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16. Abstract

The feasibility of building a large scale scour modeling facility to help evaluate the 26,018 bridges over water which exist in Texas is studied. First, the scour problem in Texas is reviewed and tends to indicate that many bridges are built on clay. Second, the fundamental laws of hydraulic and soil modeling are detailed. These laws show that it is not possible to scale all the components of the problem properly. It is also shown that when the model soil has particles smaller than 0.1 mm, additional difficulties occur. This makes the physical modeling of clays nearly impossible. Third, five bridge case histories are used to calculate the necessary size of scaled models. Scales of 1/15 to 1/100 would be used in the proposed facility. Fourth, the results of a paper survey and visits of prominent scour modeling laboratories in the USA are presented. They show that these laboratories are relatively well equipped. Fifth, the new facility is designed and the cost is estimated at $6.7 M. Finally, the advantages and disadvantages of building a new facility versus using existing facilities are outlined.

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FEASIBILITY STUDY FOR HYDRAULIC MODELING FACILITY FOR SCOUR PROBLEMS

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Texas Department of Transportation
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TEXAS TRANSPORTATION INSTITUTE
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IMPLEMENTATION STATEMENT

The implementation of this project is in the hands of the Texas Department of Transportation (TxDOT). TxDOT needs to decide if it wants to build a scour facility or not. The estimated cost of such a facility as well as its advantages and disadvantages are included in the Conclusions of this report. These conclusions are reached on the basis of literature search, data collection, numerical, similitude and dimensional analysis, laboratory visits and expert interviews and cost estimating. It is the opinion of the researchers that the facility should be built if TxDOT needs to simulate 2 bridges or more per year. It is also felt that research needs to be conducted to develop alternatives to the physical modeling approach. In particular, a site specific technique for the prediction of scour in clay is necessary since many Texas bridges are in clay and since no physical modeling approach is likely to give a reliable prediction in this case.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation. This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of the project was Jean-Louis Briaud, Texas P.E. # 48690.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new useful improvement thereof, or any variety of plant, which is or may be patentable under the patent laws of the United States of America or any foreign country.

ACKNOWLEDGMENTS

The sponsor of this project is the Texas Department of Transportation. The Project Director at TxDOT was Mr. Jay Vose; his help, constructive criticisms, and very positive attitude created an excellent atmosphere during the course of this research and Ms. Melinda Luna for her help and information. We also wish to thank Mr. Antony Schneider and Mr. George Odom of TxDOT, Mr. Peter Smith, formerly with TxDOT, and Mr. David Dunn of the United States Geological Survey (USGS), Texas district.

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LIST OF SYMBOLS AND ABBREVIATIONS

s	=	Shear strength
σ	=	Normal stress of the soil grains
u	=	Pore water pressure
ϕ	=	Angle of internal friction between the soil grains
s_u	=	Undrained shear strength
c	=	Cohesion
τ_o	=	Unit tractive force
γ	=	Specific weight of water
R	=	Hydraulic mean radius
S	=	Slope of the channel bottom
u_*	=	Shear velocity
ρ_w	=	Density of water
ρ_s	=	Density of soil
$(\tau_o)_{cr}$	=	Critical shear stress for initiation of motion
γ_s	=	Specific weight of sediment particles
d	=	Grain size diameter
ν	=	Kinematic viscosity
R_*	=	Boundary Reynolds number
ψ	=	Shields parameter
F_r	=	Froude number
g	=	Acceleration due to gravity
D	=	Depth of water
V	=	Velocity of water
R_e	=	Reynolds number
μ	=	Dynamic viscosity
F	=	Force on the fluid flow
M	=	Mass of the fluid
a	=	Acceleration of the fluid
F_V	=	Viscous force
F_g	=	Gravity force
n	=	Mannings roughness coefficient
L	=	Length
L_H	=	Horizontal scale length
L_V	=	Vertical scale length

d	=	Particle size
W	=	Weight of water element
b	=	Width of the water element
h	=	Height of water element
α	=	Angle of the bed
χ	=	Ratio of u_* to V_s
A_s	=	Cross sectional area of sump
Q	=	Discharge
H	=	Total head
η	=	Efficiency of the pump
c_d	=	Coefficient of discharge
a_1	=	Cross sectional area of the pipe
a_2	=	Cross sectional area of the throat of the venture meter

A suffix 'r' indicates scale ratio; a suffix of 'm' indicates the parameter related to model; a suffix of 'p' indicates the parameter related to prototype; a subscript of H indicates the parameter is related to horizontal scale; and a subscript of V indicates a parameter related to vertical scale.

SUMMARY

This project entitled "Feasibility Study for Hydraulic Modeling Facility for Scour Problems" was undertaken to determine if the development of a scour facility in Texas would be a sound idea. The perceived need was based on the following reasons and the questions:

1. The TxDOT has a need to evaluate bridges for scour problems.
2. There are no adequate facilities for modeling scour problems in Texas.
3. Are the hydraulic modeling facilities available elsewhere appropriate for Texas problems?
4. What are the required dimensions for a facility dedicated to the Texas scour problem?
5. What would be the approximate cost of such a facility?

The work consisted of a study of the Texas scour problem including the hydraulic and soil characteristics of the rivers in Texas, a study of fundamental principles of hydraulic and soil modeling, model analysis by similitude theory of five bridge case studies in Texas, discussions with recognized scour experts, and a survey and visit to four leading scour facilities in the country.

The following conclusions were reached :

1. The facility should have two basins: a 2-D flume for local scour studies and a 3-D basin for global scour studies.
2. The 2-D flume should be above ground, 36.6 m long (120 ft), 6.1 m wide (20ft), and 3.6 m deep (12 ft). The sump should be below ground; it should surround the flume and be 3 m (10 ft) wide and 3 m (10 ft) deep. A 240 HP pump delivering 2.8 m^3/sec (100 cfs) is necessary to feed this flume.
3. The 3-D basin should be above ground, 45 m (150 ft) long, 30 m (100 ft) wide and 1 m (3 ft) deep. The sump should be below ground under the center of the basin, parallel to the 50 m side of the basin; it should be 3 m wide (10 ft) and 1.8m (6 ft) deep. A 24 HP pump delivering 0.4 m^3/sec (12 cfs) is necessary to feed this basin.
4. The 2-D flume would allow local scour models with undistorted scales in the range of 1/15 to 1/25.
5. The 3-D basin would allow general scour models with undistorted scales in the range of 1/50 to 1/100.

6. The cost of the facility and its major components is estimated to be as follows:

The overall facility	= \$6.70 M
The building	= \$4.25 M
The 3-D basin with sump and pump	= \$0.19 M
The 2-D basin with sump and pump	= \$0.35 M
Measuring instruments	= \$0.61 M

7. The advantages and disadvantages of this facility are:

Advantages	Disadvantages
1. Availability	1. Initial cost
2. Develop local expertise	2. Delay until built
3. Latest technology	3. Inexperienced personnel at first
4. Very large scale	
5. Low overhead	
6. Easy contracts	
7. Short travel time	

8. Existing facilities do not compare favorably with the facility described above. However, a few of them can provide very valuable data on scour modeling at a reasonably large scale.
9. The advantages and the disadvantages of the existing facilities are :

Advantages	Disadvantages
1. No delay for use	1. Higher overhead
2. No initial cost	2. No local expertise developed
3. Experienced personnel	3. Older equipment
	4. Longer travel time
	5. First come first serve availability

10. The decision should be based on the estimated need in the next 10 to 20 years for such a facility by TxDOT and neighboring states. Decreasing the cost by using an existing building would make a big difference. It should also be kept in mind that Texas rivers have a mixture of sand and clay beds, and the usefulness of modeling facilities for scour in clay is limited.

1. INTRODUCTION

Scour in rivers is a major problem to be addressed by the Departments of Transportation across the country. The Federal Highway Adminstration requires that all bridges over waterways be evaluated for scour by January 1997. Texas has close to 40,000 such bridges; the task is obviously enormous, yet crucial. TxDOT has decided to take advantage of this effort on scour to also evaluate the research on this topic.

The overwhelming majority of research projects on this topic have concentrated on the experimental approach. This is due to the complexity of the problem in terms of hydraulics and sediment transport. Physical models with a scale varying from 1/10 to 1/200 are tested in large basins.

This project entitled "Feasibility Study for Hydraulic Modeling Facility for Scour Problems" was undertaken to determine if the development of a scour facility in Texas would be a sound idea. The perceived need was based on the following reasons and the questions:

1. The TxDOT has a need to evaluate bridges for scour problems.
2. There are no adequate facilities for modeling scour problems in Texas.
3. Are the hydraulic modeling facilities available elsewhere appropriate for the Texas problems?
4. What are the required dimensions for a facility dedicated to the Texas scour problem?
5. What would be the approximate cost of such a facility?

The following chapters present the results of the study. First, the Texas scour problem is described. Second, a background is given on hydraulic modeling. Third, a similar background is given on soil modeling. Fourth, five case studies are analyzed to determine an appropriate model size in each case. Fifth, the results of a survey of existing facilities for scour modeling in the USA are presented. Sixth, the characteristics of a new TXDOT Scour Facility including dimensions and costs are determined and presented. Finally conclusions are presented on the advantages and disadvantages of such a facility

2. TEXAS SCOUR PROBLEM

2.1. HYDRAULIC CONDITIONS OF RIVERS AND FLOOD PLAINS IN TEXAS

The State of Texas has 15 river basins and 8 coastal basins. The 23 river and coastal basins have approximately 3700 streams and tributaries and 128,800 km (80,000 miles) of stream bed (Moody et al. 1985). Geological and climatological features may vary dramatically from the head water to outlets into other rivers or at the Gulf of Mexico. For instance, long term average annual precipitation contributing to runoff and surface water supplies varies dramatically across the state, ranging from 1.4 m (56 inches) near Beaumont in East Texas to 0.2 m (8 inches) in far West Texas near El Paso (USGS, 1988-89). Average annual runoff is about 6.04×10^{10} m^3 (49 million ac-ft). The average annual precipitation and annual stream flow are shown in the Figures 2.1. and 2.2. Between 1940 and 1970, state wide runoff varied from an average 7.027×10^{10} m^3/year (57 million ac-ft/year) during the wettest period (1940-50) to as little as 2.835×10^{10} m^3/year (23 million ac-ft/year) during the most severe recorded state wide drought of the early 1950s (Moody et al. 1985). There are currently 188 major reservoirs and 6.16×10^{6} m^3 (5000 ac-ft) or greater storage capacity in Texas. Figure 2.3 illustrates the 23 major river and coastal basins and zones. Some of the major features of rivers and their basins are discussed briefly in the following sections..

2.1.1. CANADIAN RIVER

The Canadian River heads in northeastern New Mexico, flows across the Texas Panhandle, and merges with the Arkansas River in eastern Oklahoma. The total length of the river is 1459 km (906 miles). The Texas part of the basin comprises a total area of about 32,920 km^2 (12,700 mi^2) out of the total drainage area of 123,656 km^2 (47,705 mi^2). The average discharge (arithmetic average of annual average discharges during the period of analysis) during the period (1939-83) of analysis is 9.3 m^3/sec (331 ft^3/sec) near Amarillo where the drainage area is 39,856 km^2 (15,376 mi^2). The 100-yr. flood at that location is 3780 m^3/sec (135,000 ft^3/sec). Average annual runoff to the Canadian River during the 26-year period (1939-1964) ranged from 11,890 m^3/km^2 (25 ac-ft/mi^2) in the western part of the basin to 21,401 m^3/km^2 (45 ac-ft/mi^2) in the eastern part of the basin (Moody et al. 1985). Large floods occur infrequently in the basin, and these floods are characterized by rapid rise and fall and high stream velocities.

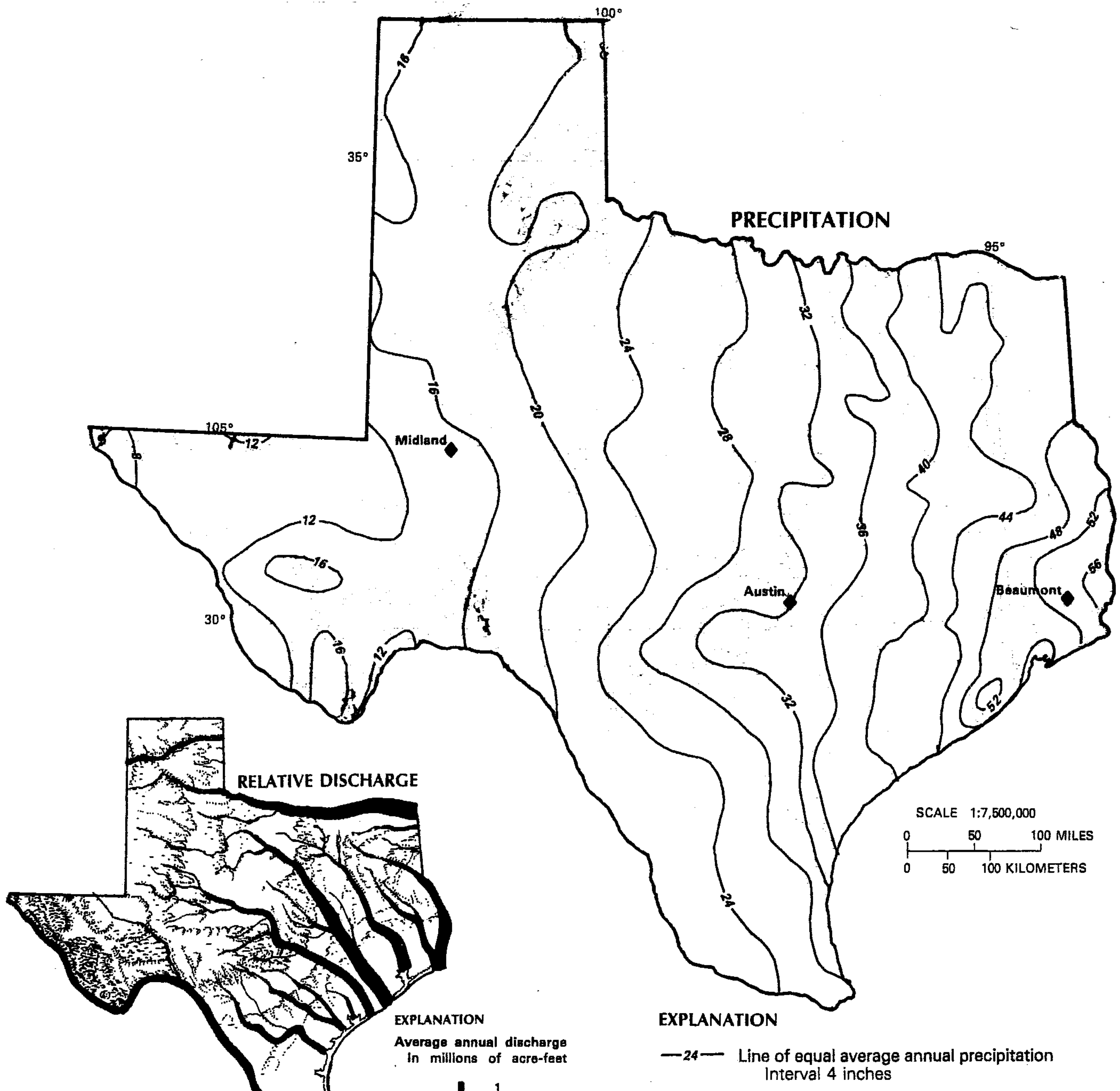

Figure 2.1. Average Annual Precipitation in Texas (Moody et al. 1985)

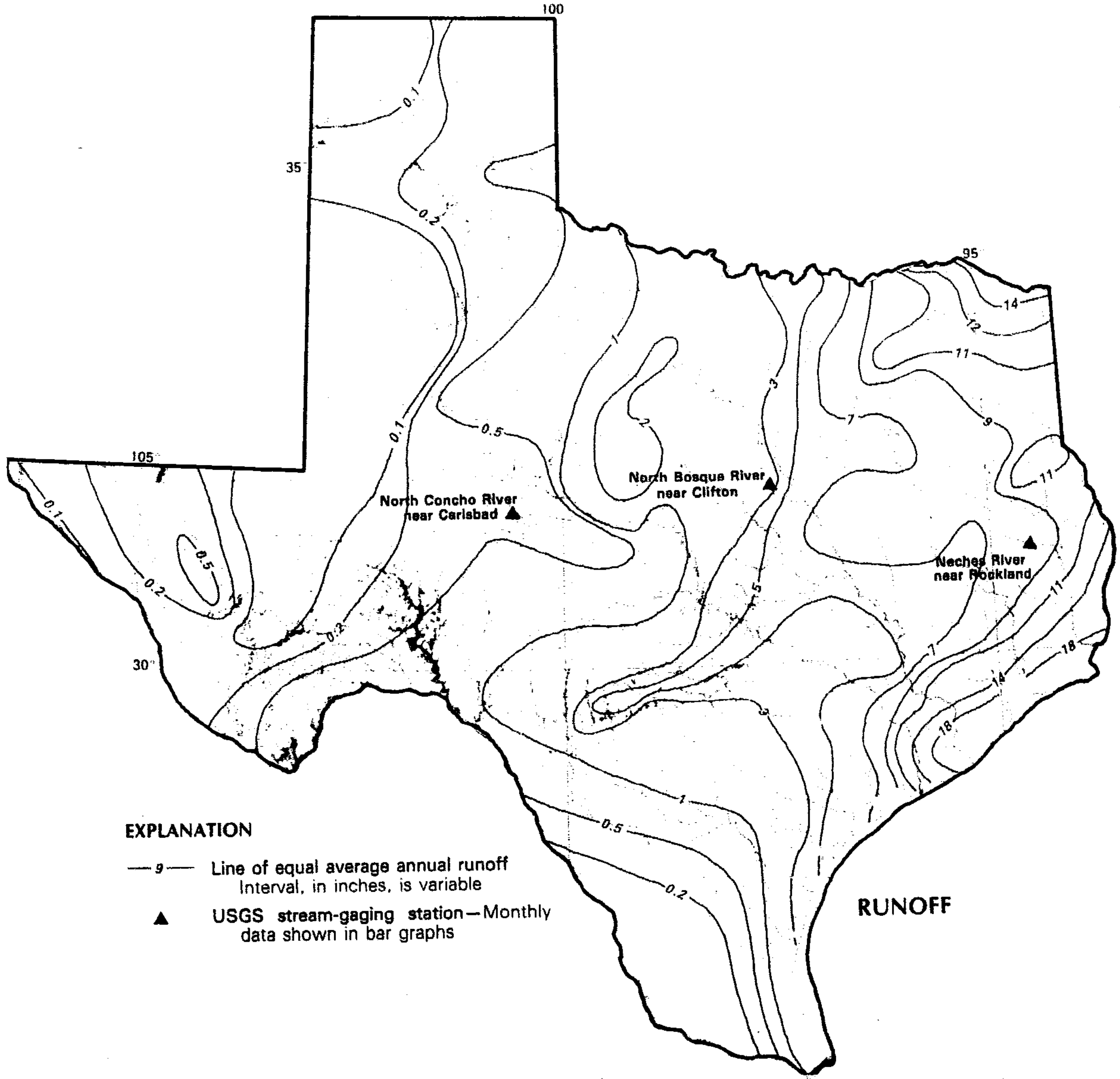

Figure 2.2. Average Annual Runoff in Texas (Moody et al. 1985)

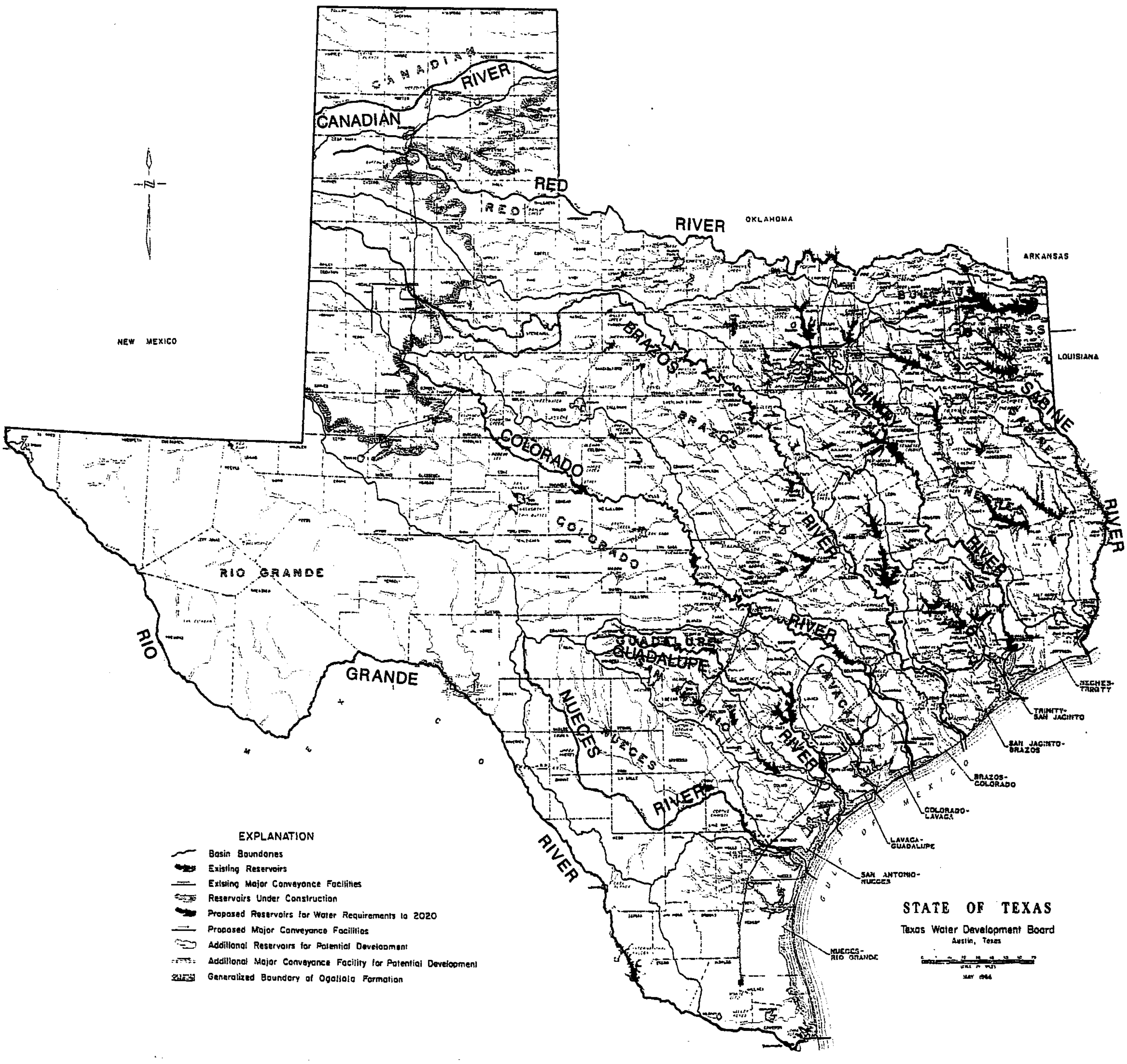

Figure 2.3. River and Coastal Basins in Texas (TWDB, 1968)

2.1.2. RED RIVER

The total length of the Red River is 2,190 km (1,360 miles). The Red River is bounded on the north by the Canadian River basin and on the south, from west to east by the Brazos, Trinity, and Sulfur River basins (TWDB, 1968). Beginning in the High Plains of eastern New Mexico at an elevation of about 1,454 m (4,800 feet), the Red River flows east, forming the northern boundary of Texas east of the Panhandle. The average discharge is 60 m^3/sec (2117 ft^3/sec) near Terral, Oklahoma where the drainage area is 59,066 km^2 (22,787 mi^2). The total drainage area of the Red River upstream from the northeast corner of Texas is 124,499 km^2 (48,030 mi^2). The total drainage area of the basin in Texas is 63,411 km^2 (24,463 mi^2). Average annual runoff within the basin in Texas ranges from more than 380,463 m^3/km^2 (800 acre-ft/mi^2) at the northeast corner of the state to less than 23,779 m^3/km^2 (50 acre-ft/mi^2) in contributing areas of the basin west of the 100th meridian. Large floods occur infrequently in the upper part of the Red River.

2.1.3. TRINITY RIVER

The Trinity River basin is bounded on the north by the Red River basin, on the east by the Sabine and Neches River basins and the Neches-Trinity Coastal basin, and on the west by the Brazos and San Jacinto River basins and Trinity San Jacinto Coastal basins (TWDB 1968). West Fort Trinity River rises in southeastern Archer County at an elevation of about 364 m (1,200 ft) and flows southeasterly to be joined successively by Clear Fork at Fort Worth and Elm Fork at Dallas. The total drainage area of the basin at the mouth of the river is 46,578 km^2(17,969 mi^2). Average annual runoff ranges from the maximum of about 309,126 m^3/km^2 (650 ac-ft/mi^2) near the mouth of the river to a minimum of about 47,558 m^3/km^2 (100 ac-ft/mi^2) near the head waters. The Trinity River basin has widely varying flood characteristics. In the upper basin, floods rise and fall rapidly and with higher velocities than floods in the lower basin. However, large floods have occurred throughout the basin causing extensive and costly damage. Major flooding has occurred on the average of once every four years in the upper basin, and about every five years in the lower basin. The average discharge at Romayor (near its mouth) is 210 m^3/sec (7,417 ft^3/sec) and the drainage area is 44,548 km^2(17,186 mi^2) (Moody et al. 1985).

2.1.4. BRAZOS RIVER :

The Brazos River originates in the high plains of New Mexico and discharge to the Gulf of Mexico. The total length of the river is 1353 km (840 miles). The Brazos River is bounded on the north by the Red River basin on the east by the Trinity and San Jacinto River basins and the San Jacinto-Brazos Coastal basin, and on the south and west by the Colorado River basin and the Brazos Colorado coastal basin. The basin has a total drainage area of 118,130 km^2 (45,573 mi^2) of which 42,840 km^2 (42840 mi^2) is in Texas. The Brazos River basin varies in width from about 113 km (70 miles) in the High Plains to 177 km (110 miles) in the vicinity of Waco (Moody et al. 1985). Average discharge at the mouth of the Brazos River is 207 m^3/sec (7,320 ft^3/sec). Runoff decreases east to west.

2.1.5. COLORADO RIVER

The length of the Colorado River is 1,393 km (865 mi). The Colorado River basin is bounded on the north and the east by the Brazos River basin and Brazos-Colorado Coastal basin, and on the west and south by the Rio Grande, Nueces, Guadalupe, and Lavaca River basins (TWDB, 1968). The river flows southeasterly along its entire length. The basin has a total drainage area of 109,692 km^2 (42,318 mi^2) at the mouth, of which 103,407 km^2 (39,893 mi^2) is in Texas. Average annual runoff in the basin ranges from a maximum of about 166,452 m^3/km^2 (350 ac-ft/mi^2) near the mouth of the Colorado River to less than 23,779 m^3/km^2 (50 ac-ft/mi^2) in the contributing area of the basin west of Coke County. There have been many large floods throughout the Colorado basin. Extensive overflows are restricted mostly to the coastal plains downstream from Austin. Average discharge at the mouth of the Colorado River is 68 m^3/sec (2,395 ft^3/sec) (Moody et al. 1985).

2.1.6. GUADALUPE RIVER

The Guadalupe River basin is bounded on the north by the Colorado River basin, on the east by the Lavaca River basin and Lavaca-Guadalupe Coastal basin, and on the west and south by the Nueces and San Antonio River basins. The total drainage area of the River basin is 15,734 km^2 (6,070 mi^2). Average annual runoff in the Guadalupe River basin ranges from a maximum of about 95,116 m^3/km^2 (200 ac-ft/mi^2) in the eastern part of the basin to a minimum of about 47,558 m^3/km^2 (100 ac-ft/mi^2) in the western part of the basin (National Water Summary, 1985). The average discharge at the Spring branch is 8.80 m^3/sec (311 ft^3/sec).

2.1.7. FLOOD PLAINS IN TEXAS

Most of the State of Texas is made of plains with cohesive soils. TxDOT provided a typical range of values of geometric parameters for rivers in the State of Texas which are given below.

Average channel velocity	=	0.30 to 3.03 m/sec (1 to 10 ft/sec)
Channel discharge	=	85 to 5,097 m^3/sec (3000 to 180,000 ft^3/sec)
Flood plains width	=	0.91 to 6.44 km (300 ft to 4 miles)
Maximum water depth	=	1.53 to 15.3 m (5 to 50 ft) (not including scour depth)

Damaging floods have occurred frequently throughout Texas resulting in serious economic losses. In the eastern part of Texas where rainfall is abundant, streams are commonly characterized by broad, flat valleys. Runoff is comparatively slow and stream velocities are generally low. These conditions generally produce broad, flat-crested floods which move slowly in the lower regions of the basins. Runoff is more rapid in the central and western parts of Texas due to steep to moderately steep slopes; high peak flows with higher stream velocities occur there.

2.2. GEOLOGY AND SOIL CONDITIONS IN TEXAS

The earth cooled down sufficiently to form the first hard rock crust 4.6 billion years ago. Since that time, weathering has transformed some of the rocks into soils. Those soils either stayed in place (sedentary or residual soils) or were transported (transported soils) (Figure 2.4). The transport mechanisms are erosion due to water, wind, or ice. Alluvium soils are transported by water; dunes and loess are transported by wind; till or glacial drift are transported by glaciers; and colluvium soils are moved downhill by gravity. Soils have widely varying grain sizes. Clays have many particles smaller than 0.002 mm (0.000078 in), silts have many particles in the range 0.002 mm (0.000078 in) to 0.075 mm (0.0029 in), sands have many particles in the 0.075 mm (0.0029 in) to 4.75 mm (0.187 in) range, and gravels have many particles larger than 4.75 mm (0.187 in).

Geologic history goes back to the first hardening of the molten rock and is shown in Tables 2.1 and 2.2, as well as in Figure 2.5 for the soils of Texas. Also, the physiography of Texas is shown in Figure 2.6.

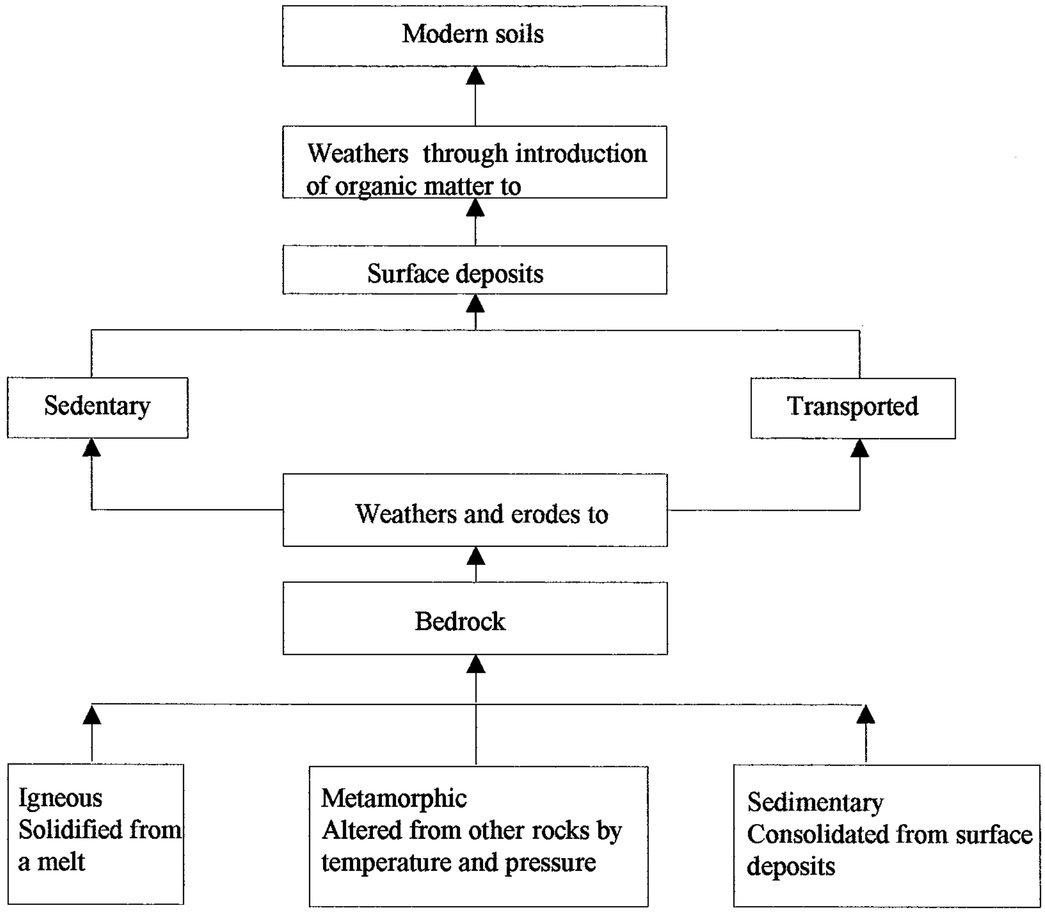

Figure 2.4. Diagram Illustrating the Formation of Modern Soils (Hunt, 1972)

Table 2.1. Geologic Time (Hunt, 1972)

Millions of years ago	**Eras**	**Periods**
Today	Cenozoic (Recent Life)	Quaternary
50		Tertiary
100	Mesozoic (Middle Life)	Cretaceous
150		Jurassic
200		Triassic
250	Paleozoic (Ancient Life)	Permian
300		Pennsylvanian
350		Mississippian
400		Devonian
450		Silurian
500		Odrovician
550		Cambrian
600	Precambrian	
4,700		

Table 2.2. Cenozoic Time (Hunt, 1972)

Period	Epoch	Glaciation	Interglaciation	Years ago (Estimated)
Quaternary	Holocene			Today
	Pleistocene	Wisconsinan		11,000
			Sangamon	70,000
		Illinoian		
			Yarmouthian	
		Kansan		
			Aftonian	750,000
		Nebraskan		
			Blancan ?	1,000,000 ?
Tertiary	Pliocene			3,000,000
	Miocene			10,000,000
	Oligocene			
	Eocene			30,000,000
	Paleocene			60,000,000

The soils of Texas formed by repeated marine regressions and transgressions finally ending with a major regression. This type of low energy geologic environment favors the deposition of very fine particles. As a result, many clay deposits are found in Texas. This is exemplified by Figures 2.7.a and 2.7.b.

Hallmark et al. (1986) gathered data on Texas soils between the ground surface and a depth of 2 to 3 m (6.6 ft to 10 ft). They indicate the soil type as well as many other index properties. The data shows that approximately 80% of the 0.2 m (0.66 ft) deep zone is made of clay. This and the geology of Texas tends to show that scour in clay for Texas rivers is likely to be an important problem.

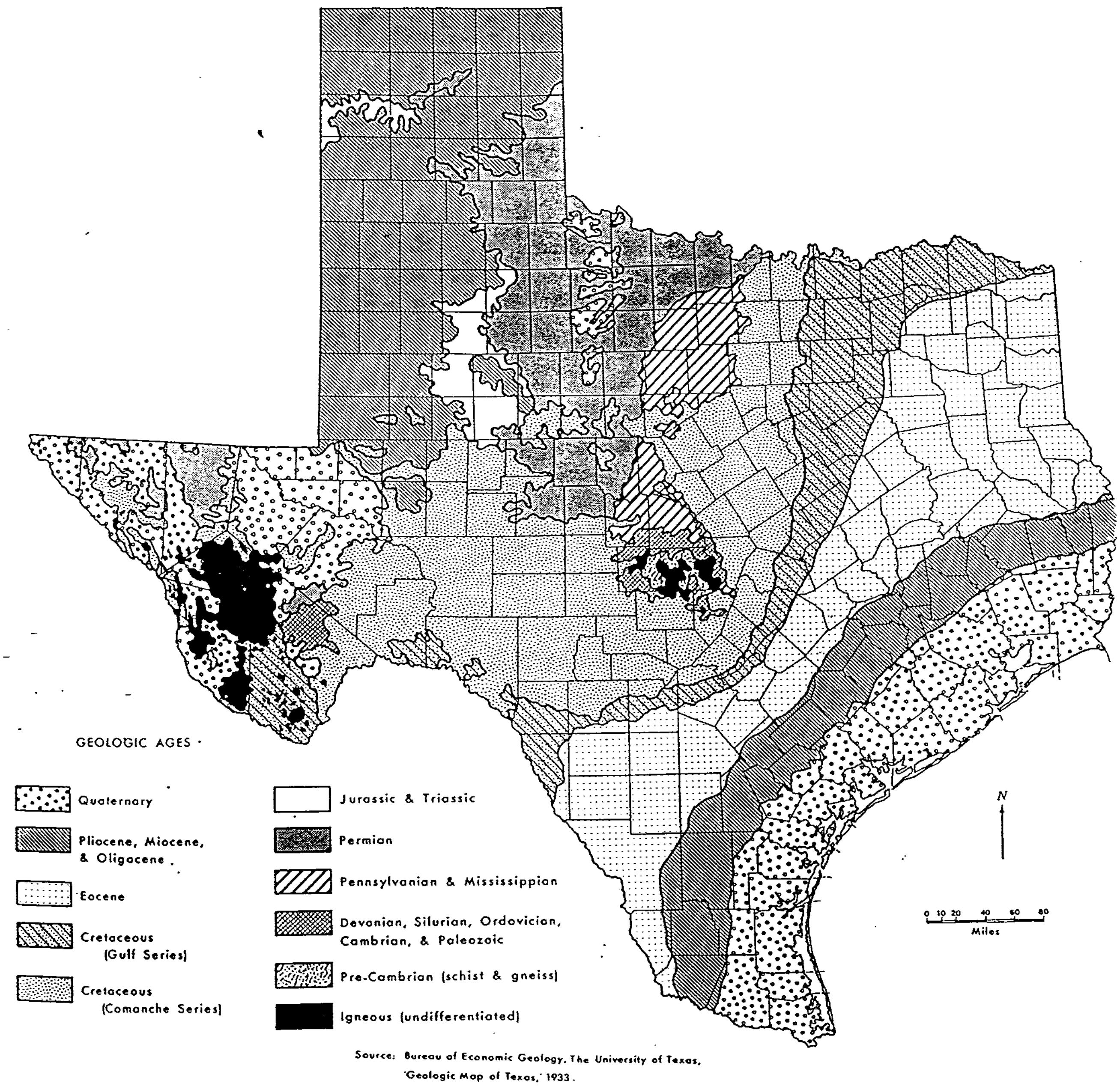

Figure 2.5. Surface Geology of Texas
(Arbingast et al. 1976)

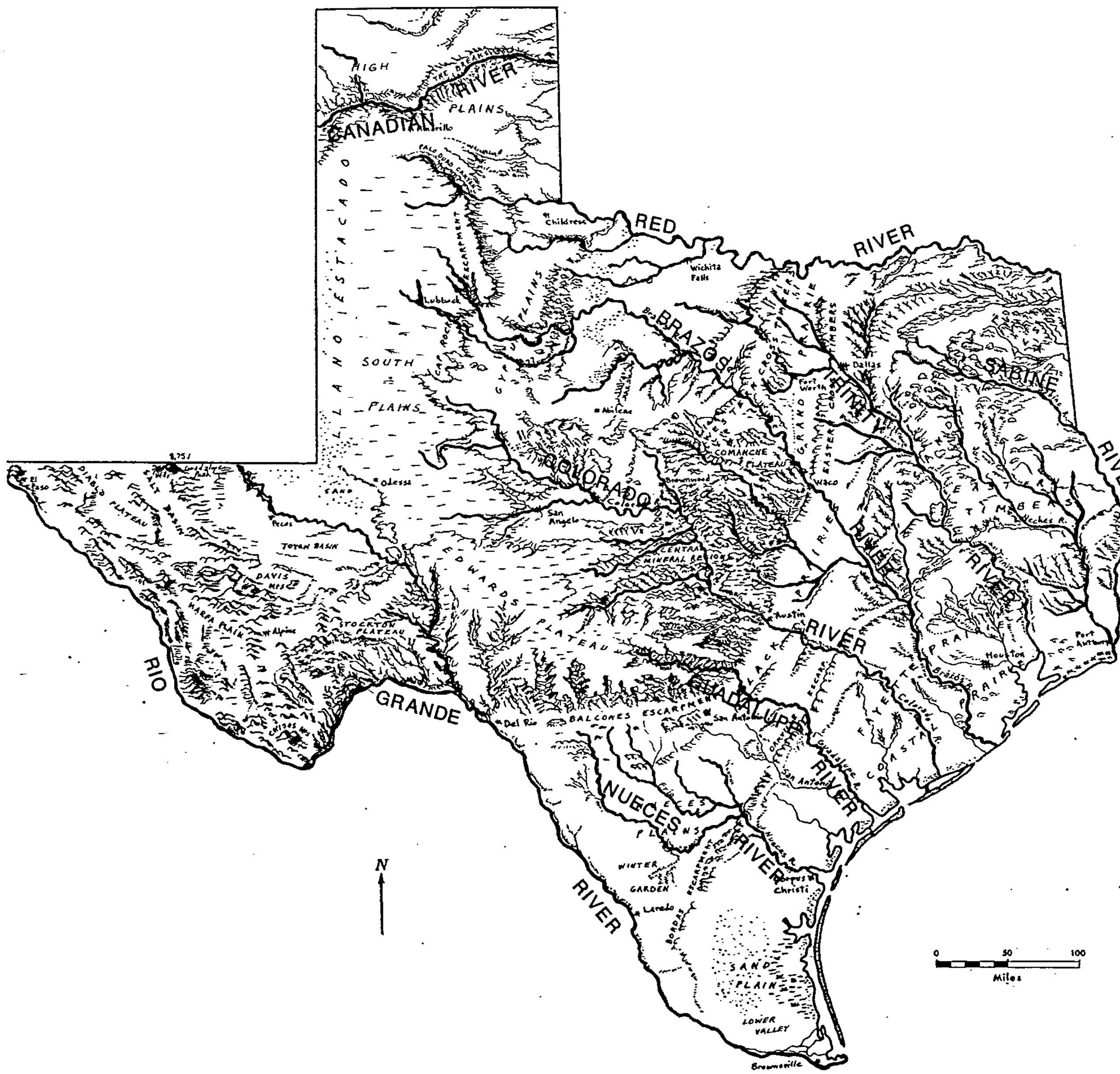

Figure 2.6. Physiography of Texas
(Arbingast et al. 1976)

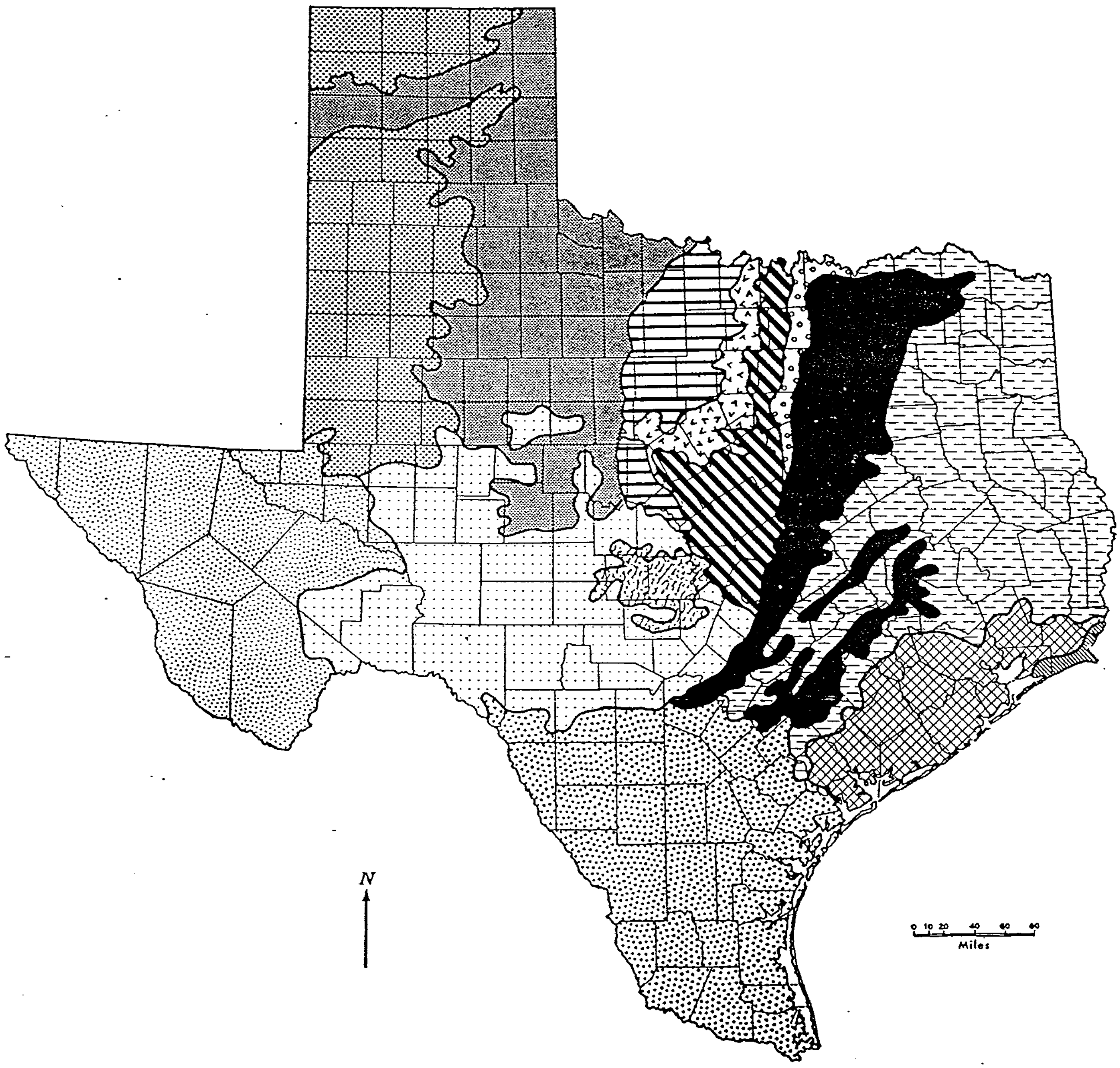

Figure 2.7. (a) Generalized Soils of Texas

(Source : Texas Agricultural Experiment Station, Types of Farming in Texas, Bulletin 964, 1960)

EAST TEXAS TIMBERLANDS

Uplands--Light-colored, acid, sandy loams and sands, some red soils.
Bottomlands--Light-brown to dark-gray, acid, sandy loams, clay loams, and some clays.

COAST MARSH

Light- and dark-colored, acid sands, sandy loams, and clays.

COAST PRAIRIE

Uplands--Dark-colored, neutral to slightly acid clay loams and clays, with some lighter colored sandy loams; acid soils mostly east of Trinity River.
Bottomlands--Reddish-brown to dark-gray, calcareous clay loams and clays.

BLACKLAND PRAIRIE

Uplands--Dark-colored calcareous clays. Some grayish-brown, acid sandy loams and clay loams along eastern edge of the major prairie and interspersed in the minor prairies.
Bottomlands--Dark-gray to reddish-brown calcareous clay loams and clays.

EAST CROSS TIMBERS

Light-colored, acid loamy sands and sandy loams.

GRAND PRAIRIE

Uplands--Dark-colored, deep-to-shallow and stony calcareous clays over limestone.
Bottomlands--Reddish-brown to dark-gray clay loams and clays.

WEST CROSS TIMBERS

Light-colored, slightly acid sandy loams, loamy sands, and sands.

NORTH CENTRAL PRAIRIES

Reddish-brown to grayish-brown, neutral slightly acid sandy loams and clay loams, some areas of stony soils.

CENTRAL BASIN

Reddish-brown to brown, neutral to slight acid gravelly and stony sandy loams.

RIO GRANDE PLAIN

Uplands--Dark calcareous to neutral cla and clay loams. Reddish-brown, neutral slightly acid sandy loams. Grayish-brown neutral sandy loams and clay loams; some saline soils near coast.
Bottomlands--Brown to dark-gray, calcareous clay loams and clays; some saline soil

EDWARDS PLATEAU

Dark, calcareous stony clays and some clay loams.

ROLLING PLAINS

Dark-brown to reddish-brown, neutral to slightly calcareous sandy loams, clay loams and clays.

HIGH PLAINS

Dark-brown to reddish-brown neutral sands, sandy loams, and clay loams; some very shallow calcareous clay loams.

TRANS-PECOS

Uplands--Light reddish-brown to brown sands, clay loams, and clays, mostly calcareous, some saline; and rough stony lands.
Bottomlands--Dark grayish-brown to reddish brown calcareous clay loams, and clays, some saline.

Figure 2.7. (b) Legends for the Soils in Texas
(Arbingast et al. 1976)

2.2.1. BACKGROUND ON SOIL SHEAR STRENGTH

Soils are usually dealt with by distinguishing between cohesionless soils and cohesive soils. Cohesionless soils are frictional materials; their resistance to shear is linked directly to the normal stress on the failure plane (contact between grains). The shear strength law is (see Figure 2.8):

$$s = (\sigma - u)\tan\phi \qquad \text{Eq..2.1}$$

where s = the shear strength

$(\sigma - u)$ = the normal stress due to the buoyant weight of the soil grains

$\tan\phi$ = the coefficient of friction between soil grains

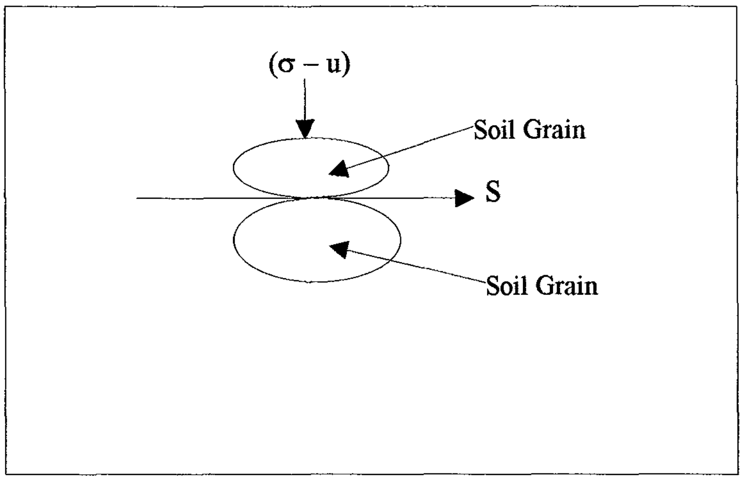

Figure 2.8. Shear Strength for Cohesionless Soils

If one considers only one particle on top of another (ground surface), the larger the particles are, the larger $(\sigma - u)$ is and the larger "s" is. Therefore, the larger the particles, the higher the resistance to scour. This is part of the reason why gravel resists scour better than sand.

Cohesive soils are fine grained soils. Since the grains are small (< 0.075 mm or 0.0029 in), water does not flow easily through the voids. As a result, two extreme types of behavior are considered: the undrained behavior and the drained behavior. The undrained behavior refers to the case where the soil is loaded fast enough not to allow any drainage. The undrained shear strength is:

$$s = s_u \qquad \text{Eq..2.2}$$

The value of S_u varies from a few kPa for very soft clay to over 200 kPa for hard clays. The drained behavior refers to the case where the soil is loaded slowly enough to allow complete drainage. The drained shear strength s is:

$$s = c + (\sigma - u)\tan\phi \qquad \text{Eq..2.3}$$

where c is the cohesion. The cohesion can be significant in over-consolidated clays. By comparing the shear stress imposed by the flowing water on the soil surface to the shear strength available, one can predict whether scour will occur or not.

2.3. DIFFERENT TYPES OF SCOUR

Scour is the erosive action of water which excavates and transports material from the stream beds and banks. The erosive action may start when the boundary shear stress exceeds a certain threshold value called the critical tractive force. Note that the shear stress is proportional to the square of the velocity. High velocities frequently occur at bridge piers and abutments.

From the point of view of bridge engineering, three types of scour can be recognized:

2.3.1. General Scour: Scour of the stream bed that occurs as a result of natural processes whether there is a structure or not.

2.3.2. Constriction Scour: Scour caused by the constriction of the waterway by placement of a structure.

2.3.3. Local Scour: Scour resulting directly from the interference of the structure with the natural flow. Local scour can occur concurrently with general and constriction scour.

Two scouring regimes may be identified according to the condition of sediment transport in the river:

2.3.4. Clear Water Scour: The bed material upstream of the scour area is at rest. The velocity and bed shear stresses away from the scour area are less than the threshold values for initiation of particle movement. In clear-water scour, the material is removed from the scour hole, but not replenished by the approach flow. As the scour depth increases, the strength of the flow decreases near the bottom of the scour hole until finally it can no longer dislodge particles. This condition represents the maximum scour to be attained by the prevailing flow conditions.

2.3.5. Live Bed Scour: There is sediment transport in the stream. The velocity and bed shear stresses upstream of the scour area are greater than the threshold values for the initiation of particle movement. In the scour hole, the strength of flow near the bottom decreases with increasing scour depth, but maximum scour is attained when the rate of sediment removal is equal to the rate of sediment transport into the scour hole by the stream. For a given pier and sediment, this depth is less than the maximum scour depth achieved in clear water conditions.

It is important to differentiate between clear-water scour and live-bed scour because both the development of the scour hole with time and the relationship between scour depth and approach flow velocity depend upon which type of scour is occurring. Figure 2.9 (a) shows variations of the scour depth with time in clear-water scour and live-bed scour. Clear-water scour approaches equilibrium asymptotically over a short period. This is because clear water scour occurs mainly in coarse bed material streams. Live-bed scour approaches equilibrium rapidly, and its depth fluctuates in response to the passage of bed features. Figure 2.9.(b) shows the scour depth as a function of shear velocity. Note that the maximum scour depth occurs at the transition between clear-water and live-bed scour. For live-bed scour, a hydraulic facility must have the capability to recirculate the soil-water mixture in order to simulate live-bed scour.

2.4. THE TEXAS SCOUR APPROACH

The State of Texas has 26,018 bridges over waterways (on system), one of the largest inventories in the nation in this category. The Federal Highway Administration (FHWA) has mandated that all state highway agencies evaluate existing and proposed bridges for susceptibility to scour related failure. This requirement must be completed before January 1997. Through an initial screening process (known as "Level 1" analysis), the Texas Department of Transportation (TxDOT) has identified 7,803 bridges as being possibly scour susceptible and in need of further evaluation.

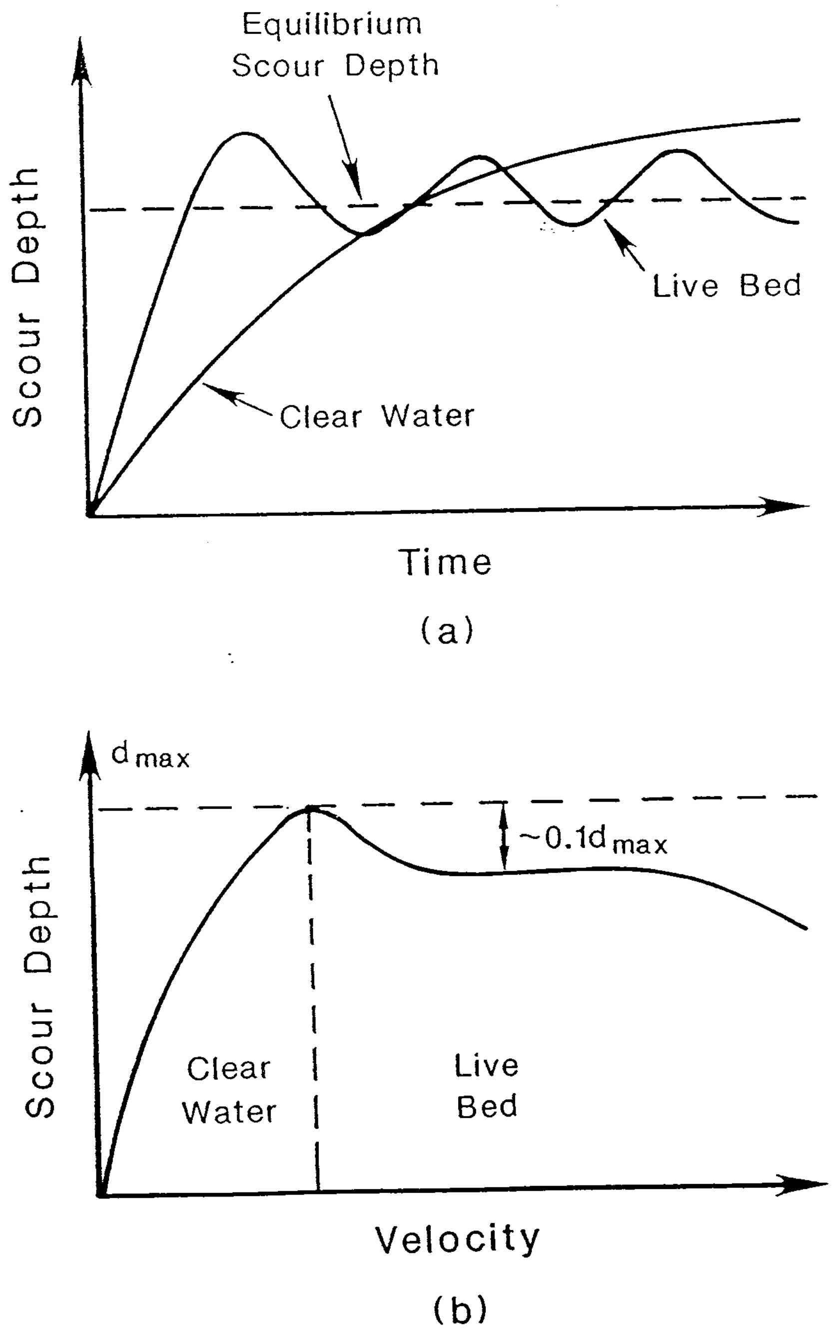

Figure 2.9. Scour Depth for a Given Pier and Sediment as a (a) Function of Time, (b) Function of Approach Velocity (Raudkivi et al. 1993)

The detailed evaluation involves hydraulic and scour analysis, often known as "Level 2" analysis. The important constraint for the Level 2 analysis is its cost. At an estimated cost of $ 10,000 or more per bridge the cost to TxDOT would be $ 20,000,000 per year over the next four years to complete the on-system bridges only. A state wide training program for scour evaluation for TxDOT engineers has increased the department's ability to perform evaluations.

To assess the preliminary stability of the Texas bridges, a plan is established which is known as the Texas Bridge Scour Evaluation And Mitigation Plan (TBSEAMP). This plan is carried out in two phases. The first phase takes place in the office. Necessary bridge plans, topographic maps of the site, and a questionnaire regarding hydraulic information of the bridge are prepared. The next phase is a field investigation. It includes channel bed measurements in the vicinity of the bridge, recording the measurements on the existing bridge plan set, and a geomorphic survey. In order to categorize the bridges, data obtained in the above two phases is used to complete a questionnaire titled "Scour Vulnerability Examination and Ranking Format" (SVEAR). This process (Level 1 analysis) provides an indication of the vulnerability of a bridge to scour and of the overall stability of the channel.

As a result of this process, TxDOT has found that :

Total bridges susceptible to scour	=	7,018
Bridges with known scour problem	=	621
Bridges with high susceptibility to scour	=	4,153
Bridges with medium susceptibility to scour	=	2,244
Bridges with low risk	=	3,186

Bridges over waterways and Average Daily Traffic (ADT) of over 150 vehicles per day (vpd) have been subjected to the SVEAR process. Each bridge inspected using SVEAR received a coding indicating its scour vulnerability. This coding was entered in the Bridge Inventory, Inspection, and Appraisal Program (BRINSAP) database. The prioritization procedure was based on elements of risk that pertain to scour vulnerability, foundation type, span type, and safety of traveling public. For example, a bridge receiving top "priority" for scour evaluation would have a known scour problem, high ADT, spread footings, and single spans.

HEC-18 (Richardson, et al. 1993) and HEC-20 (Lagasse, et al. 1991) are design manuals related to scour susceptibility. A bridge scour evaluation modeled with HEC-18 and HEC-20 consists of three stages:

1. A quantitative assessment largely based on stream geomorphology.
2. An interdisciplinary engineering analysis.
3. A hydraulic model considering sediment transport.

This evaluation is called a comprehensive Level 2 analysis.

To perform a comprehensive Level 2 analysis on all the bridges would require considerable engineering cost and effort, as pointed out earlier. Hence, a simplified Level 2 analysis called the Texas Secondary Evaluation and Analysis for Scour (TSEAS) was developed by TxDOT, proposed to FHWA (Federal Highway Administration), and accepted by FHWA. The United States Geological Survey (USGS), Texas district, assisted TxDOT to perform the TSEAS on a number of bridges. The format for a TSEAS analysis report is as follows:

1). Introduction
2). Procedure
   A). Field survey and site data
   B). Topography map showing locations
3). Hydrology
4). Bed samples, if necessary
5). Hydraulic modeling
6). Results and discussions
   A). Summary of findings
   B). Waterway adequacy
   C). Substructure
   D). Channel and channel protection
7). Computations - Scour equation forms or HY-9
8). Plot of original ground surface under bridge vs the present ground surface
9). Plot of ultimate scour envelope
10). Recommendation

The completed scour evaluation for a bridge over a waterway with scourable bed is then forwarded to the Division of Bridges and Structures, Hydraulics Section, TxDOT. There, an Interdisciplinary Scour Evaluation Team (ISET) determines whether or not a bridge is vulnerable to scour. ISET proposes an action plan for each bridge and provisions for bridge closure, if necessary. This plan also includes the timely inspection of scour counter measures to mitigate the scour potential of the bridge.

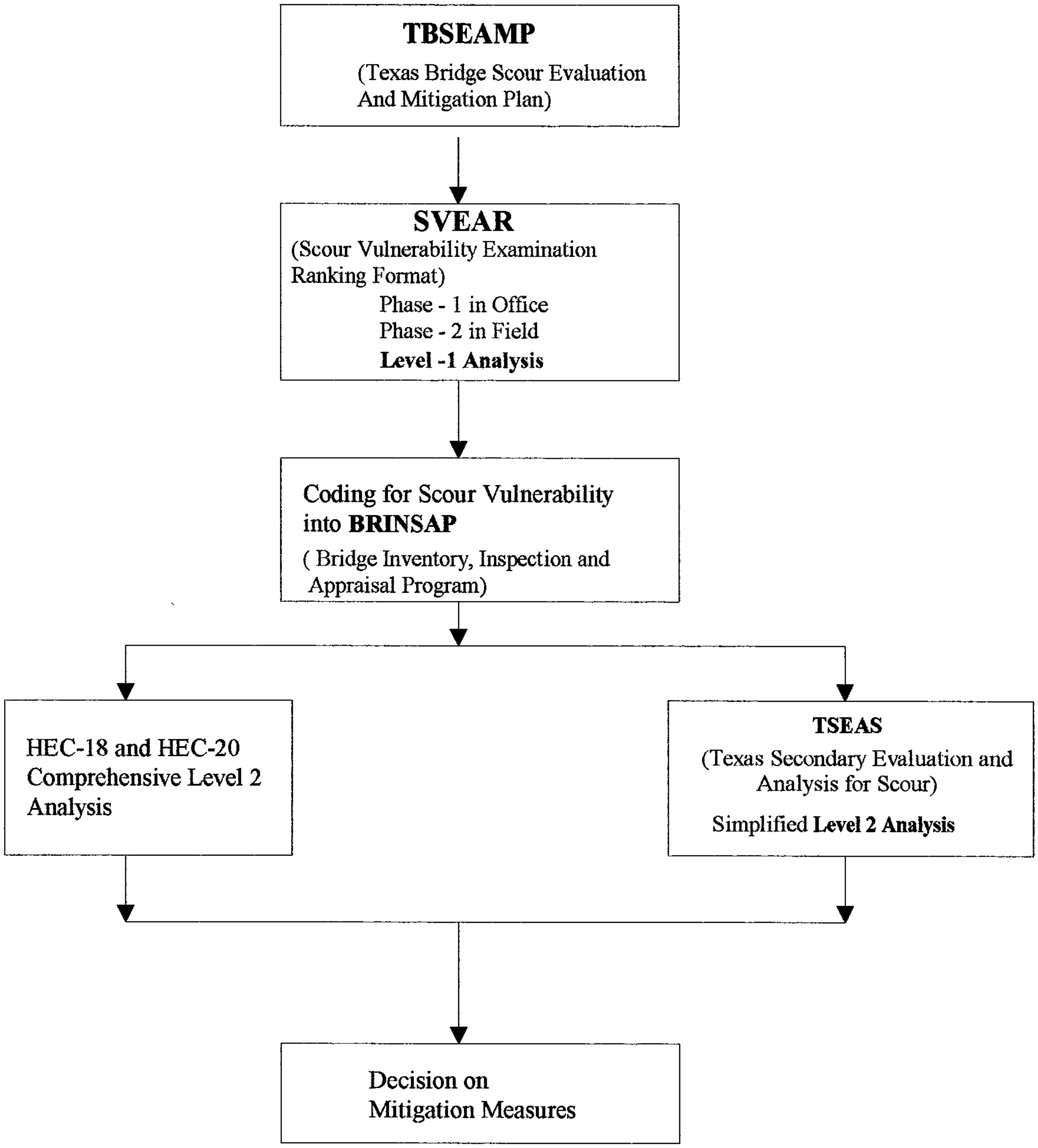

Figure 2.10. Schematic Adopted by TXDOT for Scour Evaluation

The Texas Bridge Scour Evaluation and Mitigation Plan (TBSEAMP) presented above (Figure 2.10) has made substantial progress in the assessment of scour vulnerability of Texas bridges. However, research is needed for better scour prediction procedures as well as cost-effective scour mitigation and design.

2.5. THE PROJECT OBJECTIVES

Many transportation structures are built over streams where the stream bed is susceptible to scour. Scour can lead to structural failures which can both endanger human welfare and be extremely expensive to repair. There are relatively few experts with experience in these types of hydraulic problems. Even if such experts were available for all problems which arise, the unique characteristics of different structures mean that it frequently would be much more desirable and reliable to conduct hydraulic model studies of particular structures than to rely on experience which was gained from other situations and which, therefore, may not be applicable to the problem being considered. However, the required cost and time may prohibit building individual models for each structure which needs detailed study. It would be much more feasible to conduct problem-specific hydraulic model studies for structures if a general modeling facility were available, designed specifically for riverine sediment movement.

A feasibility study was performed for developing a general-purpose hydraulic modeling facility for studying scour problems. This study included the development of a preliminary design and an evaluation of the cost. Factors to be considered in evaluating the feasibility are the size and length of the facility, whether an adjustable slope is needed, the required water flow rates and flow control devices, types and sizes of bed materials to be used for different types of problems, and instrumentation and testing procedures. The objective is to consider a facility large enough and functionally flexible enough to allow the placement of a scale model of an entire structure or to study single structural components such as piers and embankments as is presently possible.

3. HYDRAULIC MODELING

3.1. BASIC OPEN CHANNEL HYDRAULICS

A physical model is a useful tool for predicting the behavior of some physical phenomena. Physical models are usually more accurate than mathematical models and usually more reliable when they are designed properly. The reproduction of a physical phenomenon at a small scale can be a valid model if its pertinent quantitative characteristics are related to their counterparts in the prototype by the appropriate laws of similitude. To construct a physical model, one needs to understand the concepts of bed shear stress, bottom roughness, Reynolds number, Froude number, similarity laws, and types of models. Some of the basic concepts in open channel hydraulics are explained briefly in the following sections.

3.1.1. Bed Shear Stress

Bed shear stress is an important parameter in bed-load dominated sediment transport and movable bed models. To get proper scaling of hydrodynamic forces, similarity in shear stress must be attempted. When water flows in a channel, a force will be developed on the channel bed which will act in the direction of flow. This force is developed as a pull of water on the wetted area, and it is known as the tractive force. This tractive force is equal to the effective component of the gravity force acting on the body of water and parallel to the channel bottom. For a very wide open channel in which the hydraulic radius is equal to the depth of flow 'y', the unit tractive force, τ_o, is

$$\tau_o = \gamma RS \qquad \text{Eq. 3.1}$$

where γ = specific weight of water

R = hydraulic mean radius

S = slope of the channel bottom.

The above tractive force is also known as shear force or drag force. The unit tractive force or shear force is not uniformly distributed along the wetted perimeter due to the difference in the roughness along the wetted perimeter of the channel. Turbulent conditions in the channel are generally expressed with a quantity 'u_*' called shear velocity, which is a measure of the intensity of turbulent fluctuations. The friction velocity is defined as:

$$u_* = \sqrt{\frac{\tau_o}{\rho}} \qquad \text{Eq.3.2}$$

Sediment in the bed will start to move when the lift and drag exerted on individual grains by the water flow exceeds the stabilizing force due to the immersed weight of the grains. Shields (1936) proposed a criterion which is obtained by expressing the mobility, i.e., the ratio of the fluid shear stress to immersed weight of the surface layer of particles as a function of the Reynolds number of the grains. For the initiation of motion, Shields proposed the dimensionless relationship which is shown in the following equation.

$$\frac{(\tau_o)cr}{(\gamma_s - \gamma)d} = f\left(\frac{u_* d}{\nu}\right) \qquad \text{Eq.3.3}$$

where γ_s = specific weight of sediment particles

γ = specific weight of water

$(\tau_o)_{cr}$ = critical shear stress for initiation of sediment motion

d = grain size diameter

ν = kinematic viscosity.

By substituting equation (3.1) into equation (3.3), the above relationship can also be expressed as

$$\psi = f(R_*) \qquad \text{Eq.3.4}$$

where $R_* = \left(\frac{u_* d}{\nu}\right)$ is the shear Reynolds number, and the left hand side of the equation is the critical non-dimensional boundary shear stress (Shields parameter) which is defined as

$$\psi = \frac{SR}{[(G_s - 1)d]} \qquad \text{Eq.3.5}$$

where ψ = Shields parameter

R = hydraulic mean radius

S = slope of the channel

d = grain size diameter

$(G_s - 1)$ = submerged specific gravity of the sediment particles.

The above functional relationship between ψ and R_* was established by Shields from experimental data. The result is plotted in Figure 3.1.(Vanoni, 1964) and is known as the Shields diagram with dimensionless critical shear stress vs. shear Reynolds number.

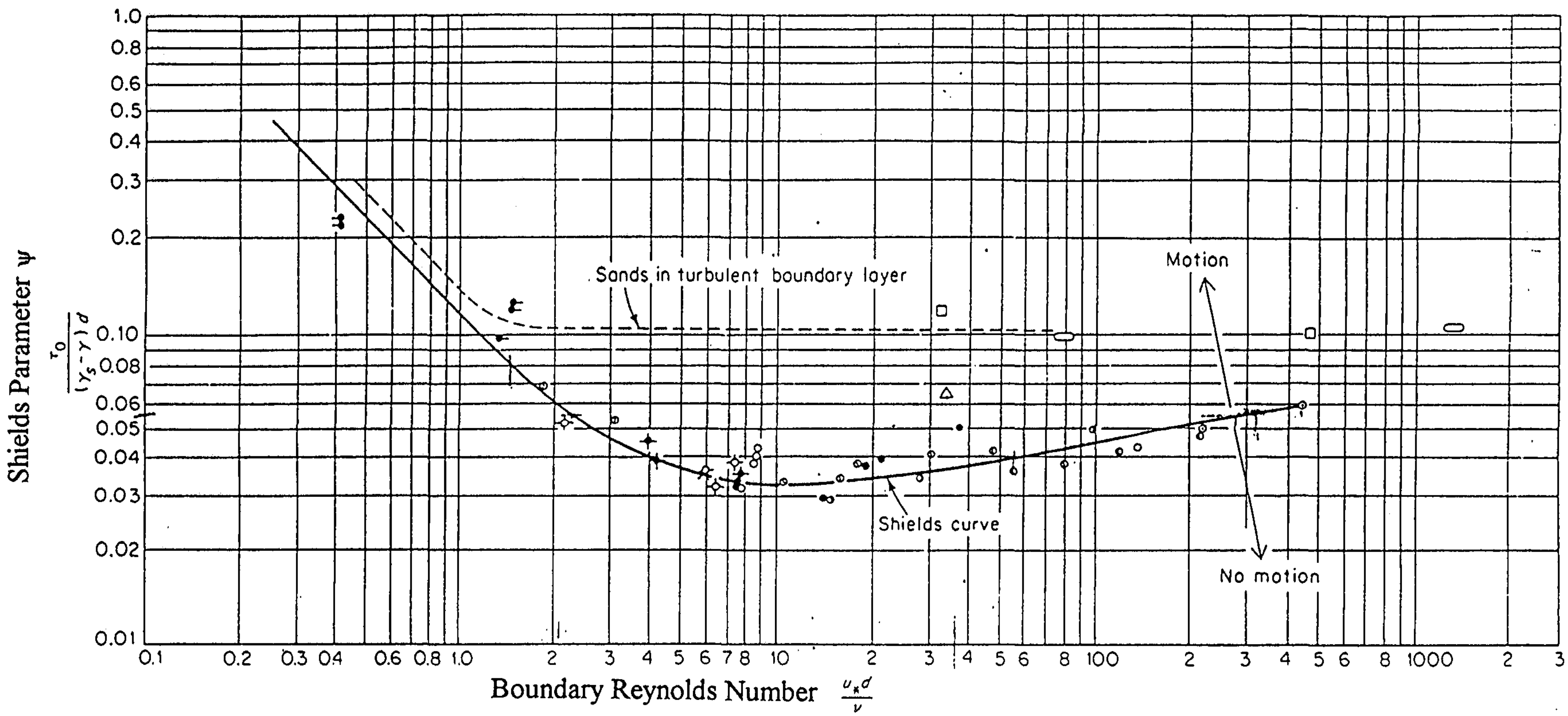

Figure 3.1 Shields Diagram (Vanoni, 1964)

From the diagram it can be observed that for any given shear Reynolds number, if the value of the critical shear stress is above the Shields curve, the sediments will be in motion. The condition of similarity may be simply derived from equating the value of '$\frac{SR}{d(G_s - 1)}$' in the model and prototype.

3.1.2. Froude Number

Froude number 'F_r' is defined as the ratio of inertial forces to gravity forces and can be written as

$$F_r = \sqrt{\frac{V^2}{gD}} \qquad \text{Eq.3.6}$$

where V = velocity of water
D = depth of water
g = acceleration due to gravity

Keeping the Froude number the same is the basic similitude criterion in river models because gravity is the predominant force. In free surface flows, the inertia forces are balanced primarily by gravity forces which can be expressed with the Froude number. The flow is said to be critical if the Froude number is equal to one. Velocity in this state of flow (critical state) is called critical velocity. If the velocity is less than the critical velocity and the depth is more than the critical depth, the Froude number is less than one and the flow is sub critical. If the Froude number is greater than one, the flow is super critical. Super critical flows normally occur in channels with steep slopes. In the models constructed with steep slopes, controlling super critical flows by tailgates at the downstream side is difficult. In this case, gates may need to be provided at the upstream side to control the flow. In case of sub critical flow, gates can be provided at the downstream side. So, the type of flow must be known to determine the location of flow controlling structures in the model.

3.1.3. Reynolds Number

Reynolds number is defined as the ratio of inertial forces to viscous forces and can be written as

$$R_e = \frac{\rho VD}{\mu} \qquad \text{Eq.3.7}$$

where V = velocity
D = depth
μ = dynamic viscosity
ρ = density of water.

Reynolds number is a non-dimensional ratio and quantifies the relative importance of the inertia to the viscous forces occurring in the flow system. Flow becomes turbulent if the Reynolds number is greater than 2000. In physical modeling, the importance of the Reynolds number progressively decreases when its numerical value increases. In physical modeling, if viscosity is the predominant force the Reynolds number similarity must be satisfied. However, it was found from many model calculations that it is very difficult to satisfy the Reynolds number completely at a reduced scale. In the prototype the viscous forces usually are not dominant. Therefore, it is advisable to have the model as large as possible to ensure that the viscous forces are not dominating.

3.2. SIMILITUDE

There are, in general, three types of similarities to be established for complete similarity to exist between the model and its prototype. These are:

1) Geometric Similarity
2) Kinematic Similarity
3) Dynamic Similarity.

1). Geometric Similarity: Geometric Similarity exists between the model and the prototype if the ratios of corresponding length dimensions in the model and the prototype are equal.

2). Kinematic Similarity: Kinematic similarity can be achieved between the model and the prototype if (a) the paths of the homologous moving particles are geometrically similar, and (b) if the ratios of the velocities, as well as accelerations of the homologous particles, are equal. Kinematic similarity can be attained if flownets for the model and the prototype are geometrically similar, which in turn means that by mere change in scale the two flownets- one for the model and the other for the prototype can be superimposed.

3). Dynamic Similarity: Dynamic similarity exists between the model and the prototype which are geometrically and kinematically similar if the ratio of all the forces acting at the homologous points are equal. In problems concerning fluid flow, the forces acting may be any one, or a combination of the several of the many forces in existence such as inertia forces, friction or viscous forces, gravity forces, pressure forces, elastic forces, and surface tension forces. For complete dynamic similarity, the ratio of inertia forces of the two systems must be equal to the ratio of the resultant forces as shown in the following equation.

$$\frac{(\Sigma F)_m}{(\Sigma F)_p} = \frac{(Ma)_m}{(Ma)_p} \qquad \text{Eq.3.8}$$

where F = force on the fluid flow

M = mass of the fluid

a = acceleration of the fluid flow.

In addition to the above condition, the ratio of the inertia forces of the two systems must also be equal to the ratio of individual component forces as shown in the following relationship.

$$\frac{(F_v)_m}{(F_v)_p} = \frac{(F_g)_m}{(F_g)_p} = \frac{(Ma)_m}{(Ma)_p} \qquad \text{Eq.3.9}$$

where F_v = viscous force

F_g = gravity force

M = mass of the fluid

a = acceleration of the fluid flow.

Thus, when the two systems are geometrically, kinematically, and dynamically similar, then they are said to be completely similar.

3.2.1. Similarity Laws

The results obtained from the model tests may be transferred to the prototype by the use of model laws which may be developed from the principles of dynamic similarity. Various model laws such as Reynolds Model Law, Froude Model Law, Mach Model Law, and Euler Model Law have been developed depending upon the significant influence of each of the forces on the different phenomena.

1). Reynolds Model Law

For the flows where in addition to inertia, viscous force is the only other predominant force, the similarity of flow in the model and its prototype can be established if the Reynolds number is same for both the systems. This is known as Reynolds Model Law, according to which

$$(R_e)_m = (R_e)_p.$$

or
$$\frac{\rho_r V_r L_r}{\mu_r} = 1 \qquad \text{Eq.3.10}$$

where $\rho_r = \frac{\rho_m}{\rho_p}$ i.e., density of the fluid in the model / density of the fluid in prototype

$V_r = \frac{V_m}{V_p}$ i.e., velocity in the model / velocity in the prototype

$L_r = \frac{L_m}{L_p}$ i.e., length dimension in the model / length dimension in the prototype

$\mu_r = \frac{\mu_m}{\mu_p}$ i.e., dynamic viscosity in the model/dynamic viscosity in the prototype

Some of the phenomena for which the Reynolds Model Law can be a sufficient criterion for similarity of flow in the model and the prototype are: flow of incompressible fluid in closed pipes, motion of airplanes and flow around structures without a free surface.

2). Froude Model Law

When the force of gravity can be considered to be the only predominant force which controls the motion in addition to the force of inertia, the similarity of the flow in any two such systems can be established if the Froude number for both systems is the same. This is the Froude Model Law according to which

$$(F_r)_m = (F_r)_p.$$

or
$$\frac{V_r}{\sqrt{g_r D_r}} = 1 \qquad \text{Eq.3.11}$$

where V_r = velocity scale ratio

g_r = gravitational force ratio

D_r = depth scale ratio

Some of the phenomena for which the Froude model law can be a sufficient criterion for dynamic similarity to be established in the model and the prototype are: free surface flows such as flows over spillways, and through sluices in which gravity is the driving force.

3.2.2. Other Model Laws

Model laws are very important in establishing the relationships between various parameters of the prototype and the model. The following scale ratios are derived based on the Froude law of similarity. Froude law is the basic similitude criteria in the river models. In addition to this, surface roughness must also be given careful consideration (Modi and Seth, 1984). For a given discharge, there will be significant change in the velocity due to the change in the roughness parameter, as shown in the Manning's equation. It is obvious that the change in velocity will affect the water surface elevation. When all the dimensions of the prototype are scaled down, depth and velocities will be reduced depending on the scale. To maintain these reduced values, it is required to scale down the roughness in order to simulate the resistance to flow in the model, which can be done based on the Manning's relation. Therefore, it is required to determine the roughness in the model for a given roughness in the prototype. The scale relationships for river models are usually based on Manning's formula given in the following equation.

$$V = \frac{R^{2/3} S^{1/2}}{n} \qquad \text{Eq.3.12}$$

where V = velocity
R = hydraulic mean radius
S = slope
n = Manning's roughness

It is a relationship between various parameters, including roughness, from which the following ratios can easily be derived. Most of the river models are distorted with either vertical exaggeration or slope exaggeration. Based on Froude scaling, the following relationship can be found using Equation 3.12.

$$V_r = \frac{L_r^{2/3} S_r^{1/2}}{n_r} \qquad \text{Eq.3.13}$$

where V_r = the ratio between the velocity in the model and the velocity in the prototype

L_r = the ratio between the length dimensions in the model and the prototype

S_r = the slope scale ratio in the model and the prototype

n_r = the ratio of Manning's roughness in the model and in the prototype

If the velocities, slopes, and depths are known in the model and prototype, then the roughness scale ratio can be found from the above equation. The hydraulic radius 'R' is dependent upon both horizontal and vertical dimensions. As an approximation for wide rivers, $R_r = D_r$. Also, the slope scale ratio 'S_r', is:

$$S_r = \frac{D_r}{L_r} \qquad \text{Eq.3.14}$$

where R_r = hydraulic radius scale ratio

D_r = depth scale ratio

L_r = length scale ratio.

Thus, the velocity scale ratio 'Vr' may be expressed as:

$$V_r = \frac{D_r^{7/6}}{n_r L_r^{1/2}} \qquad \text{Eq.3.15}$$

The value of 'n_r' can thus be controlled by suitably fixing the scale ratios. If we assume the Froude number similarity for an undistorted model, $V_r = D_r^{1/2} = L_r^{1/2}$, then the above equation reduces to:

$$n_r = L_r^{1/6} \qquad \text{Eq.3.16}$$

From the above equation, the ratio of the Manning's roughness in the model and that in the prototype can be found for a given scale and, therefore, the roughness in the model. The value of the scale ratio for Manning's coefficient 'n_r'can also be controlled by tilting the model which is otherwise geometrically similar. Such models are called tilted models or models with slope distortion. For such models, according to Froude Law and Manning's formula:

$$V_r = \frac{D_r^{2/3}}{n_r} S_r^{1/2} \qquad \text{Eq.3.17}$$

The Manning's roughness ratio will become:

$$n_r = \frac{D_r^{2/3}}{L_r^{1/2}} \qquad \text{Eq.3.18}$$

where D_r = depth scale ratio

L_r = length scale ratio

Because the model is distorted, the above scale ratios Equations 3.16 and 3.18 are different from each other. In this case, the roughness in the model can be determined for given horizontal and vertical scale ratios.

3.2.3. Empirical Approach

With movable bed models, it is the type of bed roughness, the bed configuration, and the bed-material motion which determine the roughness. When a model is distorted, the longitudinal slope is increased (Graf, 1971). This has a direct influence on the velocity profile which, in turn, has a direct bearing on the sediment movement. Since it is difficult to control the roughness, it is equally difficult to control the velocity profile; thus, dynamic similarity may be destroyed. At the same time, distortion will allow for an easier bed material movement since the shear stress is proportional to the slope. The essentials of the empirical approach are summarized as: if a model can be adjusted to reproduce events that have occurred in the prototype, it should indicate events that will occur in the prototype.

In such a model with its low velocities and shallow depths, a very light sediment material must be used. Light weight particles may be either small particles or particles with low specific gravity. Generally, it is unreasonable to apply the scale ratio to the prototype sand grains since the resulting model grains would be much too small. However, it is customary to alter the specific gravity. It is also recommended that the distortion ratio of movable-bed models never be greater than about 6, i.e.,:

$$\left(\frac{L_p}{L_m}\right)_V : \left(\frac{L_p}{L_m}\right)_H \approx 1{:}6 \qquad \text{Eq.3.19}$$

where L_m = length scale in model

L_p = length scale in prototype

Subscripts V, H represent vertical and horizontal or a compromise between the sediment-motion and water-motion similarity.

3.2.4. Types of Models

The river flow in the prototype is normally unsteady in nature. A river model can be constructed with a fixed or movable bed. Concrete, gravel, and some other material which cannot be moved is normally used in the construction of fixed bed models. Movable bed models normally have fixed banks, and overbanks are constructed with a movable bed by using crushed coal, sand, and some other material that can be moved by the fluid. The above models are explained in detail in the following sections.

3.2.4.1. Fixed Bed Model

Use of the frictional force criterion instead of the Reynolds number criterion is common practice in fixed bed models. The usual practice is to build the model as large as possible to make the viscosity effect negligible. It is customary to conduct river models with water and, thus, with the same kinematic viscosity as the prototype. The model roughness can be obtained through a trial-and-error procedure by adjusting until the calculated model flow rate and surface elevations are obtained. In general, it is difficult to satisfy the conditions of both the Reynolds number ratio and the Froude number ratio in the model (Tebbutt, 1985). So the model scale must be large enough that viscous effects do not dominate the flow in the model. An important consideration in producing similarity between model and prototype is the simulation of resistance to flow. Surface roughness, abrupt changes in flow direction, and sudden change in the size of the channel are the causes of the resistance to flow. A fixed model must be adjusted to reproduce the stage-discharge relationship of the prototype; this can be accomplished by adjusting the roughness elements until the model reproduces the prototype stages and discharges.

3.2.4.2. Movable Bed Models

The design and understanding of movable-bed models remain intricate. In addition to friction and gravity criterion, other criteria involving the mechanics of sediment transport have to be introduced. With movable-bed models, it is the type of bed roughness, the bed configuration, and bed-material motion which determine the roughness of the sediment transport in open channel flow. Some commonly used materials are sand, crushed coal, burnt shale, sawdust, and various plastics. The quantity of bed material introduced in the model must vary with the discharge, similar to the bed movement in the prototype. A movable-bed model is almost always distorted to make it function properly (Petersen, 1986). To induce movement of the bed material, larger velocities are required for large sizes of particles.

3.2.4.3. Undistorted Models

An undistorted model is one which is geometrically similar to its prototype; that is, the scale ratios for corresponding linear dimensions of the model and its prototype are the same. The model is perfectly defined by the only choice of the geometric scale number. However, the question is now to verify that frictional forces are scaled in the same manner as the inertial reactions. Since the basic condition of perfect similitude is satisfied, prediction in the case of such models is relatively easy, and many of the results obtained from the model tests can be readily transferred to the prototype. For undistorted models, three conditions have to be fulfilled which are given in the following:

1) the Froude number must be the same in the model and the prototype,
2) the roughness of the model must be correct, and
3) the flow in the model must be turbulent.

3.2.4.4. Distorted Models

A distorted model is one in which one or more terms of the model are not identical with their counterparts in the prototype. Since the basic condition of perfect similitude is not satisfied, the results obtained with the help of a distorted model are liable to distortion. A distorted model may have either geometrical distortion, material distortion, distortion of hydraulic quantities, or a combination of these. In geometrical distortion, the distortion can be either of dimension or configuration. When different scale ratios are adopted for the longitudinal, transverse, and vertical dimensions, it is said to be a distortion of dimensions. Distortion of dimensions is frequently adopted in river models. Models with vertical scales greater than horizontal scales are called 'vertically exaggerated' models. Often, the view is taken that in case of distortion of depth, an equal distortion of slope is needed. The horizontal scale ratio for model rivers and harbors should be:

$$100 < \frac{(L_H)_p}{(L_H)_m} < 2000 \qquad \text{Eq.3.20}$$

where $\left(L_H\right)_p$ is the horizontal scale length in the prototype, and

$\left(L_H\right)_m$ is the horizontal scale length in the model.

The vertical scale ratio is given as:

$$50 < \frac{(L_V)_p}{(L_V)_m} < 150 \qquad \text{Eq.3.21}$$

where $\left(L_V\right)_p$ is the vertical scale length in the prototype, and

$\left(L_V\right)_m$ is the vertical scale length in the model.

In the physical modeling, the greater the model is distorted, the greater the exaggeration of roughness. There are reasons for adopting distorted models:

1. To maintain accuracy in vertical measurements.
2. To maintain turbulent flow.
3. To obtain suitable bed material and its adequate movement.

3.2.4.5. Advantages and Limitations

The merits of distorted models may be summed up as follows:

1. Measurement of water surface elevation will become easy due to its vertical exaggeration.
2. Reynolds number will be increased considerably.
3. Sufficient tractive force can be developed to produce adequate bed movement with a reasonably small model.

Besides the above merits, there are certain limitations, as follows:

1. The magnitude and distribution of velocities are incorrectly reproduced.
2. Slopes of river bends, earth cuts, and dikes are often so steep that they can't be molded satisfactorily.
3. Some of the flow details may not be correctly reproduced because distortion increases longitudinal slopes.

3.3. EXISTING SOFTWARE AND ITS APPLICATIONS

Some software that is useful to compute the water surface elevations, velocity distribution in the channel and also on the flood plain are explained in detail in the following section. The computed model results are often used to calibrate physical models.

3.3.1. WSPRO (Model for Water-Surface Profile Computations)

WSPRO was developed by the USGS under a contract with the FHWA. WSPRO was developed for computations of flow through bridge openings, combination of road overflow and bridge-opening flow, and multiple waterway openings. This model is a comprehensive, design oriented model, and it is very well suited for analyzing alternative designs of bridge openings and their associated approach embankments. Water-surface profile computational procedures unaffected by bridges are completely compatible with those of existing models. But computations through bridges are based upon more recent developments. This model has the capability to analyze cases where flow through the bridge occurs in combination with the flow over the approach embankments.

3.3.1.1. Surface Profile Calculations

The model uses the standard step method similar to that described by Chow (1959). The standard step method is based upon the principle of conservation of energy, i.e., the total energy head at the upstream section must be equal to the total energy at the downstream section plus any energy losses that occur between the sections. In this method, the total length of the reach will be subdivided into relatively short subreaches.

The model requires definition of the geometry and roughness of each cross section. A series of coordinates are used to describe the cross sectional geometry. These coordinates define the horizontal station and ground elevation of each ground point across the section. The roughness of the section is defined by Manning's 'n' values. Convention for computational direction in this model are 1) upstream for subcritical flow and 2) downstream for supercritical flow.

3.3.1.2. Model Capabilities

The following are some of the capabilities related to input and output.

1. Missing data in the present cross-section will be propagated from the previous section.
2. It is possible to fabricate valley cross-sections from a template cross section when two or more cross sections are very similar.
3. Bridge openings may be defined either by a series of coordinates or in terms of geometric parameters of bridge components which are combined with a valley cross section.
4. The combination of subcritical, critical, and supercritical flow profiles may be analyzed for one dimensional, gradually varied, and steady flow.
5. Up to 20 profiles for different discharges may be computed at the same time.
6. Variable Manning's roughness coefficients may be specified for any cross-section.
7. Backwater for both free-surface and pressure flow situations at a bridge can be computed.
8. The model can compute water surface profiles when road overflow occurs in conjunction with flow through the bridge opening.
9. The model is capable of computing water surface profiles through multiple waterway openings.
10. Culverts can be included in multiple opening analysis.

3.3.1.3. Limitations

1. Within each subreach, flow should be gradually varied and steady.
2. Flow should be one-dimensional.

3.3.2. FESWMS-2DH (Finite Element Surface-Water Modeling System: Two Dimensional Flow In a Horizontal Plane)

FESWMS-2DH (Frochlich, 1989) is a modular set of computer programs developed to simulate two dimensional water surface flow. This modeling system has been designed specifically to analyze flow at bridge crossings. The programs that follow the core of the modeling system are :

1. The data input module (DINMOD),
2. The depth of flow module (FLOMOD), and
3. The analysis of output module (ANOMOD).

DINMOD is used to generate two dimensional finite element networks (grid): FLOMOD simulates both steady and unsteady two dimensional surface water flow. The program is based on the finite element method of analysis to solve the unsteady integrated equations of motion and continuity to obtain velocities and flow depth. ANOMOD is used to present the results in the form of hard copies and also graphically. It acts as a post processor in the modeling system.

3.3.2.1. Assumptions

The assumptions made in developing this software are:

1. The flow is assumed to be two dimensional, and
2. The velocity in the vertical direction is assumed to be negligible.

3.3.2.2. Applications

Flow in water bodies that have irregular topography and geometrical features, such as islands and highway embankments, can be simulated using this modeling system. This modeling system can also be used to model flow over dams, weirs, highway embankments, through bridges, culverts, and grid openings.

3.3.2.3. Methodology

The Galerkin finite element method is used to solve the governing equations of motion and continuity. Application of this method causes the water body to be divided in smaller regions called elements. The shape of the element can be either triangular or quadrangular. The elements are defined by a series of nodal points. The Galerkin finite element method requires the governing equations to be weighted. Gaussian quadrative is used to perform numerical integration.

The steps generally followed to operate FESWMS-2DH are:

3.3.2.3.1. Data Collection: The data required are classified as either topographic or hydraulic data. Topographic data include geometry of the physical system and also evaluation of surface roughness, velocity measurements, high water marks, rating curves, and limit of flooding.

3.3.2.3.2. Network Design: Constructing the finite element network is the next step in operating the system. In the network design process, the surface water body is subdivided into an assemblage of finite elements. The basic objective of the design is to create a representation of the water body. The size and shape of the elements depends on the desired level of detail in that particular area. Any combination of 6-node triangular, 8-node quadrangular, or 9-node quadrangular elements that have straight or curved sides can be used for complex geometries.

3.3.2.3.3. Calibration: The model dimensions have to be adjusted so that values computed by a model reproduce as closely as possible values measured on site. Measured values of water surface elevation, total flow rates, and velocities can be used for calibration of this modeling system.

3.3.2.3.4. Validation: The testing of a calibrated model, to see if computed values compare reasonably to measured values, is called validation of the results. If the model is able to reproduce the additional measured values without any further adjustment of model parameters, then the model can be used to simulate conditions outside the range of calibration. Often, it is impossible to validate a model because of insufficient data.

3.3.2.3.5. Application: After the completion of the above steps, a model can be used to simulate a variety of flow conditions. After a model has been calibrated and validated, it can be used to gain valuable insights to the response of a surface water flow system.

4. SOIL MODELING

4.1. BACKGROUND

The model in the laboratory is significantly smaller than the prototype because of scaling limitations. As a result, the water depth in the model is smaller. In order to maintain the same hydraulic condition in the model and in the prototype, the Froude number (F_r), which is a ratio of viscous force to gravitational force, must be the same in the model and the prototype (Section 3.1.2).

$$F_r = \frac{V_m}{\sqrt{gh_m}} = \frac{V_p}{\sqrt{gh_p}} \qquad \text{Eq.4.1}$$

where,
F_r = Froude Number,
V_m and V_p = velocities in the model and prototype, respectively,
g = acceleration due to gravity, and
h_m and h_p = water depths in model and prototype, respectively.

Also

$$h_m = \frac{h_p}{L_r} \qquad \text{Eq.4.2}$$

where,
L_r = scale ratio.

Therefore,

$$\frac{V_m}{V_p} = \frac{1}{\sqrt{L_r}} \qquad \text{Eq.4.3}$$

This shows that in order to keep the same hydraulic condition in the model and prototype, V_m must be smaller than V_p. The smaller water depth and the smaller velocity lead to a smaller erosion potential of the soil in the model. As a result, if the soil from the prototype is placed in the model, this soil which would be eroded in the prototype may not be eroded in the model. It is, therefore, necessary to reduce the size or the weight of the soil grains in the model in order to maintain the same erosion potential.

The selection of a model soil is based on one of two sediment transport criteria: the bed load criterion and the suspended load criterion.

4.2. BED LOAD CRITERION

The bed load criterion refers to the transport condition where the soil grains are barely dislodged from their position; they roll slightly on the river bottom and then stop.

This transport phenomenon occurs when the shear stress, τ, imposed by the water flowing over the particle becomes equal to the shear resistance between particles. For cohesionless soils, the shear resistance, S, is proportional to the normal stress, σ, on the plane of failure (Figure 4.1). The Shields parameter ψ essentially represents the ratio between τ and σ. It is defined as:

$$\psi = \frac{\tau}{(\rho_s - \rho_w)\, gd} \qquad \text{Eq.4.4}$$

Where ρ_s = soil density,
ρ_w = water density,
g = acceleration due to gravity, and
d = particle size.

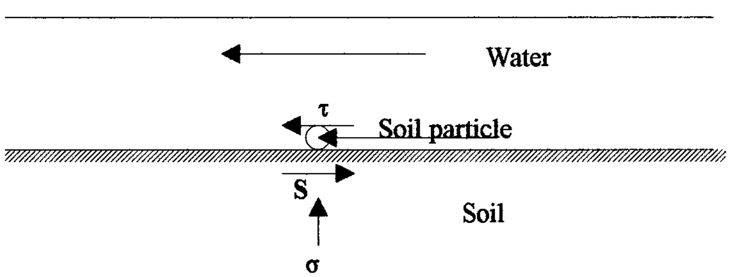

Figure 4.1. Definitions of τ and σ

The denominator represents the buoyant weight of a cubic soil particle divided by the contact base :

$$\sigma = \frac{(\rho_s - \rho_w)gd^3}{d^2} \qquad \text{Eq.4.5}$$

The shear stress τ can be calculated as shown in Figure 4.2.

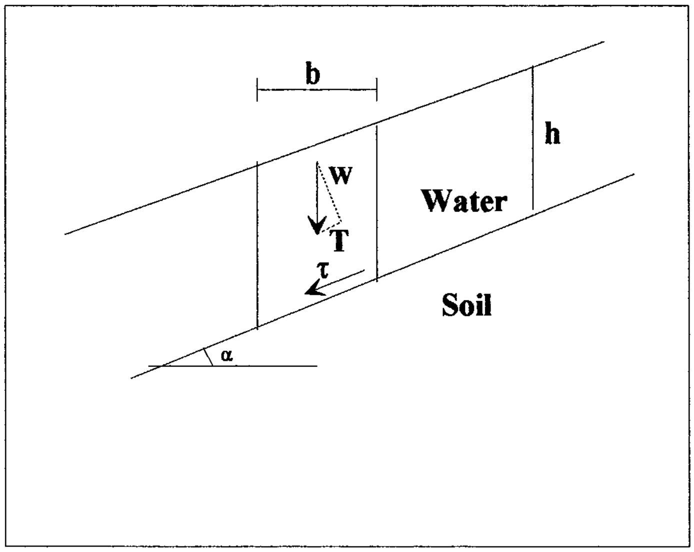

Figure 4.2. Bed Load Shear Stress

The weight of the water element W is

$$W = \gamma b h \qquad \text{Eq.4.6}$$

where $\gamma =$ unit weight of water,

b = width of the water element, and

h = height of the water element.

The component of the weight parallel to the river bottom is T:

$$T = \gamma b h \operatorname{Sin} \alpha \qquad \text{Eq.4.7}$$

The shear stress τ is, therefore,

$$\tau = \frac{T}{A} = \gamma b h \operatorname{Sin} \alpha \frac{1}{\frac{b}{Cos\ \alpha}} \qquad \text{Eq.4.8}$$

$$= \gamma h\ Sin\, \alpha\, Cos\, \alpha$$

For small angle of α, Equation 4.8 can be written as:

$$\tau = \gamma h s \qquad \text{Eq.4.9}$$

Where τ = shear strength along the soil-water interface, and

s = slope of the river bed.

Now, ψ can be expressed as

$$\psi = \frac{\gamma h s}{(\rho_s - \rho_w) g d} \qquad \text{Eq.4.10}$$

If the value of ψ becomes large, sediment transport is likely to occur. If ψ is small enough, no transport occurs. There is a boundary value for ψ; Vanoni (1964) established what that boundary was for a number of cohesionless soils (Figure 4.3). For a given case, the value of ψ is calculated together with the boundary Reynolds number (R_*); the ψ and R_* point is plotted on Figure 4.3, and a conclusion is reached on the scour potential. Note that R_* is the ratio of the inertia force to viscous force and is defined as :

$$R_* = \frac{u_* d}{\nu} \qquad \text{Eq.4.11}$$

Where u_* = shear velocity

$$= \left(\frac{\tau}{\rho_w}\right)^{1/2} \qquad \text{Eq.4.12}$$

ν = dynamic viscosity of the fluid

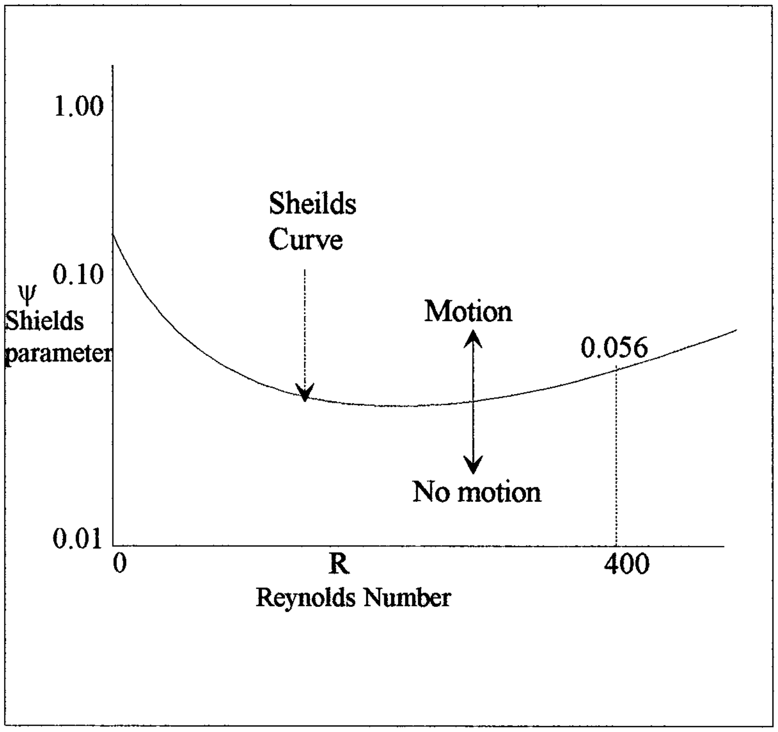

Figure 4.3. Shield's Representation (Vanoni, 1964)

In the modeling process it is essential to maintain the same value of ψ for the model and for the prototype. Since h is smaller in the model, then $(\rho_s - \rho_w)gd$ must also be reduced by the same ratio. This can be achieved by reducing $(\rho_s - \rho_w)$ (lighter soil particles) or by reducing d (smaller soil particles).

4.3. SUSPENDED LOAD CRITERION

The suspended load criterion refers to the transport condition where the soil grains are not only dislodged from their position but also stirred up into the water flow and transported in suspension downstream. This is a higher energy environment which creates a higher scour potential. It occurs when the flow is turbulent and the boundary Reynolds number is very high. In steady uniform flow carrying sediment in suspension, under equilibrium conditions, the change in concentration at any level will be minimum. The settling of the suspended particles will be balanced by the net upward flux of particles due to turbulent flow near the river bottom.

The ratio of the river bed water velocity u_* over the velocity with which a soil grain will settle in water V_s is useful in characterizing suspended load transport. This ratio χ can be expressed as:

$$\chi = \frac{u_*}{V_s} \qquad \text{Eq.4.13}$$

The velocity v_b is given by Equations 4.9 and 4.12. The velocity V_s is given by Stokes law (for very fine particles under steady state):

$$V_s = \frac{(\rho_s - \rho_w)}{18\,\mu}\, gd^2 \qquad \text{Eq.4.14}$$

Therefore, χ becomes:

$$\chi = \frac{(\gamma hs)^{0.5}(18\mu)}{(\rho_w)^{0.5}(\rho_s - \rho_w)gd^2} \qquad \text{Eq.4.15}$$

Where μ = absolute viscosity of the fluid

For proper modeling, it is necessary to maintain the same value of χ for the model and the prototype. Since h is smaller in the model, then either $(\rho_s - \rho_w)$ (lighter soil grain) or d (size of soil grains) must be reduced by the same ratio. Note that this leads to a different requirement compared to the bed load criterion. In both criteria, however, the choice is between soil with lighter grains or with smaller grains.

4.4. SOIL SIMULANTS

A number of studies have been carried out to determine a suitable bed material for use in movable bed models. Different materials have been tested by different researchers (Section 6.3.3.) including sand, crushed coal, plastics, and crushed walnut shells. They are all made of either smaller grains or lighter grains.

4.4.1. Sand

Sand is one of the most commonly used bed materials. It has a specific gravity of 2.65 like all other mineral soils. Therefore, it can only be used to model scour problems for soils having larger grains in the prototype. For example, if a uniform fine sand with a 1 mm grain size is used in a 1/10 scale model, the bed load criterion (Equation 4.10.) the soil in the prototype has a grain size of 10 mm, which is gravel.

The sand used at USAE Waterways Experiment Station (WES) has a mean diameter of 0.20 mm. Sands must be washed free of clay and silt. Sloping the model river bed can move the sand particles. When using sand in outdoor bed models, erosion is possible due to rain or other weather conditions. Otherwise, sand is not appreciably affected by weather. Any damages done to sand beds can also be easily rectified. The disadvantages of using sand in movable beds are the formation of the ripples and the larger forces required to move the sand particles. Ripples change the bed shape and alter the roughness of the channel bed.

4.4.2. Coal

Coal used in bed modeling is a special coal without any impurities. Generally, the specific gravity of the coal used is 1.3. Therefore, if the coal has a grain size equal to the grain size of the prototype soil, this lower specific gravity allows us to simulate the prototype soil in a model at a scale equal to: $\frac{(\rho_{s_1} - \rho_{w_1})}{(\rho_{s_2} - \rho_{w_2})} = \frac{(2.65 - 1.0)}{(1.30 - 1.0)} = 5.5$. Unlike sand, coal, if properly reduced in size, will not form ripples. Coal has to be crushed, screened, and washed thoroughly to remove dust. It is not suggested to use coal in outdoor facilities as it can be affected by weather.

4.4.3. Plastics

One advantage of using plastics in bed models is their shape, size, color, and specific gravity; but it is expensive to obtain plastics in small quantities with specific requirements. If plastics used in bed models have a specific gravity substantially less than coal, they may float in water. Therefore, there is a limit to decreasing ρ_s. Getting too close to ρ_w will create many problems.

4.4.4. Pumice

Pumice is a sedimentary formation with air trapped in the material. Hence, the submerged weight varies with the air trapped inside. "Pumice, which is used in some European laboratories, usually mixed with coal to provide material moving in suspension does not contribute to the model channel development but is included to indicate the movement of material that goes into suspension and areas where such material might be deposited" (Franco, 1989). The material is good in reproducing bank caving or dredged clay.

4.4.5. Walnut Shells

Ground walnut shell has a specific gravity of 1.3. Therefore, it allows the same reduction in scale as coal. Walnut shells are used for bed material at WES and University of Minnesota. Ground walnut shells have a tendency to decompose to form cakes and produce gas with an objectionable odor. This material also becomes fluffy in water and forms ripples.

4.4.6. Bakelite

"Ground bakelite has been used in outdoor movable bed river models. The particle sizes observed appeared to be rather large (about 0.63 mm) and cubical in shape. The model using this material appeared to be highly distorted, indicating the specific gravity of the bed material to be higher than that of coal but probably less than that of sand. Because of the large grain size, models observed using bakelite did not ripple but moved in rather large waves or dunes"(Franco, 1989).

4.5. CURRENT PRACTICE FOR SOIL MODELING

The current approach in soil modeling is to reduce the size of the particles or to use a soil simulant with lighter particles. Most of the laboratories visited by the principal investigators of the project scale the soil in some fashion to use the results quantitatively. One laboratory was making efforts to obtain a properly scaled soil simulant according to the principles described in Sections 4.2 and 4.3 to determine the scour depth. Another laboratory was not making such an effort and was only interested in qualitative results. Most other laboratories ensured as a minimum that the soil simulant used could be eroded at a velocity equal to or less than the model velocity.

Some researchers are of the opinion that all soils erode and the final scour depth is independent of grain size. Others disagree with this idea. Those who agree feel that by modeling the soil and measuring the final scour depth in a physical modeling facility, it can be scaled up directly to predict the final scour depth for the prototype independently of the soil type. Even if this idea is correct, it is clear that the rate of erosion will depend on the soil type and grain size.

For cohesionless soils, scaling down the soil particles presents a problem beyond 0.1 mm. When the grain size of the soil is reduced, the surface area of the particles increases. Therefore, cohesion in the model is likely to be more pronounced than in the prototype due to electromagnetic forces. This causes less erosion. The electromagnetic forces around the clay particles increase when the soil is scaled down. The soil structure is also disturbed. Thus, as the model particle size gets smaller and smaller, the changes in the behavior of the soil mass becomes more drastic. Hence, it is always better to reduce the weight of the soil particles than reduce the size of the particles, provided the soil mass behavior can be maintained. At this time, no one knows how to properly model clay beds, and the approach seems to be to simply ignore the scaling problem for clays. This is not correct and, therefore, modeling facilities are of limited use at this time to study scour in clay beds.

Most of the soil simulants discussed are used to reconstruct the bed. Reconstructing a sand bed is easier than reconstructing a clay bed. In clays, the interaction between the soil water and the clay particles plays an important role, especially from the chemistry point of view. Hence, it might be better to prepare a localized clay bed than to scale a cohesive soil and prepare a large scale model bed. Parametric studies on clay soils by preparing these localized beds around obstacles may lead to some useful results.

4.6. PREPARATION OF CLAY BEDS

Preparing a uniform bed of clay is a difficult and time consuming task. The clay may be obtained in blocks approximately 0.15 m x 0.15 m x 0.30 m in dimension. The undrained shear strength of the clay can be approximately in the range of 10 to 100 kPa. Each layer consists of placing the blocks of medium soft clay side by side. In order to properly mold that layer in place, a heavy plunger having the same area as the container can be placed on the clay, loaded with a surcharge and left for several hours. When the container is ready, it can be placed in the flume for testing.

As the depth of clay increases, it becomes more difficult to drain the soil and consolidate it. The time taken for drainage of a clay bed increases as the square of the depth of the clay bed. Most clay beds are prepared in the unsaturated state because of time limitation. For example, the time required to reach 90% consolidation of a 0.3m thick clay layer with top and bottom drainage and with an average coefficient of consolidation of 10^{-4} cm^2/s is 3 months.

5. CASE STUDIES

5.1. INTRODUCTION

The case studies are used to determine the size of the basins needed, to evaluate scaling of flow, and to determine the storage and flow capacity of the facility. Five reports were obtained from TxDOT; the potential problems for those rivers and bridges were examined and taken as the case studies. All the reports describe Level II Bridge-Scour Analysis prepared by the USGS, Water Resources Division in cooperation with TxDOT. For each bridge site, cross sections were surveyed and given in the reports. The bed-material data, mean particle size distribution, annual peak flows, roughness in the channel, and energy grade line slope were also obtained for the major rivers from the above reports. Ultimate contraction, pier, and abutment scour depths were computed by using the bridge-scour analysis procedure as documented in HEC-18 (Richardson et al. 1993). In the case studies, potential problems that might be encountered at the individual sites were considered for physical modeling in the facility. Each case study entails the computation of the hydraulic conditions using WSPRO resulting from the 500-yr peak discharge. The starting downstream water surface elevations for the water surface profiles were computed using the slope-conveyance option within WSPRO. The 100-yr and 500-yr peak discharges were obtained from a USGS gauging station for each case study. The starting Energy Grade Line (EGL) slope at the down stream section was estimated from USGS topo map and was given in the reports. Bed samples were analyzed, and the D_{50} sizes in each case study were determined from the mean particle size distribution curve. All the elevations are referenced to a benchmark near the USGS gage house located at the sites. Later, all these prototype values are scaled down by suitable scale to compute model parameters. Some of the rivers used in the case studies are Guadaloupe River, Trinity River, and Colorado River.

Consideration of engineering and economics render it desirable to construct two modeling facilities: a 3-dimensional river basin and a 2-dimensional open channel flume. The initial size of the 3-D facility is taken as 45.75 m (150 ft) long, 30.0 m (100 ft) wide, and 1.0 m (3.3 ft) deep; size of the 2-D facility is taken as 36.6 m (120 ft) long, 6.1 m (20 ft) wide, and 3.66 m (12 ft) deep. While designing the modeling facility, 500-yr recurrence interval peak flow is used. This discharge and other parameters are scaled down based on the Froude model law. The scale chosen is very much dependent on the size of the modeling facility and the nature of the problem that needs study. Some of the model to prototype ratios (based on the Froude model law) which are used in the case studies are given below.

Length ratio	$= L_r$
Depth ratio	$= d_r$
C. S. area ratio A_r	$= L_r d_r$
Time ratio	$t_r = L_r d_r^{-\frac{1}{2}}$
Velocity ratio	$u_r = d_r^{\frac{1}{2}}$
Discharge ratio	$Q_r = L_r d_r^{\frac{3}{2}}$

Difficulties are explained in each case study and alternatives examined by adopting different methods like scale distortion, slope distortion, and selection of light weight material; thus, compromises may be required. Some of the case studies are explained in detail in the following sections.

5.2. CASE STUDY 1 - GUADALUPE RIVER

This case study involves State Highway 80 crossing the Guadalupe River near Belmont. The Guadalupe River is a perennial stream with a sand and clay bed in the vicinity of this highway crossing. There are significant meanders upstream and downstream of the crossing. The bridge crossing is located immediately downstream of a large meander. Erosion and deposition pattern can change the course of the river from time to time. This would change the scour depth at the bridge section to a large extent. At this location, the 100-yr and 500-yr peak discharges are 1792 m^3/sec (63,300 cfs) and 2587 m^3/sec (91,400 cfs), respectively. The site selected to model in the facility is shown in Figure 5.1, and the cross section of the river with the predicted scour envelope at the bridge location is shown in Figure 5.2.

5.2.1. Objective

The erosion and deposition pattern in the meanders on the upstream side of the crossing can change the scour depth around the bridge piers. The main objective of this case study is to determine the erosion and deposition pattern in the meanders and to suggest the appropriate location for rip-rap placement to prevent further erosion.

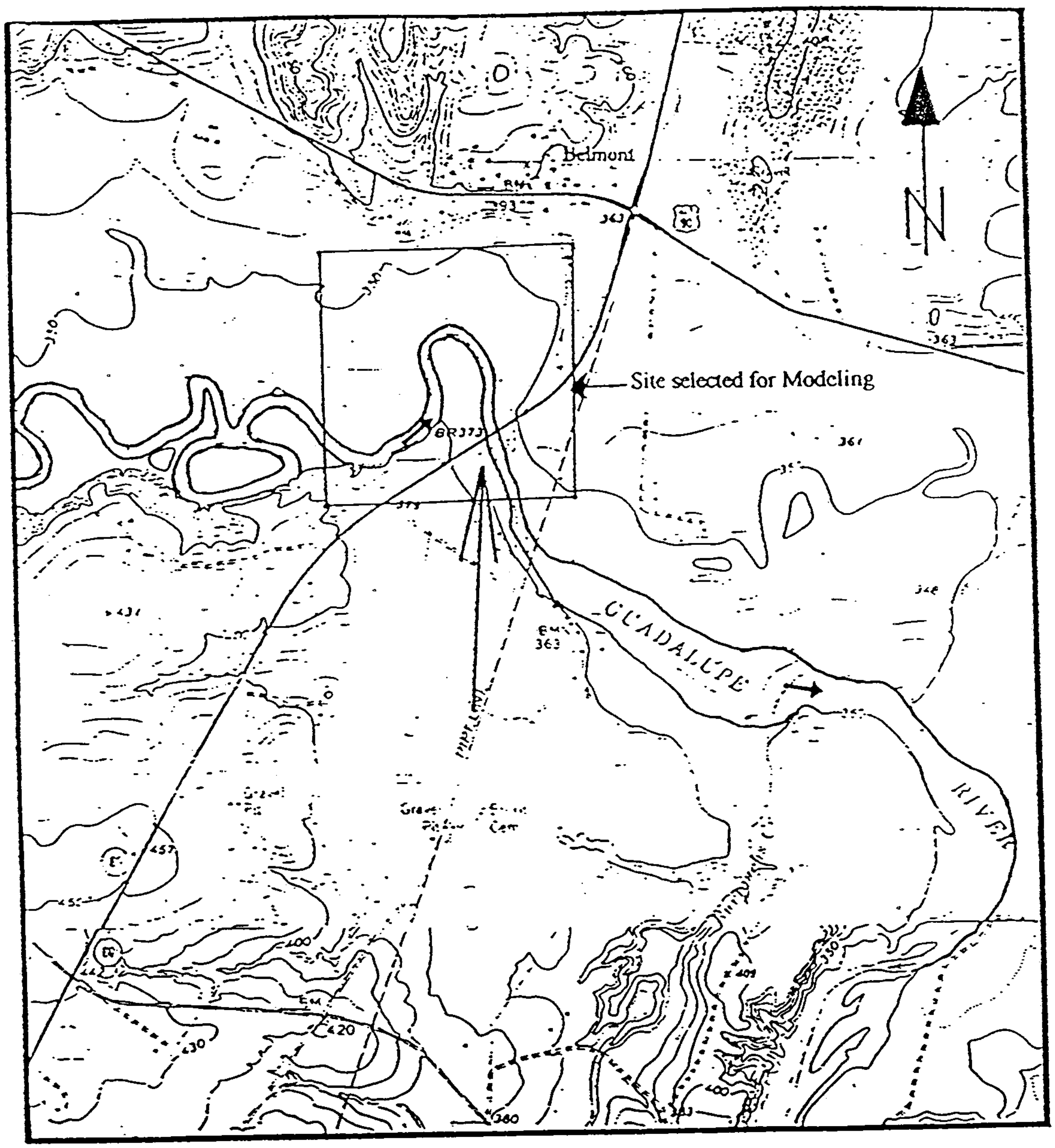

Figure 5.1. Topographic Map of Bridge Site Where State Highway 80 Crosses the Guadalupe River Near Belmont, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

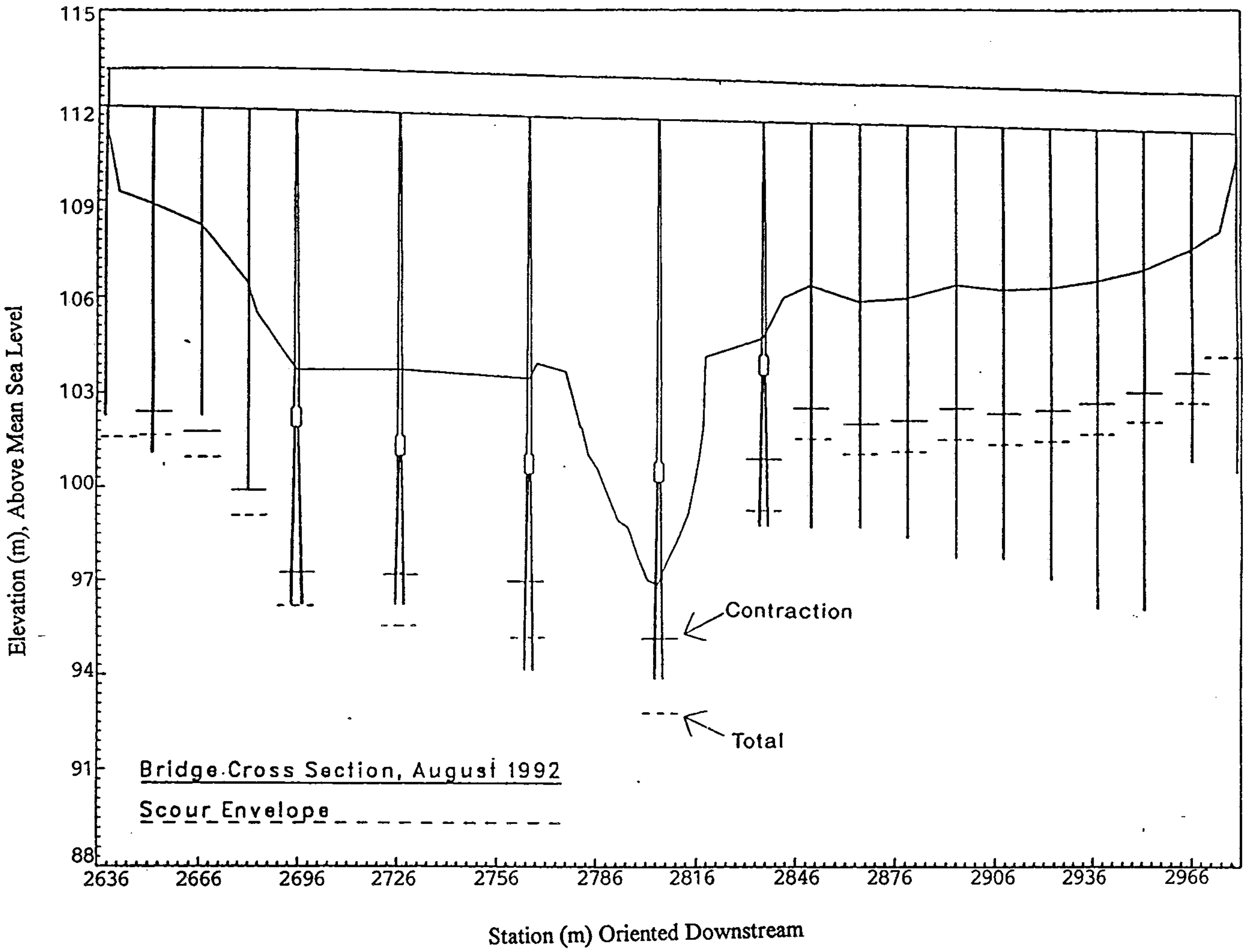

Figure 5.2. Scour Envelope (500-year Discharge) for Bridge Section Where State Highway Crosses the Guadalupe River Near Belmont, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

5.2.2. Analysis

To study this problem, a rectangular area comprising the crossing and the large meander located upstream of the crossing was selected as shown in Figure 5.1. The size of the area selected is 918 x 918 m^2 (3010 x 3010 ft^2). To study this area in the facility, the optimum scale selected was 1:36 with which the entire area would be fit into the facility. With this scale, the model parameters were calculated as shown in the table below. This kind of meandering problem can be studied only in the 3-D facility. For the above scale, the Reynolds numbers are calculated as 90,392 and 35,275 for maximum and minimum depths, respectively, using a mean velocity of 0.24 m/sec (0.79 ft/sec) in the model. The flow in the model is turbulent for both maximum and minimum depths because the Reynolds numbers are greater than 2000, as discussed in Section 3.1.3.

In this case study, Froude modeling was employed to scale the flow; that is, Froude numbers for both model and prototype were made to match for each flow. D_{50} in the model is a median bed material size which is about 0.0061 mm and is a cohesive material. Cohesive material would not move at all in the model. To overcome this problem, light weight material with a submerged specific gravity of 0.35 and D_{50} of 0.2 mm was used. This will assure a mobile bed and also some suspension. The prototype and model parameters were computed using a scale of 1:36 and given below.

<u>Condition for initiation of motion</u>: At a higher Reynolds number, any particle will be dislodged for a 'ψ' value larger than 0.056. For the above prototype conditions, ψ = 2.87. This value is about 51 times the value required for the initiation of motion. For the model conditions, if the real material from the prototype is used, ψ = 0.08. This value is about 1.43 times the value required for the initiation of the motion. If the light weight material with a submerged specific gravity of 0.35 and D_{50} of 0.2 mm is used, ψ takes a value of 0.42 which is about 7 times the value required for the initiation of motion.

	Prototype	**Model**
Mean velocity, m/sec (ft/sec)	1.45 (4.74)	0.24 (0.79)
500-yr discharge, m^3/sec (ft^3/sec)	2587 (91,400)	0.33 (11.75)
Width of the area selected, m (ft)	918 (3010)	25.5 (83.60)
Length of the area, m (ft)	918 (3010)	25.5 (83.60)
Max. flow depth, m (ft)	13.46 (44.12)	0.375 (1.23)
Mean depth, m (ft)	5.22 (17.12)	0.146 (0.48)

C.S. area of the flow, m^2 (ft^2)	1794 (19,289)	1.39 (14.88)
Hydraulic gradient	0.0002	0.0002
Fr. number	0.28	0.28
D_{50}, mm	0.22	0.0061
Flow volume, m^3 (ft^3)	1.64×10^6 (5.8×10^7)	35.21 (1244)

5.2.3. Conclusions

For the above scale, the Reynolds number in the prototype is calculated as 90,392, and it is 35,275 in the model which indicates the flow is turbulent in both cases. The flow volume in the model is computed as 35.21 m^3 (1244 ft^3) which can be used in the design of the sump. Meandering on the upstream side of the crossing may change the velocity distribution, location of the banks, and sediment load in the water at the bridge crossing, which would eventually effect the scour at the bridge piers. This kind of problem can be studied in a 3-D modeling facility.

5.3. CASE STUDY 2 - COLORADO RIVER

In this case study, prototype and model parameters are estimated for the site near Austin where State Highway 973 is crossing the Colorado River. The Colorado River in the vicinity of FM 973 is a perennial stream with a sand and clay bed. The crossing is located on a large bend with a radius of 2440 m (8000 ft). Some reservoirs are located upstream which will deplete the sediment flow through the bridge. The banks are lined heavily with vegetation cover. A D_{50} of 0.52 mm was estimated from the particle size distribution curve. The 100-yr and 500-yr peak discharges are 2103 m^3 (74,300 ft^3) and 2708 m^3 (95,700 ft^3), respectively. The site selected to model in the facility is shown in Figure 5.3, and the cross section with the predicted scour envelope at the bridge location is shown in Figure 5.4.

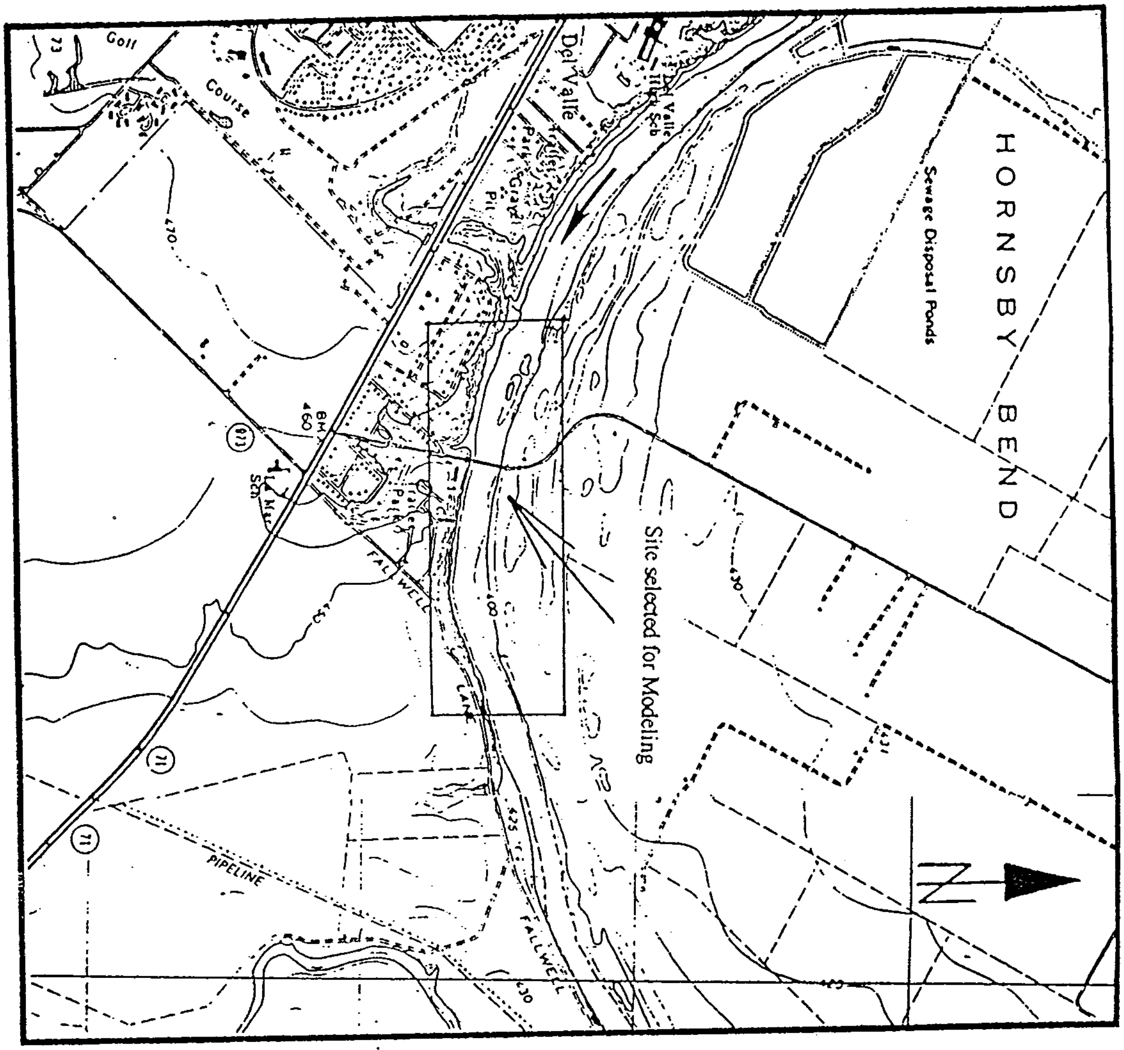

Figure 5.3. Topographic Map of Bridge Site Where FM 973 is Crossing the Colorado River Near Austin, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

5.3.1. Objective

The Colorado River at the present site has a large bend with a radius of 2440 m (8000 ft). Due to this bend, there may be considerable change in velocity distribution across the river at the bridge location. The objective here is to study the change in the velocity distribution

5.3.2. Analysis

The width of the channel at this crossing is 136.34 m (447 ft) which is taken from the report. Using WSPRO, the total width of the flood plain for the 500-yr recurrence interval peak flow is estimated as 228.14 m (748 ft). The bridge is located on a large bend with a radius of 2440 m (8000 ft). In this case study, the effect of the bend on the velocity distribution in one-dimension, can be found by using WSPRO. However, it is important to study this problem in the modeling facility to determine the change in velocity distribution in three-dimension. The site selected is 1449.36 m (4752 ft) in length and 483.4 m (1585 ft) in width, which is selected arbitrarily. However, the logical approach would be to model as large an area as possible while maintaining turbulent flow in the model. Most of this length is assumed to be on the upstream side of the bridge location because the pattern of the river on the upstream of the crossing will affect the velocity distribution at the crossing. From the survey of the bridge cross-section (Figure 5.4), it was found that the predicted scour depth is significant around all the piers across the section. Because the width of the channel is comparatively less, some part of the flood plain can also be modeled in the facility. The model and prototype parameters are given below. The scale used in this case study is 1:40.

<u>Condition for initiation of motion</u>: The Reynolds number corresponding to maximum flow depth is calculated as 84,820, and it is 31,582 for mean flow depth which indicates the flow is turbulent in both cases. At a higher Reynolds number, any particle will be dislodged for a 'ψ' value larger than 0.056. For the above prototype conditions, ψ = 2.47. This value is about 44 times the value required for the initiation of motion. For the Model conditions, if the real material from the prototype is used, ψ = 0.062. This value is about 1.1 times the value required for the initiation of motion. If the light weight material with a submerged specific gravity of 0.35 and D_{50} of 0.2 mm is used, ψ takes a value of 0.76 which is about 14 times the value required for the initiation of motion.

	Prototype	Model
Mean velocity, m/sec (ft/sec)	1.86 (6.11)	0.296 (0.97)
500-yr discharge, m^3/sec (ft^3/sec)	2708 (95,700)	0.27 (9.50)
Width of the main channel, m (ft)	136.34 (447)	3.42 (11.20)
Length considered, m (ft)	1586 (5,200)	39.65 (130.0)
Width considered, m (ft)	483.4 (1585)	12.08 (39.60)
Max. flow depth, m (ft)	11.41 (37.41)	0.287 (0.94)
Mean depth, m (ft)	4.27 (14.0)	0.11 (0.35)
C.S. area of the flow, m^2(ft^2)	1428 (15351)	0.89 (9.60)
Hydraulic gradient	0.0005	0.0005
Fr. number	0.24	0.24
D_{50}, mm	0.52	0.013
Flow volume ,m^3 (ft^3)	2.26×10^6 (7.99×10^7)	35.32(1248)

5.3.3. Conclusions

In the above analysis, the river with a large extent of flood plain is considered. The portion of the river also has a large bend 1586 m (5,200 ft) in length and 483.4 m (1585 ft) wide. This strip of the river was modeled in the facility. When this portion of the river is scaled down with a 1:40 scale, the Reynolds number is 31,170 in the model for the mean velocity of 0.296 m/sec (0.97 ft/sec) and for mean depth of 0.11 m (0.35 ft), and the flow is turbulent. For the above scale, the model dimensions will be 39.65 m (130 ft) x 12.2 m (40 ft) which can be easily fit into the 3-D facility. The above analysis also indicates that a smaller scale than 1:40 can be used without problem if only the hydraulic conditions are modeled to study the velocity distribution across the river bend.

5.4. CASE STUDY 3 - TRINITY RIVER

This case study involves the State Highway 7 crossing over the Trinity River near Crockett. The Trinity River in the vicinity of SH 7 is a perennial stream with a sand and clay bed. The banks in the vicinity of the bridge are steep and appear to be highly unstable. Significant erosion is apparent throughout the bridge reach. A D_{50} of 0.20 mm was estimated from the mean particle-size distribution curve.

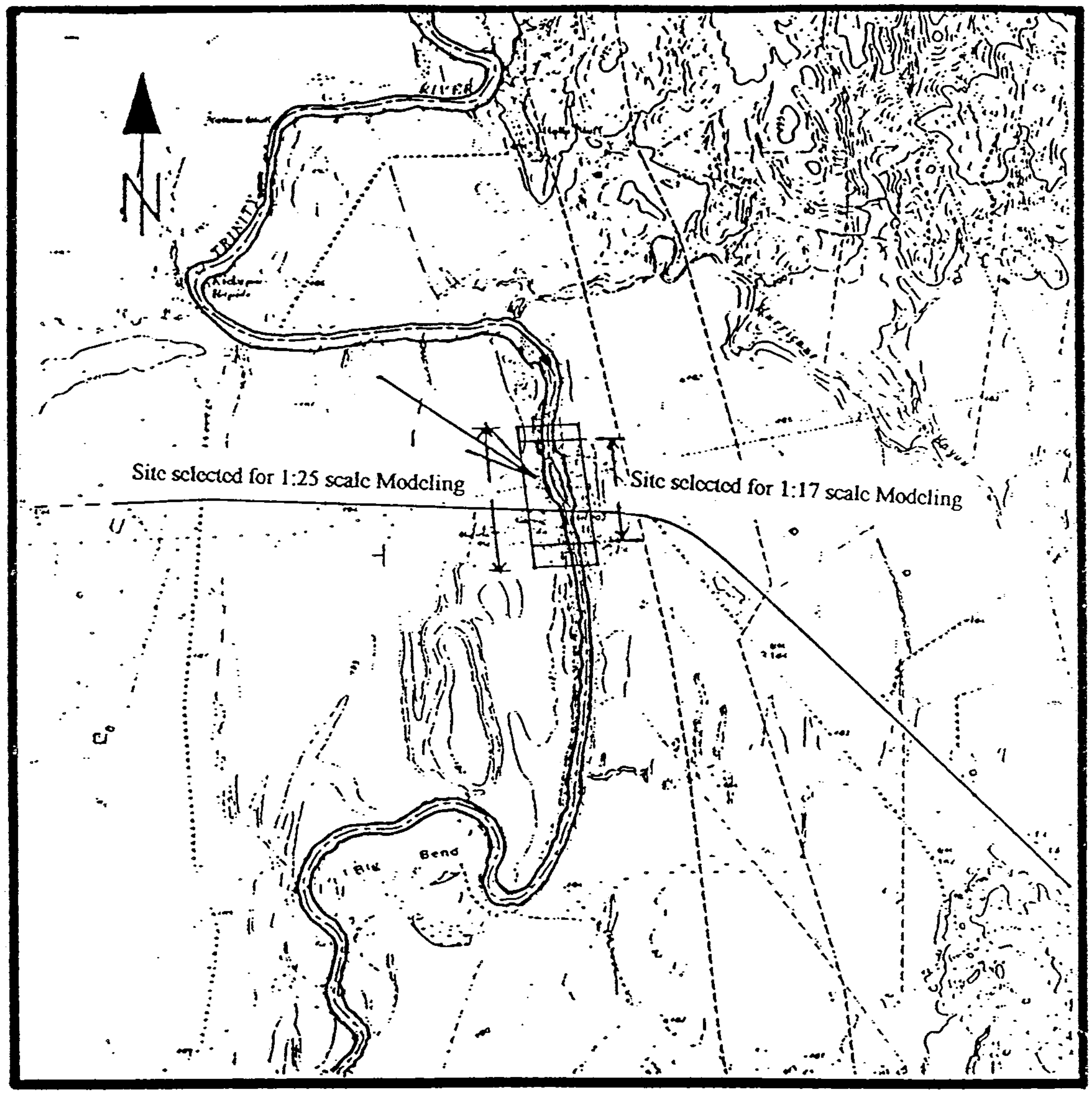

Figure 5.5. Topographic Map of Bridge Site Where State Highway 7 Crosses the Trinity River Near Crockett, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

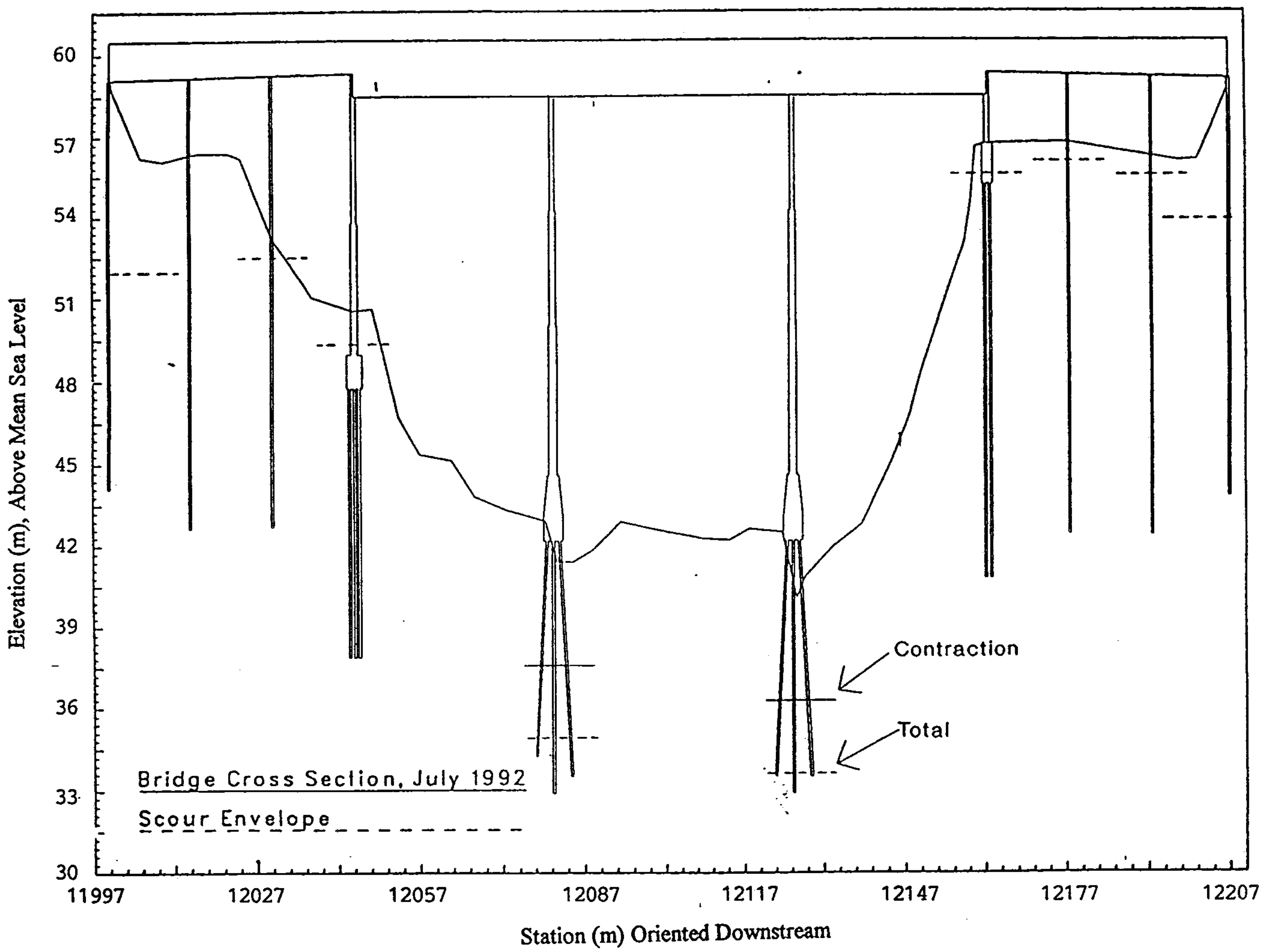

Figure 5.6. Scour Envelope (500-year Discharge) for Bridge Section Where State Highway 7 Crosses the Trinity River Near Crockett, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

The 100-yr and 500-yr peak discharges were determined to be 3297 m^3 (116,500 ft^3) and 4322 m^3 (152,700 ft^3), respectively. The site selected to model in the facility is shown in Figure 5.5, and the cross section with the predicted scour envelope at the bridge location is shown in Figure 5.6.

5.4.1. Objective

The predicted scour depth is significant in the main channel as shown in Figure 5.6, and the foundation would be undermined if it is not protected. Because the scour depth is not significant in the overbank areas, only the main channel needs to be considered for physical modeling. The main objective here is to model the main channel for detailed analysis of scour depth and placement of scour counter measures.

5.4.2. Analysis

According to the hydraulic calculations and the scour equations, significant scour occurs only at the two central piers of the bridge as can be seen in the scour envelope (Figure 5.6). Because scour around the piers in the left and right overbank areas is not significant, only a part of the channel is taken for modeling in the facility. To model this part of the channel, discharge in this part of the channel, width, depth, area of cross-section, and water surface elevation are computed using WSPRO. The flood plain is not modeled in this example because the scour depth is not significant in the flood plain. Some of the model and prototype parameters are given below. The total width of the channel considered is 103 m (338 ft). Scales of 1:17 and 1:25 are used in this case study. The length of the prototype that can be modeled in the facility is 622.2 m (2040 ft) if a 1:17 scale is used, and 851 m (2790 ft) if a 1:25 scale is used.

<u>Condition for initiation of motion:</u> At a higher Reynolds number, any particle will be dislodged for a 'ψ' value of 0.056. For the above prototype conditions, $\psi = 4.53$. This value is about 80 times the value required for the initiation of motion. For the model conditions, if the real material from the prototype is used, $\psi = 0.226$. This value is about 4 times the value required for the initiation of motion. If a light weight material with a submerged specific gravity of 0.35 and D_{50} of 0.2 mm is used, ψ takes a value of 1.056 which is about 19 times the value required for the initiation of motion.

	Prototype	**Model Scale 1:17**
Mean velocity, m/sec (ft/sec)	1.49 (4.89)	1.15 (0.35)
500-yr discharge, m^3/sec (ft^3/sec)	1885(66,600)	1.58 (55.89)
Width of the main channel, m (ft)	209.5 (687)	12.33 (40.41)
Left edge of the channel, m (ft)	159.2 (522)	
Right edge of the channel, m (ft)	262.3 (860)	
Width of the channel considered, m (ft)	103 (338)	6.1 (19.88)
Max. flow depth, m (ft)	17.6 (57.71)	1.03 (3.39)
Mean depth, m (ft)	12.66 (41.51)	0.74 (2.44)
C.S. area of the flow, m^2 (ft^2)	1304 (14,021 ft^2)	4.51 (48.52)
Hydraulic gradient	0.0001	0.0001
Fr. number	0.21	0.21
D_{50}	0.20	0.012
Flow volume, m^3 (ft^3)	$1.11 \times 10^6 (3.91 \times 10^7)$	165 (5822)

	Prototype	**Model Scale 1:25**
Mean velocity, m/sec (ft/sec)	1.45 (4.75)	0.29 (0.95)
500-yr discharge, m^3/sec (ft^3/sec)	1885(66,600)	0.60 (21.31)
Width of the main channel, m (ft)	209.5 (687)	8.38 (27.48)
Left edge of the channel, m (ft)	159.2 (522)	
Right edge of the channel, m (ft)	262.3 (860)	
Width of the channel considered, m (ft)	103 (338)	4.12 (13.52)
Max. flow depth, m (ft)	17.6 (57.71)	0.7 (2.31)
Mean depth, m (ft)	12.66 (41.51)	0.51 (1.66)
C.S. area of the flow, m^2 (ft^2)	1304 (14,021)	2.09 (22.43)
Hydraulic gradient	0.0001	0.0001
Fr. number	0.21	0.21
D_{50}	0.20	0.012
Flow volume, m^3 (ft^3)	$1.11 \times 10^6 (3.91 \times 10^7)$	71.0 (2503)

5.4.3 Conclusions

In this case study, the scour is not significant around the piers in the left and right overbanks. The scour is significant only in the main channel around the piers located in the central part of the river. Therefore, it is concluded that it is necessary to test only that part of the channel in the modeling facility. So, some part of the channel, whose width is 103.1 m (338 ft), was considered where the scour is significant. With 1:17 scale the model flow volume was computed as 165 m^3 (5822 ft^3). Since this model flow volume is relatively large, a scale of 1:25 was selected, and the model parameters were computed as shown in the above table. Because the modeling of the flood plain is not necessary, and also the depth is relatively large, the two-dimensional facility should be used for this study.

5.5. CASE STUDY 4 - GUADALUPE RIVER

This case study involves the US Highway 183 crossing over the Guadalupe River near Hochheim, Texas. The Guadalupe River in the vicinity of this crossing is a perennial stream with a sand and clay bed. Significant meanders are present upstream of the crossing. The channel is located near the left boundary of the flood plain where the terrain rises sharply up a steep hill. A D_{50} of 0.60 mm was estimated from the mean particle size distribution curve. The 100-yr and 500-yr peak discharges were found to be 4302 m^3 (152,000 ft^3) and 7839 m^3 (277,000 ft^3), respectively. The site selected to model in the facility is shown in Figure 5.7, and the cross section with the predicted scour envelope at the bridge location is shown in Figure 5.8.

5.5.1. Objective

In this case study, the width of the flood plain is 1322.18 m (4335 ft) which is considered to be very large compared to the other flood plains. The main objective of this case study is to study the scour around the piers located across the entire channel cross-section due to pressure flow which will occur if the discharge exceeds 4613 m^3 (163,00 ft^3).

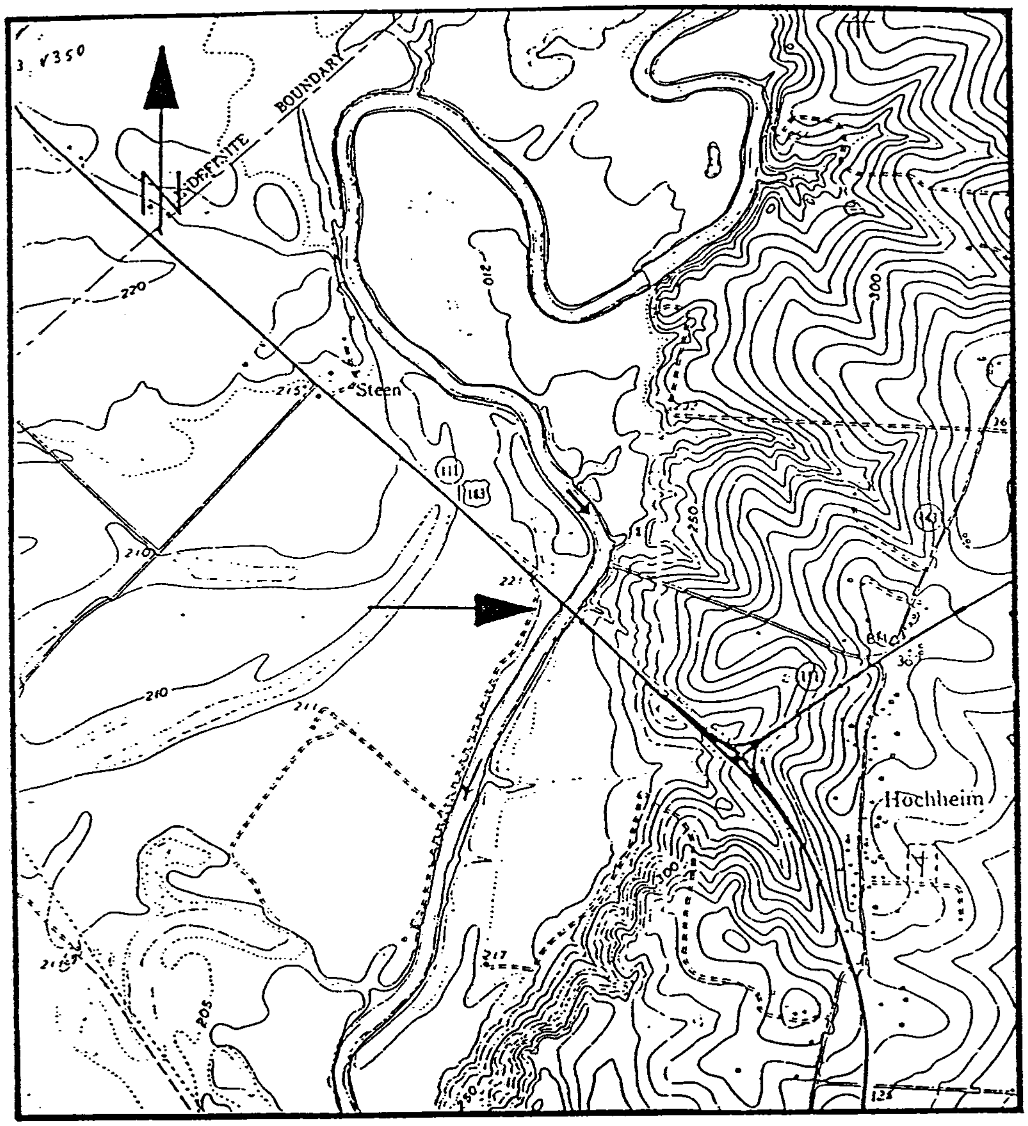

Figure 5.7. Topographic Map of Bridge Site Where US Highway 183 is Crossing the Guadalupe River Near Hochhiem, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

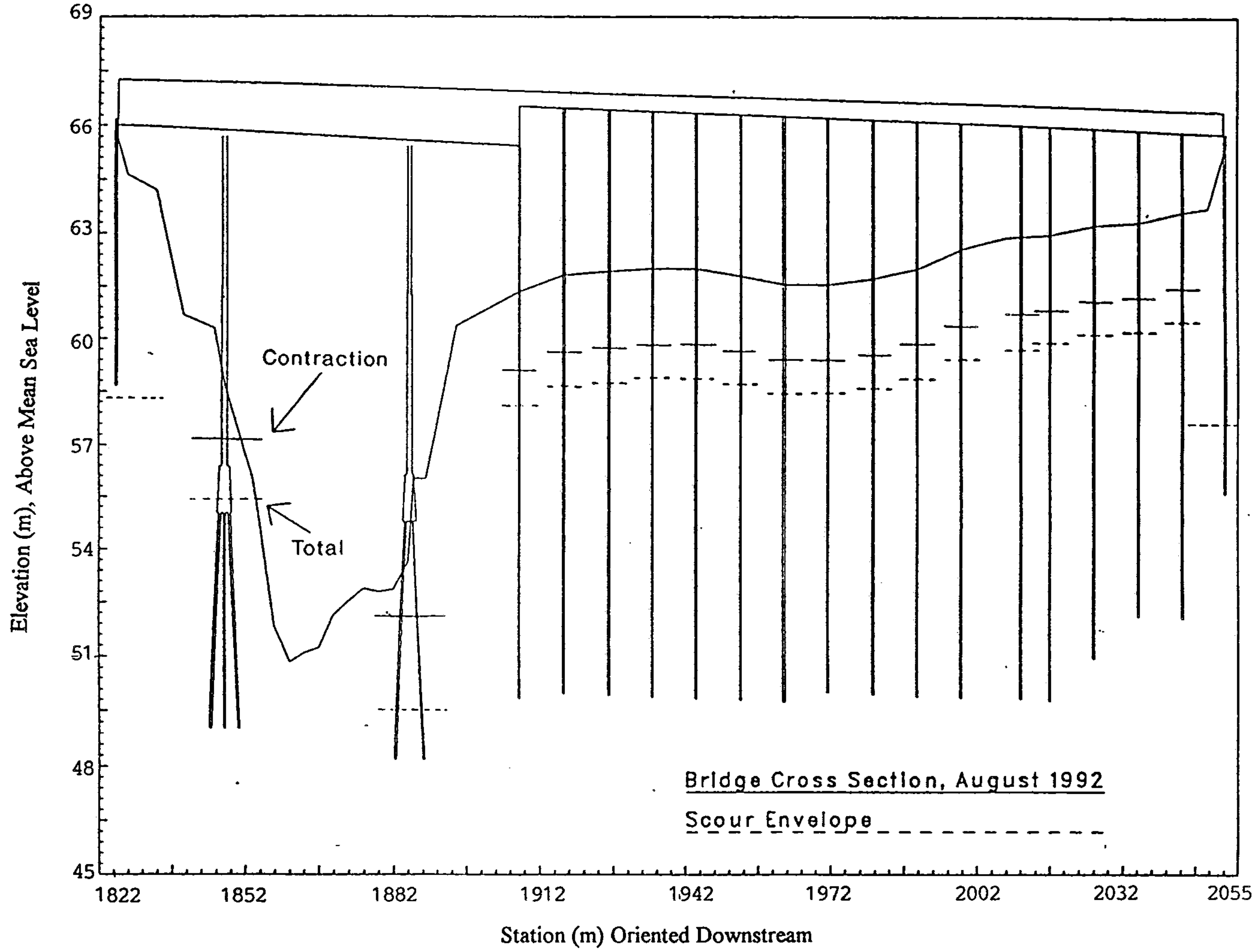

Figure 5.8. Scour Envelope (500-year Discharge) for Bridge Section where US Highway 183 Crossing the Guadalupe River Near Hochheim, Texas

(Source : Level II Bridge Scour Analysis, January, 1993 by USGS, Water Resources Division, Texas)

5.5.2. Analysis

The Guadalupe River in the vicinity of US 183 is a perennial stream with significant meanders present upstream of the crossing. The overtopping discharge that would reach the low steel elevation of the main bridge was estimated as 4613 m^3 (163,000 ft^3). This discharge will be the largest discharge in all of the case studies, and the facility designed for this discharge will probably have the largest sump and highest pump capacity if we use a large scale model in the 2-D flume. So, this case study is more useful for the design of the above components of the facility.

Because the width of the flood plain is 1322 m (4,335 ft) compared to the main channel width of 232 m (760 ft), it may also be required to model the flood plain. But it may not be necessary to model the entire flood plain. Some part of it can be truncated by analyzing velocity distribution on the flood plain using FESWMS.

Some of the prototype and model parameters computed with a scale of 1:50 are given below. For the above scale, the mean depth in the channel is very small. This very small depth might not provide reliable information on scour depth. Therefore, these values of the 3-D model may be good for studying the flow distribution. If the scour depth is to be studied only in the main channel, a scale of 1:15 is selected and the model parameters are computed as shown in the following table. In the model (1:50), the Reynolds number is calculated as 9,502 using a mean velocity of 0.21 m/sec (0.69 ft/sec) and a mean flow depth of 0.05 m (0.15 ft). The above Reynolds number indicates that the flow is turbulent in the model, since it is greater than 2,000.

Condition for initiation of motion: At a higher Reynolds number, any particle will be dislodged for a 'ψ' value of 0.056. For the above prototype conditions, $\psi = 1.18$. This value is about 21 times the value required for the initiation of motion. For the model conditions, if the real material from the prototype is used, $\psi = 0.02$. This value is about 0.41 times the value required for the initiation of motion. Therefore, the material may not move. If the light weight material with a submerged specific gravity of 0.35 and D_{50} of 0.2 mm is used, ψ takes a value of 0.32 which is about 6 times the value required for the initiation of motion.

	Prototype	Model Scale 1: 50
Mean velocity, m/sec (ft/sec)	1.49 (4.89)	0.21 (0.69)
500-yr discharge,m^3/sec (ft^3/sec)	4613 (163,000)	0.261 (9.22)
Width of the main channel, m (ft)	232 (760)	4.64 (15.2)
Width of the flood plain , m (ft)	1322.18 (4,335)	26.44 (86.7)
Length, m (ft)	2135 (7000)	42.7 (140)
Max. flow depth, m (ft)	15.39 (50.46)	0.31 (1.01)
Mean depth, m (ft)	2.35 (7.70)	0.05 (0.15)
C.S. area of the flow, m^2 (ft^2)	3103 (33,360)	1.24 (13.34)
Hydraulic gradient	0.0005	0.0005
Froude number	0.39	0.39
D_{50}, mm	0.6	0.012
Flow volume, m^3 (ft^3)	6.61×10^6 (2.34×10^8)	52.86 (1868)

To model this as a 2-D model, a part of the main channel is selected from the left edge of the channel at 1824 m (5980 ft) to the right edge of the channel at 1908 m (6257 ft), i.e., a total width of 84.5 m (277 ft). The model parameters for 1:15 scale are shown in the following table.

	Prototype	Model Scale 1:15
Mean velocity, m/sec (ft/sec)	1.49 (4.89)	0.38 (1.26)
500-yr discharge, m^3/sec (ft^3/sec)	1072 (37873)	1.23 (43.5)
Width of the channel, m (ft)	84.49 (277)	5.64 (18.5)
Width of the flood plain, m (ft)	1322.18 (4,335)	26.44 (86.7)
Length, m (ft)	549 (1800)	36.6 (120)
Max. flow depth, m (ft)	15.39 (50.46)	1.03 (3.36)
Mean depth, m (ft)	8.53 (27.96)	0.57 (1.86)
C.S. area of the flow, m^2 (ft^2)	720.5 (7745)	3.2 (34.4)
Hydraulic gradient	0.0005	0.0005
Fr. number	0.39	0.39
D_{50}, mm	0.6	0.04
Flow volume, m^3 (ft^3)	3.94×10^5 (1.39×10^7)	117 (4128)

5.5.3 Conclusions

The width of the flood plain in this case study is extremely large and can be modeled only in the three-dimensional facility. Scour is also significant in the entire cross-section. The overtopping discharge is very large in the river, which, when modeled in the facility, can be used for sizing the pump capacity and sump size. These sizes are determined later in the design of the facility. To avoid laminar flow for low discharges, some kind of artificial roughness may be required in the model. A scale of 1:50 was chosen to model the river in the 3-D model. The total length of the prototype that can be modeled in the facility is 1982.5 m (6,500 ft) with a model dimension of 39.65 m (130 ft). If this is considered in the 2-D model, the flow capacity is calculated as 1.23 m^3/sec (43.5 ft^3/sec).

5.6. CASE STUDY 5 - NAVASOTA RIVER:

In this case study, a section of the Navasota River which is shown in Figure 5.9 is taken, and velocity distribution across the flood plain is found using FESWMS. Figure 5.9 shows contours for a section of the Navasota River. The upstream open boundary is at the top of the map, while the downstream open boundary is the solid line near the bottom of the map. The flow rate is taken as 849 m^3/sec (30,000 ft^3/sec), and the corresponding downstream water surface elevation is 65.58 m (215 ft). The map scale is 1 in = 1500 ft. The roughness coeffient of the shaded area is 0.15, and the open area roughness coefficient is 0.08.

5.6.1. Objective

Modeling the entire flood plain in the physical modeling facility is difficult because of scale selection. The flood plain can be truncated knowing the velocity distribution on the flood plain. The objective here is to demonstrate how the flood plain can be truncated before modeling in the facility.

5.6.2. Analysis:

The total flood plain is divided into number of elements as shown in Figure 5.10. Initially, a network of elements is designed and plotted using DINMOD. Ground contours are also plotted using DINMOD and shown in Figure 5.11 in combination with the network of elements. A discharge of 849 m^3/sec (30,000 ft^3/sec) and a corresponding downstream water surface elevation of 65.58 m (215 ft) are used as the input data for FLOMOD and water surface elevations at regular intervals, velocity vectors, and velocity distribution are found across the flood plain. The velocity vectors and water surface contours are plotted using ANOMOD and shown in Figure 5.12. Let 'A' be a point in the flood plain which is on the left side of the river and 'B' be a point in the flood plain which is on the right side of the river as shown in Figure 5.12. Because it is difficult to model the entire width of the flood plain, it is truncated and the portion which is between A and B can be considered for modeling. The discharge passing across AB can be calculated since the depth and velocity at any given point in this portion of the flood plain is known. This discharge will be used in the model while maintaining the velocity distribution and water surface elevation across AB in the model. The width of the flood plain can be truncated along the entire length of the flood plain, i.e., from the upstream boundary to the downstream boundary so the central portion including the river can be modeled for any discharge after finding the velocity distribution and water surface elevations by using FESWMS.

5.6.3. Conclusions

The width of the flood plain in this case study needs to be truncated in order to fit into the facility using the reasonable scale. The left side of point A and right side of point B is truncated, and the portion across AB is to be modeled after knowing the velocity distribution, water surface elevation, and discharge which are computed by FESWMS. This description is for the section across the flood plain and along AB. Similarly, the width of the flood plain can be truncated along the length on both sides of the river. As shown in Figure 5.12, the portion between the lines PAQ and RBS can be modeled in the facility when the velocity distribution, water surface elevation, and discharge obtained from FESWMS is reduced to scale and by maintaining them in the model.

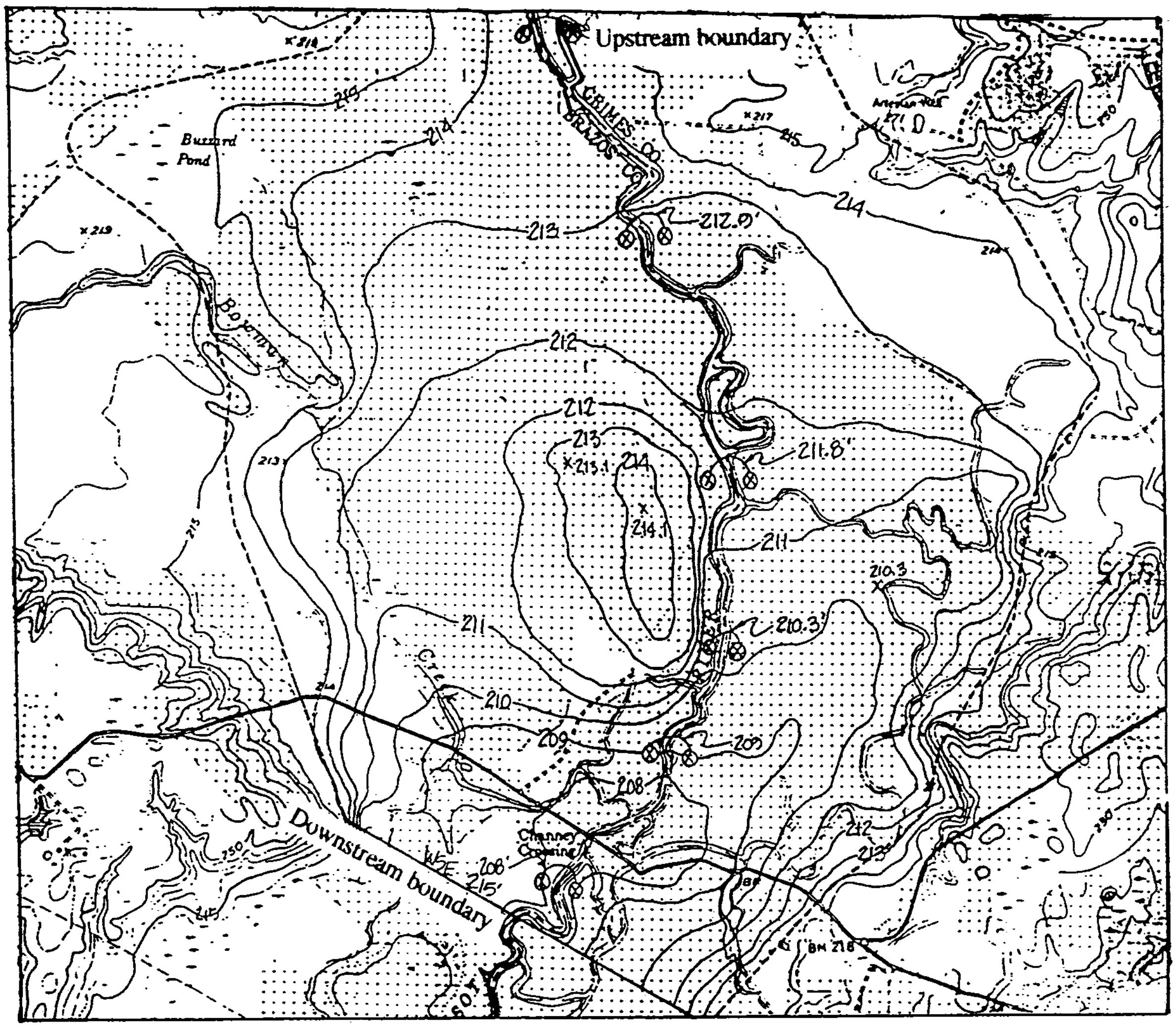

Figure 5.9. Topomap of a Section of Navasota River and its Flood Plain

(Source: Finite Element Surface Water Modeling System (FESWMS-2DH)-Users Manual)

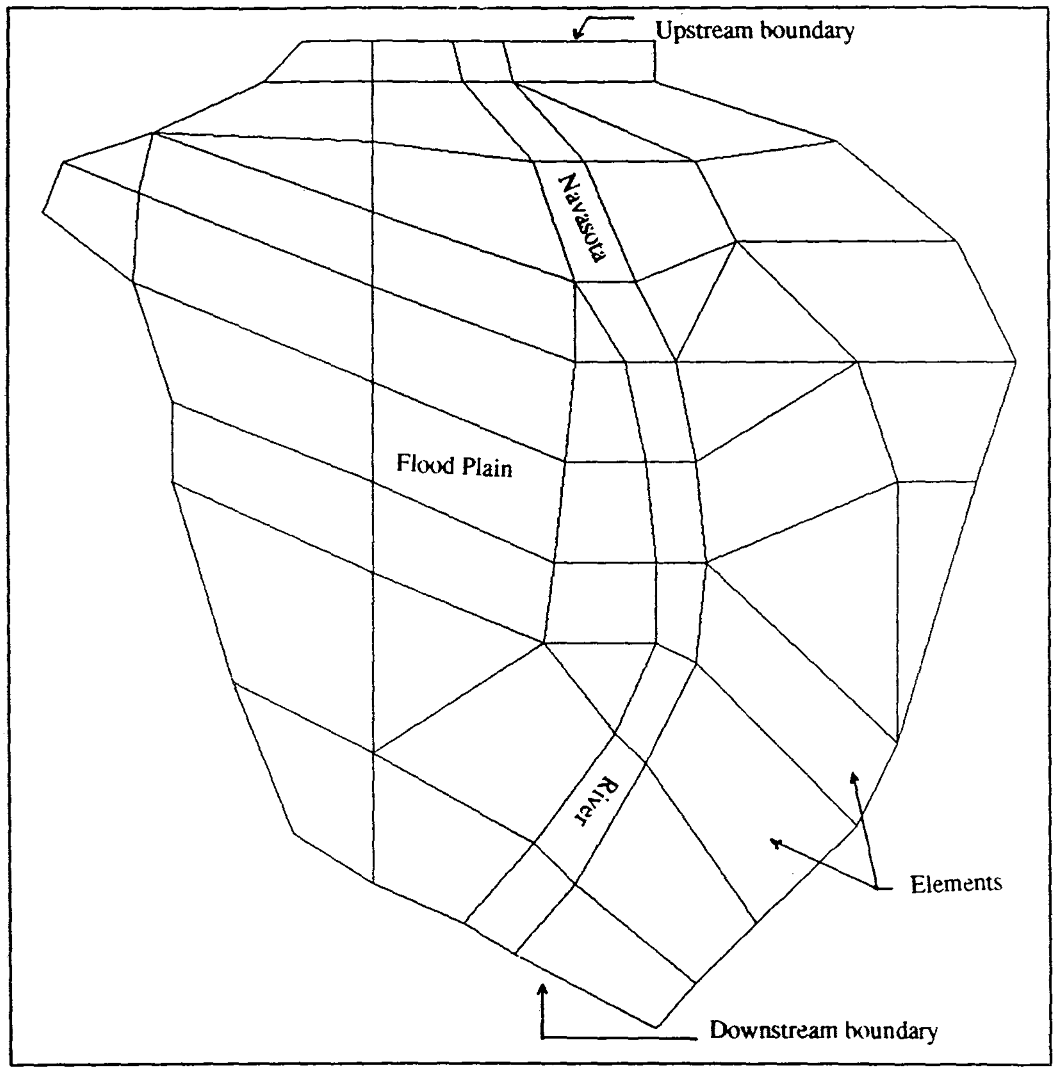

Figure 5.10. Network of Elements on the Flood Plain of Navasota River

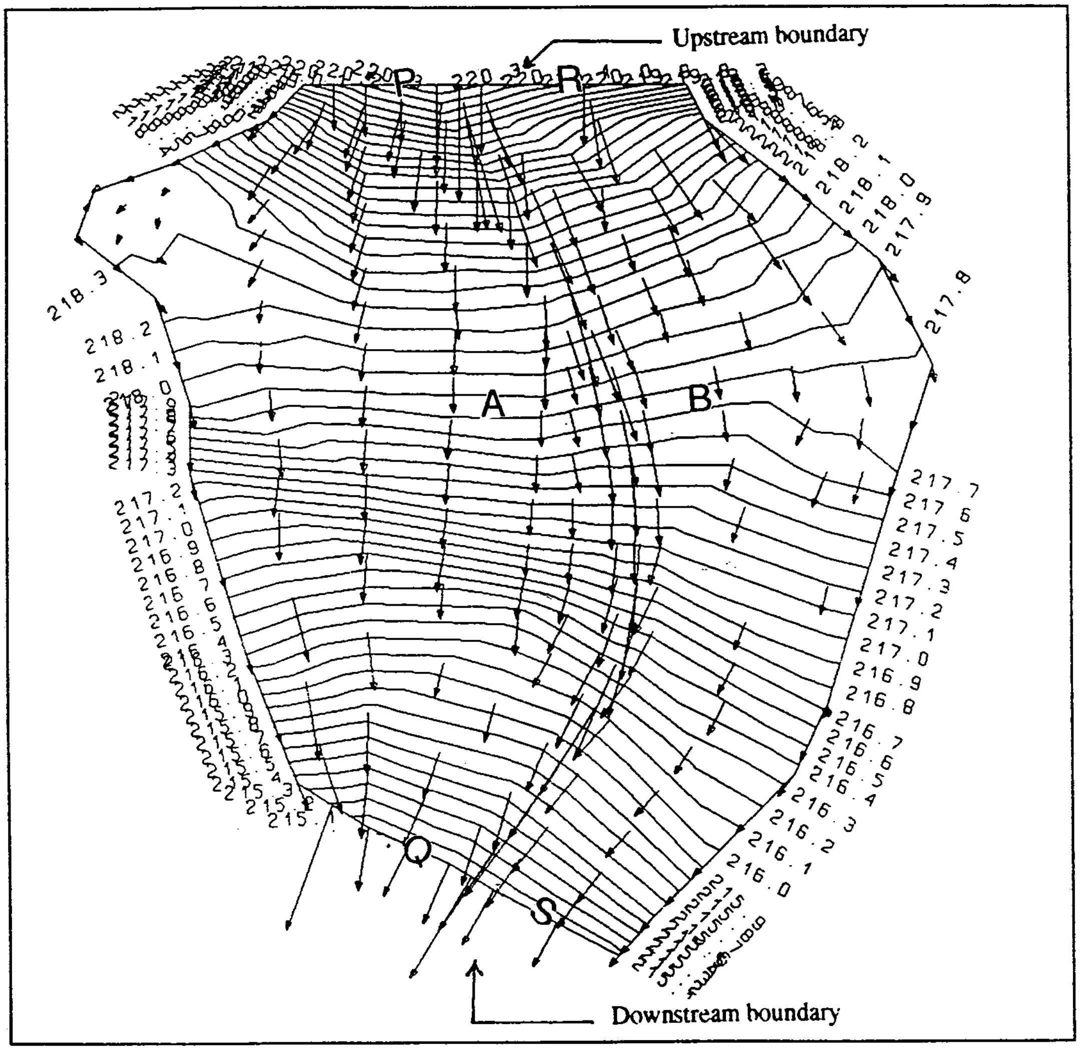

Figure 5.12. Water Surface Contours and Velocity Vectors on the Flood Plain

6. SURVEY OF EXISTING HYDRAULIC LABORATORIES

6.1. OBJECTIVE OF THE SURVEY

At the meeting held in College Station with TxDOT officials and others in November 1993, it was decided to visit some major hydraulic laboratories in the country to evaluate the type of models and scales used at existing large facilities. It was also decided at the same meeting to prepare a questionnaire to be sent to all the labs that were to be visited by the principal investigators of the project. The facilities visited were the USAE Waterways Experiment Station (WES), Vicksburg, Mississippi; the Federal Highway Administration (FHWA) Hydraulic Laboratory, McLean, Virginia; the University of Minnesota (St Antony Falls) Minneapolis, Minnesota; and Colorado State University (CSU) Fort Collins, Colorado.

6.2. THE QUESTIONNAIRE

A questionnaire was prepared to circulate to the persons-in-charge of the four above mentioned hydraulic facilities. The prepared questionnaire was circulated to all the persons present at the meeting to obtain their opinions. The questionnaire was organized into four sections as listed below:

1. Personnel and their experience
2. Physical dimensions and instrumentation
3. Physical modeling
4. Cost analysis

The questions in each section are presented here. All the questions were answered when the principal investigators visited these facilities.

6.2.1. Personnel and Experience

1. Please check one. You are a (a) Technician (2) Graduate Student (3) Professor
2. How long have you been involved with hydraulic facilities?
3. How long have you been involved with this facility?
4. Is this facility indoor or outdoor?
5. What is your opinion of an outdoor facility?

6. How many people are employed in your facility, and what is their expertise?
7. What type of problems do you model in your facility?

6.2.2. Physical Dimensions and Instrumentation

1. What kind of facility do you have to study river hydraulics and bridge scour problems?
2. What are the physical dimensions of the facility?
3. How do you arrive at these dimensions?
4. What was the criterion used to decide on the maximum flow rate that needed to be handled by the facility?
5. What flow control devices do you have to control
   (A) The discharge into the flume?
   (B) The water surface profile?
6. How do you recirculate the flow?
7. Where is the location of the pumps in your facility?
   What type of pumps do you use?
8. What is the instrumentation used in your facility to measure flow velocity and bed profile?
9. What additional instrumentation would you like to have in your facility?
10. What improvement would you like to make to your facility?

6.2.3. Physical Modeling

1. What problems associated with river modeling and scour modeling have you studied in your facility?
2. What kind of prototype to model scale ratios do you typically use?
   Do you use distorted models or undistorted models?
3. How do you model and scale cohesive soil in the facility?
4. How do you model and scale cohesionless soil in the facility?
5. Have you ever modeled rate of scour in your facility? If yes, how do you relate the rate of scour in the model to the rate of scour in the field?
6. How do you convert the scour depth in the model to the scour depth in the field ?
7. Have you ever conducted testing for stream stability problem? If yes, what kind of problem did you study?
8. How do you model a river bend?
9. How do you model non-uniform flow in similar situations?

10. How do you approach the problem of modeling the extent of a flood plain?

11. If you were asked to develop a new and large facility to simulate scour problem, what would you pay particular attention to?

6.2.4. Cost of the Facility

1. What is the cost of the major installations in your facility?
   - (a) Pumps
   - (b) Flumes
2. How much would the facility cost, if it were built today?
3. What is the current operation and maintenance cost?
4. How do you meet this expenditure?
5. What is the availability of the facility ? Is there any waiting time?
6. What are the charges normally collected for usage of the facility?

6.3. THE VISITS

The visits to the above mentioned facilities took place between January 3-6, 1994 and on January 21st, 1994 (CSU). The main objective of these visits was to evaluate the existing facilities for their use by the TXDOT and also examine whether these facilities can be effectively utilized to study the scour problem in bridges. The summary of results is presented in tabular form as follows:

6.3.1. Personnel and Experience

Q.No		WES	FHWA	U 0f Minnesota	CSU
1	Person Interviewed	Facility Manager	Facility Manger	Facility Manager and Faculty Member	Facility Manager
2	Experience with Hydraulic Facilities	32 Yrs	30 Yrs	12 Yrs & 21 Yrs	25 Yrs
3	Experience with Present Facility	32 Yrs	10 Yrs	12 Yrs & 16 Yrs	5 Yrs
4	Indoor or Outdoor Facility	Mainly Indoor	Indoor	Indoor	Both
5	Is Outdoor facility Necessary?	Limitation is Wind and Weather	Limitation is Climate	Climatic Limitation	Necessary but limited by Weather

Q. No		WES	FHWA	U of Minnesota	CSU
6	No. of Persons Employed	140 in Hyd. Lab REs, Scientists, Technicians	2 Full-time 2 Part-time	12 Faculty 7 Staff 36 Graduate Students	12 Faculty 1 RA 25 Graduate and Undergraduate Students 7 in Work Shop
7	Type of Problems Modeled	* Any Hydraulic Structure * Movable and Fixed Bed Models	*Highway drainage *Culvert *Scour *Abutments	*General Hydraulics *Wind Engineering *Intake Structures *Spillways *Movable Bed Models	*Erosion & Sedimentation *Stream Stability *Hydraulic Structures *Hydromechanics *Flow Measurements

6.3.2. Physical Dimensions and Instrumentation

Q.No		WES	FHWA	U of Minnesota	CSU
1	Type of Facility And Description	Study of Various Hydraulic Problems	One Dimensional Flume	3-D Flumes and Channel Flumes	Four Flumes Two 3-D Flumes
2	Physical Dimensions Length(m) x Width(m) x depth(m)	67 x 30 237 x 6.7 x 3.6 Rip rap facility	21 x 1.82 x 0.6	51 x 9.1 x 0.6 13.7 x 7.6 x 1.52 12.2 x 7.6 x 0.91 76.2 x 2.7 x 1.82 Channel	60.1 x 2.4 x 1.2 54 x 6.1 x 2.4 30 x 6.1 x 1.2 30 x 6.1 x 2.4 36 x 12 x 1.2 * Tilting type also but not necessary for Scour.
3	How Physical Dimensions are arrived at?	* Money Available *Space available	*Experience	*Available Space	*Project Specific

Q.No		WES	FHWA	U of Minnesota	CSU
4	Max. Flow Rate Criterion	*Produce the Design Stage Flood *Max. Sub-Critical Flow *Most Flood Channel 5.6 m^3/sec (200 cfs) for rip rap facility	*Velocities to be Created 0.42 m^3/sec(15 cfs)	* Design Flood (for Mississippi 8.5 m^3/sec or 300 cfs)	*Project Specific Up to 6 m/sec (20 ft/s) flume Upto 2.8 m^3/sec (100 cfs) Indoor >4.2 m^3/sec (150 cfs) Outdoor
5	Control Devices (A) For Flow (B) Water Surface Profiles	 * Venturi Meter * Valves, Weirs * Point Gauges to Measure * Tail Gates to Control	 * Bypass Valves, * Pumps with Variable Frequency Drive * Tail gate on Flume	 * Orifices * Venturi Meters * Valves *Point gauges *Tail gates	 * Orifice plate on pipe line with manometer or pressure transducer * Point gauges * Tail gate

Q. No		WES	FHWA	U of Minnesota	CSU
9	Additional Instrumentation Required	* Automation	* Direct Measure of Shear Stress (Velocity Profile) Laser- Expensive	* Automation	* Laser Doppler
10	Improvements like to have for the present Facility	Temperature Control	Recirculate Sediment	* Better Flow Control Devices * Maintenance * 440 Motors instead of 110 or 220	* Automation * Great variability

6.3.3. Physical Modeling

Q.No		**WES**	**FHWA**	**U of Minnesota**	**CSU**
1	Problems Studied	Bridge scour Development of navigation channel Rip rap protection against erosion	Protection around piers of different shapes	Stream stability Flow changes created by new structures	Pier and abutment scour, Particle size, Physical scour model, Rip rap and tetrapod for erosion control, Pressure scour
2	Scale Ratios	In River Engineering 1:100 Vertical 1:250 Horizontal Distorted Models For Hyd. Structures 1:24 to 1:36 1:10 to 1:120 Undistorted models	1:20 to 1:50 Undistorted models	1:10 to 1:500 Based on Reynolds and Froude numbers	1:3 to 1:40 Undistorted models

Q.No		WES	FHWA	U of Minnesota	CSU
3	Model Cohesive soils	Do not model	***	Clay Electrochemical Properties, Stratigraphy, Bentonite, Fly ash.	***
4	Modeling Criteria	Crushed Coal, Sand, Gravel, and Crushed stone	Do not Scale Soil, Match Incipient Velocity	$\tau^* = \frac{u_*^2}{(\rho_s/\rho - 1)gD}$ for Bed load $\frac{u_*}{V_s}$ for suspended load	Natural Soil, Scale by Length ratio, Scale by Specific Gravity
5	Rate of Scour	*****	*****	For Bridge Piers Constriction Scour	Not Important for cohesionless soils
6	Converting Scour Depth To Prototype	Qualitative rather than quantitative for hydraulic structures Believe deeper in the model than in field	D_{model} X Scale	D_{model} X Scale	By Scaling in Vertical Direction

Q.No		WES	FHWA	U. of Minneosta	CSU
7	Stream Stability	Limited as difficult to model	*****	Some Experience	Some Experience
8	Model River Bend	Right shape and go to next bend way	*****	Field radius of Curve = Scale X Radius of Curve in laboratory	In large flume
9	Nonuniform Flow	Let the river shape the flow, Use the model for inflow pattern surface pattern	*****	*****	By using manifolds Tail water control, Modify the model.
10	Flood Plain	Model to the levee and adjust horizontal scale, If needed, give up on the flood plain	*****	HEC 2 Runs. Truncate the model. 1/4 to 1/3 of flood plain at least.	Scale the whole plain and distort the model, Cut off based on experience.
11	Attention to be paid	Big Scale like 1:10 to 1:1. Large flow capacity	Nature of Problems, Reach Length, Channel Migration, Type of Materials	Smaller scale for global feature, Local scale large,	Velocity >10 ft/sec, Depth

6.3.4. Cost of the Facility

Q.No		WES	FHWA	U of Minnesota	CSU
1	Cost of Major Installations (A) Pumps (B) Flumes	1.46 m^3/sec (50 cfs) - \$ 40,000 0.28 m^3/sec (10 cfs) - \$ 15,000 30 m x 60 m with sump - \$ 2 million (Sump, Flume, Water supply and Building) Rip rap Facility \$ 1.2 Million	0.28 m^3/sec (10 cfs) - \$ 30,000 \$ 450,000 includes 0.14 m^3/sec (5 cfs) pump, Building - \$0.5 million	600 hp pump 1.46 m^3/sec (50 cfs) cost \$ 0.25 million ******	small \$30,000 For whole facility \$ 20 millions Big Indoor tilting type \$ 1.2 million. Outdoor \$ 0.4 million
2	Cost today	Mississippi river model - \$ 0.25 million Bridge scour facility - \$ 5-10 millions	10 yrs of inflation	\$ 10 million.	********

Q.No		WES	FHWA	U. of Minnesota	CSU
3	O&M Cost	Rip rap and river model operation cost - $ 25,000 per month. Maintenance - $1000 per month	$ 150,000 per year includes two people	Electricity, utilities and 3-4 people for the whole facility - $150,000 per year.	$ 40,000 per year plus backlog $ 1.0 Million
4	How to meet the expenditure	R&D budget	Budget + Support services Contract	Soft money over head	*****
5	Availability of the facility	Rip rap through 1996 67 m x 30 m (220 ft x 100 ft) available now with 0.56 m^3/sec (20 cfs)	Sets own priority - 5 year plan Plan for DOTs	Responsive cooperative	Depends Immediate to 4 months to 2 years Flexible work force.
6	Charges for testing	Approx $25, 000 per month for engineering and technical + model building	No charge for DOTs	No separate charge, Testing + construction for 9.1 m x 30 m (30 ft x 100 ft) is $ 150,000. Construction - $ 80-100 k.	Model - $ 65,000 Total - $ 150,000

6.4. IMPRESSIONS FROM THE VISITS

Following the visits to the facilities, a summary of the impressions of the principal investigators is presented below.

6.4.1. USAE Waterways Experiment Station

1. Very extensive and large scale facility.
2. The facility can be easily modified for scour studies.
3. The personnel is most experienced with river qualitative hydraulics studies in movable beds.
4. They have experience with movable beds in cohesionless soils only.
5. The soil simulant used is crushed coal, mostly loosely placed and drained.
6. They do not believe that scour in model can represent scour in the prototype quantitatively.
7. Cost appears to be high.
8. Availability could be a problem. The priority in this facility is to WES business.
9. They have no experience in cohesive soils and rate of scour.
10. Stream stability problems are not modeled in this facility because cohesiveness of the banks can not be modeled.
11. Most models used are 0.3 m of soil and 0.2 m of water or less.
12. They proposed a full scale scour experiment behind a Corps of Engineers dam.

6.4.2. FHWA Hydraulic Laboratory

1. A small laboratory when compared to other laboratories visited.
2. Small scale scour tests are conducted on specific requests from states.
3. Not big enough to develop as a facility that could be used by TxDOT in the future.
4. The facility works very well for what it was intended to be.

6.4.3. University of Minnesota Hydraulic Laboratory

1. The personnel is very knowledgeable and has sound fundamental principals.
2. They have experience in fundamental research using a combination of numerical simulation and physical modeling.
3. This is a very busy facility.

4. They conduct 75% of basic research and 25% of problem oriented research.
5. Testing area is smaller than the one in WES.
6. Water capacity is very large as the facility takes water directly from river.
7. They have a sound understanding for cohesionless soils scaling.
8. They have experience in stream stability problems and, to some extent, with the rate of scour also.

6.4.4. Colorado State University Hydraulic Laboratory

1. The personnel is knowledgeable in scour problems.
2. The research conducted in this facility is applied research.
3. This is a busy facility.
4. Water capacity is large.
5. The size of the facility is very large and can handle large 3-D and 2-D models.

6.4.5. Advantages and Disadvantages of Existing Facilities

The advantages and disadvantages of the existing facilities and of the new facility are:

For an existing facility, there is no initial investment, and experienced personnel are readily available. Overhead rates are many times higher (approximately 45%) than what can be offered under the SP&R program (7%). The availability of an existing facility is not very sure. Travel expenditure will be very high for TxDOT personnel when traveling to an existing facility compared to a Texas facility.

If a new facility is developed, the TxDOT will have low overheads (7%), and low travel expenditure. It will also develop local expertise. It will be easy to draw contracts, and availability will be a enhanced. The drawbacks will be a sizable initial investment and personnel with limited experience to start with.

7.0. NEW SCOUR FACILITY CHARACTERISTICS

7.1. INTRODUCTION

A physical modeling facility is a useful tool for studying problems associated with river hydraulics, pier scour, gradation, scour at site specific-location, sizing of rip-rap, pressure scour, constriction scour, and bank erosion. The design of a physical modeling facility requires hydraulic particulars at the bridge site and also the data regarding the soil. These data were obtained from TxDOT which included field measurements conducted at various locations on five major rivers in Texas. The following range of hydraulic conditions were provided by TxDOT.

Channel velocity	= 0.3048 m/s to 3.048 m/s (1 to 10 ft/s)
Channel discharge	= 85 m^3/sec to 5097 m^3/sec (3000 to 180,000 cfs)
Flood plain width	= 91 m to 6.4 km (300 ft to 4 miles)
Depth of river	= 1.524 m to 15.24 m (5 to 50 ft)

Hydraulic conditions may vary significantly from one river to another. Detailed information at five sites were obtained from reports on Level II bridge-scour analysis done by the United States Geological Survey (USGS) Texas District for TxDOT.

7.2. DESIGN OF THE 2-DIMENSIONAL FACILITY

A preliminary design is done for the hydraulic testing facility based on the data available from the reports by USGS. It is not feasible or economical to study all the features of scour problems in one modeling facility due to the wide range of prototype conditions that may be encountered. Considerable advantage and economy can be gained by constructing two modeling facilities:

a) a 2-D flume for studying bridge-scour at fairly large scale, and
b) a 3-D river basin for studying problems related to flood plain, for example, at a smaller scale than the 2-D models.

Some values from the case studies are used to design the components of the modeling facility. The Trinity River case study is used to design a 2-D modeling facility. This case study is selected because a large volume of flow is required to model this site. Similarly, data were taken from other case studies to design a 3-D river basin. For example, the flow volume from case study-4 with a 1:50 scale is used to design the sump, and the model discharge is taken from case study-1 to design the pump capacity for the 3-D modeling facility. In the design, the following components are discussed.

1. Dimensions of the 2-D flume and the 3-D river basin
2. Sump to store the water
3. Pump capacity
4. Flow control devices
5. Flow distribution
6. Surface Elevations
7. Flow Measuring Devices
8. Operation and maintenance
9. River banks
10. Soil used for sediments

7.2.1. Selection of Model Parameters

To design a 2-D modeling facility, case study-3 or case study-4 with 1:15 scale can be used. Out of these two case studies, case study-3 has the largest discharge and flow volume that can be used in the design of the pump capacity and sump capacity in the facility. The sump designed in this case study is useful for the 2-D facility because the model parameters are pertaining to a 2-D model in which the flood plain did not need to be modeled. So, in the design of 2-D facility, the parameters from case study-3, i.e., the hydraulic data near the bridge site at the State Highway 7 crossing over the Trinity River near Crockett, is used. The parameters are given below:

Width of the River	W_p	=	208.2 m (687 ft)
Width of the channel considered		=	102.4 m (338 ft)
Depth of the River	D_p	=	12.58 m (41.51 ft)
Area of Cross Section	A_p	=	1288 m^2 (14,021 ft^2)
500 year discharge	Q_p	=	1885 m^3/sec (66,600 ft^3/sec)
Width of the facility		=	6.06 m (20 ft)
Length of the facility		=	30.3 m (120 ft)
Mean velocity in the river	V_p	=	1.45 m/sec (4.75 ft/sec)

Assume a scale of 1:17 and an undistorted model.

Model width	W_m	=	6.024 m (19.88 ft)
Model Depth	D_m	=	0.74 m (2.44 ft)
Velocity in the model	V_m	=	0.35 m/sec (1.15 ft/sec)

Discharge in the model for 500 year flood, Q_m = 1.58 m^3/sec (55.89 ft^3/sec)

Cross section of the model at the bridge location, A_m= 4.51 m^2 (48.52 ft^2).

7.2.2. Preliminary Design of the Flume

As mentioned in chapter 5 the dimensions of the 2-D flume are taken as 36.6 m (120 ft) length, 6.1m (20 ft) wide, and 3.66 m (12 ft) deep which is shown in Figure 7.1. Considering a scale of 1:20, the parameters of the prototype that can be modeled in the basin are estimated as:

Width = 121 m (400 ft)

Length = 727 m (2400 ft)

In case studies 3 and 4, the widths and lengths of the prototypes are less than the above values which indicates that the basin dimensions are sufficient to model major rivers in Texas at a large scale.

7.2.3. Sump design (2-D)

The size of the sump is based on the maximum storage volume that should be handled in the model study. The storage volume of the flume must be stored in the sump when the model is not working. As mentioned in the previous section, the size of the flume is taken as 36.6 m (120 ft) length, 6.1 m (20 ft) wide, and 3.66 m (12 ft) deep. Assume the depth of water in the flume is 1.8 m (6 ft). Storage volume in the flume is equal to 36.6 x 6.1 x 1.83 = 408.57 m^3 (14,400 ft^3). The sump must be able to handle the storage volume of 408.57 m^3. Since the flow volumes in case studies 3 and 4 are smaller than the above storage volume, the sump can easily handle the flow volumes in the case studies.

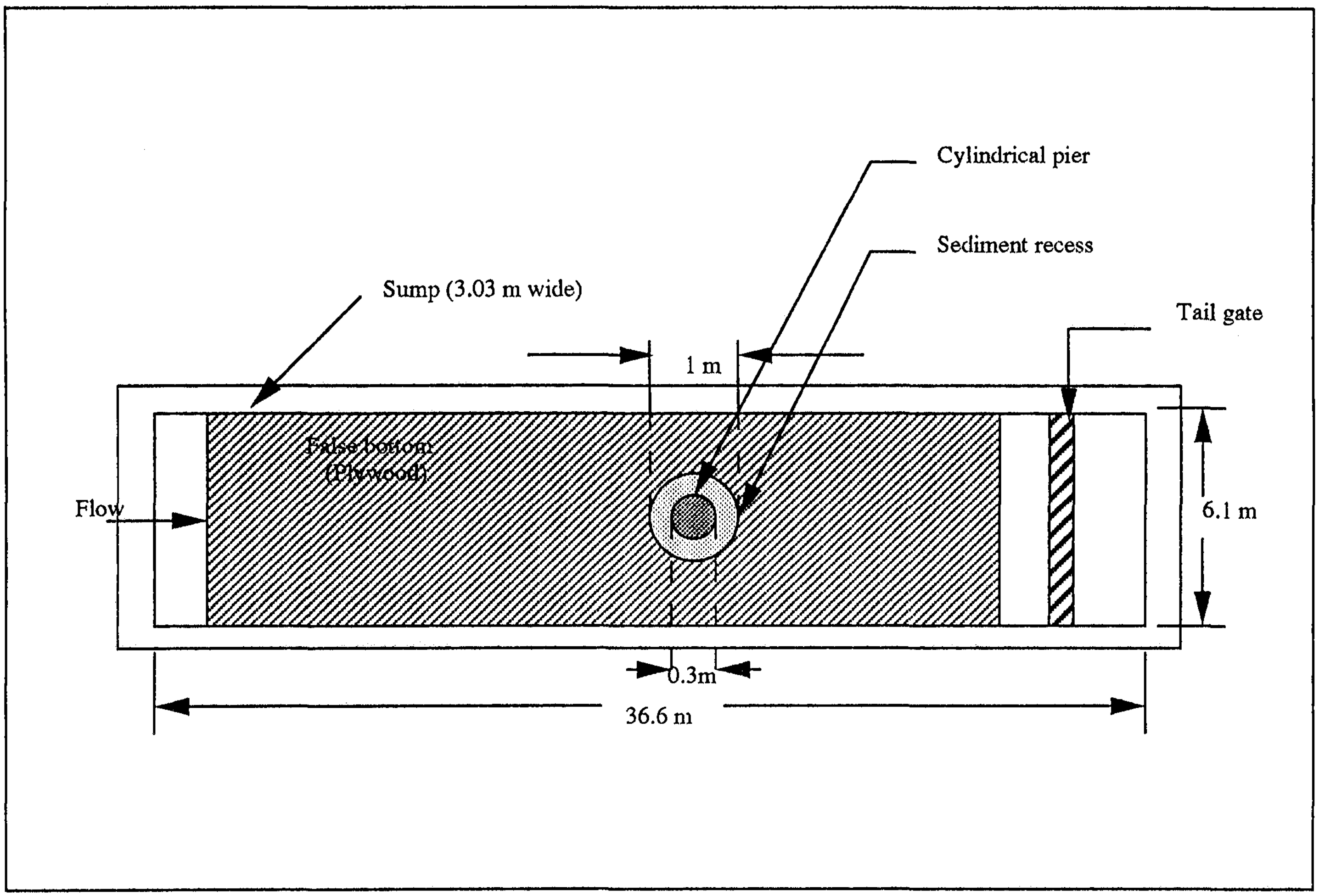

Figure 7.1. Plan of Open Channel Flume and Sump

Assuming the sump is constructed all around the facility, the cross section area of the sump is :

$$A_s = \frac{408.57}{2(36.6+6.1)} = 4.78\ m^2\ (51.43\ ft^2)$$

Assume a width of 3.03 m (10.0 ft).

Depth of the sump, D_s = 1.57 m (5.14 ft)

In order to store excess water in the sump when the model is running for a 500-yr flood, there must be some excess depth which can be taken as 0.61 m (2.0 ft). With a free board of 0.61 m (2.0 ft), the total depth of the sump becomes 2.79 m (9.14 ft). Therefore, the sump will run around the facility with a width of 3.03 m (10.0 ft) and a depth of 2.79 m (9.14 ft).

In the 2-D facility, the total cross-section of the river does not need to be modeled. If the scour around the pier is to be studied, then some portion around the pier can be taken and modeled. Similarly if the scour at the left abutment is to be studied, then the cross-section of the left overbank will be taken and modeled. This is possible if the velocity distribution is known. Let the pier be located at the center of the river. The cross-sectional area of the flow around the pier is taken as 4.51 m^2 (48.52 ft^2) (Ref: case study-3). Storage in the model for the 500 year flood simulation is calculated by multiplying the total length of the model that can be built with the cross-sectional area of the model river.

Storage of water in the model = 165 m^3 (5822 ft^3)

The sump size can be determined based on the above storage volume. Assuming that the sump is constructed all around the facility, the cross section area of the sump is:

$$A_s = \frac{165}{2(36.6+6.1)} = 1.93\ m^2\ \ (20.74\ ft^2)$$

Assume a width of 3.03 m (10 ft)

Depth of the sump, D_s = 0.63 m (2.08 ft)

In order to store excess water in the sump when the model is running for the 500-yr flood, there must be some excess depth which can be taken as 0.61 m (2.0 ft). With a free board of 0.61 m (2.0 ft) the total depth of the sump becomes 1.85 m (6.08 ft). The sump will run around the facility with a width of 3.03 m (10.0 ft) and a depth of 1.85 m (6.08 ft). The sump designed for this case study is smaller than the previous one. So, the sump which is designed by taking the flume storage will be sufficient for all the 2-D case studies. The drawing of the sump is shown in Figure 7.2.

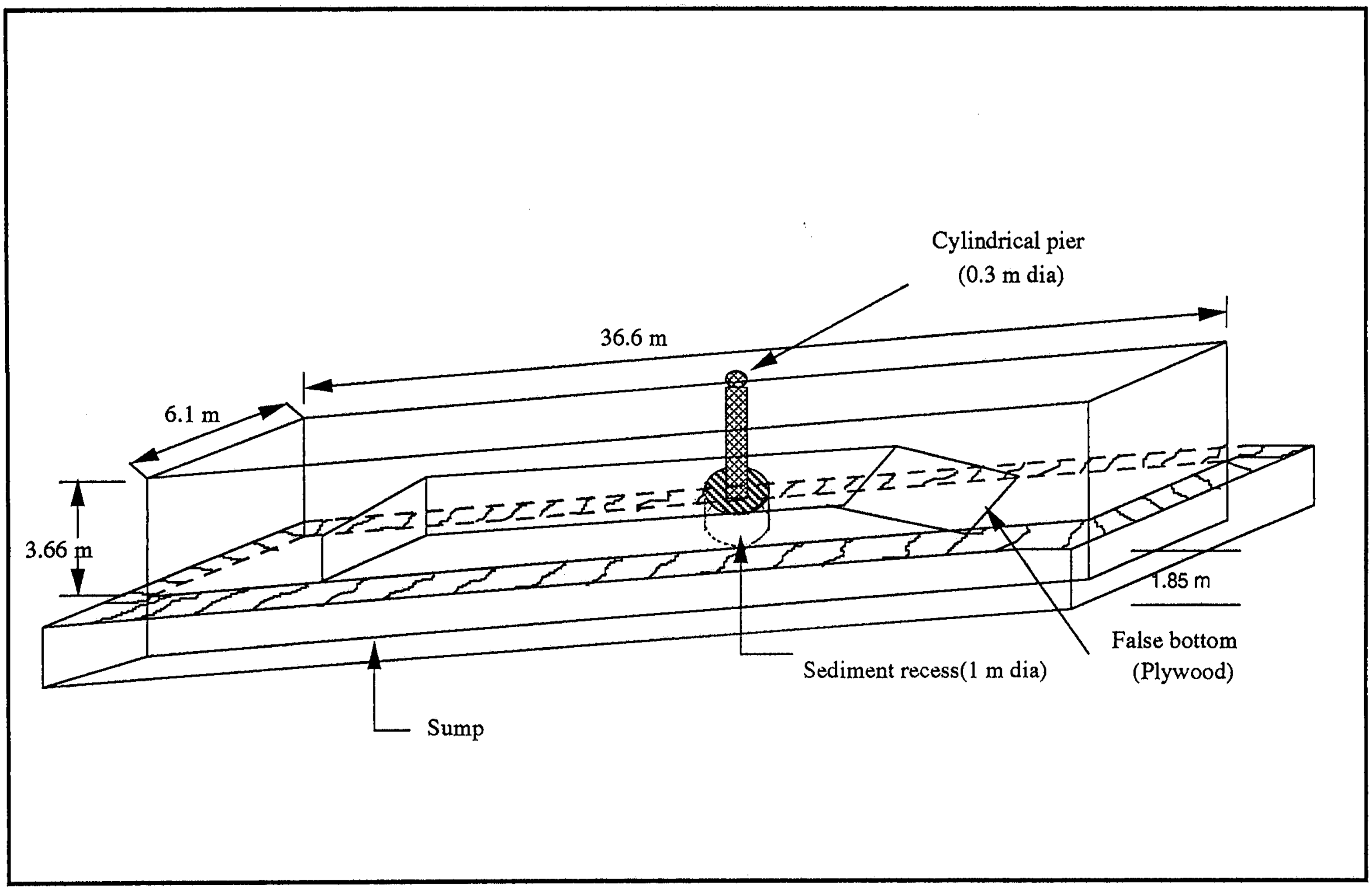

Figure 7.2. 2-D Open channel Flume and Sump

The sump could also be placed at one location, or more than one can be constructed based on the requirements at various locations; all these sumps may be interconnected. When the pumps are running with maximum flow capacity, the flow can be directed from other sumps, and when the pumps are running slow, water can be redirected back into the other sumps. The other alternative is to build a large sump and have it compartmented when its full capacity is not required.

7.2.4. Pump Capacity (2-D)

The design of the pump capacity is done in two steps. First, preliminary design is done by taking the maximum discharge from the range of hydraulic particulars in Texas which are given in the introduction of this chapter.

$$Q_p = 5094 \text{ m}^3\text{/sec } (180{,}000 \text{ ft}^3\text{/sec})$$

Assuming a scale of 1:20 for an undistorted model,

$$Q_m = \frac{Q_p}{l_r^{\frac{5}{2}}} = \frac{5094}{20^{\frac{5}{2}}} = 2.85 \text{ m}^3\text{/sec } (100.62 \text{ ft}^3\text{/sec}).$$

The above value will be the maximum discharge that must be handled by any pump when it is required to model the largest 500-yr flood in Texas. If this value is used to design the pump capacity, the pump can handle any discharge in the case studies which are smaller. The capacity of the pump is designed as follows:

Assume a soil thickness of 1.0 m (3.03 ft)above the floor.

Suction head		=	2.79 m (9.0 ft)(taken from sump design)
Delivery head		=	2.00 m (6.0 ft)
Total head	H	=	4.79 m (15 ft)
Discharge	Q	=	2.85 m^3/sec (100.62 ft^3/sec)

Since future expansion of the facility may require higher flow rate, the discharge is taken as 3.00 m^3/sec.

Assume the efficiency, η = 80 %

$$\text{Capacity of the pump} = \frac{WQH}{75\eta} = \frac{1000\text{x}3.0\text{x}4.79}{75\text{x}0.8}$$

$$= 239.5 \text{ H.P}$$

Threfore, the maximum capacity of the pump that would be needed for a general purpose modeling facility is 239.5 H.P.

In the second step, the capacity of the pump is determined by taking the maximum flow discharge from the case studies.

Assume a soil thickness of 1.0 m above the floor.

Suction head		=	2.79 m (9.0 ft)
Delivery head		=	2.00 m (6.6 ft)
Total head	H	=	4.79 m (15.0 ft)
Discharge	Q	=	1.95 m^3 /sec (68.84 ft^3/sec)

While designing, the discharge is taken as 2.5 m^3/sec (88.26 ft^3/sec) for future expansion of modeling capability.

Assume the efficiency, η = 80 %

$$\text{Capacity of the pump} = \frac{WQH}{75\eta} = \frac{1000 \times 2.5 \times 4.79}{75 \times 0.8} = 199.58 \text{ HP}$$

A pump of 199.58 HP may be provided or, depending on the availability of the pumps in the commercial market a suitable number of pumps may be chosen. The capacity of this pump is smaller when compared to the previous pump capacity. One pump with a large capacity may be more suitable for future expansion of the modeling facility. Pumps are kept at higher elevations and dry and can be installed at the end of the sump.

7.3. DESIGN OF THE 3-DIMENSIONAL FACILITY

In the 3-D facility, the total cross-section including the flood plain will be considered. The 3-D facility is preferable if the scour is significant in the entire cross section of the river as well as in the flood plain. In order to model the flood plain it is necessary to know the velocity distribution in the prototype. The flow distribution across the river and the flood plain can be found using FESWMS. The dimensions of the 3-D basin are shown in Figure 7.3.

7.3.1. Selection of Model Parameters

Flow volume and flow discharge are some of the important parameters required in the design of the sump and pump capacities of the facility.

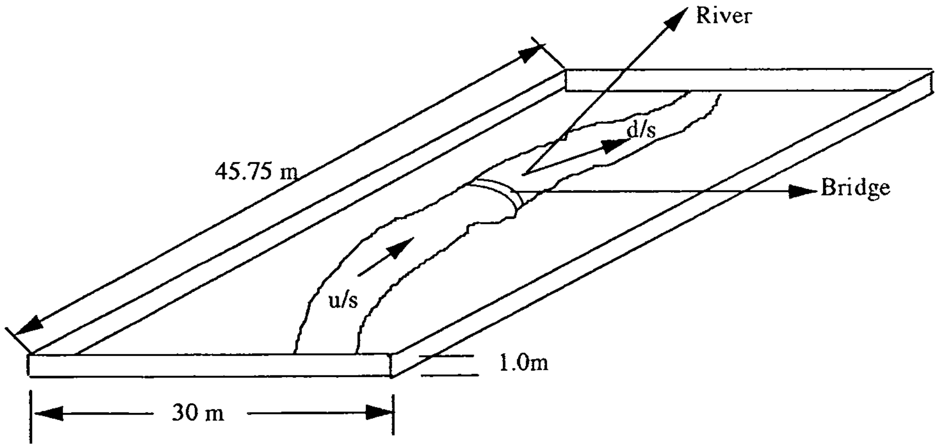

Figure 7.3. 3-D River Hydraulics Sediment Transport Basin

Out of all the case studies, case study-4 with 1:50 scale has the largest volume, and case study-1 has the largest flow discharge. These values should be used in the design of sump and pump capacity. The values are given in the following:

Flow volume at any instant of time for 500-year discharge = 52.86 m^3 (1868 ft^3)

Discharge = 0.33 m^3/sec (11.75 ft^3/sec).

7.3.2. Sump Design (3-D)

The size of the sump depends on the flow volume in the model corresponding to a 500-yr flood in the prototype, which should be modeled. Storage in the model (Case study-4) for a 500-year flood simulation is calculated by multiplying the total length of the model that can be built with the cross-sectional area of the model.

Storage of water in the model = 52.86 m^3 (1868 ft^3)

Assuming that the sump is constructed all along the facility, the cross section area of the sump, A_s will be obtained by dividing the flow volume by the length of the model.

$$A_s = \frac{52.86}{45.75} = 1.16 \text{ m}^2 \quad (12.45 \text{ ft}^2)$$

Assume a width of 3.03 m (10 ft)

Depth of the sump, D_s = 0.379 m (1.25 ft)

In order to store excess water in the sump when the model is running for a 500-yr flood, there must be some excess depth which can be taken as 0.61 m (2.0 ft). With a free board of 0.61 m (2.0 ft), the total depth of the sump becomes 1.59 m (5.25 ft).

The sump will be constructed at the center and parallel to the length of the facility with a width of 3.03 m (10.0 ft) and a depth of 1.59 m (5.25 ft). The drawing of the sump is shown in Figure 7.4. The sump could be placed at one location as a large one, or more than one can be constructed based on the requirements at various locations; all these may be interconnected. When the pumps are running fast, flow can be directed from other sumps, and when the pumps are running slow, water can be redirected back into the other sumps.

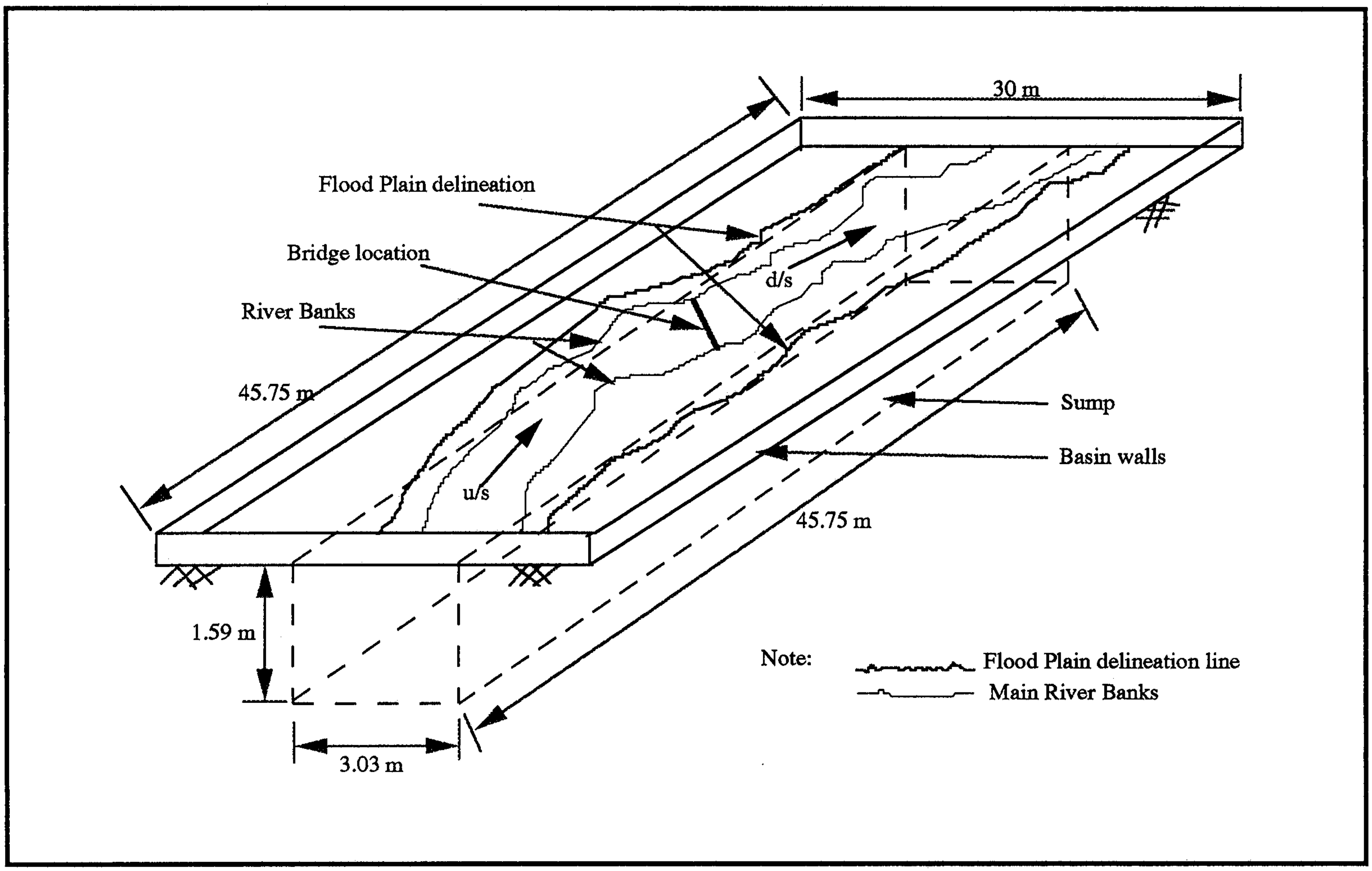

Figure 7.4. 3-D River Hydraulics Sediment Transport Basin with sump below the ground

7.3.3. Pump Capacity(3-D)

The design of the pump capacity is done in two steps. First, preliminary design is done by taking the maximum discharge from the range of hydraulic particulars in Texas which are given in the introduction of this chapter and by using the 1:50 scale.

$$Q_p = 5094 \text{ m}^3/\text{sec } (180{,}000 \text{ ft}^3/\text{sec})$$

Assuming a scale of 1:50 for an undistorted model,

$$Q_m = \frac{Q_p}{l_r^{\frac{5}{2}}} = \frac{5094}{15^{\frac{5}{2}}} = 0.29 \text{ m}^3/\text{sec } (10.18 \text{ ft}^3/\text{sec}).$$

The above value will be the maximum discharge that must be handled by any pump when it is required to model the largest 500-yr flood in Texas. If this value is taken to design the pump capacity, the pump can handle any discharge in the case studies which are relatively smaller. The capacity of the pump is designed as follows:

Assume a soil thickness of 1.0 m above the floor.

Suction head		=	1.59 m (5.2 ft)(taken from the sump design)
Delivery head		=	2.00 m (6.6 ft)
Total head	H	=	3.59 m (12.0 ft)
Discharge	Q	=	0.29 m^3/sec (10.18 ft^3/sec)

Since future expansion of the facility may require higher flow rate, the discharge is taken as 0.35 m^3/sec (12.35 ft^3/sec).

Assume the efficiency, η = 80 %

$$\text{Capacity of the pump} = \frac{WQH}{75\eta} = \frac{1000 \times 0.35 \times 3.59}{75 \times 0.8} = 20.95 \text{ H.P}$$

Therefore, the capacity of the pump that would be useful in all cases for this 3-D modeling capability is 20.95 H.P.

In the second step, the capacity of the pump is determined by taking the maximum flow discharge from the case study-1.

Assume a soil thickness of 1.0 m above the floor.

Suction head		=	1.59 m (5.2 ft)
Delivery head		=	2.00 m (6.6 ft)
Total head	H	=	3.59 m (12.0 ft)
Discharge	Q	=	0.33 m^3/sec (11.65 ft^3/sec)

While designing, the discharge is taken as 0.39 m^3/sec (13.75 ft^3/sec) for future expansion of modeling capability.

Assume the efficiency, η = 80 %

$$\text{Capacity of the pump} = \frac{WQH}{75\eta} = \frac{1000 \times 0.39 \times 3.59}{75 \times 0.8} = 23.34 \text{ H.P}$$

One pump of 23.34 HP may be provided or depending on the availability of the pumps in the commercial market, a suitable number of pumps may be chosen. The capacity of this pump is larger when compared to the previous pump capacity. One pump with a large capacity may be more suitable for future expansion of the modeling facility. Pumps are kept at higher elevations and dry and can be installed at the end of the sump.

7.4. FLOW CONTROL DEVICES

The control of flow in the model should be given proper attention. To maintain the required water-surface elevations, model discharge, and model flow velocity, it may be necessary to have different flow control devices. For example, a tail gate is required to maintain the water surface elevations; a venture meter or an orifice meter is required to measure the flow rate. Variable speed pumps are desirable to provide a required rate of flow in the model close to the design flow rate, while a bypass valve can be used to divert excess flow back to the sump. Knowing the surface elevations at different cross sections along the length of the river, it is possible to maintain the water surface profile in the model. Prototype water surface elevations can be obtained by running WSPRO or HEC-2. In all the case studies, it was found that the Froude number is less than unity, which indicates that the flow is subcritical. Because the flow is subcritical, a tilting flume is not essential for the model study.

7.5. FLOW DISTRIBUTION

Knowing the velocity distribution at any cross-section on the upstream side of the bridge location is very important. After the velocity distribution is predicted at a particular section by using WSPRO or FESWMS, manifolds can be used to reproduce the predicted velocity distribution in the model at that particular section. The reproduction of velocity distribution at the upstream section will be useful in reproducing the velocity distribution at the bridge location where the scour takes place. WSPRO or HEC-2 can be used to determine the distribution of the flow in the river and also on the flood plain. WSPRO will divide the entire cross-section into twenty small cross-sections parallel to the flow direction. Each of these twenty channels will carry 5% of the total discharge. Velocity in all these channels parallel to the flow is obtained using WSPRO. Mannings equation is used to compute the velocity. This leads to the assumption that the flow is uniform in the river. Sometimes it is difficult to measure the velocities and the depth because they are very low in the scaled model. The most common measuring device is the miniature current meter with a plastic rotor. The hot film anemometer has been discarded because of the fragility of the film and errors due to temperature fluctuations. A laser doppler anemometer can be used, but is more expensive.

FESWMS (Finite Element Surface Water Modeling System) can also be used to determine the two dimensional flow distribution on a horizontal plane in the river cross-section, as well as in the flood plain. The velocity distribution at the bridge section can also be modeled using FESWMS. The model will give the water surface elevation contours across the flood plain. After knowing the water surface elevation at any cross-section across the flood plain, the flood plain can be truncated to some limited extent.

7.6. SURFACE ELEVATIONS

Surface elevation along the river can be obtained by running WSPRO. The elevations that can be obtained are bed elevation, bank elevation, maximum water surface elevation, bridge low chord elevation, and road elevation. These surface elevations can be determined at required sections along the length of the river.

Hook or point gauges can be used in case of steady state studies to measure the water elevations. In case of unsteady state flows, it is advisable to equip the model with water elevation recorders connected to a computer. These are moving probes which follow the water for slow water movements or capacitance gauges for rapid water phenomena like flood waves. For measuring bed surface elevations, a surveying staff and bed profiler can be used.

7.7. FLOW MEASURING DEVICES

To determine the rate of flow, velocity of flow, and bed profile in the model, some of the measuring devices listed below can be used.

(a) Venture Meter: A venture meter is a device which is used for measuring the rate of fluid flow through a pipe. The basic principle on which a venture meter works is that by reducing the cross-sectional area of the flow passage, a pressure difference is created and the measurement of the pressure difference enables the determination of the discharge through the pipe. The discharge through the venture meter is given by:

$$Q = C_d \frac{a_1 a_2 \sqrt{2gh}}{\sqrt{a_1^{\,2} - a_2^{\,2}}} \qquad \text{Eq.7.1}$$

where C_d = coefficient of discharge,

a_1 = cross-sectional area of the pipe,

a_2 = cross-sectional area of the throat of the venture meter, and

h i= head difference between the two sections.

(b) Orifice Meter: An orifice meter is another simple device used for measuring the discharge through pipes. An orifice meter works on the same principle as the venture meter. An orifice meter consists of a flat circular plate with a circular hole called an orifice, which is concentric with the pipe axis. The thickness of the plate is less than or equal to 0.05 times the diameter of the pipe. The discharge through the orifice meter can be computed as given below:

$$Q = C_d \frac{a_1 a_2 \sqrt{2gh}}{\sqrt{a_1^{\,2} - a_2^{\,2}}} \qquad \text{Eq.7.2}$$

where C_d = coefficient of discharge through the orifice meter,

a_1 = cross-sectional area of the pipe,

a_2 = cross-sectional area of the orifice meter, and

h = head difference between the two sections.

Venture and orifice meters are installed in pipes to measure the flow discharge into the modeling basin.

(c) Pitot Tube: A pitot tube is a simple device used for measuring the velocity of flow. The basic principle used in this device is that if the velocity of flow at a particular point is reduced to zero, which is known as the stagnation point, the pressure there is increased due to conversion of the kinetic energy into pressure energy; and by measuring the increase in the pressure energy at this point, the velocity of flow may be determined. The velocity through the pitot tube can measured by the following equation:

$$\text{velocity, } v = c_v \sqrt{2gh} \qquad \text{Eq.7.3}$$

where c_v = coefficient of velocity, and

h = increase in the pressure energy at the stagnation point.

The accuracy of the pitot tube is very limited in complex flow situations.

(d) Laser-Doppler Anemometer (LDA): This instrument measures the Doppler shift in light scattered off tracer particles in moving fluids. The measured frequency shift is then used to compute the fluid velocity. Laser-Doppler anemometer is expensive, but it has numerous advantages over other less expensive velocity measuring devices such as the electro-magnetic current meter and acoustic Doppler anemometer. All three components of the velocity can be measured at the same point. The measuring volume is well below 1 mm^3, and a fiber-optic immersible probe can be used to bring the light beams close to the measuring point. The LDA has a frequency response typically above 1000 Hz so it is capable of measuring turbulent velocity fluctuations.

(e) Acoustic Doppler Velocimeter (ADV): This instrument uses Doppler shift of sound waves in water to measure the single point three-dimensional water velocity. The Doppler shift is derived from signals scattered from small particles in the flow. The occurrence of the suspended sediments in the water column may act as natural tracer particles, or seeding particles can be introduced. The ADV is rather insensitive to water quality and works well in turbid water. It is relatively expensive compared to the LDA and is generally adequate if the turbulent velocity fluctuations need not be measured.

(f) Electro-Magnetic Current Meter: This instrument senses changes in magnetic flux in the water created by flows around the sensor. The resulting voltage change in the electric circuit is proportional to the speed of the flowing water. This instrument can measure two velocity components simultaneously. Its measuring volume is much larger than the LDA and the ADV. In addition, the response time is of the order of 10 Hz thus, it cannot detect the turbulent velocity.

(g) Electronic Bed Profiler: This instrument consists of a probe placed vertically in the water. A servo-mechanism maintains the tip of the probe at a constant distance above the bed. When the instrument is being displaced in a horizontal direction, the probe will follow the configuration of the bed continuously. The principle of operation is the appreciable difference between the electronic conductivities of water and the bed material. The probe outputs an analog voltage whose magnitude is directly proportional to the vertical movement of the probe. The probe can be used without stopping the flow. Therefore, time development of the bed configuration can be obtained.

7.8. LIVE-BED SCOUR

To study live-bed scour, sediments can be introduced at the upstream end and collected at the downstream end. At the upstream end, specially designed feeders may be used to ensure a controlled sediment rate of inflow. At the downstream end, sediment can be collected in a sediment trap which can be reused. The recording of bed evolution can be made at the end of each run by draining the model carefully in depth increments equal to the required contour intervals. At each step, a thin, clearly visible cord is laid along the water edge. This represents a contour line and its position may be recorded by overhead photography. Bed elevation can also be measured continuously by a bed profiler that is moved across a beam spanning the model and basin. There are two types of bed profilers. One operates by measuring the conductance between two electrodes, and the other is an ultrasonic device.

7.9. RIVER BANKS

River banks should be non-erodable and stable. They should not contribute to the sediment in the model. The most common method for accurately reproducing the river cross sections is the use of the templates with cement mortar. Fine sheets of metal following the contour lines are more suitable to model flood plain or hilly sections of the river. For qualitative study of river migration and river bends, cohesionless soils could be used. Radius of curvature in the field is the product of radius of curvature in the model and the scale.

7.10. SOILS

The soil grains need to be scaled down to ensure that they will be subjected to the same erosion potential after the velocity of the water has been scaled down. The laws governing the reduction factor for the soil grains are described in Sections 4.2 and 4.3. There are two ways to scale the soil grains appropriately. The first one consists of decreasing the size of the grains. The limit for this process is 0.1 mm. Once the soil grains in the model become smaller than 0.1 mm, the soil in the model ceases to be cohesionless like the one in the prototype because the electromagnetic forces between grains start to become sizable compared to the gravity forces. If the required size of the grains for proper modeling are smaller than that, it is possible to use a different and lighter material for the grains in the model. Coal, crushed shells, and plastics are all used and provide an additional reduction factor approximately equal to 5.5. Such light particles with sizes in the 0.1 mm range are, therefore, equivalent to 0.02 mm. If this is still not small enough, the soil cannot be properly scaled. Then, the practice is simply to use such small and light particles to ensure that the soil in the model can be eroded at the model velocity. Generally, fine sands, silts, and clays cannot be properly modeled, and much work remains to be done for these soils.

7.11. COST ESTIMATE FOR PROPOSED MODELING FACILITY

The modeling facility is proposed to have both a 3-D basin and a 2-D open channel flume in the same building as shown in Figure 7.5. It will also have an overhead crane and pumps for both the 3-D basin and 2-D flume. Some space in the building is provided for different purposes such as office, loading zone, storage area, and workshop. The cost of the overall facility, including the components, is given below.

ITEM DESCRIPTION		ESTIMATED COST
Building		
1. Building 51.85×76.25 m (170 ft × 250 ft)		
@ \$1075/m^2(\$100/ft^2) (Figure 7.5)	\$	4,250,000
Office & Workshop		
2. Office 4.58 ×30.5 m (15 ft × 100 ft)		
@ \$376/m^2(\$35/ft^2)	\$	52,500
3. Workshop 9.15 × 22.88 m (30 ft × 75 ft)		
@ \$806/m^2(\$75/ft^2)	\$	168,750
Subtotal		221,250
Fixed Equipment		
4. River Basin	\$	125,000
Underground Sump (3-D)	\$	50,000
Pump (12 cfs, 25 H.P)	\$	15,000
5. Open Channel Flume (2-D)	\$	85,000
Underground Sump	\$	160,000
Pumps 3 No.s (Each of 35 cfs, 100 H.P)	\$	100,000
6. Overhead Crane	\$	100,000
Subtotal	\$	635,000

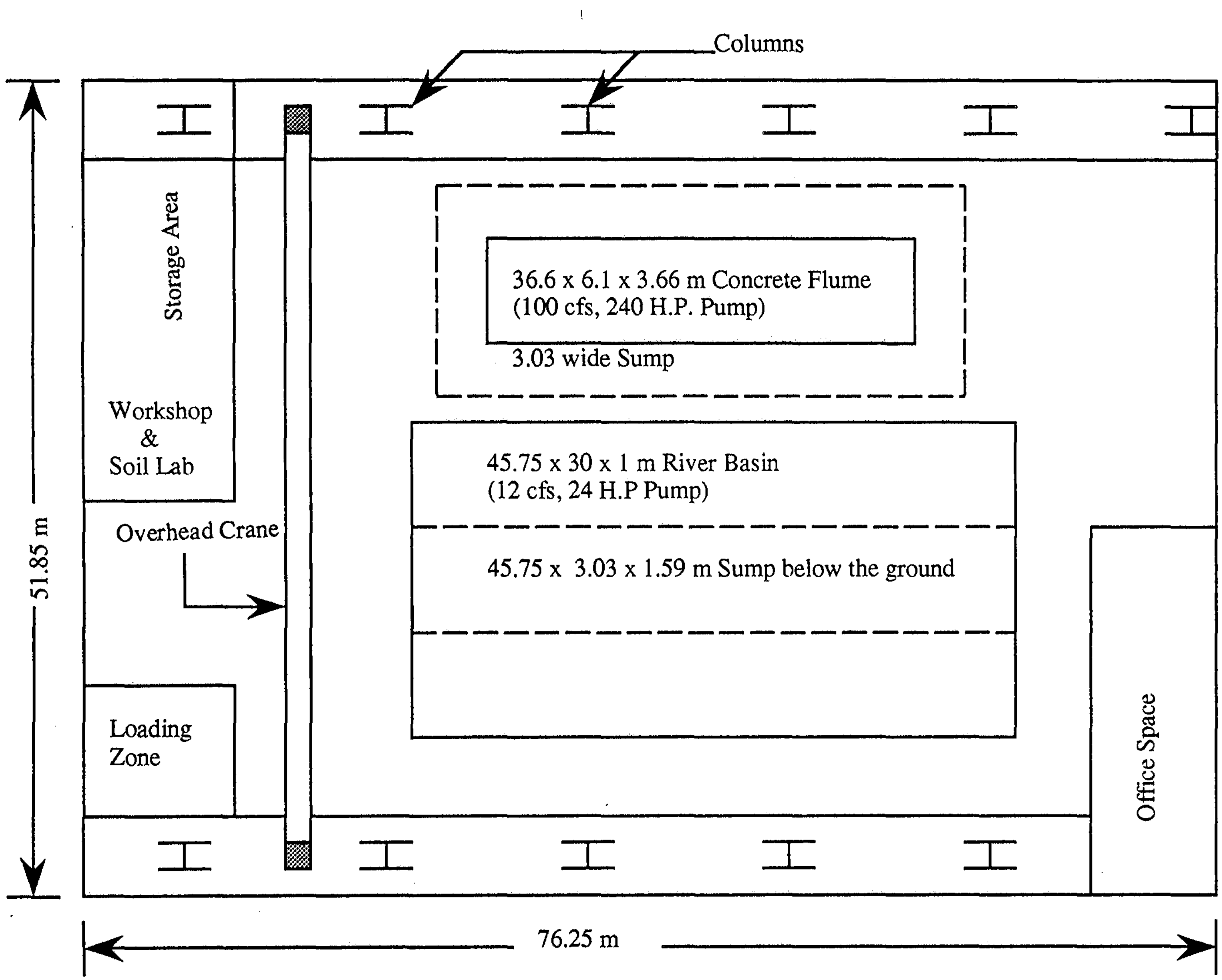

Figure 7.5. Plan of Hydraulic Modeling Facility

Various Equipment & Instrumentation

7. Data acquisition (4 PC/486, acquisition board and accessories, 2 Laser jet printers)	$	20,000
8. One three-component, fiber optic Laser-Doppler anemometer	$	200,000
9. Four two-component, electro-magnetic current meters	$	50,000
10. One bed profiler	$	25,000
11. Flowmeters	$	15,000
12. Suspended sediment concentration measuring device	$	200,000
13. Water surface elevation measuring devices (resistance/capacitance gages)	$	50,000
14. Instrument carriages	$	20,000
15. Video camera and recorder	$	10,000
16. Computer software	$	20,000
Subtotal	$	610,000
Miscellaneous		
17. A&E Fee (10% of building cost)	$	425,000
18. Utilities (Water, electricity, gas)	$	25,000
19. Contingency	$	500,000
Subtotal	$	950,000
Total Project Cost	$	**6,666,250**

8. CONCLUSIONS

These conclusions are based on a study of the Texas scour problem, including the hydraulic and soil characteristics of the rivers in Texas; on the study of the known fundamental principles of hydraulic and soil modeling; on the model analysis by similitude theory of five bridge case studies in Texas; on the discussions with recognized scour experts; and on a survey and visit of four leading scour facilities in the country.

The following conclusions are reached :

1. The facility should have two basins: a 2-D flume for local scour studies and a 3-D basin for global scour studies.

2. The 2-D flume should be above ground, 36 m long (120 ft), 6 m wide (20 ft), and 3 m deep (10 ft). The sump should be below ground; it should surround the flume and be 3 m (10 ft) wide, and 3 m (10 ft) deep. A 240 HP pump delivering 28 m^3/sec (100 cfs) is necessary to feed this flume.

3. The 3-D basin should be above ground, 45 m (150 ft) long, 30 m (100 ft) wide and 1 m (3 ft) deep. The sump should be below ground under the center of the basin, parallel to the 50 m side of the basin; it should be 3 m wide (10 ft) and 1.8m (6 ft) deep. A 25 HP pump delivering 0.4 m^3/sec (12 cfs) is necessary to feed this basin.

4. The 2-D flume would allow local scour models with undistorted scales in the range of 1/15 to 1/25.

5. The 3-D basin would allow general scour models with undistorted scales in the range of 1/50 to 1/100.

6. The cost of the facility and its major components is estimated to be a follows:

The overall facility	= \$6.70 M
The building	= \$4.25 M
The 3-D basin with sump and pump	= \$0.19 M
The 2-D basin with sump and pump	= \$0.35 M
Measuring instruments	= \$0.61 M

7. The advantages and disadvantages of this facility are:

Advantages	Disadvantages
1. Availability	1. Initial cost
2. Develop local expertise	2. Delay until built
3. Latest technology	3. Inexperience personnel at first
4. Very large scale	
5. Low overhead	
6. Easy contracts	
7. Short travel time	

8. Existing facilities do not compare favorably with the facility described above. However, a few of them can provide very valuable data on scour modeling at reasonably large scale.
9. The advantages and the disadvantages of the existing facilities are:

Advantages	Disadvantages
1. No delay for use	1. Higher overhead
2. No initial cost	2. No local expertise developed
3. Experienced personnel	3. Older equipment
	4. Longer travel time
	5. First come first serve availability

10. The decision should be based on the estimated need in the next 10 to 20 years for such a facility by TxDOT and neighboring states. Using an existing building is a way to decrease the cost that would make a big difference. It should also be kept in mind that Texas rivers have a mixture of sand and clay beds, and that the usefulness of modeling facilities for scour in clay is limited.

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