LA-UR-95-3191

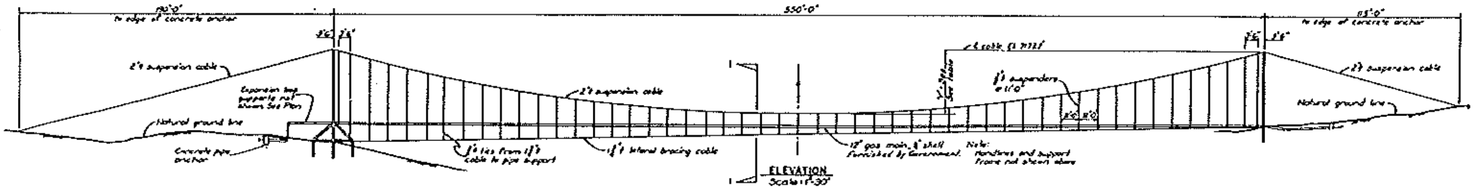
Infrastructure Support Facilities (ISF) Gas Line - Los Alamos Townsite:

The "Peggy Sue" Bridge

Historic Building Survey Report

Los Alamos National Laboratory
Cultural Resource Management Team
Report No. 116

September 2, 1995
Survey No. 663



prepared by

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ABSTRACT

In August 1995, a historic building survey of the "Peggy Sue" Bridge was conducted at Technical Area (TA) 0, Los Alamos National Laboratory (LANL), New Mexico. This is the location for a proposed LANL demolition project involving a gas line suspension bridge built in 1949 or 1950.

Based on the information gathered during this historical survey, the Peggy Sue Bridge, Laboratory of Anthropology (LA) site number 110645, is not eligible for nomination to the National Register of Historic Places. As a result of this survey, this project complies with the National Historic Preservation Act of 1966 (as amended) and with Executive Order 11593. The New Mexico State Historic Preservation Officer is requested to concur in a "Determination of No Effect".

PROVENIENCE AND ENVIRONMENTAL SETTING

Location: TA-0 (Townsite), Los Alamos National Laboratory (LANL)

Land Owner: Los Alamos County

Bridge/Gas Line Manager: Department of Energy (DOE)

Legal Description: Township 19 North
Range 6 East
N ½ of the SE ¼ of the SE ¼ of Section 9

Map: USGS Guaje Mountain 7.5 Minute Series (Map 1)

Topography: The bridge is located on portions of the main Los Alamos "Townsite" Mesa and is suspended over "Acid Canyon" (a branch of Pueblo Canyon).

Nearest Drainage: "Acid" and Pueblo Canyons

Elevation: 2170.2 meters (7120 feet)

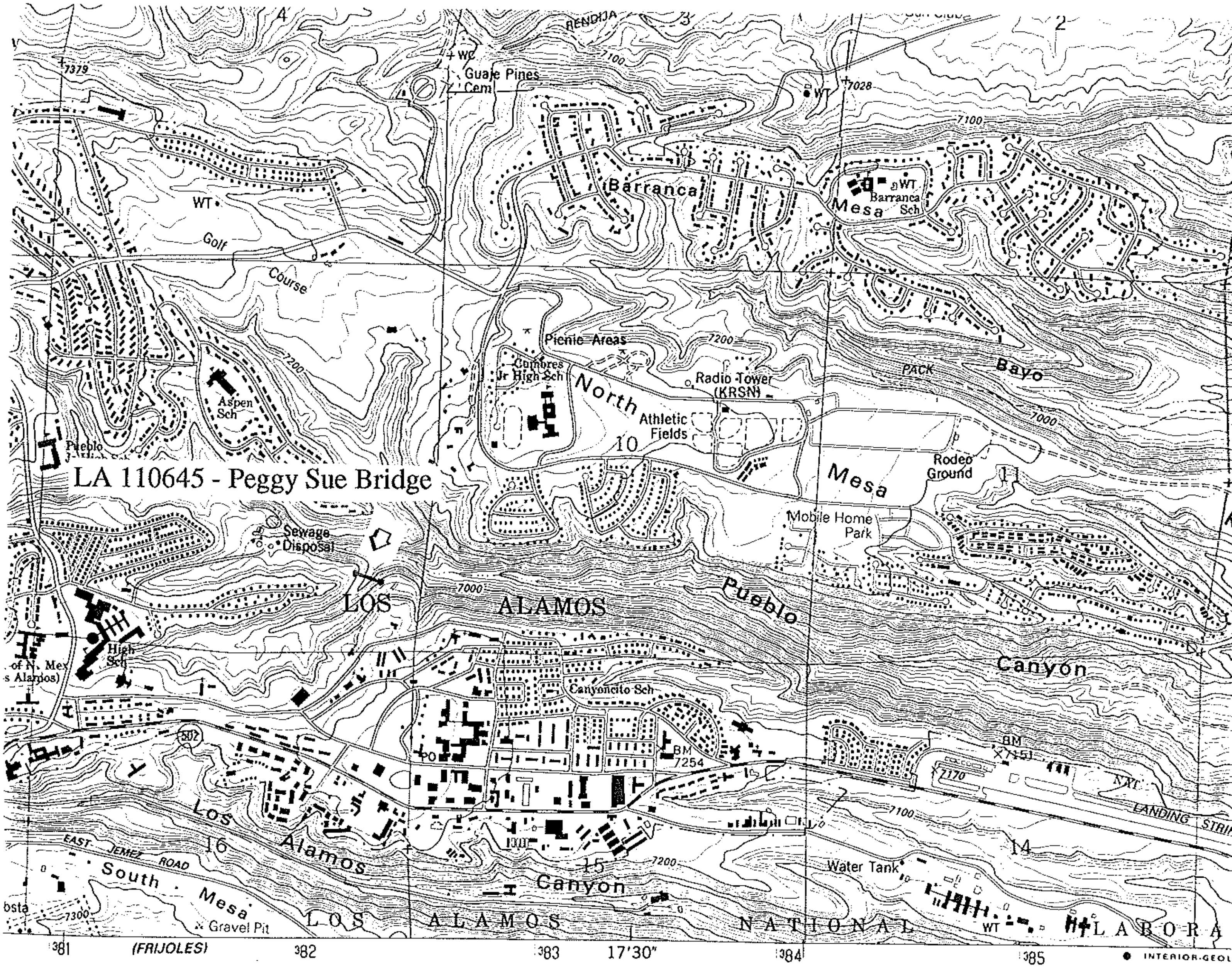
Current Land Use: This area of Los Alamos County is characterized by undeveloped steep canyon terrain. The Orange Street residential area is located to the west of the bridge and the Los Alamos Jewish Center (and downtown Los Alamos) is located to the south of the bridge. Several county trails are located in the "Acid" Canyon vicinity.

PROJECT DESCRIPTION

In August 1995, a historic building evaluation of the Peggy Sue Bridge was conducted by Ellen D. McGehee, Archaeologist, Environmental Assessments and Resource Evaluations Group (ESH-20), Los Alamos National Laboratory (LANL). This evaluation was conducted prior to a proposed LANL demolition project involving the pipeline suspension bridge associated with the 12-inch Infrastructure Support Facilities (ISF) gas line. Demolition project activities include the removal of the steel bridge, attached gas line piping, and associated anchors and cables. A previous cultural resource survey report documented the entire ISF gas line replacement project (Manz *et al.* 1993). The proposed demolition activities will take place on county land near downtown Los Alamos, New Mexico (Map 1). Access to the project area will be by existing paved and dirt roads.

SURVEY METHODOLOGY

This historic building evaluation was accomplished by first conducting a field visit to the bridge location. The bridge was recorded on a New Mexico Historic Building Inventory Form (Appendix A). Photographs were taken and are included in this report (Appendix B). Records research at Los Alamos County and at LANL was also carried



USGS 7.5 Guaje Mountain Quad
Scale 1:24000

●━━● = Bridge

MAP 1

N ⬆

ISF GAS LINE - LOS ALAMOS TOWNSITE
PEGGY SUE BRIDGE LOCATION

out. Bridge structural information was obtained and is included in Appendix C of this report. The bridge's designer, Black and Veatch, was consulted in order to assess the architectural significance of the bridge's design. Historical information was also acquired from the Los Alamos Historical Museum's archives. Several offices of the Gas Company of New Mexico and the Roswell office of the Transwestern Pipeline Company were contacted in order to identify similarly designed utility suspension bridges in New Mexico.

HISTORICAL BACKGROUND

Prehistoric land use on the Pajarito Plateau is characterized by Paleo-Indian and Archaic Period hunting and resource exploitation from about 10,000 B.C. to A.D. 600. A more formal Anasazi settlement of the Plateau occurred from A.D. 1100 until A.D. 1600, ending about the time of the arrival of the Spanish.

Historic land use on the Plateau begins during the Homesteading Period, from about 1890 to 1943, and was an outgrowth of earlier, undocumented, seasonal resource exploitation of the Plateau by Hispanics and Euro-Americans from neighboring communities.

In 1942, President Franklin Delano Roosevelt gave the official approval to develop the world's first atomic bomb. Brigadier General Leslie Groves was given complete military authority for the project. This project came to be known as Project Y, a subset of the Manhattan Project. Groves, in turn, chose J. Robert Oppenheimer to coordinate the design of the bomb. Because of its isolated location, Los Alamos was selected as the site of the bomb's design and construction. Project Y became a success when the world's first atomic device was detonated near Alamagordo on July 16, 1945. After the subsequent bombings of Hiroshima and Nagasaki in August of 1945, the end of WWII came fairly quickly (LANL 1994).

The Peggy Sue Bridge was designed in 1949 and constructed in 1949 or 1950. This was an important time period in the post-war history of Los Alamos. The Manhattan Project had come to a close with the end of WWII and many of the Los Alamos scientists and site workers had gone back to their pre-war existences. The future of Los Alamos was in question (LANL 1993).

In 1946, the U.S. Atomic Energy Commission (AEC) was established to act as a civilian steward for the new atomic technology born of WWII. With the beginning of the Cold War, continued weapons research was a top priority. In 1947, the AEC had formally taken over Los Alamos and had made a commitment to revitalize both the laboratory and the town. Once the decision was made to retain Los Alamos as a weapons laboratory, a new permanent facility had to be built. Although some of the earlier scientific facilities were kept operating, many buildings were torn down and new outlying facilities were constructed. This construction boom extended to the townsite as well. A post office, schools, central downtown area, library, medical center, perimeter housing areas, and associated infrastructure support facilities, like the Peggy Sue Bridge, were all built during the late 1940s to mid 1950s (LANL 1993).

STRUCTURE DESCRIPTION

Laboratory of Anthropology (LA) 110645

The Peggy Sue Bridge, LA 110645, was designed by Black and Veatch of Kansas City, Missouri in 1949. The bridge was constructed in 1949 or 1950 by Morrison Construction Company of Austin, Texas (Kesler 1994). The Peggy Sue Bridge is a steel utility suspension bridge that supports a section of 12-inch gas line across "Acid" Canyon, a branch of Pueblo Canyon. The bridge is a cable stayed, open spandrel arch design and is approximately 550 feet long with a 3-ft wide bridge deck. The main span is suspended from two 2-inch diameter cables which are supported by steel towers on either side of the canyon. The cables are anchored into large concrete blocks (White and Maggard 1990). The towers are built out of 12 ¾-inch diameter steel pipe that has a thickness of approximately ½ inch (Black and Veatch 1949). The east tower is 55 feet tall and the west tower is 43 feet tall. According to a 1990 LANL inspection report, the anchor blocks show signs of long-term freeze-thaw damage. The bridge has loose cables and is missing some turn-buckles. Rusting is also evident on the deck and the support towers (White and Maggard 1990). Black and Veatch provided a copy of the original bridge design specifications to the Los Alamos County Public Works Department. These specifications are included in Appendix C along with copies of several LANL bridge inspection reports. Los Alamos County recently commissioned a feasibility report for the remodeling of the Peggy Sue Bridge, from its existing function as a gas line bridge to a pedestrian and bicycle bridge (Gordan and Associates 1994). While this report is not included in the appendices, it is on file at the Los Alamos County Public Works Department.

Research involving several offices of the Gas Company of New Mexico was conducted in order to identify other similarly designed utility suspension bridges in the state of New Mexico. Based on information provided by Gas Company of New Mexico personnel, three bridges were identified: the "Little Peggy Sue" effluent bridge in Los Alamos County; the Gas Company of New Mexico's suspension bridge across the San Juan River near Bloomfield, New Mexico; and the Transwestern Pipeline Company's suspension bridge across the Rio Grande near Berino, New Mexico. Owing to travel constraints, the two bridges outside of Los Alamos County were not visited, making it difficult to verify the degree of similarity between these two bridges and the Peggy Sue Bridge.

The "Little Peggy Sue" Bridge

The Little Peggy Sue Bridge is located to the west of the Peggy Sue Bridge and spans upper Pueblo Canyon, just north of an abandoned wastewater treatment plant. This bridge, although shorter than the Peggy Sue Bridge, was also designed by Black and Veatch. The AEC built the bridge across Pueblo Canyon in 1950 in order to support an 8-inch effluent water pipeline and a 10-inch sewer influent pipeline. The Little Peggy Sue Bridge is approximately 300 feet long from tower to tower and has a total width of 4 feet. The decking is approximately 1 ft 3 inches wide and three pipelines are located on either side of the decking (an additional 6-inch gas line was added in 1970 and is located along the eastern side of the bridge near the 8-inch effluent pipe). This walkway area is different than the Peggy Sue Bridge's walkway because the pipeline supported by the Peggy Sue Bridge is located underneath the decking. The Little Peggy Sue Bridge's north tower is approximately 38 feet above subgrade and the south tower is slightly taller, at 39.5 feet. The steel pipes used to construct the tower have an outside diameter of 10 ¾ inches and are ½ inch thick. The main vertical support cables are 1 ½ inches in diameter and, like the Peggy Sue Bridge, are connected to concrete anchors at each end of the bridge (Frank Henri and Associates 1985). Photographs and drawings of the Little Peggy Sue Bridge are included in Appendix D.

LOCAL FOLKLORE

The Peggy Sue Bridge is a part of the folklore of Los Alamos. Several local myths are associated with the bridge. The most commonly repeated story concerns a girl named Peggy Sue who purportedly jumped or fell off the bridge to her death. In 1981, the Los Alamos Historical Museum collected some of the Peggy Sue Bridge stories for the museum's archives (Hunn 1981). According to this research, the name "Peggy Sue" was well associated with the bridge in the 1950's and school children from that era were familiar with the Peggy Sue Bridge stories. Several variations exist: she fell off trying to catch a falling bicycle, falling homework papers, or a falling dog. It is interesting to note that the Los Alamos Police Department had, up until 1981, no record of anyone, Peggy Sue or otherwise, who had jumped off the bridge (Hunn 1981). A recent article in the Los Alamos Monitor (1995) does, however, allude to a suicide connected with the bridge. Some associate the bridge's name with Peggy Church, an early resident of Los Alamos and the daughter of Ashley Pond. A film using the "Peggy Sue" story was made in 1978 by a Cumbres Jr. High (now Los Alamos Middle School) class. The film is supposed to be a melodrama and involves a landlord's thug throwing Peggy Sue off the bridge (Hunn 1981). A copy of this film could not be located; however, a recent documentary of the bridge is being produced for the local Los Alamos public access channel (PAC-8).

NATIONAL REGISTER ELIGIBILITY DETERMINATION

Since the bridge is not yet fifty years old, the four National Register Criteria of Eligibility can only be applied in conjunction with Criteria Consideration G, "[exceptionally important] properties that have achieved significance within the last fifty years" (U.S. Department of the Interior 1991:41).

Criterion A, "properties ... associated with events that have made a significant contribution to the broad patterns of our history" (U.S. Department of the Interior 1991:12).

While the Peggy Sue Bridge was built during the early Cold War era in Los Alamos and was a product of the AEC's revitalization of the Laboratory, the activities associated with the bridge were not of exceptional historical importance. The bridge's primary purpose was to serve as infrastructure support. The Peggy Sue Bridge, since its original construction in 1950, has been a utility structure associated with the 12-inch ISF gas line. This gas line provides natural gas to the Laboratory and to the town of Los Alamos.

The secondary use of the bridge, as an unauthorized pedestrian bridge, is of interest to the folk culture of the community but does not play an exceptionally significant role in local history.

Criterion B, "properties ... associated with the lives of persons significant in our past" (U.S. Department of the Interior 1991:14).

The bridge is not associated with the life of any historically significant person.

Criterion C, "properties ... [that] embody the distinctive characteristics of a type, period, or method of construction, or that represent the work of a master, or that possess high artistic values, or that represent a significant and distinguishable entity whose components may lack individual distinction" (U.S. Department of the Interior 1991:17).

According to Black and Veatch, the designer of the bridge, the construction of the Peggy Sue Bridge was not an architectural feat. The design is a fairly common one for the time (it was based on an existing design for a road bridge) and, as constructed, the bridge's overall dimensions are not exceptional.

Criterion D, "properties ... [that] have yielded, or may be likely to yield, information important in prehistory or history" (U.S. Department of the Interior 1991:21).

The information included in this report and in the appendices has exhausted the research potential of the Peggy Sue Bridge, LA 110645.

In view of the information presented above, the Peggy Sue Bridge is not eligible for inclusion on the National Register of Historic Places since it does not meet the requirements for eligibility under Criterion A, Criterion B, Criterion C, or Criterion D. Furthermore, the bridge is not yet fifty years old and it would have to be an exceptionally important property (Criteria Consideration G) in order to be considered for eligibility.

RECOMMENDATION

As a result of this historical evaluation, this project complies with the National Historic Preservation Act of 1966 (as amended) and with Executive Order 11593. The New Mexico State Historic Preservation Officer is requested to concur in a "Determination of No Effect".

REFERENCES CITED

Black and Veatch Consulting Engineers

1949 *Specifications for Natural Gas Transmission Line, Los Alamos Project, Los Alamos, New Mexico.* Invitation No. 291-49-122. On file at ESH-20, Los Alamos National Laboratory, Los Alamos, New Mexico.

Frank Henri and Associates Consulting Engineers

1985 *Los Alamos County Utilities Department Pueblo Canyon Suspension Bridge, Engineering Report, Physical Inspection and Structural Evaluation.* On file at the Los Alamos County Public Works Department, Los Alamos, New Mexico.

Gordan and Associates Consulting Engineers

1994 *A Feasibility Report for the Remodeling of the Peggy Sue Suspension Bridge From an Existing Pipeline Support Bridge to a Trail Bridge for Pedestrians and Bicyclists.* On file at the Los Alamos County Public Works Department, Los Alamos, New Mexico.

Hunn, Joyce

1981 "The Peggy Sue Bridge". Single page document on file at the Los Alamos Historical Museum's archives, Los Alamos, New Mexico.

Kesler, Joel R.

1994 Letter from Joel R. Kesler, Senior Partner, Black and Veatch, to Jim Janecek, Los Alamos County Public Works Department, July 28, 1994. Copy on file at ESH-20, Los Alamos National Laboratory, Los Alamos, New Mexico.

Los Alamos Monitor

1995 "Save Peggy Sue petition gets 100 signatures". Newspaper article, Vol. 32, No. 173, Friday, September 1, 1995.

Los Alamos National Laboratory

1993 *Los Alamos: beginning of an era 1943-1945.* Reprinted by the Los Alamos Historical Society, Los Alamos, New Mexico

1994 "The "City on the Hill": From Atoms to Zygotes (Part 1)" in *Perspectives on Science*, Vol. 1, No. 1, Winter, Los Alamos National Laboratory, Los Alamos, New Mexico.

Manz, Kari L., Steven R. Hoagland, Ellen D. McGehee, and Beverly M. Larson

1993 *Infrastructure Support Facilities (ISF) Gasline - Los Alamos Townsite.* Cultural Resource Survey Report #60, Survey #546, on file at ESH-20, Los Alamos National Laboratory, Los Alamos, New Mexico.

U.S. Department of Interior

1991 "How to Apply the National Register Criteria For Evaluation", *National Register Bulletin*, No. 15. National Park Service, Interagency Resources Division, Washington, D.C.

White, Kenneth R. and Samuel P. Maggard

1990 *University of California, Los Alamos National Laboratory, Inspection of Utility Bridge, Project 9-LQO-Q4974.* On file at ESH-20, Los Alamos National Laboratory, Los Alamos, New Mexico.

POSSIBLE SOURCES FOR ADDITIONAL INFORMATION

Los Alamos County Public Works Department

The Los Alamos Historical Museum's Archives

The Los Alamos Monitor

LANL FSS Division (Engineering Department and FSS-3 Records)

The Los Alamos Police Department

Los Alamos Public Access Channel (PAC-8): film on the Peggy Sue Bridge (in progress 1995, Susan Yurkovic, contact)

APPENDIX A

LA 110645, The Peggy Sue Bridge

New Mexico Historic Building Form

NEW MEXICO HISTORIC BUILDING INVENTORY FORM

LA # 110645

structure threatened? yes	surveyed: date 10/27/94, 8/24/95 & 8/25/95 by E. D. McGehee	county Los Alamos	ID no. Pueblo Canyon 12" Pipeline Suspension Bridge, LANL Property # C00104579

field map number USGS Guaje Mtn. Quad 1:2400	UTM reference: zone 13 West Tower - 382260 easting 3972160 northing East Tower - 382370 easting 3972140 northing

location description Bridge is suspended over "Acid" Canyon, a tributary of Pueblo Canyon. The eastern end of the bridge is located near the Jewish Center in Los Alamos.	city/town Los Alamos land grant/reservation n/a

structure name Peggy Sue Bridge	legal description USGS Guaje Mountain 7.5 Series tnsp 19N range 6E section 9, N ½ SE ¼ SE ¼

film roll by E. McGehee no. 220 & 222	negative nos. #220: 2-21, 23-37 #222: 3-24, 27-37	location of neg. ESH-20, LANL	date of construction 1949-1950 estimate ______ actual source FSS-3 records (LANL), Black and Veatch records

style suspension bridge	foundation material steel wall material/surface steel	use present residential ✓ other abandoned historic residential ✓ other pipeline suspension bridge	condition ___ excellent _X_ good ___ fair ___ deteriorating

degree of remodeling _X_ minor ___ moderate ___major describe: routine maintenance	surroundings wooded canyon environment	relationship to surroundings ___ similar _X_ not similar bridge is suspended above canyon	district potential ___ yes _X_ no

significance not eligible ____ eligible _X_ of interest ____ none if eligible, why?	similar structures? _X_ yes what type? if inventoried, list ID nos. The "Little Peggy Sue" effluent bridge, Los Alamos Co.; The Gas Company of New Mexico's suspension bridge across the San Juan near Bloomfield, NM; and the Transwestern Pipeline Company's suspension bridge across the Rio Grande, near Berino, NM.	photos, drawings, and architectural information are on following pages The bridge is approximately 550 feet long by 3 feet wide.

architectural features The Peggy Sue Bridge is a steel pipeline suspension bridge. The bridge supports an abandoned 12" gas line which is mounted under the bridge deck. The bridge is a cable stayed, open spandrel arch and is 550 ft long with a 3 ft wide bridge deck. The main span is suspended from two 2-inch diameter cables supported by steel towers on either side of the canyon. The cables are anchored into large concrete blocks. According to a 1990 LANL inspection report, the anchor blocks show signs of long-term freeze-thaw damage. The bridge has loose cables and is missing some turn-buckles. Some rusting is also evident on the deck and the support towers. Remodeling feasibility and inspection reports have been written for this bridge:	comments The name Peggy Sue is a local name. Several local myths are associated with the bridge. The most commonly repeated story concerns a girl named Peggy Sue who purportedly jumped off the bridge to her death. According to a 1981 document in the Los Alamos Historical Museum archives, the Los Alamos Police Department had [from 1950 to 1981] no record of anyone, Peggy Sue or otherwise, who has jumped off the bridge. Although it has a common design and is not an engineering feat, the bridge is aesthetically pleasing and is situated in a quiet and beautiful canyon location. Research involving Gas Company of NM engineering offices has located three bridges in New Mexico that might be similar in design (see above).

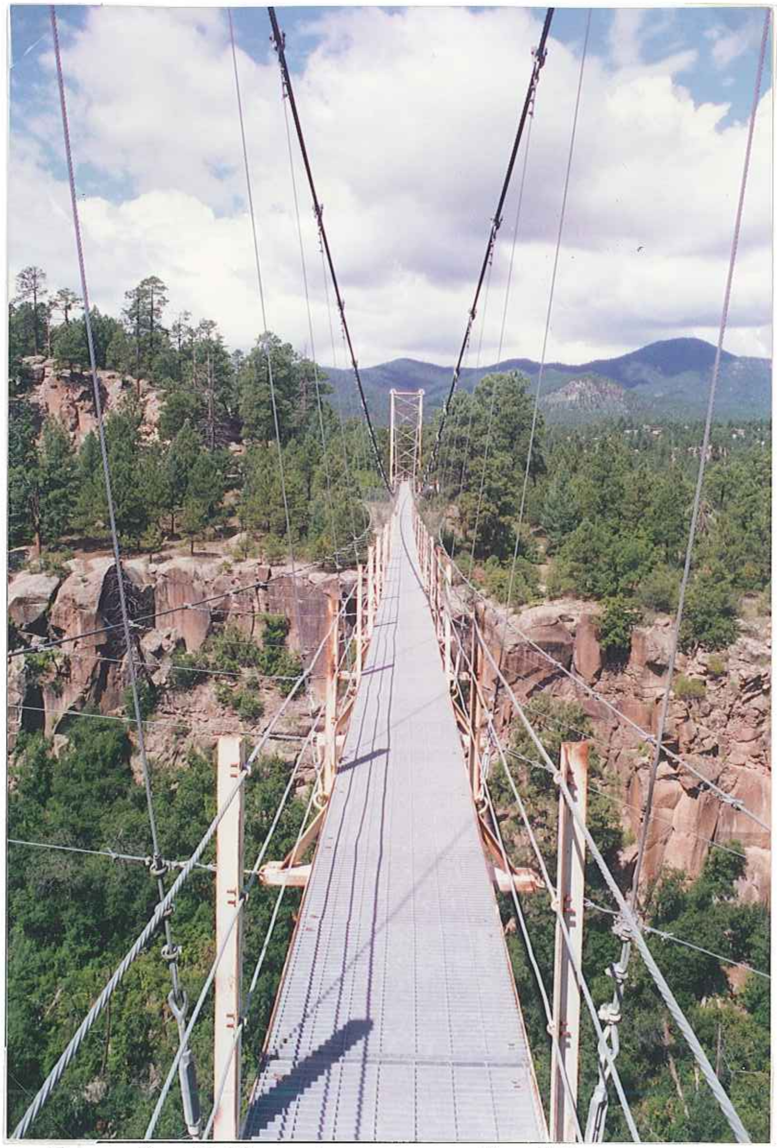
APPENDIX B

Peggy Sue Bridge Photographs

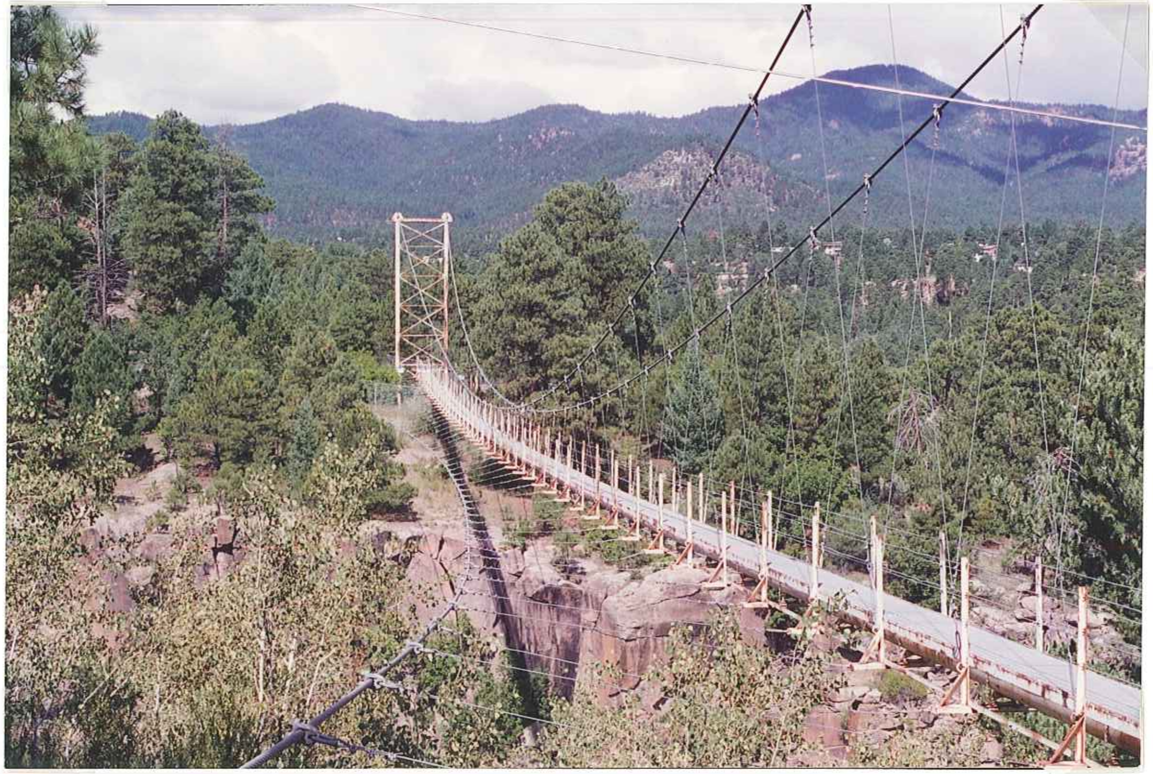
Peggy Sue Bridge with view of east tower, dir. ~ENE

Peggy Sue Bridge, view from Walnut Street playlot, dir. ~S

Peggy Sue Bridge, span west to east, dir. ~E



Peggy Sue Bridge, view to west, dir. ~W



Peggy Sue Bridge, dir. ~W

Peggy Sue Bridge, west tower, dir. ~W

Peggy Sue Bridge, east tower, dir. ~NE

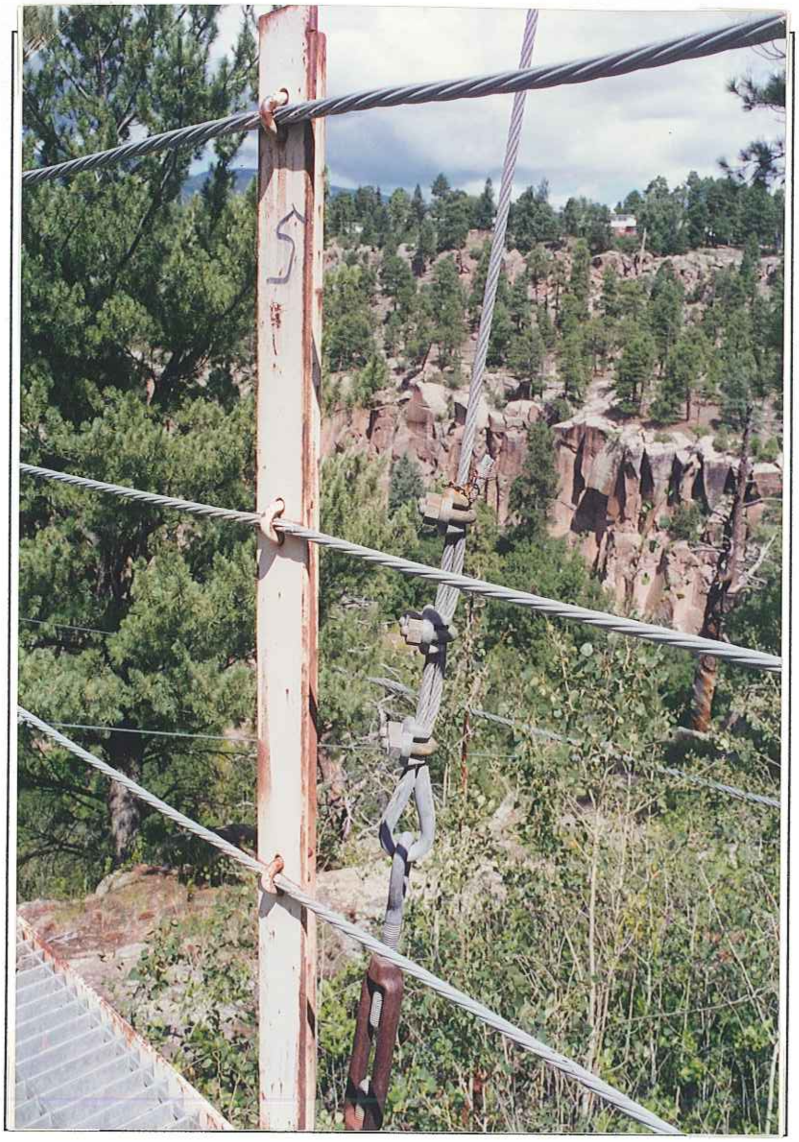
Peggy Sue Bridge (west end), gas piping, dir. ~W

Peggy Sue Bridge (west end), western-most concrete anchor, dir. ~NNE

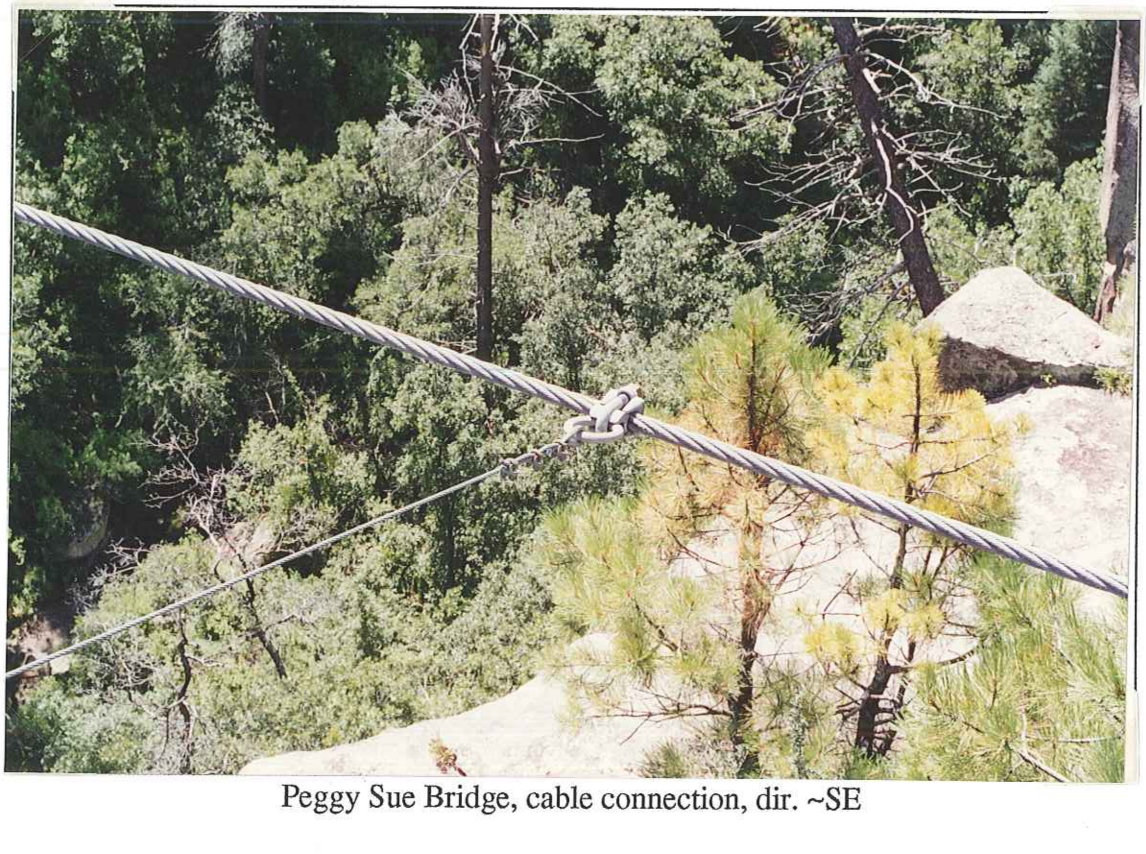
Peggy Sue Bridge (east end), detail of anchor inside fence, dir. ~E

Peggy Sue Bridge (west end), view of underside, dir. ~E

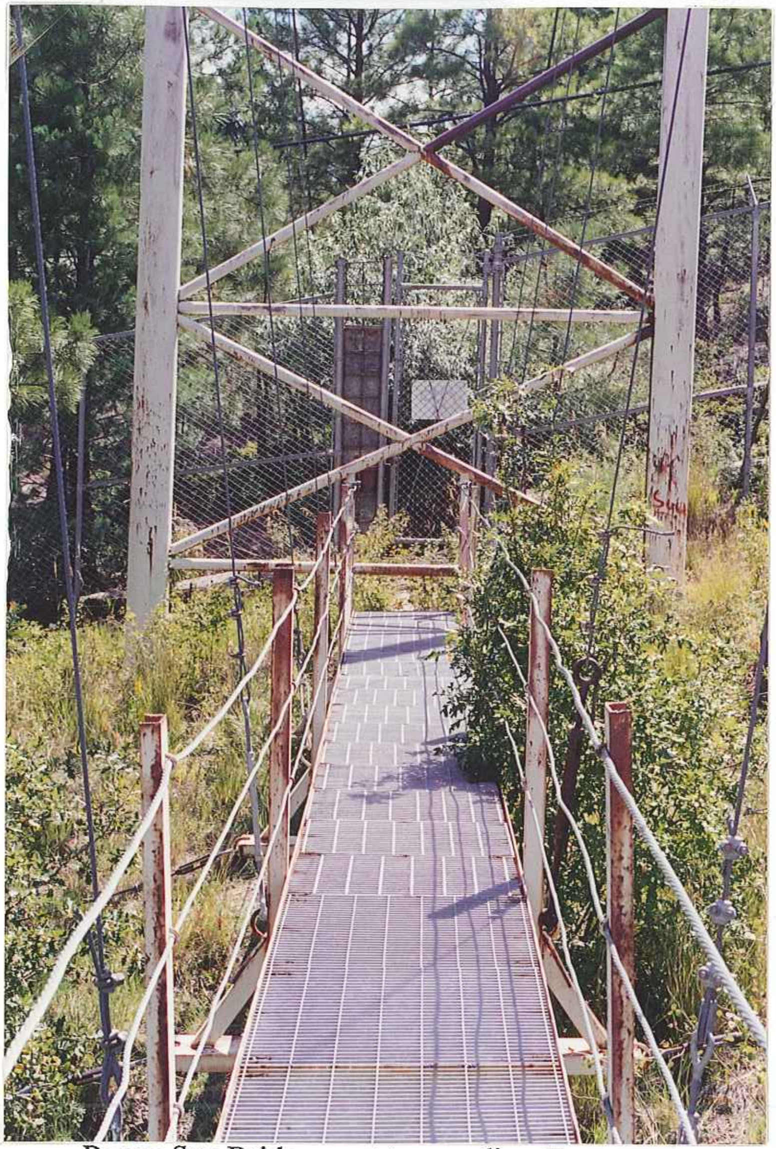
Peggy Sue Bridge (west end), view of pipeline, dir. ~W



Peggy Sue Bridge, walkway support detail, dir. ~NW



Peggy Sue Bridge, cable connection, dir. ~SE



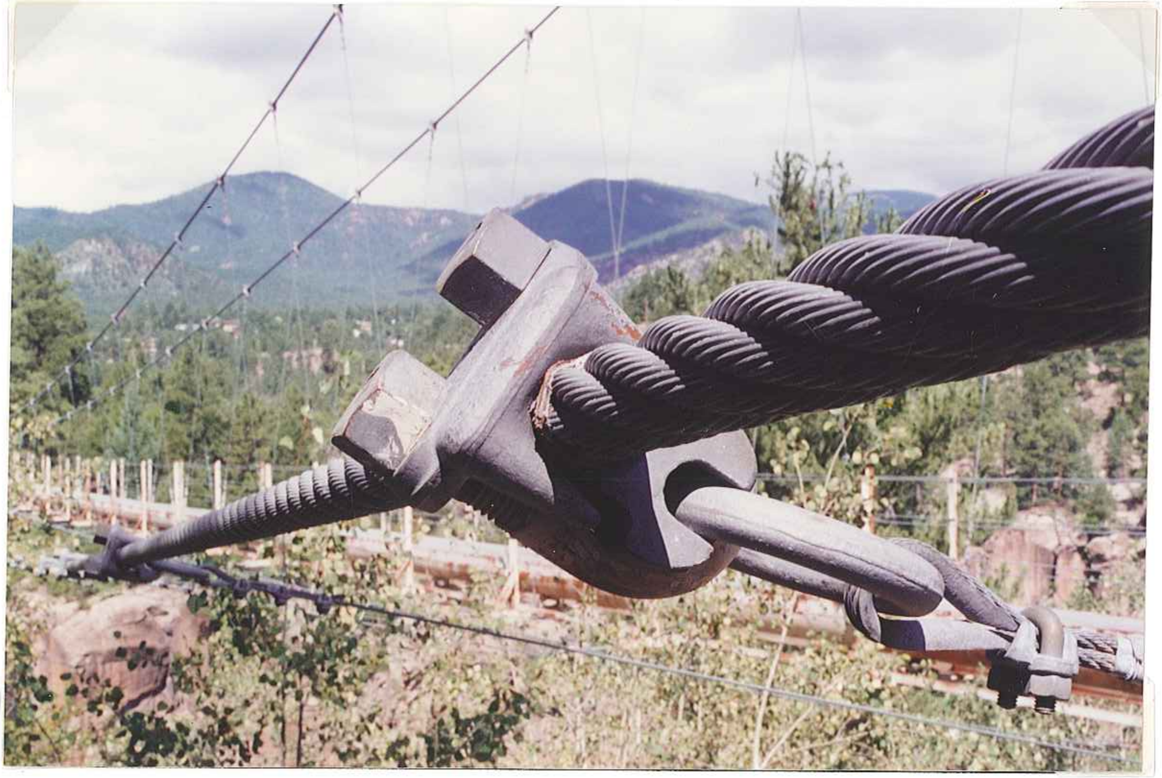
Peggy Sue Bridge, east tower, dir. ~E

Peggy Sue Bridge (east end), view of old gas line, dir. ~W

Peggy Sue Bridge (east end), walkway detail and pipe, dir. ~N

Peggy Sue Bridge (east end), decking detail, dir. ~E

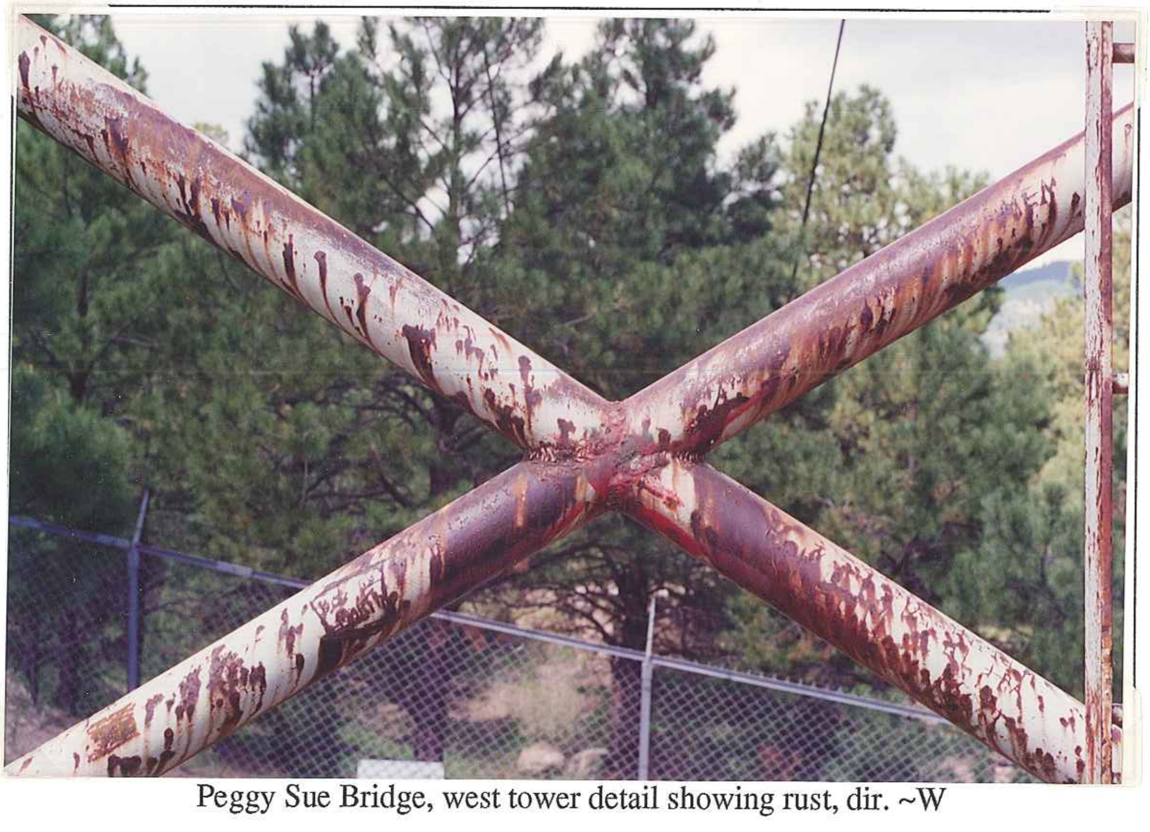
Peggy Sue Bridge, detail of decking and pipe, dir. ~down & east



Peggy Sue Bridge (east end), cable detail, dir. ~WNW

Peggy Sue Bridge (east end), cable connection detail, dir. ~W

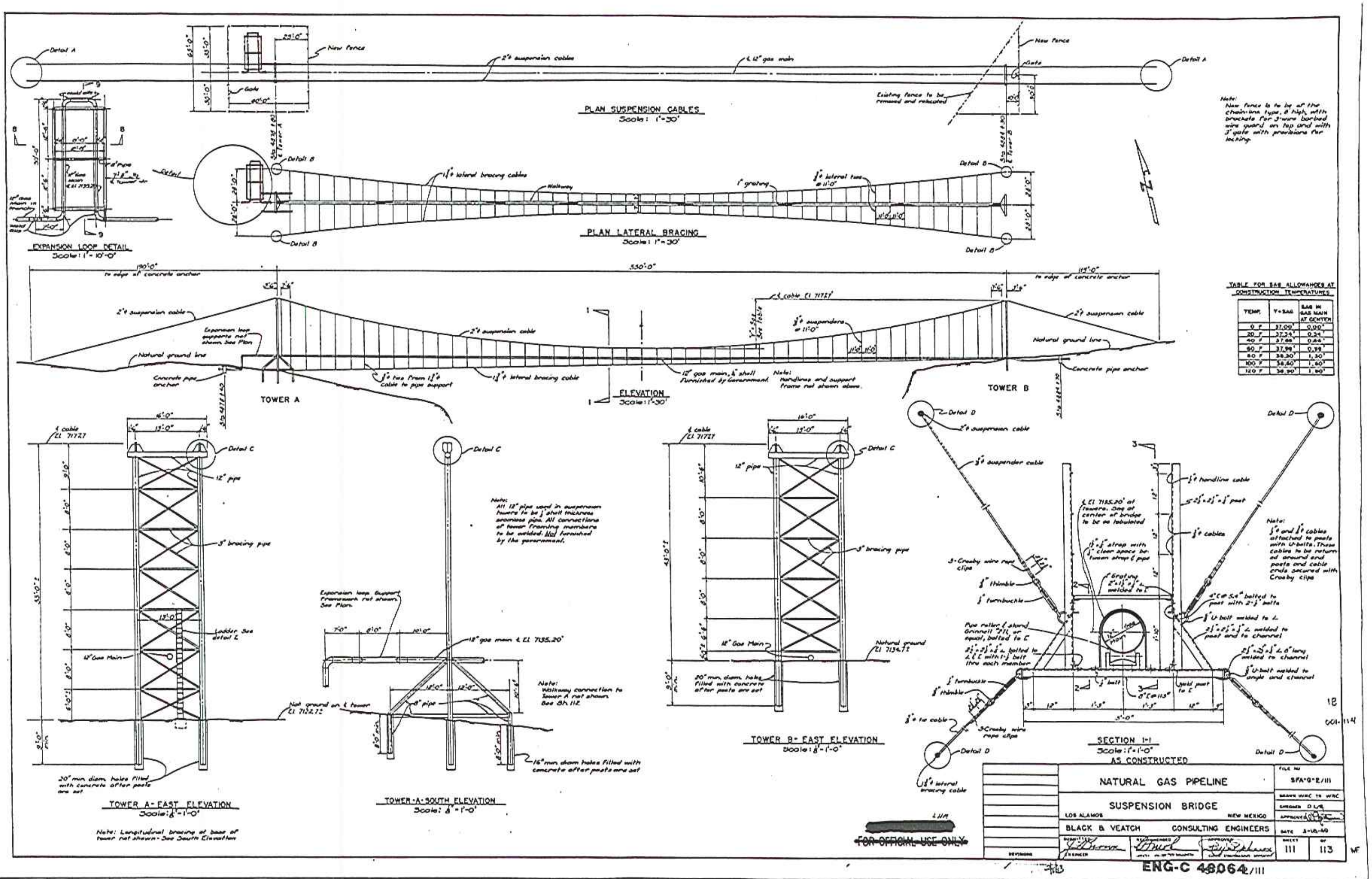
Peggy Sue Bridge, west tower detail showing rust, dir. ~W



Peggy Sue Bridge, west tower detail showing rust, dir. ~W

APPENDIX C

Peggy Sue Bridge Drawings and Associated Engineering Reports



PLAN SUSPENSION CABLES
Scale: 1" = 30'

PLAN LATERAL BRACING
Scale: 1" = 30'

EXPANSION LOOP DETAIL
Scale: 1" = 10'-0"

ELEVATION
Scale: 1" = 30'

TOWER A - EAST ELEVATION
Scale: ⅛" = 1'-0"

Note: Longitudinal bracing at base of tower not shown - See South Elevation

TOWER-A-SOUTH ELEVATION
Scale: ⅛" = 1'-0"

TOWER B - EAST ELEVATION
Scale: ⅛" = 1'-0"

SECTION 1-1
Scale: 1" = 1'-0"

Note: New fence is to be of the chain-line type, 8' high, with brackets for 3-wire barbed wire guard on top and with 3' gate with provisions for locking.

Note: All 12" pipe used in suspension towers to be ¼" shell thickness seamless pipe. All connections of tower framing members to be welded. Not furnished by the government.

Note: Walkway connection to Tower A not shown. See Sh. 112

Note: ⅜"⌀ and ⅝"⌀ cables attached to posts with U-bolts. These cables to be returned around end posts and cable ends secured with Crosby clips

TABLE FOR SAG ALLOWANCES AT CONSTRUCTION TEMPERATURES

TEMP.	Y+SAG	SAG IN GAS MAIN AT CENTER
0 F	37.00'	0.00'
20 F	37.34'	0.34'
40 F	37.66'	0.66'
60 F	37.99'	0.99'
80 F	38.30'	1.30'
100 F	38.60'	1.60'
120 F	38.90'	1.90'

AS CONSTRUCTED

NATURAL GAS PIPELINE

SUSPENSION BRIDGE

LOS ALAMOS NEW MEXICO

BLACK & VEATCH CONSULTING ENGINEERS

FILE NO.	SFA-0-2/111
DRAWN	WRC TR WRC
CHECKED	DLR
APPROVED	
DATE	3-15-49
SHEET	111
OF	113

SUBMITTED J. Brown ENGINEER

RECOMMENDED

APPROVED

REVISIONS

WF

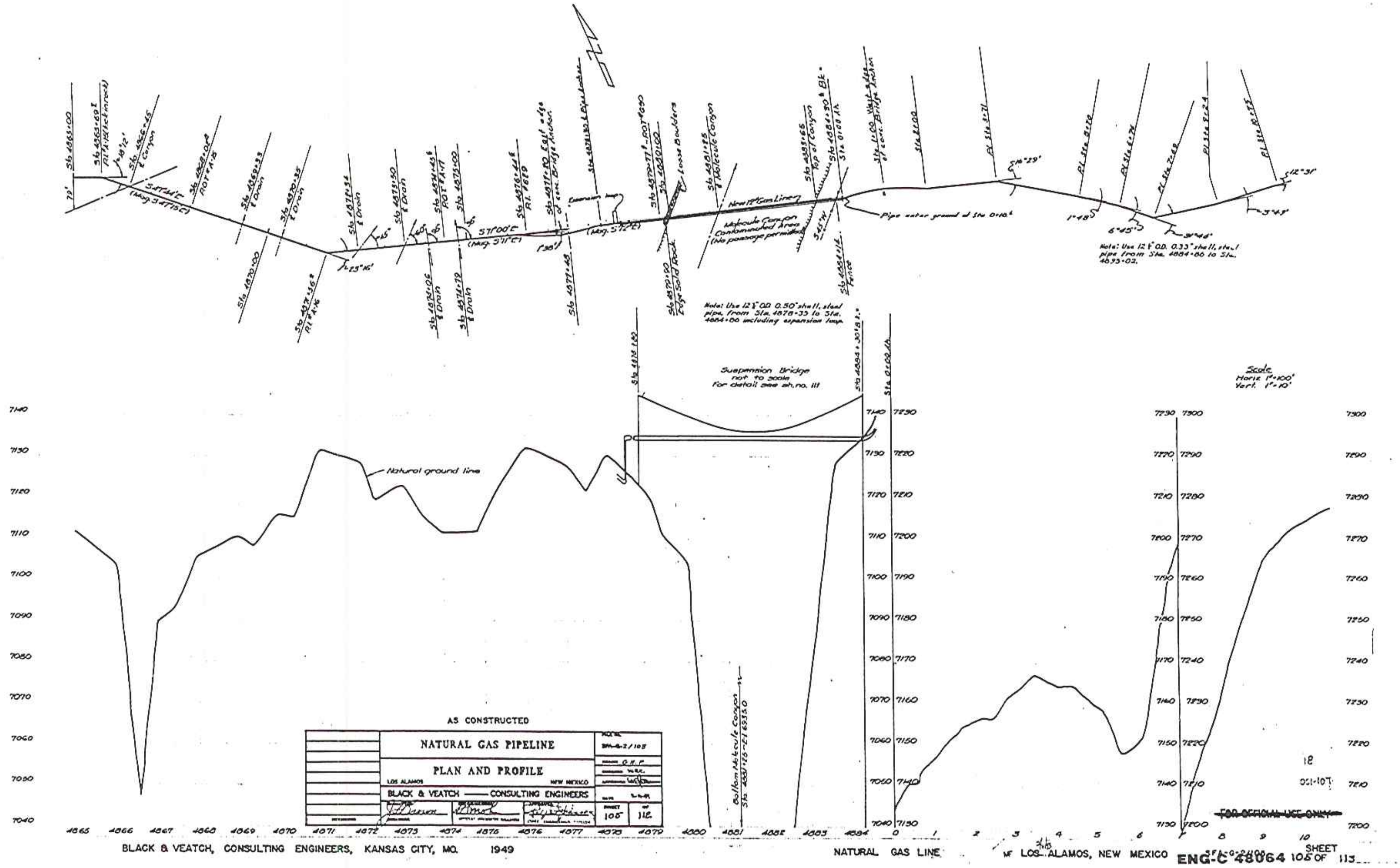
LHA

[redacted]

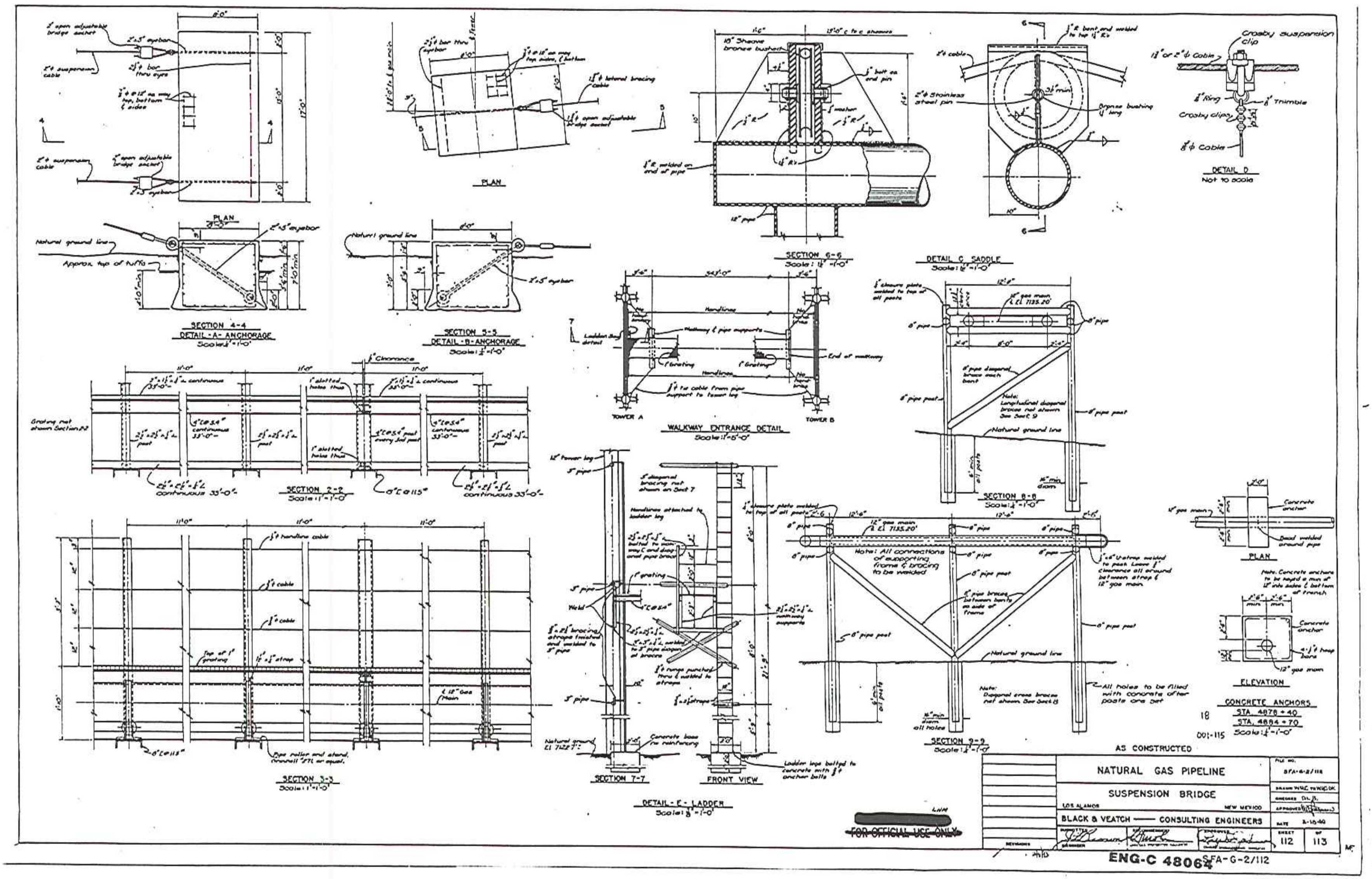
~~FOR OFFICIAL USE ONLY~~

ENG-C 48064 2/111

Peggy Sue Bridge, LA 110645



Peggy Sue Bridge and ISF Gas Line



[redacted]

Peggy Sue Bridge construction details

Invitation No. 291-49-122

SPECIFICATIONS

FOR

NATURAL GAS
TRANSMISSION MAIN

LOS ALAMOS PROJECT
LOS ALAMOS, NEW MEXICO

UNITED STATES
ATOMIC ENERGY COMMISSION
SANTA FE AREA
SANTA FE, NEW MEXICO

BLACK & VEATCH
CONSULTING ENGINEERS

1949

NATURAL GAS TRANSMISSION MAIN
LOS ALAMOS, NEW MEXICO

SPECIFICATIONS

SECTION VI
SUSPENSION BRIDGE

6-01. SCOPE.- This specification covers the furnishing of all materials and the construction of a suspension bridge for supporting the natural gas transmission main across a canyon from station 4878+80 to station 4884+30, in accordance with the details shown on the Contract Drawings and as specified herein.

The Contractor is notified that he shall not require nor permit any of his employees to enter this canyon for any purpose without first having obtained specific permission from the Contracting Officer. The Contractor and his employees shall comply with all requirements prescribed by the Contracting Officer with reference to security or precuationary measures to be observed by persons entering this canyon.

6-02. TOWERS.- The towers shall be built of steel pipe welded into structural frames, as shown, set into the tufa formation to a depth not less than that indicated by the details. The main tower legs shall be made of 12 3/4-inch outside diameter steel pipe having a shell thickness of approximately 1/2 inch. Other portions of the tower shall be built of various sizes of black steel pipe, schedule 40, or heavier and of other steel members and details, as shown. All the 12 3/4-inch pipe required for the construction of the pipe line will be furnished by the Government in the same manner and subject to the same conditions as the pipe for the pipe line.

6-02.01. Anchorage of Towers.- The Contractor shall excavate round holes into the tufa formation to the depth shown on the drawings, or to such increased depth as directed by the Contracting Officer. The holes for anchorage shall be accurately located in position and direction, and they shall be checked after excavation, to insure proper alignment. The holes shall then be carefully cleaned of all loose material. The bottom of each hole shall be not less than one foot lower than the bottom of the tower post as shown or indicated on the drawings. The tower legs shall be carefully lowered into position and securely braced to prevent displacement. After they have been checked for position and alignment, the space between the steel work and the sides and bottoms of the holes shall be slowly and carefully filled with structural concrete, puddled and compacted so as to completely fill the space. The top of the concrete shall be finished, as shown. After the main tower legs have been anchored the bottom tower braces shall be accurately placed, concreted in and welded to the main posts.

6-02.02. Steel Construction.- The connections of all members of the tower framework, except as otherwise shown, shall be made by welding. The tower shall be assembled in a horizontal position, and welded together in that position. In general welding methods shall be equal to those specified for making welded joints in the pipe line, and the welding shall be done by welders of equal competence and qualification. All members of the main tower framing shall be so aligned that their axes shall lie in the same straight plane. Special precautions shall be taken to prevent distortion due to heating and cooling during the process, and the work shall be maintained in alignment so that when erected all members shall be straight and free from warping or deformation of any kind. At each joint, the full area of weld shall be continuous around the pipe, and such area shall be not less than is required to develop the full strength of the weakest connection member.

Any splice in the main tower posts shall be located at approximately one-fourth the distance from one panel point to the next.

6-03. CABLES AND ANCHORS.- All wire ropes and cables shall be of the sizes shown and shall be spun from galvanized, plow steel bridge wire. They shall be the product of a recognized manufacturer of wire rope, and within the limitations of these specifications, shall be of a design suitable for the use to which they are put. All ropes or cables shall be shipped and handled to the point of their installation on suitable reels, handled in such a way as to preserve them from damage of all kinds. Care shall be taken in unreeling and handling cables to prevent loosening of strands and the kinking of the cable due to any cause. The ends of all wire ropes or cables shall therefore be properly seized, as recommended by the manufacturer, before any cutting thereof. Because of the importance of such precautions, all handling and installation shall be done by competent workmen under the direction of a superintendent experienced in the carrying out of similar work.

6-03.01. Main Cables.- The main suspension and main side sway cables shall be of 7 x 19 construction. They shall be cut to proper length for installation with due allowances made for installation condition. The main suspension cables shall be installed hanging free from the towers and anchored at the ends, and adjusted at such a height in the center of span that when loaded with the dead load of pipe and bridgework the amount of sag will be that indicated on the drawings. The Contractor shall provide himself with all necessary snubbing cables and other equipment to make all preliminary and final adjustment.

The end anchorage of main suspension and side sway cables shall be made by means of steel eyebars embedded and secured in concrete anchor blocks. The cable ends shall be attached to adjustable bridge sockets which will be linked to the eyebars by pins of suitable size.

The method of attaching the end of the rope or cable to the bridge socket shall be such as will develop 100 percent efficient connection. It shall be such as is approved by the wire rope manufacturer, and, except as otherwise authorized by the Contracting Officer, shall be as follows:

1. The rope shall be securely seized and served with soft wire ties before cutting, and at least two additional seizings shall be placed at a distance from the end equal to the length of the basket of the socket. The seizing shall be of adequate length, and securely wrapped with a serving iron.

2. The rope being properly seized, the end seizing shall be taken off. The strands shall then be separated. Any fiber core shall be cut off back to the first seizing, but wire strands used as a core shall not be cut. The wires shall then be separated, untwisted and broomed out.

3. The wires, for the distance that they are to be inserted in the socket, shall be carefully cleaned with benzine, naphtha, or gasoline. They shall then be dipped for a distance not greater than three-quarters of the cleaned length of the wire in commercial muriotic acid for from 30 seconds to 1 minute, or until the acid has thoroughly cleaned each wire. Care shall be taken that the acid does not come into contact with any other portion of the rope.

4. Dip the ends of the wires in boiling water containing a small amount of soda to neutralize the acid. Wipe dry. Serve the end temporarily so that the socket will slip over all the wires, being careful not to let grease or oil touch the cleaned wires.

5. Heat the socket and slip it over the end of the wires. Cut the temporary seizing, and distribute all wires evenly in the basket with the ends flush with the top. Be sure that the socket is in line with the axis of the rope.

6. Holding the rope vertically in a vise, seal the bottom of the socket from the outside with fire clay or similar substance. Check position and alignment of socket.

7. Pour molten zinc into the basket until it is full. Use only high-grade zinc, heated not above 830 degrees Fahrenheit. Do not use babbit or other anti-friction metal. When the zinc has solidified sufficiently it shall be allowed to cool. The clay shall then be removed. Remove all seizings except the one nearest the socket. If properly made, this seizing will be at the end of the socket.

The main suspension rope or cable shall be supported at the top of the towers in accordance with details, as shown on the drawings.

6-04. VERTICAL AND LATERAL TIES.- The weight of the pipe line and the walkway and their supports shall be supported at the panel points as indicated on the drawings by means of ties or suspenders attached to the main suspension cable. Likewise the bridge shall be held in line against wind load by similar ties at the panel points which are attached to the lateral sway cables.

All ties except as otherwise provided or approved, shall be 7-wire zinc coated, Siemens-Martin grade, strand with "extra galvanized" (double galvanized) zinc coating, or a galvanized strand or rope of equal size, strength and durability under service conditions.

It is important that the suspenders which carry the weight of the pipe line and walkway shall be capable of ready adjustment in order to secure uniform tension. They shall be provided with turnbuckles which shall be of a size and pattern to develop the full strength of the strand. The ties between the walkway and the side sway cables shall likewise be capable of sufficient adjustment to secure and maintain uniform tension, such adjustment being obtained with either a turnbuckle or by other approved means.

Attachments to the main cables, both suspension and lateral, shall be made with galvanized drop forged steel clips equal to genuine Crosby Perfected Suspension Clip, with specially adapted filler blocks, as made by the American Hoist and Derrick Company. Wherever the strands pass through U-bolts or eye bolts of clips, turnbuckles or other attachments, they shall be protected with galvanized oval thimbles. All eyebolts shall be welded shut. Where the length of ties is short, rods and turnbuckles may be substituted for the strand, but shall be of equal or greater strength. All fittings or parts used for making up the ties shall be heavily galvanized after fabrication.

6-05. WALKWAY AND PIPE SUPPORT.- The pipe shall be supported across the bridge on a structural steel framework which shall include a steel grating walkway protected on each side with a railing. Construction shall conform to the details shown on the drawings.

The pipe line shall be supported at each panel point on a cast iron roller of heavy pattern, designed for service in an outdoor location.

The flooring shall be constructed of grating which shall be of a rectangular type, either welded or pressure-locked, and shall be hot-dip galvanized after fabrication. The load carrying bars of the grating shall be not less in depth than 1 inch nor less than 3/16 inch in thickness. The span of the grating shall be cross-wise of the bridge, the ends being positively attached to supporting members so that the grating sections may be removed for work on the bridge or pipe, but cannot be accidentally dislodged.

The handrailing shall be composed of galvanized wire rope strands or cables of the size and spacing shown on the drawings. The ropes shall be securely attached to the posts with U-bolts of suitable design. Posts shall be strongly braced to prevent side sway.

All portions of the framework shall conform to the specifications and standards of the American Institute of Steel Construction. Ungalvanized surfaces shall be shop painted.

6-06. FENCING.- A chain-link fence is now installed along the rim of the canyon. The Contractor shall carefully take down and store such panels of the existing fencing as are in the way of construction of the suspension bridge. After construction of the bridge has been completed the Contractor shall relocate the fence to enclose the bridge end, as shown on the drawings, adding to the panels previously removed, additional sections, gates, etc., to complete the installation. The new parts shall be similar to the existing fence, and shall comply with applicable portions of Paragraph 5-11 and its several subparagraphs of this specifications. A new fence with gate shall be constructed to enclose the other end of the bridge, as shown.

6-07. PAINTING.- After erection of the bridge, all ungalvanized metal surfaces of bridge and towers shall be given a field coat of red lead paint and two coats of oil vehicle paint of colors selected by the contracting officer.

The red lead paint shall conform to the provisions of Federal Specification TT-P-86. The succeeding coats shall be Rust-Oleum "L-O", Sherwin-Williams "Metalastic," Truscon "Bar-Ox", Tnemec Industrial Coatings, or equal.

Preparation of surfaces, and the handling, application of paints and other general provisions governing painting work shall be in compliance with provisions of Paragraph 5-07 and its various subparagraphs of these specifications.

The portion of the pipe line that is exposed for the full length of the bridge shall be painted as specified herein for the tower framework, instead of being coated and wrapped, as provided for pipe laid underground.

6-08. EXPANSION LOOP AND CONCRETE ANCHORS.- An expansion loop in the pipe line shall be constructed at one end of the bridge, as shown. The loop shall be of welded construction using the same size and thickness of pipe as used in the portion of the pipe line on the bridge together with welding tube turns having a shell thickness of at least 1/2 inch. The loop shall project horizontally and shall be supported on a welded pipe framework anchored into the tufa as shown. Beyond the loop from the bridge a concrete anchorage shall be constructed to prevent creep or motion of the pipe. The loop and its steel supports shall be painted as specified for the framework of the bridge towers. A concrete anchorage shall be constructed where the pipe leaves the trench at the end of the bridge opposite the expansion loop.

6-9. METHOD OF PAYMENT.- Payment for the construction complete of the suspension bridge and the installation of the gas transmission main thereon as well as the expansion loop, and the concrete anchorage at either end will be made at the lump sum price stated under Item 21 of the Bid, which shall constitute compensation in full for all labor, construction plant and materials required to complete the suspension bridge and the installation of the gas main thereon as required by the contract drawings and these specifications.

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SAG RATIO 550/n	SAG	DIST. BOTT. SAG TO BOTT PIPE	HEIGHT TOWER B = SAG + 6'	DIFF. ELEV. TOWERS A&B	HEIGHT TOWER A = HT. B + DIFF. EL.	LOADS: PIPE t=0.5 #/ft.	ICE #/ft.	WALK-WAY #/ft.	CABLES #/ft.	WIND #/ft.	√(V²+H²) TOTAL	TENSION CABLES ON EACH TOWER	TENSION EACH CABLE TOP OF TOWER	REACTION CABLES ON TOWER #	WORKING STRESS IN WIRE ROPE #	SIZE CABLE NEEDED	SIZE CABLE WTS. COMPUT. FOR (SUSP. CABLE)	AREA CABLE COMP FOR.	CABLE SATIS-FACTORY
l/8	69'	6'	75'	2'	87'	65.4		11.5											
l/10	55'	6'	61'	2'	73'	654		11.5			200	148,000	74,000		47,500	1.56		1.767	O.K
l/12	46'	6'	52'	12	64'	654	69.5	11.5	36.9	43	189	159,500	79,750	202,000 202,000 202,000	47,500	1.68	1½"φ	1.767	O.K
l/15	31'	6'	43'	12	55'	654	65	11.5	35.9 57.6	43	184 224 200	94,500 240,000 216,000	97,250 120,000 108,000	420,000 420,000 420,000	47,500 47,500 47,500	2.04 2.52 2.28	1½"φ 2¼"φ 2"φ	1.767 4.91 3.1	No. O.K O.K use

Hor. cable load at towers = $\frac{100 \times 550^2}{8 \times 37}$ = 102,000#

max. tension in cables = $102{,}000\sqrt{1+\frac{16 \times 37^2}{550^2}}$ = 106,000# OK

Reference
Sag Ratio — page 1279. A.C.E. Hdbk merriman
Length Cable with sag 1279 above
Tension form. Cables 278 above
Strength wire ropes 701 above
working stress in ropes 702 above
Proportion vertical load acting on top of tower 1282 above
Allowable stresses for l/r ratio 210 A.I.S.C. Hdbk
Pipe description 139 above

TOWER A — 55' high
Reccomend 18" pipe (5/8) thick 12" pipe ½" thick

TOWER B — 43' high
Reccomend 12" pipe page 138

Cable size
2"φ cable with a minimum ultimate stress of 138,000#

	SAG RATIO	SIZE PIPE IN TOWER LEGS	L/r A B	I	LONG. LOAD TOP TOWERS REAC. ON TOWER / 100 / 2	WIND LOAD per leg	WIND MOMENT	LONG. MOMENT	TOTAL MOMENT	$S_1 = \frac{P}{nca}$ S_2 = Reaction / Pipe Area	$S_2 = \frac{Mc}{I}$	
TOWER A	l/8											
	l/10											
	l/12	12"	177	361.5	505	1966	54300	32,300	86,600	2630#	17200#	.56 + .867 >1 Too high.
1" thick		16"	145	1331.5	505	2694	71,900	32,300	104,200	1070#	7510#	.147 + .376 <1 overdesigned
5/8" thic.		14"	163	588.5	505	2471	58,400	32,300	90,700	1920#	12,950#	.336 + .646 <1 O.K .982
	l/15	8"	239	162	560#/leg	1803	31,800#	30,800#	62,600#	4½ 3348#	20,000# 18,900#	.79 + .95 Too high
		10"	182	211.9	"	1499	31,300	"	72,000	2920#	15,250#	.44 + .76 >1 Too high
		12"	152	361.5	"	1690	41,200	"	76,800	2130#	11,000#	.27 + .55 <1 O.K .82
5/8" th.		14	140	588.5	560	1886	46,000	30,800	76,800	2640#	15,800#	.33 + .78 >1 Too high.
1/2" th.		4'	138	483.8	560	1886	46,000	30,800				
TOWER B	l/15	8"	187	162	560	1025	20,900	24,100	44,600	2630#	14,200#	.65 + .71 >1 Too high
		10"	142	211.9	560	1184	23,480	24,100	47,780	3740#	14,500#	.46 + .72 >1 Too high
		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	24,100	50,940	2910#	11,750#	.29 + .59 <1 .88 OK

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Try Tower B at Station 4884 + 30

Try Tower A at Station 4878 + 80

5 + 50 Span = 550' = l

Refer to page 1279, American Civil Eng. Hdbk - Merriman, Wiggin for following.

$$\text{Sag Ratio or Dip Ratio} = \frac{\text{sag of cable}}{\text{span}} = \frac{h}{l} \quad \left[\text{Varies from } \frac{1}{8} \text{ to } \frac{1}{15}\right]$$

Investigate both conditions -

$$\text{Sag}_1 = \frac{l}{8} = \frac{550}{8} = 68.75 \quad \text{say } 69'$$

$$\text{Sag}_2 = \frac{l}{15} = \frac{550}{15} = 36.7 \quad \text{say } 37'$$

$S_1 = 69'$

$S_2 = 37'$ ← Use

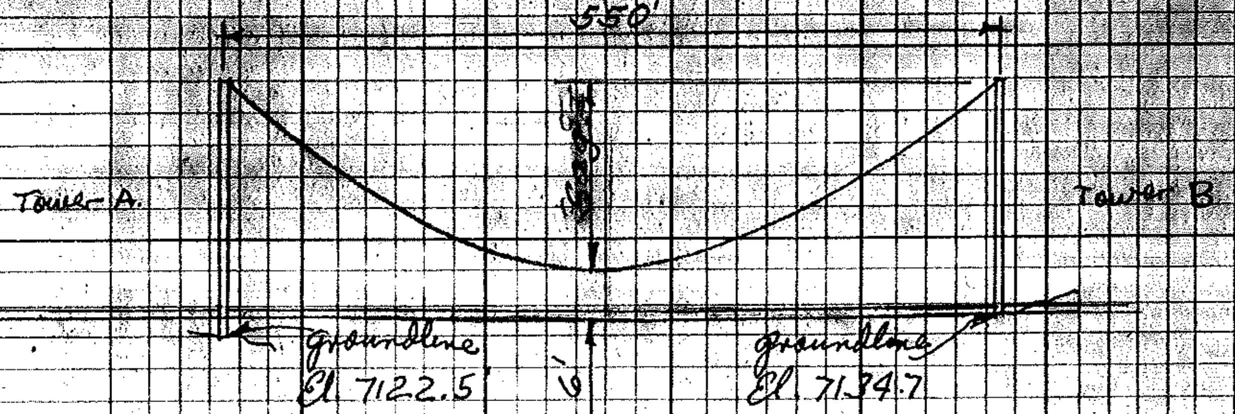
Tower Height determination for sags -

Elevation Tower B at Sta 4884 + 30 = 7134.7'

Elevation Tower A at Sta 4878 + 80 = 7122.5'

12.2' = Difference

Take pipe into ground at ground elevation Tower B



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Height of Tower B = sag + 6'. (Assume 6' as distance between Cable and bottom of pipe)

For sag of l/8 = 69'—
Height = 69 + 6 = 75'

For sag of l/10 = 55'
Height = 55 + 6 = 61'

For sag of l/15 = 37'
Height = 37 + 6 = 43'

For sag of l/12 = 45.9 = 46
Height = 46 + 6 = 52'

Height of Tower A = Height of Tower B + difference in elevation

Sag of l/8
Height = 75 + 12 = 84'

Sag of l/10
Height = 61 + 12 = 73'

Sag of l/15
Height = 43 + 12 = 52

Sag of l/12
Height = 52 + 12 = 64'

Sag Ratio l = 550'	Sag	Dist from bott of sag to bott pipe	Height Tower B = Sag + 6	Diff in Elev. tower A and B	Height Tower A = Ht. Tower B + Diff El.
l/8	69'	6'	75'	12'	87'
l/10	55'	6'	61'	12'	73'
l/12	46'	6'	52'	12'	64'
l/15	37'	6'	43'	12'	55

From primary investigation and for a tower using 8" pipe it seems best to use l/15 to reduce l/r ratios. However stresses in pipe will have to be determined.

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Load Determination

Assume live load as negligible

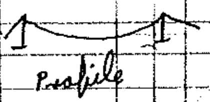
Dead load = Wt. pipe + wt. cable + wt. walkway + wt. ice + wt. wind

Refer to page 139 AISC Handbook

Weight of 12" pipe .500" thick = 65.42 $\frac{\#}{ft}$

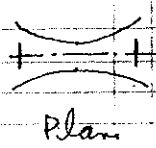
Refer to page 1279 of Amer. Civil Eng. Hdbk.

Length of cable with sag = $l\left(1+\frac{8}{3}\frac{s^2}{l^2}\right)$



1. Approx. length cable tower to tower plus backstay lengths equals $2\times l\left[1+8\frac{\times\overline{37}^2}{3\times\overline{550}^2}\right]+360$ Using sag ratio of $\frac{1}{15}$

$=[570+360]2 = 2\times 930' = 1860'$



~~2. Approx. length cables for lateral sway bracing plus backstay lengths.~~

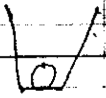
~~Assume pipe 3' from sway wire at ℄ of span and 18' at towers giving an s of 15.~~

~~$L = 2\times l\left[1+8\times\frac{\overline{15}^2}{3\times\overline{550}^2}\right] = 1100\left[1+\frac{1800}{302{,}500}\right] = 1100[1.00595]$~~

~~$= 1106.5'$~~

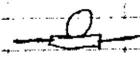
~~Add 100' for anchorages~~

~~$L = 1207'$~~



3. Approx. length suspender cables supporting pipe placed at 20' intervals between towers

$\text{Length} = \frac{550'}{20'}\left[\frac{44'+6'}{2}\right][2] = 1375'$



4. Approx. length wind bracing at 20' intervals

$\text{Length} = \left[\frac{33.5'+3.5'}{2}\right]\left[\frac{550}{20}\right] = 509'$

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Load determination, cont.

Assume following sizes —

	Length	Size	weight per foot	Total weight
1. Main suspension Cables	1860'	1½" φ	6 #/ft	11,180
2. Main lateral bracing cables	1207'	1½" φ	6 #/ft	7242
3. Suspender Cables	1375'	½" φ	.7 #/ft	962
4. Sway bracing ties	509'	½" φ	.7 #ft	357

total = 19,741 #

Weight of Cable per foot of span = $\frac{19741}{550}$ = 35.9 #/ft.

Approximate weight of walkway —

Refer to page 10 Irving Bulletin A I.A. No. 14 P20 - weight of Hex-Weld Grating HW-A = 4.21 #/sq.ft

Assume walkway 2'-6" wide

Weight = 4.2 × 2.5 = 10.5 #/ft.

Assume ∠ handrails every 20' to be 3'-3" long and made of ∠ 1½" × 1½" × ¼"

weight per foot span = $\frac{\frac{550}{20}[3.25][2][2.34]}{550}$ = .76 say 1 #/ft

Total weight walkway = 10.5 + 1 = 11.5 #/ft.

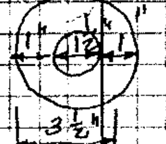
Weight of ice —

Assume 1" of ice ~~on twice the horizontal projection of Cables (both sides covered)~~ on all Cables, pipe, and walkway.

Cables: (1½" φ)

Area of ice on Cable = $(.7854)(3.5^2 - 1.5^2)$

= $(.7854)[12.25 - 2.25]$ = 7.85 sq. in



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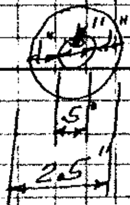
Weight of ice continued.

Cables $\frac{1}{2}$"ϕ

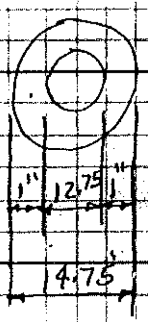


Area of ice on Cables $= (.7854)\left[\overline{2.5}^2 - \overline{.5}^2\right]$

$= (.7854)(6) = 4.72$ sq. in.

Pipe

Area of ice on pipe



Area of ice on pipe $= (.7854)(\overline{14.75}^2 - \overline{12.75}^2)$

$= (.7854)[21.75 - 162.5] = (.7854)(55)$

$= 43.2$ sq. in

Walkway

Area = 2.5' wide × ~~length~~ × 2" thick = 60 sq. in.

Part.	Area of cross-section (sq. in.)	Length (in ft)	Volume ice $\frac{(A \times L \times 12)}{1728}$ (cu. ft).	Weight of ice (Vol. × 57.5)	Ice weight 57.5 #/cu ft (page 663 Am. Civil Eng. Hdbk.)
Cables (1½"φ)	7.85	(1860+1207)	167	^{3780}9600	
Cables (½"φ)	4.72	(1375+509)	62	3560	
Pipe	43.2	550	165	9500	
Walkway	60	550	230	13,200	
				35,860#	

Weight of ice per foot of span

$= 65\ \#/\text{ft}$

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Wind load

Assume velocity of 90 m.p.h. which gives a pressure of 20 #/sq.ft. on exposed surface. Use as most dangerous condition of 20 #/sq.ft on bridge when ice covered.

Area of Cables per foot length

$1\frac{1}{2}$"φ = 12 × 1.5 = 18 sq.in. = .125 sq.ft

$\frac{1}{2}$"φ = 12 × $\frac{1}{2}$ = 6 sq.in. = .042 sq.ft

Area of walkway per foot

= .25 (2" ice, $\frac{3}{4}$" gret

and posts)

say .3

Part	Exposed area (sq. ft)	Pressure = (Area x length x 20)	=	
$1\frac{1}{2}$"φ	.125	.125 × [1860 + 1207] [20]	=	7670#
$\frac{1}{2}$"φ	.042	.042 × [1375 + 509] [20]	=	1580#
Pipe	1	1 × 550 × 20	=	11,000#
Walkway	.3	.3 × 550 × 20		3300#
				23,550#

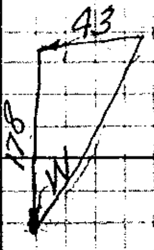
Weight per foot of span = $\frac{23,550}{550}$ = 42.7#

Total unit vertical loads

Pipe	= 65.4#
Cable	= 35.9
walkway	= 11.5
Ice	65.0
	177.8 #/ft.

Total unit horizontal loads

Wind = 42.7 #/ft.



$W = \sqrt{43^2 + 178^2}$

$= \sqrt{1899 + 31,684} = \sqrt{33,533}$

$= 184$ #/ft

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Actual sizes of wire rope.

sag of 37'

Max. tension at tower A & B $= \frac{w}{8h}\sqrt{l^4+16h^2l^2} = \frac{184}{8\times37}\sqrt{550^4+16\times37^2\times550^2}$

$= \frac{184}{296}\sqrt{\cancel{90,000,000,000} \; 91,506,250,000 + 21,904\times302,500}$

$= \frac{184}{296}\sqrt{91,506,250,000 + 6,300,000,000}$

$= \frac{184}{296}\sqrt{97,806,250,000} = \frac{184}{296}\times312,740$

$= \boxed{294,000}\#$ 194,500 #

Use 2 cables —

Each cable takes $\frac{\overset{194,500}{\cancel{294,000}}}{2} = \overset{97,250}{\cancel{147,000}}\#$

Refer to page 701 of Amer. Civil Engrs. H'dbk.

Use plow steel wire rope with an ultimate stress of 190,000#.

Use working stress of $\frac{1}{4}$ × ultimate [page ~~78~~ 702 Amer. Civil Engrs. H'dbk]

$= \frac{1}{4}\times190,000 = 47,500\#$.

Area Cable $= \frac{\overset{97,250}{\cancel{147,000}}}{47,500} = \cancel{3.1}$ sq. in. ~~2"ϕ = 3.1416~~

2.04 sq in needed $1\frac{1}{2}$"ϕ = 1.767 sq in.

2.04 > 1.767

Note: For above loads $1\frac{1}{2}$"ϕ cable is too small; with a sag of 37'.

Check tensile stress with same loads for a sag of 46'

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sag of 46'

Max. tension at towers A & B $= \frac{W}{8h}\sqrt{\ell^4+16h^2\ell^2}$

$= \frac{184}{8\times46}\sqrt{91,506,250,000 + 16\times2116\times302,500}$

$= \frac{184}{8\times46}\sqrt{91,506,250,000 + 10,241,440,000}$

$= \frac{184}{368}\sqrt{161,747,690,000} = \frac{184}{368}\times318,980$

$= 159,500$ #

Use 2 Cables

Each Cable takes $\frac{159,500}{2} = 79,750$ #

Area Cable $= \frac{79,750}{47,500} = 1.68$ 1½" φ Cable satisfactory with 46' sag.

For investigation of tension increase load to 200#/ft. and increase sag from 46' to 55'

Max. tension at towers A & B $= \frac{200}{8\times55}\sqrt{91,506,250,000 + 16\times55^2\times302,500}$

$= \frac{200}{8\times55}\sqrt{91,506,250,000 + 14,641,000,000}$

$= \frac{200}{440}\sqrt{106,147,250,000} = \frac{5}{11}\times 325,803 = 148,000$ #

Each Cable takes $\frac{148,000}{2} = 74,000$ #

Area Cable $= \frac{74,000}{47,500} = 1.56$ sq. in. Use 1½" φ (1.767)

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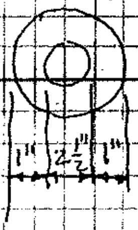
Investigation of $2\frac{1}{2}$"φ Cables for main suspension Cables

Revised loading Cables = 8561 + (1860 × 16.7) = 8561 + 31,100 = 39,661#

Revised weight per foot = $\frac{39,661}{550}$ = 72.2#

Revised loading ice on cables

Area = (.7854)[(4.5²) - (2.5²)] = .7854 × 140 = 11 sq. in.



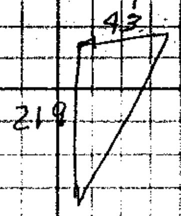
weight = $\frac{(11)(1860)\cancel{(12)}}{\cancel{1728}\ 144}$ × 57.5 = 8160#

Revised loading = $\frac{30,040 + 8160}{550}$ = $\frac{38200}{550}$ = 69.5

Total ~~unit~~ vertical loads

Pipe = 65.4
Cable = 72.2
walkway = 11.5
Ice = 69.5
218.6

Say wind = 43 #/ft



Total W = $\sqrt{43^2 + 219^2}$
= $\sqrt{49,810}$
say 224 #/ft

Max tension at A & B for sag of 37'

$= \frac{224}{8\times37}\sqrt{550^4 + 16\times37^2\times550^2} = \frac{224}{8\times37}\sqrt{97,806,250,000}$

$= \frac{28}{37} \times 312,740 = 240,000$#

Each cable takes 120,000#
Use working stress of 47,500

Area Cable = $\frac{120,000}{47,500}$ = 2.52

Area $2\frac{1}{2}$"φ Cable = 4.9087

4.91 > 2.52

Note: $2\frac{1}{2}$"φ Cable too large. Try 2" to give needed area for span with 37' sag

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Cable size Cont.

Loading w $1\frac{1}{2}''\phi = 184 \frac{\#}{ft.}$

" " " $2\frac{1}{2}''\phi = 224$ Try $204 \frac{\#}{ft}$ for $2''\phi$ Cable

max. tension $= \frac{204}{8 \times 37} \times 312{,}740 = 216{,}000^{\#}$

Each cable takes $108{,}000^{\#}$

Area $= \frac{108{,}000}{47{,}500} = \underline{2.28}$ sq. in needed

$\underline{2''\phi}$ Cable has $\underline{3.14}$ sq. in.

For Crucible steel wire rope and with an ultimate strength from 130,000 to 190,000 #, 2"φ will cover upper range.

minimum stress in cable needed $= \left(\frac{108{,}000}{3.14}\right)4 = \underline{138{,}000}$ #
ultimate strength

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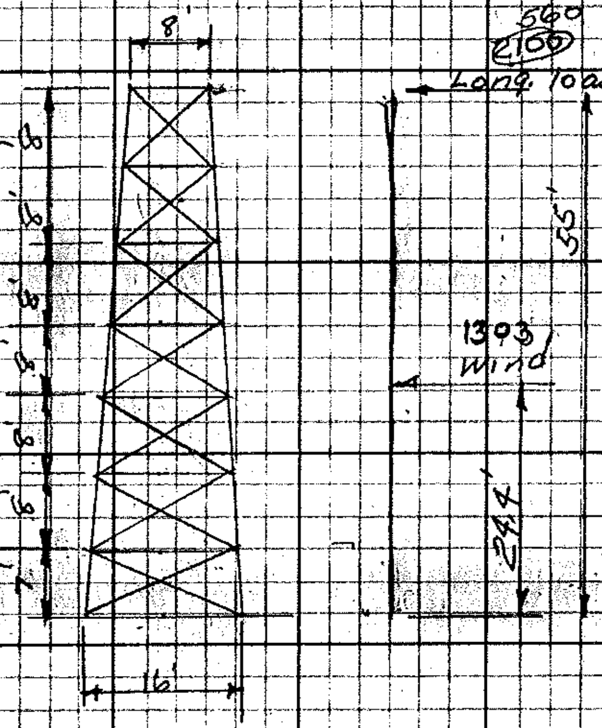
Investigation of stresses on Tower A with 8" legs and with a sag in line of 37' and suspension Cables of 2-2"ϕ



Wind loading - 20 #/sq ft.

Tower legs = 2×55×.67×20 = 1475

Top = 8×.67×20 = 107

Horiz. pieces = 7×13×.17×20 = 310

Diag. = 14×15×.17×20 = 714

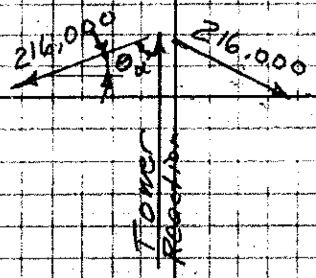
Total = 2606 #

c.g. from top = $\frac{55(32+8)}{3(16+24)} = \frac{55\times40}{\cancel{3\times40}\ 72}$

= ~~[illegible]~~ = 30.6

Arm = ~~55 - [illegible]~~

= 55 - 30.6 = 24.4'



Tangent slope at tower = $\frac{4s}{l} = \frac{4\times37}{550} = .269$

θ = 15°-03'

α = 90° - θ = ~~64°~~ 74°57'

ΣFy = 0

Reaction = 2×216,000 # × cos 74°-57'

= 412,000 #

Longitudinal load = $\frac{\text{Reaction}}{100}$

= ~~4120~~ 1120 #

Note: Half longitudinal load and half wind load act on each column.

Mom. = $\overset{560}{\cancel{2100}}$ × 55 + 1303 × 24.4 = $\overset{30,800}{\cancel{115,600}}$ + 31,800 = $\overset{62,600 \#}{\cancel{147,400}}$

$S = \frac{Mc}{I} = \frac{\overset{62,600}{\cancel{147,000}} \times 12 \times 4.3}{162} = \overset{20,000 \#}{\cancel{46,800 \#}}$

Too high

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Try 10" pipe on Tower A with 37' sag

$\frac{l}{r} = \frac{55 \times 12}{3.63} = 182$

Wind loading at 20#/ft²

Tower legs = 2×55×.83×20 = 1820

Top = 8×.83 × 20 = 133

Bracing = 1024

2977

1499 goes each leg tower

$M = \overset{1499}{\cancel{2977}} \times 24.4 + 560 \times 55$

$= \overset{31,300}{\cancel{72,600}} + 30,800$

$= 62,100$ '#

$S = \frac{Mc}{I} = \frac{62,100 \times 12 \times 5.37}{211.9}$

$= 18,900\psi$

$S = \frac{P}{a} = \frac{56,000}{16.10} = 3480\psi$

$\frac{f_a}{F_a} + \frac{f_b}{F_b} < 1$

$\frac{3480}{4370} + \frac{18,900}{20,000}$

$.795 + .957 > 1$ member is overstressed

Try 12" pipe

$\frac{l}{r} = \frac{55 \times 12}{4.34} = 152$

Wind loading at 20#/ft²

Tower legs = 2×55×20 = 2200

Top = 8×20 = 166

Bracing = 1024

3380

of which 1690 goes each tower leg

$M = 1690 \times 24.4 + 30,800$

$= 41,200 + 30,800$

$= 72,000$ '#

$S = \frac{Mc}{I} = \frac{72,000 \times 12 \times 6.37}{361.5} = 15,250\psi$

$S_2 = \frac{P}{a} = \frac{56,000}{19.24} = 2920\psi$

$\frac{2920}{6,620} + \frac{15,250}{20,000} < 1$

$.44 + .76$

$1.20 > 1$ member is overstressed

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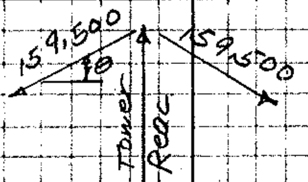
Investigate sag of 46'. Try 12" pipe Tower becomes 64' high

$\frac{l}{r} = \frac{64 \times 12}{4.34} = 177$

$\tan \theta = \frac{4 \times 46}{550} = \frac{184}{550} = .334$

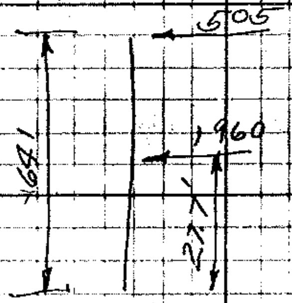


θ = ~~19°-31'~~ 18°-28'

α = 90 − ~~19-31~~ = ~~70-29~~ ; 89 − 18-28 = 71-32'

Reacton = 2 × 159,500 cos 71°-32'
= 319,000 × .3167 5
= 101,000 #

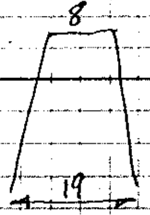
Long. load = 1010
Note Half or 505 goes to each leg tower



c.g. from top
$= \frac{64(38+8)}{3(19+8)} =$
$= \frac{64 \times 46}{3 \times 27} = 36.3$

Arm = 64 − 36.3 = 27.7

Wind load.
Tower = 2 × 64 × 20 = 2560
Top = 8 × 20 = 160
Bracing = 1200
3920 #



Mom. = ~~3920~~ 1960 × 27.7 + 505 × 64
= ~~108,600~~ 54,300 + 32,200 = ~~140,800~~ 86,500 '#

$S_1 = \frac{Mc}{I} = \frac{\cancel{140,800}\ 86,500 \times 6 \times 12}{361.5}$
= 17,200 ϕ

$S_2 = \frac{P}{a} = \frac{50500}{19.24} = 2630$ ϕ

$\frac{2630}{4700} + \frac{17,200}{20,000}$
.56 + .8671
1.4271

member overstressed

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Investigate 16" pipe 1" thick for tower A with a sag of 46'

$\frac{l}{r} = \frac{64 \times 12}{5.3} = 145$ $r = \sqrt{\frac{256+196}{4}} = \frac{\sqrt{452}}{4} = \frac{21.3}{4} = 5.3$

$I = .049087\left[16^4 - 14^4\right] = .09908\ 7(27,120)$

$= 1331\ in^4$

Wind loading at 20 #/ft²

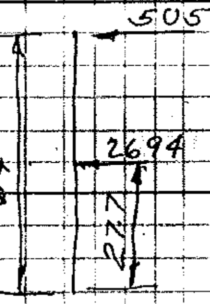
Tower legs = ~~226~~ $2560 \times \frac{4}{3} = 3410$

area = $.78539\ (16^2 - 14^2) = .785398(60)$

Top = $160 \times \frac{4}{3} = 213$ = 47.2 sq. in.

3" Bracing: $1200 \times 1.47 = 1765$

5388 of which 2694 goes each tower leg.



moment = $64 \times 505 + 2694 \times 27.7$

$= 32,300 + 71,900$

$= 104,200$

$S_1 = \frac{mc}{I} = \frac{104,200 \times 8 \times 12}{1331} = 7510\#$

$S_2 = \frac{50,500}{47.2} = 1070\#$

$\frac{1070}{7260} + \frac{7510}{20,000} < 1$

$.147 + .376 < 1$

$.523 < 1$

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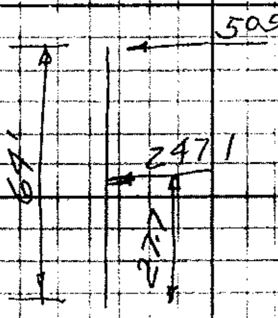
Investigate 14" pipe 5/8" thick for Tower A with eq. value 46'

$I = 588.5 \text{ in}^4$ $\frac{l}{r} = \frac{64 \times 12}{4.734} = 163$

$r = 4.734"$

Area = 26.34



Wind loading

Tower legs = $2560 \times \frac{7}{6} = 2990$

Top = $160 \times \frac{7}{6} = 187$

3" Bracing = 1765

4942

of which 2471 goes each tower leg.

moment = 2471 × 27.7 + 505 × 64

= 58,400 + 32,300

= 90,700 '#.

$S_1 = \frac{Mc}{I} = \frac{90,700 \times 7 \times 12}{588.5} = 12,950 \#$

$S_2 = \frac{P}{a} = \frac{50,500}{26.34} = 1920 \#$

$\frac{1920}{5,710} + \frac{12,950}{20,000} < 1$

$.336 + .646 < 1$

$.982 < 1$ O.K.

Try 14" pipe $\frac{5}{8}$" thick for tower A with 37' sag

$I = 588.5 \text{ in}^4 \quad A = 26.34 \text{ sq. in.}$

$r = 4.734$

$\frac{l}{r} = \frac{55 \times 12}{4.734} = 140$

Wind loading

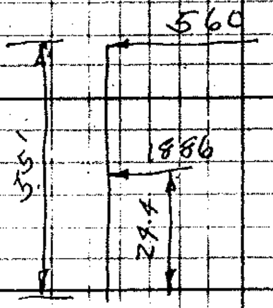
Tower legs = $\frac{7}{6} \times 2200 = 2560$

Top = $\frac{7}{6} \times 160 = 187$

Bracing = 10 1024

3771

of which 1886 goes each tower leg.



$M = 29.4 \times 1886 + 560 \times 55$

$= 46,000 + 30,800$

$= 76,800$ '#

$S = \frac{Mc}{I} = \frac{76,800 \times 12 \times 7}{588.5} = 11,000\#$

$S_2 = \frac{P}{a} = \frac{56,000}{26.34} = 2130\#$

$\frac{2130}{7,760} + \frac{11,000}{20,000} < 1$

$.274 + .55 < 1$

$.82 < 1$ O.K.

Try 14" pipe $\frac{1}{2}$" thick for same conditions as above

$I = .049087 [\overline{14}^4 - \overline{13}^4]$

$= .049087 [9,855]$

$= 483.75 \text{ in}^4$

$A = .785398 (\overline{14}^2 - \overline{13}^2)$

$= .785398 (27)$

$= 21.21$ sq. in.

$r = \sqrt{\frac{\overline{14}^2 + \overline{13}^2}{4}}$

$= \sqrt{\frac{365}{4}}$

$= 4.78$

$\frac{l}{r} = \frac{55 \times 12}{4.78}$

$= 138$

$S = \frac{Mc}{I} = \frac{907000 \times 12 \times 7}{483.75} = 15,800\#$

$S_2 = \frac{P}{a} = \frac{56,000}{21.21} = 2640\#$

$\frac{2640}{7960} + \frac{15800}{20000} < 1$

$.343 + .78 > 1$

$1.12 > 1$ Overstressed

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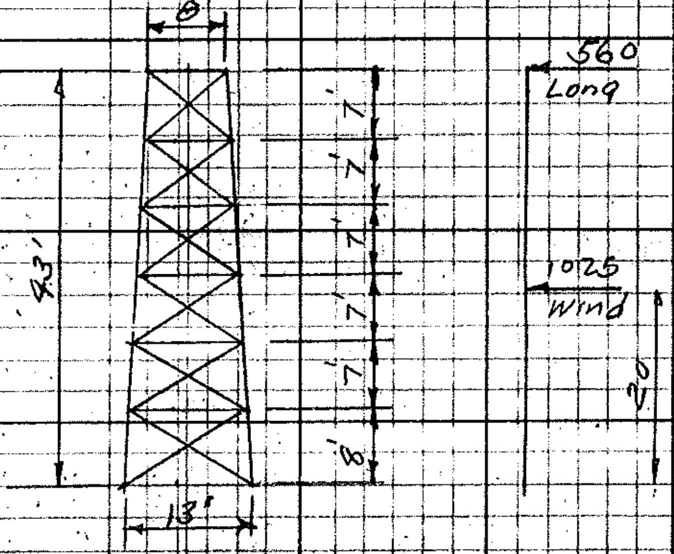
Tower leg determination for Tower B for a sag of 37'

Try 8" pipe

c.g. from top $= \frac{43(26+8)}{3(13+8)} = \frac{43 \times 34}{3 \times 21}$

$= 23.1$

Arm from bottom $= 43 - 23 = 20'$



Wind loading 20#/ft²

Tower legs $= 2 \times 43 \times 0.67 \times 20 = 1150$

Top $= 8 \times 0.67 \times 20 = 107$

Bracing:

Horz $= 7 \times 11 \times 17 \times 20 = 262$

Diag $= 12 \times 13 \times 17 \times 20 = 531$

2050

of which 1026 goes each tower leg

moment $= 560 \times 43 + 1025 \times 20$

$= 24{,}100 + 20{,}500$

$= 44{,}600$ '#

$S_1 = \frac{M c}{I} = \frac{44{,}600 \times 12 \times 4.3}{162} = 14{,}200$ #

$S_2 = \frac{P}{a} = \frac{56{,}000}{21.30} = 2630$ #

$\frac{l}{r} = \frac{43 \times 12}{2.76} = 187$

$\frac{2630}{4070} + \frac{14200}{20{,}000} < 1$

$.646 + .71 > 1$

$1.36 > 1$ Overstressed

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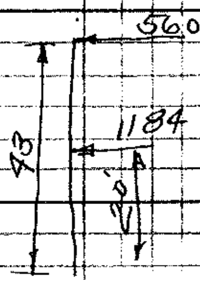
Investigation of 10" pipe for Tower B for sag of 37'

$\frac{l}{r} = \frac{43 \times 12}{3.63} = 142$

Wind loading at 20 #/sq'.
Tower legs = 1150 × 5/4 = 1440
Top = 107 × 5/4 = 134
Bracing = 793
2367
of which 1189 goes each tower leg.



Moment = 43 × 560 + 1184 × 20
= 24,100 + 23,680
= 47,780 '#

$S_1 = \frac{Mc}{I} = \frac{47,780 \times 5.37 \times 12}{211.9} = 14,500\#$

$S_2 = \frac{P}{a} = \frac{56,000}{16.10} = 3470\#$

$\frac{3470}{7560} + \frac{14500}{20000} < 1$

.46 + .72 > 1

1.18 > 1

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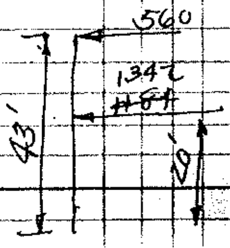
Investigation of 12" pipe for tower B for sag of 37'

$\frac{l}{r} = \cancel{56}\ \frac{43 \times 12}{4.34} = 119$



Wind loading

Tower legs $= 1440 \times \frac{6}{5} = 1730$

Top $= 134 \times \frac{6}{5} = 161$

Bracing $= 793$

2684

of which 1342 goes to each tower leg.

$\text{Moment} = 43 \times 560 + 1342 \times 20$

$= 24{,}100 + 26{,}840$

$= 50{,}940$ '#

$S = \frac{Mc}{I} = \frac{50{,}940 \times 6.37 \times 12}{361.5} = \overset{10{,}800}{\cancel{11{,}750}}\ \#$

$S_2 = \frac{P}{a} = \frac{56{,}000}{19.24} = 2916\ \#$

$\frac{2910}{10{,}130} + \frac{\overset{10{,}800}{\cancel{11{,}750}}}{20{,}000} < 1$

$.288 + \overset{54}{\cancel{.59}} < 1$

$\overset{83}{\cancel{.88}} < 1$ member O.K

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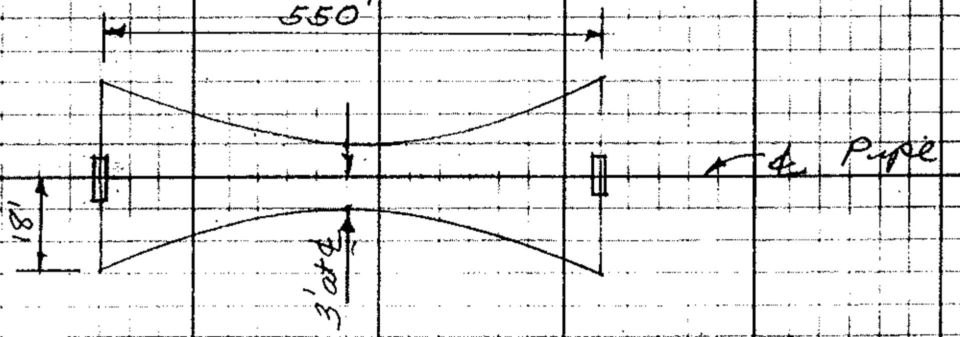
Main Cables for lateral sway bracing

Tension at point 18' from ℄ pipe at towers.



Horizontal sag of Cable = 18 - 3' = 15' = h

Use 43 #/lin. ft. of span = w

Max tension $= \frac{w}{8h}\sqrt{l^4 + 16 h^2 l^2}$

$= \frac{43}{8 \times 18}\sqrt{91{,}506{,}250{,}000 + 16 \times \overline{15}^2 \times \overline{550}^2}$

$= \frac{43}{144}\sqrt{91{,}506{,}250{,}000 + 1{,}080{,}000{,}000}$

$= \frac{43}{144}\sqrt{92{,}586{,}250{,}000} = \frac{43}{144} \times 304{,}280$

= 91,000#

Use working stress in cable of 47,500 #

Area Cable $= \frac{91{,}000}{47{,}500} = 1.91$ sq. in needed

Use $1\frac{5}{8}$" φ with an area of 2.07 sq. in.

Use $1\frac{5}{8}$" φ rope made of plow steel and with an ultimate strength of 2.07 × 47,500 = 98,500#

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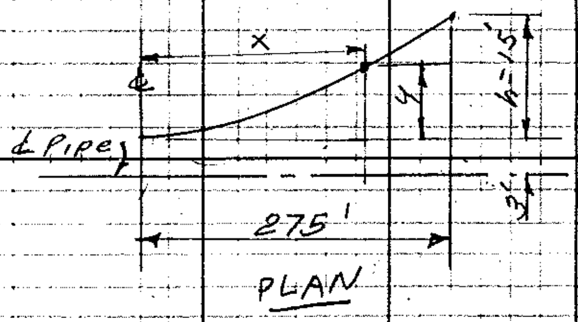
Coordinates for lateral sway bracing cable



x coordinates measured from ℄
y coordinates measured from bottom of sag.

$$y = \frac{4h}{l^2} x^2$$

y ft	h ft	$4h$ ft	$\frac{4h}{l^2}$ ft	x	x^2
.08	15	60	.000198	20	400
.32	"	60	" "	40	1600
.71	"	60	" "	60	3600
1.27	"	60	" "	80	6400
1.98	"	60	" "	100	10,000
2.85	"	60	" "	120	14,400
3.88	"	60	" "	140	19,600
5.06	"	60	" "	160	25,600
6.41	"	60	" "	180	32,400
7.92	"	60	" "	200	40,000
9.60	"	60	" "	220	48,400
11.4	"	60	" "	240	57600
13.4	"	60	" "	260	67600
15'	15	60	" "	275	75,625

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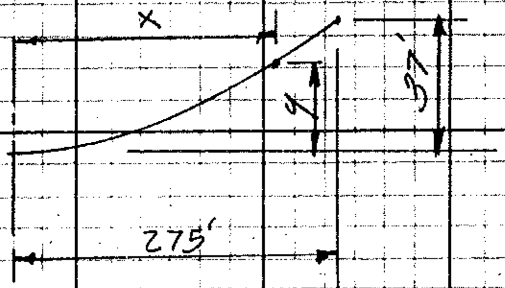
Coordinates for main suspension Cables when pipe is level.



$y = 4\frac{h}{l^2}x^2$

$\frac{4h}{l^2} = \frac{148}{302,500} =$

y	h	4h	4h/l2	x	x²
.196	37'	148	.000489	20	400
.781	"	"	"	40	1600
1.76	"	"	"	60	3600
3.13	"	"	"	80	6400
4.89	"	"	"	100	10,000
7.05 ~~8.49~~	"	"	"	120	14,400
9.68	"	"	"	140	19,600
12.5	"	"	"	160	25,600
15.8	"	"	"	180	32,400
19.6	"	"	"	200	40,000
23.6	"	"	"	220	48,400
28.2	"	"	"	240	57,600
33.1	"	"	"	260	67,600
37	"	"	"	275	75,626

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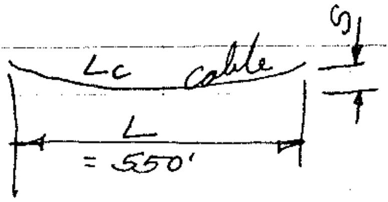
Sag on pipe for laying

Assume pipe is level at 0° F which gives a sag of 37' in cable for which a max. tension has been computed.

Refer to page 1296 of Amer. Civil Eng. Hdbk.



$$L_c = L + \frac{8S^2}{3L} = 550 + \frac{8\times 37^2}{1650}$$

$$= 550 + \frac{8\times 1369}{1650} = 550 + \frac{10,952}{1650}$$

$$= 550 + 6.64 = 556.64'$$

Increase in length due to temp

of 20° = 556.64 × .0000095 × 20 = .1057'   ~~New length 556.64~~
of 40° = " × " × 40 = .2111'
of 60° = " × " × 60 = .3167'
of 80° = " × " × 80 = .4222'
of 100° = " × " × 100 = .5278' $S = \sqrt{\frac{(L_c - L)(3L)}{8}}$
of 120° = " × " × 120 = .6333'

New length 20° = 556.64 + .11 = 556.75 $S = \sqrt{\frac{(6.75)(1650)}{8}} = \sqrt{1392.2} = 37.31$
" " 40° = " " + .21 = 556.85 $= \sqrt{1412.8} = 37.59$
" " 60° = " " + .32 = 556.96 $= \sqrt{1435.5} = 37.89$
" " 80° = " " .42 = 557.06 $= \sqrt{1456} = 38.16$
" " 100° = " " .53 = 557.17 $= \sqrt{1479} = 38.46$
" " 120° = " " .63 = 557.27 $= \sqrt{1500} = 38.73$

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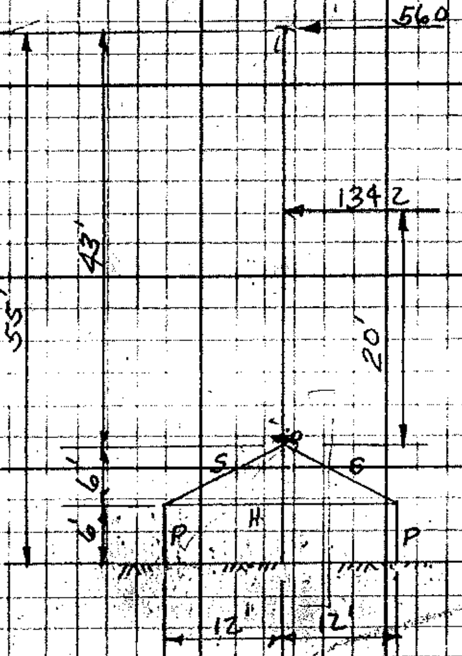
Revised tower design for tower A with bracing to reduce pipe size from 14" to 12"



Assume bracing is strong enough and rigid so that tower bends from point 0 instead of ground.

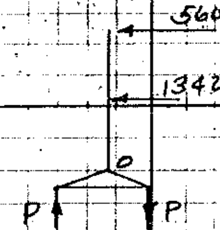
Then wind loading =

Tower legs = 1440 × 6/5 = 1730
Top = 134 × 6/5 = 161
Bracing = 793
2684

of which 1342 goes to each tower leg.

Use 8" pipe can take ~~115,000~~ 109,000# p 248 AISC

make space frame by cutting posts —



$\Sigma M_0 = 0$

Compression in one post equals tension in other

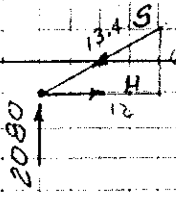
$2 \times P \times 12 = 560 \times 43 + 1342 \times 20$

$24P = 24{,}100 + 26{,}840 = 50{,}940$

$P = 2080$ #

Try 3" pipe posts.

$\frac{l}{r} = \frac{6 \times 12}{1.16} = 62$ Pipe can take 33,000# O.K



$\Sigma F_V = 0$

$S = 2080 \times \frac{13.4}{6}$

$= 4650$ #

$\frac{l}{r} = \frac{13.4 \times 12}{1.16}$

$= 139$

3" pipe capable carrying 18,000 #

$\Sigma F_H = 0$

$H = 4650 \times \frac{12}{13.4} = 4160$ #

$\frac{l}{r} = \frac{12 \times 12}{1.16} = 124$

3" pipe capable carrying 21,000 #

Refer to page 248 AISC Hdbk for allowable concentric

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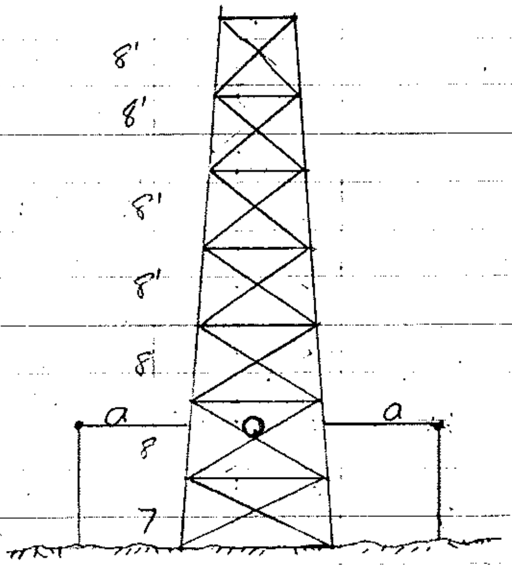
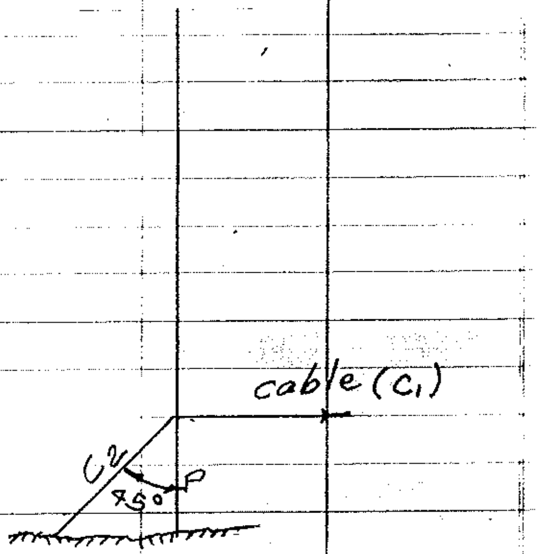
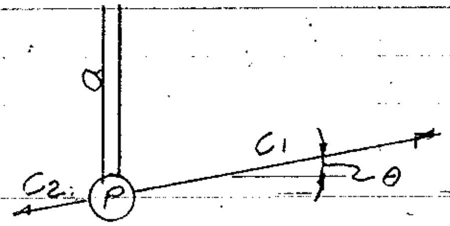
Lateral sway anchorage (cable)



FRONT VIEW



SIDE VIEW



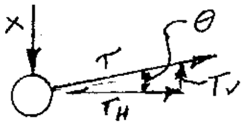
PLAN

$\tan\theta = \frac{4s}{l} = \frac{4 \times 15}{550} = \frac{60}{550} = .109$

$\theta = 6°\text{-}13'$

Investigate Condition as if Cable C_2 was not working

$x = T_V = T \sin\theta$



$= 91{,}000 \times .10829$

$= 9850\#$

9850 bears on connecting pipe x

Try 3" pipe for x

$\frac{l}{r} = \frac{12 \times 12}{1.16} = 124$

pipe Can Carry 33,000#

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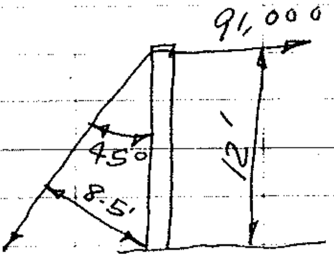


$12^2 + 12^2 = 288$

$\frac{1}{2}$ diag $= \frac{17}{2} = 8.5'$

Arm $= 8.5'$

$8.5 \times \cancel{12} \times C_2 = 91,000 \times 12, \times \cancel{12}$

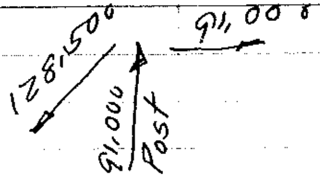
$C_2 = \frac{91,000 \times 12}{8.5} = 128,500$

Using a working stress of 47,500 # for the cable to ground

$A_c = \frac{128,500}{47,500} = 2.7$ sq. in.

2" ϕ Cable gives 3.19 sq. in.

Stress in post



$\Sigma F_v = 0$

Post stress $= 128,500 \times .707 = 91,000^{\#}$

Try 8" pipe

$\frac{l}{r} = \frac{12 \times 12}{2.76} = 52$

Standard 8" pipe can take 115,000 to 133,000#.

D O   N O T   W R I T E   I N   T H I S   S P A C E

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WORK Pipe supports.

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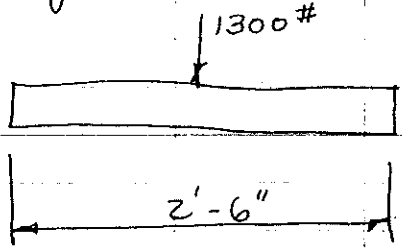
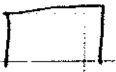
Place pipe supports every 11' along line

Each support Carries	11' of pipe @	65.4 #/ft
" " "	11 of walkway @	11.5 #/ft
" "	ice on pipe @	17.3 #/ft
" " "	ice on walkway @	24. #/ft
		118.2 #/ft

Total weight = 11 × 118.2 = 1300#



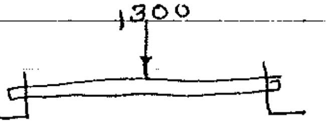


$$\text{moment} = \frac{1300 \times 2.5 \times \overset{3}{\cancel{12}}}{\cancel{4}} = 1300 \times 7.5 = 9750\#$$

$$\frac{I}{c} = \frac{M}{S} = \frac{9750}{20000} = .487 \text{ in}^3$$

Use 6" [@ 8.2#

Roller supports



Each angle has to Carry 650#
Try angle 2 × 2 × 1/4" with 1/2" φ bar as roller.

$$S = \frac{650}{.5 \times .25} = \frac{650}{.125}$$

$$= 5200\# \text{ O.K.}$$

shear in 1/2" bar

$$= \frac{650}{.1963} = 3300\#$$

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Size of suspender Cables

$w = 118.2$ #/ft.

Load Cables Carry = 118.2 × 550 = 65,000#

Number suspenders = 100

Each Cable Carries $\frac{65000}{100}$ = 650#

Use $\frac{3}{8}$"φ suspenders

Sway Cable ties.

$w = 43 \frac{\#}{ft}$ (wind))

Load = 550 × 43 = 23,600#

number ties = 50

Each Cable Carries $\frac{2\overset{3}{3}60\cancel{0}}{5\cancel{0}}$ = 472#

Use $\frac{3}{8}$"φ suspenders

Bracing in Towers   Try 3" pipe

greatest $\frac{l}{r} = \frac{15 \times 12}{1.16} = \frac{180}{1.16} = 155$

Refer to page 248 AISC Hdbk.
Pipe Can Carry 17,000#
Use 3"

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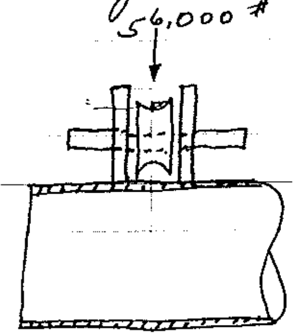
Design of saddle

56,000 # acts down on ea. sheave
Pin is under double shear. Casting is under single bearing.



$A_{pin.} = \frac{P}{S} = \frac{56,000}{32,000} = 1.75$ sq. in.

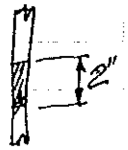
Use 2" pin.

$1\frac{1}{2}$"φ gives 1.767 sq. in.

Use 2"φ pin for extra protection.

Each part of Casting takes $\frac{56,000}{2}$ = 28,000.#

See page 33



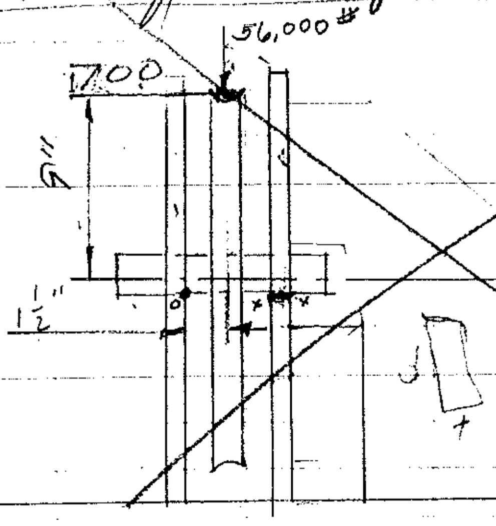
Plan- one casting

2 × t × 32,000 = 28,000

64,000 t = 28,000

$t = \frac{28,000}{64,000} = \underline{.437}$"

Use $\frac{1}{2}$" plates for Castings:



Refer to page 31 and revise plates to allow for wind load on Cables. At worst condition wheel and pin pivot at 0

M = 8620 × 9 + 56,000 × 2
= 77,580 + 112,000 = 187,580"#

Area bearing area x-x = $\frac{187,580}{20,000}$
= 9.38 sq. in.

Revise see ~~sheet~~ page ㉝

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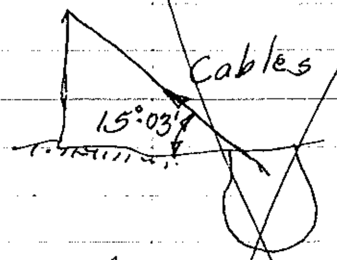
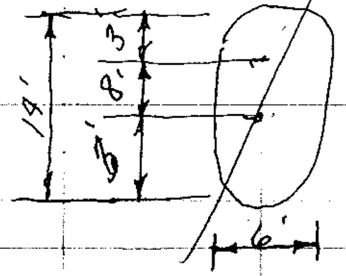
Anchorage for backstays Tower A & B Detail C



Vertical force lifting up
= tension in two Cables × sin 15°-03'
= 216,000# × .25966
= 56,200 #

Elevation



Depth Conc = $\frac{56,200}{6 \times 14 \times 150}$ = $\frac{56,200}{11,200}$ = 5'

Use 6' depth.

See Pg 36

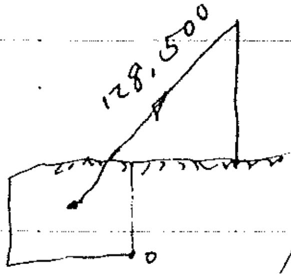
Area of eye-bar.

Each bar has to resist a tension of 108,000#

$a_b = \frac{108,000}{20,000} = 5.4$ sq. in.

5" bar 2" thick gives = 10 sq. in.

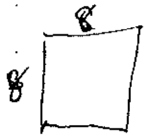
Lateral anchorage Tower A.



Use 2"ϕ cable and eyebar

$a_b = \frac{128,500 \times .707}{20,000} = \frac{91,000}{20,000}$ = 4.6 sq. in.

Depth Conc. = $\frac{91,000}{64\ ~~38~~ \times 150}$ = ~~379~~ = 9.5' say 10



$\Sigma M_o = 0$

128,500 × 6 = 8 × 8 × d × 150 × 4

d = 771,000 = 20.1

See Pg 37

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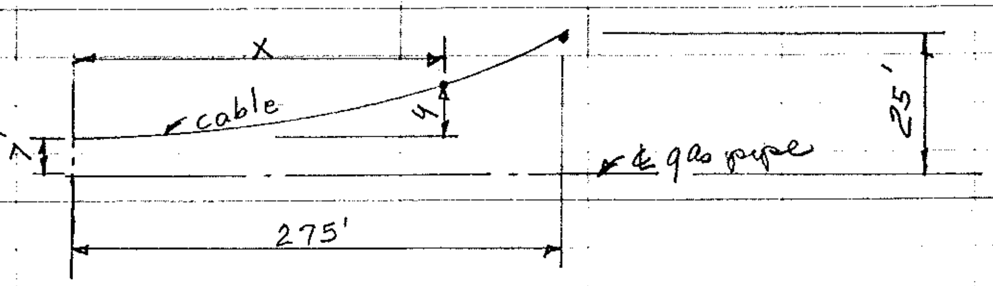
Lateral bracing cable - coordinates and tension

Anchorage points at Towers A & B were changed from 18-
from ℄ of gas pipe to 25-0" by DLB with a change
to 7' from ℄ ~~gp~~ gas pipe at ℄ span. This change
sag ratio and change tension.

new Coordinates in plane along axis of cable



h = 25 - 7 = 18'

y ft	h ft.	4h ft.	4h/l²	x	x²
.1	18'	72	.0002379	20	400
.38	"	"	"	40	1600
.855	"	"	"	60	3606
1.52	"	"	"	80	6400
2.38	"	"	"	100	10,066
3.49	"	"	"	120	14,400
4.63	"	"	"	140	19,600
6.07	"	"	"	160	25,600
7.7	"	"	"	180	32,400
9.5	"	"	"	200	40,000
11.5	"	"	"	220	48,400
13.7	"	"	"	240	57,600
15.9	"	"	"	260	67,606
18	"	"		275	75,625

~~Computed tension with assumption load is pulling~~

l = 550'
w = 43 #/ft
h = 18

Computed tension with assumption load is pulling evenly against Cable. ~~Refs~~ Formula in book references, w = 43 #/ft. page 6

max. tens. = $\frac{w}{8h}\sqrt{l^4 + 16h^2 l^2} = \frac{43}{8 \times 18}\sqrt{91,506,250,000 + 16 \times 324 \times 302,500}$

$= \frac{43}{144}\sqrt{91,506,250,000 + 1,568,160,000} = \frac{43}{144}\sqrt{93,074,410,000}$

$= \frac{43}{144} \times 305,081 = 91,000\#$ ← Use 1¾" cable

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Check for need of bracing for Tower A in plane of tower

It was decided with talk with DLB that there would be a force acting on top of the towers due to wind action on the main suspension cables and the suspenders. For this check the force acting on top of the tower leg will be the total of the wind force acting on one half of one suspension cable and the total of the wind forces acting on one half the suspenders ~~or~~ in one half the span. These loads apply to one pipe leg. Wind at 20 #/□'

Refer to page 3 - One half length one suspension cable equals 930/2 = 465'

Suspenders at 11' intervals -

$$\text{Length} = \frac{550'}{11'}\left[\frac{44+6}{2}\right] = 1250' \text{ in one span}$$

$$\text{length acting on one tower} = \frac{1250}{4} = \boxed{313'}$$

Wind loads on cables

Max. Condition 2"φ Cable with 1" ice (2"+1"+1") = 4" = .33'
Wind load = 465 × .33 × 20 = 3070 #

Min. Condition (2"φ cable) no ice
Wind load = ~~16~~ 465 × .167 × 20 = 1550 #

Max. Condition 3/8"φ Cable with 1" ice (3/8"+1"+1") = 2.375" = .198'
Wind load = 313 × .198 × 20 = 1250 #

Min Condition 3/8"φ Cable no ice 3/8" = .0313'
Wind load = 313 × .0313 × 20 = ⓪71# 196#

Total maximum wind load = 3070 + 1250 = 4320#

Total minimum wind load = 1550 + 196 = 1746#

See pg. 33 & 34

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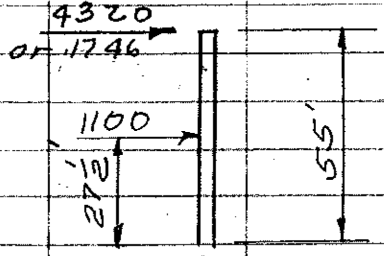
Try loads determined previous page on tower with no bracing.



Wind load on 12" tower leg
$= 55 \times 1 \times 20 = 1100^{\#}$

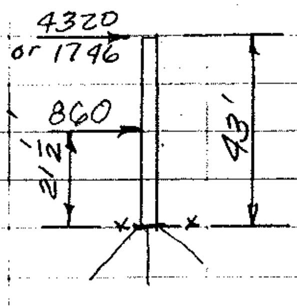
Max. Mom $= 4320 \times 55 + 1100 \times 27.5$
$= 238{,}000 + 30{,}200 = \underline{268{,}000}\ ^{1\#}$

Min mom $= 1746 \times 55 + 1100 \times 27.5$
$= 96{,}000 + 30{,}200 = \underline{126{,}200}\ ^{1\#}$

$S_{max} = \frac{Mc}{I} = \frac{268{,}000 \times 12 \times 6.37}{361.5} = \underline{56{,}700\#}$ Too high

$S_{min} = \frac{Mc}{I} = \frac{126{,}200 \times 12 \times 6.37}{361.5} = \underline{26{,}800\#}$. Too high

Put in braces similar to tower legs in other plane. This will reduce moment arm to 43'. Maximum wind will probably not come with maximum icing conditions



Wind load on 12" pipe
$= 43 \times 1 \times 20 = 860^{\#}$

Max mom $= 4320 \times 43 + 860 \times 21.5$
$= 186{,}000 + 18{,}500 = \underline{204{,}500}\ ^{1\#}$

Min. mom $= 1746 \times 43 + 860 \times 21.5$
$= 75{,}100 + 18{,}500 = \underline{93{,}600}\ ^{1\#}$

$S_{max} = \frac{Mc}{I} = \frac{204{,}500 \times 12 \times 6.37}{361.5} = \underline{43{,}300\#}$

$S_{min} = \frac{Mc}{I} = \frac{93{,}600 \times 12 \times 6.37}{361.5} = \underline{19{,}800\#}$

See Pg 33 & 34

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REVISION

Towers are to be checked as a bent. The bent is to be computed as a stiff frame fixed at the base. The loads computed on page 32 and 33 are to be doubled, since those loads represent 1/4 of one suspension rope and suspenders.

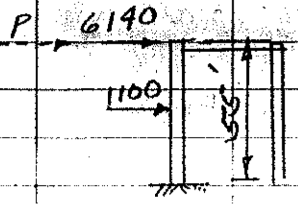
Refer Pg. 31 & 32.

Then max. reaction from 2"φ cable with ice on tower = 3070 × 2 = 6140 #

min. reaction from 2"φ cable with no ice = 1550 × 2 = 3100 #

max. reaction from 3/8"φ cables with ice = 1250 × 2 = 2500 #

min. reaction from 3/8"φ cables no ice = 196 × 2 = 398 #



To compute bent get equivalent loading for wind load on tower leg and on cables.

$P_{max.\,2"\phi} = \frac{55 \times 6140 + 1100 \times 27.5}{55} = 6140 \times 550 = \underline{6690}\ \#$

$P_{min.\,2"\phi} = \frac{3100 \times 55 + 1100 \times 27.5}{55} = 3100 + 550 = \underline{3650}\ \#$

$P_{max\,\frac{3}{8}"\phi} = \frac{2500 \times 55 + 1100 \times 27.5}{55} = 2500 + 550 = \underline{3050}\ \#$

$P_{min\,\frac{3}{8}"\phi} = \frac{398 \times 55 + 1100 \times 27.5}{55} = 398 + 550 = \underline{948}\ \#$

$\text{Total } P_{ice} = P_{max\,2"\phi} + P_{max\,\frac{3}{8}"\phi} = 6690 + 3050 = \underline{9740}\ \#$

$\text{Total } P_{no\ ice} = P_{min\,2"\phi} + P_{min\,\frac{3}{8}"\phi} = 3650 + 948 = \underline{4598}\ \#$

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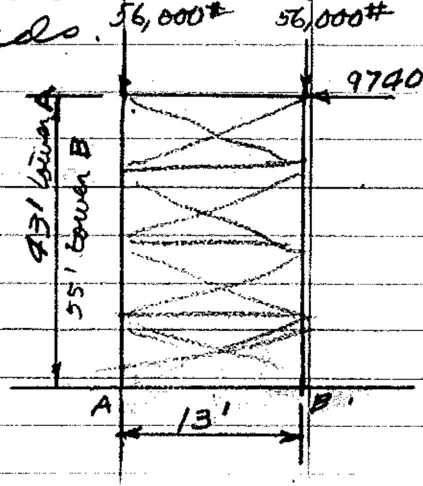
Reference to calculations on towers - pg. 34 A by W.R.C.
for loads.

Tower A



Reaction at A taking moments about B =

$$= 56,000 + \frac{9740 \times 43}{13} = 56,000 + 31,200 = 87,200\#$$

Direct compressive stress in tower leg $= \frac{87,200}{19.24} = 4530\#/\square''$

With extremely low direct stress, bending in trussed tower will not be critical so tower A is

Tower B.

Reaction at A taking moments about B

$$= 56000 + \frac{9740 \times 55}{13} = 56,000 + 41,000 = 97,000\#$$

Leg on windward side of tower is still in compressio
so extra brace transverse to tower at base not
required & will be eliminated.

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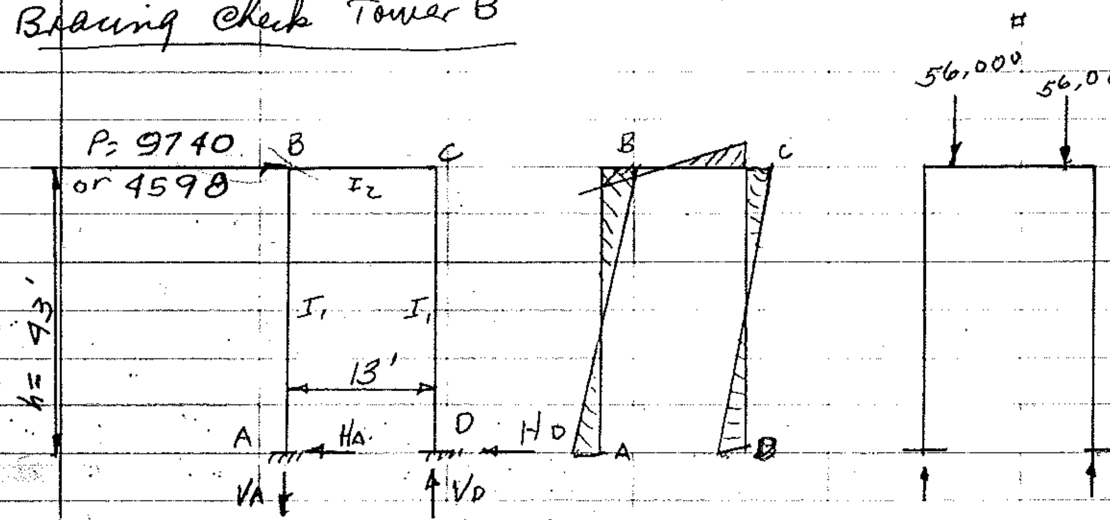
Bracing check Tower B



Void - assumptions not in agreement with actual conditions

Direct stress each tower leg

$S_1 = \frac{P}{a} = \frac{56,000}{19.24} = 2910\#$

$\frac{l}{r} = \frac{43 \times 12}{4.34} = 119$

Allowable stress = 10,130#
Page 209 A.I.S.C. Hdbk.

Refer to Fig 8 page 693 Milo S Ketchum Structural Engs. Hdbk.

$K = \frac{I_2}{I_1} \cdot \frac{h}{l} \quad I_2 = I_1$

$M_A = \frac{Ph}{2} \cdot \frac{1+3K}{1+6K}$

$M_A = M_D$
$M_B = M_C$

$K = \frac{43}{13} = 3.3$

$M_B = \frac{Ph}{2} \cdot \frac{3K}{1+6K}$

M_A is maximum

$M_{A\,(1''\,ice)} = \frac{Ph}{2} \cdot \frac{1+3K}{1+6K} = \frac{9740 \times 43}{2} \times \frac{(1+9.9)}{(1+19.8)} = 4870 \times 43 \times \frac{10.9}{20.8}$

$= 110,000'\#$

$M_{A\,(no\ ice)} = \frac{4598 \times 43}{2} \times \frac{10.9}{20.8} = 51,600'\#$

$S_2\ max = \frac{Mc}{I} = \frac{110,000 \times 12 \times 6.37}{361.5} = 23,300\#$

$S_2\ min. = \frac{Mc}{I} = \frac{51,600 \times 12 \times 6.37}{361.5} = 10,950\#$

Maximum

$\frac{2910}{10130} + \frac{23,200}{20,000} < 1$

$\cancel{.99}$
$.29 + 1.16 > 1$

Minimum

$\frac{2910}{10130} + \frac{10,950}{20,000} < 1$

$.29 + .55 < 1$

$.84 < 1$

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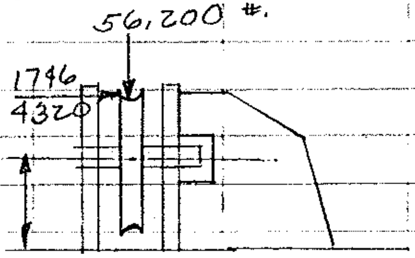
Saddle Detail

Each saddle has to carry 56,000# direct load from cable and a horizontal load acting on top of sheave which comes from a wind load on the cables. This varies from 1746# to 4320#.



Combined load = $\sqrt{56200^2 + 4320^2}$

= $\sqrt{\cancel{2,500,000,000}\ 3,170,000,000 + 16,000,000}$

= ~~$\sqrt{2,516,000,000}$ =~~

= $\sqrt{3,186000,000}$ = 56,400#

Pin is under double shear, each plate casting is under single bearing and shearing

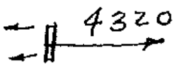
Area pin = $\frac{56,400}{32,000}$ = 1.76 sq. in.^{2} Use $1\frac{3}{4}$" ϕ pin (2.04 sq. in.)

Each ~~pin~~ plate casting takes ~~approx~~ 30,000#

2t 13,000 = 30,000
13000 t = 15000
t = 1.76"

Use $1\frac{1}{4}$" plates.

Bolt in pin
Wind stress will cause bolt to be under double shear.



Area bolt = $\frac{4320}{40,000}$ = .108 sq. in.

Use $\frac{1}{2}$" ϕ bolt

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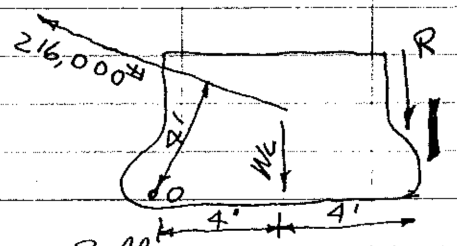
Revision of anchorages

Backstay anchorages Towers A & B

A 5'-0" × 17'-0" × 8'-0" anchorage was taken anchored 3' deep in rock. The Concrete in the anchorage alone is not enough to resist pull of cables. Purpose of check is to find out stress on rock above anchorage.



Refer to sheet 112 of drawings for dimensions. Lever arms scaled off.

~~Tension~~ Pull Cables = 2 × 108,000 = 216,000#

W_c = Wt. Conc. = 8 × 17 × 5 × 150 = 102,000#

ΣM_o

+216,000 × 4 ~~+~~ 102,000 × 4 = 456,000 '# rock has to provide

8R = 456,000

R = 57,000#

Shear area of rock approx. $2\frac{1}{2}$' deep and 17' long.

$v_{rock} = \frac{57,000}{2.5 \times 17 \times 144} = \underline{9.3\#}$

Refer to page 346 AISC Hdbk. Shearing stress limestone or sandstone = $\underline{150\#}$

Anchorage slight

Steel - temperature = .3 to .4 % cross sectional area.

~~.004 .003 × 12 × 5 × 12 = 2.16 sq. in.~~

Use $\frac{7}{8}$" ϕ @ 12" ea way

Pin diam. in Conc.

area = 108,000 = 5.4 sq. in. Use $2\frac{1}{2}$" ϕ bars

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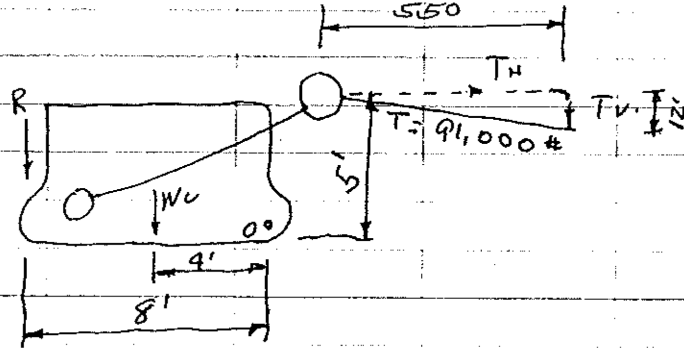
Revision of anchorages

Lateral anchorages Towers A & B

See preceding page. Refer to sheet 112 of drawings for dimension.



Angle is so small that ~~assume~~ take TH as 91,000# acting horizontal

Wc = wt. conc. = 8 × 8 × 5 × 150 = 48,000#

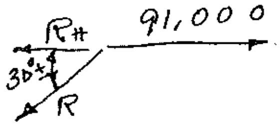
Overturning moment = 91,000 × 5 − $\overset{3}{4}$8,000 × 4 = 455,000 − 192,000
= 263,000'#

8 R = 263,000
R = 32,875#

V rock = $\frac{32,875}{2.5 \times \underset{8}{\cancel{17}} \times 144}$ = 11.4# allowable 150# O.K.

Use 7/8" φ @ 12" ea-way

Eyebar size



RH = 91,000

R = $\frac{91,000}{.866}$ = 104,000#

area = $\frac{104,000}{20,000}$ = 5.2

Use 5" × 2" eyebar

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Check sags with & without ice load.

Length of back tie cables = $\frac{190+115}{\cos 15^\circ} = \frac{305}{.966}$ = ~~316' total~~ 316' total

Length of suspension cable with 37' span.

Pg. 285 Union Wire Rope Handbook for formula.

$L = 2\sqrt{\frac{s^2}{4}+\frac{4d^2}{3}} = 2\sqrt{\frac{550^2}{4}+\frac{4\times37^2}{3}} = 2\sqrt{75{,}625+1825} = 2\sqrt{77{,}450}$

s = span
d = sag.

$= \overset{556.60}{\boxed{556.96}}$ ft. ~~Say~~ Say 557 ft.

Total length of cable = 557 + 316 = 873 ft.

From WRC calculations, ice load for 1" ice was calculated as 65#/ft. or 32.5#/ft. each cable.

Check stretch in cable using 30#/ft. additional load over normal load on bridge.

Tension in cable from Pg. 1278 of Merriman & Wiggin Am. Civil Eng. Handbook

$T = \frac{W}{8h}\sqrt{l^4+16h^2l^2} = \frac{30}{8\times37}\sqrt{550^4+(16+42^2+550^2)} = 31{,}800^{\#}$ additional due to ice.

From pg. 309 of Union Wire Rope Handbook, modulus of Elasticity of 7×19 ~~galv.~~ bridge cable is 17,000,000 -

Metalic area of 2" galvanized bridge cable is 1.97 sq. in.

Increase in unit stress due to ice load $= \frac{31{,}800}{1.97} = 16{,}150^{\#}$/sq. in.

Stretch in cable due to increase in load

$= \frac{873\times16{,}150}{17{,}000{,}000} = .83'$

Length of suspension cable after stretch due to ice load

$\overset{556.60}{\boxed{556.96}} + .83 = \boxed{557.79}$ 557.43

Clearing above formula for sag.

$\frac{4d^2}{3} = \frac{L^2}{4} - \frac{s^2}{4} = \frac{\overset{557.43}{\boxed{557.79}}^2 - 550^2}{4}$ ~~[illegible]~~ $= \frac{\overset{8228}{\cancel{8630}}}{4} = \overset{2057}{\cancel{2158}}$

$d^2 = \frac{3\times\overset{2057}{\cancel{2158}}}{4} = \overset{1543}{\cancel{1618}} \quad d = \overset{37.27}{\cancel{40.22}}'$

If pipe is level without ice load, sag in pipe with ice load will be approx. ~~3'-3"~~. 2'-3"

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Check shortening of pipe due to sag with ice load.
Clearing formula at top of pg. 1 for span length.

$$\frac{S^2}{4} = \frac{L^2}{4} - \frac{4d^2}{3} = \frac{550^2}{4} - \frac{4 \times 3.25^2}{3} = 75,625 - 14.08 = 75,610.92$$

~~$s = \sqrt{\frac{75,610.92}{4}} = \sqrt{18,902.73} =$~~

$$s = \sqrt{75,610.92 \times 4} = 549.95.$$

shortening due to sag = 105 ft.

Shortening of pipe with 100°F temperature drop
$= 550 \times 100 \times .0000065 = .36$ ft.

To be safe should allow for 4" longitudinal movement of pipe at each end of bridge.

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Gas main should be approx. level in cold weather and allowed to sag in hot weather. Will use a sag of 37' with pipe level at 0°F and calculate sag for temperatures above 0°F.

Length of cable at 0°F - 37' sag from pg. 1 = 873'

Length of suspension portion of cable at 37' sag = ~~556.96~~ 556.60'

Coefficient of expansion of cable = .0000066

at. 20°F.

Increase in length = 873 × .0000066 × 20 = .12'

Suspension length = ~~556.96~~ 556.60 + .12 = ~~557.08~~ 556.72

From formula at top of pg. 1. $d = \sqrt{\frac{3(L^2 - S^2)}{16}}$

where d = sag, L = suspension length, S = susp. span.

. S = 550 $S^2 = 302,500$

Sag = $\sqrt{\frac{3(556.72^2 - 302,500)}{16}} = 37.34'$

at. 40°F

Increase in length = 873 × .0000066 × 40 = .23'

Suspension length = 556.60 + .23 = 556.83'

Sag = $\sqrt{\frac{3(556.83^2 - 302,500)}{16}} = 37.66'$

at 60°F

Increase in length = 873 × .0000066 × 60 = .35'

Suspension length = 556.60 + .35 = 556.95'

Sag = $\sqrt{\frac{3(556.95^2 - 302,500)}{16}} = 37.98'$

at 80° F

Increase in length = 873 × .0000066 × 80 = .46'

Suspension length 556.60 + .46 = 557.06'

Sag = $\sqrt{\frac{3(557.06^2 - 302,500)}{16}} = 38.28'$

at 100° F

Increase in length = 873 × .0000066 × 100 = .58'

Suspension length = 556.60 + .58 = 557.18'

Sag = $\sqrt{\frac{3(557.18^2 - 302,500)}{16}} = 38.61$

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at 120° F

Increase in length = 873 x .0000066 x 120 = .69'

Suspension length = 556.60 + .69 = 557.29'

Sag = $\sqrt{\frac{3(557.29^2 - 302,500)}{16}}$ = 38.90

File 4.6.3.79

PAN AM WORLD SERVICES, INC.

ENGINEERING REPORT

INSPECTION, EXAMINATION AND EVALUATION
FOR
CONCRETE ANCHOR BLOCKS
OF THE
PUEBLO CANYON 12" PIPELINE SUSPENSION BRIDGE

OCTOBER 24, 1989

WORK ORDER # 4442-27
PROJECT NAME: ENG. SERVICES -
PUEBLO CANYON BRIDGE

PREPARED BY: Thomas L Brake PE

REVIEWED BY: Will [illegible] P.E.

Per Discussion with Lon, [illegible], Riess, and Jim G., Tom Brake
at 1315 hrs, 10/25:

① We will submit this plan for review by LANL ENG-DIV. Structural Engr before going to bid.

② Lon stated it is on our Pan Am property books and is therefore our maint. responsibility. Said to go ahead with work, and that he would authorize us and "take the hit" if any problems came up.

③ Lon instructed ENG-8 to prepare a list of where our maintenance responsibilities begin and end on [illegible] [illegible] etc.)

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PAN AM WORLD SERVICES, INC.

ENGINEERING REPORT

INSPECTION, EXAMINATION AND EVALUATION
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OF THE
PUEBLO CANYON 12" PIPELINE SUSPENSION BRIDGE

1.0 SUMMARY OF CONCLUSIONS AND RECOMMENDATIONS:

1.1 All six of the anchor blocks have deteriorated concrete and have reached severe to complete failure. Exposure to moisture and freeze-thaw temperatures greatly accelerated an alkali - silica reaction between aggregates and cement and have contributed to a minor sulfate attack. The reinforcement steel and cables [illegible] anchor bolts and sockets show no visual evidence [illegible] deterioration.

1.2 Rehabilitation is recommended as soon as possible. Based on the results of physical examinations, the method of rehabilitation recommended is to first [illegible] bond the cracks and fractured concrete using epoxy grout and seal the exposed surface of the blocks using a penetrating epoxy sealer (in accordance with Appendix G, Specification for Epoxy Rehabilitation) with the expectation of extending the life of the blocks for several years; then evaluate restored blocks and the soil surrounding them to determine the need for specific additional stabilizing devices. The bridge should be inspected regularly (at least annually) to enable appropriate preventive maintenance.

2.0 PROBLEM AND NEED

2.1 The concrete anchor blocks that support the suspension cables of the Pueblo Canyon 12" Pipeline Bridge visually appear to be deteriorating rapidly and are in need of immediate attention for repair or replacement. More complete descriptions are given in Appendix I., Pan Am Design Engineering memo (J. Garcia) (10-24-89).

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2.2 Discussions among LA County, DOE, LANL and Pan Am Utilities indicate possible changes in the function of the bridge in the foreseeable future including the remote possibility of abandoning this pipeline and bridge altogether.

2.3 However, in the absence of any clear plans for change, the current need is to restore the cable anchoring system to a safe condition as economically as possible.

2.4 The "fix" may be: 1) a quick one to carry the bridge through the next freeze-thaw season coupled with restoration work in 1990; 2) a relatively "long term" rehabilitation coupled with annual inspections and regular preventive maintenance activities; or 3) replacement of the existing anchor blocks with a "permanent" system coupled with preventive maintenance.

2.5 There is no firm required life expectancy for the bridge, nor is there a budget for repairing and maintaining the bridge. Acceptable repair costs must be determined as the investigation and evaluations of alternatives is developed.

3.0 SCOPE OF ENGINEERING SERVICES TO BE PERFORMED:

3.1 W.O. #4442-27 - The services requested in this work order are described as an "inspection and report on the structural integrity of the Pueblo Canyon Suspension Bridge". After an initial meeting and visit to the site, the scope was further defined and limited to include only the concrete blocks that anchor the suspension cables. Considerations of the integrity of other parts of the bridge is not included here. The following items describe the extent of work to be performed.

3.2 Recommend the method(s) of examination and analyses required to determine the existing integrity of the six concrete anchor blocks.

3.3 Proceed with the approved methods of measuring, testing, examining, inspection and evaluating.

3.4 Recommend the most appropriate method(s) of correcting the problems determined in the evaluations. This would include a discussion of alternatives and preparation of a technical specification for procurement of the required construction or rehabilitation services.

3.5 Analyze bids for construction or rehabilitation work.

3.6 Inspection of the work in support of the Contract Administrator, including final inspection.

3.7 Examine and evaluate the rehabilitated blocks. This is to be accomplished by pulsed velocity (sonic) analysis and interpretation of the results.

4.0 ALTERNATIVES DISCUSSED:

4.1 Complete replacement of all 6 anchor blocks with new blocks - same design but with low alkali concrete that would not deteriorate. Current requirements for use of low alkali cement generally eliminates the chemical reactivity, but the cost of replacement is estimated at greater than $100,000.

4.2 Complete removal of existing blocks and replacement with a drilled pier/auger cast pier of concrete and make use of concrete weight and skin friction resistance. Same conditions apply as in 4.1 above.

4.3 Remove the worst of the deteriorated concrete, leave the re-bar and cable anchor hardware in place, and replace voids with special concrete and bonds and cap the repair with a shell of reinforced concrete. This would be a labor-intensive repair, would require temporary restraints for the cables during repair, and has the risk that the "worst" of the concrete may finally be all of it as the removal progresses.

4.4 Drill and grout rock anchor bolts through the existing blocks into the rock under the blocks, utilizing large steel plates to partially bind the cracked concrete pieces together, along with some epoxy grouting of cracks and surface sealing. Some grout injection would be necessary to allow the anchor bolts to be effective, but this combination would be a temporary fix and would not stop further deterioration of concrete.

4.5 Inject an epoxy grout into the cracks and delaminations throughout the entire block and apply epoxy penetrant sealer to the exposed surface of the block with the intent to re-bond the concrete pieces and to greatly reduce, possibly eliminate, the alkali reaction by cutting off exposure to moisture and the resultant freeze-thaw effect. This would produce a solid bonded mass with characteristics similar to the original concrete block; the life expectancy is several years, perhaps permanent if the chemical reactions can be stopped; the work can be done quickly and the cost is less than other alternatives.

5.0 INVESTIGATION WORK SELECTED AND PERFORMED:

5.1 Pulsed velocity analysis of all blocks (a non-destructive examination) was chosen to obtain preliminary indications of the extent and size of internal cracks. It is not totally conclusive of concrete strength, but does broadly categorize the location and extent of cracking, thus permitting the best choice of core boring locations for obtaining physical evidence.

5.2 Based on the very low velocities obtained by sonic examination, a core sample was drilled and removed from the eastern main block and examined visually.

The poor condition of the concrete in that sample dictated cutting and removing a sample from the western main block to determine if all the blocks should be presumed to be badly cracked.

Results and interpretations of both cores are given in Appendix A (Western Technologies).

5.3 The severe cracks and fractures observed in the core samples indicate a significant mineral reactivity.

This prompted the next step in the investigation, a petrographic examination to obtain specific physical and mineralogical properties and to assess the degree of alkali-aggregate reactivity and degree of cement hydration.

Results and interpretations are given in: Appendix B (Micro-Chem Laboratories).

5.4 Water is required to promote mineral reactivity and to cause cracking due to freeze-thaw temperature changes. A section of core sample was examined for: 1) water content in a core sample and 2) ability of the concrete to absorb moisture. This is a factor in determining the method(s) of rehabilitation.

Results are given in Appendix E

5.5 Epoxy resins bond well to concrete and aggregate surfaces, providing a bond exceeding that of normal cement hydration and concrete strength development, and providing a water repellant product. On this premise, the following sources were reviewed and analyzed for development of specific criteria for rehabilitation of the blocks:

ASTM Standards
ACI Standards
New Mexico State Highway Department Bridge Engineers and Specifications
Manufacturer's Technical Data (3 sources)
Application Contractors (2 sources)

Results and interpretations are given in:
Appendix C (Vic Peery Construction)
Appendix G (Specification EMTD-EGROUT-100)

6.0 CONCLUSIONS:

6.1 All six of the anchor blocks are extensively cracked and delaminated throughout the mass and have reached severe to complete failure. The concrete is, for all practical purposes, a mass of tightly knit pieces of various sizes held together by the cage of #7 steel reinforcing bars. The ability of these blocks to withstand the type loads for which they were designed has been greatly diminished; however, the life of the blocks in their current condition is indeterminate, depending entirely on the risk of ultimate safety factor loading. Reactivity will continue at an accelerated rate as long as air and moisture have access to carry on the chemical reactions.

6.2 Steel re-bar and cable anchor bolts and sockets [redacted] sound condition, based on visual examinations.

6.3 Weight, shape and placement of blocks is adequate to support the bridge as designed.

6.4 Epoxy injection can, when properly applied, re[redacted] cracks and delaminations in the blocks stronger than new.) The re-bond will produce a restored, solid mass with integrity similar to normal concrete. The mass will also be water repellant. There is no way to know in advance how extensive the crack fill will be, but core sampling and sonic analysis of the work will provide reliable indicators of how well the epoxy penetrates the cracks and bonds the surfaces.

6.5 There are many suppliers of epoxy and contractors who profess to be able to properly apply the products. However, most suppliers either have no track record or their track record has been blemished by numerous law suits. And there are relatively few contractors who have a successful track record in this specialty of epoxy injection.

7.0 RECOMMENDATIONS:

7.1 Recommend rehabilitation through competitive bidding in accordance with the terms of Appendix G (Specification for Rehabilitation) and Appendix H (Field Requisition).

7.2 The work should be completed before the end of November unless allowance is made for extensive weather protection and heating throughout the application and curing period.

7.3 The blocks should be covered temporarily now and kept covered continuously until the injection contractor can provide adequate protection from rain, snow and low temperature. At the date of this report, Utilities is arranging for this work. 7

7.4 Pan Am should prepare the site by excavating the top soil around each block down to tuff, and with a trench wide enough to allow the epoxy applicator to operate drilling and injection equipment. This work has been scheduled by Roads and Grounds.

7.5 After injection is complete, the blocks should be examined by selected coring and analysis to evaluate for % of crack filling and for bonding strength through the use of compression testing.

7.6 The blocks should also be examined nondestructively by pulsed velocity (sonic) analysis after core holes have been filled and after any supplementary injections resulting from core sample analysis.

7.7 After the epoxy injection process is complete, the blocks and surrounding soils should be given a final evaluation to determine the need for any other strengthening or safety modifications such as drilling and grouting rock anchor bolts through the blocks and into the rock below the blocks. For information only, Appendix D (Calculations of Cable and Block Loads) is attached as a preliminary step toward this final

PAN AM WORLD SERVICES, INC.

MEMORANDUM

TO: Jerome Gonzales, USTP

THRU: Manager, Materials Engineer, EMTD

FROM: Sr. Materials/Geotechnical Engineer, EMTD

EMTD90.130

DATE: March 23, 1990

SUBJECT: W.O. #4442-27, ENGINEERING SERVICES, PEGGY SUE BRIDGE FOUNDATION ANALYSIS OF CONCRETE ANCHOR BLOCKS

As requested, we have completed an investigation of the foundation supporting the two principal anchor blocks and an analysis of the potential for the installation of anchor bolts for additional support.

The attached report of this work describes the recommended type and configuration of anchors required to provide adequate support.

A contract for this installation should be about $5,000, plus detailed design, specification preparation, administrative and inspection costs.

A copy of this report is being given to ENG-3 to give them an opportunity to review and provide any additional recommendations.

This report completes our scheduled work on this phase of the project.

Thomas L. Brake

Thomas L. Brake

TLB:mjh

XC: C.E. Baxter, EMDO
Doug Volkman, ENG-3, M984
Lamar Nowland, EDED

ATTACHMENT: Anchor Bolt Investigation Report

WORLD SERVICES, INC.

ANCHOR BOLT INVESTIGATION REPORT

March 23, 1990

W.O. #4442-27
ENGINEERING SERVICES - PEGGY SUE BRIDGE
FOUNDATION ANALYSIS OF CONCRETE ANCHOR BLOCKS

PURPOSE: The objective of this endeavor has been the geotechnical investigation of the substratum to determine its capacity to support pre-tensioned anchors and then to devise an anchor configuration that would satisfy the mechanics pertaining to the blocks.

SUMMARY OF CONCLUSIONS: The subgrade materials are composed of incompetent tuff. Its consistency is that of a dense sand and this analogy was employed in the suggested design included. The type of anchor assumed herein for the blocks is a hollow-core prestressable anchor, adapted with a tubular hollow at its center for injecting grout downward through the bar and upward through the annulus in the hole.

OBSERVATIONS AND CONCLUSIONS: The tests, measurements and design example are summarized as below. Supporting details and computations are provided within the text.

1. <u>Borings</u>: Two borings were drilled, one adjoining each of the two major concrete blocks (see Boring Locations on page 1). Boring logs follow on the next two pages. A hollow-stem auger was used to drill the borings, with sampling taking place through the aperture. Continuous sampling procedures yielded a meager recovery, indicative of the soil's lack of coherence.

2. <u>Profile</u>: The Boring Profiles are provided on pages 2 and 3. These disclose that the consistency of the substrata on opposite sides of the canyon are very similar. The boring on the east side of the canyon was drilled to a depth of 25 feet; that on the west-side to a depth of 23.5 feet.

3. <u>Soils</u>: The material encountered in both borings was incompetent tuff. A design procedure should treat this material as a dense sand. The average dry unit weight is 95 pcf having an average moisture content of 0.6%.

4. <u>Soil Testing</u>: The predominant test procedure was that of the direct shear test and a total of 10 of these tests were completed (pp. 4 through 6). The selected angle of internal friction for design purpose is $\phi=31^\circ$ and a contracted cohesion

of C=250 psf was also decided upon. The coefficient of passive lateral pressure, K_p, is 3.12; the coefficient of at-rest lateral pressure, K_o, is 0.485. A Newmark method of stress analysis was computed (page 10) to determine the block's contribution to vertical stresses at various depths.

5. Anchor Design: The method used for pull-out resistance of the anchors was devised by Su and Fragaszy (3) and is outlined on pages 8 and 9. Only the resistance to pull-out of the grout shaft was considered. The top resistance would apply only in the case of a buried anchor. A displacement-resistance curve was then drawn to illustrate the movement of the grout-shaft necessary to attain maximum resistance (page 12). After a number of trial calculations (Appendix B), the final and workable configuration was arrived at (page 13) being derived from the data outlined on page 14.

The suggested design requires 3-25 foot anchors having grout-shafts 18 feet long. The factor of safety against overturning about the toe of the block is 1.06.

The shear resistance to sliding provided by the material of the anchors and grout is 240 kips (page 15), coupled with frictional resistance of the block amounting to 112.5 kips plus passive pressure resistance of 140.7 kips (page 14) totals 493.2 kips. The factor of safety is 2.5.

6. Anchor: The 1.375 inch hollow-core prestressable anchor is grouted up to the base of the block. Within the remaining 7 feet of borehole through the block, the anchor bar is sheathed and the annulus filled with a passive material to arrest corrosion. With a prestress force of 5 tons on each anchor, the uninhibited 7 feet of bar should displace about 0.019 inches. This applied force enhances sliding friction resistance. The manufacturer for this type of anchor is the Williams Form Engineering Corp (4).

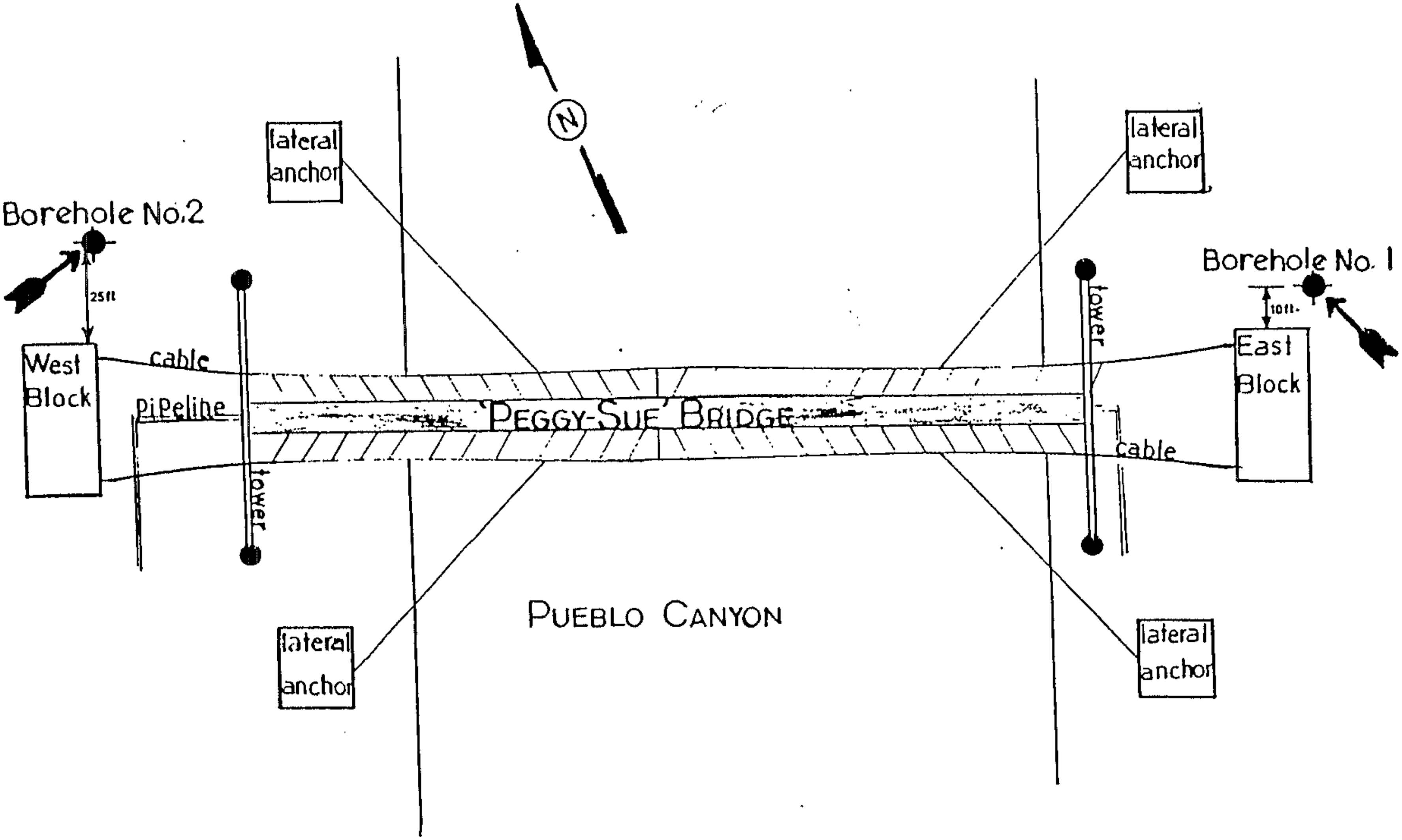
7. Earthquake Data: Los Alamos lies in seismic risk zone 2. This status implies a possibility of an earthquake having an intensity of VII on the Modified Mercalli Scale. This intensity can produce a ground acceleration of from 0.07 g to 0.15g and a velocity of from 7cm/sec to 20cm/sec. The anchor bolt design should address this condition.

Respectfully submitted,

Thomas L. Brake

Thomas L. Brake, P.E.

CONTENTS



BORING LOCATION MAP

Pan Am World Services, Inc.

TEST BORING LOG

PROJECT: PUEBLO CANYON BRIDGE W.O.#: 4442-27

BORING NO.: 1 BY: WTI DRILLING RIG: CME-75 AUGER: 6 5\8"

ELEVATION: N\A DATUM: N\A BY: N\A

ASSUMED ELEVATION: N\A WATER: NONE DEPTH: N\A TIME: N\A

DATE DRILLED: 11-09-89 DRILLED BY: WTI

LOG	ELEV.	CLASSIFICATION	WT. ON BIT	DRILL SPEED	% M	WT.DRY #\C.F.	SAMPLE
▨		Rhyolite Porphyry:tan, subrounded, subangular, moderately sorted, poor to moderately indurated,	1500	32 MIN\ FOOT			NO RECOVERY GRAB SAMPLE GRAB SAMPLE
▨	5	friable, aphanitic texture, 30% quartz, 10% blue sanidine inclusions, 60% welded tuffaceous matrix.		1 MIN\ FOOT			GRAB SAMPLE
▨	10	light purple		2 MIN\ FOOT	1.1 1.0	94 93	FULL RECOVERY BUT MOSTLY INCOMPETENT
▨	15			2 MIN\ FOOT	0.5 0.3 0.2	93 102 93	
▨	20			2 MIN\ FOOT			
	25	AUGER REFUSED @ 25' STOPPED SAMPLING @ 25'					
	30						
	35						
	40						

Pan Am World Services, Inc.

TEST BORING LOG

PROJECT: PUEBLO CANYON BRIDGE W.O.#: 4442-27

BORING NO.: 2 BY: WTI DRILLING RIG: CME-75 AUGER: 6 5\8"

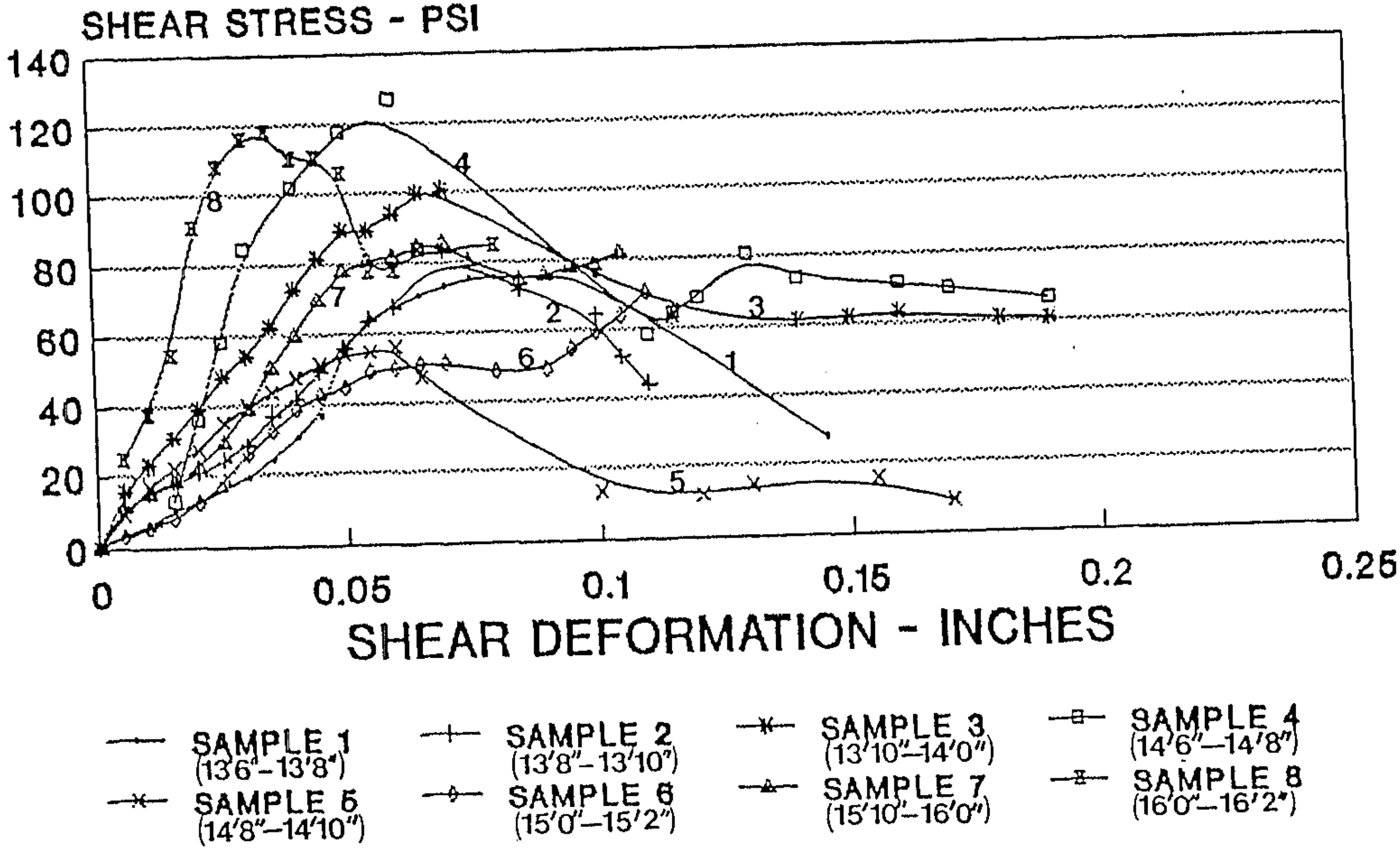
ELEVATION: N\A DATUM: N\A BY: N\A

ASSUMED ELEVATION: N\A WATER: NONE DEPTH: N\A TIME: N\A

DATE DRILLED: 11-09-89 DRILLED BY: WTI

LOG	ELEV.	CLASSIFICATION	WT. ON BIT	DRILL SPEED	% M	WT. #\C.F.	SAMPLE
▨		Rhyolite Porphry:tan, subrounded, subangular, moderately sorted, poor to moderately indurated,	1500	6 MIN\ FOOT			POOR RECOVERY
▨	5	friable, aphanitic texture, 30% quartz, 10% blue sanidine inclusions, 60% welded tuffaceous matrix.		20 SEC\ FOOT			
▨	10	light purple to tan in color		2 MIN\ FOOT			
▨	15						GRAB SAMPLE
▨	20			32 MIN\ FOOT			
▨				4 MIN\ FOOT			
	25	STOPPED SAMPLING @ 23.5'					
	30						
	35						
	40						

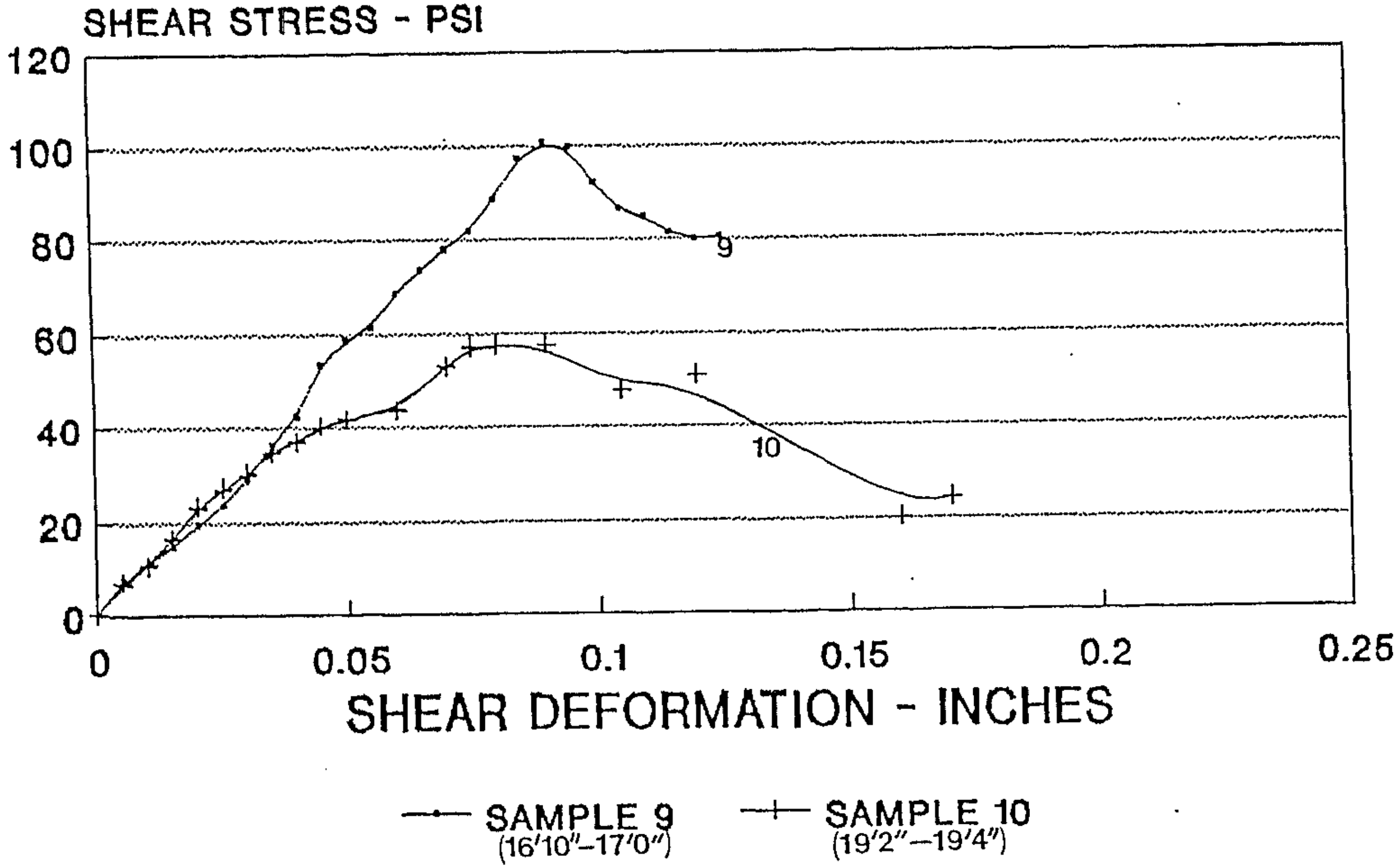
SHEAR STRENGTH GRAPHS
PEGGY SUE BRIDGE



Boring No. 1

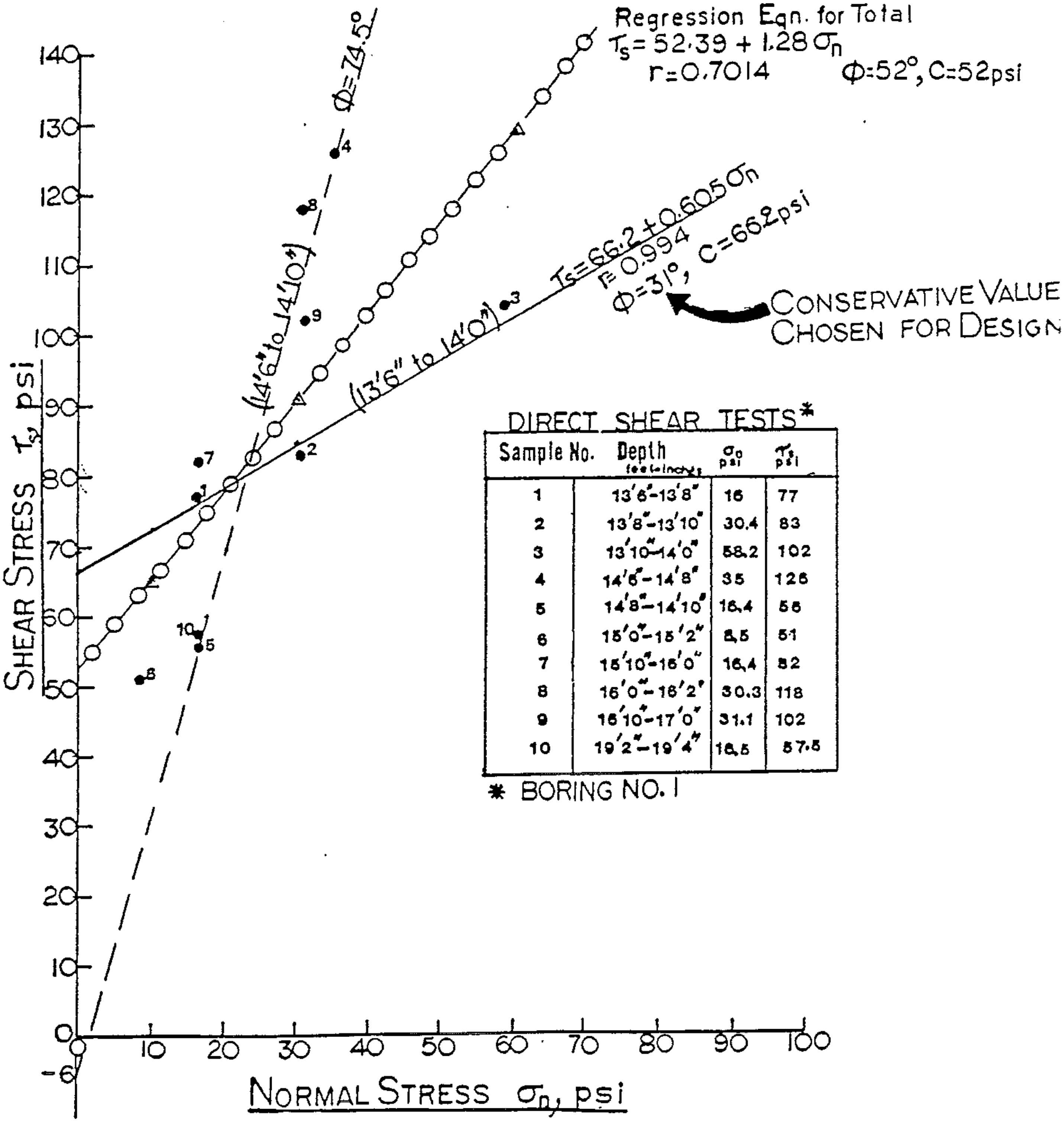
SHEAR STRENGTH GRAPHS

PEGGY SUE BRIDGE



(CONTINUED)

DIRECT SHEAR TESTS
PEGGY SUE BRIDGE



DIRECT SHEAR TESTS*

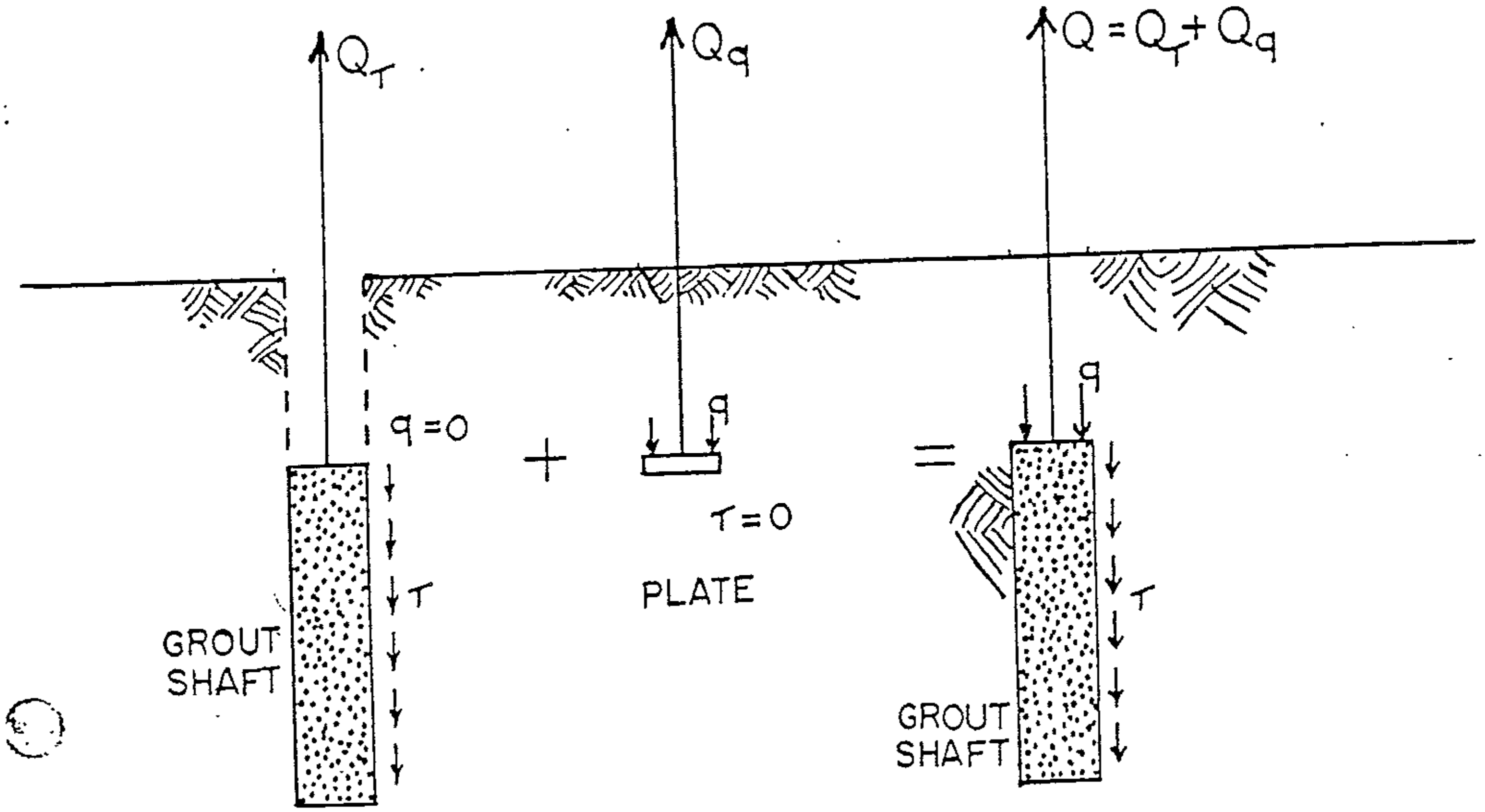
Sample No.	Depth (feet-inches)	σ_n psi	τ_s psi
1	13'6"-13'8"	16	77
2	13'8"-13'10"	30.4	83
3	13'10"-14'0"	58.2	102
4	14'6"-14'8"	35	126
5	14'8"-14'10"	16.4	56
6	15'0"-15'2"	8.5	51
7	15'10"-16'0"	16.4	82
8	16'0"-16'2"	30.3	118
9	16'10"-17'0"	31.1	102
10	19'2"-19'4"	16.5	57.5

* BORING NO. 1

GENERAL DESIGN PARAMETERS USED IN THE PRESENT DESIGN ON NEXT 3 PAGES

SHAFT RESISTANCE OF THE GROUT COLUMN IS ONLY RESISTANCE TO PULL-OUT CONSIDERED HEREIN.

GROUND ANCHORS



The load applied to a single gound anchor is carrfied jointly by the soil on the top of the grout shaft and by the soil around the shaft.

The common expression for the bearing capacity of a gound anchor is

$$Q = Q_\tau + Q_q$$

where Q_τ = the shaft resistance; and Q_q = the top resistance.

Q_τ increases rapidly as loading begins and reaches a peak value after a small displacement.

Q_q increases more slowly and reaches its peak value at a relatively large displacement. The soil is usually in a plastic state when Q_q reaches peak value due to shear failure.

ANCHOR RESISTANCE FORMULAE (3)

For a homogeneous soil and a linear overburden stress increase, the shear stress around the cylindrical grout shaft is assumed to increase linearly. The skin resistance of a dense sand is

$$Q_\tau(s) = (\pi D L)\frac{s}{L s_f}\exp\left(1 - \frac{s}{L s_f}\right)(c + K\gamma Z_m \tan\phi)$$

where Z_m = the distance between the ground surface and the midpoint of the grout shaft;

K = the lateral stress coefficient, which depends on the density of the soil and the injection pressure.(1) K_0 for loose sand and K_p for dense sand.

s = displacement

s_f = displacement at failure in direct shear test

L = length of the grout shaft

D = diameter of grout shaft

γ = unit weight of the soil

ϕ = angle of internal friction of the soil

c = cohesion

The maximum value of the top resistance, s_t, occurs at s = 2 to 3D; therefore it is suggested that s_t = 2.5D in calculation. The total top resistance is given by

$$Q_q(s) = \frac{\pi}{4}(D^2 - d^2)q_{ult}\frac{s}{2.5D}\exp\left(1 - \frac{s}{2.5D}\right)$$

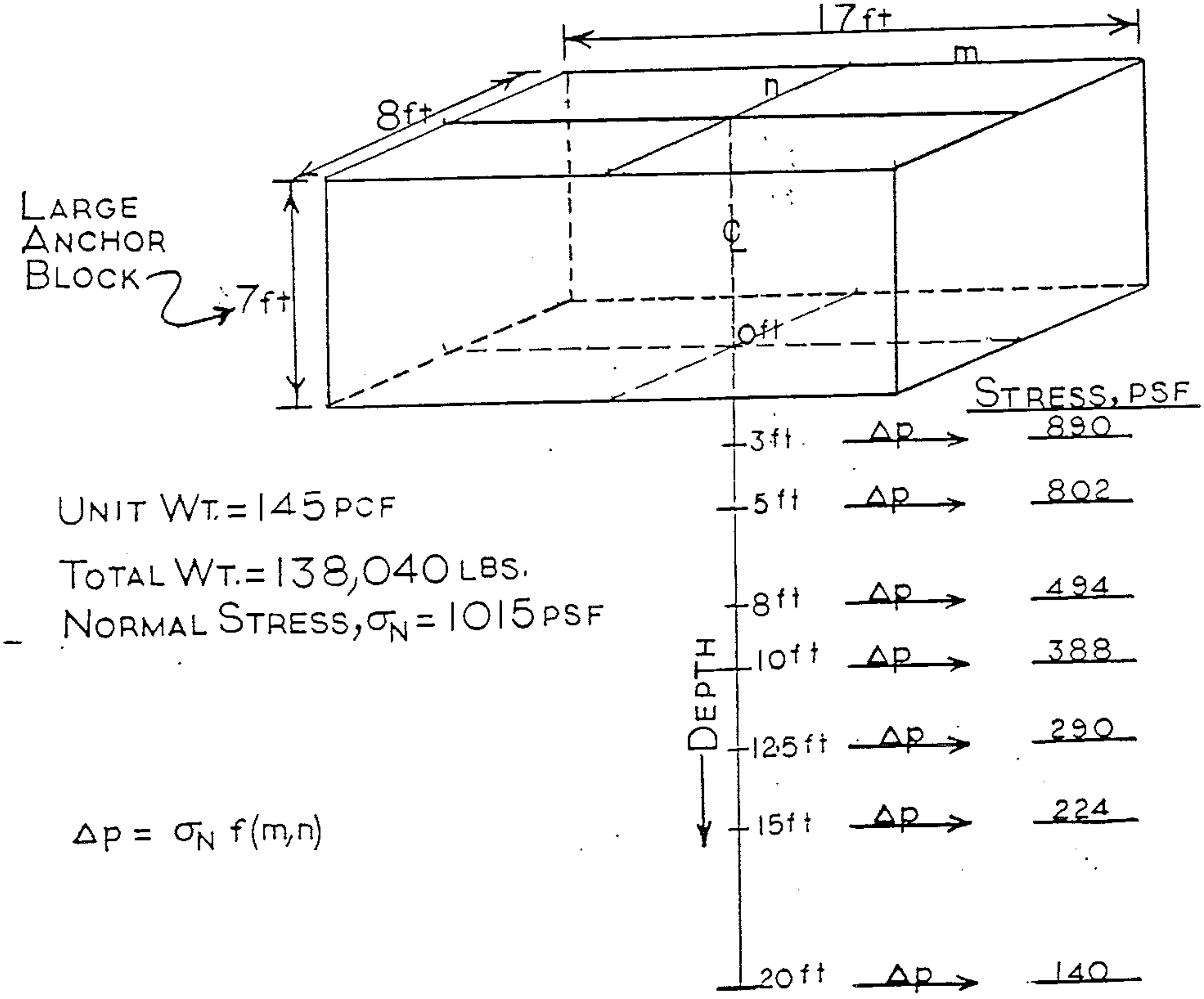
where d = diameter of tendon

q_{ult} = ultimate bearing capacity

The total capacity of the anchor is $Q(s) = Q_\tau(s) + Q_q(s)$

GROUND ANCHOR DESIGN

NEWMARK STRESS ANALYSIS



UNIT WT. = 145 PCF

TOTAL WT. = 138,040 LBS.

NORMAL STRESS, σ_N = 1015 PSF

$\Delta p = \sigma_N \, f(m,n)$

RECOMMENDED ANCHOR DESIGN

THE FOLLOWING FIVE PAGES COMPRISE THE FINAL RECOMMENDED DESIGN FOR ANCHORING THE CONCRETE BLOCKS.

THE DESIGN ASSUMES THE FOLLOWING:

1. THREE 1 3/8-INCH DIAMETER HOLLOW-CORE RE-BAR ANCHORS PER BLOCK. (4)

2. THE ANCHORS ARE 25 FEET IN LENGTH.

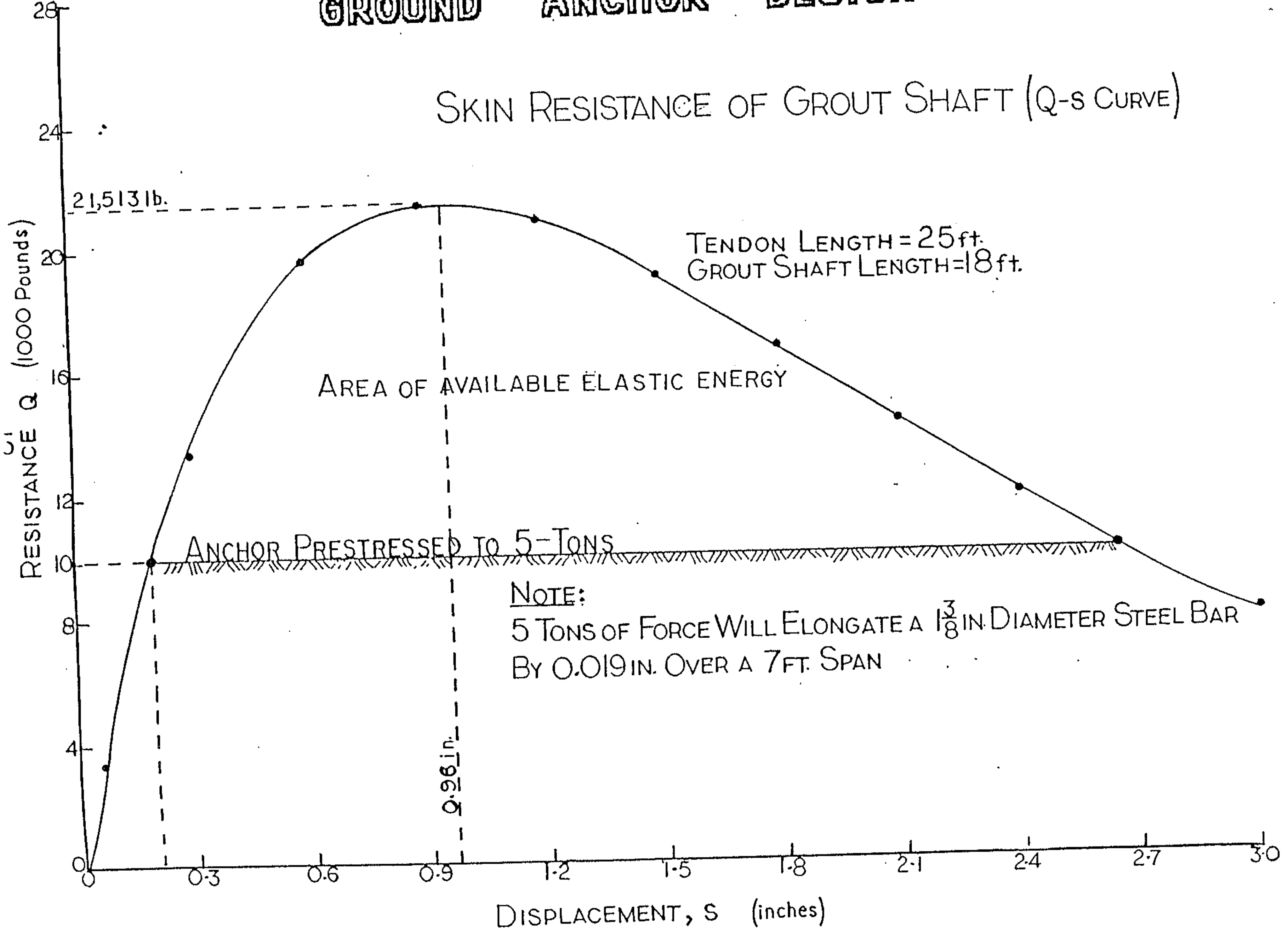
3. THE GROUT-SHAFT LENGTH IS 18 FEET.

4. BOREHOLE DIAMETER OF 2 1/2 IN.

5. EPOXY-RESIN GROUT.

6. ANCHOR CONFIGURATION 'E'

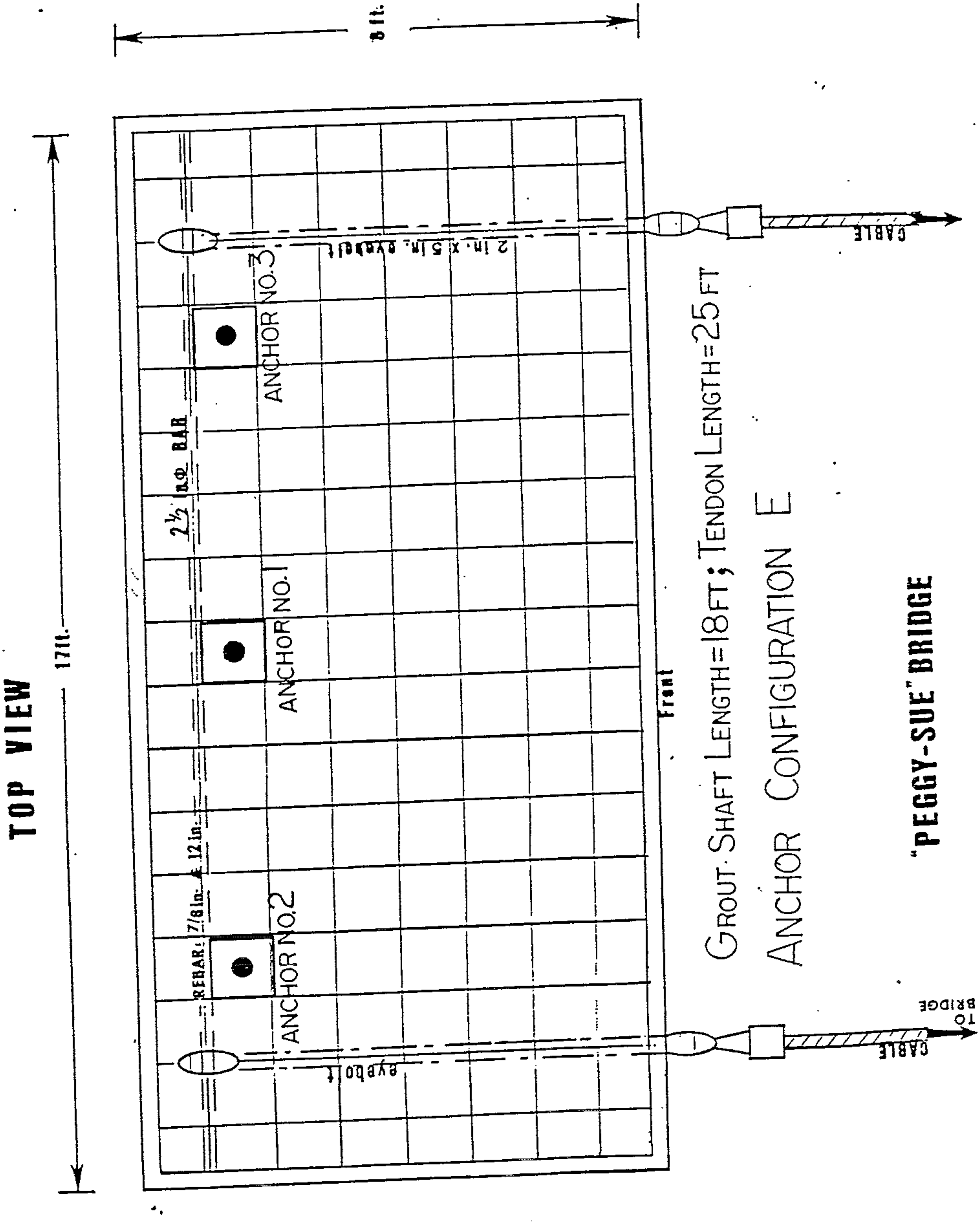
GROUND ANCHOR DESIGN



SKIN RESISTANCE OF GROUT SHAFT (Q-S CURVE)

TENDON LENGTH = 25 ft.
GROUT SHAFT LENGTH = 18 ft.

NOTE:
5 TONS OF FORCE WILL ELONGATE A $1\frac{3}{8}$ IN. DIAMETER STEEL BAR
BY 0.019 IN. OVER A 7 FT. SPAN

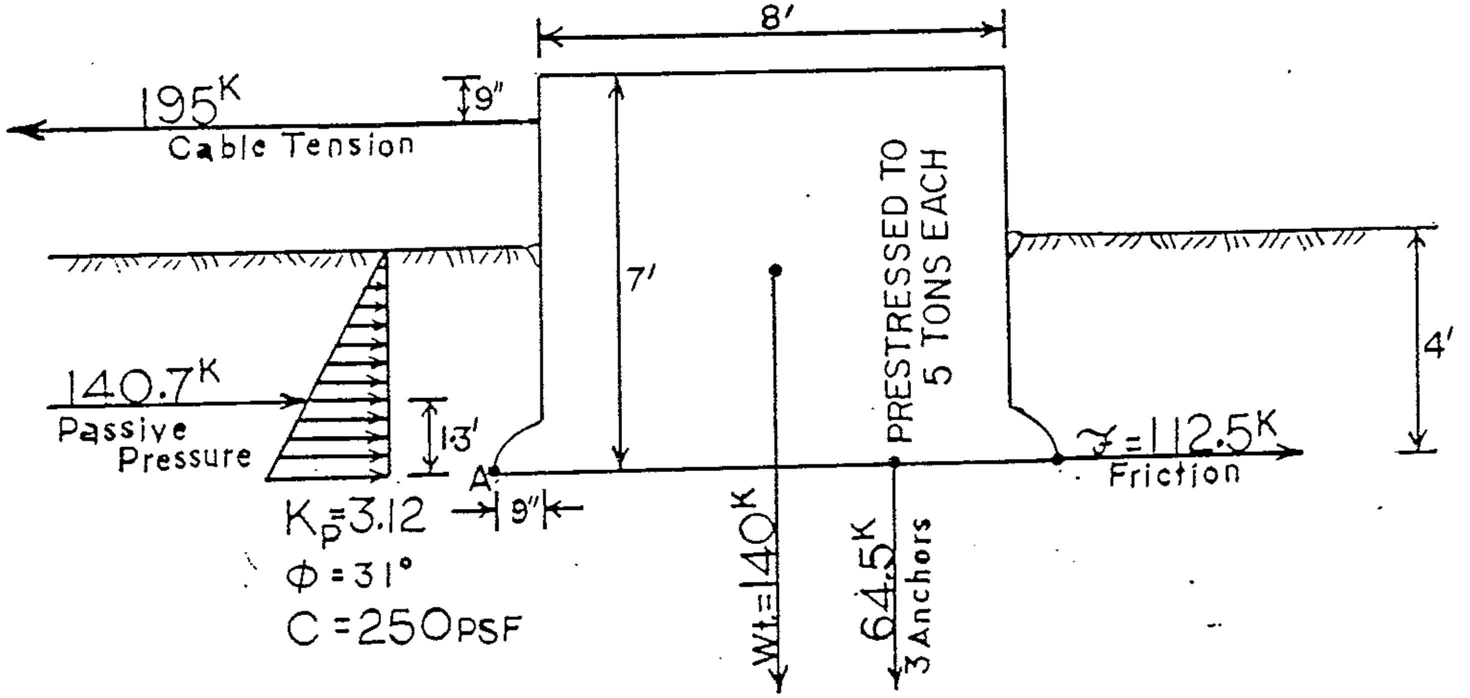


Grout Shaft Length=18FT; Tendon Length=25FT

Anchor Configuration E

"PEGGY-SUE" BRIDGE

RECOMMENDED DESIGN
ANCHOR CONFIGURATION 'E'



$$\Sigma M_A = 1218.75'^{k} - 1288'^{k} = -69.25'^{k} \text{ CW} \rightarrow \text{STABLE}$$

$$\Sigma F_H = 195^{K} - 140.7^{K} - 112.5^{K} = -58.2^{K} \text{ (TO RIGHT)}$$

SHEAR RESISTANCE PROVIDED BY ANCHORS AGAINST SLIDING OF CONCRETE BLOCK

ASSUMPTIONS:

(1.) HOLLOW CORE GROUTABLE RE-BAR ROCK BOLT, $1\frac{3}{8}$ in. DIAMETER
  (1.1) SHEAR MODULUS, $E_S = 11 \times 10^3$ KSI
  (1.2) CROSS-SECTIONAL AREA = $1.48\ \text{IN.}^2$
  (1.3) ELASTIC LIMIT = 40 KSI = τ
  (1.4) SHEAR STRAIN, $\phi = \tau/E_s$
(2.) EPOXY RESIN GROUT
  (2.1) DIAMETER = $2\frac{1}{2}$ in.
  (2.2) SHEAR STRENGTH $\cong$ 6 KSI

STEEL:

$$\phi = \tau/E_s = \frac{40\text{ksi}}{11\times10^3\text{ksi}} = 3.64\times10^3\text{rad.}$$

$$\tau = F/A = E_s\phi = 11\times10^3\text{ksi} \times 3.64\times10^{-3}\text{rad} = 40.04\text{ksi}$$

$$F = 1.48\text{in}^2 \times 40.04\text{ksi} = \underline{59.5\text{KIPS}}$$

EPOXY:

SECTION AREA = $4.91\text{in}^2 - 1.48\text{in}^2 = 3.43\text{in}^2$

SHEAR FORCE $\doteq$ $6\text{ksi} \times 3.43\text{in}^2 = \underline{20.6\text{kips}}$

TOTAL SHEAR FORCE RESISTANCE PER ANCHOR
= 59.5 KIPS + 20.6 KIPS = $\underline{80\text{ KIPS}}$

3 ANCHORS = $\underline{240\text{ kips}}$

APPENDIX A

REFERENCES

1. BOWLES, J. E. (1982), "Foundation Analysis and Design," 3rd ed., McGraw-Hill Book Company, New York, 816 pp.

2. DAS, B. M. (1983), "Advanced Soil Mechanics," McGraw-Hill Book Company, New York, 511 pp.

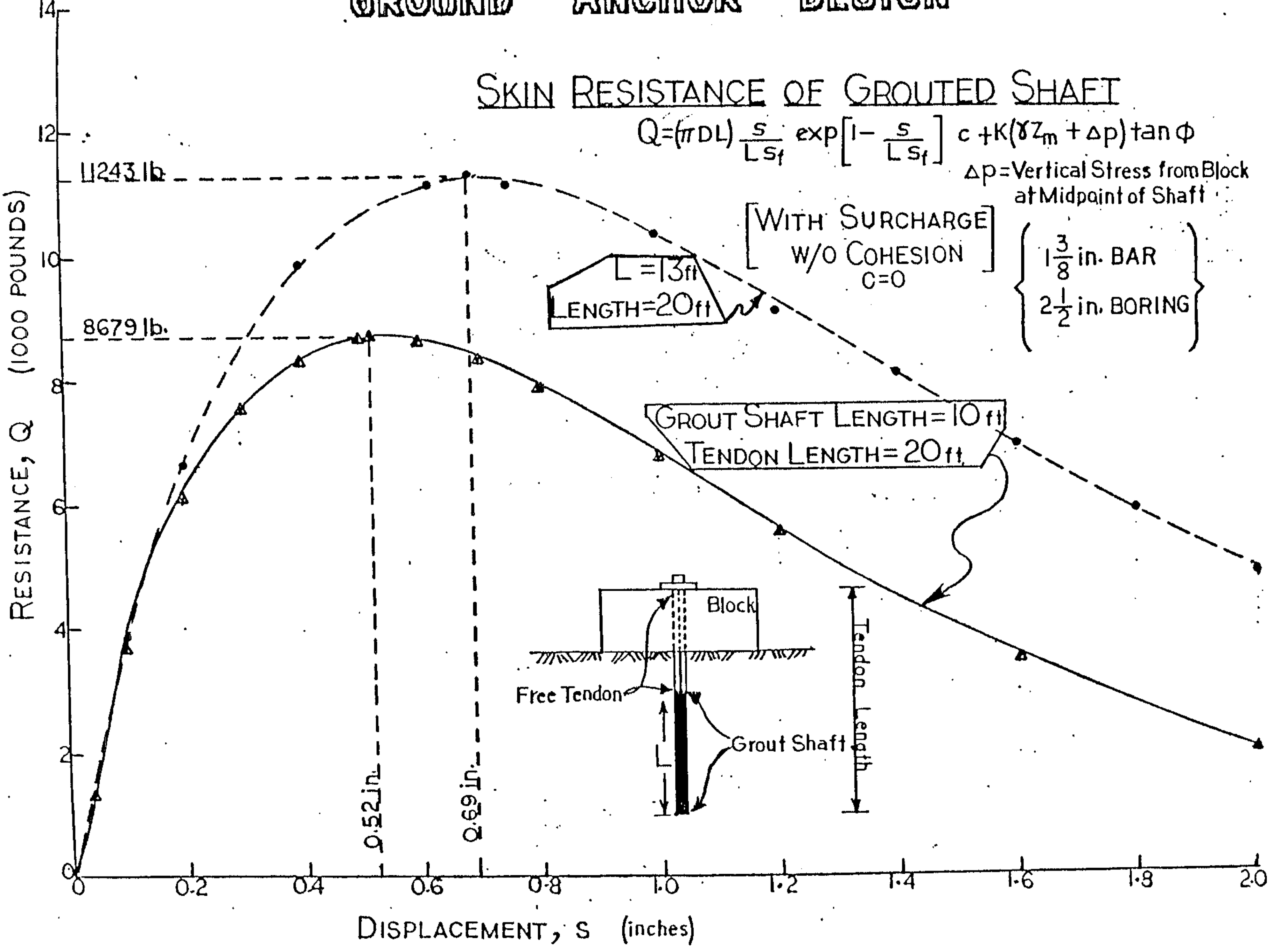
3. SU, W. and FRAGASZY, R. J. (1988), Uplift Testing of Model Anchors, Journal of Geotechnical Engineering, ASCE, Vol. 114, No. 9, September, pp. 961-983

4. WILLIAMS FORM ENGINEERING CORP., Anchor Bolting Systems, Pub. No. 198, 1988

APPENDIX B

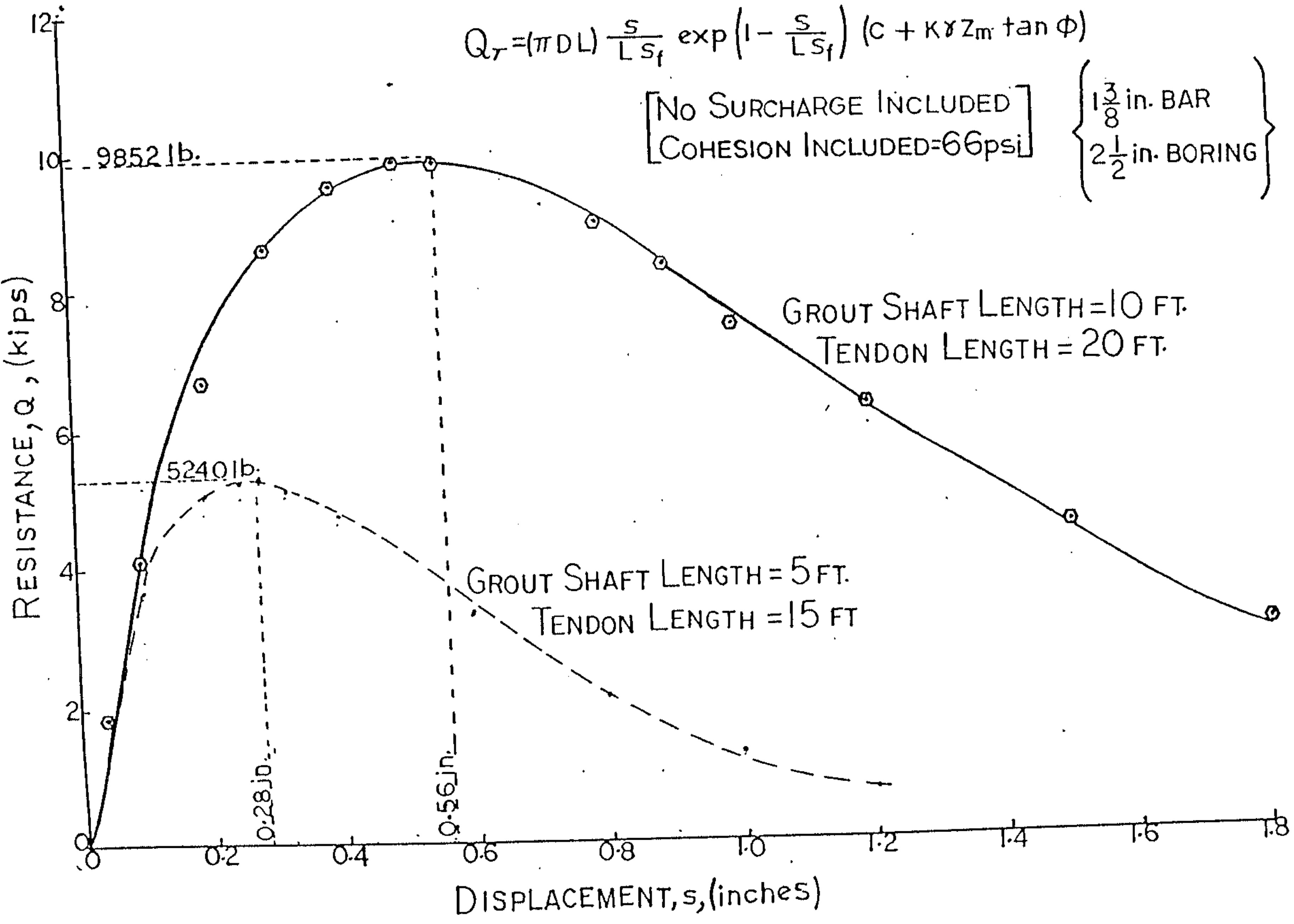
PRELIMINARY ANCHOR DESIGNS THAT WERE FOUND TO PROVIDE INSUFFICIENT RESISTING MOMENT TO PREVENT OVERTURNING OF THE CONCRETE BLOCK.

GROUND ANCHOR DESIGN



GROUND ANCHOR DESIGN

SKIN RESISTANCE OF GROUT SHAFT [Q-s CURVE]

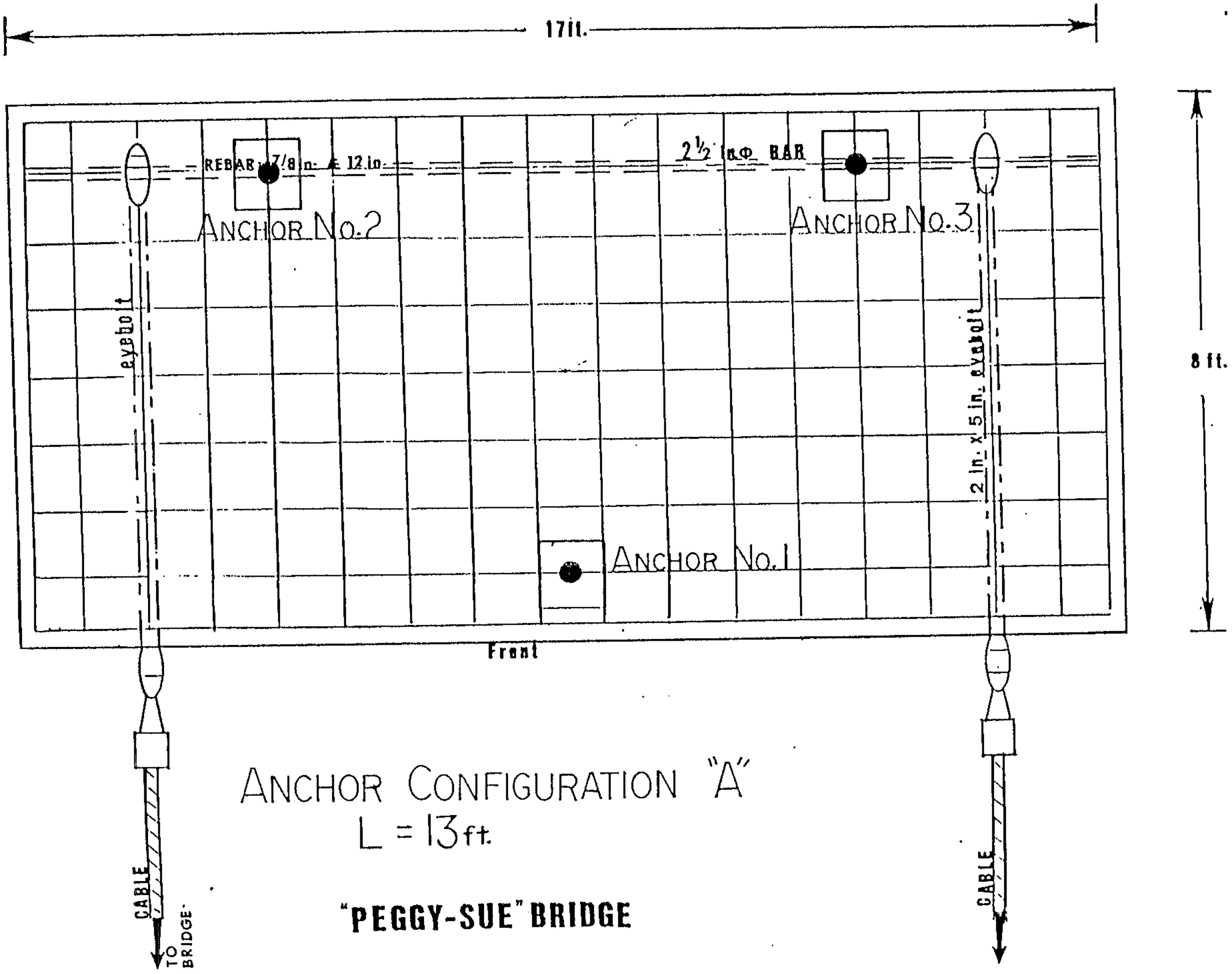


$$Q_r = (\pi D L) \frac{s}{L s_f} \exp\left(1 - \frac{s}{L s_f}\right) (c + K \gamma Z_m \tan \phi)$$

[No Surcharge Included
Cohesion Included=66psi]

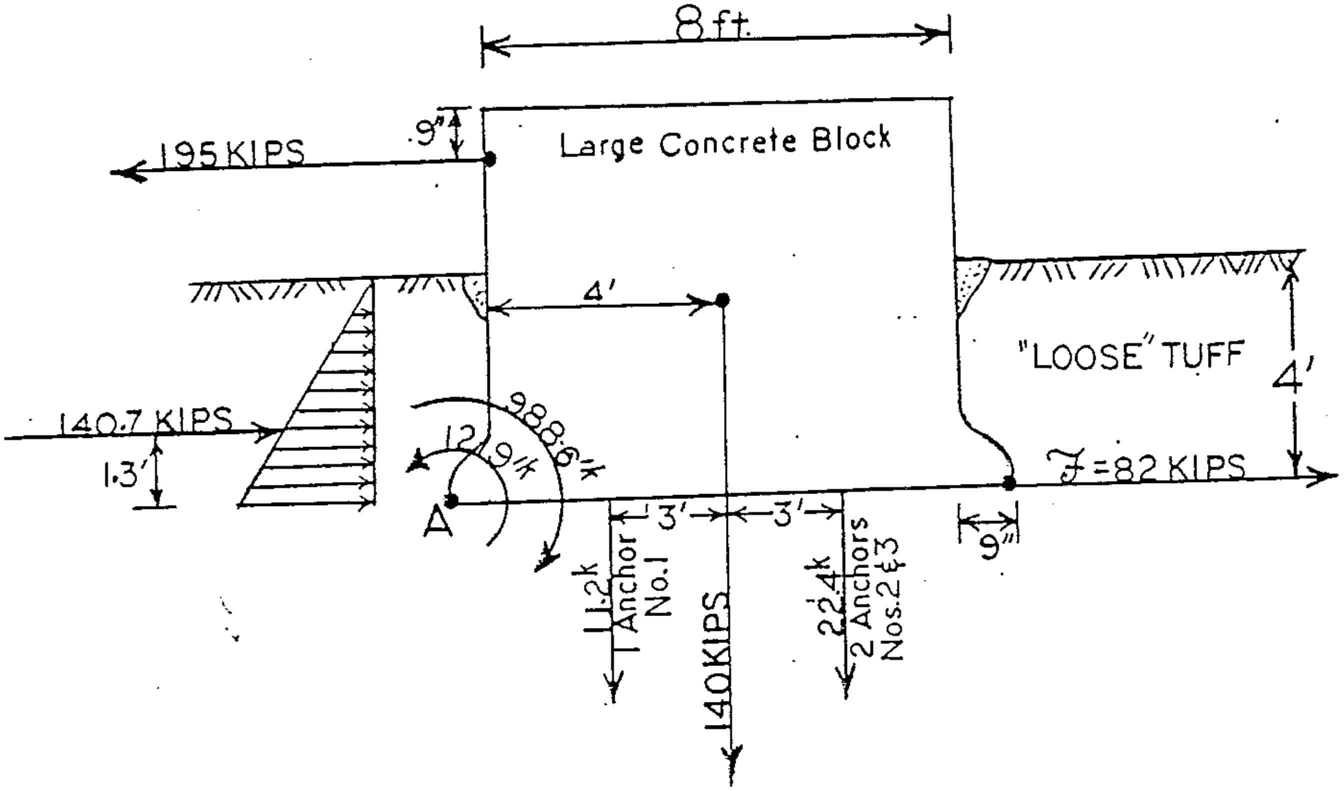
{$1\frac{3}{8}$ in. BAR
$2\frac{1}{2}$ in. BORING}

TOP VIEW



ANCHOR CONFIGURATION "A"
L = 13ft.

"PEGGY-SUE" BRIDGE

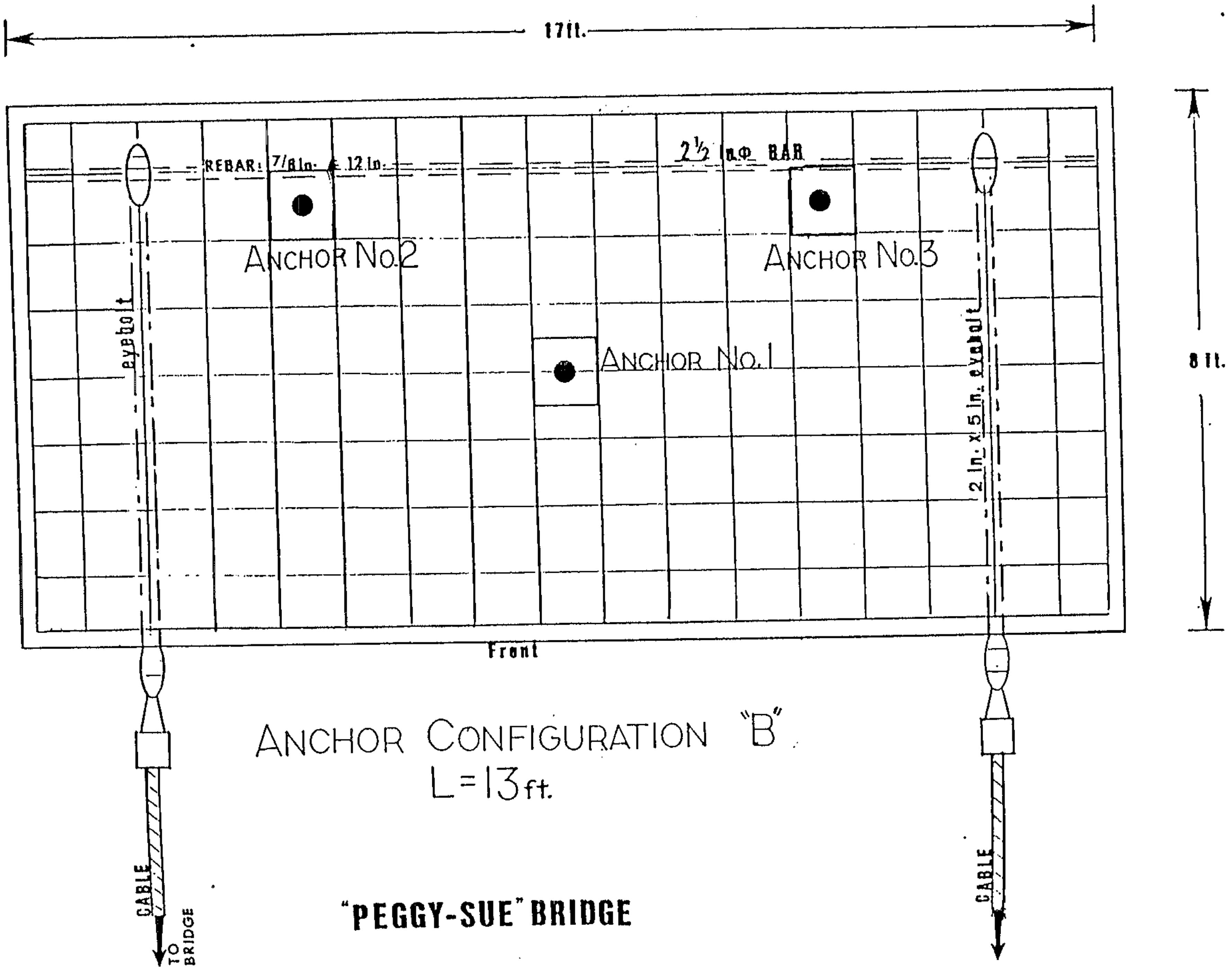


$$\Sigma F_H = 195^K - 140.7^K - 82^K = -27.7^K \qquad \text{OK}$$

$$\Sigma M_A = 1219'^k \circlearrowleft - 1041.1'^k \circlearrowright = \underline{177.9'^k} \circlearrowleft$$

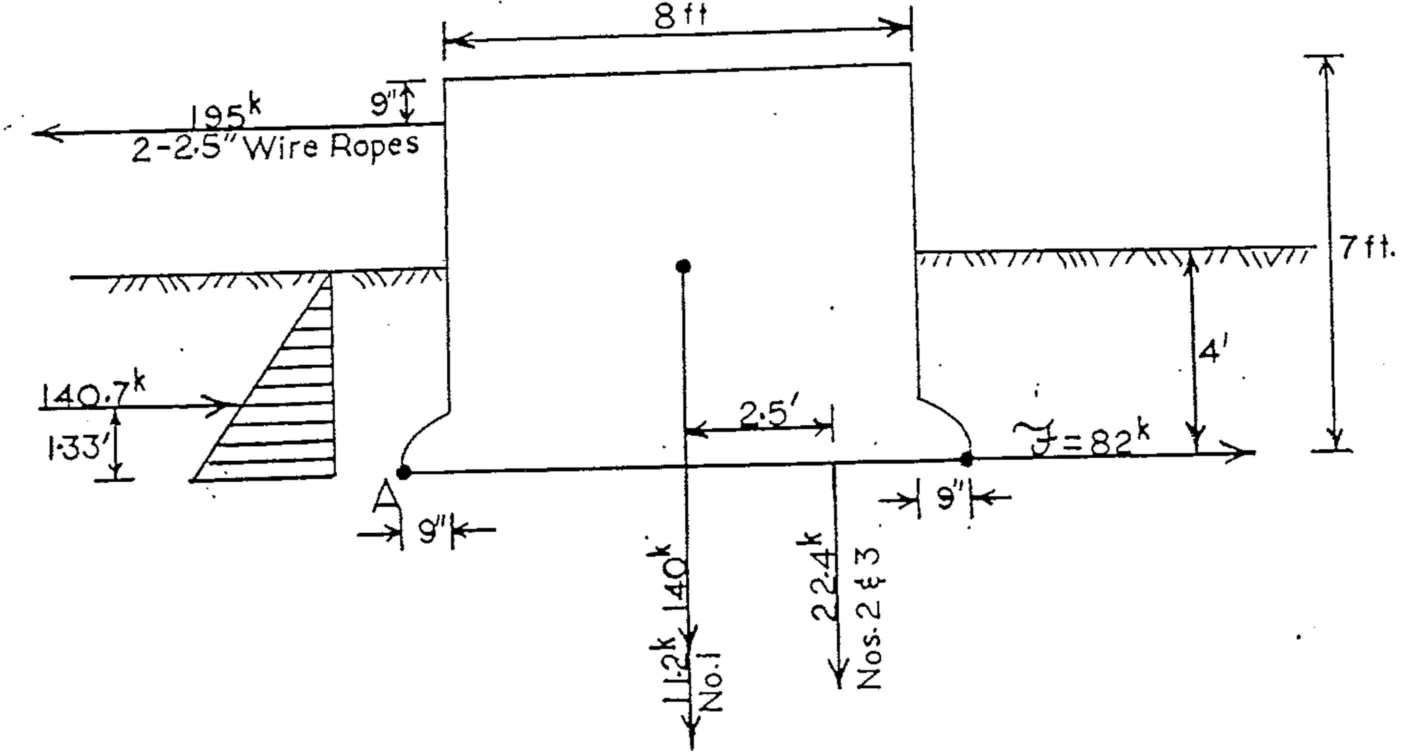
[UNBALANCED]

TOP VIEW



ANCHOR CONFIGURATION "B"
L=13ft.

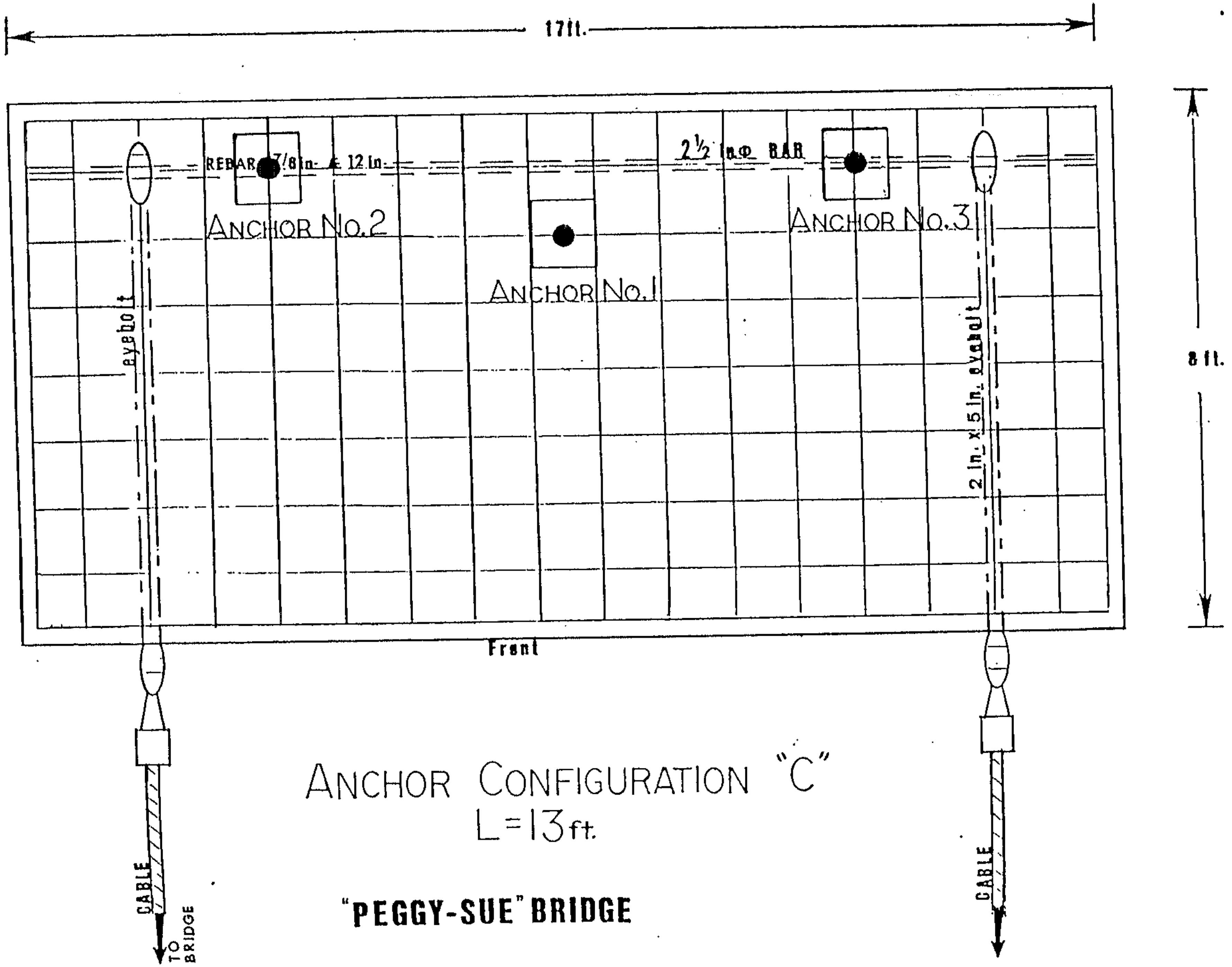
"PEGGY-SUE" BRIDGE



$$\Sigma F_H = 195^k - 140.7^k - 82^k = -27.7^k \quad \text{OK}$$

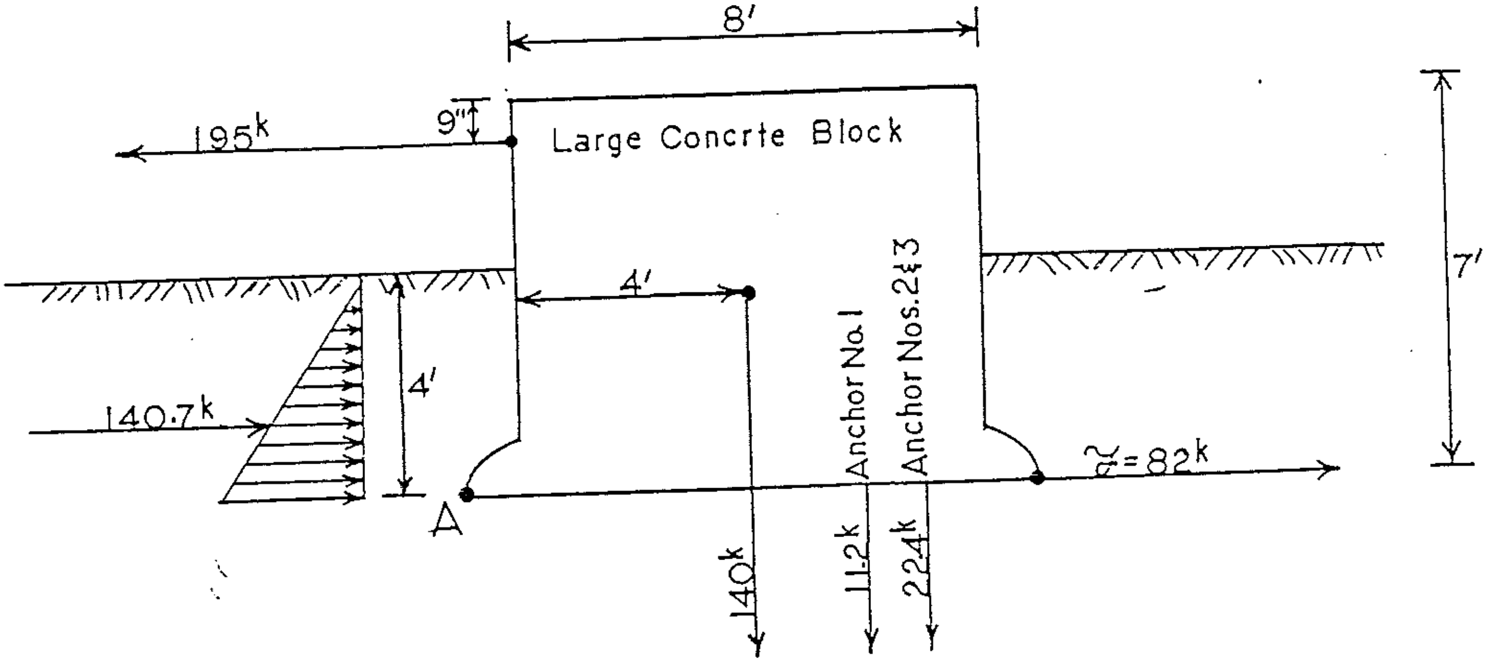
$$\Sigma M_A = 195^k(6.25') - 140.7^k(1.33') - 151.2^k(4.75') - 22.4^k(7.25')$$
$$= 1218.75'^k - 1067.7'^k = 151.05'^k \text{ ccw} \quad [\text{UNBALANCED}]$$

TOP VIEW



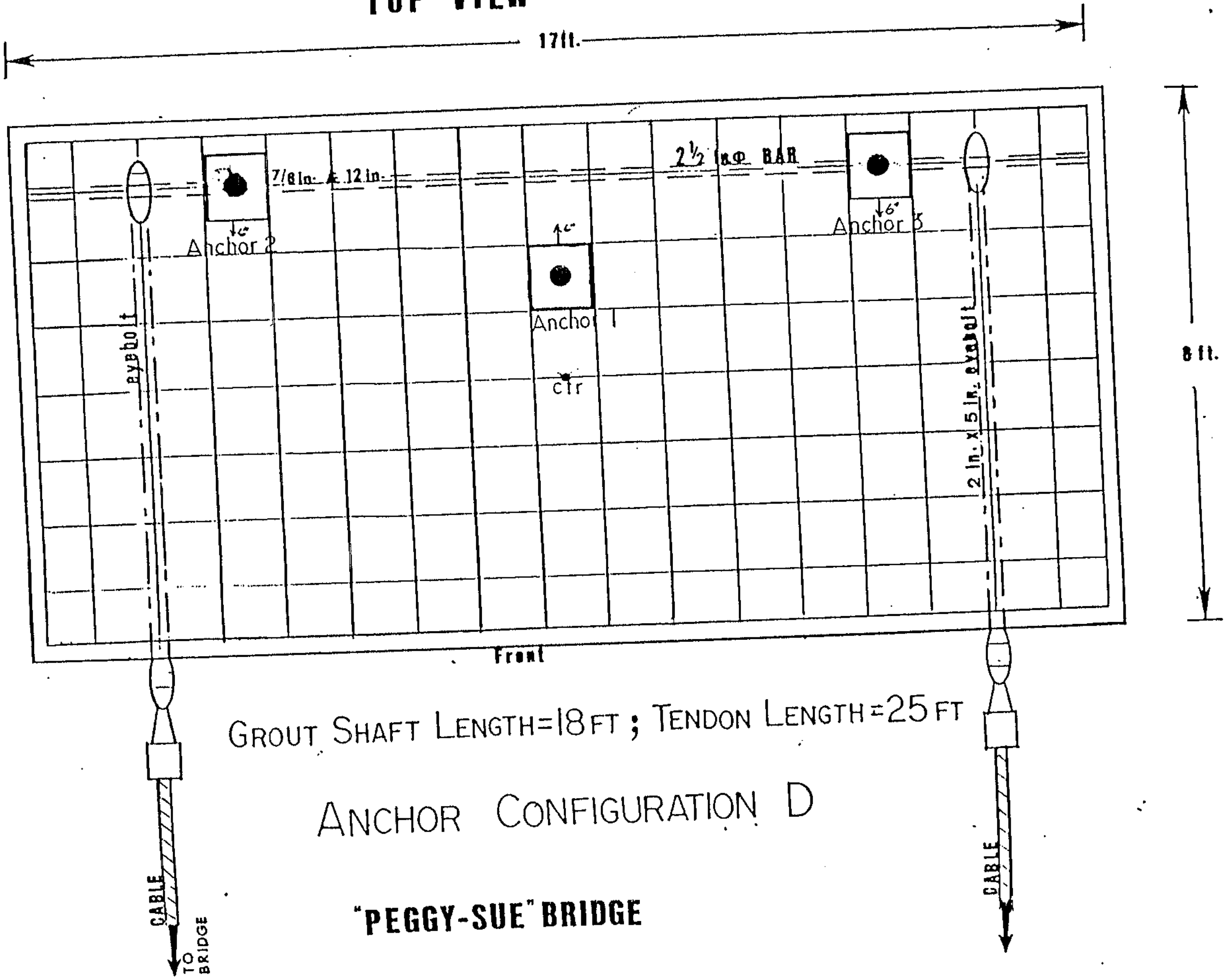
ANCHOR CONFIGURATION "C"
L=13ft.

"PEGGY-SUE" BRIDGE



$$\Sigma M_A = 1218.75'^k - 1101.33'^k = 117.4'^k \text{ ccw}$$

TOP VIEW



GROUT SHAFT LENGTH=18 FT ; TENDON LENGTH=25 FT

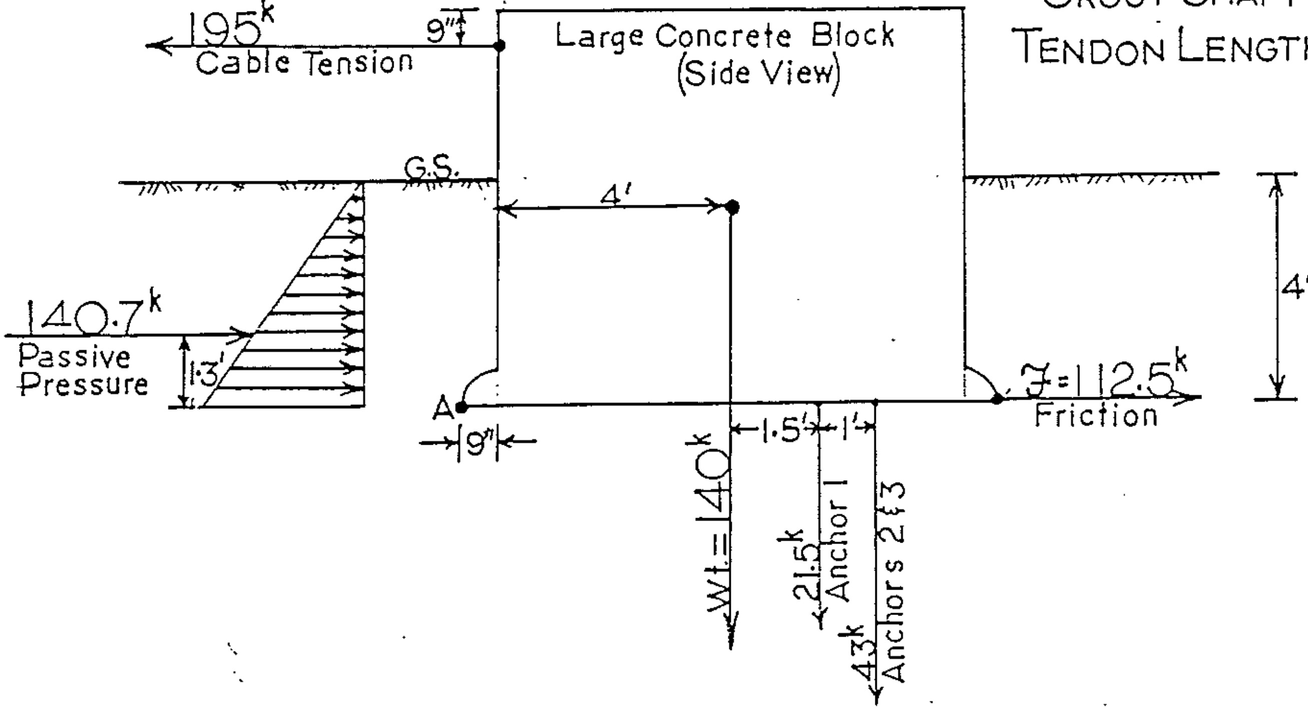
ANCHOR CONFIGURATION D

"PEGGY-SUE" BRIDGE

GROUND ANCHOR DESIGN

Grout Shaft Length=18'

Tendon Length=25'



M_A=Moment about point "A"

Anchor Configuration D

$$\Sigma M_A = 195^k(6.25') - 140^k(4.75') - 140.7^k(1.33') - 21.5^k(6.25') - 43^k(7.2$$

$$\Sigma M_A = 1218.75'^k - 1298.3'^k = -79.6'^k$$

∴ Sum of Resisting Moments Greater Than Turning-Over Moment by 79.6 ft-kips

$\Sigma F_{\text{Horizontal}}$ $195^k_{\text{Left}} - 253^k_{\text{Right}} = 58^k$ RESISTANCE TO SLIDING

---

*Note: If all 3 anchors were placed 2 ft to the right of ℄

$\Sigma M_A = 1218.75'^k - 1288'^k = -69.25'^k$ Resisting Moment EXCESS

Anchor Configuration E

University of California
Los Alamos National Laboratory

INSPECTION OF UTILITY BRIDGE

Project 9-LQO-Q4974

Submitted by
Kenneth R. White, P.E.
Samuel P. Maggard, P.E.

October 1990

INSPECTION OF UTILITY BRIDGE

Project 9-LQO-Q4974

The bridge supporting the gas utility line in the city of Los Alamos has been inspected and evaluated as requested in RFP 9-LQO-Q4974. The bridge is a cable stayed, open spandrel arch spanning Pueblo Canyon. The bridge is 550 feet long and consists of one main span suspended from two 2-inch diameter cables supported by one steel tower on each side of the canyon with the cables anchored into large concrete blocks cast into the volcanic tuff that makes up the plateau on each side of the canyon. Figure 1 in Appendix A shows a deck view of the bridge and Figure 2 shows a profile view.

The field inspection was conducted by Dr. Kenneth R. White, P.E., Dr. Samuel P. Maggard, P.E., and Mr. Ruben Gallegos on September 6-7, 1990. The inspection included a "hands-on" inspection of each joint of the bridge superstructure and careful observation of every cable anchor clamp of the bridge. The inspectors climbed the cable-support towers on each side of the canyon and carefully inspected each joint and bearing on the structures. The field inspection team checked each substructure component for deterioration or damage. The substructure components included the six concrete cable-anchor blocks and the base of the cable support towers.

The field inspection included careful visual inspection of the bridge components as well as "soundings" using a hammer. Other inspection equipment such ultrasonic probes, "feeler wires", and dye penetrant were available but were not used since the inspection team found no signs of significant duress on the steel superstructure.

The bridge is rated as fracture critical according to the National Bridge Inspection Standards (NBIS) since failure of one of the major components (main support cable, cable support tower, or cable anchorage) would most likely result in catastrophic collapse of the bridge. However, during the field inspection the team found no indications of significant duress. Despite the age of the bridge (41 years), the structure appears to be functioning very well. The cables, joints, cable clamps, floorbeams, and stringers are all in fair to good condition. Figure 3 shows a typical view of the cables, joints, cable clamps, and floorbeams. The hinges, rollers, and bearings are all functioning and show no signs of significant duress. Figure 4 shows hinges, bearings, and rollers.

The concrete cable anchorage blocks do show signs of long-term, freeze-thaw damage. The exposed surfaces of the concrete blocks are cracked with spalling and delamination. The south main-cable anchor block has some efflorescence caused by water leaching through the cracks in the concrete. All the anchorage blocks have been treated with epoxy crack sealant that is in good condition at this time and makes the concrete much more resistant to water penetration.

Various forms of corrosion were found on a majority of the bridge components. Corrosion forms included galvanic, crevice, and pitting. The galvanic corrosion is caused by the interfacing of dissimilar metals in the presence of an electrolyte (water). The crevice corrosion occurs within the gaps of mating surfaces of the structure where evaporation of moisture is slow. Pitting is the localized formation of deeper, narrow penetrations into the steel surfaces caused by scratches and nicks in the protective paint surfaces. Fortunately, none of the corrosion has progressed to a point of causing significant loss of section and/or duress in any of the primarily structural members. Further damage from the corrosion may be prevented by cleaning and painting the bridge superstructure and support towers.

The Structural Inventory and Appraisal Ratings required by the National Bridge Inspection Standards (NBIS) for highway bridges is shown in Appendix A. Several of the items do not apply to the utility bridge. Under the structural data; the bridge is classified as a steel suspension bridge (Item 43) over a waterway (Item 42).

The Condition and Appraisal Ratings are defined in the Recording and Coding Guide for the Nation's Bridges and the appropriate definitions have been reproduced and placed in Appendix B as reference material. The applicable condition ratings are as follows:

Deck - good condition with minor problems (Item 58).

Superstructure - good condition with minor problems (Item 59).

Substructure - satisfactory condition, some structural elements show some minor deterioration (Item 60).

The applicable appraisal ratings are:

Structural condition - equal to present minimum criteria (Item 67).

Waterway adequacy - superior to present desirable criteria (Item 71).

Routine maintenance on numerous components is needed to prevent damage or further deterioration of the structure. Most of these items are associated the cable connections, e.g. replacing missing cable clamps or missing nuts, tightening the sway cables, replacing frayed cables, and replacing a few cat-walk gratings. Specific maintenance items and their location are listed in detail in Appendix C.

Recommendations:

The following general recommendations are made with regard to the utility bridge.

1. Clean and paint the bridge superstructure and main cable supports. The bridge stringer, floorbeams, bearing, and walkway need protection from further corrosion damage. Typical corrosion is shown in Figure 5.
2. It is not necessary to paint the cables at this time since they appear in good condition and show no signs of corrosion.
3. Monitor the cable anchorage blocks to check on further freeze-thaw damage cause by water penetration and any sign of movement of the anchorage blocks or cable eyebars. Freeze-thaw damage is shown in Figure 6.
4. Tighten the loose sway cables and monitor the cable tensions on a regular schedule to prevent damage from wind loading.
5. Replace the securing bolt in the pin of the main cable anchor on the west end, south side. Proper tension on this bolt holds the pin sleeve in position as originally designed. The pin arrangement is shown in Figure 6.

6. The east tower requires some further preventative maintenance to minimize corrosive activity as detailed in Item 27 of Appendix C.

Appendix A

PHOTO PAGES

STRUCTURE: GAS ULITITY PIPE COUNTY: LOS ALAMOS

FEATURE CROSSED: PUEBLO CANYON DATE: SEPTEMBER 7, 1990

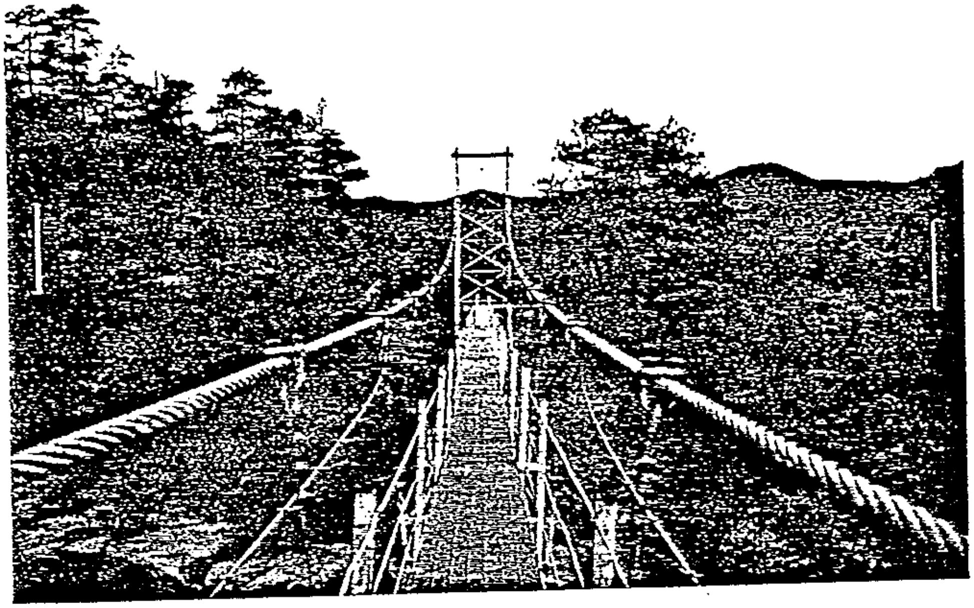


FIGURE 1: EAST - WEST DECK VIEW

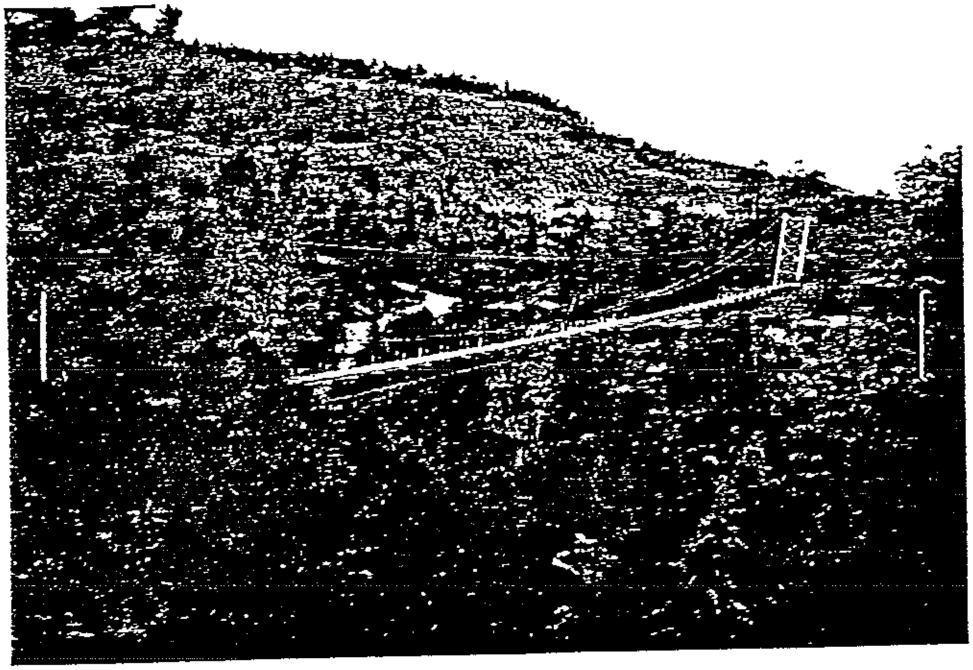


FIGURE 2: NORTH - SOUTH PROFILE

STRUCTURE: GAS ULITITY PIPE

COUNTY: LOS ALAMOS

FEATURE CROSSED: PUEBLO CANYON

DATE: SEPTEMBER 7, 1990

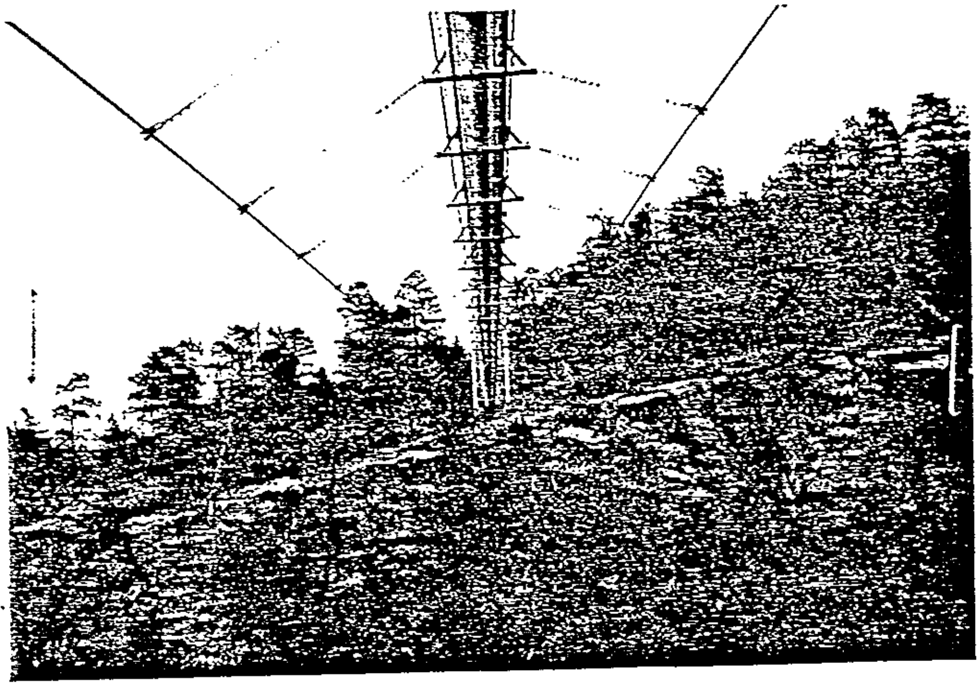


FIGURE 3: FLOORBEAM/STRINGER/SWAY CABLE; WEST - EAST

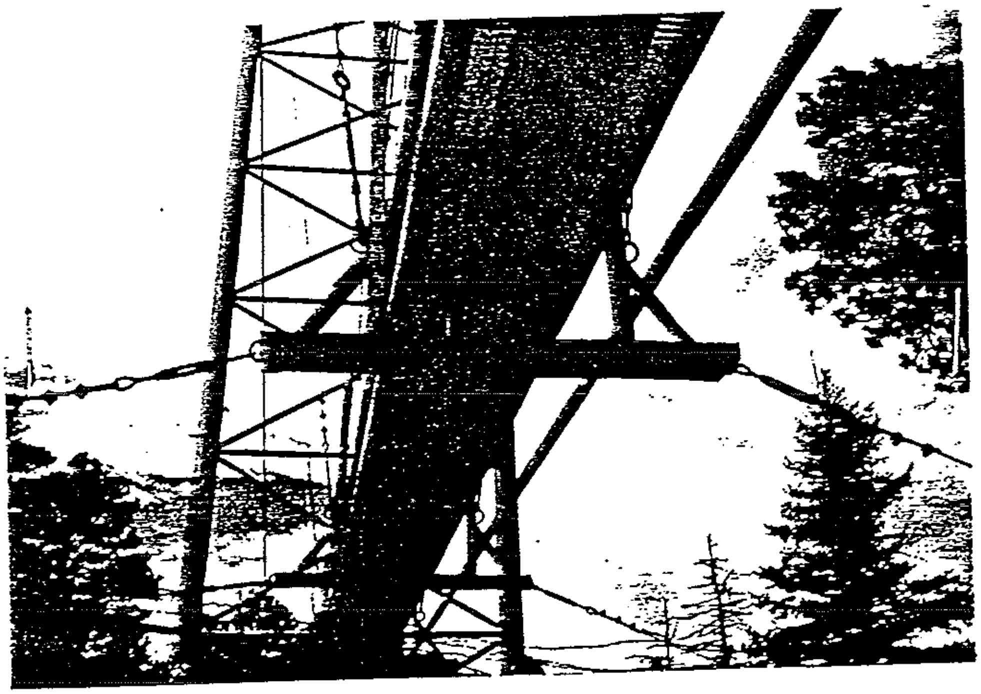


FIGURE 4: HINGES/ROLLERS/BEARINGS

STRUCTURE: GAS ULITITY PIPE COUNTY: LOS ALAMOS

FEATURE CROSSED: PUEBLO CANYON DATE: SEPTEMBER 7, 1990

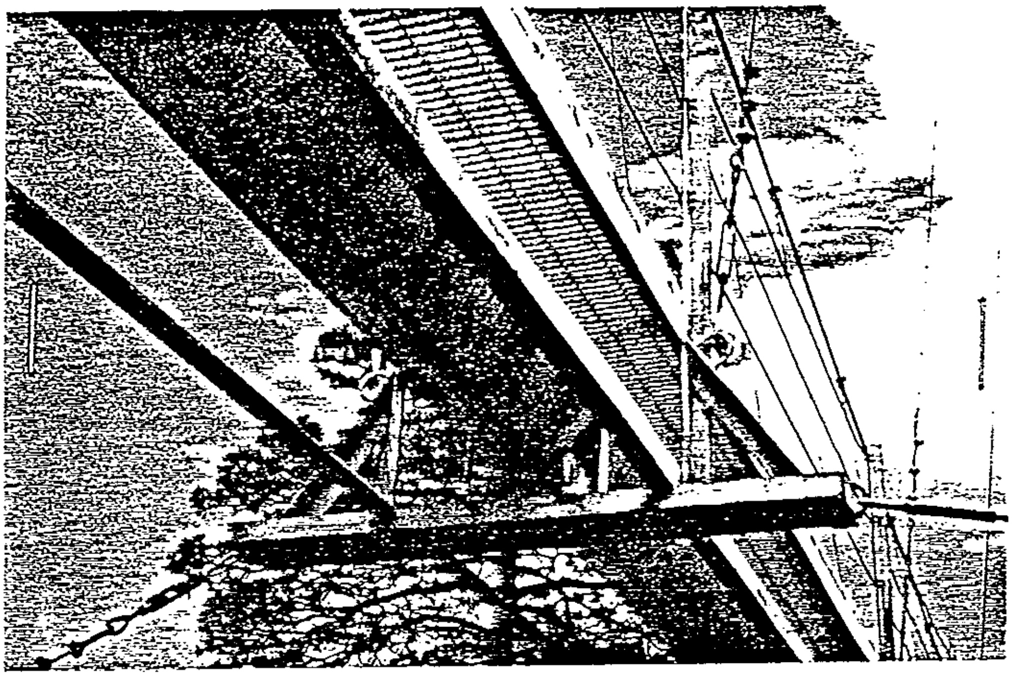


FIGURE 5: TYPICAL CORROSION ON THE BRIDGE

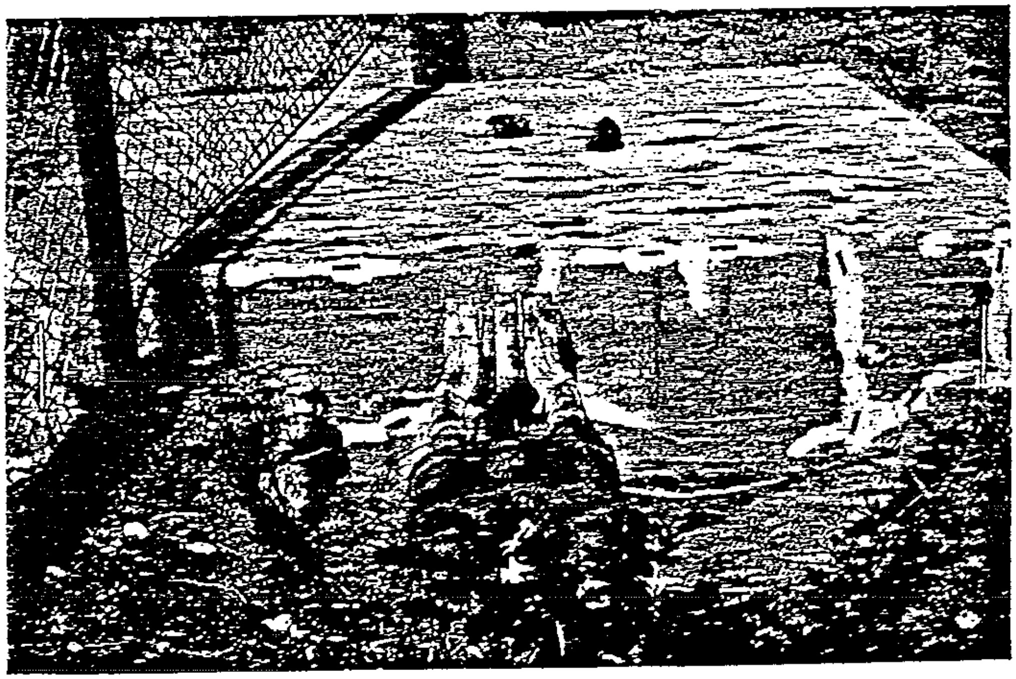


FIGURE 6: CABLE ANCHORAGE BLOCK AND ANCHOR PIN

STRUCTURE: GAS ULITITY PIPE

COUNTY: LOS ALAMOS

FEATURE CROSSED: PUEBLO CANYON

DATE: SEPTEMBER 7, 1990

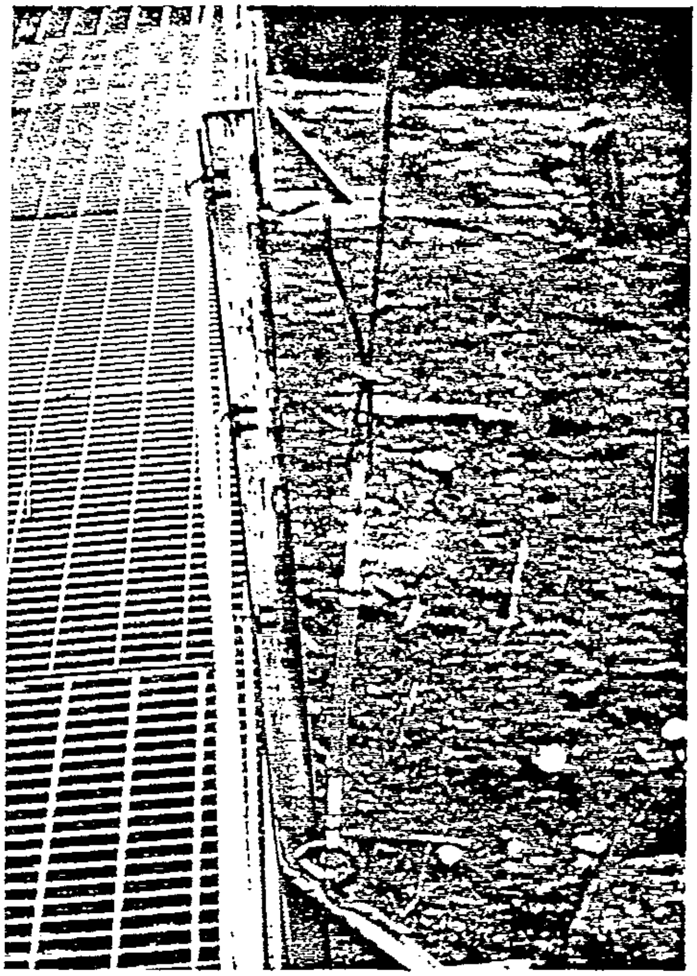


FIGURE 7:
MISSING CABLE CLAMPS

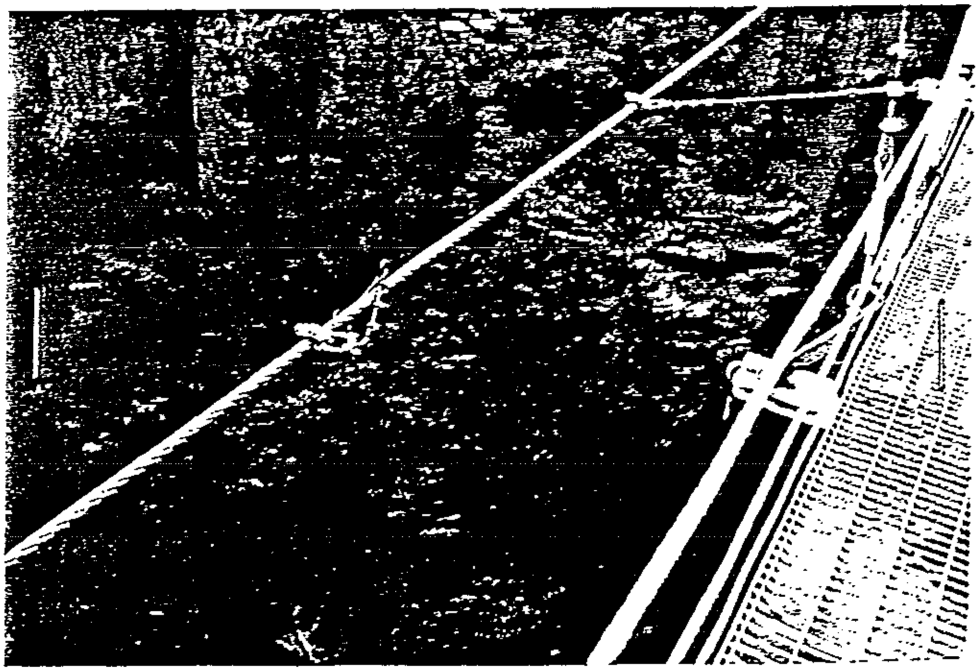


FIGURE 8:
MISSING TURNBUCKLE

STRUCTURE: GAS ULITITY PIPE　　　COUNTY: LOS ALAMOS

FEATURE CROSSED: PUEBLO CANYON　　　DATE: SEPTEMBER 7, 1990

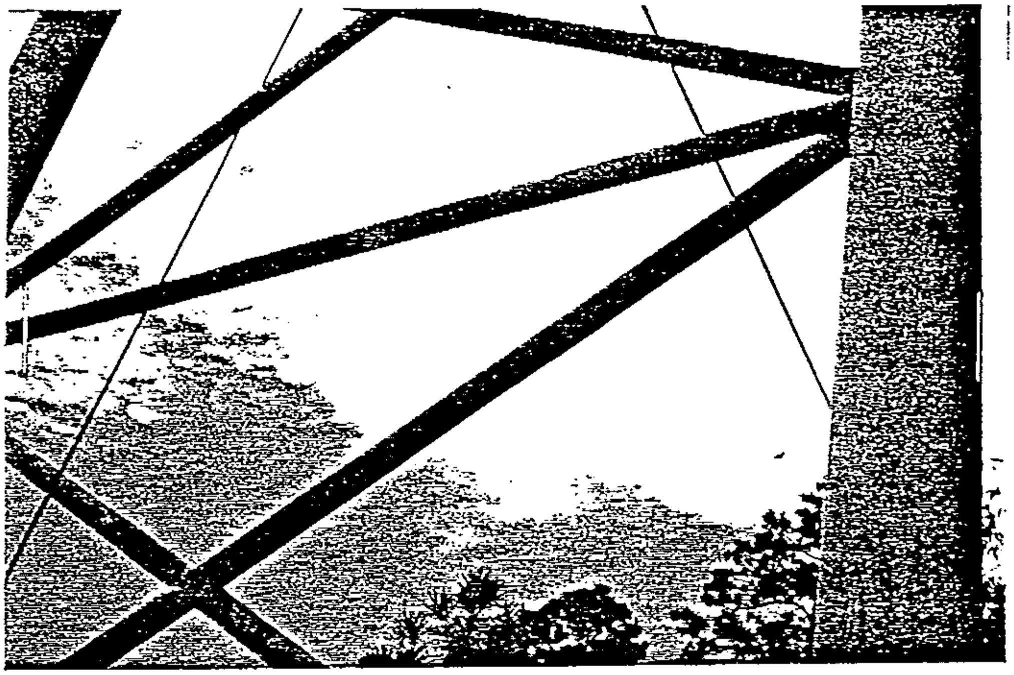


FIGURE 9: RUSTED X-BRACING

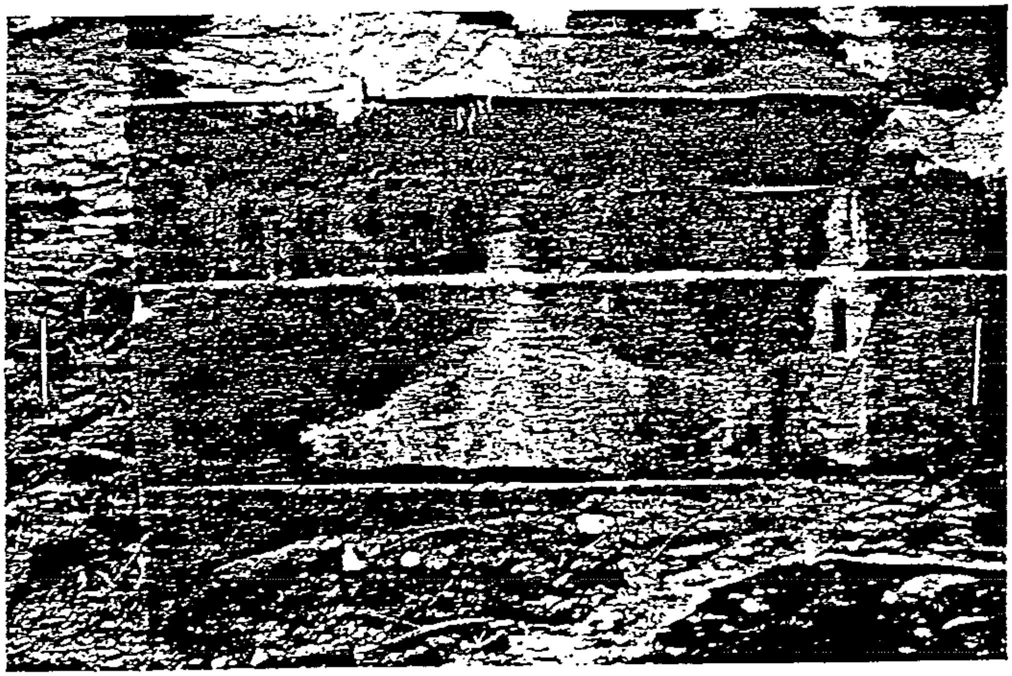


FIGURE 10: LEACHING OF ANCHORAGE BLOCK

Appendix B

STRUCTURAL INVENTORY AND APPRAISAL RATINGS

STRUCTURAL INVENTORY & APPRAISAL SHEET

STRUCTURE DATA:

Item	
36 - Traffic Safety Features	Not Applicable
41 - Open, Posted, or Closed	N/A
42 - Type Service	05
43 - Structure Type - Main	313
57 - Wearing Surface	N/A

CONDITION:

Item	Material	Condition anal.	Rating
58-Deck	steel grate	good except one panel	7
59-Superstructure	steel	good	7
60-Substructure	concrete	satisfactory	6
61-Channel & Ch Protection			N/A
62- Culvert & Retain. Walls			N/A
63- Est. Remaining Life			N/A
64- Operating Rating			N/A
65- Approach Roadway Condition			N/A
66- Inventory Rating			N/A

APPRAISAL:

	Deficiencies	Rating
67- Structural Condition	Controlled by Cond rating	6
68- Deck Geometry		N/A
69- Underclearances		N/A
70- Safe Load Capacity		N/A
71- Waterway Adequacy		9
72- Approach Roadway Alignment		N/A

Items 58 through 62 - Indicate the Condition Ratings

In order to promote uniformity between bridge inspectors, these guidelines will be used to rate and code Items 58, 59, 60, 61, and 62.

Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Evaluation is for the materials related, physical condition of the deck, superstructure, and substructure components of a bridge. The condition evaluation of channels and channel protection and culverts is also included. Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated.

The load-carrying capacity will not be used in evaluating condition items. The fact that a bridge was designed for less than current legal loads and may be posted shall have no influence upon condition ratings.

Portions of bridges that are being supported or strengthened by temporary members will be rated based on their actual condition; that is, the temporary members are not considered in the rating of the item. (See Item 103 - Temporary Structure Designation for the definition of a temporary bridge.)

Completed bridges not yet opened to traffic, if rated, shall be coded as if open to traffic.

The following general condition ratings shall be used as a guide in evaluating Items 58, 59, 60, 61, and 62:

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Item 58 - Deck 1 digit

This item describes the overall condition rating of the deck. Rate and code the condition in accordance with the above general condition ratings. Code N for all culverts.

Concrete decks should be inspected for cracking, scaling, spalling, leaching, chloride contamination, potholing, delamination, and full or partial depth failures. Steel grid decks should be inspected for broken welds, broken grids, section loss, and growth of filled grids from corrosion. Timber decks should be inspected for splitting, crushing, fastener failure, and deterioration from rot.

The condition of the wearing surface/protective system, joints, expansion devices, curbs, sidewalks, parapets, fascias, bridge rail, and scuppers shall not be considered in the overall deck evaluation. However, their condition should be noted on the inspection form.

Decks integral with the superstructure will be rated as a deck only and not how they may influence the superstructure rating (for example, rigid frame, slab, deckgirder or T-beam, voided slab, box girder, etc.). Similarly, the superstructure of an integral deck-type bridge will not influence the deck rating.

Item 59 - Superstructure 1 digit

This item describes the physical condition of all structural members. Rate and code the condition in accordance with the previously described general condition ratings. Code N for all culverts.

The structural members should be inspected for signs of distress which may include cracking, deterioration, section loss, and malfunction and misalignment of bearings.

The condition of bearings, joints, paint system, etc. shall not be included in this rating, except in extreme situations, but should be noted on the inspection form.

On bridges where the deck is integral with the superstructure, the superstructure condition rating may be affected by the deck condition. The resultant superstructure condition rating may be lower than the deck condition rating where the girders have deteriorated or been damaged.

Fracture critical components should receive careful attention because failure could lead to collapse of a span or the bridge.

Item 60 - Substructure 1 digit

This item describes the physical condition of piers, abutments, piles, fenders, footings, or other components. Rate and code the condition in accordance with the previously described general condition ratings. Code N for all culverts.

All substructure elements should be inspected for visible signs of distress including evidence of cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion. The rating given by Item 113 - Scour Critical Bridges, may have a significant effect on Item 60 if scour has substantially affected the overall condition of the substructure.

The substructure condition rating shall be made independent of the deck and superstructure.

Items 67, 68, 69, 71, and 72 - Indicate the Appraisal Ratings

The items in the Appraisal section are used to evaluate a bridge in relation to the level of service which it provides on the highway system of which it is a part. The structure will be compared to a new one which is built to current standards for that particular type of road as further defined in this section except for Item 72 - Approach Roadway Alignment. See Item 72 for special criteria for rating that item.

Items 67, 68, 69, 71, and 72 will be coded with a 1-digit code that indicates the appraisal rating for the item. The ratings and codes are as follows:

Code	Description
N	Not applicable
9	Superior to present desirable criteria
8	Equal to present desirable criteria
7	Better than present minimum criteria
6	Equal to present minimum criteria
5	Somewhat better than minimum adequacy to tolerate being left in place as is
4	Meets minimum tolerable limits to be left in place as is
3	Basically intolerable requiring high priority of corrective action
2	Basically intolerable requiring high priority of replacement
1	This value of rating code not used
0	Bridge closed

Tables are provided to evaluate items 67, 68, 69 and 71, and shall be used by all evaluators to code these items. They have been developed to closely match the descriptions for the appraisal evaluation codes of 0 to 9. The tables shall be used in all instances to evaluate the item based on the designated data in the inventory, even if a table does not appear to match the descriptive codes. For unusual cases where the site data does not exactly agree with the table criteria, use the most appropriate table to evaluate the item.

Level of service goals is a concept that several States have introduced into their bridge management to determine the need for bridge improvements.

Level of service goals are target values for selected bridge characteristics that are used to assess bridge adequacy. The goals may vary depending on the highway functional classification, traffic volume, and other factors. The goals are set with the recognition that widely varying traffic needs exist throughout highway systems and that many bridges on local roads can adequately serve traffic needs with lower load and capacity geometric standards than would be necessary for bridges on heavily traveled main highways. The degree to which a bridge is deficient can be measured by comparing bridge characteristics with level of service goals. Shortfalls from the goals determine the type and extent of improvement needs. The shortfalls are useful for comparing bridge needs and setting improvement priorities. Needs determined by level of service goals that are graduated to traffic levels and the characteristics of the vehicles served can differ greatly from those determined by a single standard that applies to all bridges, for example the AASHTO *A Policy on Geometric Design of Highways and Streets 1984*.

19. VC45-N rusted turnbuckle.

20. SC46-N,S rusted turnbuckles.

21. VC46-N missing cable clamp.

22. VC48-S rusted turnbuckle.

23. SC48-N,S rusted turnbuckle.

24. SC49-N,S rusted turnbuckle, nut missing at both ends of cable.

25. VC50-N,S rusted turnbuckle, missing cable clamps.

26. SC50-N,S rusted turnbuckle, loose cable on south side.

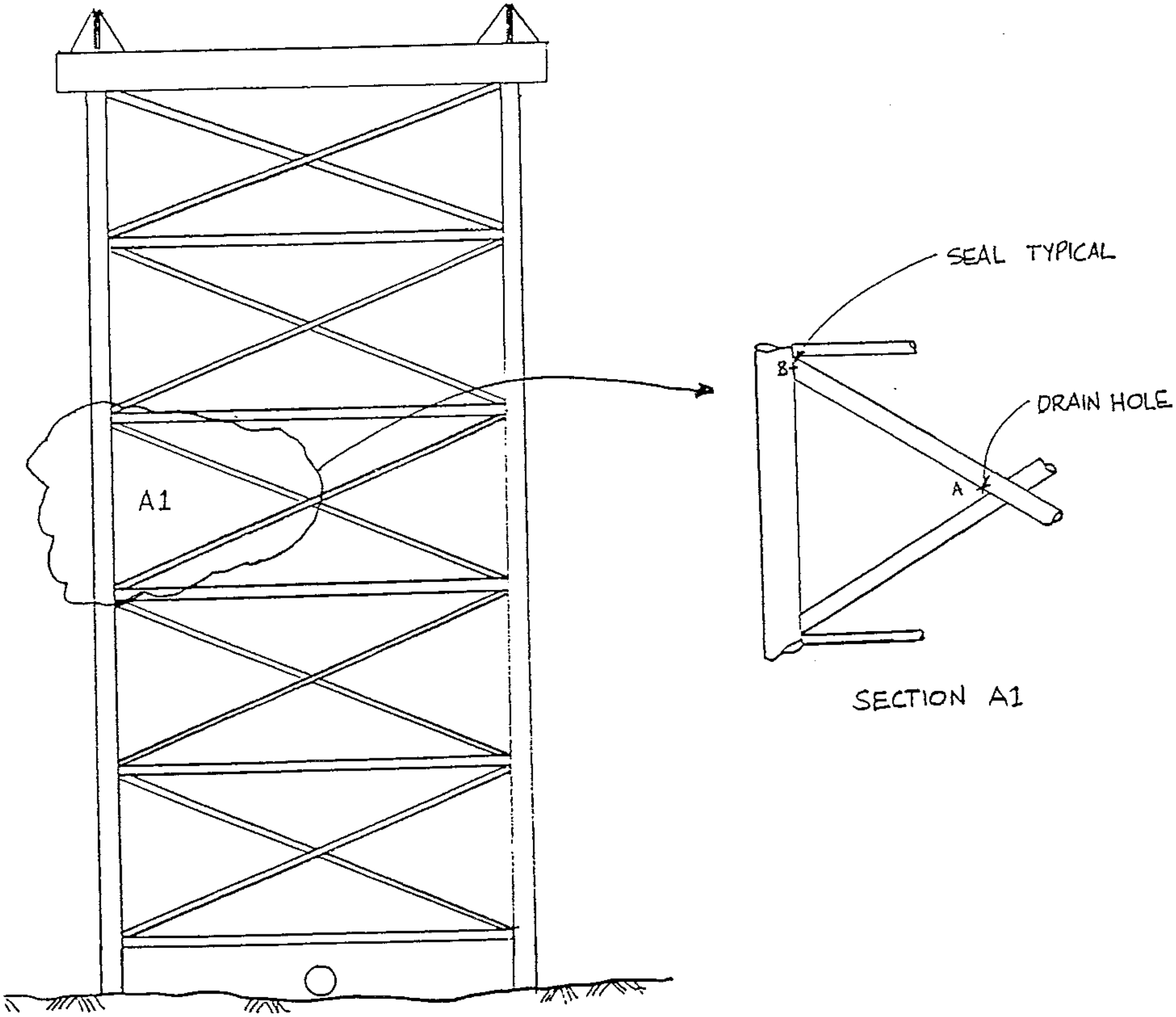
27. Support tower, east end - water entering into X-bracing at 4th from bottom, S.E. end. Pipe bracing is rusting uniformly. The rest of the tower exhibits pit corrosion. Tower should be sand blasted and painted. Defects are shown in Figure 9. The joints of the X-bracing should be sealed to prevent water out of pipes. Drains at lower ends of bracing would drain any water trapped at the present time. See the attached sketch.

28. Southeast main cable anchorage block is leaching at N.E. corner as shown in Figure 10.

Appendix C

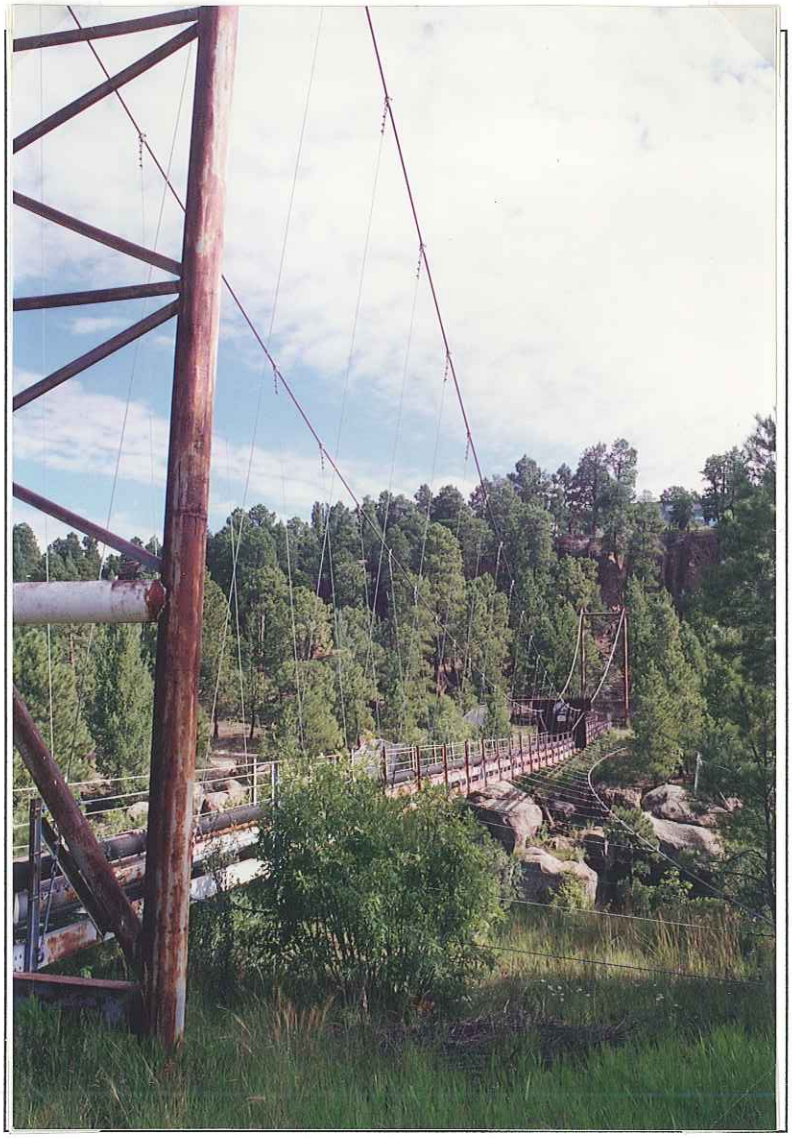
Listed below are specific items that require maintenance or repair. Each item is located according to joint number attached to the cables (metal tags at cable clamps). VC will refer to the vertical cable (suspension cable) between the main cable (MC) and bridge floorbeams (attachment to the bridge superstructure). SC will refer to the sway cables. N will reference cables on north side and S will reference cables on south side of the bridge.

1. Section of walkway grating missing at west end of bridge.
2. VC1-S (vertical cable, number 1, south side) is missing cable clamps as shown in Figure 7.
3. VC1-N has frayed cable.
4. SC4-S (sway cable, number 4, south side) cable is loose.
5. VC5-S has oversized turnbuckle that is rusting and has odd spacing of cable clamps.
6. SC8-S has a rusted turnbuckle.
7. SC9-N,S both sway turnbuckles are rusting.
8. SC19-N nut missing on sway cable clamp.
9. VC23-S nut missing on cable clamp.
10. VC25-N two nuts missing on cable clamps.
11. VC25-S frayed cable ends and corroded turnbuckle. Nut also missing from sway cable clamp.
12. SC27-S turnbuckle missing as shown in Figure 8.
13. SC35-S loose cable.
14. SC37-S loose cable.
15. SC38-S loose cable.
16. SC39-S loose cable.
17. SC40-S loose cable.
18. SC45-S loose cable, rusted turnbuckle.


A1
SEAL TYPICAL
B
DRAIN HOLE
A
SECTION A1

APPENDIX D

Little Peggy Sue Bridge Photographs and Drawings

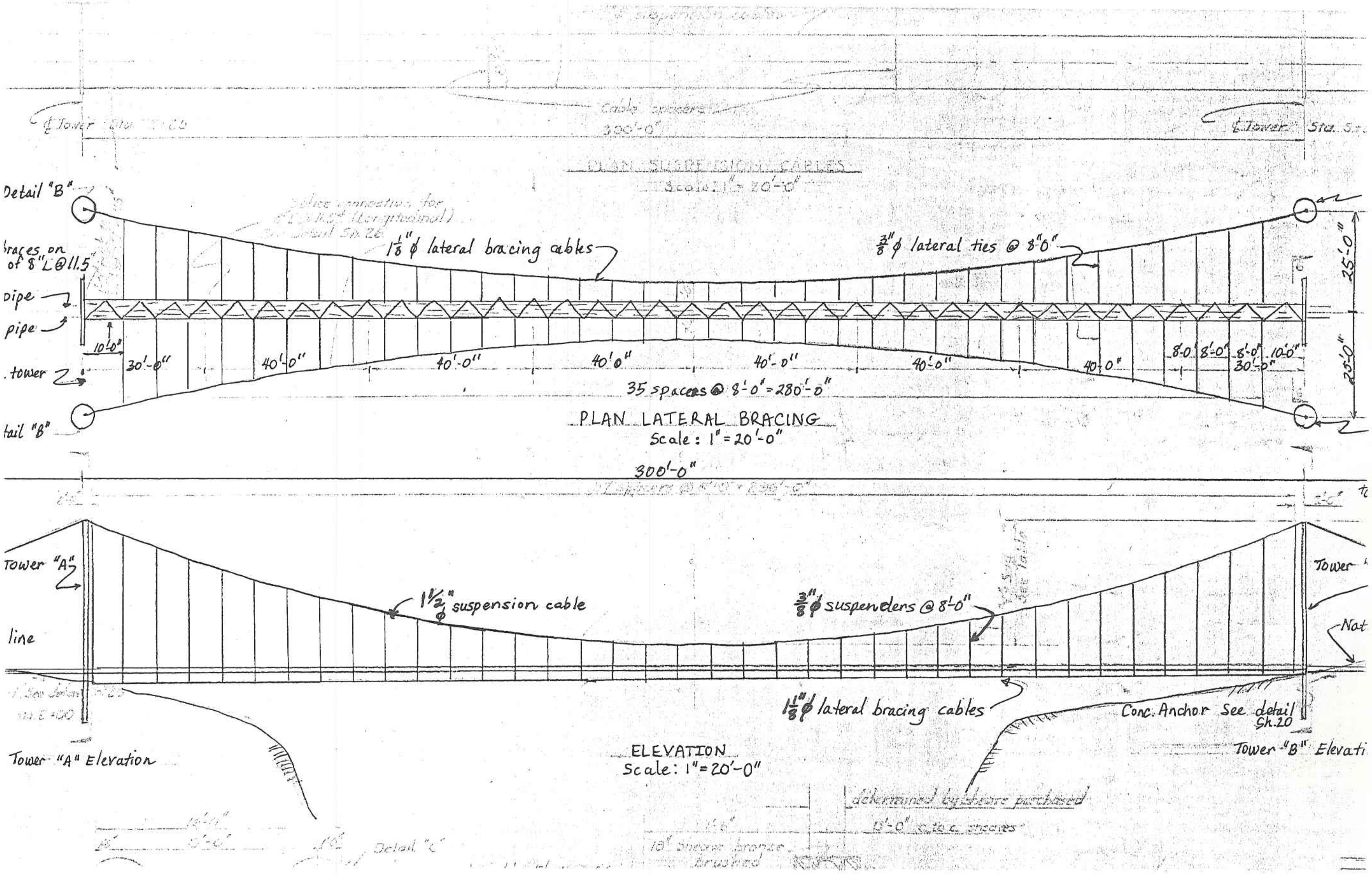


Little Peggy Sue Bridge, dir. ~NNW

Little Peggy Sue Bridge, view from south end, dir. ~NNE

Little Peggy Sue Bridge, dir. ~N

Little Peggy Sue Bridge, south tower, dir. ~W



"Little Peggy Sue" Bridge
(dwg. redrawn from poor quality original)
#SFA-FL-1/25 U.S. AEC