ft. per sec.

* This condition occurs in upper extremity of watershed only.

** These values vary with the channel size and other conditions so that the ones given are the averages of a wide range. Where possible more accurate determinations should be made for particular conditions by the Manning Channel Formula for velocity.

TABLE NO. 2

CONSTANTS FOR USE IN FORMULA $I = \frac{b}{(t+d)^e}$

Based on Texas Reclamation Dept. publication

"EXCESSIVE RAINFALL IN TEXAS"

County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Anderson	0.855	100	20.0	115	22.0	130	26.0	160	29.0
Andrews	0.860	70	13.0	90	16.0	120	20.5	160	27.0
Angelina	0.848	115	23.0	140	27.0	175	31.0	200	36.0
Aransas	0.860	130	29.0	165	34.0	205	40.0	260	48.0
Archer	0.860	100	18.0	110	20.0	130	21.0	155	25.0
Armstrong	0.860	80	15.0	86	16.0	105	16.0	115	19.0
Atascosa	0.890	120	22.0	145	24.0	180	28.0	205	30.0
Austin	0.880	135	22.0	155	26.0	195	30.0	220	32.0
Bailey	0.848	65	13.0	80	15.0	95	16.0	110	17.0
Bandera	0.875	110	22.0	140	26.0	165	30.0	195	36.0
Bastrop	0.890	160	27.5	200	34.5	230	40.0	320	46.0
Baylor	0.840	80	14.0	95	18.0	115	18.0	130	21.0
Bee	0.890	135	27.0	160	30.0	200	35.0	250	40.0
Bell	0.883	160	29.0	190	36.0	250	46.0	325	52.0
Bexar	0.880	110	21.5	140	23.5	170	27.5	190	30.0
Blanco	0.883	120	21.0	150	23.0	185	27.0	220	31.0
Borden	0.860	78	13.0	100	16.0	130	20.0	155	23.0
Bosque	0.880	120	24.0	160	28.0	200	33.0	240	41.0
Bowie	0.827	88	19.0	103	22.0	123	25.5	140	28.0
Brazoria	0.850	140	26.0	170	31.0	215	40.0	250	38.0
Brazos	0.880	135	24.0	160	28.0	195	32.0	235	36.0
Brewster	0.940	105	19.0	120	20.0	145	23.0	175	25.0
Briscoe	0.855	75	14.0	85	14.5	110	16.0	120	18.0
Brooks	0.940	210	32.0	220	36.0	320	44.0	360	48.0
Brown	0.882	110	19.0	130	22.0	155	22.0	170	26.0
Burleson	0.882	145	25.0	180	30.0	205	34.0	255	37.0
Burnet	0.887	135	21.0	160	24.0	195	27.0	255	36.0
Caldwell	0.890	150	25.0	190	30.0	230	35.0	295	44.0
Calhoun	0.850	130	28.0	165	34.0	205	40.5	248	45.0
Callahan	0.875	100	18.0	125	21.0	150	23.0	165	28.0
Cameron	0.960	235	37.0	285	41.0	390	50.0	470	54.0
Camp	0.837	90	19.0	105	22.0	130	25.0	150	28.0
Carson	0.880	85	17.0	90	18.0	110	16.0	120	19.0
Cass	0.836	90	19.5	110	22.0	130	25.0	150	27.0
Castro	0.854	70	14.0	80	15.0	100	16.0	115	20.0
Chambers	0.850	140	28.0	180	33.0	220	39.0	250	40.0
Cherokee	0.850	100	20.0	110	23.0	145	26.0	170	28.0

Page 2, Table 2

County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Childress	0.840	70	13.0	80	14.0	100	14.0	110	16.0
Clay	0.860	100	19.0	120	21.0	140	23.0	170	27.5
Cochran	0.847	65	12.0	80	15.0	100	16.0	115	17.0
Coke	0.875	100	18.0	120	20.0	145	23.0	165	25.0
Coleman	0.880	100	20.0	125	23.0	155	24.0	170	30.0
Collin	0.860	110	21.0	125	24.0	145	28.0	170	30.0
Collingsworth	0.860	80	15.0	90	14.0	100	14.0	115	18.0
Colorado	0.880	130	22.0	150	27.0	170	30.0	200	30.0
Comal	0.880	120	22.0	150	24.0	190	28.0	225	30.0
Comanche	0.881	110	19.0	130	21.5	165	24.0	205	30.0
Concho	0.880	105	20.5	135	24.0	155	28.0	180	33.0
Cooke	0.860	105	21.0	130	26.0	150	29.0	170	32.0
Coryell	0.884	145	24.0	170	28.0	220	34.0	280	42.0
Cottle	0.840	70	12.0	85	14.0	100	14.0	115	16.0
Crane	0.890	85	17.0	112	19.0	140	23.0	175	28.0
Crockett	0.880	95	18.0	115	21.0	145	24.0	175	29.0
Crosby	0.850	75	12.0	95	14.0	110	15.0	130	16.0
Culberson	0.900	80	16.0	100	19.0	120	21.0	140	24.0
Dallam	0.870	75	16.0	88	19.5	100	20.0	120	24.0
Dallas	0.865	110	21.0	133	24.0	150	28.0	180	33.0
Dawson	0.858	73	12.0	95	15.0	120	20.0	155	24.0
Deaf Smith	0.855	70	14.5	80	17.0	100	17.0	125	25.0
Delta	0.840	90	18.0	110	22.0	125	25.5	145	29.0
Denton	0.862	110	22.0	132	26.0	150	29.0	175	33.0
DeWitt	0.885	135	26.0	170	30.0	210	35.0	265	45.0
Dickens	0.850	75	12.0	95	14.0	115	15.0	135	16.0
Dimmit	0.920	145	25.0	180	28.0	225	31.0	280	40.0
Donley	0.860	80	15.0	88	15.0	100	15.0	120	18.0
Duval	0.920	170	28.0	195	31.0	240	39.0	295	42.0
Eastland	0.875	105	18.0	125	21.0	165	24.0	200	31.0
Ector	0.880	80	16.0	105	17.0	140	21.0	170	27.0
Edwards	0.875	105	21.0	135	25.0	155	28.0	180	36.0
Ellis	0.870	110	21.5	135	24.0	165	28.0	195	35.0
El Paso	0.910	67	16.0	80	18.0	96	21.0	120	26.0
Erath	0.880	110	20.0	130	22.0	180	30.0	220	37.0
Falls	0.880	135	26.0	165	31.0	195	38.0	270	48.0
Fannin	0.840	95	20.0	115	23.0	130	26.0	148	29.0
Fayette	0.885	140	25.0	180	32.0	195	36.0	250	38.0
Fisher	0.865	85	14.0	110	18.0	140	20.0	155	22.0
Floyd	0.850	75	12.5	90	14.0	110	16.0	125	16.0
Foard	0.835	70	12.0	83	14.0	100	16.0	120	17.0
Fort Bend	0.865	140	24.0	160	26.0	200	34.0	235	33.0
Franklin	0.835	90	18.5	105	22.0	125	25.0	145	28.5
Freestone	0.860	100	21.0	125	23.0	155	27.5	195	34.0
Frio	0.890	125	23.0	155	25.0	200	29.5	255	34.0
Gaines	0.855	67	12.0	85	15.0	110	18.0	135	26.0
Galveston	0.850	145	28.0	180	34.0	220	40.0	250	40.0

Page 3, Table 2

County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Garza	0.855	75	12.0	97	15.0	120	16.0	145	19.0
Gillespie	0.880	115	21.0	140	24.0	180	28.0	210	31.0
Glasscock	0.870	85	16.0	105	18.5	140	23.0	168	28.0
Goliad	0.885	135	27.0	165	31.0	210	36.0	265	45.5
Gonzales	0.890	140	25.0	175	29.0	220	35.0	280	44.0
Gray	0.880	90	18.0	100	18.0	115	17.0	140	20.0
Grayson	0.850	100	21.0	120	25.0	140	27.0	160	29.5
Gregg	0.840	100	20.0	115	23.5	140	26.0	160	28.0
Grimes	0.880	140	24.0	160	26.0	195	30.0	225	32.0
Guadalupe	0.885	130	23.0	160	26.0	205	31.0	255	36.0
Hale	0.850	70	13.0	90	14.0	105	16.0	123	18.0
Hall	0.850	75	14.0	85	14.0	100	14.0	110	16.0
Hamilton	0.883	125	21.0	145	23.0	185	27.0	240	34.0
Hansford	0.910	110	20.0	130	23.0	150	23.0	170	24.0
Hardeman	0.835	70	12.0	80	14.0	100	15.0	115	16.0
Hardin	0.840	130	27.0	160	31.0	190	35.0	245	42.0
Harris	0.865	140	25.0	165	27.0	210	34.0	235	34.0
Harrison	0.838	100	20.2	120	24.0	140	26.0	160	28.0
Hartley	0.860	70	15.5	87	19.5	100	21.0	120	25.0
Haskell	0.850	80	14.0	105	18.0	120	17.5	130	20.0
Hays	0.880	140	23.0	175	25.0	220	30.0	270	37.0
Hemphill	0.880	95	18.0	115	19.0	125	20.0	150	21.5
Hénderson	0.855	100	20.0	120	22.0	140	26.0	175	30.0
Hidalgo	0.980	255	36.0	300	40.0	370	41.0	430	42.0
Hill	0.875	110	23.0	150	27.0	180	29.0	220	40.0
Hockley	0.848	69	12.0	87	15.0	102	16.0	125	18.0
Hood	0.876	110	21.5	130	23.5	170	30.0	200	34.0
Hopkins	0.840	90	18.5	108	22.0	125	25.5	150	29.0
Houston	0.855	110	22.0	120	23.0	150	26.0	180	28.0
Howard	0.865	80	14.0	105	17.0	135	21.5	163	25.0
Hudspeth	0.920	80	16.0	100	20.0	120	21.0	130	24.0
Hunt	0.850	97	19.5	117	22.0	135	26.5	160	29.5
Hutchinson	0.890	90	19.0	110	21.0	130	19.0	140	21.0
Irion	0.870	100	18.0	120	20.0	148	25.0	180	31.0
Jack	0.867	103	20.0	127	22.0	145	27.0	190	28.5
Jackson	0.850	130	27.0	160	32.0	195	38.0	240	39.0
Jasper	0.838	120	24.0	140	25.5	170	30.0	210	34.0
Jeff Davis	0.905	85	17.5	107	20.0	130	23.0	150	25.5
Jefferson	0.840	135	28.0	170	33.0	205	38.5	250	42.0
Jim Hogg	0.960	225	32.0	245	36.0	325	40.0	360	42.0
Jim Wells	0.900	160	29.0	175	31.5	215	38.0	260	41.0
Johnson	0.873	110	23.0	138	26.0	165	28.0	195	35.0
Jones	0.870	90	16.0	115	19.0	140	20.0	150	22.0
Karnes	0.890	130	25.0	160	27.0	200	32.0	250	37.0
Kaufman	0.860	107	20.0	125	23.5	145	27.0	185	32.0
Kendall	0.880	112	21.0	140	24.0	175	27.5	205	30.0
Kenedy	0.920	205	34.0	215	38.0	300	46.0	370	50.0

Page 4, Table 2

County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Kent	0.860	79	12.0	100	14.5	125	16.0	145	19.0
Kerr	0.875	110	22.0	138	26.0	170	30.0	200	36.0
Kimble	0.875	120	22.0	150	26.0	180	30.0	230	38.0
King	0.850	75	12.0	90	14.0	115	15.5	130	17.5
Kinney	0.880	115	22.0	140	26.0	160	27.0	195	32.0
Kleberg	0.900	170	30.0	180	34.0	230	41.0	290	43.0
Knox	0.840	75	13.0	90	16.0	110	17.0	125	18.0
Lamar	0.830	87	18.0	105	22.0	120	25.0	135	28.0
Lamb	0.850	69	13.0	85	15.0	100	16.0	120	18.0
Lampasas	0.885	130	20.0	145	23.0	180	26.0	230	31.0
LaSalle	0.910	150	25.0	175	26.0	210	32.0	300	35.0
Lavaca	0.880	130	26.0	160	29.5	195	35.0	230	38.0
Lee	0.880	150	27.0	195	34.0	210	38.0	290	40.0
Leon	0.860	110	22.0	125	24.0	160	27.0	200	32.0
Liberty	0.850	140	27.0	175	30.0	210	35.0	240	38.0
Limestone	0.870	110	22.0	140	26.0	170	31.0	220	42.0
Lipscomb	0.900	105	20.0	125	21.0	140	23.0	160	24.0
Live Oak	0.895	135	26.0	155	27.0	195	32.0	235	35.0
Llano	0.883	120	20.0	145	23.5	175	26.5	210	30.0
Loving	0.880	77	15.5	95	17.0	125	20.0	140	24.0
Lubbock	0.850	72	12.0	92	14.5	105	16.0	127	17.0
Lynn	0.855	72	12.0	92	15.0	110	16.0	140	20.0
Madison	0.870	125	23.0	135	25.0	170	28.0	205	32.0
Marion	0.835	95	20.0	115	22.5	135	26.0	155	28.0
Martin	0.863	75	14.0	100	16.0	132	21.0	165	26.0
Mason	0.880	120	21.5	150	25.0	180	28.0	210	34.0
Matagorda	0.840	130	26.0	165	32.0	210	40.0	245	40.0
Maverick	0.910	130	24.5	155	27.0	190	29.0	220	35.0
McCulloch	0.880	107	20.5	135	24.5	160	27.0	185	32.5
McLennan	0.880	130	26.0	170	31.0	195	36.0	270	46.0
McMullen	0.900	140	25.0	160	24.0	190	31.0	250	32.0
Medina	0.880	115	22.0	145	25.0	180	29.0	220	34.0
Menard	0.880	115	21.5	145	26.0	175	29.5	205	36.0
Midland	0.875	80	16.0	105	18.0	140	22.0	170	29.0
Milam	0.883	145	27.0	185	35.0	210	40.0	290	46.0
Mills	0.883	115	19.0	135	22.0	158	22.0	185	24.0
Mitchell	0.870	85	15.0	110	18.0	140	21.5	162	24.5
Montague	0.860	105	20.5	130	24.0	160	27.0	185	30.0
Montgomery	0.875	140	25.0	165	27.0	205	30.0	225	31.0
Moore	0.880	80	17.0	100	21.0	115	19.0	145	20.0
Morris	0.837	90	19.0	105	22.0	130	25.0	145	28.0
Motley	0.845	70	12.0	90	14.0	110	14.0	120	16.0
Nacogdoches	0.843	110	22.0	135	26.0	165	30.0	190	33.0
Navarro	0.865	110	21.0	130	23.0	165	28.5	200	36.0
Newton	0.835	110	23.0	130	24.0	155	28.0	200	32.0
Nolan	0.870	90	17.0	115	19.0	145	21.0	163	24.0
Nueces	0.890	140	29.0	167	33.0	207	38.0	255	43.0

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County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Ochiltree	0.910	110	20.0	130	22.0	150	23.0	170	24.0
Oldham	0.860	70	15.0	85	18.5	100	21.0	130	26.0
Orange	0.835	125	25.5	155	30.0	190	35.0	230	38.0
Palo Pinto	0.870	100	19.0	125	21.5	155	30.0	195	32.0
Panola	0.840	103	21.0	125	25.0	147	27.0	175	29.0
Parker	0.870	105	21.0	130	23.0	155	30.0	190	31.0
Parmer	0.850	68	13.8	78	16.0	95	16.0	115	21.0
Pecos	0.910	95	18.0	120	20.0	140	24.0	175	27.5
Polk	0.850	125	25.0	155	28.0	185	32.0	220	36.0
Potter	0.870	77	16.0	90	19.0	105	16.0	125	20.0
Presidio	0.920	90	18.0	110	20.0	130	22.0	160	24.5
Rains	0.850	98	19.0	115	22.0	135	26.0	160	29.5
Randall	0.860	73	15.0	85	16.0	105	16.0	115	21.0
Reagan	0.877	90	17.0	110	19.5	145	24.5	180	32.0
Real	0.875	110	22.5	137	26.5	160	30.0	190	38.0
Red River	0.830	85	18.5	100	22.0	115	25.5	135	29.0
Reeves	0.900	85	17.0	105	19.0	135	22.0	160	26.5
Refugio	0.870	132	28.0	165	33.0	205	38.0	260	47.0
Roberts	0.900	105	19.0	120	20.0	135	20.0	155	21.5
Robertson	0.875	130	22.5	150	28.0	185	33.0	240	41.0
Rockwall	0.860	110	20.0	125	23.5	145	27.5	180	31.0
Runnels	0.880	100	19.5	125	21.5	150	24.0	168	29.0
Rusk	0.843	105	20.5	120	24.0	150	27.0	175	29.0
Sabine	0.837	105	21.5	120	23.5	145	26.0	185	30.0
San Augustine	0.840	110	23.0	135	25.0	160	29.0	195	33.0
San Jacinto	0.860	135	25.0	155	27.0	190	29.0	220	32.0
San Patricio	0.880	135	28.5	165	32.0	205	37.0	255	45.0
San Saba	0.880	115	20.0	137	23.0	165	25.0	190	26.0
Schleicher	0.870	105	20.0	140	23.0	165	29.0	200	33.0
Scurry	0.860	83	13.0	105	16.0	135	20.0	155	22.0
Shakelford	0.865	100	16.5	120	20.0	140	21.0	160	24.0
Shelby	0.840	105	21.5	130	25.0	150	27.0	180	30.0
Sherman	0.890	90	20.0	110	21.5	135	21.0	150	22.0
Smith	0.848	97	19.5	110	22.0	135	26.0	165	28.0
Somervell	0.878	115	22.0	135	24.0	180	30.0	210	37.0
Starr	0.980	255	33.5	300	37.5	360	38.0	390	38.0
Stephens	0.870	95	17.5	123	21.0	150	24.0	190	30.0
Sterling	0.870	90	17.0	110	19.0	143	23.0	168	26.0
Stonewall	0.860	80	13.0	105	16.0	125	17.0	140	19.0
Sutton	0.870	105	20.0	140	24.0	170	29.0	215	36.0
Swisher	0.855	72	14.0	85	15.0	105	16.0	120	19.0
Tarrant	0.870	110	22.0	135	27.0	160	28.0	180	33.0
Taylor	0.875	95	18.5	120	20.5	145	22.0	162	24.0
Terrell	0.910	100	18.0	120	20.0	142	23.5	165	25.0
Terry	0.850	70	12.0	87	15.0	105	16.0	130	20.0
Throckmorton	0.850	90	15.5	110	19.5	130	20.0	150	23.0
Titus	0.833	90	19.0	105	22.0	125	25.0	145	28.0

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County	e	5 year		10 year		25 year		50 year	
		b	d	b	d	b	d	b	d
Tom Green	0.875	105	19.0	130	22.0	150	26.0	175	30.0
Travis	0.890	155	25.0	200	31.0	250	35.0	300	44.0
Trinity	0.855	120	24.0	135	25.0	170	28.0	195	31.0
Tyler	0.840	120	25.0	150	28.0	185	33.0	230	38.0
Upshur	0.840	95	19.0	110	22.0	135	25.0	155	28.0
Upton	0.880	85	17.0	112	19.0	145	24.0	175	30.0
Uvalde	0.880	115	23.0	145	26.5	185	30.5	235	38.0
Val Verde	0.885	100	19.0	120	22.0	145	24.0	165	26.0
Van Zandt	0.850	98	19.0	117	22.0	135	26.0	170	30.0
Victoria	0.860	130	27.5	160	33.0	200	39.0	250	42.0
Walker	0.870	130	24.0	145	26.0	180	27.0	210	30.0
Waller	0.880	140	23.0	160	25.0	200	30.0	230	29.0
Ward	0.890	85	17.0	110	18.5	135	22.0	165	26.0
Washington	0.885	140	24.0	170	28.0	200	32.0	240	34.5
Webb	0.930	170	27.0	215	32.0	255	36.0	300	39.0
Wharton	0.860	130	23.0	155	28.0	185	34.0	215	33.0
Wheeler	0.870	90	17.0	100	17.0	105	17.0	135	20.0
Wichita	0.840	90	17.0	105	19.0	120	20.5	140	23.0
Wilbarger	0.835	75	13.5	90	16.0	105	17.0	125	20.0
Willacy	0.950	230	36.0	270	40.5	370	50.0	440	54.0
Williamson	0.890	160	28.0	210	36.0	280	44.0	340	56.0
Wilson	0.885	125	23.0	155	25.0	195	30.0	240	34.0
Winkler	0.880	75	16.0	100	17.0	130	21.0	165	25.0
Wise	0.867	108	21.0	130	25.0	160	30.0	185	31.0
Wood	0.843	95	19.0	110	22.0	130	25.0	155	28.5
Yoakum	0.850	65	12.0	85	14.5	103	16.0	120	20.0
Young	0.863	100	18.0	120	20.5	140	22.0	180	27.0
Zapata	0.970	210	30.0	270	36.0	320	37.5	360	38.0
Zavalla	0.900	130	24.0	160	27.0	210	31.0	260	38.0

TABLE 3

VALUES OF C (RUN-OFF COEFFICIENT) IN FORMULA Q = CIA

SLOPE	LAND USE	CLASSIFICATION OF SOIL					
		Rolling Plains		Sand or Sandy Loam Soils (Pervious)		Black or Loessial Soils (Impervious)	
		Min.	Max.	Min.	Max.	Min.	Max.
	Timber			0.15	0.20	0.15	0.20
Flat (0% - 1%)	Pasture			0.20	0.25	0.25	0.30
	Cultivated			0.25	0.35	0.30	0.40
	Timber			0.15	0.20	0.18	0.25
Rolling (1% - 3.5%)	Pastures	0.25	0.30	0.30	0.40	0.35	0.45
	Cultivated	0.40	0.45	0.45	0.65	0.50	0.70
	Timber			0.20	0.25	0.25	0.30
Hilly (3.5% - 5.5%)	Pasture			0.35	0.45	0.45	0.55
	Cultivated			0.60	0.75	0.70	0.85
	Timber					0.70	0.80
Mountainous (5.5% +)	Bare					0.80	0.90

TABLE 4

Values of n for Manning's Formula
as Applied to Natural Streams

PRIMARY CHANNEL	Straight Alinement	Tortuous Alinement
Smooth banks, little or no vegetation	.030	.035
Rough, irregular banks and bed, little or no vegetation	.035	.040
Light brush or scattered trees on banks	.040	.045
Heavy brush and trees on banks	.045 to .050	.050 to .055
Very dense brush and trees on banks	.050 to .055	.055 to .060
Very rough, densely vegetated banks, large boulders or trees in bed	.060 to .070	.070 to .080

Note: With the exception of the last case, above values are based on the condition of little or no vegetation in bed of stream. When such condition is encountered, or when trees on banks overhang stream and obstruct flow in major portion of channel, values of n should be increased.

FLOOD PLAINS	Smooth Surface	Rough, Irregular Surface
Bare soil or grass sod, no high weeds	.030	.035
Scattering brush	.035	.040
Medium brush, scattering trees	.040 to .050	.050 to .060
Thick trees, little or no undergrowth	.060 to .070	.070 to .080
Heavy brush or heavy brush and trees	.080 to .090	.090 to .100
Very dense brush and trees	.100 to .130	.130 to .150

Note: Large weeds which will not flatten under flood flow may be considered the same as brush.

Special Exception: For streams with beds of deep loose sand, where the sand flows and conforms to the stream flow, such as in the Canadian River, the value of n is considerably lower than shown above. Values of the order of .020 should be used in such instances.

General Note: The above values of n are based on tests conducted by the U. S. Department of Agriculture and the U. S. Geological Survey and are believed to represent the best information presently available. Due to the limited amount of experimental work of this nature which has been performed and to the difficulty of describing and classifying the degrees of roughness, these values must be treated as approximate only.

HYDRAULIC REQUIREMENTS OF SMALL STRUCTURES, MULTIPLE BOX CULVERTS AND SELECTION OF CULVERT SIZES

1. Hydraulic Requirements for Structure Size.

Now that the runoff from rainfall has been determined, the next step is to select a structure of proper size and design to carry the discharge beneath the highway satisfactorily. The function of a highway drainage structure is to carry the water from the upstream side of the road to the downstream side without causing excessive backwater head and without creating excessive velocities. The designer should keep the losses of head and velocities within safe limits and select the structure of minimum cost that will perform as required to meet design needs as to appearance, strength and performance.

The size and type of structure selected for a culvert site will be influenced by the location characteristics, the relation of the grade line of the highway to the flow line of the channel, tailwater elevation, discharge velocity, backwater elevation and losses of hydraulic head through the structure.

The cost of maintaining highways in good condition is directly related to the adequacy of the means provided for drainage. Storm water for which adequate provision is not made may cause severe erosion of embankment slopes, shoulders and stream channels, may undermine culvert outlets and may cause base and pavement failure. Good drainage design depends on anticipating where the runoff will occur, in what amount and at what frequency and on making provision for removal of excess water as rapidly as is necessary to avoid undue interference with operation of vehicular traffic or excessive cost for maintenance.

A culvert or similar conduit of intermediate length is in a category between weirs and short tubes at one extreme and long pipe lines at the other. When water discharges freely over a weir or through a short tube, the discharge under low head depends primarily on the geometry of the cross section of flow and the elevation of the headwater pool. So-called frictional resistances are neglected, and a consistent pattern of the hydraulics phenomena can be developed.

At the other extreme, flow in a long conduit, where water is flowing under substantially steady, uniform conditions, can be fitted into a pattern in which the head loss between two sections is primarily a function of the geometry of the conduit, the surface resistance and the rate of flow.

Between these two extremes lies a very important group of conduits of intermediate length in which neither the entrance and outlet conditions nor the surface resistances may be neglected. In other words, the hydraulic problems of conduits of intermediate length afford interesting challenges to the experimenter and the analyst.

At least nine variables must be considered and controlled in the comprehensive study of conduits of intermediate length in addition to the usual assumptions of the relationship expressed in Manning's formula or some other hydraulic flow formula. These variables are as follows:

a. Culvert material. This is reflected in the roughness coefficient in Manning's formula which may vary from 0.013 for smooth concrete or metal surfaces to 0.022 or more for corrugated material surfaces.

b. Diameter of conduit or its geometrical proportions if it is not circular in section.

c. Length of the conduit.

d. Slope of the conduit invert.

e. Entrance conditions. The shape of head wall and the character of the entrance to the conduit has important bearing on the capacity of the structure.

f. Headwater elevation above the invert.

g. Tailwater elevation referred to the invert at the outlet and also to the headwater pool level.

h. Outlet conditions. Whether operating as a free outlet or a submerged or partially submerged outlet.

i. Rate of flow through the conduit.

The design frequency to be selected in preparing hydraulic designs for drainage structures is based principally on the importance of the highway route from a traffic volume standpoint. Other factors affecting the establishment of the proper design frequency are initial cost and cost of maintaining the drainage installation.

As a general guide in determining the proper design flood frequency, it is considered proper that for structures of culvert classification located on the two lower classes of farm roads, a two year flood frequency design is the minimum desirable. For the highest class of farm road and less important highway routes, a five year frequency is suggested. In providing designs for medium and important classes of roads on the Primary System, a 10 year flood frequency is usually followed. For major bridges a higher design is followed, except for structures classified as low water bridges, as discussed in the paper on the Hydraulic Design of Bridges.

2. Operating Conditions for Culverts.

In considering the detailed requirements for a culvert installation, it is desirable that a number of factors commonly involved in the design be reviewed. Fundamentally we design on the basis of providing a culvert that will carry a quantity of water to be drained and not by a formula that determines an indicated waterway opening direct. It is false economy to confine a wide shallow stream in a tall narrow structure. The upper part of the culvert is not used even under severe flood conditions and the effect of the change in the characteristic of flow is to interfere with the normal state of channel flow.

For our purpose of hydraulic design drainage structures may be classified into two general groups: Those operating with submerged or partially submerged outlets and those operating with free outlets. Generally major bridges and structures placed in confined channels are in the first classification named. Structures discharging in a broad flat channel and those discharging into steep outfall channels operate with a free outlet. The greater percentage of culvert installations are in the classification of structures operating with free outlet.

If the operating conditions cannot be determined by visual inspection of the channel characteristics at the structure site, a check must be made by the application of Manning's formula to determine the depth of flow in the outfall channel under design discharge flow. This depth so determined is referred to as the tailwater elevation. If this depth is greater than the critical depth of flow which would occur in a structure with assumed free outlet, then it is apparent that the structure should be designed to operate with a submerged outlet condition.

When the tailwater elevation is equal to or lower than the critical depth of flow, the structure will discharge with a free outlet and any lowering of the tailwater elevation below the critical depth will not affect the discharge of the structure.

Uniform flow will occur in the culvert if it is operating with a submerged or partially submerged outlet and its flow line is placed on the frictional grade. For slight variation from the exact frictional grade the flow for all practical purposes may be considered uniform and the culvert will flow at a uniform depth for its entire length. The velocity of flow will be the same throughout the length of the structure and is represented by the equation:

$$V = \frac{Q}{W}$$

Where: V = Velocity of flow in feet per second.

W = Waterway area in square feet.

Q = Discharge in c.f.s.

For a culvert operating with a free outlet, the design velocity is the velocity occurring at the critical depth and is represented by dividing the design discharge by the waterway area at the critical depth. For rectangular culverts the critical velocity is determined from the nomograph attached, Chart 3.

3. Definition of terms.

At this point definition of certain hydraulic terms will clarify the discussion. In practical terms critical depth can best be illustrated as the depth at which water flows over a weir, this depth being attained automatically because it is the depth at which the energy content of the flow is a minimum. The critical velocity is the velocity occurring at the critical depth and is the controlling flow velocity in determining the proper width of waterway area for a structure operating with free outlet.

There are three hydraulic head losses which occur as the flow passes through the structure. These are velocity head, entrance head and friction head losses. The velocity head is that required to compensate for energy losses caused by velocity differential at the structure. The entrance head loss is that caused by the contraction of the channel at the structure. The amount of this loss is dependent on the type of culvert wings used. The friction head is the head loss caused by roughness of the culvert barrel.

The velocity head loss may be expressed by the following equation:

$$h_v = \frac{V_1^2 - V_2^2}{2g} \text{ in which}$$

h_v = Velocity head in ft.

V_1 = Discharge velocity in ft./sec.

V_2 = Channel velocity or velocity of approach in ft./sec.

g = Acceleration of gravity = 32.2

The entrance head loss:

$$h_e = \frac{Ce\ V^2}{2g}$$

h_e = Entrance head in ft.

V = Average velocity in culvert barrel in ft. per sec.

Ce = 0.10 for parallel wings.

Ce = 0.25 for flared sloping wings.

Ce = 0.50 for flared sloping wings with Weir, The friction head loss:

$$h_f = \frac{29.2\, n^2\, L}{r^{4/3}} \frac{V^2}{2g}$$

h_f = Friction head in ft.

n = Coefficient of roughness usually taken as 0.013 for Concrete.

V = Average velocity in culvert barrel in ft. per sec.

r = Hydraulic radius = Area divided by wetted perimeter.

g = Acceleration of gravity = 32.2.

L = Length of culvert barrel in feet.

The backwater elevation is the elevation of the water on the upstream side of the highway and for a structure operating with free outlet is determined by adding the sum of the velocity head, entrance head and frictional head losses to the critical depth. The tailwater elevation is the elevation of the water downstream from the structure and represents the elevation of the flow in the unobstructed stream channel.

4. Establishment of backwater elevation.

One of the important considerations in the hydraulic design of drainage structures is estimating how much the water flowing in the stream is going to be backed up on the highway embankment and over the adjacent property. No all-inclusive rule can be cited as to where the allowable backwater elevation should be set. This backwater elevation is often established with regard to probable damages which may occur to upstream properties due to inundation of crops or flooding of improvements, such as houses, store buildings or industrial plants. Higher backwater elevations are generally satisfactory in rolling or mountainous country than in flat regions as smaller inundated areas are affected. In any event the backwater elevation corresponding to the design discharge should be so established as to provide a reasonable amount of freeboard against overflowing the highway grade line.

If the total hydraulic head is such as to produce an excessive backwater elevation, the head must be reduced by increasing the width of the structure, or if feasible, this condition may be alleviated by lowering the culvert flow line and lowering the flow line of the outfall channel accordingly. Generally a box culvert should provide a minimum height equal to the critical depth plus the total hydraulic head. For fill type structures as a matter of economy, a cost comparison should be made between the culvert meeting this culvert height and one of greater height with reduction in barrel length corresponding to the reduction in fill height.

5. Allowable Discharge Velocities.

In culverts the velocity of discharge is usually limited to 8 to 10 Ft./Sec. The downstream channel should be protected against erosion by riprap or sodding if the discharge velocity exceeds 8 Ft./Sec. or 8.5 Ft./Sec. In general, a small amount of erosion at the design velocity is not considered serious as it will be corrected by siltation during runoffs of less magnitude. A culvert almost invariably contracts the flow from the open channel and develops a velocity greater than the velocity in the channel. Some degree of scour is inevitable at the culvert outlet unless the channel is in material which can withstand the increased velocity.

6. Selection of Span Lengths.

No definite rules can be stated for appropriate selection of a minimum size of structure for small drainage areas except that the selection of size should be based on capacity to pass the weeds, farm drift or other materials which may reasonably be expected to be delivered to the structure. General practice dictates the use of a 3 ft. x 2 ft. as a minimum size of box and 18 inch round as the minimum pipe size. The volume and nature of drift should be noted in order that spans of adequate lenght can be provided consistent with drift requirements.

One of the principal causes of embankments being washed out in flat country is the accumulation of drift on a fence line upstream from the culvert. When the fence gives way suddenly, the entire mass of drift lodges on the culvert entrance, almost entirely blocking the flow of water. A gate in the fence line, hinged at the top to a cable above flood elevation and free to swing up to pass drift, may help to alleviate this difficulty for flood flows. The gate would automatically remain closed during low flows. The property owner may be encouraged to provide this type of construction.

7. <u>Channel Changes.</u>

As a general principle the natural drainage pattern should be disturbed as little as possible. An apparent saving in cost obtained by channel relocation to combine the flow of two or more channels, thus eliminating a culvert installation may be offset by a higher cost of protecting the highway embankment from erosive stream velocity. A hydraulic analysis of the effect of such a change should be made in advance of incorporating it in the project plans. Insofar as culvert installations are concerned, channel changes should be limited to minor relocations to improve the hydraulics of the channel in the immediate vicinity of the structure. Major channel relocations should be avoided except in very unusual cases as it is likely that the established equilibrium of the natural stream will be disrupted unless the channel is not subject to erosion. The straightening of a natural channel for long reaches of the stream will usually increase the hydraulic gradient to such extent as to produce erosive velocities.

Channel relocations of limited length are highly desirable in many instances to eliminate culverts where the stream crosses and recrosses the highway location, lessen hazard of scour to the highway embankment where the stream bends close to the road and provide improved approach and outlet channels to a structure. A slight relocation of the channel may be desirable to permit the reduction in skew angle of a culvert. This practice should not be abused, however. Where a well defined channel dictates a skewed culvert installation, it is poor practice to provide a square structure and place sharp bends in the channel to meet the culvert barrels. Unless the stream flow velocities are extremely low, an installation of this type will operate at a very low hydraulic efficiency.

Provision should be made to obtain drainage easements beyond the right-of-way limits if construction or future maintenance operations appear desirable in this area.

The effectiveness of any culvert is materially influenced by the manner in which the roadway ditch water is delivered to the culvert. It should be borne in mind that in the channel proper the stream has assumed, in general, a moderate gradient. Where side ditches are on a steep gradient drainage entering the stream channel from this source may cause severe turbulence and interference with flow and efficiency of operation of the culvert. In unusual cases of this nature it may be advisable to swing the ditches to run near the right-of-way line and as they approach the structure to curve them near the point of delivery to approximate tangency with the main channel. Ditches on the downstream end should likewise be swung towards the right-of-way line to enter the channel with the minimum of disturbance.

If the main stream channel is considerably below the flow line of the side ditches riprap flumes or pipes may be required to control erosion at the junction of the ditches and the stream channel.

8. Skewed Installations.

It is important, especially in areas of rolling or mountainous topography, to fit culverts to the average slope of the streambed and to the alignment of the channel. The natural skew may be slightly reduced by keeping the culvert inlet in the streambed and making a limited change in channel alignment at the outlet as hereinbefore mentioned. This practice is satisfactory only within reasonable limits and should be followed only in instances where cost comparisons between providing a skew to fit the natural stream channel and providing the alternate installation, including cost of the necessary channel excavation, favor the latter installation. To provide culverts satisfactorily fitting the channels, it is seldom necessary to use odd skew angles. Usually skew angles of 15 degrees, 30 degrees, 45 degrees or 60 degrees will suffice.

Square culverts provided in channels crossing the highway on a skew and culverts constructed on improper skews result in the stream flow being diverted to one side of the channel with a consequent increase in erosion on that side and the creation of an eddy on the opposite side in which sediment is deposited. As a result the culvert barrel on one side will silt up during medium and low flow and cause serious obstruction at the flood stage.

9. Drop Inlet and Broken-Back Installations.

On steep side hill installations culverts are necessarily placed on appreciable grades requiring provision for reducing outlet velocities. The so-called broken-back culvert is frequently used for this purpose. The grade from the inlet to some point near the center of the structure is made sufficiently steep so that the distance from this point to the outlet may be laid on a very flat grade. If the grade is steep enough to cause a velocity greater than the critical velocity, the resulting hydraulic jump will fall within the box and harmlessly dissipate its energy if the flat grade portion of the culvert barrel is of sufficient length.

Usual practice dictates that the flat grade portion of the barrel for the usual highway culvert installation should have a minimum length of 20 feet. The broken-back type of culvert has the advantage over the drop inlet type for the average installation of this kind in that less excavation is required for its construction.

A drop inlet culvert is one having a vertical shaft or riser at the upstream end of the barrel. The barrel is usually of the same cross section as the riser and is placed on a nearly flat grade. This type of structure is used to intercept drainage in the median between multiple lane divided highways and occasionally on steep side hill locations. However, for the latter case broken-back installations will usually prove more satisfactory and be of less cost. Where it is desired to raise the culvert flow line substantially above the natural streambed and to impound water upstream from the highway, the drop inlet culvert is appropriate. The pool may serve as a debris basin. When such ponding is not objectionable, the drop inlet culvert will usually represent a cost saving as compared to a straight culvert extending across the base of high roadway embankments.

If the drop inlet type of culvert is to be developed to its hydraulic capacity, it is necessary to provide ample head room between the top of the lip and the highway profile. Placing a grate over the entrance of a drop inlet culvert does not change the weir coefficient appreciably except as trash accumulates on the bars. If trash accumulates the hydraulic capacity becomes indeterminate. The use of grates should be avoided in rural areas insofar as possible, since partial clogging by trash is almost certain.

10. Overflow Structures.

Experience has shown that roadway embankments can be overflowed without severe damage, providing the flow is well distributed along the roadway, the difference between upstream and downstream water surfaces is not excessive, shoulders are well stabilized and embankments are protected by ground cover. In arid regions where vegetation is sparse or non-existent, riprap protection is advisable. The justification for the use of overflow culverts depends principally on the traffic service to be rendered. For roads of low traffic volume, interruptions to the movement of traffic may not be of sufficient importance to justify expenditure of additional funds required for a higher type of structure. In our effort to provide more mileage of low cost roads to meet a specific low volume traffic requirement, the overflow type of installation serves a need in helping to realize this objective.

Except for the larger streams where it is readily apparent that the low water structure represents a less costly installation, a comparative estimate should be made between a culvert installation adequate for the usual design discharges and the proposed overflow installation including cost involved in providing the necessary erosion protection. A justified cost saving should be evident before selecting the overflow installation.

Where cost savings are not considerable, the use of overflow culverts should be avoided if the structures are under more than a minimum of embankment fill and where for other reasons excessive discharge velocities will cause frequent damages of serious magnitude to embankment slopes and outfall channels. Other factors being equal, it is good engineering practice to be consistent in the design frequency for all small structures on any given project.

A very important factor in the design of an overflow structure is to provide sufficient culvert opening to carry the design discharge at a velocity which will create little or no hydraulic head on the structure. For this type of structure the design discharge is represented by a flood with flow elevation equal to the elevation of the highway grade line over the structure. In other words, we should lower the highway grade line to meet the elevation of flow of the flood that we can afford to design the culvert to carry. This practice will eliminate excessive overflow velocities across the highway embankment.

11. Hydraulic Analysis of Culverts to be Extended.

From a practical standpoint a culvert which has proved to have a satisfactory hydraulic operating efficiency should be considered acceptable for extending although computed velocities based on the desired discharge design frequency are found to be relatively high. Most any culvert which has been on the highway system sufficient time to need extending should have a flood history long enough to have established its suitability or lack of it.

An inspection of the installation will indicate how the culvert has been operating. If siltation has occurred, it is probable that the condition can be alleviated by opening up the outfall channel. Any appreciable erosion at the outlet should be arrested by filling in the areas affected and providing riprap protection. If streambed erosion is too serious and overflow of the grade line is of such frequency as to cause unjustified delay to traffic, consideration should be given to adding one or more culvert barrels to the structure when the widening project is undertaken. Where overflow of the highway has been objectionable, it should not be assumed that the addition of culvert waterway opening alone will necessarily alleviate this condition. It is possible that the highway grade line is below the elevation of the flood flow of the desired design flood. If this is true the only solution is to raise the grade line. Whether or not overflow is caused by this condition or because the inadequacy of the culvert is causing an excessive hydraulic head at the inlet can be established by observation during flood or by computing elevation of the flood flow in the unobstructed stream channel.

12. Hydraulic Data for Project Plans.

The field survey is to supply all data governing the detailed hydraulic design of the culvert layout including the channels, side ditches and other hydraulic features within the limits of the drainage areas. These data should be presented on the project plans in sufficient detail to justify the design. If contour maps or aerial survey maps are available, much of the information needed for adequate hydraulic analysis may be obtained from this source.

The channel investigation should include a plat of the stream meander and profile extending far enough upstream and downstream to encompass any probable channel change with a minimum of about 300 feet in each direction from the highway. If the highway centerline profile across the stream does not reflect a representative channel section, profile at a typical section should be taken. Several sections across the channel will be required if a channel change is contemplated in order to have required data to compute excavation quantities. Data as to whether the channel is silting or scouring and the nature and volume of drift should be obtained. Observation as to flow characteristics including nature of vegetation is necessary to establish the "n" factor for use in Manning's formula.

In actual design it may be found desirable or necessary to select a culvert which materially influences the head of water above the structure. In the event conditions exist which definitely limit the maximum head of water or the minimum flow line elevation, these conditions should be noted. The permissible head is a matter of judgment and probable damage and structure costs should be carefully weighed. If backwater will flood crops or residential or industrial property, a head of 0.5 foot to 0.8 foot is considered the maximum that should be permitted in the design.

The amount and frequency of discharge for which the culvert is to be designed should be shown on the plans as well as the pertinent data from which this quantity was computed. From the design discharge the required width of structure, the corresponding discharge velocity and total hydraulic head should be indicated for a culvert operating with a free outlet. For a culvert operating with a submerged outlet, the computed flood flow elevation in the unobstructed stream channel and the required waterway opening of the structure with the corresponding discharge velocity and finally the size of the structure selected should be shown. For culverts requiring a bridge layout sheet, these data may be recorded on this sheet or the drainage area sheet may be used to show this information for all structures.

13. Pipe Culverts.

As to whether a box culvert or pipe culvert is more suitable for an installation depends both on the hydraulic factors already discussed and on the relative construction costs. No general rule is practical as there are too many variable factors. From a structural standpoint, box culverts can be designed to carry any required height of embankment, while the capacity of pipe culverts in this respect is limited. Pipe culverts are adapted to speedy construction. This feature may dictate their use where a minimum of traffic interruption, either highway or railroad, is important.

For the hydraulic requirements the box culverts may be constructed for direct traffic, thus having advantage where the highway grade line is low. A square culvert flowing full is more efficient hydraulically than a circular pipe having the same cross-sectional areas flowing full, as the crown elevation at the outlet will be lower for the box culvert and for a given backwater elevation the square section can discharge more water than the circular section (assuming the same roughness and unsubmerged outlet for both).

Between the use of box culverts and pipe culverts, experience will enable the designer to establish an approximate dividing line for use under usual installation conditions. For border line cases comparative estimates should be made between box culvert and pipe culvert designs giving approximately the same hydraulic advantages in order to make an intelligent decision. The relative costs of materials in the various areas of the State will have an influence on this determination.

For the pipe culvert the selection is between concrete pipe of standard strength and extra strength; full circle corrugated galvanized metal pipe and the corrugated galvanized metal pipe arch type. From a hydraulic standpoint the concrete pipe has an advantage over the CGM type because of the lower frictional head loss in a smooth pipe. However, this advantage is relatively insignificant for short culverts. The arch pipe has the advantage of affording a lower crown elevation as compared to the full circle pipe of equal capacity. Where the water contains a high acid or alkali concentration or other chemicals attacking the material from which the pipe is made, satisfactory service will not be obtained from either type. The CGM pipe possesses the advantages of ease of handling, light weight reducing shipping cost and minimum equipment required for installation. In highly inaccessible regions and in areas requiring long shipping distances from pipe sources it should show some price advantage in place. For the usual size of pipe and average conditions of installation, recent bids indicate some price advantage in favor of reinforced concrete pipe. Furthermore, past experience has shown concrete pipe to be of somewhat longer life than metal pipe.

The concrete pipe shows less deterioration when subjected to flow carrying considerable amounts of sand and gravel and has the advantage of requiring less head room as a lower minimum fill is acceptable for the concrete pipe for pipe sizes over 24 inches in diameter. The use of extra strength concrete pipe in place of standard strength should be provided as required for the higher fills as indicated by the standard specifications and should be used under railroad embankments.

For the usual pipe culvert installation where the structure operates with a free outlet, a schedule showing the essential hydraulic characteristics of full circle pipe for both concrete and CGM pipe for sizes from 18 inch diameter to 72 inch diameter is incorporated in this paper as Chart 4.

In selecting the proper size of pipe arch, reference is made to the table of equivalent sizes of full circle pipe and pipe arches in Item 413.2 of the standard specifications. On the basis of a culvert operating with free outlet and at a discharge velocity of about 8 Ft./Sec., the pipe arch designs numbers 2 to 7 correspond very closely from a hydraulics standpoint to the size of full circle pipes listed as equivalent. For the smaller size the full circle pipe is more favorable hydraulically and for the sizes above design No. 7 (48 inch diameter full circle pipe) the pipe arch is of slightly higher capacity. In case there is need for a more exact analysis of flow through the pipe arch type, reference is made to the series of critical flow curves compiled by one of the metal pipe manufacturers. These were distributed to field engineers some time ago. Additional copies may be obtained from the Bridge Division.

14. <u>Typical Hydraulic Design for Box Culvert with Free Outlet.</u>

Assuming that the 10 year frequency design discharge and slope of channel to be:

Q_{10} = 320 c.f.s.

S = 0.008 Ft./Ft.

For this installation the water will discharge from the proposed culvert into a wide outfall channel which indicates that the culvert will likely operate with a free outlet. However, a check will be made of the elevation of flood flow in the unobstructed channel at design discharge. By the use of Manning's formula, solution of which is shown on Chart 1 with a trial depth of flow of 0.7 foot; area under flood flow elevation, A = 106 Sq. Ft.; wetted perimeter p = 151 feet and "n" factor of 0.035 (see chart 2 attached). The average flood flow velocity in the unobstructed stream channel, V = 2.98 Ft./Sec.

Therefore discharge equals AV = 106 x 2.98 = 316 c.f.s.

The assumed depth of 0.7 foot was a good guess for depth of flood flow.

Limiting the discharge velocity to 8.5 Ft./Sec. which is considered desirable to eliminate objectionable erosion at the culvert outlet the critical depth in the structure would be 2.3 feet (see Chart 3). Since the elevation of flood flow in the stream channel (depth 0.7 foot) is less than the critical depth (2.3 feet), the structure would operate with a free outlet.

From Chart 3 we find the needed width of culvert to be 16 feet and for a culvert with flared wings the total hydraulic head is 1.5 feet. (It is assumed that the culvert will be constructed on frictional grade.)

To satisfy the required culvert width of 16 feet, four 4 foot spans are tentatively selected.

On the basis that it is desirable for the structure to operate with little or no head above the culvert crown at the design discharge because of the usage of the adjacent property upstream, we select a culvert with a height equal to the critical depth plus the total hydraulic head (entrance head plus velocity head), equal to 2.3 feet plus 1.5 feet equals 3.8 feet (Use 4 foot culvert height).

Therefore, a 4-4 foot x 4 foot culvert will satisfactorily meet the requirements.

15. Typical Hydraulic Design for Box Culvert with Submerged Outlet.

Assuming that the 10 year frequency design discharge and slope of channel to be:

$$Q_{10} = 400 \text{ c.f.s.}$$

$$S = 0.0C25 \text{ Ft./Ft.}$$

This stream channel is a relatively deep, well defined channel, and it is probable that the elevation of the flood flow in the unobstructed channel will be above the critical depth of a structure operating with a free outlet and the structure finally selected will likely operate with a submerged outlet. To prove this the flood flow elevation will be determined by "trial and error" method using Manning's formula. (Graphic solution which appears on Chart 1).

Take a trial depth of 5.0 Ft.; $A = 115$ Sq. Ft.; $p = 37.6$ Ft. and "n" factor of 0.045. From a solution of Manning's formula $V = 3.51$ Ft./Sec.

Therefore the discharge equals $AV = 115 \times 3.51 = 403.6$ c.f.s.

Again this estimated depth of 5 feet for depth of flow is approximately correct as the computed discharge approximately equals the actual discharge of 400 c.f.s.

As the critical depth for a discharge velocity of 8.5 Ft./Sec. which velocity we consider desirable and proper to control possible erosion at the structure outlet is 2.3 feet (Chart 3). We find that a structure on this stream channel will operate with a submerged outlet since the flood flow depth of 5 feet is greater than the critical depth of 2.3 feet.

For a structure operating under this condition the required waterway area is the discharge ($Q_{10} = 400$ c.f.s.) divided by the allowable velocity (8.5 Ft./Sec).

$$\text{Required waterway area} = \frac{400}{8.5} = 47.2 \text{ Sq. Ft.}$$

From an inspection of the proposed highway profile, it is considered desirable to use a culvert of 5 foot height.

Try 2-5 foot x 5 foot = 50 Sq. Ft.

Corrected discharge velocity: $V = \frac{Q}{A} = \frac{400}{50} = 8$ Ft./Sec.

The head losses are computed from formula given earlier:

(Assuming flow line of culvert is placed on frictional grade.)

Velocity head:

$$h_v = \frac{V_1^2 - V_2^2}{2g} = \frac{8^2 - 3.51^2}{64.4} = 0.08 \text{ ft.}$$

Entrance head:

$$h_e = C_e \frac{V^2}{2g} = 0.25 \frac{(8^2)}{(64.4)} = \underline{0.25 \text{ ft.}}$$

Total head 1.05 ft.

If the increased backwater elevation resulting from this head is objectionable, it may be reduced by increasing the size of the structure.

Try 2 = 6 foot x 5 foot - 60 Sq. Ft.

Corrected velocity:

$$V = \frac{400}{60} = 6.67 \text{ Ft./Sec.}$$

Corrected head losses:

$$h_v = \frac{6.67^2 - 3.51^2}{64.4} = 0.50$$

$$h_e = 0.25 \frac{(6.67^2)}{(64.4)} = \underline{0.17}$$

Total corrected head 0.67

If the increase in cost of providing the larger structure is justifiable to lower the backwater elevation, the 2 - 6 foot x 5 foot culvert should be selected.

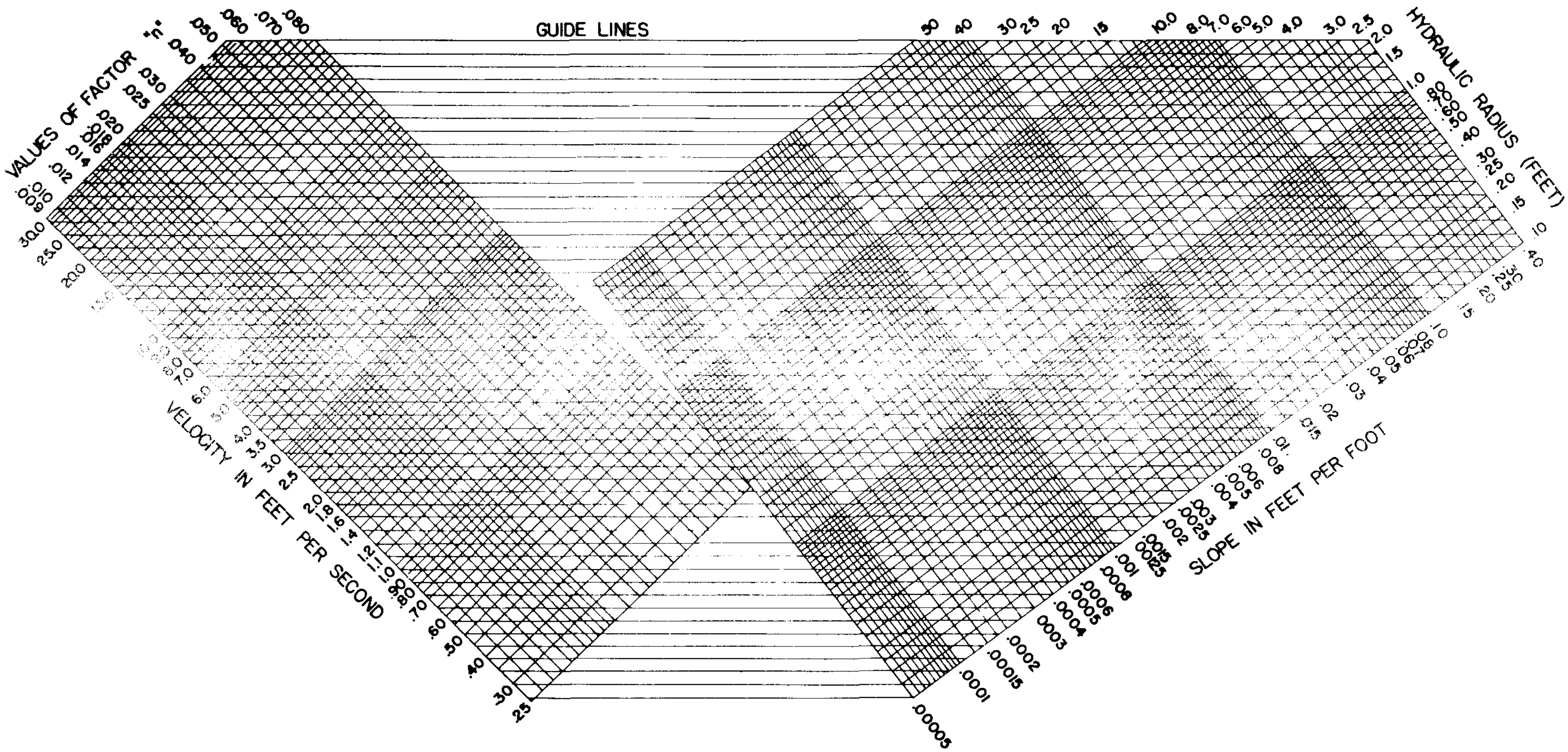


CHART 1

DIAGRAM FOR SOLUTION OF MANNING'S FORMULA

VALUES OF "n" FOR MANNING'S FORMULA
CHART 2

	Value of "n" for Manning's Formula		
	Good	Fair	Bad
Canals and Ditches:			
Earth - Straight and Uniform	0.020	0.022	0.025
Earth - Fairly Rough	0.025	0.030	0.035
Earth - Sodded - No Rank Growth	0.035	0.040	0.045
Earth - Rank Growth	0.040	0.045	0.050
Earth - Dredged Channels	0.027	0.030	0.033
Rock Cuts - Smooth and Uniform	0.030	0.033	0.035
Rock Cuts - Jagged and irregular	0.035	0.040	0.045

Natural Streams:

See Table 2, page 137.

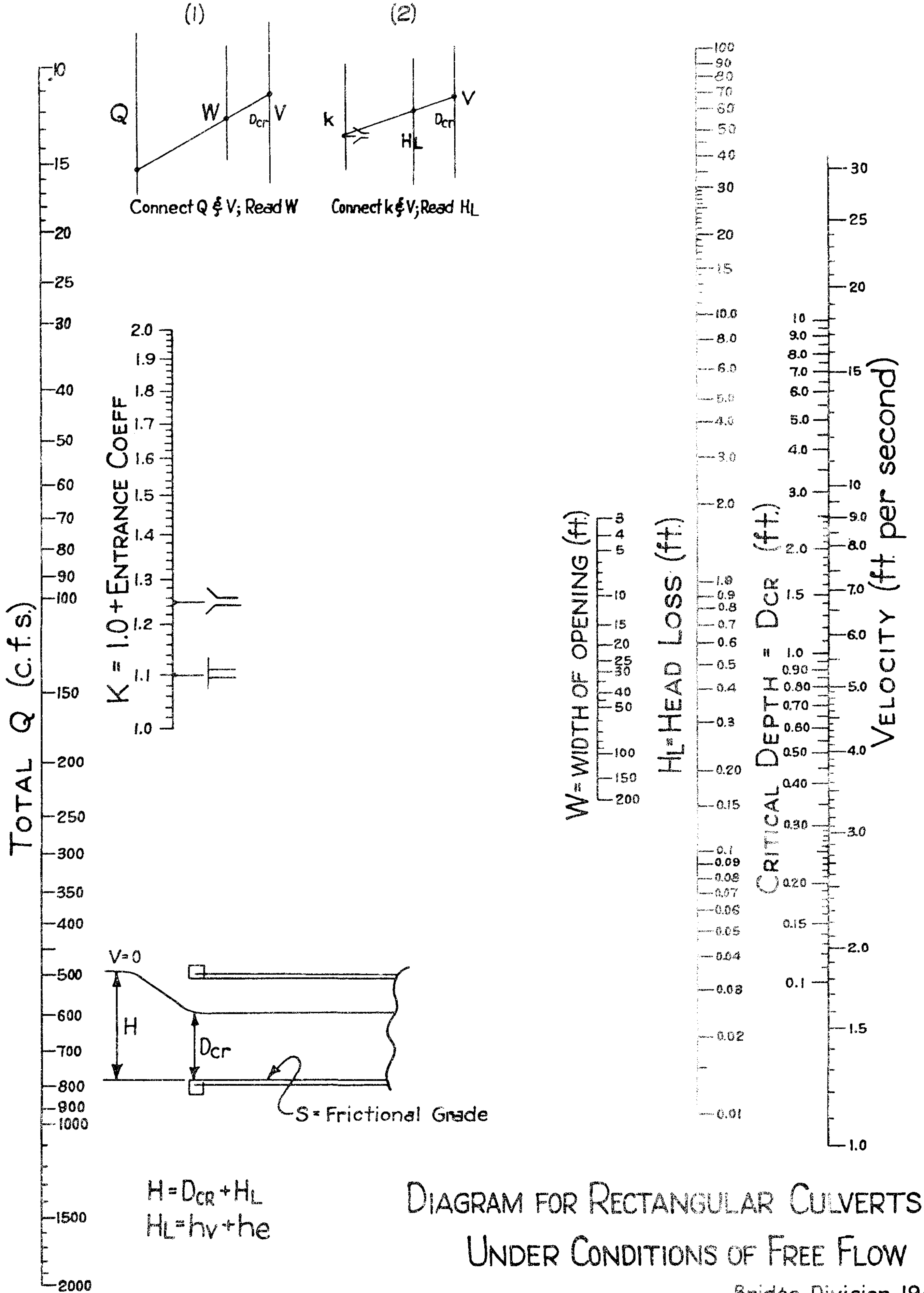


DIAGRAM FOR RECTANGULAR CULVERTS
UNDER CONDITIONS OF FREE FLOW

Bridge Division 1945
W. R. Welty

CHART 3

PIPE CULVERTS - FREE OUTLET

CHART 4

Pipe Size Inches	Discharge C.F.S.	Crit. Vel. Ft/sec.	Crit. Depth Ft.	Area @ Crit. Depth Sq. Ft.	Depth at Entrance Ft.	Critical Grade %*		Total Area Pipe Sq. Ft.
						Conc.	CGM	
18	13.4	7.93	1.37	1.69	3.0	1.46	3.80	1.77
24	22.8	8.00	1.70	2.85	3.4	0.96	2.50	3.14
30	32.8	7.98	1.95	4.11	3.45	0.71	1.86	4.91
36	42.7	7.95	2.13	5.37	3.5	0.57	1.50	7.07
42	52.9	7.99	2.28	6.62	3.6	0.49	1.27	9.62
48	63.3	8.04	2.40	7.87	3.7	0.43	1.12	12.57
54	71.9	8.02	2.48	8.96	3.8	0.39	1.02	15.90
60	78.0	7.95	2.50	9.81	3.8	0.37	0.95	19.63
66	88.0	8.02	2.59	10.97	4.0	0.34	0.89	23.76
72	96.4	8.05	2.64	11.98	4.0	0.32	0.84	28.27

* n = 0.013 Conc.
n = 0.021 CGM

HYDRAULIC DESIGN OF BRIDGES

1. Scope

This paper will deal with the proportioning of medium and large bridges as distinguished from multiple box culverts and small bridges of equivalent proportions. We will arbitrarily take a structure length of 50 feet as the dividing line and confine our study to a consideration of bridges greater than 50 feet in length.

The procedures to be followed in determining the quantity of water carried by a stream have been covered by others and will not be repeated herein. We will take up the problem at the point where the amount of water carried by the river or creek under consideration has been determined and will go into the various phases of the procedure for proportioning a structure which will afford a safe and economical facility for passing this amount of water under the highway.

At this point it should be emphasized that the hydraulic proportioning of major structures is far from an exact science. Actually the engineering profession knows so little about this subject that most engineers hesitate to display their lack of knowledge by discussing it. Our engineering colleges avoid it in setting up their hydraulics courses, and we know of no text book that devotes more than a paragraph or two to this very important problem.

In view of this situation you will readily understand why this discussion is confined to the simplest features of the matter under discussion. No attempt will be made to cover crossings of very wide flood plains where one or more relief structures in addition to the main channel structure are necessary since the time allotted will not permit consideration of this very complicated problem. This paper will cover only streams of medium or narrow flood plain width which may be served adequately by a single structure.

2. The Overall Problem

The overall problem is to provide a structure which will permit the uninterrupted passage of traffic across the stream at all, or practically all, times; which will withstand without major damage all floods except possibly those of very rare occurrence and unusual magnitude; which will not require extensive maintenance and repair; and to provide all this at a reasonable cost. The problem would be greatly simplified if cost were no consideration. All that would then be necessary would be to determine the greatest possible width and height of the stream under maximum flood conditions and provide a single span bridge, high enough in the air to clear this maximum possible high water plus any drift which might be carried and long enough to span the entire flood plain. Such a solution would of course entail structures of enormous cost for the crossing of all but the most innocuous streams. The item of initial cost must be weighed against the inconvenience to traffic resultant from infrequent closures due to high water and against the hazard of severe damage or total destruction of the bridge by floods of rare intensity. In short, we must take a calculated risk in proportioning bridges, just as we take a calculated risk in most of the acts we perform in daily life. It easily can be shown that it is uneconomical to design a structure to clear floods of 1000 year frequency when the useful life of the structure cannot be estimated at above 75 to 100 years. A subsequent section on "Design Frequencies" goes into this subject further and gives recommended values for use on various classes of roads.

Having chosen the proper flood frequency for which the structure is to be designed, and assuming that the quantity of flow for this frequency has been pre-determined, the next step is to obtain data upon, or analyze, the channel flow and determine the elevation and velocity of the flow in the open channel for this flood frequency. The grade line of the bridge is then established at such height that the lowest portions of the superstructure will clear the elevation of the flood crest by a suitable margin to allow for passage of drift. The total length of structure is then determined. In the interest of economy in initial cost the length of structure should be kept to a minimum since, within the usual ranges, the cost of a bridge per unit of length is much greater than the cost of an embankment of corresponding height. On the other hand, if the bridge is made too short, the crowding of a large quantity of water through too narrow an opening results in destructive current velocities, and, sometimes in an objectionable backwater level. In short the bridge should be made long enough to preclude the development of excessive velocities or an objectionable piling up of the water above the bridge, but no longer than necessary to satisfy these requirements.

The final step is to fix the lengths of the individual spans. The main channel spans should be made of sufficient length to permit the free passage of drift of the size and amount reasonably to be expected in the main body of the stream and the approach spans likewise should be of sufficient length to clear drift of the character to be expected along the stream edges. Beyond this point the problem becomes a question of economy, in which the additional cost of longer spans is balanced against the advantages gained from fewer and more favorably located piers; or, in other words, the superstructure cost is weighed against the substructure cost. This feature as well as the other factors enumerated above are covered in greater detail in the subsequent paragraphs.

3. Flood Design Frequencies

Our current manual on the ''Rational Design of Culverts and Bridges'' recommends that small bridges be designed to pass floods of 25 to 50 year frequency and that large bridges be designed for 50 to 100 year frequency. These criteria are good general guides but do not take into account the relative importance of the highway route under consideration. This range in relative importance has been greatly broadened since the manual was written by the development of very heavily travelled highways of the expressway class at one extreme and the taking over of very light traffic farm to market roads at the other. The occassional closure, by high water, of an unimportant farm to market road is of minor consequence as compared to traffic stoppage on a major highway artery, hence it is apparent that this factor of highway importance should be given due weight in determining the flood design frequency. The following table gives the frequencies which are recommended as a general guide to be followed in proportioning structures. The values given are intended to apply to average conditions and are subject to appropriate modification as dictated by sound engineering judgment wherever unusual conditions prevail. As an illustration let us consider the case of a very light traffic road crossing a small stream with a deep primary channel and very little flood plain. The table calls for a 5 year design frequency but investigation shows that the cost of a structure which would pass a 25 year flood would be only a few dollars more than that of a 5 year structure. The proper solution in such a case is, of course, to build the 25 year structure.

TABLE 1

Recommended Flood Frequencies To Be Used
In the Design of Bridges

Daily Traffic Volume (Annual Avg.)	Minor Streams	Major Streams
1 to 100	5	10
100 to 400	10	25
400 to 6000	25	50
6000 and over	50	100

As mentioned above, the values given in this table are intended only as a general guide and are subject to modification as dictated by local conditions, the availability of alternate routes across the stream, and the cost differential involved by an increase or decrease of the design frequency.

There is a rather prevalent misconception of the proper use of flood frequencies in bridge design which should be corrected. Some of our engineers apparently believe that, if a structure is to be designed for, say a ten year frequency, the effect of any larger flood on the structure may be completely disregarded. This is a false conception, particularly when designing for low frequencies. The structure and approaches should be so proportioned that a ten year flood will not top the highway or damage the structure and they should furthermore be so designed that a 25 or 50 year flood, although closing the road to traffic, will not ordinarily destroy the bridge. This is best accomplished by keeping the grade of approaches to a level just above the design high water so that a relief is afforded to take the water flow of floods greater than the design flood. In short our bridges and approaches should be built to take the design flood <u>without overflow</u> and to take larger floods <u>with overflow</u> but without developing velocities high enough to take out the bridge or extensive amounts of the approaches.

4. Channel Flow

Having selected the flood frequency to be used and having determined the quantity of water to be carried as based on this chosen flood frequency the next step is to determine the elevation of water surface (i.e. the tail water elevation) and average current velocity of the unrestricted stream when carrying this amount of water.

Wherever authentic records on the flow of the stream at flood stage are available, such as rating curves for the stream, high water elevations for known amounts of discharge, slope of the water surface, etc., this information should be used. The records of the Texas Board of Water Engineers, the U. S. Geological Survey, the Bureau of Reclamation, the Corps of Engineers and other similar agencies, should be consulted to accumulate all data available on the stream in question at or near the proposed crossing. Even when such data is not applicable to the proposed location or is not of sufficient adequacy to give all the information needed, it is very valuable as a check against the stream flow computations.

In many instances, particularly on the smaller streams, it will be found that there are no available records of stream flow at points close enough to the proposed crossing to serve as a guide or that the available records cover too short a period of time or are not of sufficient detail to give the information required. Hence it is necessary in most instances to compute the depth of flow and the average velocity. Manning's formula is probably the best tool for this purpose and its use is recommended.

Manning's formula for flow in open channels is:

$$V = \frac{1.486}{n} r^{2/3} s^{\frac{1}{2}} \quad \text{where,}$$

V = mean velocity in ft. per sec.
n = coefficient of roughness
r = hydraulic radius = area divided by wetted perimeter
s = slope in feet per foot

Having obtained the mean velocity the quantity of flow is determined by multiplying the mean velocity by the area, or in equation form:

$$Q = AV$$

The selection of the proper values of n is the most troublesome feature in applying this formula as this factor varies widely with the degree of roughness of the stream bed and banks, the tortuosity of the channel and the amount and character of vegetation or other obstructions, such as large boulders, in the stream bed and on the flood plains. The value will vary with the seasons of the year, being larger in spring and summer when vegetation is rank and smaller during the fall and winter when the trees are bare of leaves and grass and weeds have died away. It will also vary from year to year depending upon whether the vegetation in flood plain is allowed free growth or whether it is cleared away by man or diminished by long drouths or destructive floods. Table 2 gives approximate average values which are recommended for general use.

TABLE 2

Values of n for Manning's Formula
as Applied to Natural Streams

PRIMARY CHANNEL	Straight Alinement	Tortuous Alinement
Smooth banks, little or no vegetation	.030	.035
Rough, irregular banks and bed, little or no vegetation	.035	.040
Light brush or scattered trees on banks	.040	.045
Heavy brush and trees on banks	.045 to .050	.050 to .055
Very dense brush and trees on banks	.050 to .055	.055 to .060
Very rough, densely vegetated banks, large boulders or trees in bed	.060 to .070	.070 to .080

Note: With the exception of the last case, above values are based on the condition of little or no vegetation in bed of stream. When such condition is encountered, or when trees on banks overhang stream and obstruct flow in major portion of channel, values of n should be increased.

FLOOD PLAINS	Smooth Surface	Rough, Irregular Surface
Bare soil or grass sod, no high weeds	.030	.035
Scattering brush	.035	.040
Medium brush, scattering trees	.040 to .050	.050 to .060
Thick trees, little or no undergrowth	.060 to .070	.070 to .080
Heavy brush or heavy brush and trees	.080 to .090	.090 to .100
Very dense brush and trees	.100 to .130	.130 to .150

Note: Large weeds which will not flatten under flood flow may be considered the same as brush.

Special Exception: For streams with beds of deep loose sand, where the sand flows and conforms to the stream flow, such as in the Canadian River, the value of n is considerably lower than shown above. Values of the order of .020 should be used in such instances.

General Note: The above values of n are based on tests conducted by the U. S. Department of Agriculture and the U. S. Geological Survey and are believed to represent the best information presently available. Due to the limited amount of experimental work of this nature which has been performed and to the difficulty of describing and classifying the degrees of roughness, these values must be treated as approximate only.

The slope $\underline{s}$ is the gradient of the surface of the water. In a few instances it will be possible to determine this slope directly by waiting for a sizeable flood and then measuring the fall of the water surface over a given length of the channel. Water elevations for this purpose should never be taken on each side of an existing bridge or similar obstruction as this procedure will not give the true slope of the natural stream. A reach of the stream should be selected which is as uniform in character throughout as possible and free from abrupt changes in cross section, slope and amount of vegetation or other features which might cause wide variation in the coefficient of roughness.

In other instances authentic high water marks may be available, spaced a suitable distance apart on an acceptable reach of the stream from which the true slope can be computed. In such cases it must be established that the high water marks to be used represent the same stage of the flood.

Wherever data of the character described above is available, it should be used, but in most instances dependable information of this nature will not be available and the usual practice, which is of satisfactory accuracy, is to assume that the water surface will be parallel to the bottom of the channel and to use the mean channel slope as the value for $\underline{s}$. In determining the channel gradient a reach of at least 2000 feet should be used, profile run on the bottom of channel through this reach and a straight line drawn, passing as closely as possible through the high points of this profile as illustrated in Figure I. The slope of this line is the value to be used in the formula.

The procedure to be followed will be illustrated by carrying through the computations on a typical crossing as follows:

ILLUSTRATIVE PROBLEM

As an illustration we will take the bridging of the small stream as shown in Figures I and II. For simplicity we will assume a right angle crossing. The bridge is to be designed for a flood of 25 year frequency and the discharge for this frequency has been computed and found to be approximately 12,000 sec. ft. We need to know the height of water and the average velocity in the unrestricted channel when the stream is carrying 12,000 cu. ft. of water per second. We first must determine the value of the slope factor $\underline{s}$. Since we have no records of the actual slope of the water surface at flood stages, we will determine the mean slope of the stream bed and assume that the water surface will slope in the same amount. A profile of the stream bed covering a reach from 500 feet upstream to 1500 feet downstream from the crossing is run and plotted as shown in Figure I. A straight line is then drawn to approximate the average slope of the stream bed, this is done by placing the line in such manner as to pass through, or nearly through, the high points of the stream bed. The slope of this line is then computed. In our case the fall is 3.6 feet in 2000 feet or $s = 3.6 \div 2000 = .0018$.

The next step is to obtain a typical cross section of the stream a short distance downstream from the crossing. This involves running levels on a line at right angles to the stream extending from a point well above the high water level on one side to a corresponding point on the other side. In choosing the location of this typical cross section care should be taken to avoid deep holes in the stream bed and local irregularities in the contour of the banks or flood plains, and to choose a section of regular contour whose depth and width represent the general average of the reach under consideration. Also the line chosen preferably should be at a point where the banks are slightly converging in the downstream direction rather than diverging. As a general rule this line should lie somewhere between the center line of crossing and a point 500 feet downstream but it is better to go as far as 1000 feet or 1200 feet downstream or even a short distance upstream if a suitable cross-section does not exist at a more favorable location.

In our example a point 500 feet downstream from the crossing has been chosen and the cross-section at this point is shown in Figure II. The primary channel of the stream is reasonably straight and fairly well free of vegetation, large boulders or other obstructions. The west flood plain, however, is covered with a medium growth of brush and trees while a heavy growth is encountered throughout the east flood plain. Accordingly, there will be a large variation in the value of the roughness factor $\underline{n}$ and it will be necessary to divide the stream up into 3 sections, as indicated in Figure II, compute the water carried by each separate section and add the three together to obtain the total discharge of the stream. Referring to Table 2, we choose values of n = .035 for the main channel, .050 for the west flood plain, and .080 for the east flood plain.

From the rather meagre high-water records available on our stream, we are able to estimate that the water level for a 25 year flood will lie somewhere between Elev. 510 and Elev. 516; hence we will compute the discharge of the stream for flood levels at 2 foot intervals between Elev. 510 and 516 and plot the conveyance curve of the stream for this range. From this curve we will be able to read off the flood level which will correspond to a discharge of 12,000 sec. ft.

The computations are shown in Figure III. To explain these computations in detail we will take the line for Elev. 516. The waterway area "A", below El. 516 for the west flood plain is computed (or planimetered) from the cross section in Figure II and found to be 387 sq. ft. The wetted perimeter, p, is measured at 150 feet. The hydraulic radius, r, is equal to A $\div$ p or 387 $\div$ 150 = 2.58. The velocity, V, is then determined by Manning's formula $V = \frac{1.486\ r^{2/3}\ s^{\frac{1}{2}}}{n}$, and found to be 2.35. The discharge Q, is then obtained by multiplying A by V, or 387 x 2.35 = 910 sec. ft. In a like manner the discharges of the primary channel section and the east flood plain section are computed and the three added together to give a total discharge of 16,800 sec. ft. The average velocity at this level is determined by dividing the total discharge by the total waterway area, in our case 16,800 $\div$ 2539 = 6.6 ft. per second.

The conveyance curve as shown in Figure IV is then drawn by plotting the flood water elevation against the discharge, and the velocity curve as shown in Figure V is prepared by plotting the water elevation against the average velocity.

As previously mentioned our design is to be based on a discharge of 12,000 sec. ft. From the conveyance curve we find that a flood height of Elev. 514.3 gives this amount of discharge, and from the velocity curve we find the average velocity of the stream will be 6.5 ft. per sec. Note that the water elevation so determined is at the point of the typical cross section which is not necessarily immediately adjacent to the structure. Since the average channel slope is .0018, the elevation of tail water at the structure will be 514.3 + .0018 x 500 = 515.2.

5. Height of Structure.

The height of structure is determined by placing low steel (or "low concrete") a short distance above the computed tailwater elevation as necessary to allow for the passage of drift. Our usual practice is to allow a free board of about 2 feet for small streams carrying ordinary amounts of drift and 3 feet to 4 feet for major rivers carrying heavy drift.

In this connection it might be well to mention the fallacy of basing the height of a structure wholly on past high water marks unless authentic records have been maintained over a long period of time. In the first place the so-called high-water mark may actually be several feet in error and in the second place it may represent a 10 year flood or possibly even a 500 year flood. Another misconception is that the restriction of a channel resultant from the building of a bridge affects the tail water elevation below the bridge. This is not true; the stream levels below the bridge are not changed by the insertion of the bridge and the same applies for all practical purposes to the water level immediately beneath the bridge. This level can be taken to be the same as that computed for the unrestricted channel. There will be a slight rise in the water level a short distance upstream from the bridge but this occurs far enough away that it may be disregarded in determining amounts of freeboard.

In our examples we have determined that the elevation of water surface under the bridge for the design discharge will be El. 515.2. We will allow about 2 feet freeboard, which will place low concrete elevation at El. 517.2. We will assume that standard continuous concrete slab units will be used with depth from crown of road to low concrete of about 1.25 feet, which gives us a required grade elevation of 518.45. We will round this off to 518.50 and place our roadway grade at that elevation.

6. Length of Structure.

The structure must be made of a length sufficient to accommodate the quantity of water to be passed without developing destructive current velocities or excessive back water head. Under average conditions, the usual practice is to keep the average velocity thru the bridge to a value of from 6 to 8 feet per second. This value is often lowered to the order of magnitude of 4 ft./sec., for sluggish streams with loose, easily eroded, sand and silt stream beds such as are found in many parts of East Texas, whereas it may safely be raised even as high as 16 to 18 ft./sec. on streams in the hill country of West Texas where the velocities in the natural channels are very high and the rocky stream beds are very resistant to erosion. A large measure of sound judgement is required in determining the velocity to be allowed at any specific crossing. The degree of resistance to erosion of the stream bed and banks must be considered as well as the effect of possible deep scour around piers and header banks. The current velocity in the primary channel of the unrestricted stream can often be used as a general guide, keeping the average velocity through the bridge opening to not more than, say, 1-1/3 times that velocity.

Another factor to be considered in fixing the bridge length is the width of flood plain dammed by the approach embankments. If the bridge end is set in too far from the edge of flood plain, erosive velocities will develop around the bridge ends and the water may pile up along the upstream side of the embankment until it is overflowed. Where wide flood plains are encountered, the usual solution is to use one or more relief structures in addition to the structure at the primary channel. This develops into an intricate and specialized problem which will not be covered in this paper.

The actual fixing of the structure length reduces to a cut and try proposition. A trial layout is drawn up, with bridge ends placed at favorable points as governed by the contour of the crossing, and with total length based on a suitable arrangement of individual simple spans or continuous units. The effective waterway area of the bridge opening thus provided is calculated and the average velocity and backwater head computed therefrom. If these values are suitable the layout is used, if too high or too low, the layout is lengthened or shortened and the procedure repeated until suitable values are obtained.

In our sample problem, we will try a bridge 180 feet in length as shown in Figure VI. After sketching in the header banks and bents, we compute the net waterway area below El. 515. 2 which we have determined to be the tail-water height for a 25 year flood. This area is found to be 1540 sq. ft. The average velocity through the bridge is then: 12,000 ÷ 1540 = 7.8 feet per second. The stream bed and banks at this location are of soils not easily eroded and the average velocity will be practically the same as that of the primary channel of the unrestricted stream. Hence, the assumed structure length of 180 feet will be ample.

We will now compute the backwater head by the formula:

$$H = \frac{C\,(V_1^2 - V_2^2)}{2g}$$

where C is a factor which takes into account the entrance loss and friction losses due to piers or bents, V_1 is the average velocity through the bridge opening and V_2 is the average approach velocity in the unrestricted stream. The entrance losses and friction losses are generally rather small, and a value of 1.10 for C will be close enough for usual conditions. The head for our particular case then becomes $H = 1.10 \times \frac{(7.8)^2 - (6.5)^2}{64.4} = 0.32$ ft., which is of negligible amount. The backwater elevation is then: 515.2 + 0.3 = 515.5.

Since we have proportioned our bridge for the comparatively low frequency value of 25 years, it would now be well to check the effect of a 50 year flood. We will assume that the discharge for a 50 year flood has been computed at 16,300 sec. ft. From the conveyance curve in Figure IV we read off a water elevation of 515.8 for this value of Q. Projecting this back up the channel to the bridge we obtain a tail water elevation of 516.7. Therefore our proposed structure with low concrete at Elev. 517.25 will clear the 50 yr. flood with about a half foot to spare. The net waterway area below Elev. 516.7 is found to be 1780 sq. ft. This gives an average velocity of 16,300 ÷ 1780 = 9.2 feet per second which should not cause excessive damage. Accordingly we can consider our bridge length satisfactory.

7. Length of Individual Spans and Pier Locations.

In fixing the individual span lengths of a structure, the main points to be considered from a hydraulic standpoint are the clearances necessary to allow the free passage of drift and the placing of the piers or bents at points where they will not be undermined or weakened by excessive scour and will not catch and pile up large masses of drift. As previously mentioned, the ideal bridge from a hydraulic standpoint would be a single span extending across the entire width of the stream, with no interior supports to interfere with the passage of water and drift. On the other hand, as a general rule, a bridge composed of a number of spans can be built much more cheaply than a single span structure of the same total length. For bridges of medium height and with favorable foundation conditions the general rule is that the shorter the span the cheaper the initial cost of the bridge, at least down to the range of 20 foot to 30 foot spans. Hence the fixing of span lengths usually becomes a problem of compromising between hydraulic and economic advantages.

Since there are so many different factors involved which vary widely with the particular conditions involved at each crossing, no set rules for fixing the span lengths can be established. The following general rules, however, should be observed:

(1) The spans across the primary channel should be of sufficient length to pass the largest trees or other forms of drift normally carried by the stream. For our larger rivers, such as the Brazos and Colorado, span lengths in the range of 100 feet to 150 feet are believed to be adequate for this purpose in most instances, and proportionately shorter spans may be used on the lesser streams.

(2) Span lengths for the portions of the structure across the flood plains, where the velocity of the water is moderate, generally should be considerably shorter than the main channel spans but should be commensurate with the magnitude of drift normally to be expected in these portions of the stream. Lengths of 40 feet to 70 feet are usually adequate for the flood plains of our largest streams, and shorter lengths down to a minimum of 15 feet to 25 feet will usually be proper for small creeks.

(3) Placing piers at or near the middle of the main channel where stream velocity is highest should be avoided wherever feasible.

(4) Placing a pier on or near a steep bank subject to sliding or undercutting is also to be avoided.

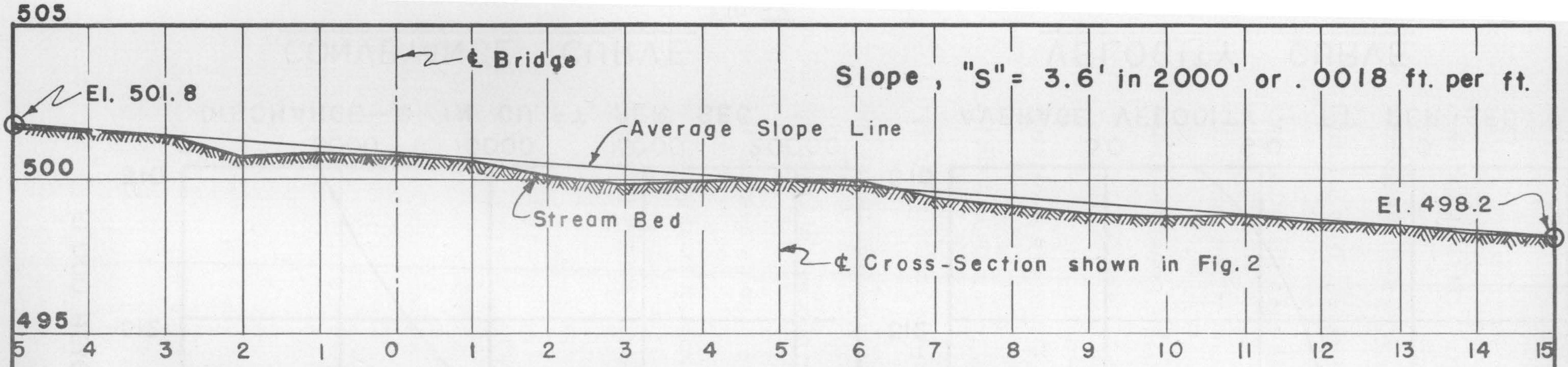
(5) The economical ranges of the various types of span in general use should be taken into consideration, bearing in mind that there is generally a sharp increase in cost in each step from the shorter type of span to the longer. The usual maximum span lengths of the types of bridges most widely used in our work are listed below as a guide:

Simple Concrete Slab Spans - - - - - - - - - - - - - -	25 ft.
Simple Concrete Slab - Girder Spans - - - - - - - -	30 ft.
Continuous Concrete Slab Units, Interior Span - - -	30 ft.
Simple Concrete Girder Spans- - - - - - - - - - - -	50 ft.
Simple Steel I-Beam Spans - - - - - - - - - - - - - -	70 ft.
Continuous Steel I-Beam Units, Interior Span - - -	90 ft.
Continuous Steel Girder Units, Interior Span - - - -	250 ft.
Through Truss Spans - - - - - - - - - - - - - above	250 ft.

We will now consider our typical crossing as an illustration of the determination of span lengths. The layout shown in Figure VI is based on the assumptions that ample bearing capacity can be obtained with piles or drilled shafts of moderate length, that the stream does not carry heavy drift, and that the stream bed is not susceptible to deep scour. Under these conditions we have chosen the continuous concrete slab type of structure as the most economical and suitable and have provided a 30 foot span for the central portion of the primary channel, with 25 foot spans over the remainder. Note that we have avoided locating a bent at mid-channel.

Now let us assume that the stream carries a fairly heavy amount of drift which might pile up against the two central bents which are pretty well out into the main channel. In such case we would want to move the bents back to locations at or near the edges of the main channel. One solution would be to use a 60 foot simple I-Beam as the central span, flanked by two 30 foot slab girder spans of our standard CG series on either end, maintaining the 180 foot total length of bridge. Another solution would be to use 3-60 foot simple I-Beam spans and a third solution would be to use a 55 ft.-70 ft.-55 ft. continuous I-Beam unit. This last solution would be the best from a hydraulic standpoint, and, if deep foundations were required, might be the cheapest of the three since it would require two interior bents or piers as compared to four for the first layout. On the other hand it would require a special design whereas standard plans could be used on the other two layouts. This is an item which must be considered, particularly with our present manpower shortage. Solution No. 1 would probably be the cheapest if foundation conditions were favorable but would be less desirable than the other two layouts if appearance were a consideration because of the unsightly breaks in the lines of the structure at the junctions of the concrete spans and the I-Beam span.

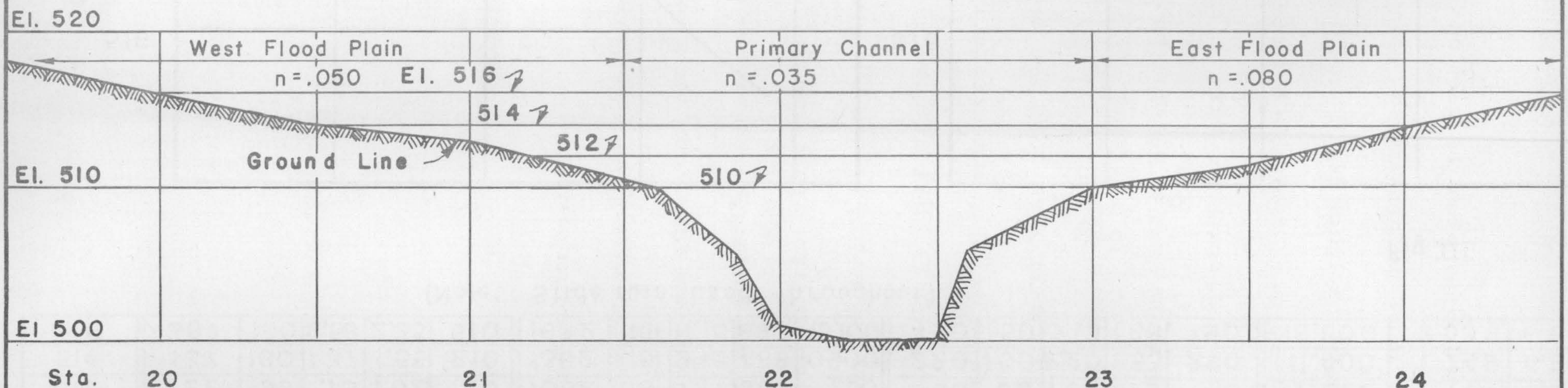
It can be seen from the above example that there are numerous factors to be taken into account in working up a bridge layout and that a layout which is ideal from one standpoint may be highly undesirable for another. The only practicable procedure in many instances is to lay out several different arrangements, make an estimate of cost of each and weigh the advantages afforded by the more expensive layouts against the saving in cost afforded by the cheaper layouts. In short, it is impossible to divorce entirely the problem of hydraulic capacity from the problem of construction costs.



PROFILE OF STREAM BED

Scale: 1" = 200' Horiz., 1" = 5' Vert.

Fig. I



TYPICAL CROSS SECTION OF UNRESTRICTED CHANNEL

Scale: 1" = 50' Horiz., 1" = 10' Vert.

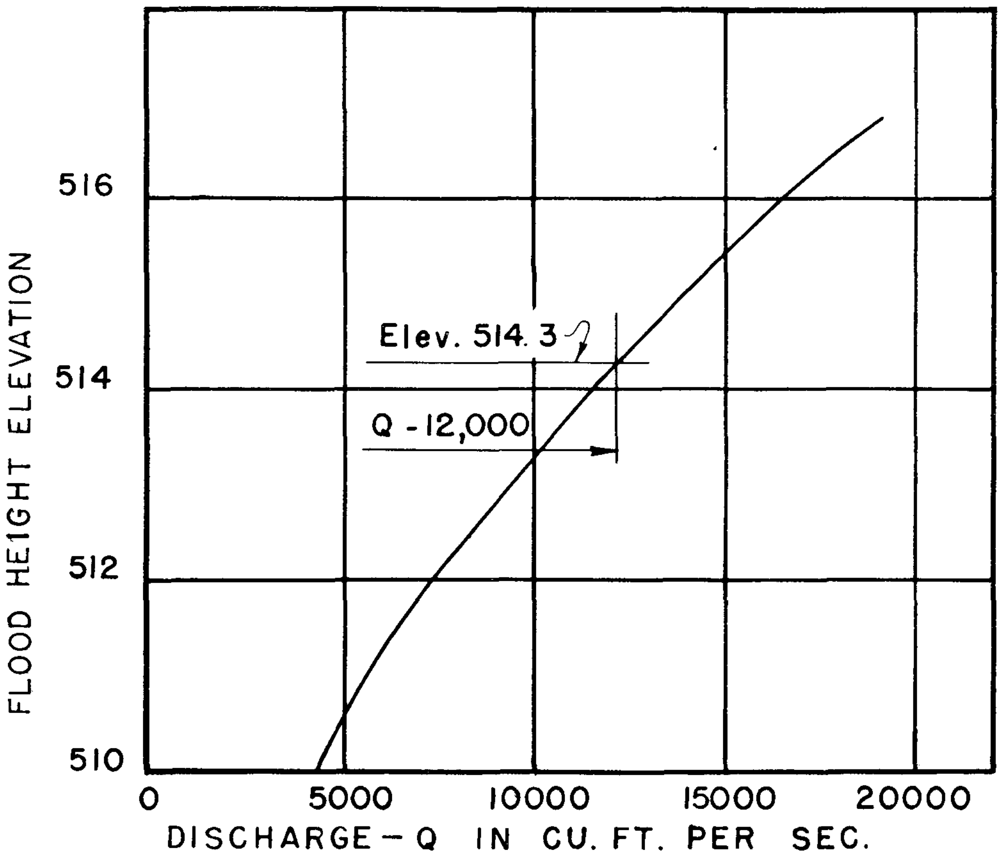
Fig. II

COMPUTATIONS FOR CONVEYANCE CURVE

Elev. Water	West Flood Plain (n=.050)					Primary Channel (n=.035)					East Flood Plain (n=.080)					Total Q	Total A	Avg. V
	A	P	R	V	Q	A	P	R	V	Q	A	P	R	V	Q			
510	0	-	–	–	–	785	141	5.56	5.62	4410	0	–	–	–	–	4410	785	5.6
512	22	30	.73	1.02	20	1082	151	7.17	6.69	7250	60	60	1.00	.79	50	7.320	1.164	6.3
514	137	100	1.37	1.55	210	1382	151	9.15	7.88	10.900	220	100	2.20	1.33	290	11.400	1 749	6.5
516	387	150	2.58	2.35	910	1682	151	11.15	8.97	15,100	470	150	3.13	1.68	790	16,800	2 539	6.6

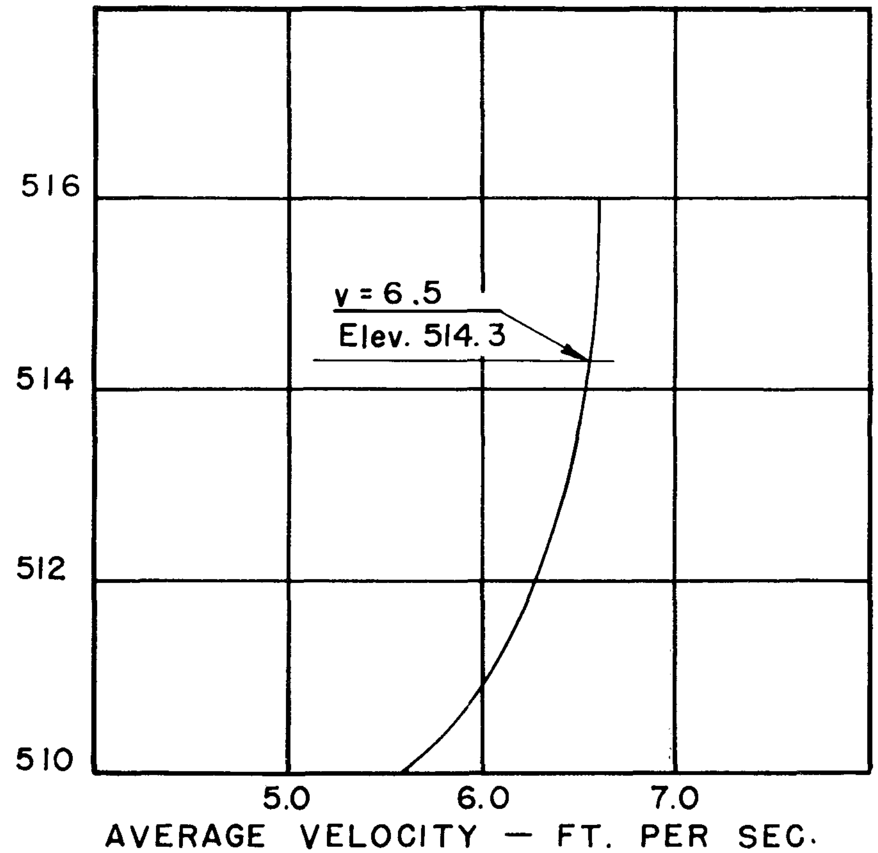
(Note: Slide rule used throughout)

Fig. III



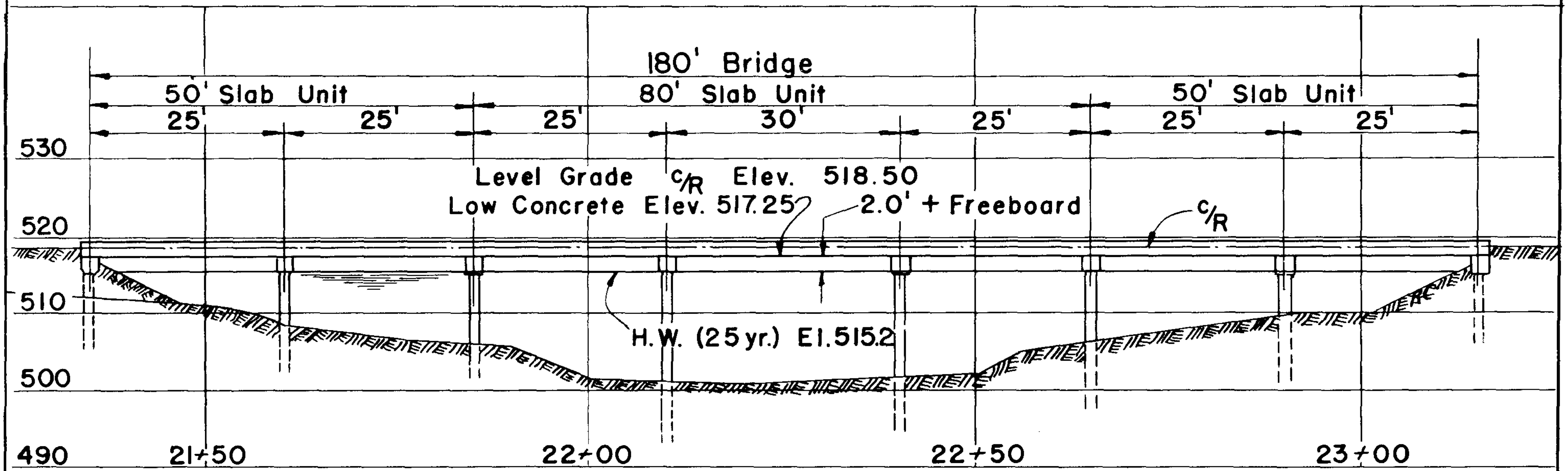
CONVEYANCE CURVE

Fig. IV



VELOCITY CURVE

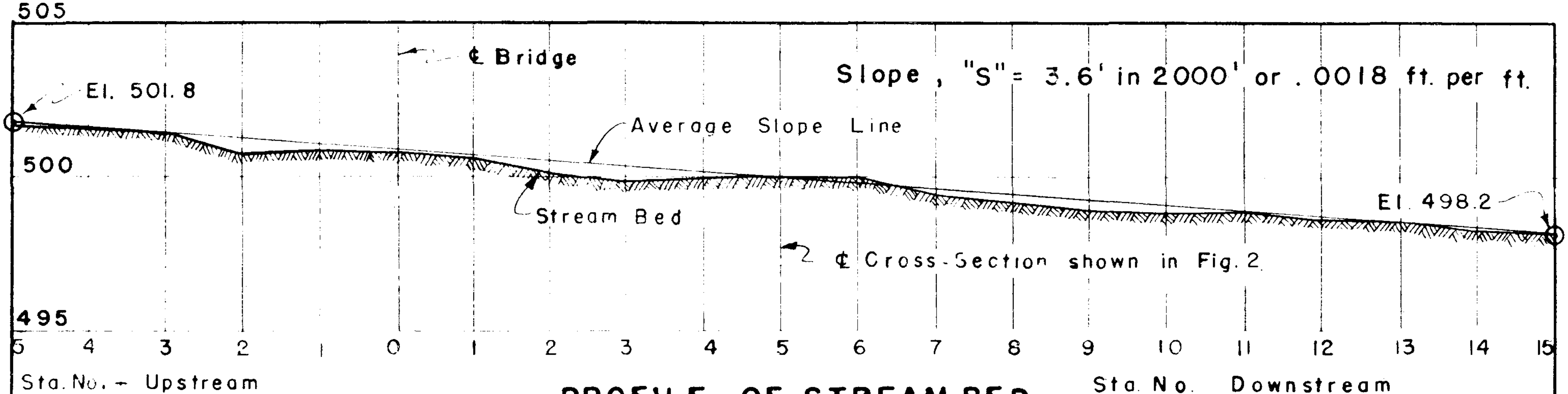
Fig. V



BRIDGE LAYOUT

Scale: 1" = 20'

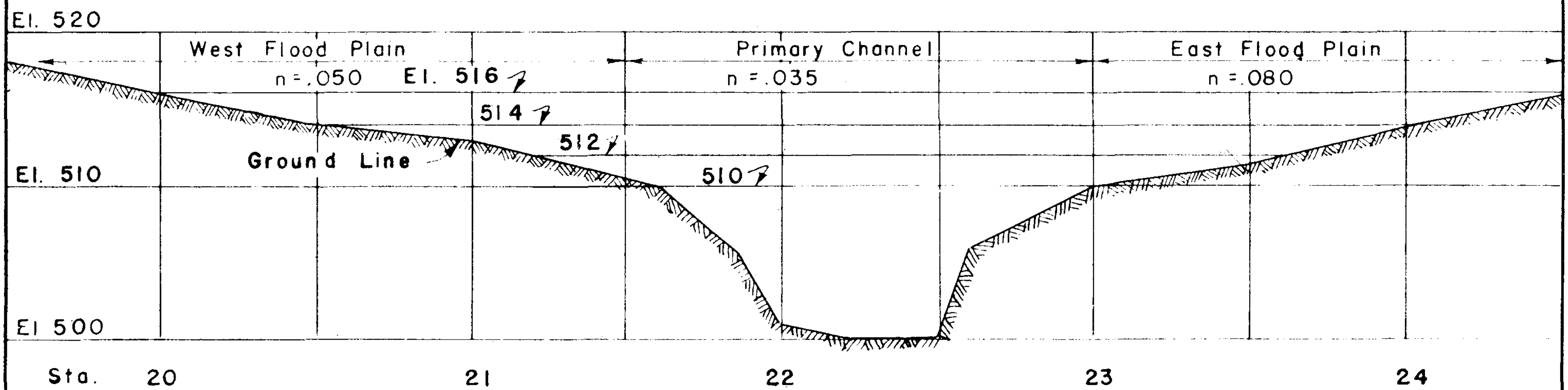
Fig. VI



PROFILE OF STREAM BED

Scale: 1" = 200' Horiz., 1" = 5' Vert.

Fig. I



TYPICAL CROSS SECTION OF UNRESTRICTED CHANNEL

Scale: 1" = 50' Horiz., 1" = 10' Vert.

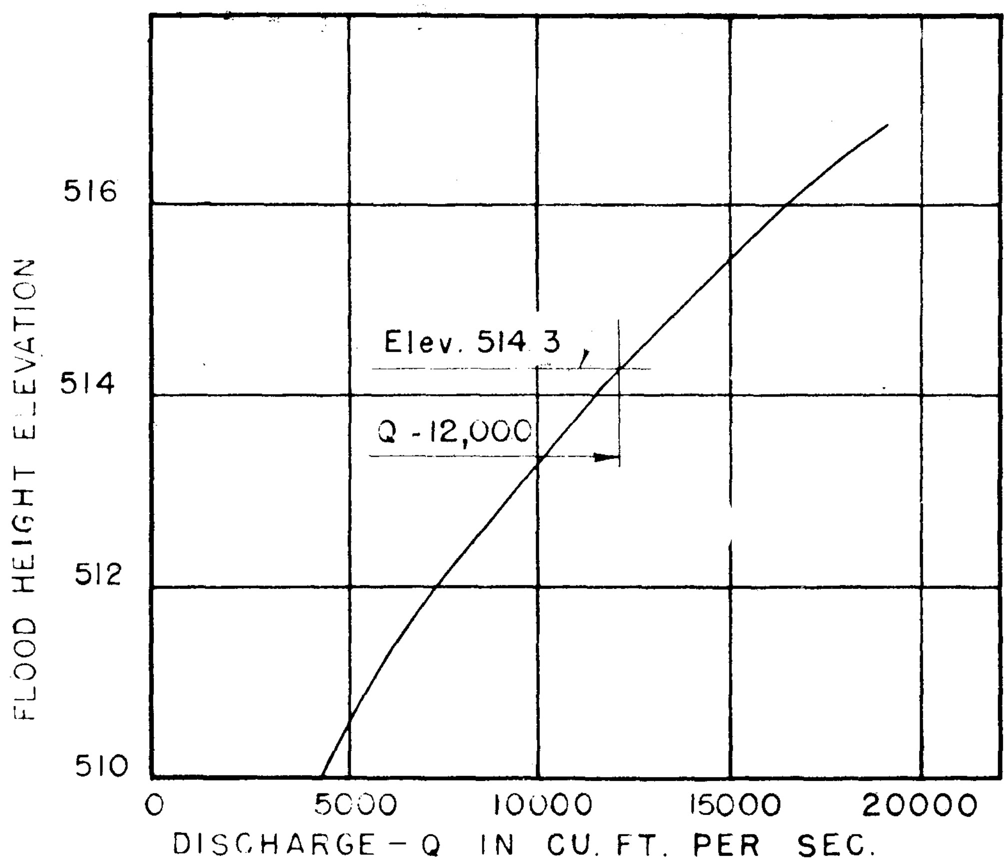
Fig. II

COMPUTATIONS FOR CONVEYANCE CURVE

Elev. Water	West Flood Plain (n=.050)					Primary Channel (n=.035)					East Flood Plain (n=.080)					Total Q	Total A	Avg. V
	A	P	R	V	Q	A	P	R	V	Q	A	P	R	V	Q			
510	0	-	-	-	-	785	141	5.56	5.62	4410	0	-	-	-	-	4410	785	5.6
512	22	30	.73	1.02	20	1082	151	7.17	6.69	7250	60	60	1.00	.79	50	7.320	1.164	6.3
514	137	100	1.37	1.55	210	1382	151	9.15	7.88	10,900	220	100	2.20	1.33	290	11,400	1 749	6.5
516	387	150	2.58	2.35	910	1682	151	11.15	8.97	15,100	470	150	3.13	1.68	790	16,800	2 539	6.6

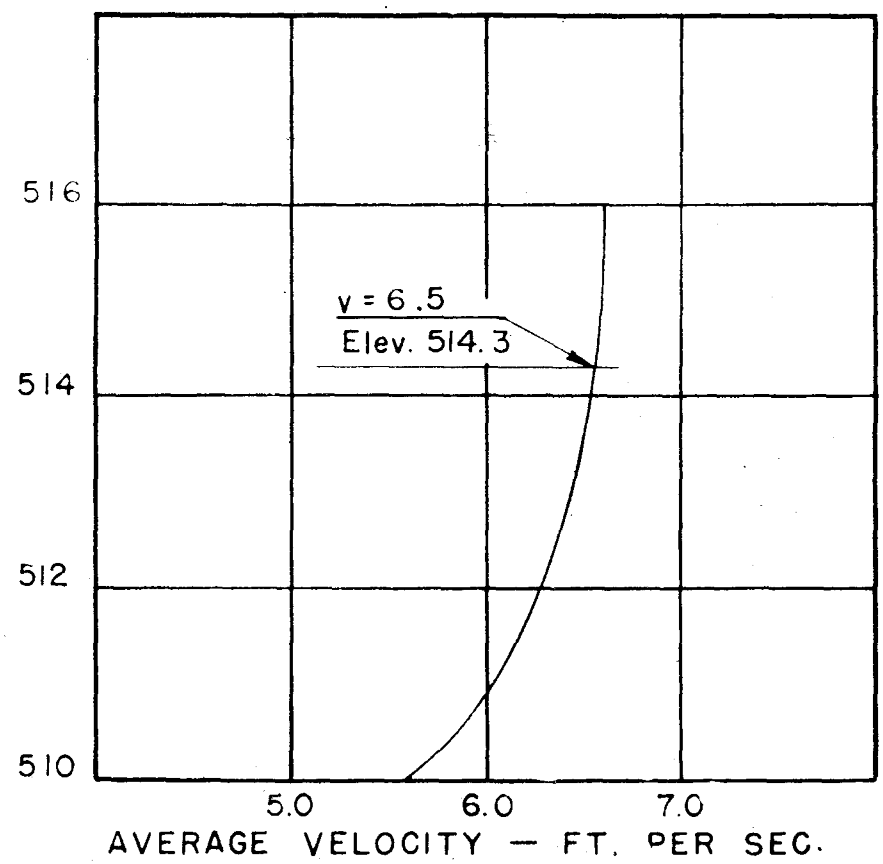
(Note: Slide rule used throughout)

Fig. III



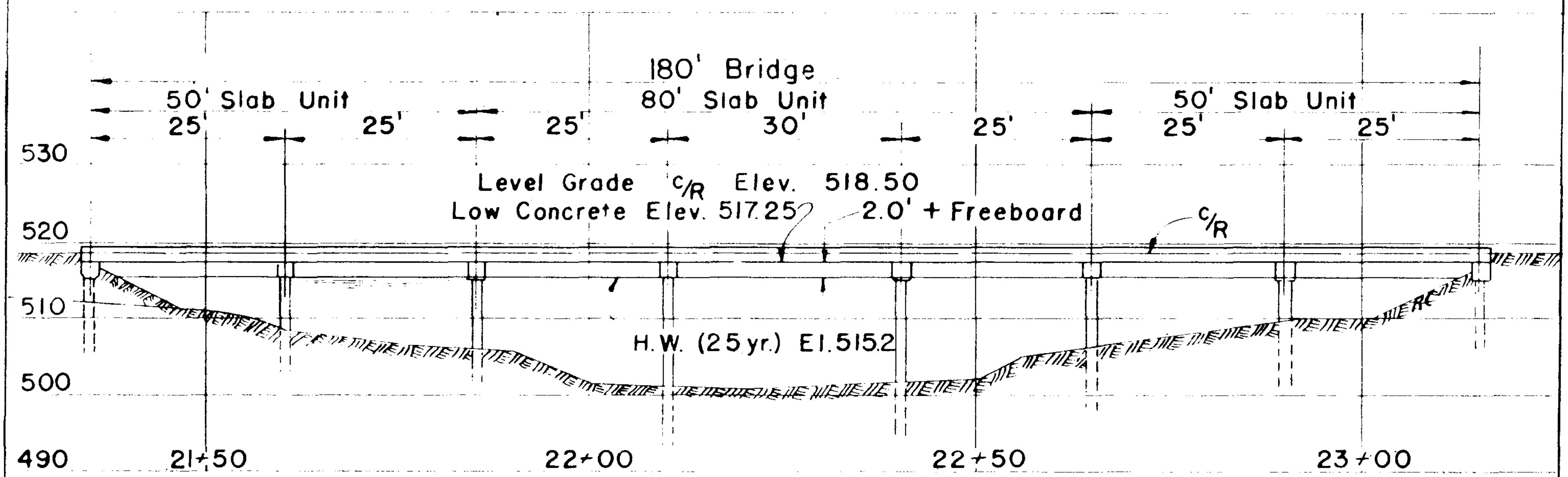
CONVEYANCE CURVE

Fig. IV



VELOCITY CURVE

Fig. V



BRIDGE LAYOUT

Scale: 1" = 20'

Fig. VI

HYDRAULIC DESIGN OF STORM SEWER INLETS

GENERAL

The purpose of this discussion is to summarize the contents of the Texas Highway Department's "Storm Sewer Manual," published in April 1951 with special reference to the method of determining the length of curb openings which should be provided for a predetermined runoff; grate and curb inlets, catch basins; and such revisions as have come to our attention since this manual was published. Street drainage in cities has become quite an important phase in our highway work and since this type of drainage is rather complicated from the design viewpoint it is our thought that a uniform design procedure should be developed in order to facilitate the computations at a rapid rate and to reduce the mathematical aspects connected with any hydraulic problem to a minimum. Many empirical formulas have been developed, yet in the absence of actual tests on full scale models it appears that very little has been accomplished. It is believed, however, that the principal objective is to provide the designer with some tools he may use in making a more intelligent design for storm sewer inlets. Please bear in mind, that the construction and maintenance problems and cost may overshadow purely hydraulic considerations in many instances.

Inlets, in general, should be spaced to limit the spread of water on the pavement to a predetermined width that will not interfere with traffic. In some instances the entire roadway may have to be ponded, for a short duration. This condition, however, should be held to a minimum, and should only be permitted if local conditions so dictate. Under no condition should ponding of water in the gutters be permitted which would cause damage to surrounding property caused from overflowing the roadway curbs.

Under ordinary conditions, inlets are normally placed upstream from crosswalks and street intersections. To drain sag vertical curves properly, it is considered good practice to place three inlets in each curve; one at the low point and one each at the points of tangency or at convenient points adjacent to them. In so doing, water will be removed before it begins to spread out, and in addition, the deposit of sediment on the pavement will be reduced.

Where the pavement is warped in transitions between superelevated and normal sections, water should be normally picked up before the cross-slope of the pavement begins to change. This is particularly important in areas where icing occurs.

Large run-off from areas off the project that enters the project from side streets should, whenever possible, be picked up on the side street before it reaches the project.

Curb Inlets: The capacity of a curb inlet on a continuous grade, when intercepting 100 per cent of the flow in the gutter is given in the chart "Inlet Capacity for Variable Gutter Depressions" on page 11 of the storm sewer manual. This chart has been prepared for zero to 5 inch depressions. According to these curves, the capacity or intercepting values increase rapidly per foot of inlet as the gutter depression increases. It has been recommended in our Storm Sewer Design Manual that the depth of depression may vary from 0 to 1 inch where the gutter is within the theoretical traffic lane. It has been found, however, that a 2 to 3 inch depression of the gutter within the traffic lane is not objectionable. For low point inlets in vertical curves, the wier formula has been graphed on page 13 of the Storm Sewer Manual.

Grate Inlets: This type of inlet is normally used only to drain low places outside the roadway area.

Curb and Grate Inlets: As explained in the Storm Sewer Manual, the combination of curb and grate inlets should be discouraged since no great gain has resulted with this type of installation. The opinions of the various hydraulic research laboratories differ widely; however, the Illinois Division of Highways has developed an improved grate which is currently being tested by the University of Illinois. The end results of these tests are not available at this time.

Our usual pavement crowns are parabolic, straight line or a combination of straight lines. The method of determining the depth of flow in a gutter of a parabolic crowned section as explained on page 19 of the Storm Sewer Manual should not be followed inasmuch as the error introduced in obtaining and using an average slope is considered too great.

If the pavement cross-section is parabolic or is made up of other than straight lines the following procedure should be followed:

Plot cross-section of pavement surface, using distorted scales with horizontal scale about ten times the vertical scale. Calibrate the horizontal scale in feet from the curb and the vertical scale in feet from the flow line of the gutter.

Divide the pavement cross-sections into arbitrary sectors about one foot wide for the first four feet out from the curb then two feet wide to the centerline. These sectors may be lettered a, b, c, etc. (See Plate No. I).

The general procedure is to assume a number of different widths of flow, measured from the curb. Then for each assumed width, using Manning's formula, compute the discharge (q) for each sector comprising this width. To facilitate computations, the assumed width should coincide with sector boundaries. Use a 1 per cent ($s = 0.01$) longitudinal gutter slope. The total gutter flow for each assumed width is the summation of the sector discharges. Plot the total gutter flow (Q) as ordinate and the corresponding assumed width as abscissa. Each assumed width of flow will give one point on this Discharge - Width curve which is for a 1 per cent gutter slope. Other gutter slopes will give a family of curves above and below the 1 per cent curve.

Points on the Discharge - Width curves for other gutter slopes can be obtained by multiplying the discharges found above by 10 times the square root of the new slope.

The use of the table on the following page is suggested to facilitate the above computations.

Discharges for other depths or widths are computed in the same manner assuming either depth at curb or level water surface.

When the cross-section is a straight line or a combination of straight lines, the procedure on page 10 of the Storm Sewer Manual should be followed.

A simple problem to explain the step by step computations for a series of inlets will now be considered.

Let us assume a 76 ft. pavement section with a 3/16 inch per foot cross-slope. The inlets are to be designed for a 5 year flood frequency. Ponding is to be limited to two lanes or 24 feet, leaving one traffic lane or 12 feet unobstructed to flow.

On the Map of Drainage Areas included in a later discussion on Storm Sewer Design, the run-offs (Q) have been indicated. The method of determining these various run-offs will not be discussed here since it is amply covered in our Storm Sewer Design Manual.

n = 0.015 Longitudinal Slope 1.0%

Sector Symbol	Sector Width	Sector Depth	Mean Sector Depth (Hydraulic Radius)	Sector Area	Mean Sector Velocity	Sector Discharge	Total Discharge
1	2	3	4	5	6	7	8
a.		0.431					
	0.5		0.423	0.212	5.6	1.19	
b.		0.415					
	0.5		0.408	0.204	5.4	1.10	
c.		0.400					
	1.0		0.385	0.385	5.1	1.96	
d.		0.369					
	1.0		0.354	0.354	4.8	1.70	
e.		0.339					
	1.0		0.324	0.324	4.6	1.44	
f.		0.309					
	2.0		0.281	0.562	4.2	2.36	
g.		0.252					
	2.0		0.224	0.448	3.6	1.61	
h.		0.196					
	2.0		0.170	0.340	2.9	0.99	
i.		0.144					
	2.0		0.119	0.238	2.3	0.55	
j.		0.093					
	2.0		0.069	0.138	1.7	0.23	
k.		0.045					
	2.0		0.063	0.046	0.8	0.04	13.17
l.		0					

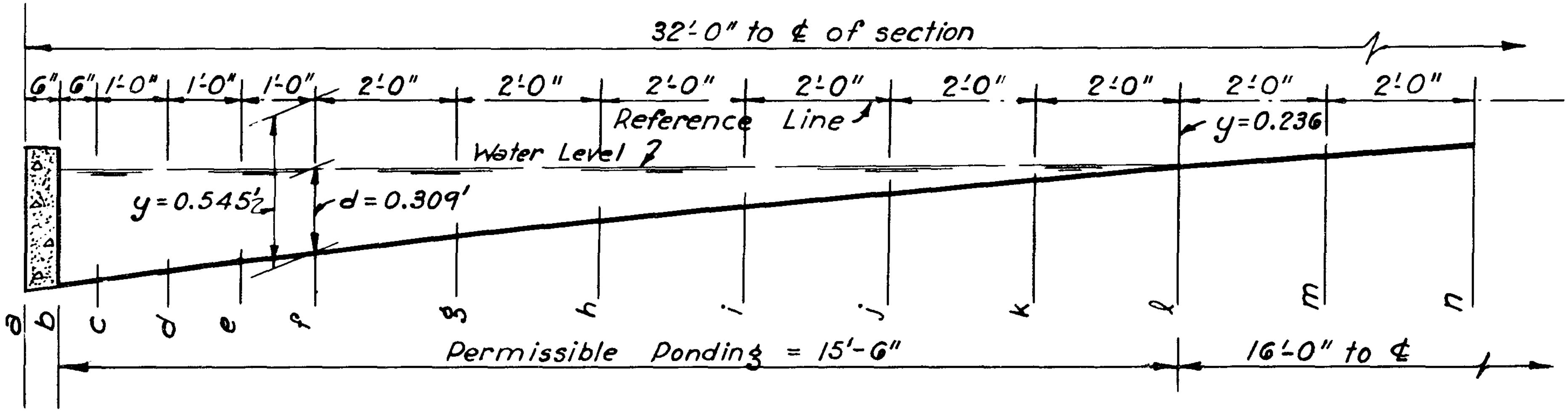
Col. 1 - Symbol designing sector.
Col. 2 - Width of sector in feet.
Col. 3 - Depth of sector in feet.
Col. 4 - Mean depth of water in sector. This is also the hydraulic radius $(R = \frac{A}{WP})$ since the wetted perimeter (WP) is assumed to be equal to the width of the sector.
Col. 5 - Area of cross section of flow in sector in square feet = Col. 2 times Col. 4.
Col. 6 - Mean sector velocity in feet per second = $\frac{1.486}{n} S^{\frac{1}{2}}$ times 2/3 power of Col. 4. (See Plate II)
Col. 7 - Sector discharge in cubic feet per second = Col. 5 times Col. 6 = Q = AV.
Col. 8 - Total discharge of section in cubic feet per second.

It will be noted that a curb inlet is to be provided at Station 9+35 Rt. for a run-off of 4.7 cfs shown in Column 6. (See Plate III). The immediate problem on hand is to determine the depth of flow (y) in the gutter. This depth is to be determined from the nomograph for flow in triangular channels on page 10 of the storm sewer manual in the following manner: The ratio z/n is the reciprocal of the cross-slope divided by the coefficient of roughness. In our example z = 12 divided by 3/16 = 64, and z/n = 64 divided by .015 = 4,266 or 4300 for all practical purposes. The longitudinal gutter slope is 0.25 per cent as shown on the plan-profile sheet. All of the above values are indicated on the inlet computation sheet in columns 7, 8 and 9. We now turn to the nomograph on page 10 of the Storm Sewer Manual and connect the z/n ratio with slope (s) or 4300 with .0025 and mark the point where this line intersects the turning line.

We now intersect the point on the turning line with the discharge (4.7 cfs) and read a depth of flow (y) of 0.29 feet. The ponded width is equal to y times z or 0.29 x 64 which is 18.6 feet, or less than the permissible ponding width. Since the area next to the gutter acts as a parking lane, it is decided to use a 4 inch gutter depression at the inlets with suitable transitions so as to provide a comparatively smooth riding surface. The interception by this inlet (qL) with a 0.29 ft. depth of flow and a 0.33 ft. gutter depression will be 0.59 cfs per foot of curb opening as shown on the graph on page 11 of the Storm Sewer Manual. This value is recorded in column 13 of the inlet computation sheet. Since the run-off is 4.7 cfs and the interception per foot of curb opening is 0.59 cfs. the length of curb opening required as shown in column 14 is 4.7 divided by 0.59 or 8 ft. It has been decided to provide for a 5 foot opening as indicated in Column 15.

Since the balanced capacity of Curb Inlet 1 is exceeded, the ratio of the flow intercepted to the total flow in the approach gutter may be read from the graph "Ratio of Intercepted to Total Flow" page 12 of the Storm Sewer Manual, using the two ratios L/La and a/y. L is the actual length of the inlet and La the theoretical length found in column 14. In our particular example, L/La = 5.0/8.0 = 0.63 and a/y = 0.33/0.29 = 1.14. By referring to the above mentioned chart and using a L/La of 0.63 and an a/y of 1.14, we find a Q/Qa ratio of 0.74 which is the per cent interception of the 5.0 foot inlet. The actual interception, therefore, is 4.70 x 0.74 = 3.48 cfs. and the carry over is 1.22 cfs.

The procedure for determining curb openings for all other inlets is identical to that explained heretofore with the exception that carryovers will have to be included in the following inlet as shown for Curb Inlet 2. It will be noted that at Curb Inlet 10, the ponded width is 30.7 ft. which is excessive on a 36 ft. pavement. In this particular instance, however, we assume there is no other way to obtain a lesser width unless the City will install a storm sewer system along the side streets within the drainage area, in order that the bulk of the run-off is intercepted before entering the highway.



Data:

Width of pavement 64'-0"
8" Parabolic Crown
Crown Formula: $y = kx^{3/2}$
Limit of ponding = 15'-6"

To Determine Depth of Flow:

$y = kx^{3/2}$; $k = \frac{y}{x^{3/2}} = \frac{0.6667}{32^{3/2}} = 0.003683$

Point "l": For $x = 16$ ft., $y = 0.003683 \times 16^{3/2} = 0.236$ ft.
Point "f": For $x = 28$ ft., $y = 0.003683 \times 28^{3/2} = 0.545$ ft.
Depth of water at point "f" — coordinate "f" minus coordinate "l" = 0.545' − 0.236' = 0.309 ft.

PLATE I

EQUATION: $V = \frac{1.486}{n} R^{2/3} S^{1/2}$

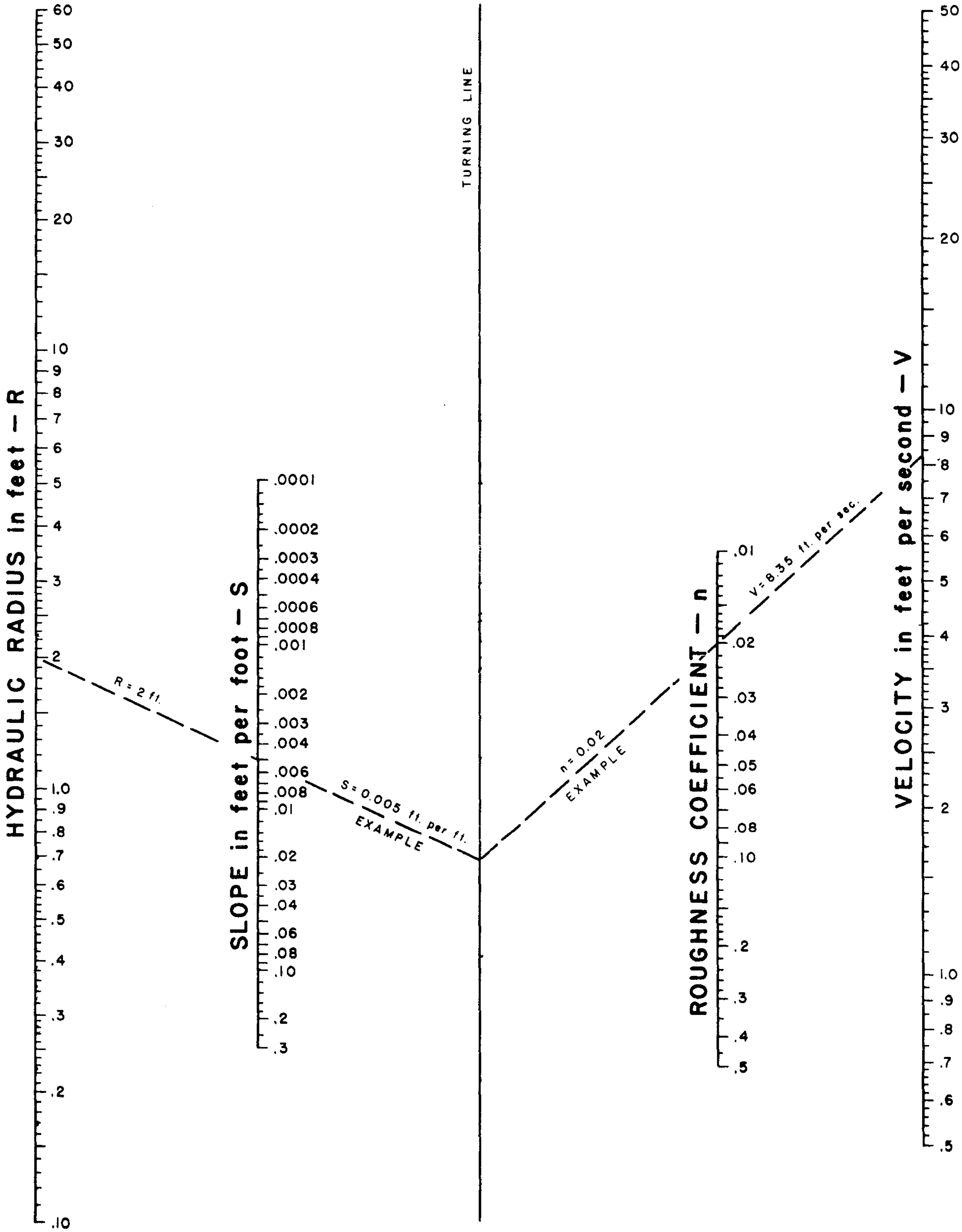


PLATE II
NOMOGRAPH FOR SOLUTION OF MANNING EQUATION

Inlet No.	Inlet Station	D.A. No	Q_a cfs	Carry-over cfs	Total Q_a cfs	z	z/n	s ft/ft.	y ft.	Ponded Width y x z ft.	a ft.	q_L cfs	$L_a = Q_a/q_L$ ft.	L ft.	L/L_a	a/y	Q/Q_a	$Q = Q_a \times Q/Q_a$ cfs	Carry-over cfs	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21
CI-1	9+35 Rt.	2.26	4.70	—	4.70	64	4300	.0025	0.29	18.6	0.33	0.59	8.0	5.0	0.63	1.14	0.74	3.48	1.22	
CI-2	10+52 Rt.	5.57	8.68	1.22	9.90	64	4300	.0025	0.39	25.0	0.33	0.68	14.6	15.0	—	—	—	—	—	
CI-3	13+35 Rt.	5.57	9.51	—	9.51	64	4300	.0025	0.38	24.3	0.33	0.67	14.2	10.0	0.70	0.87	0.81	7.70	1.81	
CI-4	14+50 Rt.	4.83	8.78	1.81	10.59	64	4300	.0025	0.40	25.6	0.33	0.69	15.3	15.0	0.98	0.83	0.99	10.48	0.11	Disregard
CI-5	17+35 Rt.	2.19	4.98	—	4.98	64	4300	.0025	0.30	19.2	0.33	0.60	8.3	5	0.60	1.10	0.71	3.54	1.44	
CI-6	18+50 Rt.	1.61	3.67	1.44	5.11	64	4300	.0025	0.31	19.8	0.33	0.60	8.5	5	0.59	1.06	0.70	3.58	1.53	
CI-7	21+35 Rt.	3.42	7.28	1.53	8.81	64	4300	.0025	0.37	23.7	0.33	0.66	13.3	10	0.75	0.89	0.84	7.40	1.41	
CI-8	22+50 Rt.	1.87	4.14	1.41	5.55	64	4300	.0025	0.31	19.8	0.33	0.60	9.3	10	—	—	—	—	—	
CI-9	25+35 Rt.	3.99	8.24	—	8.24	64	4300	.0025	0.36	23.0	0.33	0.65	12.7	10	0.79	0.92	0.87	7.17	1.07	
CI-10	26+50 Rt.	17.30	17.47	1.07	18.54	64	4300	.0025	0.48	30.7	0.33	0.78	23.8	20	0.84	0.69	0.91	16.87	1.67	

PLATE III ~ INLET COMPUTATIONS

STORM SEWER DESIGN

Because of the amount of work being handled by the Highway Department within urban areas, the personnel of the Department engaged in plan preparation is being called on to design an increasing number of storm sewers. As storm sewer design is a problem somewhat different from that of the usual drainage installation which we are in the habit of designing for rural areas, some comments and a brief illustrative problem on storm sewer design have been included in this instruction course.

A storm sewer is a hydraulic structure designed to intercept and carry runoff from certain specified areas. For the Highway Department Designer this area is usually well defined through the municipality by the location of the highway proposed for construction or improvement and generally includes only those areas draining to the highway. Consequently, the basic layouts of sewers designed by the Department usually follow the alignment of the highway with occasional short branches to inlets just across the street.

As the systems or methods of determining the quantity of water to be intercepted from a given area, that is the relation between rainfall and runoff, and the hydraulic design of storm sewer inlets have been covered in previous discussions, we will confine our attention primarily to picking up the water after it has passed through the inlets and conveying it to a convenient point of discharge off the highway. However, it might be well to mention here that in determining the quantity of runoff to be carried by a storm sewer, a storm frequency of 2 to 5 years is recommended depending on the importance of the highway. For freeways, carrying very heavy traffic, using a 10 year frequency storm is not out of line.

The layout of inlets and connecting pipe lines should be such that the water collecting on the thoroughfare will be intercepted by the inlets and transferred into the sewer before excessive ponding takes place on the highway. In placing these inlets and connecting lines the layout should be so arranged that the water will be carried in as nearly a straight line as possible from the upper end of the sewer to the point of discharge. Sometimes the economic relation between pipe sizes, gradients, cut sections and the presence of man made structures, will make it impractical to lay out the ideal straight line. Pipe sizes and gradients should be selected to provide a minimum velocity of flow of 2.5 to 3.0 ft. per sec. In preparing the layout and working up the final design, consideration must always be given to the possibility of future connections by providing stub lines and larger pipes in the proposed installation if financing of these additions is possible. Manholes should be provided at all abrupt changes in alignment, intersections with branch lines, and at intervals of 400 feet to 600 feet in long lines to provide entrance for inspecting and cleaning.

Once the sewer layout has been established, the type of sewer should be determined. It is the practice of the Department at this time to construct all storm sewers of concrete, either precast concrete pipe, cast in place concrete pipe of circular or eliptical cross section, or cast in place rectangular section corresponding to our standard box culverts modified if necessary for extra fill depth over the structure. As a properly designed storm sewer will invariably function as an open channel, its capacity can be determined by a proper application of the Manning's formula which is available in any hydraulics textbook. Practically all of the storm sewers with which the Department is concerned are constructed of precast reinforced concrete pipe. The method of selecting the appropriate size of pipe will be illustrated in an example later on in this lecture. Except for very short branch lines where 12 inches diameter pipe may be used, pipe sizes selected should be not less than 18 inches diameter.

At a point in the sewer where there is an increase in pipe size, care should be taken to lay the invert profile in the proper manner. Such changes in pipe size will ordinarily take place at an inlet or at a manhole provided as a junction chamber where a branch line discharges into a trunk line sewer. The change in pipe size should be cared for by placing the discharge pipe invert below the inlet pipe invert an amount equal to the difference in pipe diameters. In other words, maintain continuity of gradient at the crown line of the pipe and drop or offset the invert. If this method of establishing the gradient is not carried out, at any time the larger pipe is flowing full, the smaller pipe will be operating under pressure which condition is to be avoided in a properly designed storm sewer.

In laying the profile grade care should be taken to provide the minimum specified cover over the various sizes of pipe. This requirement is usually met automatically in that a headroom or freeboard of 1.5 feet to 2.0 feet should be maintained between the top of pipe and the gutter or lip of inlets. Pinching this freeboard down to a few inches will decrease the capacity of the inlet and cut down the headroom necessary to develop the velocity and entrance head required to force the water into the line when it is operating at maximum capacity.

As soon as approximate pipe sizes and invert gradients have been determined for the several reaches of the sewer, test borings should be made along the line to determine the nature of the material to be encountered during construction as well as to determine the material available below the sewer to give it structural support. When borings are being made, the presence of ground water should be noted in the boring log and later shown on the sewer profile. Although excavation for sewer lines is usually set up in project proposals for bidding as unclassified structural excavation, the nature of the material to be encountered during construction is of great importance to the Contractor in figuring his costs and should be shown on the plans. Occasionally boring information will dictate a revision in selected pipe sizes and invert gradients. If it is found that the proposed sewer line will run through rock or gravel containing large boulders, provision should be made in the plan details to remove the rock and boulders a minimum depth of 8 inches below the pipe and to replace it with suitable bedding material such as sand or fine gravel. If the line runs through soft or mucky material not adequate to form a structural support for the sewer, the soft material should be removed and replaced with gravel and in some cases with lean concrete to form a supporting medium for the pipe. In particularly bad soil it is occasionally necessary to provide for short piles to be driven under the concrete support or cradle to carry the weight of the structure.

A knowledge of the nature of the material to be encountered under a sewer is very important. This is particularly true when lines must be laid on a flat grade where any settlement with resulting departure from the theoretical gradient would materially reduce the capacity of the sewer. The hydraulic capacity of the line can be greatly improved if care is exercised during construction to lay the pipe on true line and grade and by showing on the plans certain details which are frequently omitted. Provision should be made for shaping the bottom of inlets and manholes by filling corners and edges with concrete rounded to conform to the periphery of the intake and discharge pipes up to about half of the depth of the pipes. With this detail carefully carried out during construction there is very little loss of head and resulting sewer capacity through inlets and manholes.

When the layout indicates that the sewer line must be carried under a railroad track, the Bridge Division should be furnished sketches of the proposed construction in accordance with instructions in Bridge Circular No. 4-44 so the Department will have the installation approved by the Railroad Company well in advance of contracting the project. Early submission of these sketches is particularly important in view of the fact that negotiations with the railroad will frequently indicate the possibility of installing the pipe under tracks in an open trench excavation between trains, which method of construction is considerably cheaper than so called jacking methods. Bear in mind that all concrete pipe placed under railroad embankments must be extra strength reinforced concrete pipe and that a 30 inch diameter pipe is the absolute minimum size that will provide working room for excavation at end of pipe. Thirty-six inch diameter pipe is the recommended minimum. Even though a smaller pipe might be sufficient to carry drainage, the larger pipe required for jacking should be used.

In laying out a storm sewer the designer is frequently confronted with the problem of intersections between the proposed line and existing storm sewers or sanitary sewers. A difficulty of this type is both a public relations and engineering problem, the solution of which must be satisfactory to the agencies owning the existing facilities. As an existing storm sewer will usually dictate or follow a logical location to a point of discharge for the proposed line, parallel lines can be run or the two installations can be combined. Limited right-of-way or the expense of procuring the required right-of-way for two lines may dictate the latter course of action. The presence of existing sanitary lines presents a more difficult problem because it is not good practice from a public health standpoint to combine sanitary and storm flow into a single line. Also the gradients of sanitary lines are frequently so flat as not to lend themselves to adjustment to clear any proposed installation. However, this can sometimes be done and is the practical solution when possible. More often it will be necessary to run the proposed storm sewer by the sanitary line either by a true siphon over the line, an inverted siphon under the line, or to carry the sanitary line through the storm sewer. When a sanitary line is carried through a storm sewer, vitrified clay or concrete pipe in that portion of the sanitary line within the storm sewer should be replaced with cast iron pipe. As the sanitary sewer will constitute an obstruction within the cross section of the storm sewer it may be necessary to increase the size of the storm sewer line for a few lengths of pipe on both sides of the sanitary line. Although sometimes necessary, this is admittedly objectionable as the increase in pipe size will decrease the velocity of flow causing solid material being carried by the water to be deposited in the larger pipe. In some cases this problem can be solved by placing a junction chamber at the intersection of the two lines; the junction chamber being amply large to pass all the water which it is anticipated will come down the storm sewer. Such a junction chamber should be provided with a manhole so that it can be cleaned out if it becomes clogged with debris or gravel.

In designing a storm sewer, erosion protection at the outlet of the line must be given consideration. Usually the outfall section of a sewer line is a rather large size pipe and when operating even at a moderate velocity of flow will discharge a large quantity of water with considerable potential capacity for excessive erosion. Usually outfall lines discharge at velocities which definitely present an erosion problem unless discharging onto a highly resistant material such as rock or coarse gravel. It is occasionally necessary to discharge a sewer into an arroyo or creek channel with a channel elevation several feet below the sewer invert elevation as it approaches the point of discharge. In establishing the invert elevation it is always desirable to keep it as high as possible in order to reduce to a minimum the amount of excavation to be done during construction, which results in a sudden drop in grade at or near the discharge. With this type of outlet, erosion protection is invariably necessary. In some cases this can be accomplished by turning say 75 feet to 100 feet of the lower end of the sewer abruptly down on a steep gradient and providing riprap and baffels at the discharge. This type of construction may involve considerable excavation in which case a surge chamber provides a reasonable solution to the erosion problem.

This type of surge chamber would consist of a rectangular box with the approach flow line at normal gradient elevation of the storm sewer line and the discharge invert at such lower elevation as may be necessary to obtain a comparatively flat grade from the surge chamber to the point of discharge. Irrespective of whether or not special features of this type are required at the discharge end of an outfall line, the end of the sewer should be provided with a headwall or if not a headwall some riprap to give the job a finished appearance and to prevent erosion around the end of the pipe from overland flow above the outlet. Riprap placed to dissipate high discharge velocities should be carried well beyond the end and sides of the pipe and up over the line in which case the headwall would not be necessary. However, riprap in connection with a headwall makes a well finished outlet.

Three different time intervals or time concepts are used in storm sewer design; inlet time, sewer time, and time of concentration. The inlet time is the time required for the water to flow overland from the remotest part of a drainage area to the inlet intercepting drainage from that area. Sewer time at any point in the sewer is the time required for the water in the line to flow from the first (generally) or highest inlet in the line to a given point in the sewer. The time of concentration is the sum of the inlet and sewer time. The word "generally" has been inserted parenthetically because as will be shown in a problem to be developed shortly, sewer time is not always figured from the

the first or highest inlet in the line. The time of concentration is the time interval used in determining pipe or conduit sizes.

A simple problem in storm sewer design will now be given brief consideration. This problem has been set up as the storm sewer line to drain a few blocks of Main Street in a small city which for convenience has been called Levelville, Texas. By reference to the drainage map it is noted that ten drainage areas are involved with ten inlets correspondingly numbered as shown on the sewer plan and profile. In problems of this type, if at all possible, it simplifies plan interpretation to number an inlet to correspond with the number given the drainage area from which it receives runoff. The amount of runoff to be carried by the sewer has been determined by the rational method which is well explained in the Department's manual "Rational Design of Culverts and Bridges." The several drainage areas have been determined by calculations using scale dimensions from the drainage map. The coefficient of runoff has been taken as 0.5 for convenience and as being applicable to residential areas with some pavement. CA is the product of the areas times this runoff coefficient, which product multiplied by the intensity of rainfall "I" gives the quantity "Q" of runoff in cubic feet per second for the several areas involved. The inlet sizes as indicated on the sewer profile have been determined by methods discussed in the previous lecture.

The selection of pipe sizes and invert gradients begins at the upper end of the sewer where a pipe has to be provided only large enough to carry the runoff from drainage area No. 1. Also, there being no sewer above Inlet No. 1, the sewer time is zero and the time of concentration is the inlet time only. By reference to the tabulation on the drainage map sheet it is found that the inlet time for Inlet No. 1 is 12.5 minutes with a runoff from drainage area No. 1 of 4.7 c.f.s. to be carried in the first section of the sewer between CI-1 and CI-2. The size of the line can now be determined by the Manning's formula. Opposite page 16 in the Storm Sewer Manual which was passed out in the preceding lecture, is found a graphic solution of this formula for various sizes of concrete pipe. Referring to the first page of this graph it is found that an 18 inch diameter pipe placed on 0.2% grade has a capacity of 4.7 c.f.s. and a velocity of flow of 2.7 feet per sec. This size pipe will be used for the reach of sewer between CI-1 and CI-2.

To design the second run of this sewer, that is the section between CI-2 and CI-3, it is necessary to make a comparison between the inlet time for CI-2 and the time of concentration as determined from the inlet time at CI-1 plus the sewer time between CI-1 and CI-2. This sewer time is the length of sewer between CI-1 and CI-2 or 117 feet divided by 60 x 2.7 or 0.7 minutes, which interval added to the 12.5 minutes inlet time at CI-1 gives a time of concentration at CI-2 of 13.2 minutes. Now by reference to the runoff calculations on the drainage map, we note that the inlet time for CI-2 is 24.2 minutes. In this particular case 24.2 minutes is used at the time of concentration in designing the reach of sewer between CI-2 and CI-3 as it is a reasonable conclusion that water entering the system at CI-1 in the first 13.2 minutes of a 24.2 minute interval will have passed CI-2 before any runoff from the remote portions of drainage area No. 2 could enter the system. The intensity of rainfall "I" for 24.2 minutes time is 3.11 in. per hr. which intensity must be applied to the sum of drainage area 1 and 2 as both areas will be subject simultaneously to this rainfall. The cumulative CA for the two areas is 3.92 which multiplied by an I of 3.11 gives a runoff of 12.2 c.f.s. to be taken care of between CI-2 and CI-3. By reference to the graphic solution of Manning's formula, it is found that a 24 inch pipe placed on 0.3% grade will carry 12.4 c.f.s. at a velocity of 3.9 feet per sec.

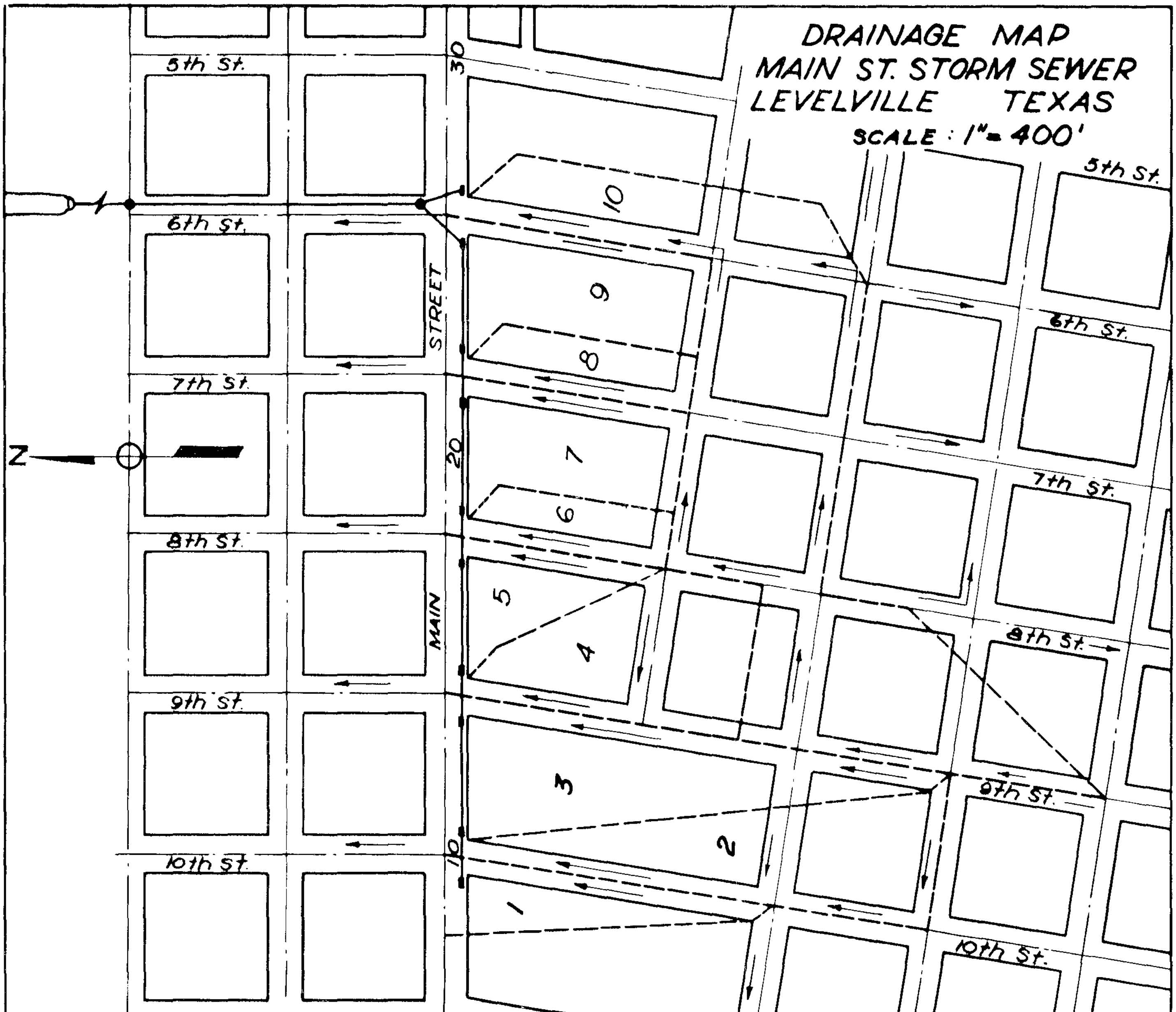
The time of concentration at CI-3 is obtained by adding to 24.2 minutes, the inlet time at CI-2, a time interval representated by the calculation $\frac{283 \text{ feet}}{60\text{x}3.9}$, the 283 feet being the length of sewer between CI-2 and CI-3, the 60 x 3.9 being the velocity of flow converted to feet per minute. This gives the time of concentration at CI-3 of 25.4 minutes which time being larger than the inlet time for CI-3 is used in designing the line between CI-3 and CI-4. The intensity of rainfall for a time of 25.4 minutes is 3.02 in. per hr. which multiplied by 6.71 the cumulative CA for drainage areas one to three inclusive, gives a runoff of 20.3 c.f.s. to be carried in the sewer between CI-3 and CI-4. A 24 inch pipe on 0.8% grade will carry this quantity of water, but it is noted on the sewer profile that a 30 inch pipe on 0.45% grade is proposed. In selecting pipe sizes it was found that a 30 inch line would be needed beyond CI-4 and by using this size pipe between CI-3 and CI-4 the invert elevation is held up and a straight grade maintained between CI-3 and CI-5.

By a series of similar calculations the remaining pipe sizes and gradients for the sewer are determined until manhole No. 1 is reached.

By reference to the plan layout of this sewer it is noted that at CI-9 the line is run diagonally across Sixth Street to a manhole placed at the northeast corner of the street intersection and that the runoff intercepted in CI-10 is proposed to be carried in a 21 inch line directly from the inlet to MH-1. If the main line were run from CI-9 to CI-10 with the outfall line running north on Sixth Street the direction of flow would be turned back against itself in CI-10 or in a combination inlet and manhole placed at that location. Consequently, the line has been run from CI-9 to MH-1 which avoids the more than 90 degree turn at CI-10. Even 90 degree changes in direction of flow should be avoided because of head losses due to turbulence and entrance. This is particularly true in flat areas where every bit of available fall must be utilized to provide flow and not dissipated in faulty design.

In determining the size of the outfall Line C the designer is confronted with the problem of selecting the proper time of concentration at MH-1. As the sewer time between CI-10 and MH-1 is only a fractional part of a minute, the time of concentration at MH-1 will, for all practical purposes, be the same as the inlet time at CI-10, or 50 minutes. If the corresponding rainfall intensity of 2.02 in. per hr. is applied to the total drainage area, one to ten inclusive, or 48.61 acres with its corresponding CA of 24.31, a runoff of 49.1 c.f.s. is obtained. The complete design of Line A above MH-1 yields a time of concentration at MH-1 of 29.5 minutes and if the corresponding rainfall intensity of 2.80 in. per hr. is applied to the total drainage area, 48.61 acres, a runoff of 68.02 c.f.s. is obtained. Because of the 20.5 minutes difference in the two concentration times at MH-1, it is a logical conclusion that the runoff from drainage areas one to nine inclusive will pass MH-1 before water from the remote part of drainage area No. 10 reaches the manhole. For this reason, in determining the size of outfall Line C only a portion of drainage area No. 10 is used in combination with areas one to nine inclusive. By simple calculations we find that runoff from the 1,770 feet of the drainage area above CI-10 will reach MH-1 in the 29.5 minutes time of concentration through Line A. Scaling 1,770 feet up drainage area No. 10 from CI-10, it is found that the portion of drainage area 10 to be drained in 29.5 minutes terminates at approximately Eighth Street. This represents an area of 11.2 acres to be added to areas one to nine inclusive or a total of 42.51 acres which with the 0.5 runoff factor has a CA of 21.26. Multiplying this CA by 2.80 in. per hr., the intensity of rainfall for a time of 29.5 minutes, a runoff of 59.5 c.f.s. is obtained. This figure seems more reasonable for use in designing the outfall than either the low figure of 49.1 c.f.s. or the high of 68.0 c.f.s. The 42 inch line on 0.403% grade with a capacity of 64 c.f.s. and the discharge velocity of 6.6 ft. per sec. is proposed for the outfall. Manhole No. 2 is provided about halfway down the outfall. A standard CH-11-B head-wall is designated at the outlet to keep the bank out of the flowage way and to provide a finished appearance to the work. A 500 feet outfall channel carries the water from the sewer to a convenient disposal point.

A careful scrutiny of the preceding problem will reveal at least two apparent falacies, both of which have been introduced to bring out an important point in the storm sewer design and that is "Don't split hairs." The drainage areas have been calculated from dimensions scaled off the drainage map and are undoubtedly subject to minor discrepancies, but the degree of accuracy obtained is consistent with the runoff coefficient of 0.5 and the five year intensities of rainfall calculated by an imperical formula based on existing rainfall records. Additional records could show the constants in this formula to be in error. The line of demarcation between the areas is very uncertain and subject to change due to property improvements. Also, the selection of the runoff coefficient is one of engineering judgment and it is unlikely that two well qualified engineers would both select the same coefficient for a given area. It is also noted that at CI-2 and MH-1 different methods have been used in handling different times of concentration. At CI-2 an intensity of rainfall based on a time of concentration of 24.2 minutes has been applied to the total drainage area above CI-2, while at MH-1 an intensity of rainfall based on a time of concentration of 29.5 minutes, an interval less than the inlet time at CI-10 has been applied to all of the area drained by Line A together with only a portion of the drainage area No. 10. The simplier method used at CI-2 appears justifiable, as any difference in intensity applied to the small total area above CI-2 i.e. drainage area 1 plus 2, would result in only a small change in calculated runoff, whereas it has been shown that the same procedure applied at MH-1 would result in an appreciable difference in runoff.



$$I = \frac{b}{(t+d)^e}$$

$b_5 = 77$, $d_5 = 16.0$, $e = 0.870$

Approach Velocity $V_a = 1'/s$ $Q = CIA$

*L = Length of D.A. $t = L/60\,V_a$

Acres Drained			Total CA	Time of Concentration Minutes	I Inches per hour	Q c.f.s.
Drainage Area		Paved Residential C = 0.5				
No.	Ac.					
1	2.26	2.26	1.13	*750/60 = 12.5	4.16	4.70
2	5.57	5.57	2.79	1450/60 = 24.2	3.11	8.68
3	5.57	5.57	2.79	1200/60 = 20.0	3.41	9.51
4	4.83	4.83	2.42	1050/60 = 17.5	3.63	8.78
5	2.19	2.19	1.10	480/60 = 8.0 (Use 10)	4.53	4.98
6	1.61	1.61	0.81	600/60 = 10.0	4.53	3.67
7	3.42	3.42	1.71	730/60 = 12.1	4.26	7.28
8	1.87	1.87	0.94	650/60 = 10.8	4.40	4.14
9	3.99	3.99	2.00	780/60 = 13.0	4.12	8.24
10	17.30	17.30	8.65	3000/60 = 50.0	2.02	17.47

From	To	Drainage Area No.	Total D. A. Ac.	Total C. A.	Length ft.	Time of Concentration — Minutes: Along Sewer Line	Inlet Time	Used in Design	I In./Hr.	Q cfs	Design: Dia. In.	Slope %	Cap. cfs	Vel. ft/sec.	Remarks
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
C.I.-1	C.I.-2	1	2.26	1.13	117	* $\frac{750}{60} = 12.5$	12.5	12.5	4.16	4.70	18	.20	4.7	2.7	
C.I.-2	C.I.-3	1 & 2	7.83	3.92	283	* $\frac{1450}{60} = 24.2$	24.2	24.2	3.11	12.2	24	.30	12.4	3.9	
C.I.-3	C.I.-4	1, 2 & 3	13.40	6.71	115	$24.2 + \frac{283}{60 \times 3.9} = 25.4$	20.0	25.4	3.02	20.3	30	.45	28	5.6	
C.I.-4	C.I.-5	1 to 4 Incl.	18.23	9.12	285	$25.4 + \frac{115}{60 \times 5.6} = 25.7$	17.5	25.7	3.02	27.5	30	.45	28	5.6	
C.I.-5	C.I.-6	1 to 5 Incl.	20.42	10.21	115	$25.7 + \frac{285}{60 \times 5.6} = 26.5$	10.0	26.5	2.96	30.2	36	.23	32	4.6	
C.I.-6	C.I.-7	1 to 6 Incl.	22.03	11.03	285	$26.5 + \frac{115}{60 \times 4.6} = 26.9$	10.0	26.9	2.94	32.4	36	.23	32	4.6	
C.I.-7	C.I.-8	1 to 7 Incl.	25.45	12.74	115	$26.9 + \frac{285}{60 \times 4.6} = 27.9$	12.1	27.9	2.90	36.9	36	.30	37	5.3	
C.I.-8	C.I.-9	1 to 8 Incl.	27.32	13.67	285	$27.9 + \frac{115}{60 \times 5.3} = 28.3$	10.8	28.3	2.85	39.0	36	.35	40	5.6	
C.I.-9	M.H.-1	1 to 9 Incl.	31.31	15.66	135	$28.3 + \frac{285}{60 \times 5.6} = 29.1$	13.0	29.1	2.80	43.8	36	.43	44	6.2	
C.I.-10	M.H.-1	10	17.30	8.65	91	* $\frac{3000}{60} = 50.0$	50.0	50.0	2.02	17.5	21	1.20	17.5	7.2	
M.H.-1	Outlet	1 to 9 Plus †1,770' of DA-10	42.51	21.26		$29.1 + \frac{135}{60 \times 6.2} = 29.5$		29.5	2.80	59.5	42	.403	64	6.6	
			48.61	24.31				29.5	2.80	68.0					
			48.61	24.31				50.0	2.02	49.1					

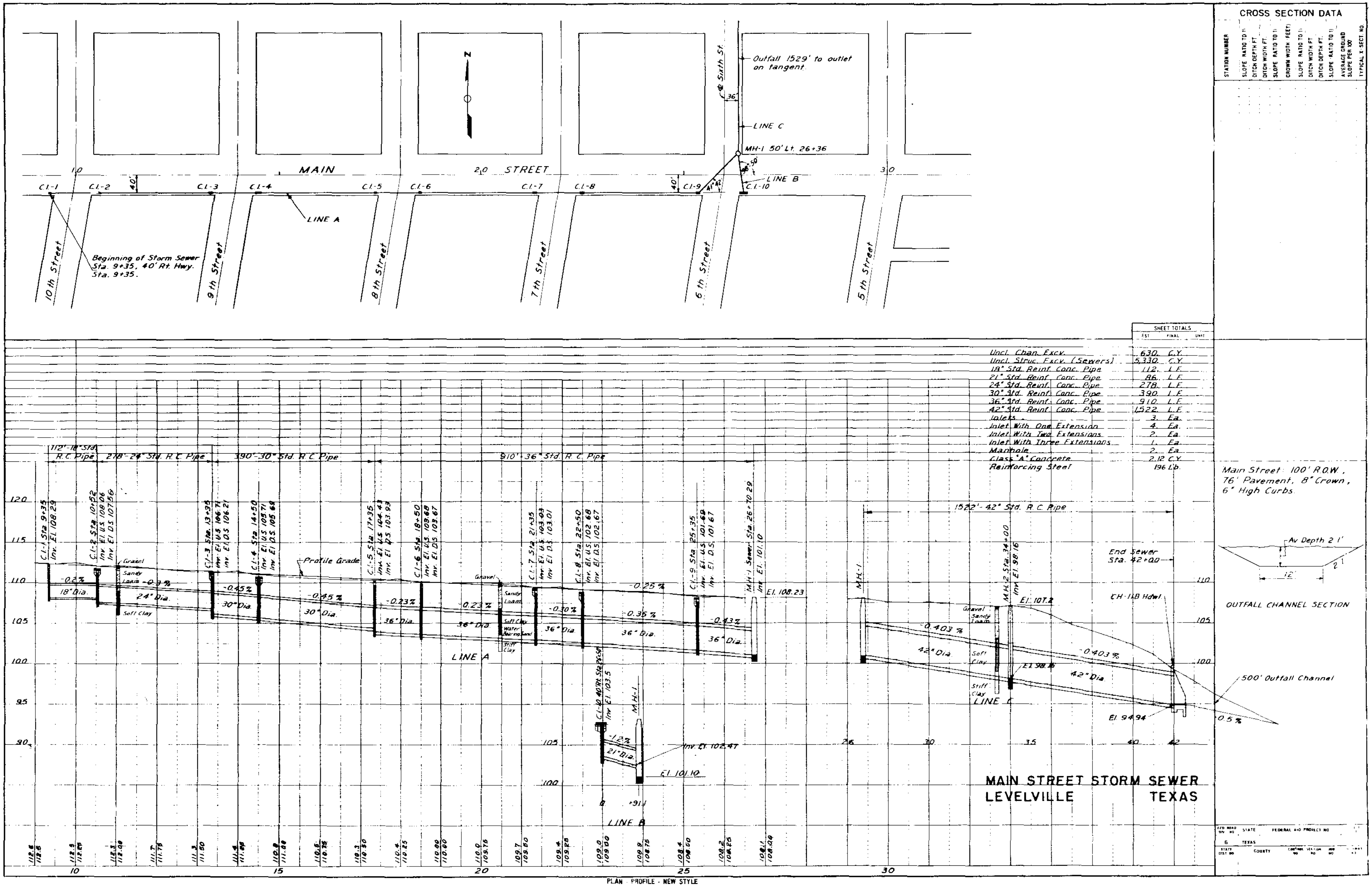
MAIN ST. STORM SEWER

LEVELVILLE TEXAS

* Overland Flow

† 1,770' Length of D.A. #10 drained to C.I.-10 in 29.5 Min. Time of Concentration at M.H.-1

STORM SEWER COMPUTATIONS


CROSS SECTION DATA
STATION NUMBER
SLOPE RATIO TO 1
DITCH DEPTH FT.
DITCH WIDTH FT.
SLOPE RATIO TO 1
CROWN WIDTH (FEET)
SLOPE RATIO TO 1
DITCH WIDTH FT.
DITCH DEPTH FT.
SLOPE RATIO TO 1
AVERAGE GROUND SLOPE PER 100'
TYPICAL X-SECT NO
MAIN
STREET
10th Street
9th Street
8th Street
7th Street
6th Street
5th Street
℄ Sixth St.
Outfall 1529' to outlet on tangent
LINE C
MH-1 50' Lt. 26+36
LINE B
LINE A
C.I.-1
C.I.-2
C.I.-3
C.I.-4
C.I.-5
C.I.-6
C.I.-7
C.I.-8
C.I.-9
C.I.-10
Beginning of Storm Sewer Sta. 9+35, 40' Rt. Hwy. Sta. 9+35.
SHEET TOTALS
EST FINAL UNIT
Uncl. Chan. Excv. 630. C.Y.
Uncl. Struc. Excv. (Sewers) 5,330. C.Y.
18" Std. Reinf. Conc. Pipe 112. L.F.
21" Std. Reinf. Conc. Pipe 86. L.F.
24" Std. Reinf. Conc. Pipe 278. L.F.
30" Std. Reinf. Conc. Pipe 390. L.F.
36" Std. Reinf. Conc. Pipe 910. L.F.
42" Std. Reinf. Conc. Pipe 1,522. L.F.
Inlets 3. Ea.
Inlet With One Extension 4. Ea.
Inlet With Two Extensions 2. Ea.
Inlet With Three Extensions 1. Ea.
Manhole 2. Ea.
Class "A" Concrete 2.12 C.Y.
Reinforcing Steel 196 Lb.
112'-18" Std. R.C. Pipe
278'-24" Std. R.C. Pipe
390'-30" Std. R.C. Pipe
910'-36" Std. R.C. Pipe
1522'-42" Std. R.C. Pipe
Profile Grade
18" Dia.
24" Dia.
30" Dia.
36" Dia.
42" Dia.
21" Dia.
Gravel
Sandy Loam
Soft Clay
Stiff Clay
Water Bearing Sand
End Sewer Sta. 42+00
CH-11B Hdwl.
El. 108.23
El. 101.2
El. 98.16
El. 94.94
Inv. El. 102.47
El. 101.10
500' Outfall Channel
0.5%
Main Street: 100' R.O.W., 76' Pavement, 8" Crown, 6" High Curbs.
Av. Depth 2.1'
12'
OUTFALL CHANNEL SECTION
MAIN STREET STORM SEWER
LEVELVILLE TEXAS
PLAN - PROFILE - NEW STYLE