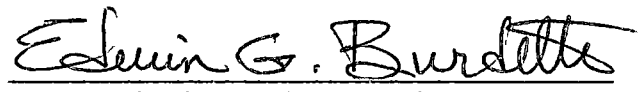
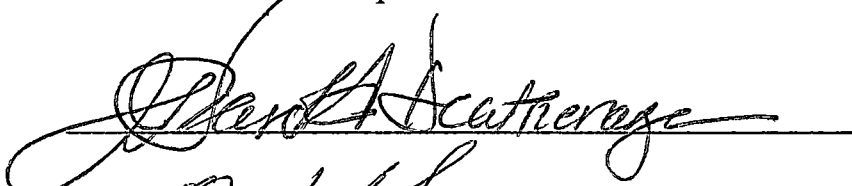
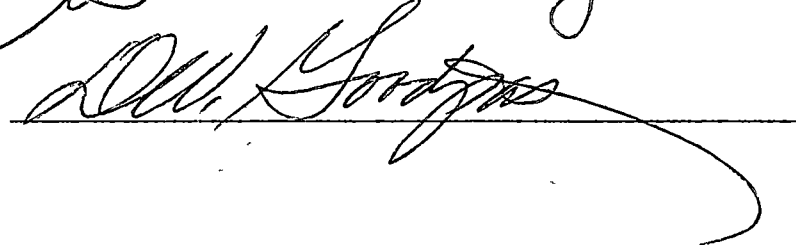


To the Graduate Council:

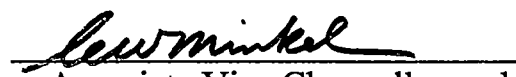
I am submitting herewith a thesis written by Ali Dawood Wehbi entitled "Bridge Design of S.R. 136 over Caney Fork River". I have examined the final copy of this thesis for form and content and recommend that it be accepted in partial fulfillment of the requirements for the degree of Master of Science, with a major in Civil Engineering.


Dr. Edwin G. Burdette, Major Professor

We have read this thesis and
Recommend its acceptance:

Accepted for Council


Associate Vice Chancellor and
Dean of the Graduate School

BRIDGE DESIGN OF
SR 136 OVER CANEY FORK RIVER

A Thesis

Presented for the

Master of Science Degree

The University of Tennessee, Knoxville

Ali Dawood Wehbi

May 2000

ABSTRACT

In October of 1995, the Tennessee Department of Transportation approved the replacement of the bridge on S.R. 136 over Caney Fork River. The original bridge consists of 8 concrete deck girder spans at 41'-0", 43'-0", 43'-0", 45'-0", 220'-6", 220'-4", 39'-2", and 35'-3" for a total of 687'-3". Spans 5 and 6 consist of riveted steel trusses and the piers are made of reinforced concrete. The replacement bridge is continuous and stretches over 6 equal spans for a total length of 721'-6". The superstructure consists of an 8.25" concrete slab and 72" prestressed bulb-t beams. All piers are reinforced concrete.

This thesis has two primary objectives: (1) to explain why the existing bridge has been replaced, and (2) to explain the reasons for the choice of abutment types. This thesis explains several issues involving several state departments that reflected the decision to replace the existing bridge. The choices of an integral abutment at Abutment No. 2 and expansion device at Abutment No. 1 are explained, and the differences in design philosophy between these two types of abutment are discussed. This study concludes that replacement of the bridge is necessary and that the choice of abutment types is justified.

DEDICATION

This project is dedicated to my mother, who has been my main support during my academic years and throughout my life. I thank her for all the love and support she gave me, and I hope to return some of it one day.

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Chapter One

Introduction

In 1982, Congress acknowledged the seriousness of our nation's bridge need by continuing the Highway Bridge Replacement and Rehabilitation Program (HBRRP). The Surface Transportation Assistance Act of 1982 increased funding for HBRRP through fiscal year 1986. The 1987 Surface Transportation and Uniform Relocation Assistance Act provides authorizations for the HBRRP through 1999.

The State Route 136 bridge over Caney Fork River is considered a top priority for replacement. The bridge is located off U.S. Highway 70 South in Warren and White Counties over Caney Fork River which feeds into the Great Falls Reservoir Dam downstream from the site (figures 1 and 2).

This thesis discusses two major issues. The first issue involves the decision to replace the existing bridge, and the reasons behind this decision are discussed. To obtain necessary information, many issues involved several state agencies. The state agencies involved in this research are as follows:

- 1) Bridge Evaluation and Inspection (Tennessee Department of Transportation)

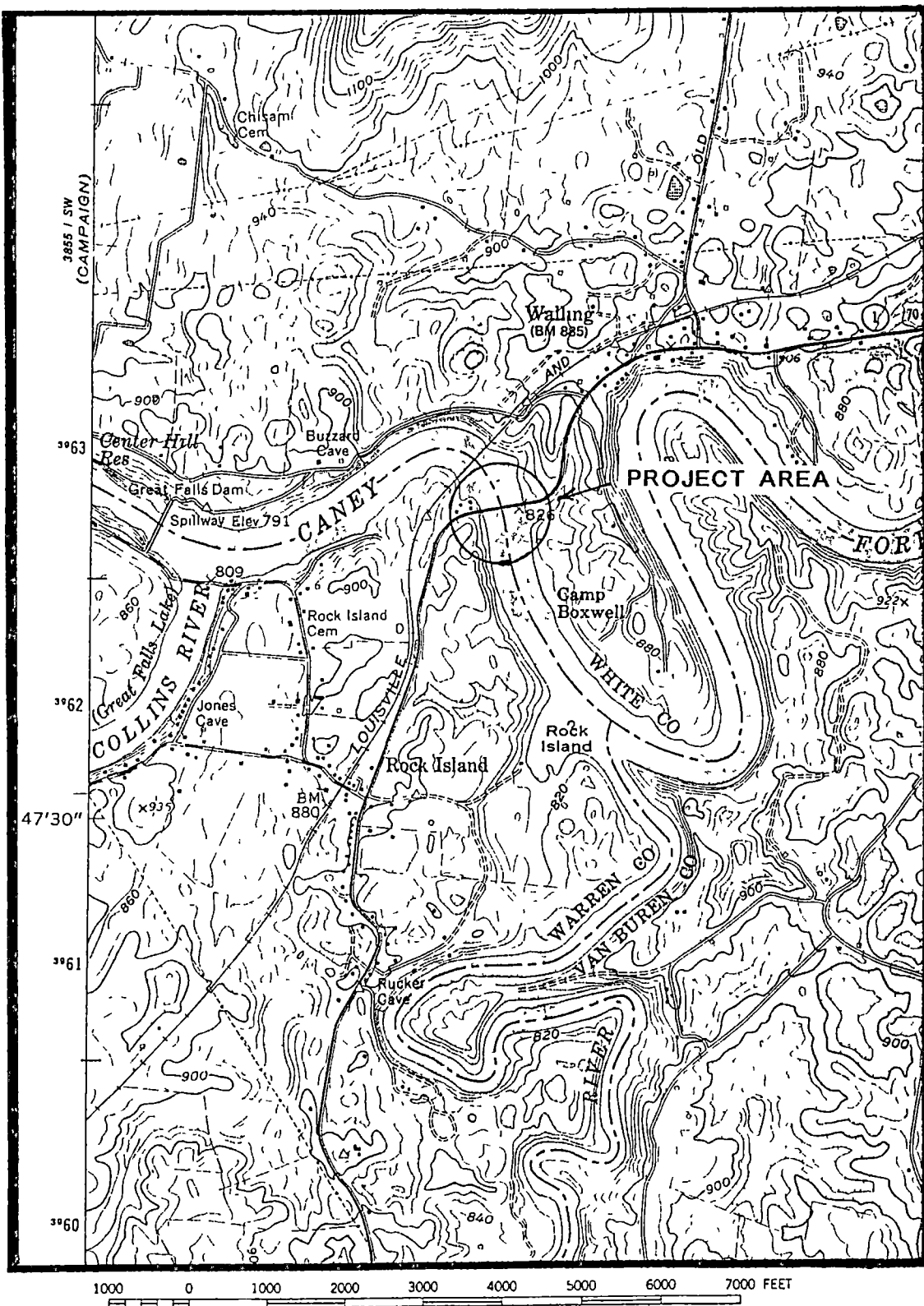


Figure 1: Project location map

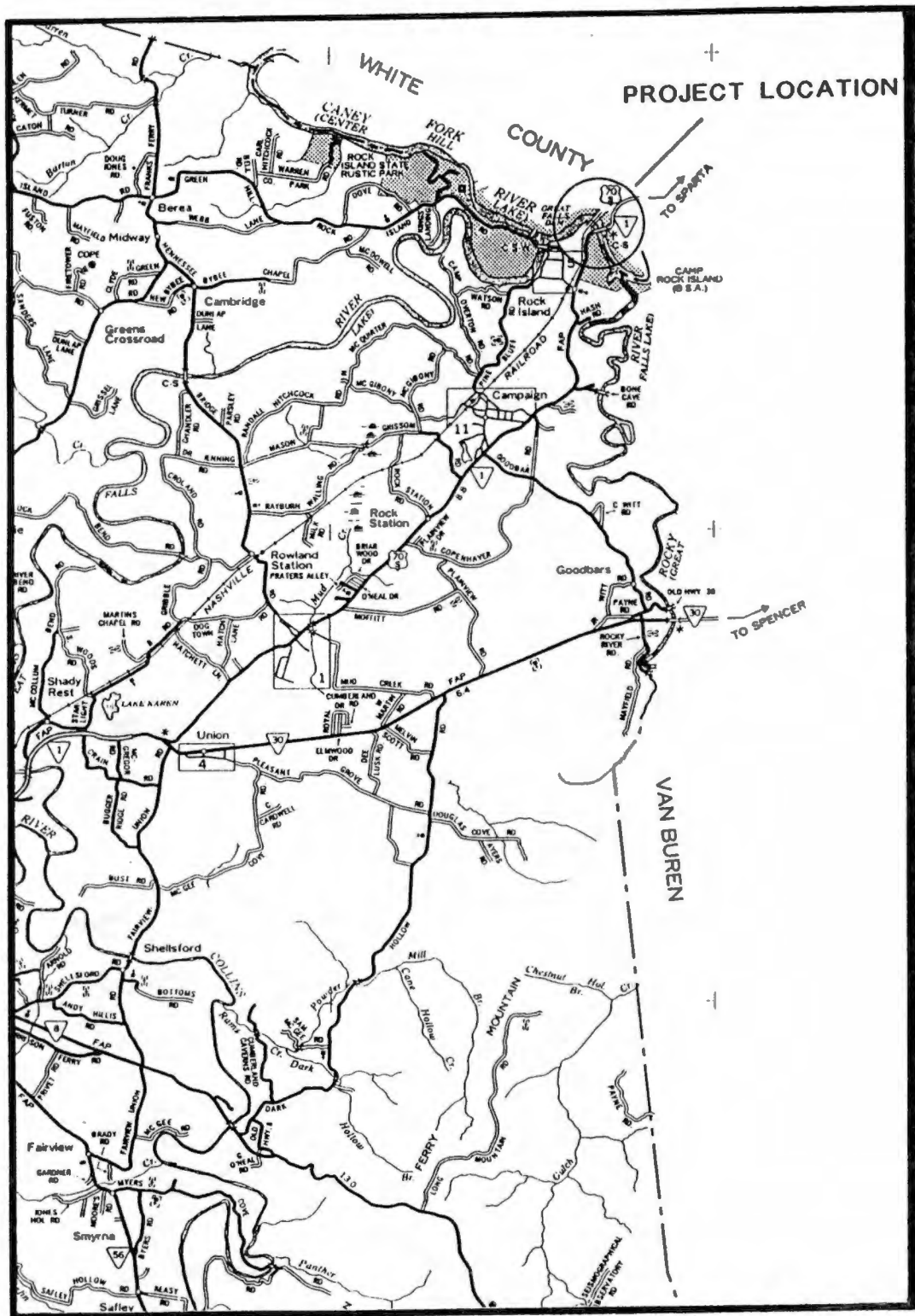


Figure 2: County map showing the project location

- 2) Hydraulics Design Division (Tennessee Department of Transportation)
- 3) Soils and Geology Division (Tennessee Department of Transportation)
- 4) Environmental Studies (Tennessee Department of Transportation)
 - a) Ecological Studies
 - b) Archaeological Studies

The information provided by these state agencies helped in the decision to replace the existing bridge and helped this writer understand some of the criteria behind it.

Second, the choice of the type of abutment at each end of the bridge is explained. There are several types of abutments the designer can choose based on several factors including the length of the bridge, location, type of soil, and depth of rock layer. In this study an integral abutment was chosen at Abutment No. 2, because a rock layer is $30\pm$ feet below the abutment's footing. The entire footing is standing on HP 10 x 42 steel piles, driven into soft silty clay soil to rock. The method used to determine number of piles needed is based on point bearing. An expansion device was chosen for Abutment No.1, which is sitting on double short staggered piles driven to refusal (rock layer), but only 11'- 0" long.

The objective of this paper is to explain why the existing bridge has been replaced and to make sure that the replacement bridge's thermal movement at the abutments is well contained within the expected range. Calculations in the next few chapters will show a movement less than 1" at each end. A primary purpose of this thesis is to demonstrate the advantages of integral abutment over expansion abutment design.

A special computer program (COM624P) was used to analyze the effect of thermal movement on HP10x42 steel piles at the abutments. This computer program analyzes the effect of thermal-induced lateral movement of the pile on the carrying capacity of the pile. The analytical model is based on the finite element approach, nonlinear soil behavior, and soil-pile interaction. The results of this model indicate the maximum safe length of piles to be buried into soil and that the total thermal movement of the bridge is not excessive.

In the next few chapters, the reasons behind the bridge replacement and the abutment types chosen are explained in detail.

Chapter Two

Description of Existing Bridge

2.1 Location

The Rock Island Bridge (S.R. 136 over Caney Fork River) is part of the Great Falls Reservoir located 5 miles northeast of McMinnville, a small community in Warren County (figures 1 and 2). The Caney Fork River, in the area of the bridge, is open and approximately 550 feet wide with an average daily flow (Great Falls Dam) of 3016 cubic feet/second (figures 3 and 4).

The area consists of different geological layers that are steep, well-drained, stony and loamy soils from colluvium, sandstone, shale, and limestone. The underlying bedrock is primarily limestone, chert, siltstone, sandstone, and dolomite.

The area southeast of the project location is hilly in the area located next to the river; however, the grade increases to the east. The surrounding land is primarily forest. Mixed hardwoods dominate the site with oak, hickory, and maple being the most common species (figures 5, 6, 7 and 8).

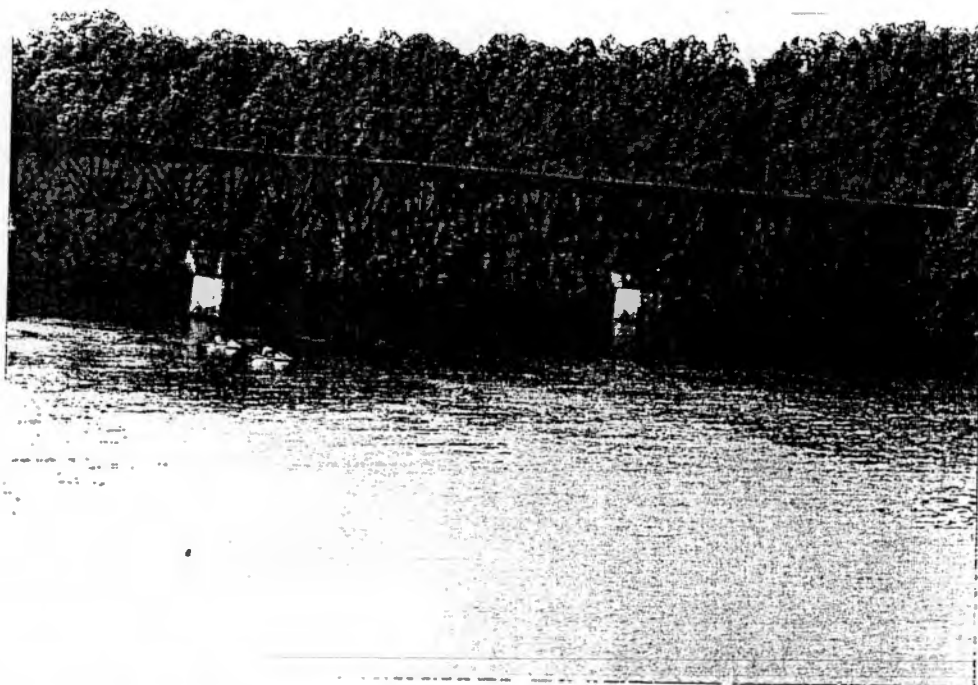


Figure 3: Photograph of the area downstream from the bridge

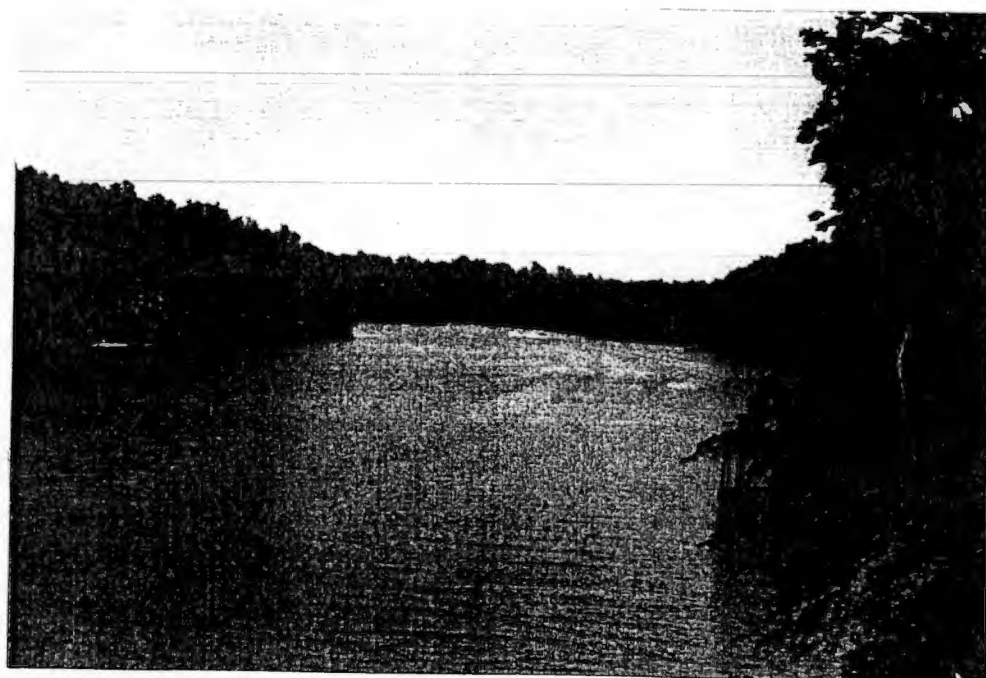


Figure 4: Photograph of the area upstream from the bridge



Figure 5: View east along existing road



Figure 6: View toward river in southeast quadrant



Figure 7: View showing spans 5 and 6, steel trusses



Figure 8: View east of the project area

2.2 Bridge Type

The existing bridge was built in 1925 with two lanes for traffic and total width from curb to curb of 20' - 0". There was no sidewalk or shoulder lane. The structure consisted of 8 concrete deck girder spans at 41'-0", 43'-0", 45'-0", 220'-6", 220'-4", 39'-2", & 35'-3", with a total length of 687'-3". Spans 5 and 6 consist of riveted steel trusses. All piers were lightly reinforced concrete piers. The riveted steel trusses were painted with inorganic zinc paint and vinyl topping; therefore, dropping pieces of the trusses during the removal of the bridge was not considered hazardous to the environment.

2.3 Geometry

The existing bridge crossed Caney Fork River at a 90° skew angle, the preferred angle for bridges that cross over waterways and channels. The approaches before and after the bridge were on a curve. The posted speed limit over the bridge was 40 mph. Although design speed for both approaches is 40 mph, there have been many accidents due to poor design of horizontal sight distance and the sharp curves. Figure 9 shows an aerial view of the existing bridge and approaches.

The cross section of the existing bridge consisted of two 10' lanes with no sidewalk or shoulders. This width, which is below standard, caused



Figure 9: An aerial view of existing bridge

traffic hazards and delays, especially when a truck was trying to cross over. The actual truck loading was calculated to be H-17^[1] while the posted truck loading on the bridge was H-15^[2]. Figure 10 shows some of the Truck loadings in detail.

2.4 Bridge Evaluation Report

A field bridge inspection is done every two years on each bridge in Tennessee, and an evaluation report is created or updated for further use within the Tennessee Department of Transportation agency. Replacement or rehabilitation of each bridge depends on this report, because bridge engineers evaluate and base their decision upon the outcome of this field inspection.

The results of field inspection for each bridge is divided into sections as follows:

- Concrete Decking and Pavement
- Rails
- Steel Trusses
- Beams
- Abutments
- Piers
- Footings, Hydraulics & Scour

^[1] H-17 Denotes: 17 Ton per axle

^[2] H-15 Denotes: 15 Ton per axle (Figure 3.7.6A AASHTO)

AXLE DEFINITIONS

All trucks operating on Tennessee's highway system will be weighed on each axle, as well as for the gross weight. No exception will be made for any type of load or truck. If additional axles are added they must meet the following conditions

- A Axles must be equipped with brakes
- B Axles must extend across the full width of the vehicle
- C Axles must be load bearing providing equal distribution of weight among all axles
DUMMY AXLES will NOT be recognized
- D At the discretion of the owner, axles may be placed in front of the permanent axle(s) or in the rear. The axle must be 40 inches from the other axles as measured from center-to-center and it must conform to the bridge formula. (See Bridge Table.)

SCHEDULE OF ALLOWABLE WEIGHTS

NUMBER OF AXLES	GROSS WEIGHT PERMITTED
2 (Straight Truck)	38,000 lbs
3 (Straight Truck)	52,000 lbs
4 (Straight Truck)	72,000 lbs
3 (Tractor-Trailer)	52,000 lbs
4 (Tractor-Trailer)	66,000 lbs
5 (Tractor-Trailer)	80,000 lbs

(See Bridge Table, p 6-7)

SIZE AND WEIGHT LIMITATIONS

Gross Weight	80,000 lbs
Tractor Steering Axle	12,000 lbs
Straight Truck Steering Axle	18,000 lbs
Single Axle	20,000 lbs
Tandem Axle	34,000 lbs
Width	96 in
Height	13 ft. 6 in.
Length	
a. Straight Truck	40 ft.
b. Combination	60 ft.
1. Combination transporting livestock	65 ft.
2. Combination transporting automobiles	65 ft.
3. Combination transporting poles or logs	75 ft.

2

WEIGHT LIMITATIONS BASED ON VEHICLE AND AXLE CONFIGURATIONS

18M = 18,000 Pounds
32M = 32,000 Pounds

No 1



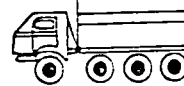
Axle 18M 20M
Max Gross 38,000

No 2



Axle 18M 34M
Max Gross 52,000

No 3



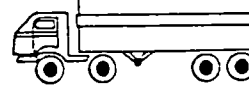
18M 20M 34M
Max. Gross 72,000

No 4



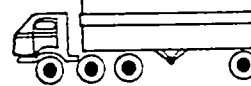
12M 20M 20M
Max Gross 52,000

No 5



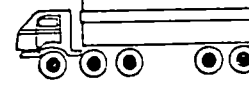
12M 20M 34M
Max. Gross 66,000

No 6



12M 34M 20M
Max Gross 66,000

No 7



12M 34M 34M
Max. Gross 80,000

Legal Loads enacted
in 1981

Figure 10: Truck weight and wheel configuration for different trucks

2.4.1 Concrete Decking Problems

The following deficiencies were found during the inspection of this bridge:

- 1) At Pier No. 1, the beams were anchored with steel rods . As the span moved, the steel rods were bent causing spalls up to 7" deep at the riser blocks.
- 2) At Pier No. 4, the ends of the beams had large diagonal cracks due to the span movement.
- 3) At Pier No. 3, the beams were breaking away from the diaphragms and would have eventually lost their bearing if the span movement had not stopped.

2.4.2 Steel Truss Problems

The following problems were found throughout the truss:

- 1) Some members had heavy corrosion.
- 2) Exterior stringers were in poor condition with some loss of section to the flanges and large holes through the webs.

- 3) Interior stringers had considerable corrosion in their top flanges, and many connections in the truss had considerable section loss.
- 4) Continued loss of section in the steel due to corrosion would have resulted in a weight posting of the bridge.

2.4.3 Roadway Traffic Problems

- 1) The bridge was too narrow for the amount of traffic it carried (ADT = 2016). It consisted of one 10'-0" lane in each direction.
- 2) The bridge rail was substandard. It was not crash tested, and reinforcement was not adequate.
- 3) Due to sharp curves at both ends of the bridge, the approach alignment was very poor (Figure 9).

2.4.4 Hydraulics and Scour Analysis⁽¹⁾

This bridge spans Caney Fork River, which is part of the main stream feeding into the Great Falls Reservoir Dam Area. The normal pool water elevation is 804.99', the 100-year water elevation is 805.58', and the 500-year water elevation is 805.94. The minimum girder elevation requested is close 810.37'.

The stream velocity is 4.0 ft/sec., which is not considered hazardous during construction for both existing and proposed bridge. The total weight of the bridge, minus water removed and minus water pressure pushing on one side of the bridge, is still a positive value, indicating that the piers during construction will not fall over.

Calculations are shown below. See Figure 11.

Checking Piers & Cofferdams from Overturning during Construction

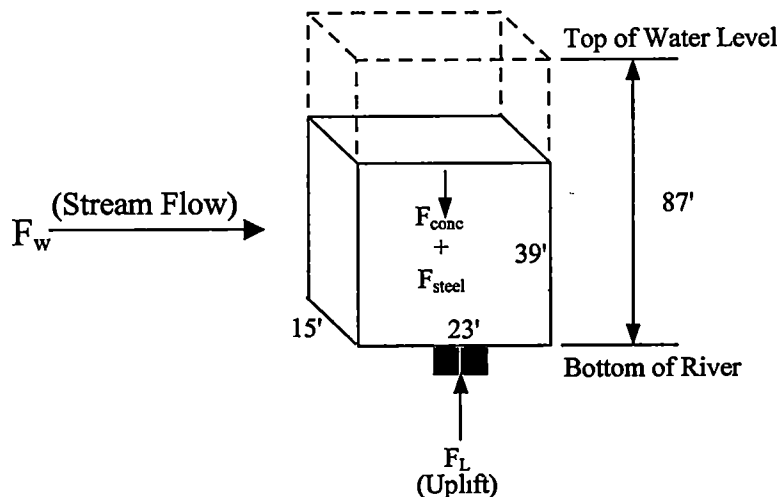


Figure 11: Elevation view of cofferdam

$$P_w = k(V_{avg})^2 \quad (\text{AASHTO 3.18.1})$$

P_w : Force exerted on the cofferdam by the stream

$$\left(\begin{array}{c} \text{Average} \\ \text{Stream} \\ \text{Velocity} \end{array} \right) = V_{avg} = 4 \text{ ft/sec}^2$$

$$\text{Constant } k = 1.0$$

$$P_w = 1(4)2 = 16 \text{ lb/FT}^2$$

$$F_w(\text{Stream Force}) = 16(87)15/1000 = 20.90 \text{ k-ft}$$

$$M_w = -20.90(43.5) = -909.0 \text{ k-ft}$$

wt. of water

$$F_L(\text{Uplift Force}) = 62.4 \text{ lb/ft}^3 (15.0')23.0'(87.0')/1000 = 1,873.0 \text{ k}$$

$$M_L = 1873.0(23.0'/2) = -21,540 \text{ k-ft}$$

$$F_{\text{conc}} = 39.0'(23.0')15.0'(145 \text{ lb/ft}^3) / 1000 = 1951.0 \text{ k}$$

$$M_{\text{conc}} = 1951.0(11.5) = 22,437.0 \text{ k-ft}$$

$$F_{\text{Steel}} = (\text{sheet piles are } 0.5'' \text{ thick}) = \frac{0.5''(23+23+15+15)}{12} 87(.49)$$

$$= 135 \text{ k}$$

Steel Wt.
k/ft³

$$M_{\text{Steel}} = 135(11.5') = 1552.0 \text{ k-ft}$$

$$\therefore \text{Overturning Moment (M)} = 22437 + 1552 - 21540 - 909 = 1540 \text{ k-ft}$$

$$\text{Factor of Safety} = \frac{22437 + 1552}{21540 + 909} = 1.07 > 1.0 \quad \underline{\text{O.K.}}$$

There was no scour at the piers. The footing at the existing Pier No. 6 was partially exposed, but below the bottom of the footing the material was reported as stone. Stone does not wash away from scour or high water velocity. All the existing piers had been replaced prior to construction of the proposed bridge.

2.4.5 Recommendations

Due to the functional obsolescence and deteriorating condition of the bridge, the final recommendations should be implemented as follows:

1) The bridge should be replaced.

OR

2) If a decision has been made to repair the bridge, the department should do the following:

- ◆ Repair spalls to the riser block and ends of the beams at Pier No. 4.
- ◆ Consideration should be given to repair/replacement of exterior stringers in the truss spans.
- ◆ Blast clean and paint structural steel as needed.

Chapter Three

Evaluation of Proposed Bridge

3.1 Soil and Geology Analysis Of Bridge Site

The main concern in this project is the interface between the substructure's foundation and the soil below and how both interact together. The type of footings which will be used in this project is based on the soil and geology report given. The Soils and Geology Department, which is part of the Tennessee Department of Transportation, is responsible for the Foundation Data Reports. The reports are the outcome of several borings or hole drillings that have been analyzed by the soil engineer for different locations determined by the design engineer. The depth of each hole is usually terminated upon reaching rock layer or up to 100'-0" if rock has not been encountered. The rock layer encountered must be at least 6'-0" minimum of sound rock before it is considered adequate for use.

This report^[1] presents the results of the foundation investigation for State Route 136 over Caney Fork River, done by the Soils & Geology Department. Two borings under each substructure were

^[1] Report No. 8909196, Soil & Geology Department, Tennessee Department of Transportation, SR 136/Caney Fork River

advanced to depths sufficient to determine subsurface conditions and make foundation recommendations.

The area is generally underlain by sandy clay resting upon shale. The overburden ranges in depth from 18'-0" on the western end of the structure to 37'-0" on the eastern end. Boring logs in figures 12 and 13 present details

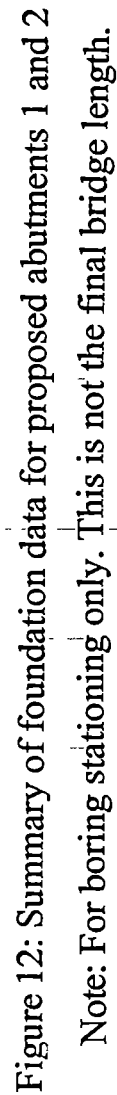
The findings and recommendation for each substructure are as follows⁽¹⁴⁾

- Abutment No 1, Station 29+13 97

“It is recommended that Abutment No 1 be supported by either point bearing steel "H" piles driven to refusal, or by a spread footing seated on rock. If steel "H" piles are to be used, refusal elevations will range from 816.92 to 818.57. Pile tip protection is recommended. If a spread footing is to be used, the footing should be seated at an elevation of 816.92 with an allowable bearing capacity of 15.84 ksf. Some rock excavation may be required. See figure 12.”⁽¹⁴⁾

- Abutment No 2, Station 36+44 82

“It is recommended that Abutment No 2 be supported by point bearing steel "H" piles driven to refusal. Pile lengths based on refusal



elevation of 793.96 may be used. Pile tip protection is recommended.

See figure 12.”⁽¹⁴⁾

- Pier No. 1, Station 30+39.39

“Two borings were drilled near the existing pier location in an attempt to verify the condition of the material underlying the existing footing. Rock elevation ranged from 782.50 to 759.50. The as-built bottom of footing elevation provided by the Structures Division is 762.17, indicating a high probability that the existing footing is seated on sound rock. An allowable bearing capacity of 19.84 ksf may be used. If a new pier is to be constructed at this location, the footing may be seated at a maximum elevation of 754.60 with an allowable bearing of 25.06 ksf. Up to 6'-0" of rock excavation will be required.

See figure 13.”⁽¹⁴⁾

- Pier No. 2, Station 31+59.38

“Borings near the existing pile location indicate rock to reside at an elevation of approximately 739.83. The as-built bottom of footing elevation provided by the Structures Division is 739.14. This indicates a high probability that the existing footing is seated on rock, however, borings indicate a continuous cavity to exist at an elevation of approximately 730.00. Due to the presence of this cavity under the

existing pier, an allowable bearing capacity of 16.71 ksf may be assumed. If a new pier is to be constructed at this location, it is recommended that the footing be seated at an elevation of 738.19 with an allowable bearing capacity of 16.71 ksf. If navigation considerations require the seal to be placed at a lower elevation, the footing should be seated at a maximum elevation of 715.22 with an allowable bearing capacity of 14.62 ksf. Up to 10'-0" of rock excavation will be required. See figure 13 ⁽¹⁴⁾

- Pier No. 3, Station 32+79.39

"It is recommended that Pier No. 3 be supported by a spread footing seated on sound rock. The footing should be seated at an elevation of 721.78 with an allowable bearing capacity of 14.62 ksf. Up to 13.12' of rock excavation will be required to remove unsuitable material (weathered and cavity ridden chert and limestone). See figure 13 ⁽¹⁴⁾

- Pier No. 4, Station 33+99.39

"It is recommended that Pier No. 4 be supported by a spread footing seated on sound rock. The footing should be seated at an elevation of 731.63 with an allowable bearing capacity of 19.48 ksf. Up to 8.25' of rock excavation will be required to remove unsuitable

material (weathered and cavity ridden chert and limestone). See figure 13.^{”(14)}

- Pier No 5, Station 35+19 39

“No drilling was done on Pier No 5 due to extreme low water conditions and steep terrain features, however, borings near the proposed pier location indicate that the upper 10’ to 16.5’ of rock is unsuitable to support proposed substructures. Projected rockline information indicates that rock will be encountered at an approximate elevation of 771.00. It shall be the contractor's responsibility to drill sufficient test borings at the proposed pier location to determine competent rock elevation. Competent rock shall consist of a minimum of 10’ of solid rock containing no cavities. Standard, down the hole hammer or air track type equipment may be used to drill the test borings. It is recommended that Pier No 5 be supported by a spread footing seated on competent rock. Preliminary design may assume a design bearing elevation of 757.87, however, the footing shall be seated at an elevation to be determined in the field by the engineer using the guidelines discussed above. An allowable bearing capacity of 9 40 ksf may be used. For estimation purposes, rock excavation of

up to 13 12' should be assumed for removal of unsuitable material.

See figure 13 »⁽¹⁴⁾

3 2 Environmental Studies

The office of Environmental Planning and Permits explained in their report ⁽⁹⁾ “The Environmental Technical Studies involve archaeologists, biologists, historians, and air quality and noise specialists. The study is responsible for the identification, avoidance, mitigation, and documentation of potential impacts of transportation facilities on streams, wetlands and other ecological and water resources, historical resources, archaeological resources, air quality, and noise. The staff conducts on-site data collection, coordinates with resource agencies and other TDOT divisions to minimize impacts, prepares designs and conducts follow-up monitoring for mitigation sites, and prepares reports for inclusion in environmental documents or to satisfy specific statutes. Avoidance of impacts to natural and cultural resources is the initial goal of this study. When unavoidable impacts to resources occur, the TDOT office works closely with internal and external entities to mitigate those impacts. This often requires designing wetland or stream segment restorations or replacements, preserving archaeological resources, providing

vegetative screening to historic properties near highway projects, and designing barriers for neighborhoods affected by highway noise”.

In this project the areas of study of particular concern are ecological and archaeological studies for the following reasons:

- 1) The location of the bridge is outside city limits;
- 2) The site is part of the Great Falls Reservoir Dam;
- 3) The proposed structure will improve traffic safety rather than increase traffic volume.

3.2.1 Ecology and Environmental Analysis⁽²⁾

The contract for this study was awarded to Chester, Ide Associates, Inc., a local firm from Nashville, Tennessee. The following subsections summarize their findings.

3.2.1.1 Soil Type

Soil in the area consist of different layers. It is, well-drained, stony and loamy soils from colluvium to rock. The underlying bedrock is primarily limestone, chert, siltstone, sandstone, and dolomite. The area around the project location is steep to the southwest, northwest, and northeast. The area southeast of the project location is hilly in the area located next to the river; however, the grade increases to the east.

3.2.1.2 Stream Quality

The Caney Fork River in the area of the bridge is open. The river at the State Route 136 Bridge is a part of the Great Falls Reservoir. There are several species of trees along the banks. These species include maple, hickory, sycamore, sweetgum, tulip poplar, and mimosa.

The river is approximately 550 ft. wide with an average daily flow (Great Falls Dam) of 3016 ft³/sec. The dissolved oxygen concentration was 8.9 mg/l, the pH was 8.90, and conductivity was 90 ohms/cm. The water had no unusual odors.

3.2.1.3 Habitat Assessment

The habitat assessment of the stream and surrounding area rated a condition considered good. However, due to the deep water, some normal habitat was not observed. There was no wetland located within the area of the bridge.

3.2.1.4 Biology

None of the rare and endangered species listed in Tables 1 and 2 were observed. The habitat does exist for *Hexastylis Contracta*^[1] in

^[1] Southern Heart leaf with a purple color usually flowers during the spring time.

TABLE - 1: LIST OF RARE AND ENDANGERED SPECIES FOR
WARREN COUNTY

15 JUN 1994

SCIENTIFIC NAME	COMMON NAME	*FEDERAL STATUS	**STATE STATUS
INVERTEBRATES			
LEPTOXIS SUBGLOBOSA UMBILICATA	UMBILICATE ROCK SNAIL	3B	
LITHASIA GENICULATA FULIGINOSA	GENICULATE RIVER SNAIL		
LITHASIA GENICULATA PINGUIS	SMALL GENICULATE RIVER SNAIL	3C	
MEDIONIDUS CONRADICUS	CUMBERLANT MOCCASINSHELL		
PEGIAS FABULA	LITTLE-WING PEARLYMUSSEL	LE	E
PLEUROBEMA GIBBERUM	CUMBERLAND PIGTOE	LE	
PLANTS			
DROSERA BREVIFOLIA	DWARF SUNDEW		T
GLYCERIA ACUTIFLORA	SHARP-SCALED MANNA-GRASS		S
HYDRASTIS CANADENSIS	GOLDENSEAL	3C	T
HYPERICUM ADPRESSUM	CREeping ST JOHN'S-WORT	C2	T
IRIS PRISMATICA	SLENDER BLUE FLAG		T
JUNCUS BRACHYCEPHALUS	SMALL-HEAD RUSH		S
LEPPEDEZA ANGUSTIFOLIA	NARROWLEAF BUSHCLOVER		T
LILIUM MICHIGANENSE	MICHIGAN LILY		T
LYCOPODIELLA ALOPECUROIDES	FOXTAIL CLUBMOSS		T
LYSIMACHIA TERRESTRIS	SWAMP LOOSESTRIPE		E
MYRIOPHYLLUM PINNATUM	CUTLEAF WATER-MILFOIL		T
PANAX QUINQUEFOLIUS	AMERICAN GINSENG	3C	T
PANICUM ACUMINATUM VAR LEUCOTHRIX	ROUGHISH WITCHGRASS		S
PANICUM HEMITOMON	MAIDENCANE		S
PEDICULARIS LANCEOLATA	SWAMP LOUSEWORT		T
PLATANThERA INTEGRa	YELLOW FRINGELESS ORCHID	3C	E
POGONIA OPHIOGLOSSOIDES	ROSE POGONIA		T
POLYGALA NANA	DWARF MILKWORT		S
PONThIEVA RACEMOSA	SHADOW-WITCH		E
RHYNCHOSPORA PERPLEXA	A BEAK-RUSH		T
RHYNCHOSPORA RARIFLORA	FEW-FLOWERED BEAK-RUSH		E-P
SPIRANTHES OVALIS	LESSER LADIES'-TRESSES		S
XYRIS IRIDIFOLIA	A YELLOW-EYED GRASS		S
VERTEBRATES			
CORYNORHINUS (=PLECOTUS)	EASTERN BIG-EARED BAT	C2	D
RAFINESQUI			
ETHEOSTOMA LUTEOVINCTUM	REDBAND DARTER		D
ETHEOSTOMA SP D	JEWEL DARTER (DORATION)	LE	D
HEMITREMIA FLAMMEA	FLAME CHUB		D
IXOBRYCHUS EXILIS	LEAST BITTERN		D
MYOTIS GRISESCENS	GRAY BAT	LE	D
MYOTIS LEIBII	EASTERN SMALL-FOOTED BAT	C2	D
MYOTIS SODALIS	INDIANA BAT	LE	D
NEOTOMA FLORIDANA	EASTERN WOODRAT		D
NOTROPIS RUPESTRIS	BEDROCK SHINER		D
TYPHLICHTHYS SUBTERRANEUS	SOUTHERN CAVEFISH		D

* LE - Listed Endangered
C2 - Designation Discontinued
3B - Designation Discontinued
3C - Designation Discontinued

** E - Endangered
T - Threatened
P - Probably endemic to the state
D - Deemed in need of management
S - Unranked, within the state

TABLE – 2: LIST OF RARE AND ENDANGERED SPECIES FOR
WHITE COUNTY

21 SEP 1993

SCIENTIFIC NAME	COMMON NAME	FEDERAL* STATUS	STATE** STATUS
INVERTEBRATES			
LITHASIA GENICULATA PINGUIS	SMALL GENICULATE RIVER SNAIL	3C	
PLEUROBEMA GIBBERUM	CUMBERLAND PIGTOE	LE	
PLANTS			
CERATOPHYLLUM ECHINATUM	PRICKLY HORNWORT		S
CONRADINA VERTICILLATA	CUMBERLAND ROSEMARY	LT	T
ERIOCAULON DECANGULARE	PIPEWORT		E
HEXASTYLIS CONTRACTA	<u>SOUTHERN HEARTLEAF</u>	C2	T
HYDRASTIS CANADENSIS	GOLDENSEAL	3C	T
LECHEA LEGGETTII	PINWEED		S
LILIUM MICHIGANENSE	MICHIGAN LILY		T
PLATANThERA FLAVA VAR FLAVA	SOUTHERN REIN-ORCHID	3C	S
PLATANThERA PERAMOENA	PURPLE FRINGELESS ORCHID	3C	T
TORREYCHLOA PALLIDA	PALE MANNA GRASS		S
XYRIS AMBIGUA	A YELLOW-EYED GRASS		E
VERTEBRATES			
ACCIPITER COOPERII	COOPER'S HAWK		T
AMBYSTOMA TALPOIDEUM	MOLE SALMANDER		D
AMMODRAMUS SAVANNARUM	GRASSHOPPER SPARROW		T
BUTED LINEATUS	RED-SHOULDERED HAWK		D
ETHEOSTOMA BARRENENSE	SPLENDID DARTER		D
FALCO PEREGRINUS	PEREGRINE FALCON	E/SA	E
HYLA GRATIOSA	BARKING TREEFROG		D
LIMNOTHLYPIS SWAINSONII	SWAINSON'S WARBLER		D
MELANERPES ERYTHROCEPHALUS	RED-HEADED WOODPECKER		D
MYOTIS GRISESCENS	GRAY BAT	LE	E
MYOTIS SODALIS	INDIANA BAT	LE	E
NOTROPIS RUPESTRIS	BEDROCK SHINER		D
OPHISAURUS ATTENUATUS	EASTERN SLENDER GLASS LIZARD		D
LONGICAUDUS			
PLECOTUS RAFINESQUII	EASTERN BIG-EARED BAT	C2	D
SOREX LONGIROSTRIS	SOUTHEASTERN SHREW		D

* LE - Listed Endangered
C2 - Designation Discontinued
Designation Discontinued
3C - Designation Discontinued

** E - Endangered
T - Threatened
D - Deemed in need of management

the area of the bridge. The area to the southeast of the bridge represents the best habitat, one with rich moist soils of deciduous forests, for the listed species. There were no aquatic species listed as endangered, therefore it was determined that aquatic sampling was not necessary.

3.2.1.5 Recommendations

- 1) Because *Hexastylis Contracta* (Southern Flower) was not found around the bridge site, no danger will be imposed on this flower from construction.
- 2) Environmental controls (Erosion Control) should be properly implemented and maintained, such as erosion silt fence, bales of hayes, silt booms, etc.

3.2.2 Archaeological and Historical Assessment⁽³⁾

The contract for this study was awarded to Duvall and Associates, Inc., Franklin, Tennessee. Mr. Glynn D. Duvall conducted the reconnaissance of the proposed bridge area complying with the National Historic Preservation Act of 1966 (Public Law 91-190; 91 stat. 852; 42 USC 4321-4347).

The reasons for study around the bridge area were as follows:

- 1) To determine if the area has any potential for archaeological sites.
- 2) To determine vegetation patterns within the bridge boundaries.
- 3) To observe exposed ground surfaces for archaeological materials.

3.2.2.1 Site Coordination & Reconnaissance

A search of the site files and coordination with the Tennessee Division of Archaeology indicated the following:

- 1) No historical sites had been previously reported around the area.
- 2) None of the properties which are included in the national registry of historic places were located within the bridge area, or in close distance to the project.
- 3) No cultural material was identified in the quadrants. Road cuts, cleared areas, and scraped driveway areas in the southwest quadrant were all negative.

3.2.2.2 Conclusions

While the bridge does meet the 50-year criteria for inclusion in the national register of historic places, for other reasons the bridge had been deemed ineligible for inclusion.

In the unlikely event that human burials or skeletal remains are unearthed during construction on the bridge and approaches, construction on that section should cease, and the Tennessee Department of Transportation, Environmental Planning Office, should be immediately notified.

Chapter Four

Integral Abutment Bridges

“In May of 1930, Professor Hardy Cross published an article in the proceedings of the American Society Of Civil Engineers⁽¹²⁾ that changed the field of structural engineering significantly”⁽¹³⁾. Professor Cross’s article explained an easier method for the analysis of continuous beams and frames. Until the Hardy Cross method, most bridges were designed as a series of statically determinate simple spans.

Moment distribution, also known as the Hardy Cross Method, was immediately accepted by most structural engineers, and with the advent of this method, continuous bridges began to appear. However, the advancement in the design and construction of continuous bridges started to be a reality when computers became available to everyone. Along with the development of continuity, the use of integral abutments to get rid of joints on bridges has progressed. Ohio, South Dakota, and Oregon appear to have pioneered the use of jointless bridges in concrete in the 1930’s and 1940’s, with California following in the mid 1950’s.

With the national interstate highway system construction boom in the 1950’s and early 1960’s, the use of continuous jointless bridge construction

began a real growth. By the mid-1960's, Tennessee and five other states had adopted continuous bridges with integral abutments as standard construction.

Many states have not felt comfortable using a system without expansion joints for temperature variation displacements. Of the 52 states, only 29 indicated that their states use integral type abutments, and the rest are considering the use of them in the near future.

The Tennessee Department of Transportation has extensive experience with integral abutment construction and its performance. It is estimated that over 300 steel and 700 concrete bridges have been built with integral abutments, and the state is very pleased with the performance of these structures.

The maximum length limits using integral abutments were determined by setting a limit of expansion or contraction of 1 inch. This figure was developed empirically over a period of several years. By using a simplified column analysis with an unsupported length of 10 feet, the state calculated the piling stresses to be slightly over yield when deflected only 1 inch. Tennessee uses the average AASHTO temperature range of 65° F for concrete structures and 120° F for steel .

The maximum bridge lengths (2L) for this allowable deflection (Δ) are about 800 feet for concrete and 400 feet for steel.

$$L \text{ (Concrete)} = \Delta / \alpha_c (\Delta T)_c = 1 / 0.0000060 (35)(12) = 397 \text{ feet}$$

$$L \text{ (Steel)} = \Delta / \alpha_s (\Delta T)_s = 1 / 0.0000065 (60)(12) = 214 \text{ feet}$$

$$\Delta T_c = \text{Avg. Temperature Range For Conc.} = (95^\circ - 25^\circ) / 2 = 35^\circ \text{ F}$$

$$\Delta T_s = \text{Avg. Temperature Range For Steel} = (120^\circ - 0^\circ) / 2 = 60^\circ \text{ F}$$

$$\alpha_c = \text{Coefficient Of Thermal Expansion For Conc.} = 0.0000060$$

$$\alpha_s = \text{Coefficient Of Thermal Expansion For Steel} = 0.0000065$$

Tennessee has recently completed a research project performed by the University of Tennessee to verify the assumptions used in design, and the results supported current TDOT practice

“Today's bridge designers are essentially striving to achieve the same goals as their counterparts were 70 years ago long term serviceability, low maintenance characteristics, and economy of construction”⁽¹³⁾ One of the more important aspects of design is a reduction or elimination of roadway expansion devices and associated bearings, an aspect which is consistently overlooked or avoided by many bridge design practitioners. However, a growing number of bridge designers are using integral abutments in order to eliminate expansion joints wherever possible.

A previous study published in 1983, in the Transportation Research Board (Transportation Research Record 903⁽⁴⁾) analyzed data collected on

420 bridges in 39 states over an eight-year period. The study showed the following:

- 1) 1/3 of the bridges had abutments that experienced horizontal movements due to thermal expansion.
- 2) Almost 50 percent of the abutments had been forced to deflect horizontally with movements ranging from 0.1 to 8.0 inches.
- 3) 26% of the stub-type abutments experienced horizontal movements ranging from 0.3 to 14.4 inches.
- 4) 62% of the spill-through abutments experienced movements due to thermal expansion ranging from 0.5 inches to 8.8 inches.
- 5) 2/3 of the damage that occurred in the bridges was due to joints and 1/3 of the damage was due to bearings.
- 6) Abutment damage from horizontal movement due to thermal expansion occurred with greater frequency for movements of 2 inches or greater.

- 7) Well over 1000 bridges with integral abutments have been constructed in the state of Tennessee alone, with calculated movements ranging from $\frac{1}{4}$ inch to 2 inches.

Structural damage related to horizontal movements due to thermal expansion is described as followed:

- 1) Misalignment of bearings and superstructure.
- 2) Beams jammed against abutments.
- 3) Girders overrunning abutments.
- 4) Cracks in the superstructure.
- 5) Shearing of anchor bolts.
- 6) Damage to expansion joints.
- 7) Misplacement of steel bearings.
- 8) Deformed elastometric pads.
- 9) Cracking of bridge seats.

Studies and observations that have been conducted on abutments sparked an interest in integral abutments. Integral abutments appear to be a cure for most of the problems discussed earlier.

Chapter Five

Methods and Analysis Used to Design Integral Abutments at Caney Fork River Bridge

5.1 Discussion

“Integral abutments have been used successfully for 50 years. Their implementation has been anything but an exact science, but rather a matter of intuition, experimentation, and observation. The primary reason that exact design approaches have not been fully developed is that the analysis of a pile under lateral loads is a problem in soil-structure interaction. The deflected shape of the loaded pile is dependent upon the soil response, and in turn, the soil response is a function of pile deflection, thus the system response cannot be determined by the traditional rules of static equilibrium. Soil response is a non-linear function of pile deflection.”⁽¹³⁾

A number of methods have been used to model soil-pile interaction. However, one of the most promising methods of analysis is found in report no. FHWA-5A-91-048, COM624P-Laterally Loaded Pile Analysis program for the microcomputer, version 2.0⁽⁸⁾. The methods used in reference 8 recognize that the solution to the problem of laterally loaded piles requires differential equations to obtain pile

deflections and iteration to solve these equations, since soil response is a non-linear function of the pile deflection along the length of the pile.

The most popular technique used to model soil-pile interaction in the United States is the P-y method. From this procedure pile response is obtained by an interactive solution of a fourth-order differential equation using finite-difference techniques. The soil response is described by a family of non-linear curves (P-y curves) that compute soil resistance P as a function of pile deflection y.

Structures must be designed to accommodate the movements and stresses caused by thermal expansion and contraction. Rather than accommodating these movements by using unnecessary bridge deck expansion joints and expansion bearings, the decision was made to use integral abutment at the end of Caney Fork River Bridge and expansion joint at the beginning of the bridge. The only reason behind using expansion joint at abutment 1 is because the total height of the piles was below 10 feet. The TDOT policy of integral abutments on piles is a minimum height equal to 10 feet.

These are the notations and criteria used for thermal movement calculations (See figure 14 for illustration):

α = Thermal Coefficient for concrete = $0.000006/F^{\circ}/ft$

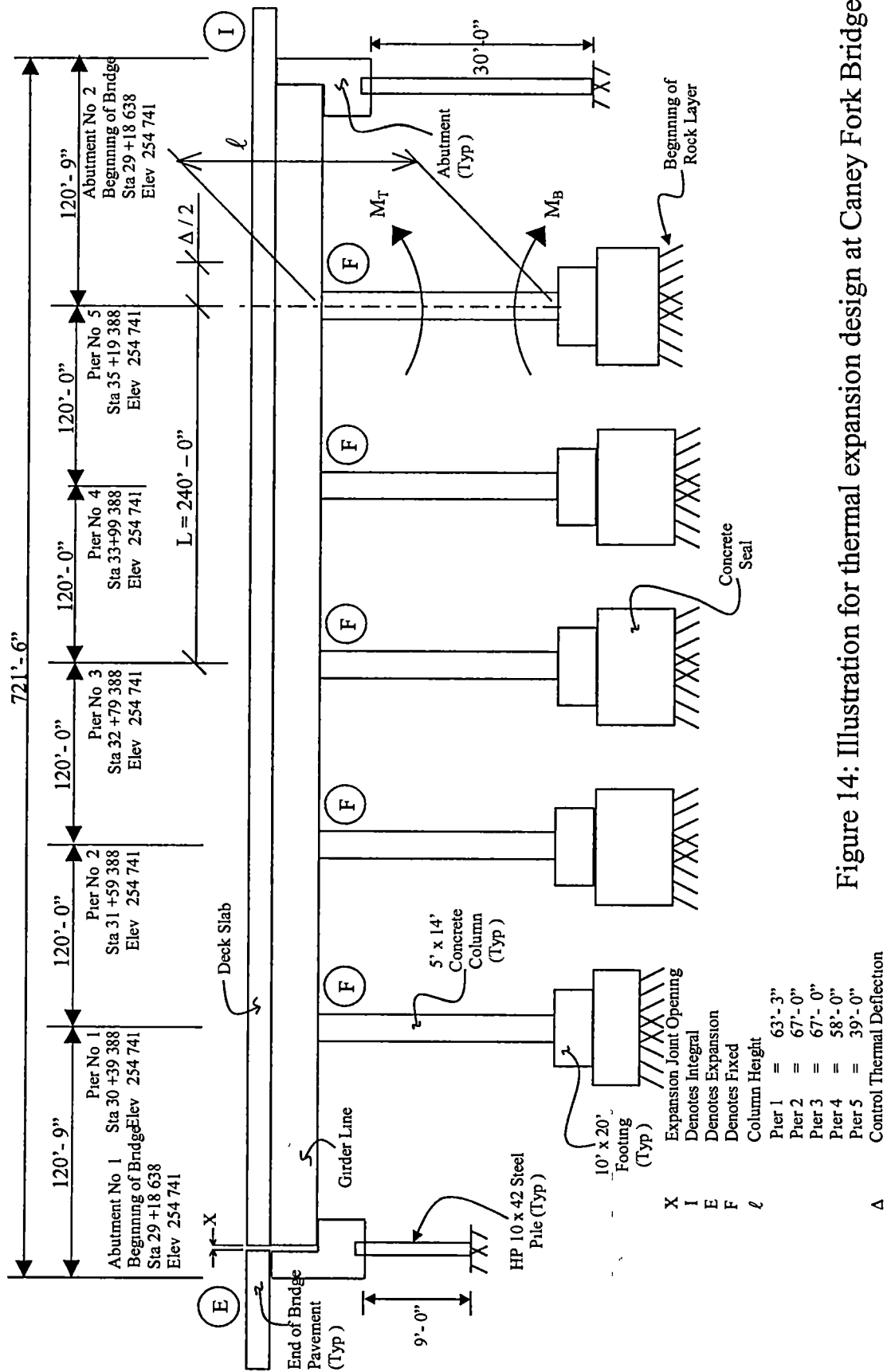


Figure 14: Illustration for thermal expansion design at Caney Fork Bridge

- Δ = Total thermal movement due to expansion and contraction (in)
- L = Length of bridge over which thermal movement occurs, measured from the point of fixity to the end of the bridge (ft).
- ℓ = Pier height (ft)
- M_B = Moment at base of column due to $\Delta/2$ displacement at top (k-ft)
- M_T = Moment at top of column due to $\Delta/2$ displacement at top (k-ft)
- P = Force required to produce $\Delta/2$ displacement (lbs.)
- E_c = 1.0×10^6 for concrete column in long term deflection (lbs./in²)
- $w_{conc.} = 145 \text{ lbs./ft.}^3$
- $f'_c = 4500 \text{ psi}$

$$\therefore E_c = (145)^{1.5} (33) \sqrt{(4500)} = 3.9 \times 10^6 \text{ psi}$$

The 4500 psi value is used instead of 3000 psi because the average concrete strength obtained for Class "A" Concrete⁽¹⁾ is about 4500 psi.

Class "A" Concrete Specifications:

Min. 28 days compression strength = 3000 psi

Min. Cement Weight = 564 lbs/c.y.

Min. Water/Cement Ratio = 0.45 lb/lb

Avg. Air Content = $6 \pm 2 \%$

Slump Sum = 3 ± 1 inches

5.2 Brief Description of COM624P Program Analysis

The analysis of a pile under lateral loading is a problem in soil-structure interaction; that is, pile response and the soil response is a function of pile deflection.

The problem cannot be solved by the equations of static equilibrium. Instead, differential equations must be solved to obtain the deflection of the pile; iteration must be employed because the soil response is a nonlinear function of pile deflection and of position along the length of the pile. See Figure 15.

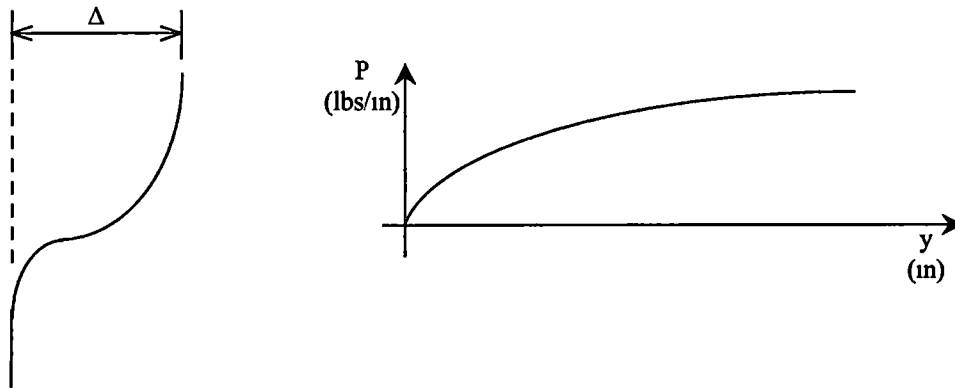


Figure 15: Soil stresses vs. pile deflection graphs and diagrams

$$P = \int F \left(\frac{\text{lbs}}{\text{in}^2} \right) dx = \text{lbs/in}$$

$$Es = \frac{P}{y} \quad \text{lbs/in}^2$$

E_s : Soil modulus, characterized as a nonlinear spring whose stiffness is largest with small deflection and decreases as the deflection of the pile increases. Nevertheless, E_s changes in value along x and y for any particular load P_t at the top of pile. One cannot assume a solution based on some presumed variation of E_s as a function of x . Iteration will invariably be necessary, and a computer is necessary to make the calculations.

Definition of Soil Response (y) See Figure 16.

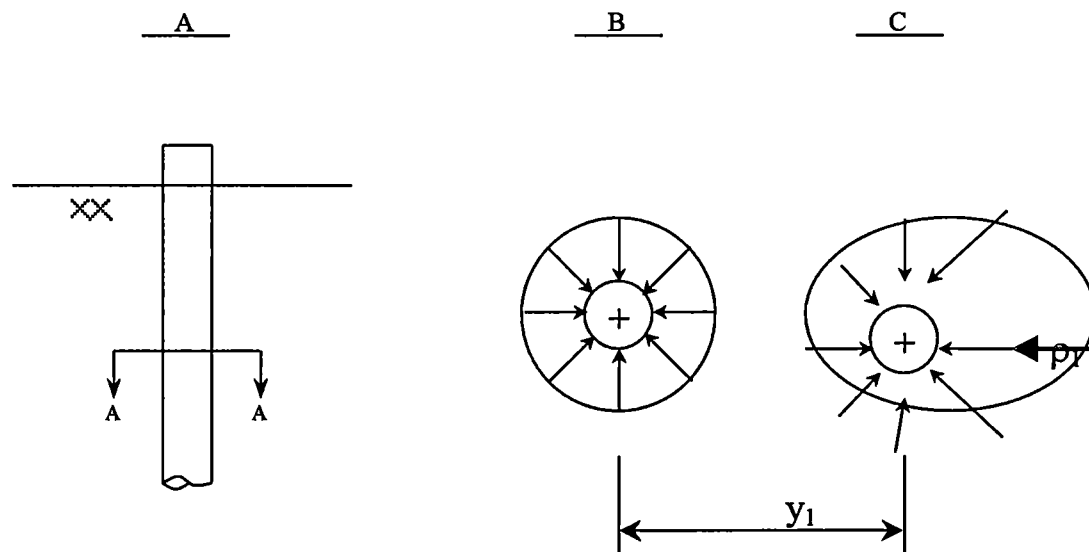


Figure 16: Soil response around pile

$$\rho_1 = \int F \left(\frac{lbs}{in^2} \right) dx$$

$$\frac{\rho_1}{y} = E_s$$

$$\rho_1 = -E_s(y)$$

$$y = \Delta \quad \text{deflection}$$

Buckling needs to be investigated when (See Figure 17):

- Axial load is high.
- Pile's unsupported portion is extending above ground.

$\sum M$ at point A

$$(M + dM) - M + P_x dy - V_v dx = 0$$

$$\frac{dM}{dx} + P_x \frac{dy}{dx} - V_v = 0$$

$$\frac{d^2 M}{dx^2} + P_x \frac{d^2 y}{dx^2} - \frac{dV_v}{dx} = 0$$

$$EI \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} + E_s y = 0$$

$$V_n = V_v \cos \theta - P_x \sin \theta$$

$$V_n = V_v = P_x \frac{dy}{dx}$$

$$\therefore \left(\frac{d^2 M}{dx^2} \right) + P_x \frac{d^2 y}{dx^2} + k y - w = 0$$

Dist. load along the pile

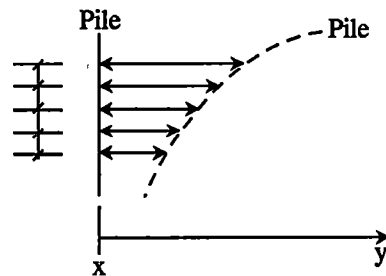
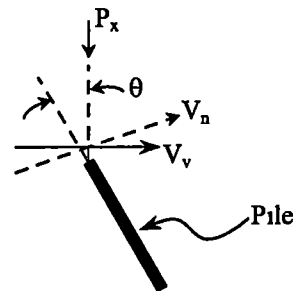
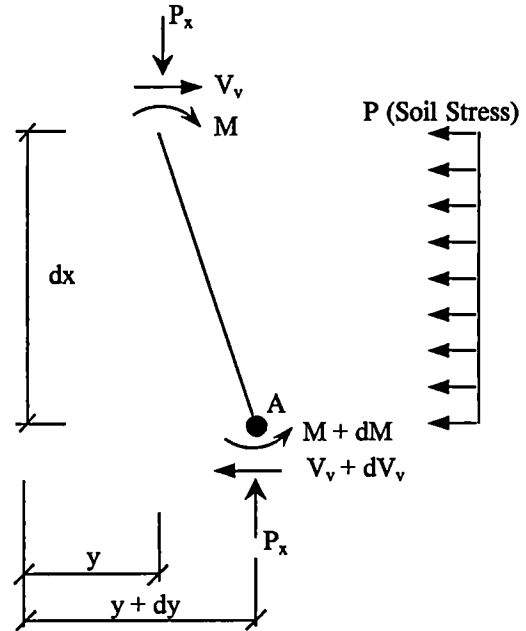


Figure 17: Moments and loads effecting pile deflection

The COMP624 program will solve the equation by
finite difference techniques as shown below.

$$\left(\frac{d^2 M}{dx^2}\right)_m = \left[\frac{y_{m-2}(R_{m-1}) + y_{m-1}(-2R_m - 2R_{m+1}) + y_m(4R_m + R_{m-1} + R_{m+1})}{y_{m+1}(-2R_m - 2R_{m+1}) + y_{m+2}R_{m+1}} \right] \frac{1}{h^4}$$

$$P_x \left(\frac{d^2 y}{dx^2}\right)_m = \frac{P_x(y_{m-1} - 2y_m + y_{m+1})}{h^2}$$

$$R_m = \text{Flexural Rigidity at Point } (m) = E_m I_m$$

The following four tasks must be completed before running the COMP64
program:

- 1) Find the bridge properties. See Appendix A.
- 2) Calculate the total axial service loads (Dead Load and Live Load) for each pile.
- 3) Determine and measure “ $\Delta/2$ ” thermal movement.
- 4) Calculate the plastic moment capacity of the HP10x42 steel pile embedded inside the abutment beam.

Calculations of HP10x42 steel pile capacity using the interaction diagram and analysis will be done after running the COMP624 program. See Appendix C.

5.3 Bridge and Soil Properties

In order to be able to run COMP624 Program, some of the bridge properties had to be determined. A modification factor ($n=0.707$) was obtained to accommodate the difference in the strength of concrete between slab (3000 psi) and girders (6000 psi). See appendix A for more details.

5.4 Total Axial Service Loads at the Abutments

Live Loads :

$$(LL + I)(4221 \text{ Service Load}) = 4(59.39) = 239.56 K$$

4221 is a computer program that analysis dead loads and live loads for continuous bridges.

Dead Loads :

$$\text{Rails} + \text{Overlay} = 18.199(4) = 72.48 K$$

$$\text{Slab} = \frac{8.25}{12} (33.83) \left(\frac{120}{2} \right) 0.15 = 209.32 K$$

$$\text{Filler} = 4 \left(\frac{1.5(42)}{144} \right) \left(\frac{120}{2} \right) 0.15 = 15.75 K$$

$$\text{Beams} = 4 \left(\frac{767}{144} \right) \left(\frac{120}{2} \right) 0.15 = 199.25 K$$

$$\text{Abut. 1 Wt.} = 136.11 K$$

$$\text{Abut. 2 Wt.} = 105.00 K$$

$$\text{Abut. 1 Wing Wt.} = 117.00 K$$

$$\text{Abut. 2 Wing Wt.} = 102.50 K$$

$$\text{Total Loads} = 987.5 K \text{ (Abut. 1)}$$

$$\text{Total Loads} = 942.0 K \text{ (Abut. 2)}$$

Unbraced Length of Pile at Abut. No. 1 = 9' - 0"

Unbraced Length of Pile at Abut. No. 2 = 31' - 0"

$$\text{Capacity of each pile} = \frac{110^T}{2} = 55^T = 110K \quad (\text{AASHTO 4.5.7.3})$$

$$\text{Abut. 1: No. of piles} = 987.5 \left(\frac{1 \text{ pile}}{110^K} \right) = 8.97 \approx 9 \text{ piles}$$

$$\text{Abut. 2: No. of piles} = 942.0 \left(\frac{1 \text{ pile}}{110^K} \right) = 8.56 \approx 9 \text{ piles}$$

Use HP 10 x 42 Steel piles ASTM A36 steel

Use: 15 piles staggered and 2 piles under each wing for Abut. No. 1

Use: 8 piles plus 2 piles under each wing for Abut No. 2

$$\begin{aligned} \text{Total Load (Factored) / Pile @ Abut. 1 (DL + LL)} &= \frac{239.56(2.17) + 750.0(1.3)}{9} \\ &= 160.00k \end{aligned}$$

$$\begin{aligned} \text{Total Load (Factored) / Pile @ Abut. 2 (DL + LL)} &= \frac{239.56(2.17) + 704.30(1.3)}{15} \\ &= 100.00k \end{aligned}$$

$$\text{Dead Loads (Service) / Pile @ Abut. 1 (DL + LL)} = \frac{750.00}{15} = 50.00k$$

$$\text{Dead Loads (Service) / Pile @ Abut. 2 (DL + LL)} = \frac{704.30}{9} = 78.00k$$

See Figure 18 for final axial service loads illustration.

The reason behind using 15 piles is to restrain abutment number 1 from deflection or movement. The expansion joint will take all the thermal expansion movement.

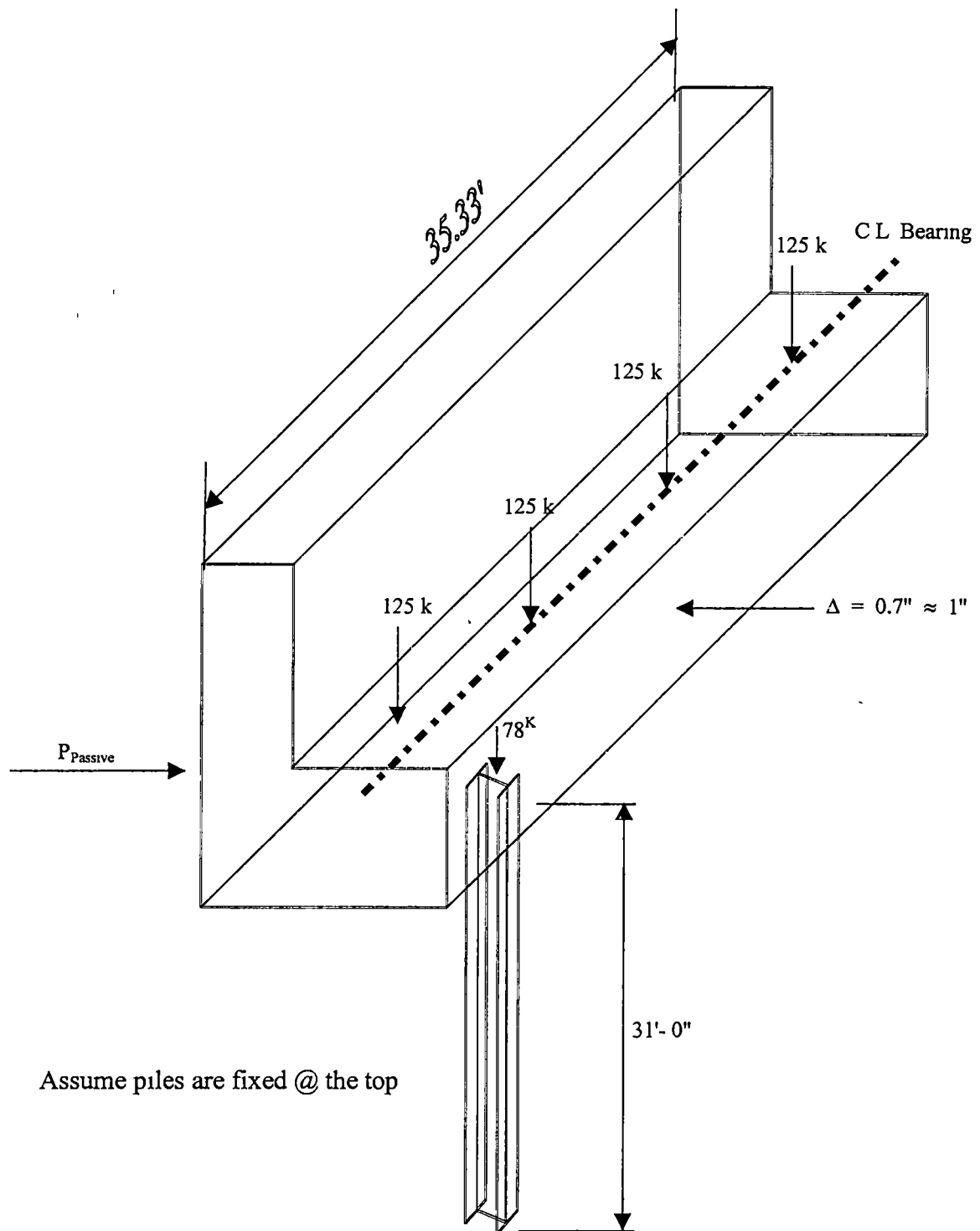


Figure 18: Vertical and horizontal loads profile

5.5 Determine “ $\Delta/2$ ” Thermal Movement

Thermal stiffness needs to be calculated before determining the thermal movement.

Procedure for finding the center of thermal stiffness of the Caney Fork Bridge:

- 1) By trial and error, pick a point along the bridge
- 2) Since Abutment No. 2 is considered to be fixed, the soil is pushing against the Abutment No. 2 endwall by the force $P = 108^K$.
- 3) The slab thermal expansion is doing work against the other end (Abutment No. 1) with expansion joint.
- 4) The coefficient of friction between the beam (steel plate embedded at the bottom) and the copper plate at the expansion device is considered to be 0.40.
- 5) The total dead load of the superstructure multiplied by the coefficient of friction of expansion device is greater than the load generated by the deflection of columns from thermal expansion.
The thermal stiffness is powered by both ends.
- 6) The thermal stiffness is almost in the middle of the bridge, but because of the variations in column heights it is off a little towards Abutment No. 2.

See Appendix B for COMP624 input and output data.

5.6 Plastic-Moment Capacity of the embedded Piles at the Abutments

HP 10 x 42 piling is embedded 12 inches into the abutment beam, with the piling oriented for bending about the strong axis. The pile must be able to develop a sizable resisting moment at the top in order to achieve double curvature within its design length under 1 inch deflection.

Using a free head condition at the top of the pile and analyzing the pile as a cantilever maximized the pile capacity at a displacement of 1.0 inch.

HP 10 x 42 pile properties (ASD p.138) see figure 19.

$$F_y = 36 \text{ ksi}$$

$$A_s = 12.4 \text{ in}^2$$

$$S_x = 43.4 \text{ in}^3$$

$$v = 4.13 \text{ in}$$

$$Z_x = 48.30 \text{ in}^3$$

$$b_f = 10.075 \text{ in}$$

$$I_x = 210 \text{ in}^4$$

(In pile calculation program use $d = 9.7 \text{ in}$)

Abutment no. 2 length of pile = 32'-0"

Unsupported pile length = 31'-0"

Calculate Plastic Moment Capacity of embedded pile

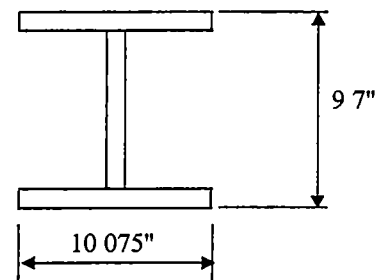


Figure 19: Pile cross section

1) Plastic moment at base of stub abutment

$$M_p = F_y Z = 36,000(48.3)/12000 = 144.90 \text{ k-ft}$$

2) Check the ability to develop the plastic moment within the 12" embedment of the pile into the abutment beam. See Figure 20.

$$c = 6"$$

$$f'_c = ? \text{ psi (Conc. Strength Around The Embedded Pile)}$$

$$a = \beta_1 c$$

$$\beta_1 = 0.85$$

$$a = 5.1"$$

$$b_f = 10.075 \text{ in}$$

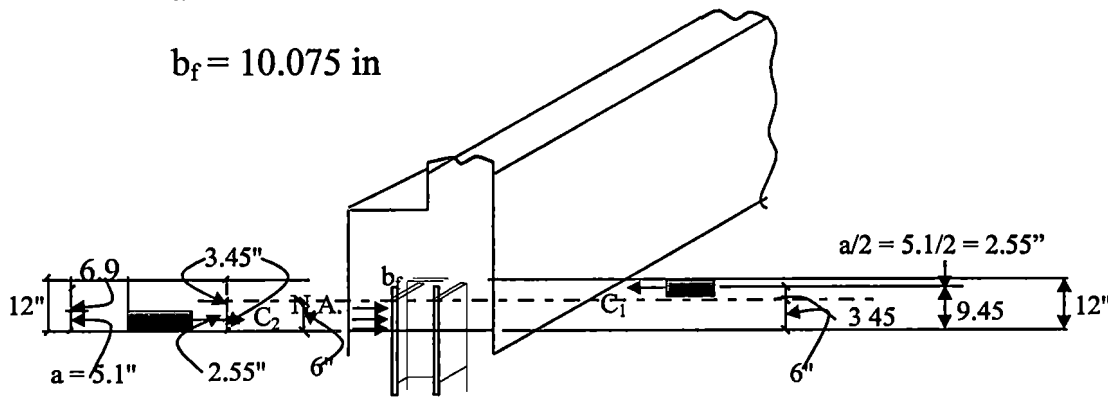


Figure 20: Pile and abutment interface reactions

$$C_1 = C_2 = 0.85 f'_c a b_f = 0.85 (f'_c) 5.1 (10.075) = 43.67 f'_c$$

$$Z = 12 - 5.1 = 6.90 \text{ in}$$

$$M_{\text{couple}} = 43.67 f'_c (6.9) = 301.32 f'_c \text{ in}^3$$

$$M_p = 301.32 f'_c$$

Determine f'_c required to develop plastic moment

$$M_p = F_y Z$$

$$301.32f_c = 144.90(12)$$

$$f_c = 5.77 \text{ ksi}$$

Research has determined that the concrete bearing capacity around large inserts can reach an average of $3.78 f_c$ bearing. The concrete strength at the abutment beam is taken as 3.00 Ksi.

$$5.77 \text{ ksi} / 3.78 = 1.53 \text{ Ksi} < 3.00 \text{ Ksi} \text{ O.K.}$$

Therefore, the plastic moment capacity can be developed in the pile head with a safety factor of nearly 2.0.

5.7 Calculate Column Capacity (HP 10 x 42) Against Bending.

From COM624P output, both graphs and the tabulated data in Appendix B have been applied in Appendix C to calculate the maximum bending moment capacity of the HP 10 x 42. The calculations indicated that the column is adequate. (See Appendices B and C for more information).

5.8 Summary

Through the following series of operations, the adequacy of a particular pile and the pile point of fixity can be determined.

- 1) Determine the total axial service load (dead load plus live load).
- 2) Determine the maximum pile deflection due to thermal movement.

- 3) Calculate the plastic moment capacity of the abutment beam with embedded piles.
- 4) Input the information into the COMP624 program and determine the length over which the pile will deflect.
- 5) Using the length calculated in Step 4, run the iteration program analysis for the particular pile. This program will determine the pile's bending adequacy.

The final results showed that the HP 10 x 42 is adequate for the integral abutment at the Caney Fork Bridge.

Chapter Six

Evaluation of Design Joints vs. Integral Abutments

The choice of whether to use expansion joints or integral abutments in a bridge is dependent upon the total length of the bridge. The following criteria have evolved from experience at TDOT with integral abutment bridges. When the total anticipated movement at an abutment is less than 2 inches and is not restrained against movement, no joint will be required and the superstructure and abutment beam can be constructed integrally. An unrestrained abutment is one that is free to rotate such as a stub abutment on one row of piles or an abutment hinged at the footing with the axis of rotation being skewed between 60° and 90° to the direction of movement. When the total anticipated movement at an abutment is less than $\frac{1}{4}$ inch, the abutment may be constructed integrally with the superstructure regardless of the support conditions.

When the total movement at an abutment is between $\frac{1}{4}$ inch and 2 inches and the abutment is restrained against movement and rotation, or the total movement is between 2 inches and 4 inches, the designer should specify a strip seal with required movement of 4 inches. When the total

movement is greater than 4 inches, the designer should specify a modular expansion joint with the required movement rounded up to the nearest inch.

In a paper published in 1992, titled “Performance Evaluation of Integral Abutment Bridges”⁽⁵⁾, the results of a survey suggested many reasons to use integral abutments and the advantages behind this approach. Following are some reasons behind the usage of integral abutments:

- (1) The force exerted by the thermal expansion movement on the abutment is far greater than the passive force of the fill material behind it. To withstand this massive force, a rigid connection is needed such as an integral abutment rather than using an expansion joint at the same location.
- (2) A paper on movements and forces in skewed bridges explained:⁽⁶⁾“The force required to stabilize rotation of the abutments on a 30° skewed abutment approaches 50% of the passive pressure on the backwall; for a 45° skewed abutment, 70% of the total passive pressure would be required. These values far exceed the frictional resistance of the backfill against the abutment backwall or the shearing resistance of the backfill”.

- (3) A paper presented by Dr. Charles Roeder and Shashi Moorthy⁽⁷⁾ discuss further the phenomenon associated with skewed bridges, both tangent and curved. The report points out that: “when full-depth joints become clogged with dirt and debris, longitudinal thermal movements induce large transverse movements. Utilizing integral abutment connections allows the entire backwall and deck to be mobilized to transmit the horizontal forces. Similarly, curved-girder bridges generate more unpredictable forces than skewed bridges. It is unclear whether the primary thermal movements follow the chord between abutments or the local tangent at abutments. This uncertainty can lead to misaligned bearings and unidirectional joints resulting in damage”.
- (4) Integral abutments can only be applied to relatively short bridges. A review of the National Bridge Inventory System in 1992 revealed that:⁽⁵⁾“80% of all bridges in the NBIS are less than 180 feet in total length. About 90% are less than 400 feet in length. These data indicate that most bridges have the

potential to be jointless given the proper conditions of flexibility at the abutments”.

- (5) Cracks develop at the interface of abutment backwalls or approach pavements with the asphalt roadway paving, causing a bump to be present at this location. A bump will be created whether or not joints are used at abutments. Subsequent paving may be carried over jointless bridges without interruption. One fact is certain; if there is a joint, there will be a bump.
- (6) Integral abutment bridges are generally limited to pile-supported abutments. While pile-supported abutments are desirable, stub abutments with up to $\frac{1}{4}$ inch of total movement have been successfully used for spread footings supported on, but not keyed into rock. Additional tall abutments that are hinged at their base have been used for movements up to 2 inches.
- (7) Cranes cannot go close to abutments to place beams since backfill is placed after beams are set. Therefore, large booms are required. The most common method of construction, even for bridges with joints, is to set beams prior to pouring

backwalls. If the backwalls for expansion abutments are poured and backfilled before setting the beams, the abutments will be at a greater risk of displacement resulting from crane loadings than jointless abutments, which have no backwalls at the time of beam erection.

- (8) Integral abutments make future widening easier than expansion abutments. Reconstruction or widening of an expansion joint is costly and time consuming.

When a joint is required, it should have the following characteristics:

- (1) Accommodate the full range of structure movements without exceeding a four (4) inch clear span at deck surface level when at maximum opening
- (2) Provide proper anchorage and structural capacity to resist the anticipated loads. Welded strap anchors are required in lieu of welded stud connectors.
- (3) Have good riding qualities.
- (4) Not impart undue stresses to the structures due to structure expansion and contraction.
- (5) Be reasonably silent and free of vibration.
- (6) Resist snow plow damage and traffic abrasion
- (7) Facilitate maintenance repair.

(8) Be leak proof with the sealing element continuous for the entire structure width. Field splices in the sealing element will not be permitted.

Although many expansion devices appear satisfactory on paper, the real test is in their performance under normal field conditions. In recent years, many expansion devices have proven to be unsatisfactory.

Departments of Transportation are still searching for a device will meet all of the above requirements.

Chapter Seven

Conclusion and Recommendations

Structures must be designed to accommodate the movements and stresses caused by thermal expansion and contraction. While some bridge designers choose to accommodate these movements by using bridge deck expansion joints and expansion bearings, this solution frequently creates more problems than it solves. Structural deterioration, due to leaking expansion joints and frozen expansion bearings, constitute major bridge maintenance problems.

To eliminate problems associated with leaking expansion joints and frozen expansion bearings, it is TDOT's policy to design and construct bridges with continuous superstructures, fixed or integral bearings at the piers and abutments, and no bridge deck expansion joints unless absolutely necessary. When expansion joints are necessary, they will normally be provided only at abutments.

The Caney Fork Bridge thermal movement analysis showed a 0.7" movement on the integral abutment side, a value within the required range by TDOT office policy. Using an integral abutment, rather than an

expansion joint at one end of the bridge, was beneficial in many ways. The simplification of construction details and the use of longer spans made life easier for the construction crew. Most importantly, however, costs were greatly reduced in both initial construction and long-term maintenance.

A great deal of attention should be given to the construction of integral abutments. Piles should be driven vertically, in a single row, and oriented for strong-axis bending to accommodate the total lateral displacement. The ends of the girders should be connected to the abutment by pouring the last portion of slab concurrently with at least the top 12" of the abutment's end diaphragm. The end of the bridge pavement should be made of concrete only, and finally the backfill material should be granular selected backfill for easy compaction and to help carry water intrusion away from the abutments.

The Rock Island Bridge (Caney Fork Bridge) serves as an excellent example of an integral abutment bridge. The clean lines of the bridge and the beautiful scenery around it make this bridge particularly attractive.

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Appendices

APPENDIX A

Bridge and Soil Properties

Strength calculations are based on the 3000 psi minimum value, and deflections are based on the 4500 psi average value. In order to account for long term effects of creep in reinforced concrete, the dead load correction diagrams^[2] to compensate for deflections should use the value of $E_c = 1,500,000$ psi. When figuring stresses in fixed columns due to thermal expansion or contraction of the superstructure, E_c should be based on long term deflection using E_c equal to 1.0×10^6 psi.

Temp. Range = 25°F → 95°F (for concrete)

Coefficient of expansion = α = 0.0000060/°F/ft

Length of structure = 721.50 ft

Total movement = $\alpha(\Delta L)$ = $721.50(0.00505) = 3.65$ in $\approx$
4.0 in

Movement @ each end = 2 in

With the girders set at a temperature of 60°F, the required movement demand at each abutment will be ± 1 inch.

^[2] Dead Load Correction Diagram = All D L. factors except prestressed & self wt. factors

Property of Soil above Water

$$\gamma \text{ (Soil Weight)} = 106 \text{ lb/ft}^3 = 0.061 \text{ lb/in}^3$$

$$\phi \text{ (Angle Of Friction)} = 25^\circ$$

$$C \text{ (Avg. Undrained Shear Strength) Cohesion} = 3.47 \text{ psi}$$

$$\epsilon_{50} \text{ (Strain at 50\% of the Maximum Principal Stress)} = 0.02 \text{ in/in}$$

$$K_h \text{ (Coefficient of horizontal subgrade reaction)} = 210 \text{ lb/in}^3$$

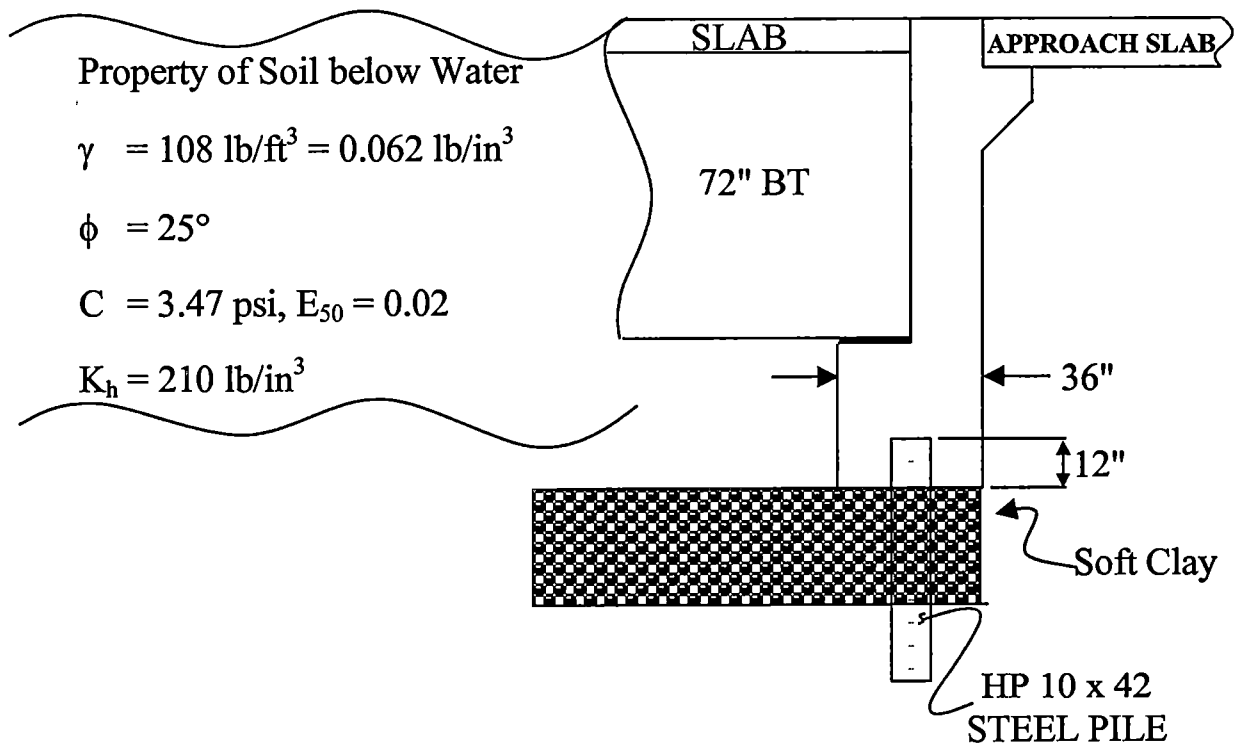


Figure A-1: Pile and soil cross section

Column Properties

$$Y_b = \frac{\sum Ay}{\sum A} = \frac{187.35}{37.47} = 5.0 \text{ in} = 0.42 \text{ ft}$$

$$I_o = \sum I + \sum Ay^2 = 1167.49$$

$$\begin{aligned}
 I &= I_o - \sum A(y_b)^2 \\
 &= 1167.49 - 37.47(5)^2 \\
 &= 1167.49 - 936.75 = 231 \text{ in}^4 = 0.011 \text{ ft}^4
 \end{aligned}$$

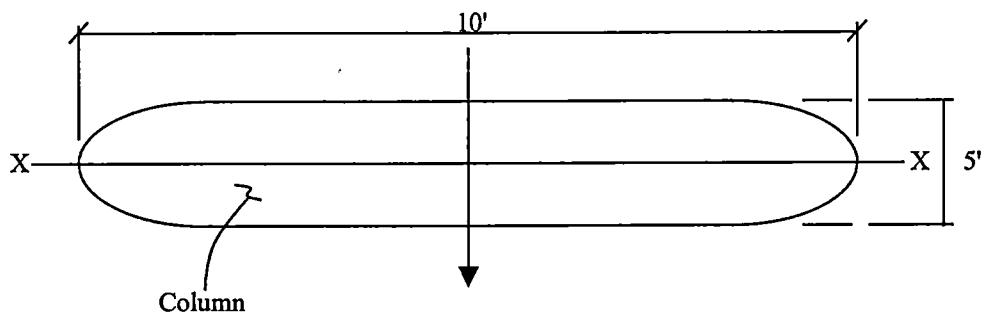


Figure A-2: Pier's column cross section

$$A = 6480 \text{ in}^2 = 45 \text{ ft}^2$$

$$I_x = 1,574,640 = 75.94 \text{ ft}^4$$

$$I_y = 7,776,000 = 375.00 \text{ ft}^4$$

$$E_C = 3321 \text{ psi}$$

Bridge Rail Properties

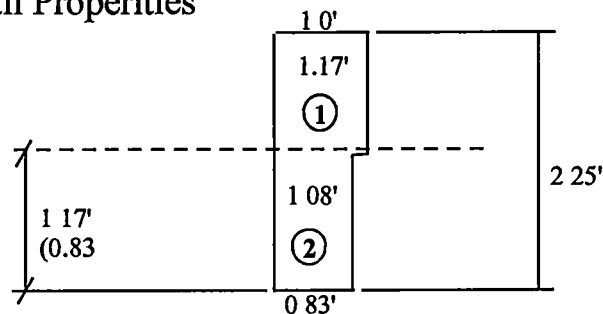


Figure A-3: Bridge handrail cross section

SECTION	A	y	Ay	Ay ²	I
1	1.17	1.66	1.94	3.22	0.133
2	<u>0.896</u>	0.54	<u>0.48</u>	<u>0.26</u>	<u>0.098</u>

2.07 2.42 3.48 0.220

$$Y_b = \frac{\sum Ay}{\sum A} = \frac{2.42}{2.07} = 1.17'$$

$$I_o = \sum I + \sum Ay^2 = 3.70$$

$$I = I_o - \sum A(y_b)^2 = 3.70 - 2.07(1.17^2) \\ = 3.70 - 2.83 = 0.87 FT^4$$

Equivalent Section ($n = E_{slab}/E_{beam} = 0.707$)

SECTION	A	y	Ay	Ay ²	I
1	0.83	1.17	0.97	1.14	0.05
2	<u>0.63</u>	0.38	<u>0.24</u>	<u>0.09</u>	<u>0.03</u>
	1.46		1.21	1.23	0.08

$$Y_b = \frac{\sum Ay}{\sum A} = \frac{1.21}{1.46} = 0.83'$$

$$I_o = \sum I + \sum Ay^2 = 0.08 + 1.23 = 1.31 FT^4$$

$$I = I_o - \sum A(y_b)^2 = 1.31 - 1.46(0.83^2) = 0.3 FT^4$$

Superstructure Properties

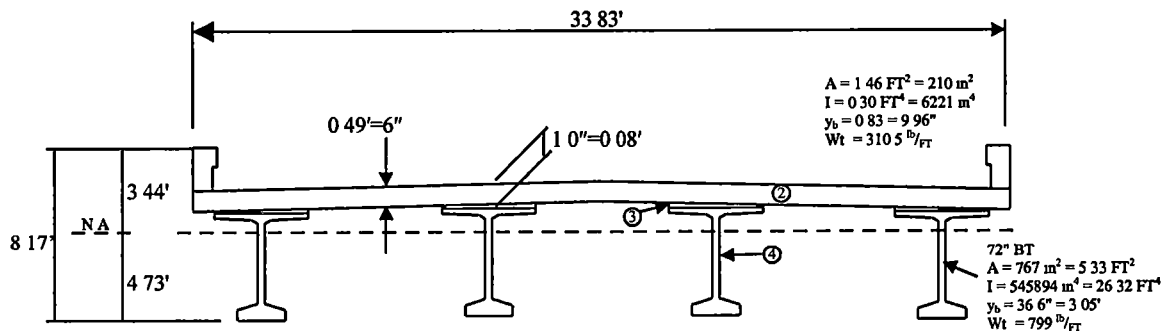


Figure A-4: Bridge superstructure cross section

$$w = 145 \text{ lb/ft}^3$$

$$E_{slab} = 33(w^{1.5})f_c^{1.5} = 3156 \text{ ksi}, f_c = 3000 \text{ psi}$$

$$E_{beam} = 33(w^{1.5})f_c^{1.5} = 4463 \text{ ksi}, f_c = 4000 \text{ psi}$$

$$n = E_{slab}/E_{beam} = .707$$

SECTION	A	y	Ay	Ay ²	I _o
1	2.92	7.41	21.64	160.33	0.3(2)=0.60
2	16.50	6.34	104.61	663.23	0.33
3	1.24	6.04	7.50	65.24	0.0002
4	<u>21.32</u>	<u>3.05</u>	<u>65.03</u>	<u>198.33</u>	<u>26.32(4)= 105.28 ft⁴</u>
	41.98		198.78	1067.13	106.21

$$y = \frac{\sum Ay}{\sum A} = \frac{198.78}{41.98} = 4.73' = 56.82''$$

$$I_o = \sum Ay^2 + \sum I = 1173.34 \text{ ft}^4$$

$$I = I_o - \sum A(y)^2 = 1173.34 - 41.98(4.73^2)$$

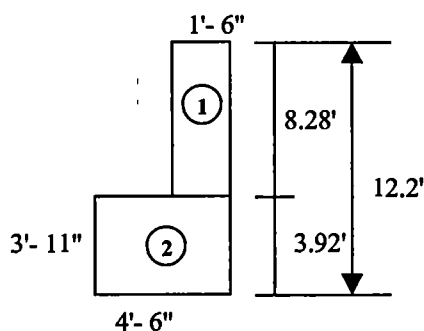
$$= 1173.34 - 939.21$$

$$= 234.13 \text{ ft}^4 = 4854919.68 \text{ in}^4$$

$$E_c = 4463 \frac{k}{\text{in}^2}$$

Abutment No.1 Properties

SECTION	A	y	Ay	Ay ²	I
1	18.30	6.10	111.63	680.94	226.98
2	<u>11.76</u>	1.96	<u>23.05</u>	<u>54.18</u>	<u>15.06</u>
	30.06		134.68	726.12	242.04



$$Y_{\text{Bot}} = \frac{\sum Ay}{\sum A} = \frac{134.68}{30.06} = 4.48 \text{ ft}$$

$$I_o = \sum Ay^2 + \sum I =$$

$$I = I_o + \sum A(y_B)^2$$

$$= 968.16 - 30.06(4.48)^2$$

Figure A-5: Abutment no. 1 cross section

$$= 969.16 - 603.32$$

$$= 364.84 \text{ ft}^4$$

$$= 7565322.24 \text{ in}^4$$

$$S = I / y = 81.44 \text{ ft}^3$$

$$A = 30.06 \text{ ft}^2$$

$$= 4328.64 \text{ in}^2$$

$$E_c = 3122 \text{ ksi} = 449568 \text{ k/ft}^2$$

$$\text{Wt.} = 30.06(0.15) = 4.51 \text{ k/ft}$$

Abutment No.2 Properties

Abutment Length = 35.00'

SECTION	A(in ²)	y(in)	Ay(in ³)	Ay ² (in ⁴)	I (in ⁴)
1	10.76	6.59	70.91	467.29	46.07
2	<u>9.0</u>	1.5	<u>13.5</u>	<u>20.25</u>	<u>6.75</u>
	19.76		84.41	487.54	52.82

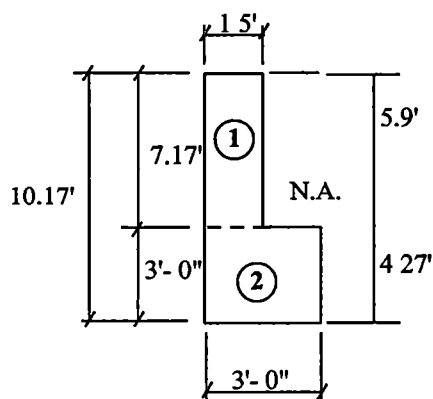


Figure A-6: Abutment no. 2 cross section

$$Y_b = \frac{\sum Ay}{\sum A} = 4.27'$$

$$I_o = \sum I + \sum Ay^2 = 540.36 \text{ ft}^4$$

$$I = I_o - \sum A(y_b)^2$$

$$= 540.36 - 19.76(4.27)^2$$

$$= 540.36 - 360.28 = 180 \text{ ft}^4$$

$$I_{X-X} = 180.0 \text{ ft}^4 = 3,732,480 \text{ in}^4$$

$$A=19.76 \text{ ft}^4=2845 \text{ in}^2$$

$$J=0.611 \text{ ft}^4$$

SECTION	A(in ²)	y(in)	Ay(in ³)	Ay ² (in ⁴)	I (in ⁴)
1	10.76	0.75	8.07	6.06	2.02
2	<u>9.0</u>	1.5	<u>13.5</u>	<u>20.25</u>	<u>6.75</u>
	19.76		21.57	26.31	8.77

$$Y_b = \frac{\sum Ay}{\sum A} = 1.09'$$

$$I_o = \sum I + \sum Ay^2 = 35.08 \text{ ft}^4$$

$$I = I_o - \sum A(y_b)^2$$

$$= 35.08 - 19.76(1.09)^2$$

$$= 35.08 - 23.48 = 11.6 \text{ ft}^4$$

$$I_{Y-Y} = 11.6 \text{ ft}^4 = 240,538 \text{ in}^4$$

$$A = 19.76 \text{ ft}^4 = 2845 \text{ in}^2$$

APPENDIX B

Input/output for COM624P

 PILE DEFLECTION, BENDING MOMENT, SHEAR & SOIL RESISTANCE

INPUT INFORMATION *****

THE LOADING IS CYCLIC
 NO. OF CYCLES = .10E-02

PILE GEOMETRY AND PROPERTIES

PILE LENGTH = 372.00 IN
 MODULUS OF ELASTICITY OF PILE = .290E+05 KIP/IN**2
 3 SECTION(S)

X	DIAMETER	MOMENT OF INERTIA	AREA
IN	IN	IN**4	IN**2
.00			
248.00	9.700	.210E+03	.124E+02
372.00	9.700	.210E+03	.124E+02
372.00	9.700	.210E+03	.124E+02

SOILS INFORMATION

X-COORDINATE AT THE GROUND SURFACE = .00 IN
 SLOPE ANGLE AT THE GROUND SURFACE = .00 DEG

2 LAYER(S) OF SOIL

LAYER 1

THE LAYER IS A SOFT CLAY

X AT THE TOP OF THE LAYER = 00 IN
 X AT THE BOTTOM OF THE LAYER = 248.00 IN
 VARIATION OF SOIL MODULUS, K = 210E+03 LBS/IN**3

LAYER 2

THE LAYER IS A SOFT CLAY

X AT THE TOP OF THE LAYER = 248 00 IN
 X AT THE BOTTOM OF THE LAYER = 400 00 IN
 VARIATION OF SOIL MODULUS, k = .210E+03 LBS/IN**3

DISTRIBUTION OF EFFECTIVE UNIT WEIGHT WITH DEPTH
4 POINTS

X, IN	WEIGHT, LBS/IN**3
.00	.61E-01
248 00	.61E-01
248 00	.61E-01
400 00	.61E-01

DISTRIBUTION OF STRENGTH PARAMETERS WITH DEPTH
4 POINTS

X, IN	C, LBS/IN**2	PHI, DEGREES	E50
.00	347E+01	25.000	200E-01
248 00	347E+01	25.000	200E-01
248 00	.347E+01	25 000	200E-01
400.00	347E+01	25 000	200E-01

FINITE DIFFERENCE PARAMETERS

NUMBER OF PILE INCREMENTS = 120
 TOLERANCE ON DETERMINATION OF DEFLECTIONS = .100E-04 IN
 MAXIMUM NUMBER OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
 MAXIMUM ALLOWABLE DEFLECTION = 15E+02 IN

INPUT CODES

OUTPT = 1
 KCYCL = 0
 KBC = 4
 KPYOP = 1
 INC = 2

SRI36/CANNEY FORA RIVER

UNITS--ENGL

O U T P U T I N F O R M A T I O N

GENERATED P-Y CURVES

THE NUMBER OF CURVE IS = 9
 THE NUMBER OF POINTS ON EACH CURVE = 17

DEPTH BELOW GS	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
10.00	9 700	.3E+01	6E-01	200E-01

Y, IN	P, LBS/IN
000	000
004	12 424
121	39 134
243	49 306
364	56 442
485	62 122
606	66 919
.727	71.112
849	74 861
970	78 269
1 091	81 403
1.213	84.313
1.334	87 034
1 455	89.595
3 880	56 477
7 275	10.306
9 700	10.306

DEPTH BELOW GS	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
50.00	9 700	3E+01	6E-01	.200E-01

Y, IN	P, LBS/IN
000	.000
004	21 731
121	68 449
.243	86.240
364	98 721
.485	108.656
606	117 046
727	124.380
.849	130.938
.970	136.898
1 091	142 380
1 213	147 469
1 334	152 229
1 455	156 709
3 880	128 826
7 275	90.131
9 700	90.131

DEPTH BELOW GS	DIAM	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
100 00	9 700	3E+01	6E-01	200E-01

Y, IN	P, LBS/IN
.000	.000
.004	30 293
121	95 417
.243	120.218
364	137 616
.485	151 466
.606	163 161

727	173 385
849	182 527
970	190 835
1 091	198 476
1 213	205 570
1 334	212 206
1.455	218.451
3 880	218 110
7 275	218 110
9 700	218.110

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
150.00	9.700	.3E+01	.6E-01	200E-01

Y, IN	P, LBS/IN
000	000
.004	30 293
121	95 417
243	120 218
364	137 616
485	151 466
606	163 161
.727	173 385
.849	182 527
970	190 835
1 091	198 476
1 213	205 570
1 334	212 206
1 455	218 451
3 880	218 110
7 275	218.110
9 700	218.110

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
200 00	9 700	3E+01	.6E-01	.200E-01

Y, IN	P, LBS/IN
.000	000
004	30 293
121	95 417
243	120 218
364	137 616
485	151.466
606	163 161
727	173 385
849	182 527
970	190 835
1 091	198 476
1 213	205 570
1.334	212 206
1 455	218 451
3 880	218 110
7.275	218 110
9 700	218 110

DEPTH BELOW GS	DIA	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
250.00	9.700	.3E+01	6E-01	200E-01

Y, IN	P, LBS/IN
.000	000
.004	30.293
.121	95.417
.243	120.218
.364	137.616
.485	151.466
.606	163.161
.727	173.385
.849	182.527
.970	190.835
1.091	198.476
1.213	205.570
1.334	212.206
1.455	218.451
3.880	218.110
7.275	218.110
9.700	218.110

DEPTH BELOW GS	DIA	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
300.00	9.700	.3E+01	6E-01	200E-01

Y, IN	P, LBS/IN
.000	000
.004	30.293
.121	95.417
.243	120.218
.364	137.616
.485	151.466
.606	163.161
.727	173.385
.849	182.527
.970	190.835
1.091	198.476
1.213	205.570
1.334	212.206
1.455	218.451
3.880	218.110
7.275	218.110
9.700	218.110

DEPTH BELOW GS	DIA	C	GAMMA	E50
IN	IN	LBS/IN**2	LBS/IN**3	
350.00	9.700	.3E+01	6E-01	200E-01

Y, IN	P, LBS/IN
.000	000
.004	30.293
.121	95.417
.243	120.218
.364	137.616
.485	151.466
.606	163.161

727	173 385
849	182 527
970	190.835
1.091	198.476
1 213	205.570
1 334	212.206
1 455	218 451
3 880	218 110
7 275	218 110
9.700	218 110

DEPTH BELOW GS IN	DIAM IN	C LBS/IN**2	GAMMA LBS/IN**3	E50
400.00	9 700	.3E+01	.6E-01	200E-01

Y, IN	P, LBS/IN
000	.000
004	30 293
121	95 417
243	120 218
364	137 616
.485	151.466
.606	163.161
.727	173 385
849	182 527
970	190 835
1 091	198 476
1 213	205 570
1.334	212.206
1 455	218 451
3 880	218 110
7.275	218.110
9.700	218 110

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PILE LOADING CONDITION

LATERAL DEFLECTION AT PILE HEAD	=	100E+01 IN
APPLIED MOMENT AT PILE HEAD	=	000E+00 IN-KIP
AXIAL LOAD AT PILE HEAD	=	780E+02 KIP

X	DEFLECTION IN	MOMENT IN-KIP	TOTAL STRESS LBS/IN**2	SHEAR KIP	SOIL RESIST LBS/IN	FLEXURAL RIGIDITY KIP-IN**2
00	100E+01	000E+00	629E+04	.971E+01	643E+02	609E+07
6 20	.935E+00	589E+02	765E+04	848E+01	718E+02	609E+07
12 40	871E+00	115E+03	895E+04	801E+01	789E+02	609E+07

18.60	808E+00	168E+03	102E+05	750E+01	855E+02	609E+07
24 80	746E+00	218E+03	113E+05	695E+01	.916E+02	609E+07
31 00	.685E+00	264E+03	124E+05	637E+01	971E+02	609E+07
37.20	625E+00	306E+03	134E+05	575E+01	102E+03	609E+07
43.40	.568E+00	.344E+03	142E+05	510E+01	106E+03	609E+07
49 60	513E+00	378E+03	150E+05	443F+01	.110E+03	.609E+07
55 80	460E+00	408E+03	.157E+05	374E+01	.113E+03	.609E+07
62.00	409E+00	.432E+03	.163E+05	.303E+01	116E+03	.609E+07
68 20	362E+00	453E+03	167E+05	230E+01	118E+03	609E+07
74 40	317E+00	468E+03	171E+05	157E+01	.119E+03	609E+07
80 60	275E+00	479E+03	173E+05	827E+00	119E+03	.609E+07
86 80	.237E+00	485E+03	.175E+05	871E-01	119E+03	609E+07
93 00	201E+00	486E+03	175E+05	- 633E+00	.113E+03	609E+07
99.20	169E+00	482E+03	174E+05	- 131E+01	106E+03	609E+07
105 40	139E+00	474E+03	172E+05	-.195E+01	.998E+02	609E+07
111 60	112E+00	462E+03	170E+05	- 255E+01	.930E+02	609E+07
117 80	897E-01	447E+03	166E+05	- 311E+01	860E+02	609E+07
124 00	679E-01	427E+03	.162E+05	- 362E-01	787E+02	609E+07
130 20	498E-01	.405E+03	156E+05	-.408E+01	709E+02	609E+07
136 40	342E-01	380E+03	151E+05	- 449E+01	.626E+02	609E+07
142.60	210E-01	352E+03	144E+05	- 485E+01	.532E+02	609E+07
148 80	101E-01	321E+03	137E+05	-.515E+01	416E+02	609E+07
155 00	115E-02	289E+03	130E+05	- 535E+01	202E+02	609E+07
161 20	- 595E-02	257E+03	122E+05	- 525E+01	- 349E+02	609E+07
167 40	-.114E-01	225E-03	115E-05	-.500E-01	- 434E-02	609E+07
173 60	- 155E-01	196E+03	108E+05	- 472E+01	- 481E-02	609E+07
179.80	- 183E-01	168E+03	102E-05	- 441E-01	- 508E+02	609E+07
186 00	- 201E-01	141E+03	955E+04	- 409E+01	- 524E+02	609E+07
192.20	- 210E-01	.117E+03	899E+04	- 376E+01	-.532E+02	.609E+07
198 40	- 211E-01	948E+02	848E+04	- 343E+01	- 533E+02	609E+07
204 60	-.206E-01	745E+02	801E+04	- 310E-01	- 529E+02	609E+07
210.80	- 197E-01	562E-02	759E+04	- 278E+01	-.521E+02	609E+07
217.00	- 184E-01	399E+02	721E+04	- 246E+01	- 509E+02	609E+07
223 20	- 168E-01	255E+02	688E+04	- 215E+01	- 494E+02	609E+07
229 40	-.151E-01	130E+02	.659E+04	-.185E+01	-.477E+02	609E+07
235.60	- 133E-01	230E+01	634E+04	-.156E+01	- 457E+02	.609E+07
241 80	- 115E-01	- 662E+01	644E+04	- 128E+01	- 436E+02	.609E+07
248 00	- 977E-02	- 139E-02	661E+04	- 102E+01	- 412E+02	609E+07
254 20	-.810E-02	-.195E+02	674E+04	- 769E+00	- 387E-02	609E+07
260 40	- 654E-02	- 237E-02	684E+04	- 537E-00	- 361E+02	609E+07
266 60	- 514E-02	- 264E-02	690E+04	- 322E+00	- 333E+02	609E+07
272 80	- 390E-02	- 279E+02	693E+04	-.125E+00	- 303E+02	609E+07
279 00	- 283E-02	- 282E+02	694E+04	537E-01	- 273E+02	609E+07
285 20	- 195E-02	- 274E+02	692E+04	213E+00	- 241E+02	609E+07
291 40	- 123E-02	- 257E+02	688E+04	352E+00	- 207E+02	609E+07
297 60	- 682E-03	- 232E+02	683E+04	.469E+00	-.170E+02	609E+07
303 80	- 276E-03	-.200E-02	675E+04	561E-00	- 125E+02	609E+07
310 00	297E-05	- 163E-02	667E+04	607E+00	306E+01	609E+07
316 20	.179E-03	- 127E-02	658E+04	559E+00	109E+02	609E+07
322 40	275E-03	- 946E+01	651E+04	486E+00	.126E+02	609E+07
328 60	311E-03	- 669E+01	644E+04	406E+00	131E+02	609E+07
334 80	305E-03	- 443E+01	639E+04	325E+00	130E+02	609E-07
341 00	271E-03	- 265E+01	635E+04	247E+00	125E-02	609E+07
347.20	.219E-03	- 136E+01	632E+04	172E+00	.116E+02	609E-07
353 40	159E-03	- 505E+00	630E+04	103E+00	.104E+02	609E+07
359 60	954E-04	- 521E-01	629E+04	435E-01	881E+01	609E+07
365 80	314E-04	642E-01	629E+04	-.341E-02	608E+01	609E-07
372.00	- 322E-04	000E-00	629E+04	000E-00	- 615E+01	609E+07

COMPUTED LATERAL FORCE AT PILE HEAD

= 88991E-01 KIP

NO OF ITERATIONS = 18
 MAXIMUM DEFLECTION ERROR = 812E-05 IN

COMPUTED LATERAL FORCE AT PILE HEAD = .88991E+01 KIP

NO. OF ITERATIONS = 2
 MAXIMUM DEFLECTION ERROR = 496E-05 IN

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PILE LOADING CONDITION

LATERAL DEFLECTION AT PILE HEAD = 100E+01 IN
 APPLIED MOMENT AT PILE HEAD = 000E+00 IN-KIP
 AXIAL LOAD AT PILE HEAD = .780E+02 KIP

X	DEFLECTION	MOMENT	TOTAL	SHEAR	SOIL	FLEXURAL
IN	IN	IN-KIP	STRESS LBS/IN**2	KIP	RESIST LBS/IN'	RIGIDITY KIP-IN**2
*****	*****	*****	*****	*****	*****	*****
00	100E+01	.000E+00	629E+04	.971E+01	643E+02	609E+07
6.20	.935E+00	.589E+02	.765E+04	848E+01	.718E+02	.609E+07
12.40	.871E+00	115E+03	895E+04	801E+01	789E+02	.609E+07
18.60	808E+00	168E+03	102E+05	750E+01	855E+02	.609E+07
24 80	746E+00	218E+03	113E+05	695E+01	916E+02	.609E+07
31 00	685E+00	264E+03	.124E+05	.637E+01	.971E+02	.609E+07
37 20	625E+00	306E+03	134E+05	575E+01	102E+03	609E+07
43 40	568E+00	.344E+03	.142E+05	510E+01	.106E+03	609E+07
49 60	513E+00	378E+03	150E+05	.443E+01	110E+03	609E+07
55 80	460E+00	.408E+03	157E+05	374E+01	113E+03	609E+07
62 00	409E+00	432E+03	163E+05	303E+01	.116E+03	609E+07
68 20	362E+00	453E+03	167E+05	230E+01	118E+03	.609E+07
74 40	317E+00	468E+03	.171E+05	157E+01	119E+03	609E+07
80 60	275E+00	479E+03	173E+05	827E+00	119E+03	609E+07
86 80	237E+00	.485E+03	.175E+05	.871E+01	.119E+03	.609E+07
93 00	201E+00	.486E+03	175E+05	- 633E+00	113E+03	609E+07
99 20	169E+00	482E+03	174E+05	-.131E+01	106E+03	.609E+07
105 40	139E+00	474E+03	172E+05	- 195E+01	999E+02	.609E+07
111 60	112E+00	462E+03	170E+05	- 255E+01	930E+02	609E+07
117 80	888E-01	447E+03	.166E+05	-.311E+01	.860E+02	.609E+07
124 00	679E-01	427E+03	162E+05	- 362E+01	787E+02	609E+07
130 20	.498E-01	405E+03	156E+05	- 408E+01	709E+02	.609E+07
136.40	342E-01	.380E+03	.151E+05	-.449E+01	626E+02	.609E+07
142 60	211E-01	352E+03	.144E+05	-.485E+01	.532E+02	.609E+07
148 80	101E-01	321E+03	137E+05	- 515E+01	.417E+02	609E+07

155.00	116E-02	289E+03	130E+05	- 535E+01	203E+02	609E+07
161.20	- 594E-02	257E+03	122E+05	- 525E+01	-.349E+02	609E+07
167.40	- 114E-01	225E-03	115E+05	- 500E+01	-.434E+02	.609E+07
173.60	- 155E-01	196E+03	108E+05	- 472E+01	- 481E+02	.609E+07
179.80	- 183E-01	168E+03	102E+05	-.441E+01	-.508E+02	.609E+07
186.00	- 201E-01	141E+03	.955E+04	- 409E+01	-.524E+02	609E+07
192.20	- 209E-01	117E+03	899E+04	- 376E+01	- 531E-02	.609E+07
198.40	-.211E-01	948E+02	.848E+04	-.343E+01	- 533E+02	609E+07
204.60	- 206E-01	745E+02	.801E+04	- 310E+01	-.529E+02	609E+07
210.80	- 197E-01	562E+02	.759E+04	- 278E+01	- 520E+02	609E+07
217.00	- 184E-01	399E+02	721E+04	- 246E+01	- 509E+02	.609E-07
223.20	- 168E-01	255E+02	688E+04	- 215E-01	-.494E+02	609E+07
229.40	- 151E-01	.130E+02	.659E+04	- 185E+01	- 477E+02	.609E+07
235.60	- 133E-01	231E+01	.634E+04	-.156E+01	- 457E+02	.609E+07
241.80	- 115E-01	- 662E+01	644E+04	- 128E+01	- 436E+02	609E+07
248.00	- 977E-02	-.139E+02	661E-04	- 102E+01	- 412E+02	609E+07
254.20	- 810E-02	- 195E+02	674E+04	- 769E+00	- 387E+02	609E+07
260.40	- 654E-02	- 237E-02	684E+04	- 537E+00	- 361E+02	609E+07
266.60	- 514E-02	- 264E-02	690E+04	- 322E+00	- 333E+02	609E+07
272.80	-.390E-02	- 279E+02	693E+04	- 125E+00	- 303E+02	609E-07
279.00	- 283E-02	- 282E+02	694E+04	538E-01	- 273E+02	609E-07
285.20	-.195E-02	- 274E+02	.692E+04	213E+00	-.241E+02	609E+07
291.40	- 123E-02	- 257E+02	.688E-04	352E+00	- 207E+02	.609E+07
297.60	- 683E-03	- 232E+02	683E-04	469E-00	- 170E+02	609E-07
303.80	- 277E-03	- 200E+02	675E+04	561E-00	- 126E+02	609E+07
310.00	207E-05	- 163E+02	667E+04	607E+00	258E+01	609E+07
316.20	178E-03	- 127E+02	658E+04	559E-00	109E+02	609E+07
322.40	274E-03	- 945E+01	651E-04	486E+00	125E-02	.609E+07
328.60	310E-03	- 669E-01	.644E+04	406E+00	131E+02	609E+07
334.80	304E-03	- 442E+01	639E+04	325E+00	.130E+02	609E+07
341.00	270E-03	- 265E+01	635E+04	246E+00	125E+02	.609E-07
347.20	219E-03	- 135E-01	632E+04	172E+00	116E-02	609E-07
353.40	159E-03	- 502E+00	630E-04	103E+00	104E+02	609E+07
359.60	953E-04	-.509E-01	629E+04	.434E-01	881E+01	609E+07
365.80	.315E-04	646E-01	629E+04	- 352E-02	.609E-01	609E+07
372.00	- 320E-04	000E-00	629E+04	000E+00	- 612E+01	.609E+07

COMPUTED LATERAL FORCE AT PILE HEAD = 88990E+01 KIP

NO OF ITERATIONS = 2
 MAXIMUM DEFLECTION ERROR = 306E-05 IN

S U M M A R Y T A B L E

LATERAL LOAD (KIP)	BOUNDARY CONDITION BC2	AXIAL LOAD (KIP)	YT (IN)	ST (IN/IN)	MAX MOMENT (IN-KIP)	MAX STRESS (LBS/IN**2)
890E+01	000E+00	780E+02	100E+01	- 104E-01	486E+03	175E+05
890E+01	000E+00	780E+02	100E-01	-.104E-01	486E+03	.175E+05
890E+01	000E+00	780E-02	100E+01	- 104E-01	.486E+03	.175E+05

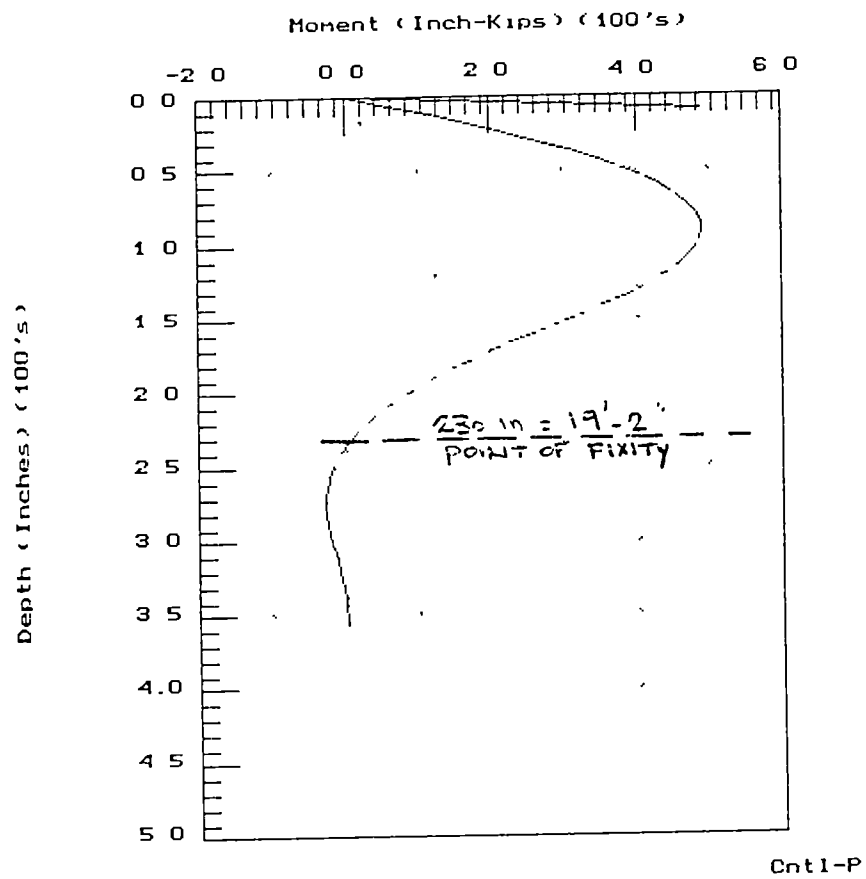


Figure B-1: Bending moment vs. depth

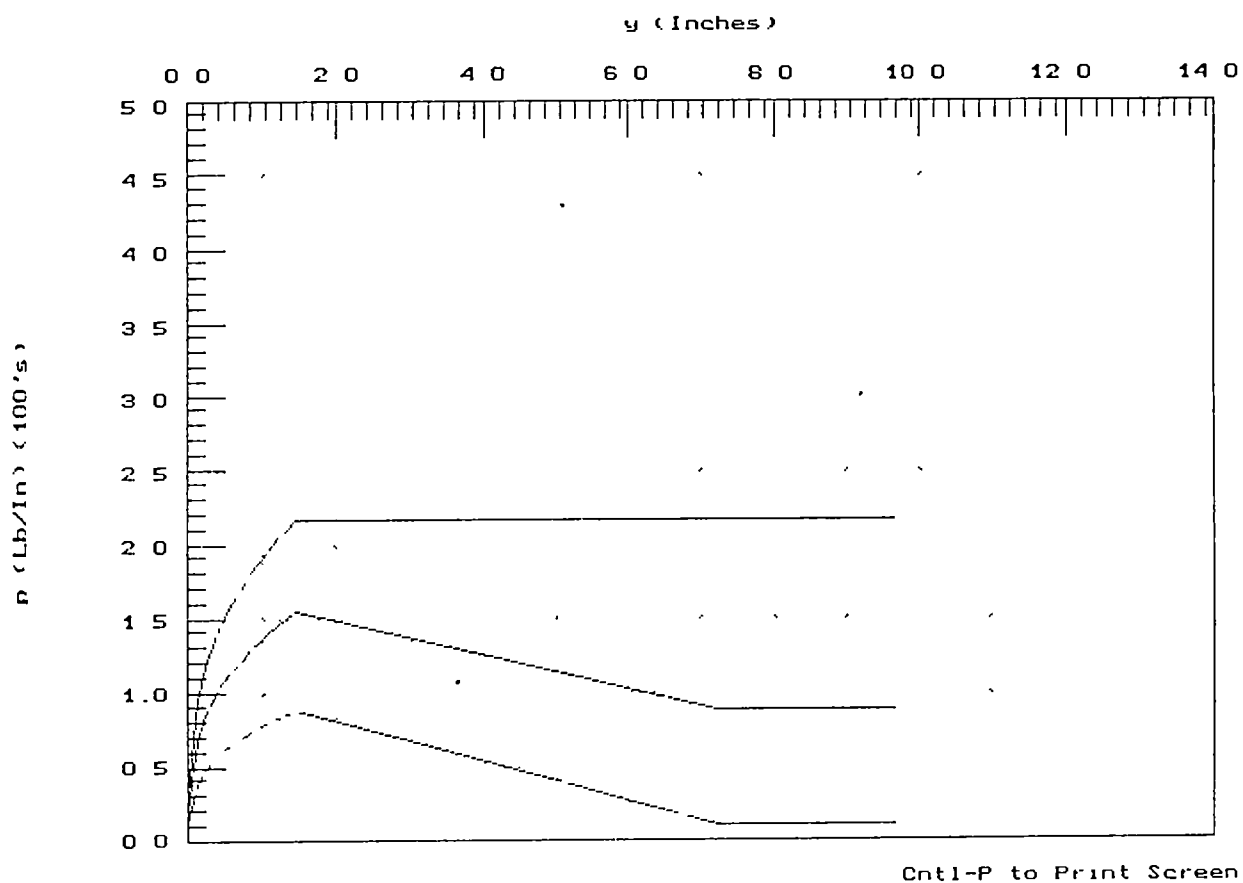


Figure B-2: Soil stress (p) vs. pile deflection (y) curves

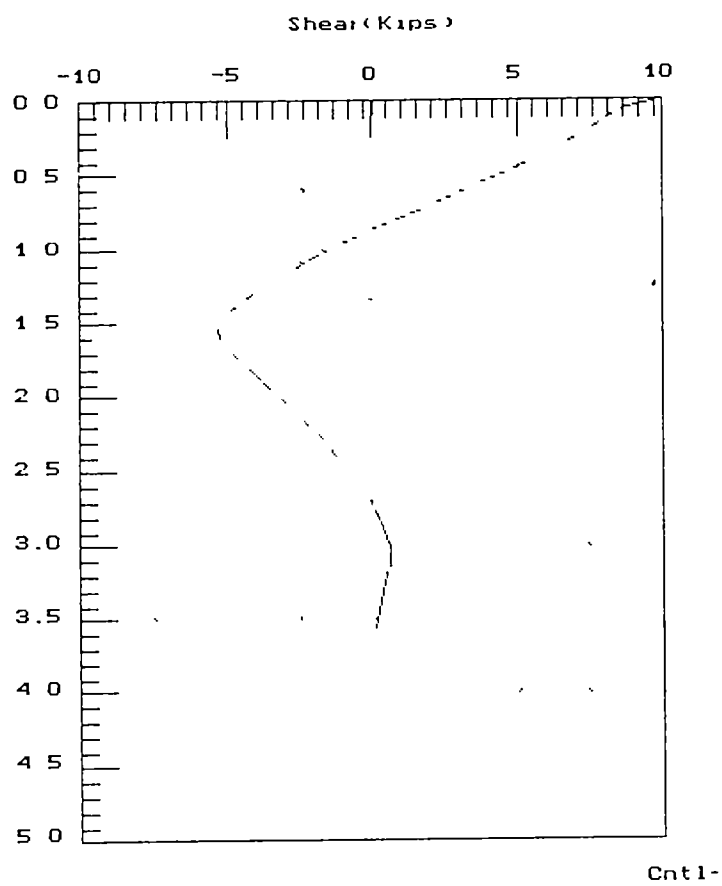


Figure B-3: Shearing force vs. depth

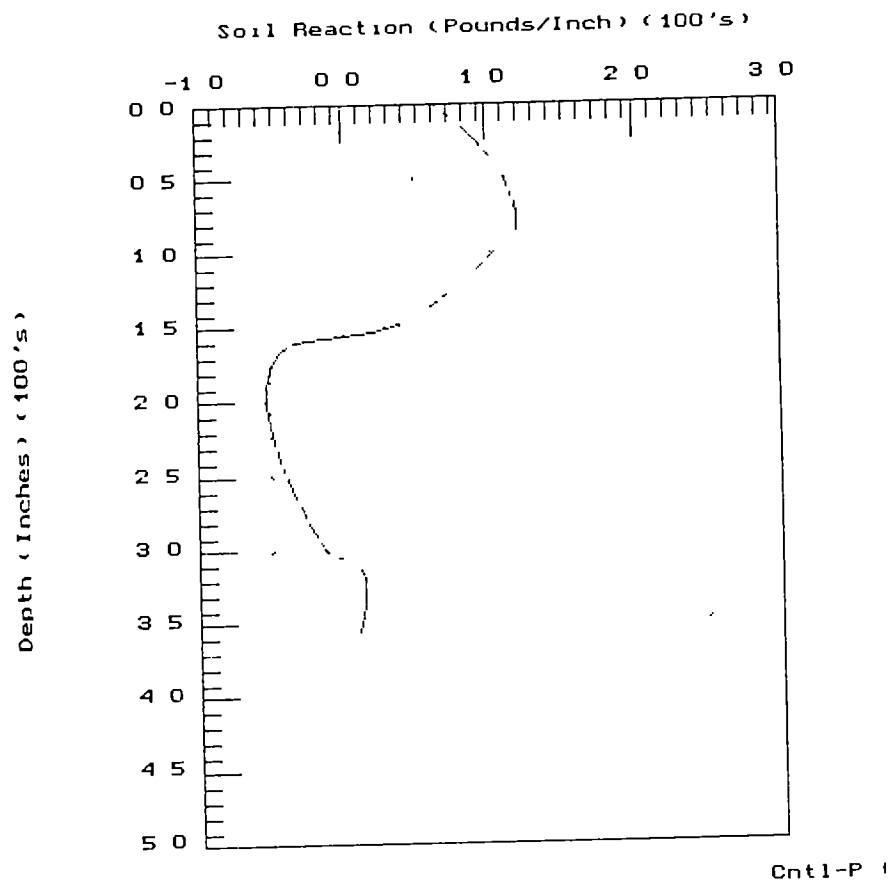


Figure B-4: Soil reaction vs. depth

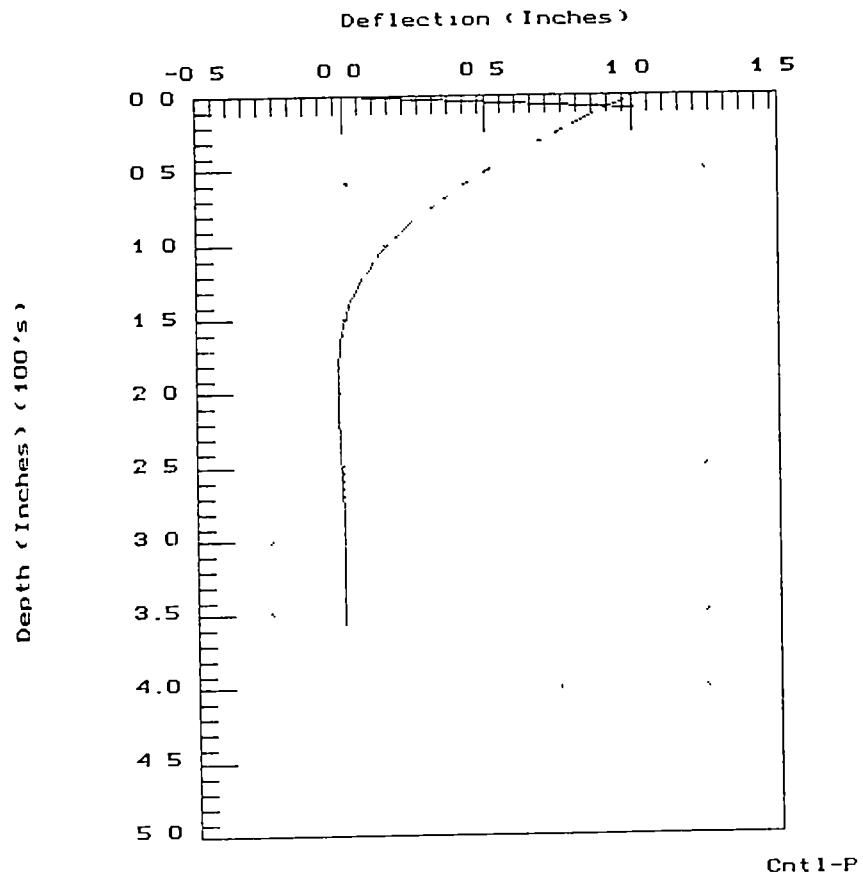


Figure B-5: Deflection vs. depth

APPENDIX C

Calculate Column Capacity (HP 10 x 42)

From COM624P output of both graph and tabulated data the following data have been obtained for each pile:

$$\text{Point of Fixity} = 230 \text{ in} \approx 19' - 2''$$

$$M_{\text{Max}} \text{ due to temperature, longitudinal and shrinkage(R,S,T)} = 486 \text{ k-in} \approx 40.50 \text{ k-ft}$$

$$f_{\text{Max}} = 17,500 \text{ lb/in}^2 \approx 2,520 \text{ lb/ft}^2$$

$$P_{\text{Axial}} = 78.00 \text{ k}$$

$$P_{\text{Lateral}} = 8.90 \text{ k}$$

$$\Delta_{\text{Max}} = 1.00 \text{ in}$$

$$\text{Total D.L. per Pile at Abutment No. 2} = 78.0 \text{ k}$$

$$\text{Total D.L. per Pile at Abutment No. 1} = 50.0 \text{ k}$$

$$\text{Total L.L. per Pile at Abutment No. 2} = 239.56 / 9 = 27.0 \text{ k}$$

$$\text{Total L.L. per Pile at Abutment No. 1} = 16.0 \text{ k}$$

Loadings due to temperature movement (1.0 in of movement for soft clay) the maximum positive moment between inflection points is 40.5 k-ft.

From AASHTO Specifications Table 3.22.1A

$$\begin{aligned} \text{Group I} &= 1.3 (\text{DL} + 1.67\text{LL}) \\ \text{Group IV} &= 1.3 (\text{DL} + \text{LL} + (\text{R}+\text{S}+\text{T})) \\ \text{Group V} &= 1.25 (\text{D} + (\text{R}+\text{S}+\text{T})) \\ \text{Group VI} &= 1.25 (\text{DL} + \text{LL} (\text{R}+\text{S}+\text{T})) \end{aligned}$$

$$\begin{aligned}
 \text{DL} &= 78.00 \text{ k} \\
 \text{LL} &= 27.00 \text{ k} \\
 \text{M} &= 40.50 \text{ k-ft (Due to R,S \& T)}
 \end{aligned}$$

Group	M (k-ft)	P (k)
I	0	160.0
IV	53.0	136.0
V	51.0	98.0
VI	51.0	132.0

Table C-1: Group Loading

Calculate Column Capacity (Cont'd)

$$\begin{aligned}
 L_c &= 19.17 \text{ ft} \\
 A_s &= 12.40 \text{ in}^2 \\
 F_y &= 36.0 \text{ ksi} \\
 S_x &= 43.40 \text{ in}^3 \\
 Z_x &= 48.30 \text{ in}^3 \\
 b_f &= 10.075 \text{ in} \\
 r_x &= 4.13 \text{ in} \\
 E_s &= 29000 \text{ ksi} \\
 k &= 0.875 \quad \text{For pinned ends (AASHTO 10.54.1.2)} \\
 I &= 210 \text{ in}^4
 \end{aligned}$$

From AASHTO 10.54 Compression members

$$kL_c/r = 0.875(19.17)12/4.13 = 48.74$$

$$\sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2(\pi)^2 29000}{36}} = 126.10$$

For

$$kL_c/L \leq \sqrt{\frac{2(\pi)^2 29000}{F_y}}$$

$$\begin{aligned} F_{cr} &= F_y \left[1 - \left(\frac{F_y}{4\pi^2 E} \right) \left(\frac{kL_c}{r} \right)^2 \right] \\ &= 36 \left[1 - \left(\frac{36}{4(3.14)^2 (29000)} \right) (48.74)^2 \right] \\ &= 36(1 - 0.075) = 33.30 \text{ ksi} \end{aligned}$$

P_U = Max. strength of concentrically loaded column

$$P_U = 0.85 A_s F_{cr} = 0.85(12.4)33.30 = 350.98 \text{ k}$$

$$M_P = F_y Z_x = 36(48.3) = 1738.8 \text{ k-in} = 144.9 \text{ k-ft}$$

$$M_U = F_y S_x = 36(43.4) = 1562.4 \text{ k-in} = 130.2 \text{ k-ft}$$

Combined Axial Load and Bending (AASHTO 10.54.2)

Maximum Capacity:

$$\frac{P}{0.85 A_s F_{cr}} + \frac{MC_m}{M\mu \left(1 - \frac{P}{A_s F_e} \right)} \leq 10 \longrightarrow \left[\begin{array}{c} \text{Equation} \\ 10-155 \\ \text{AASHTO} \end{array} \right]$$

$$\frac{P}{0.85A_sF_y} + \frac{M}{M_p} \leq 1.0 \longrightarrow \left[\begin{array}{c} \text{Equation} \\ 10-156 \\ \text{AASHTO} \end{array} \right]$$

$$F_e = \frac{E\pi^2}{\left(\frac{kL_c}{r}\right)^2} = \text{The Euler buckling stress in the plane of bending}$$

$$= \frac{29000(314)^2}{(48.74)^2} = 120.36 \text{ k}$$

$C_m = \text{Equivalent Moment Factor} = 0.6 + 0.4a \geq .4$ (AASHTO 10.54.2.2)

where $a = M_1/M_2 = 0$

Since bending is assumed to occur between inflection points

Then $a = 0$ and $C_m = 1.0$ (Because end moments are equal to zero)

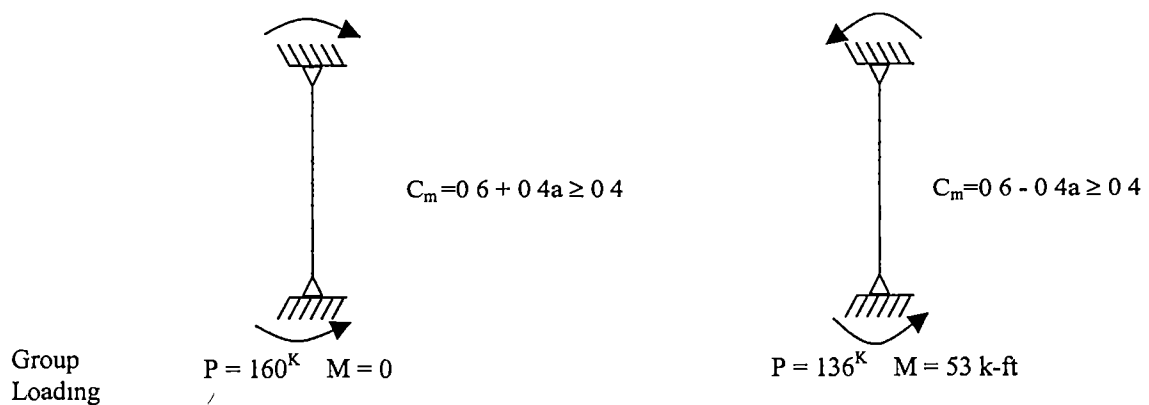


Figure C-1: C_m factor

From equation 10-155 (AASHTO)

$$\frac{160}{0.85(12.4)33.30} + \frac{40.50(1.0)}{130.2 \left(1 - \frac{160}{12.4(120.36)} \right)} = 0.46 + 0.35 = 0.81 \leq 1.0 \quad \underline{\underline{O.K}}$$

$$\frac{136}{0.85(12.4)33.30} + \frac{53(1.0)}{130.2 \left(1 - \frac{136}{12.4(120.36)} \right)} = 0.39 + 0.45 = 0.84 \leq 1.0 \quad \underline{\underline{O.K}}$$

From equation 10-156

$$\frac{160}{0.85(12.4)36} + 0 = 0.42 \leq 1.0 \quad \underline{\underline{O.K}}$$

$$\frac{136}{0.85(12.4)36} + \frac{53}{144.9} = 0.36 + 0.37 = 0.73 \leq 1.0 \quad \underline{\underline{O.K}}$$

See Figure C-2 and Table C-2 for output summary of the column iteration program.

STEEL COMPRESSION MEMBERS

LOAD FACTOR DESIGN

SEE AASHTO ART 10 54

REQUIRED DATA

Fy =	36000	PSI
As =	12.4	in ²
K =	0.875	
Lc =	19.2	Ft
S =	43.4	in ³
r =	4.13	in

$E = 29,000,000 \text{ PSI}$

SECTION HP10X42

X-X AXIS

References

DETERMINE F_{cr} . (EQNS 10-151 and 10-152)

$$K L_c / r = 48.74 \text{ IN}$$
$$\text{SQRT} (2 * \text{PI}^2 * \text{E} / \text{FY}) = 126.10$$

$F_{cr} = 33.311 \text{ 13 PSI}$ (Eqn 151 Controls)

$$P_{11} = 85 * A_s * F_{cr} = 351.099 \text{ 30 LB} \quad (\text{Eqn 150})$$
$$= 351.1 \text{ K}$$

COMBINED AXIAL LOAD AND BENDING

REQUIRED DATA

Mu = 130 2 K-FT = 1,562,400 IN-LB
C = 1 (conservatively taken as unity)
Z = 48 3 in^3

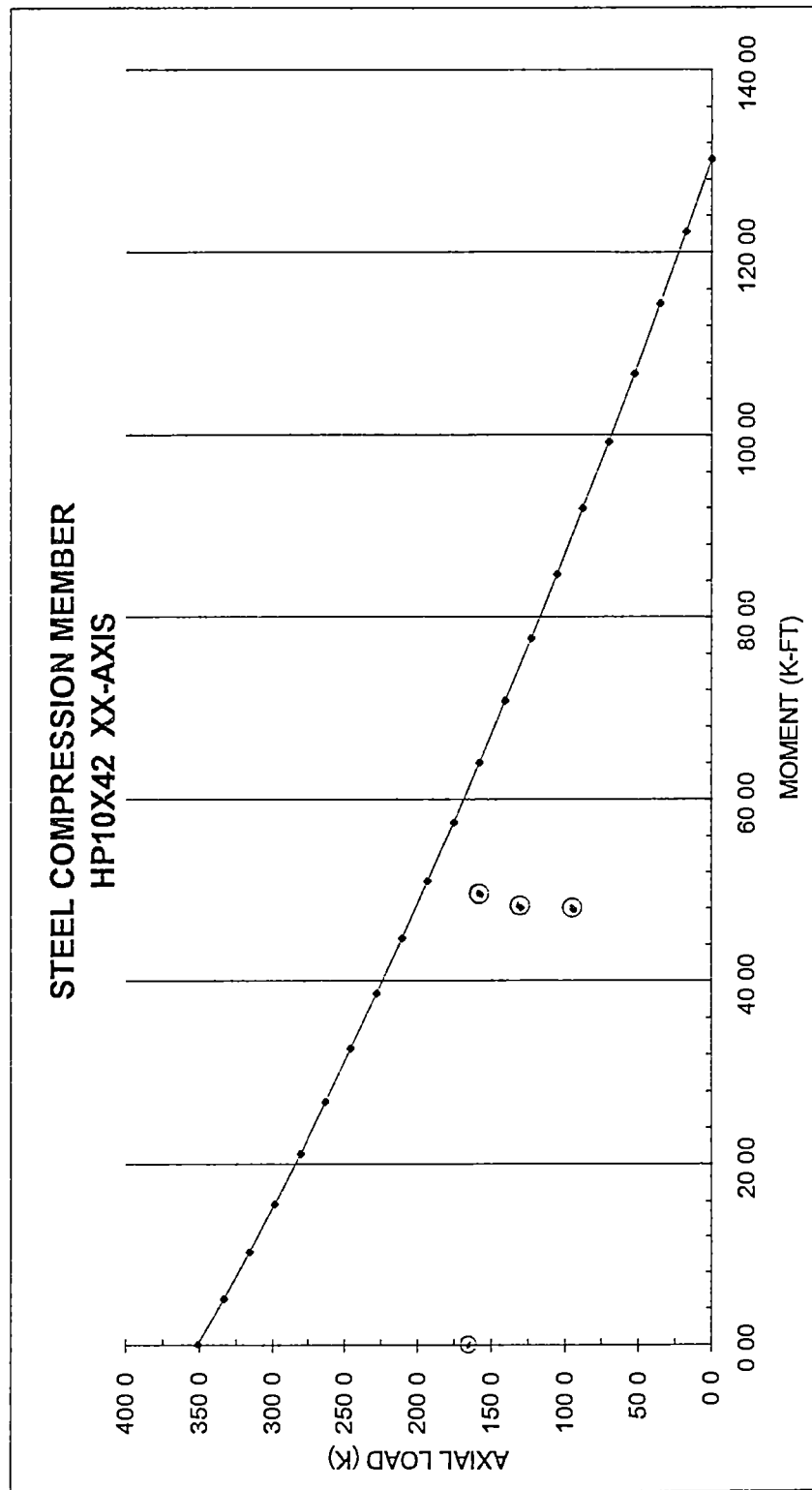
$$F_e = 120,496.66 \text{ PSI} \quad (\text{Eqn 157})$$

$M_p = F_Y \cdot Z = 1,738,800 \quad \text{IN-LB} = 144.9 \quad \text{K-FT}$

INTERACTION DATA POINTS					
P (K)	P (LB)	M EQN -155	M EQN -156	M MINIMUM	M (K-FT)
351 1	351,099 3	0 0	129,872 5	0 0	0 00
333 5	333,544 3	60,681 1	210,318 9	60,681 1	5 06
316 0	315,989 4	123,197 9	290,765 3	123,197 9	10 27
298 4	298,434 4	187,550 4	371,211 6	187,550 4	15 63
280 9	280,879 4	253,738 5	451,658 0	253,738 5	21 14
263 3	263,324 5	321,762 3	532,104 4	321,762 3	26 81
245 8	245,769 5	391,621 7	612,550 8	391,621 7	32 64
228 2	228,214 5	463,316 9	692,997 2	463,316 9	38 61
210 7	210,659 6	536,847 7	773,443 5	536,847 7	44 74
193 1	193,104 6	612,214 2	853,889 9	612,214 2	51 02
175 5	175,549 6	689,416 4	934,336 3	689,416 4	57 45
158 0	157,994 7	768,454 2	1,014,782 7	768,454 2	64 04
140 4	140,439 7	849,327 7	1,095,229 0	849,327 7	70 78
122 9	122,884 7	932,036 9	1,175,675 4	932,036 9	77 67
105 3	105,329 8	1,016,581 8	1,256,121 8	1,016,581 8	84 72
87 8	87,774 8	1,102,962 3	1,336,568 2	1,102,962 3	91 91
70 2	70,219 8	1,191,178 5	1,417,014 5	1,191,178 5	99 26
52 7	52,664 9	1,281,230 4	1,497,460 9	1,281,230 4	106 77
35 1	35,109 9	1,373,117 9	1,577,907 3	1,373,117 9	114 43
17 6	17,555 0	1,466,841 2	1,658,353 7	1,466,841 2	122 24
0 0	0 0	1,562,400 0	1,738,800 0	1,562,400 0	130 20

Table C-2: Pile interaction output

STEEL COLUMN INTERACTION DIAGRAM
LOAD FACTOR ANALYSIS
HP 10 X 42 PILE X-X AXIS



UNBRACED LENGTH = 19.17 FT.

Figure C-2: Pile interaction diagram

Vita

I was born in Glendale California in August of 1960. I spent the first few years of my life in Kuwait city, and the rest between California and Tennessee.

I finished my bachelor degree in civil engineering at the University of California and I am currently working on my master degree at the University of Tennessee in Knoxville. Currently, I am currently employed with the Tennessee Department of Transportation. I hope to obtain my PhD in Civil Engineering in the near future.