

# The Design of a Suspension Bridge Anchorage System

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*The use of a rock anchorage system with a redundant waterproofing scheme proved to be an economical and safe design solution that accommodated site conditions.*

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**T**he Father Louis Hennepin Suspension Bridge over the Mississippi River in Minneapolis, Minnesota, replaces a steel arch bridge that was constructed in 1889. The steel arch bridge was preceded by two suspension bridges, the first of which was a wood tower suspension bridge that was constructed in 1855. This bridge was the first bridge to span the Mississippi River. It was replaced by a stone tower bridge in 1876. Foundations for the earlier suspension bridges are preserved near the west tower of the new bridge.

The bridge was constructed in longitudinal halves in order to permit uninterrupted traffic movement on Hennepin Avenue during the construction. The south half of the bridge was constructed during 1988-89. The north half was

constructed during 1989-90. The main suspension span is 625 feet long, and is flanked by simply supported deck welded plate girder back spans. The overall bridge length — including the main span, back spans, and approach spans — is about 1,050 feet. The bridge was designed to carry six lanes of AASHTO Standard HS25 Truck and Lane Loading, with bikeways and wide sidewalks as shown in Figure 1. The cost for the suspension bridge was approximately \$20 million, plus an additional amount of approximately \$6 million for the approaches.

## Bridge Layout

The design of the Hennepin Suspension Bridge called for a state-of-the-art anchorage system that would restrain a large tension force from the suspension cables. Since the bridge was to be constructed in two longitudinal halves, a total of four suspension cables were required that, in turn, required eight cable anchorages. The center two cables were located seven feet apart. Thus, the interior anchor systems were designed to handle two cables; the exterior were designed to handle one. The three critical elements of this anchorage system, which is designed to economically restrain a tension force of seven million pounds in a localized area, are:

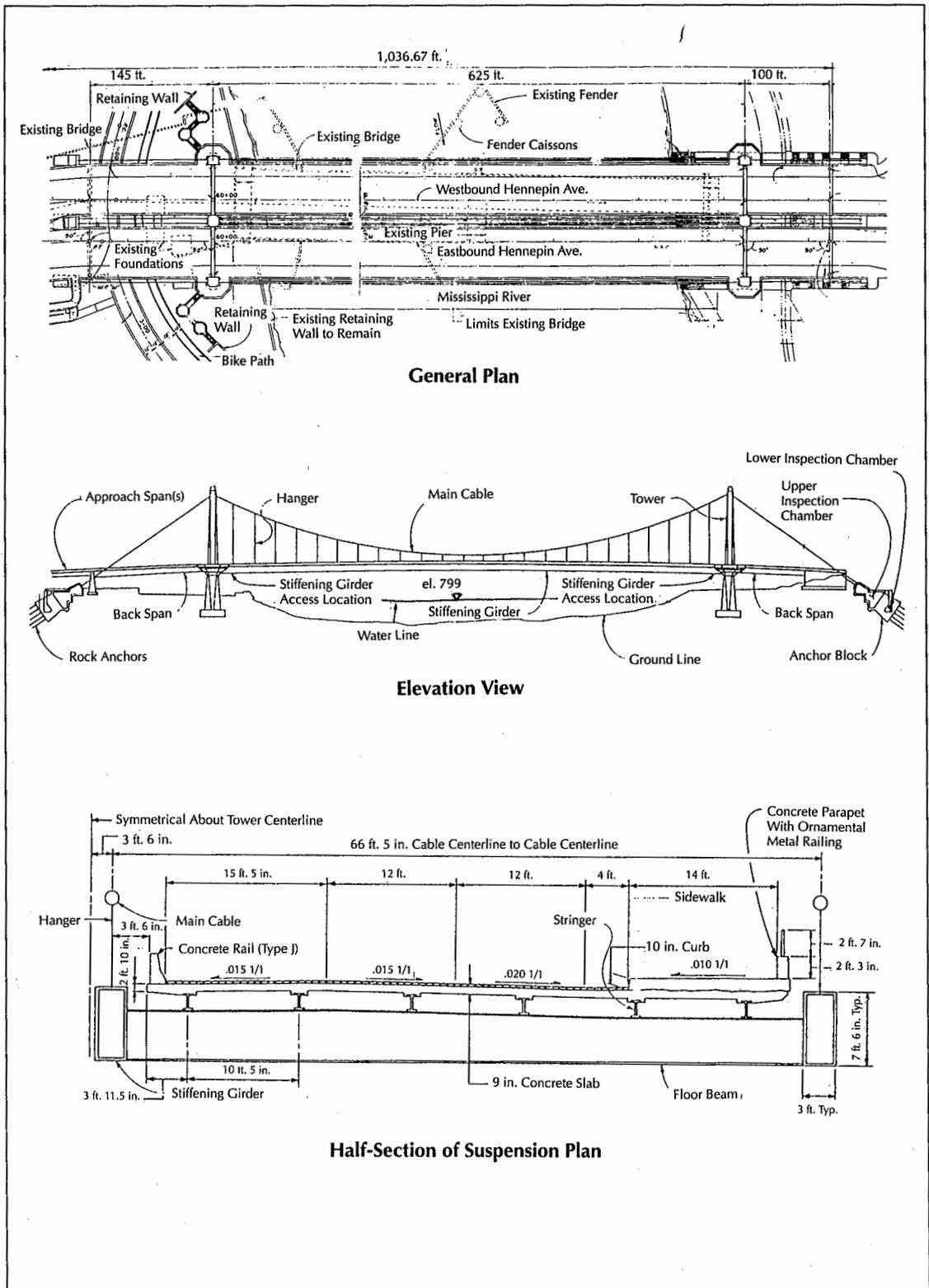


FIGURE 1. Hennepin Bridge plan/elevation.

- The permanent rock anchor tieback system
- The concrete transfer block
- The anchor chamber waterproofing system

## Permanent Rock Anchor Tieback Design/Construction

A variety of suspension bridge anchoring systems were evaluated for this project. Among them were gravity anchors, prestressed rock anchors, drilled shafts and a self-anchored system. This study encompassed cost, constructability, long service life, risk and aesthetic factors for each of the proposed systems. Site geometrics eliminated the self-anchored bridge alternative. The use of groups of 30-inch diameter drilled shafts located at several different angles of inclination was investigated, but it was dismissed due to constructability problems, cost and the lack of redundancy.

The final selection came down to a comparison between two alternatives: the gravity mass system and the rock anchor system. The mass anchor system provided the lowest design risk factor due to major dependence on the physical mass of the anchor and the relative predictability of the geotechnical bearing parameters. However, the large amounts of rock excavation and concrete placement that would have been required resulted in a high cost for this option.

The proximity of the sandstone bedrock resulted in making the prestressed rock anchors a viable choice. Potential concerns with the rock anchor system were as follows:

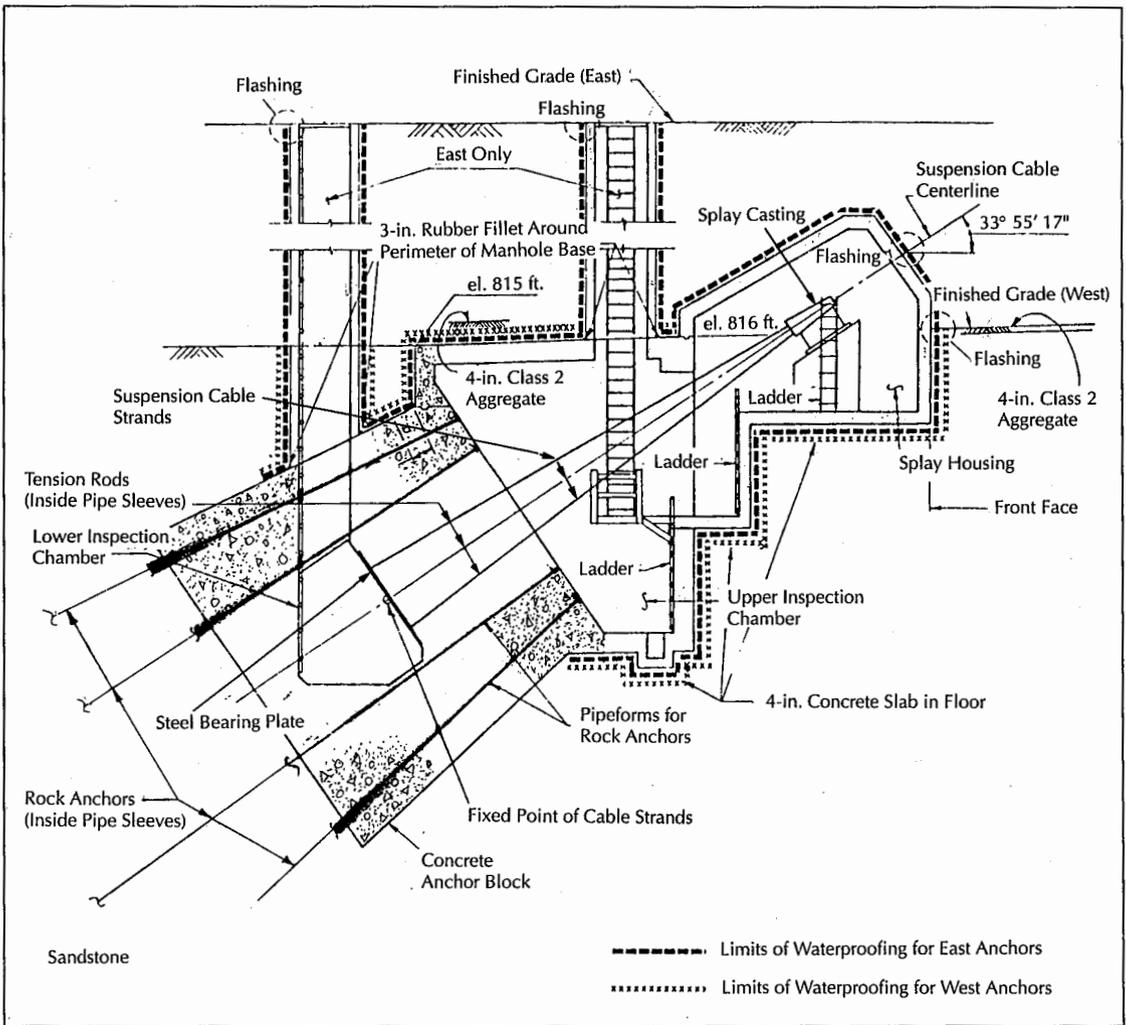
- Permanent prestressed rock anchor tiebacks are a relatively new technique to the United States. They were first installed in the 1960s in any significant quantity. Anchors have been used in Europe prior to that, however.
- The corrosion protection system has the potential to create problems. Failures have been reported, largely due to corrosion in the vicinity behind the anchor head, in aggressive environments.
- Potential long-term creep of the sandstone bedrock would reduce anchor capacity. Other anchor losses (predicted to

be fairly small) could also mean that the anchors would lose additional capacity over time.

- Confined space limitations meant that each rock anchor tendon would need a design capacity of 700 kips, which is quite large in comparison with other permanent anchors installed to date.
- The failure mode of a "wedge" or "bulb" pullout of the sandstone bedrock would need to be evaluated.
- Many rock anchor systems are proprietary and it is difficult to specify a system with the latest technology for a publicly funded project.
- The bond strength (or shear strength) between the grout and various rock types are different. Even for sandstone, available guidelines give a range. What value should be used? What will the safety factor be?

Cost estimates showed the rock anchor system would be the economical method to transfer the suspension cable tension force to the sandstone bedrock. The designers concluded that if the above concerns could be suitably addressed, this method would be selected. The overall concept is shown in Figure 2. The suspension cable locks off to a metal plate bearing against a concrete anchor block that is restrained to the sandstone by prestressed permanent rock anchors. The anchor block and chambers can be more easily visualized by reviewing the anchor isometric shown in Figure 3.

Even though the use of permanent "prestressed" rock anchors is relatively new to the United States, its longer track record in Europe provides a significant body of experience. A number of reputable rock anchor contractors have been successfully installing rock anchors in the United States. The Post-Tensioning Institute (PTI) has recently developed its "Recommendations for Prestressed Rock and Soil Anchors." The project's designer decided to require that the anchors be installed per the PTI recommendations. The designer also stipulated that the general contractor engage a specialty rock anchor foundation subcontractor that has had at least ten years of successful



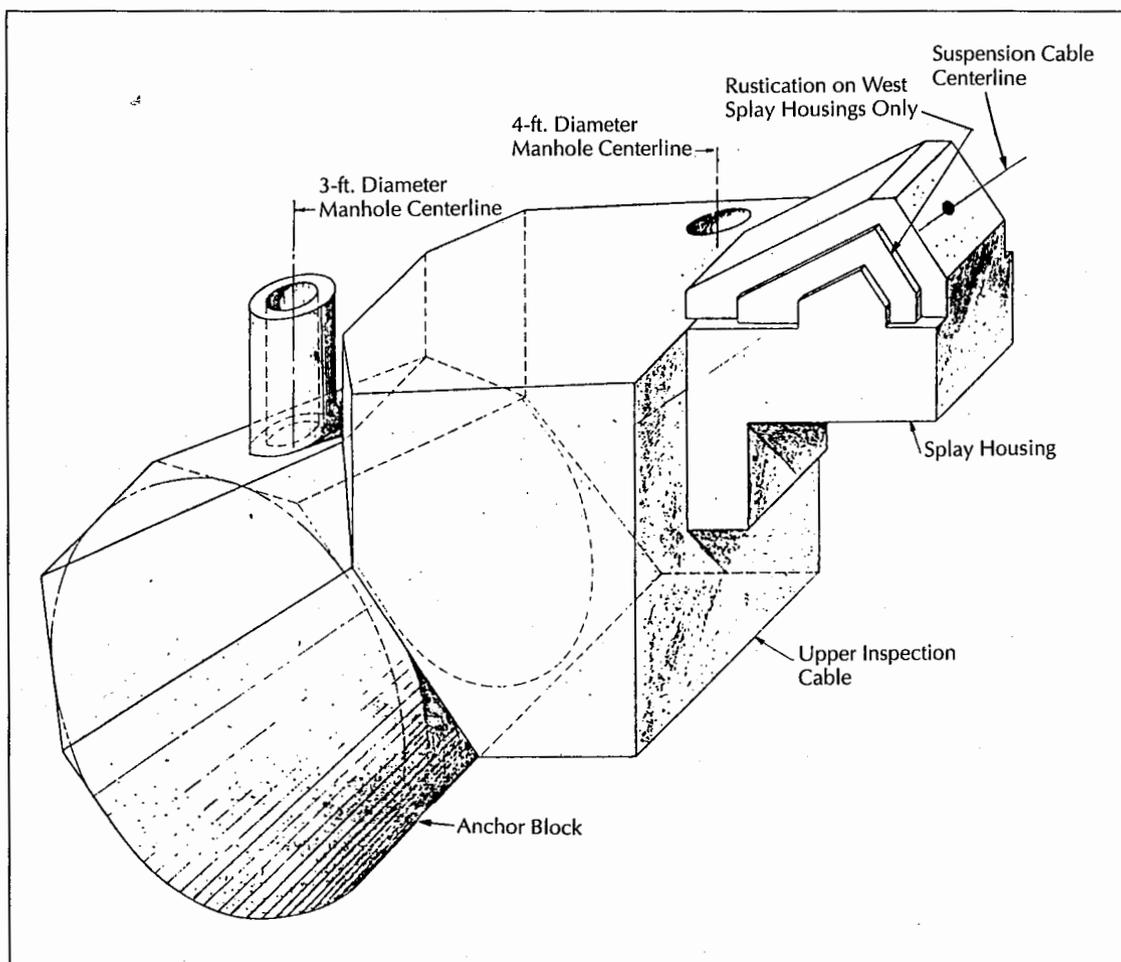
**FIGURE 2. Typical section through the anchor.**

experience installing similar high-capacity anchors. Competitive bids were received by general contractors.

Because the longest life possible was desired for the anchorages, the designer decided to require a dual corrosion protection system, which essentially is a grouted tendon encased in a corrugated polypropylene sheath that, again, is surrounded by grout (see Figures 4, 5 and 6). The grout and the sheath provide the double protection. The ground environment was evaluated and determined to be non-aggressive in regard to corrosion. In addition, the anchor head will be protected from the weather. The head is housed inside a humidity-controlled inspec-

tion chamber and is covered with a removable grease-filled cap.

The PTI recommendations require an extensive testing program as well as acceptance criteria for jacking (stressing) the anchors. Performance, proof and seven-day lift-off tests were required to ensure that the anchors were properly tensioned. The design incorporated additional (spare) locations for anchors that would not be used unless failures were encountered (anchors not meeting acceptance criteria). The anchor head was designed so that it could be jacked in the future (without unseating the wedges) to verify the anchor force. At the same time, metal shims could be inserted



**FIGURE 3. Anchorage isometric.**

behind the head to increase the anchor force, if necessary.

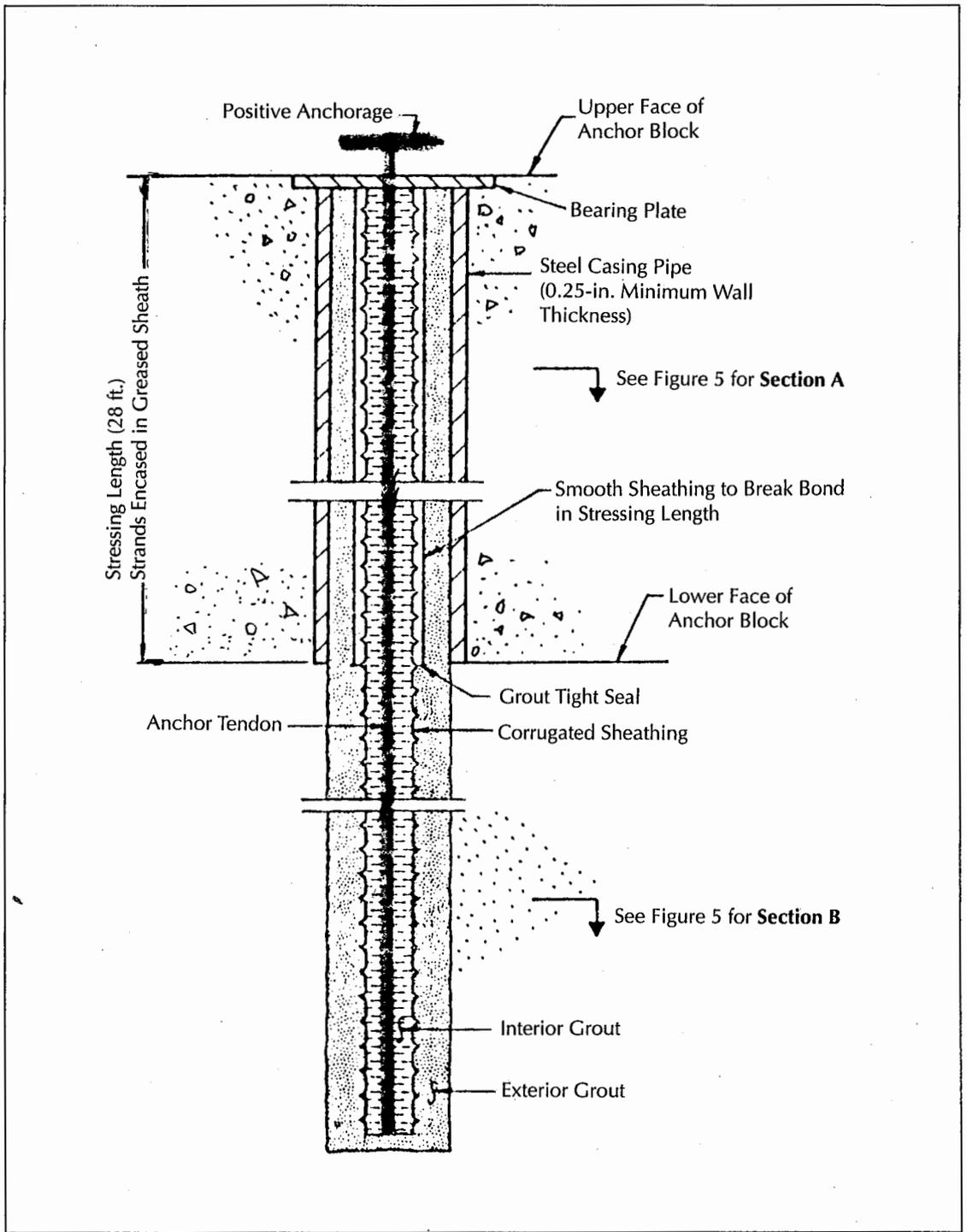
The anchors were fanned out radially (see Figures 7 and 8) and the bond lengths were staggered to engage a large volume of sandstone bedrock in order to increase the safety factor against a bulb (or wedge) failure.

The anchor system was designed and specified as a generic system that could incorporate a number of desired features of several proprietary systems available in the marketplace. Any system proposed by the contractor would need to satisfy the required features: either strands or bars would be acceptable.

A pullout test was specified to be performed by the rock anchor contractor in order to determine the ultimate shear strength of grout to sandstone. A length of approximately five feet

was grouted in sandstone and tested by jacking. The results were somewhat inconclusive due to construction variables. However, the project engineer was confident that the design shear strength was achieved.

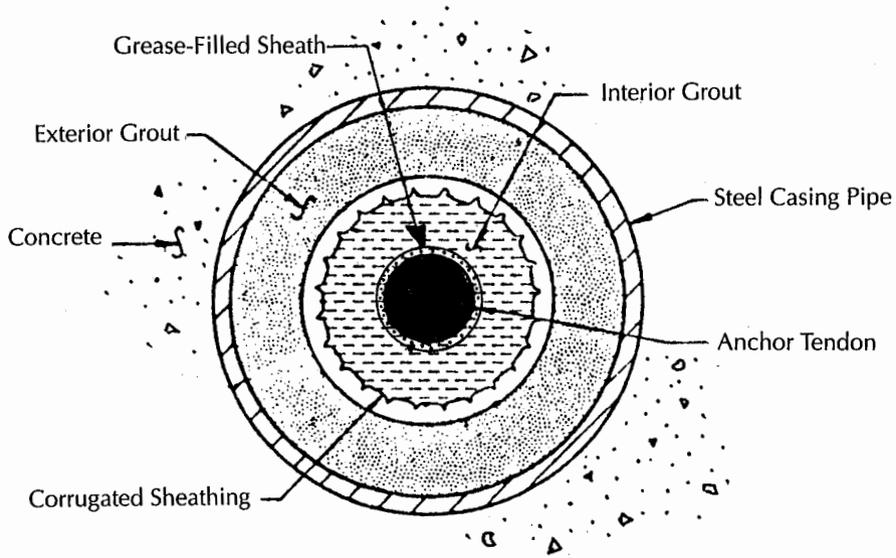
The design rock anchor load was approximately 700 kips, with total anchor lengths ranging from 80 to 105 feet. Each tendon consisted of nineteen 0.6-inch diameter seven-wire strands inside a corrugated polypropylene sheath. The tendon was grouted along its full length, including the stressing length (inside and outside of the sheath) in one step so there would be no cold joints that would possibly lead to a breach of the dual corrosion protection system. The unbonded (stressing) length consisted of grease-injected strands encased in plastic jackets.



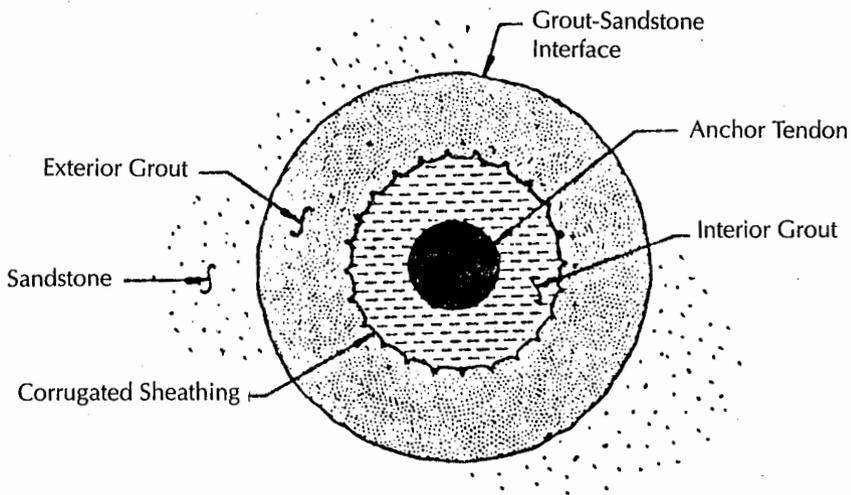
**FIGURE 4. Rock anchor detail.**

The drilled holes required for the strands/sheath (tendon) were seven inches in diameter. Holes were drilled with standard

drill rigs and cuttings were flushed to the surface with water. Careful drilling and flushing procedures were implemented in order to pre-



Section A (Stressing Length) from Figure 4



Section B (Bond Length) from Figure 4

Note: No scale.

FIGURE 5. Rock anchor sections.

vent wall sloughing and caving. Grout (with no additives) was pumped to the bottom of each hole in a plastic tube where it could rise gradu-

ally and simultaneously inside and outside the corrugated sheath. It was necessary to install the corrugated sheath at the site in order to

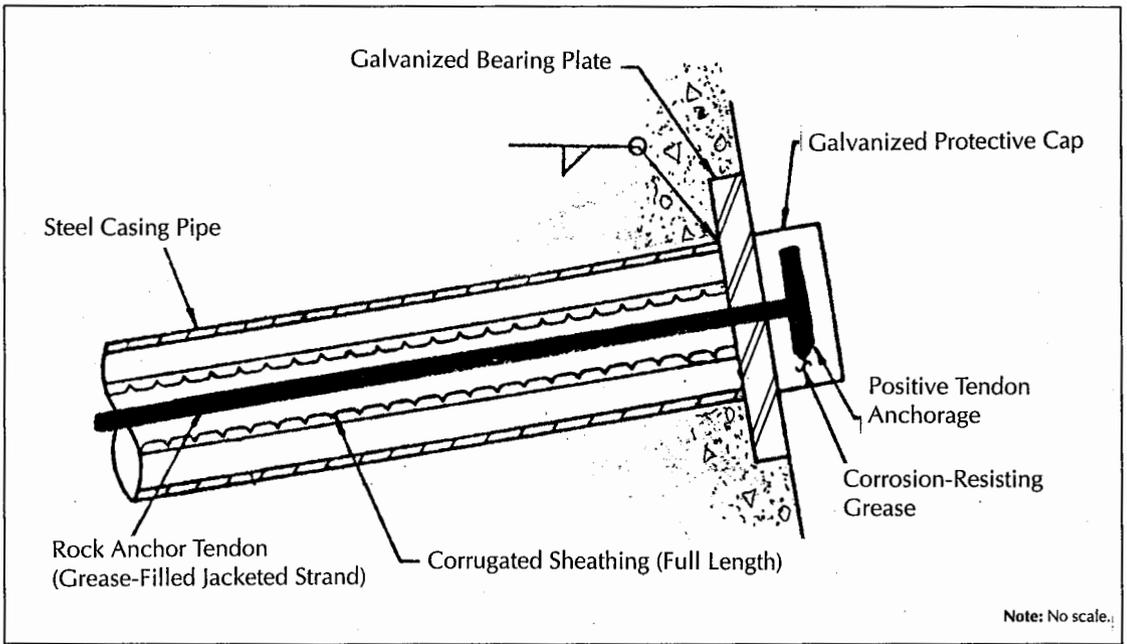


FIGURE 6. Rock anchor head detail.

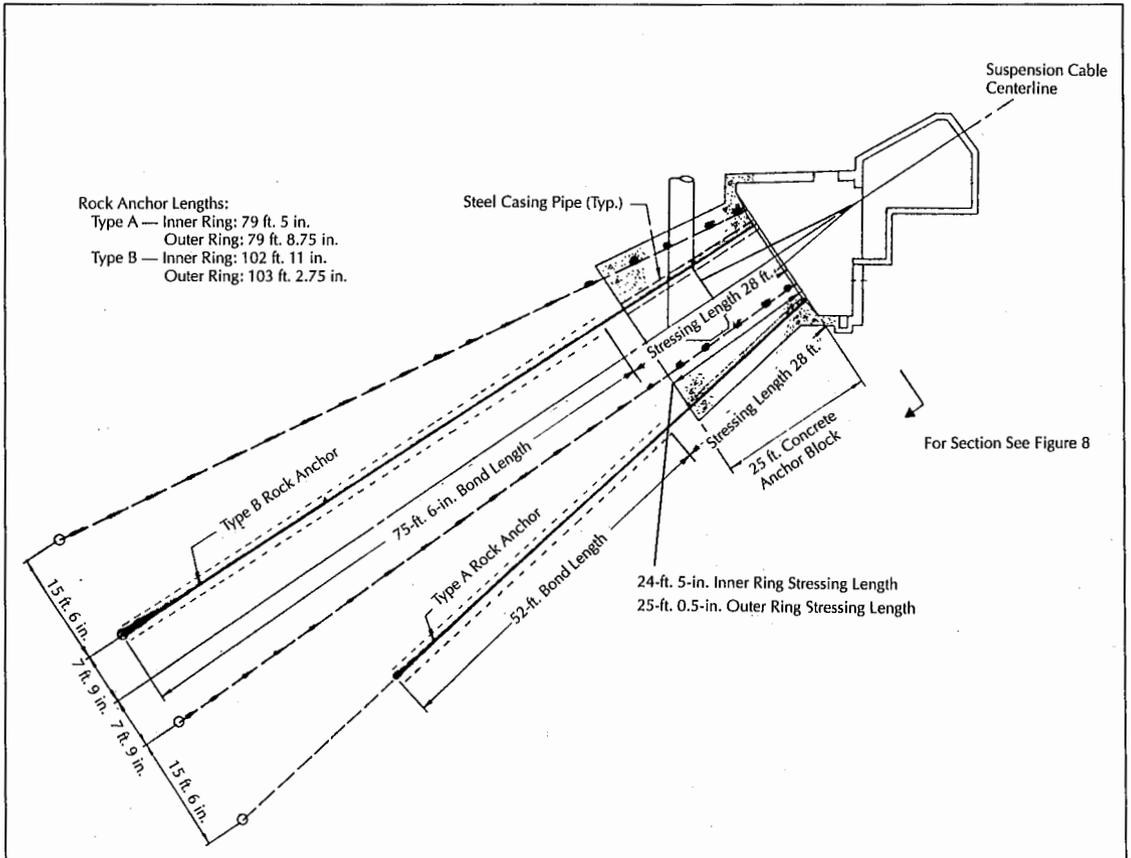
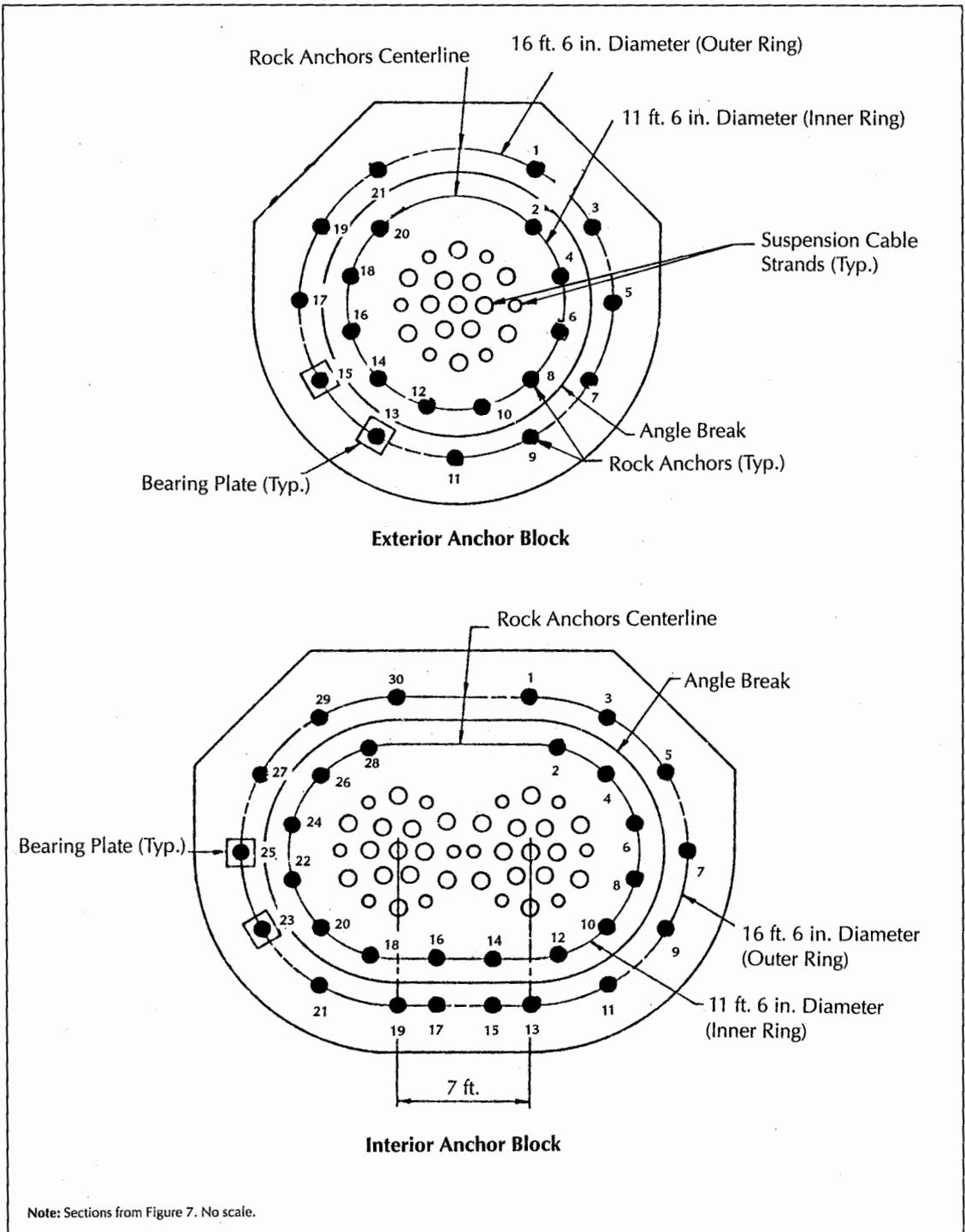


FIGURE 7. Rock anchor layout.

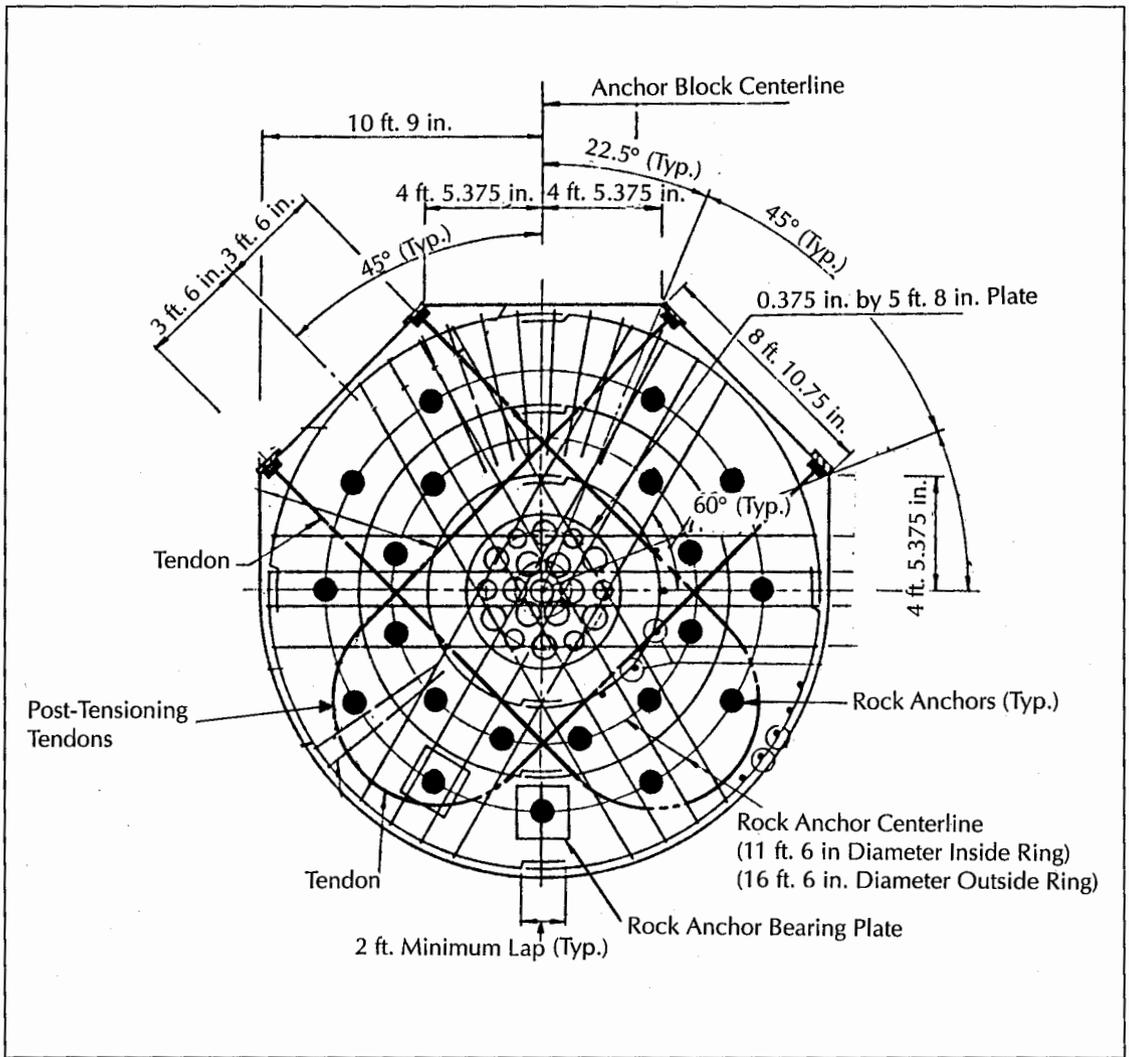


Note: Sections from Figure 7. No scale.

**FIGURE 8. Section of interior and exterior anchor blocks.**

avoid damage from shipping and coiling. Standard hydraulic rams were used for jacking. The unit bid price for installation and stressing

was \$9,000 each for the shorter tendons and \$10,000 each for the longer tendons, inclusive of drilling.



**FIGURE 9. Top face of an exterior anchor block.**

### Concrete Transfer Block Design/Construction

A structural element was required to transmit the tension load from the suspension cable to the rock anchors. The 19 strands of the suspension cable fanned out radially to a diameter of six feet. The strands were socketed and attached to a steel threaded rod that was inserted in steel sleeves through the concrete anchor block, and locked off against a steel bearing plate with a large nut (see Figure 2). The rock anchors were located around the perimeter of this plate. Therefore, an element that would transfer the load from the bearing

plate to the perimeter rock anchors was required. Metal pipe sleeves were provided through the concrete anchor block for both the suspension cable tension rods and the rock anchors in order to facilitate placement and alignment.

A cylindrical concrete transfer block was designed (see Figure 2). Potential failure modes of this block included shear (pullout) of the metal bearing plate and bursting from potentially large compressive stresses.

For the shear (pullout) stresses, the shear capacity of the concrete was supplemented with shear friction reinforcement designed per the AASHTO specifications. Possible lo-

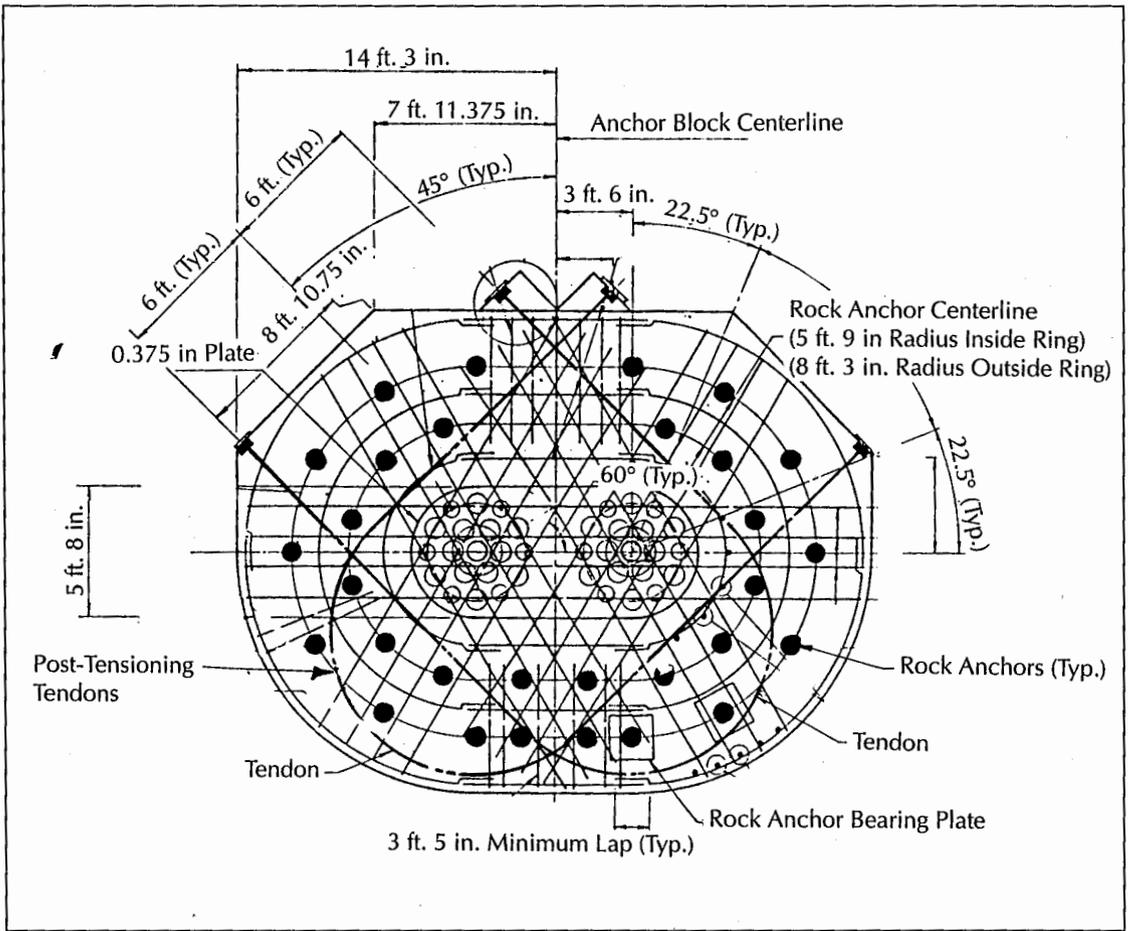


FIGURE 10. Top face of an interior anchor block.

cations for this reinforcement were identified and clearances for concrete placement were checked.

For the bursting (or splitting) stresses, mild steel reinforcement hoops were provided around the perimeter of the cylinder in addition to radial looped post-tensioning (see Figures 9 and 10). The designer's intent was to reinforce the concrete in order to counteract the bursting stresses with a combination of a passive reinforcing bar system and an active post-tensioning system, and to prevent any concrete cracking that could lead to water intrusion and corrosion. The calculation of these stresses was based on the same theory that was used to determine the bursting stresses behind a post-tensioning anchor head.

To determine the tensile (splitting) forces behind an anchor head, the equation is:

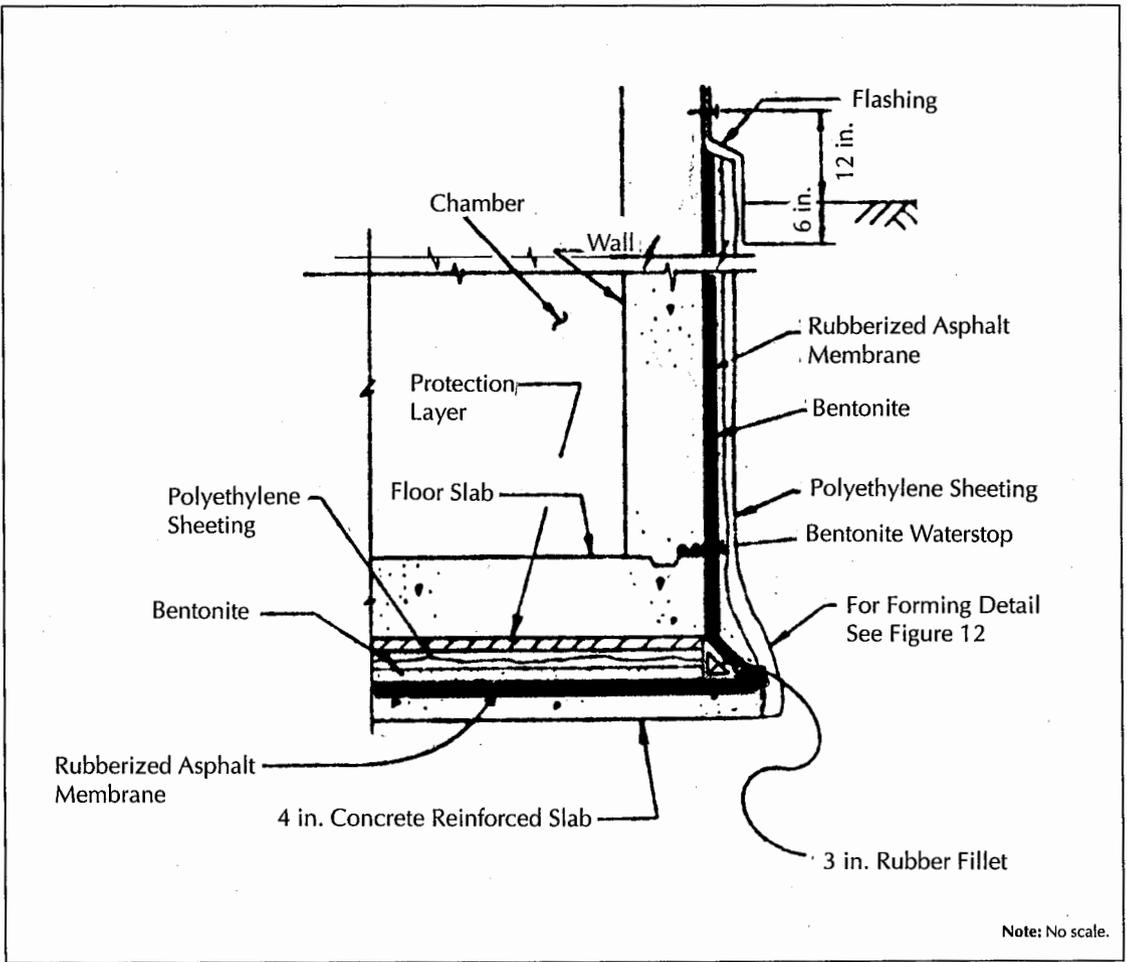
$$Z = 0.3P (1-a/d)$$

Where:

$Z$  = the total splitting or bursting force,  
 $P$  = the tendon force,  
 $a$  = the depth of the anchor place, and  
 $d$  = the depth of the concrete section.

The development of this equation is discussed in Leonhardt's "Prestressed Concrete Design and Construction."<sup>1</sup>

Either spiral (hoops) or mats of straight reinforcement can be provided directly behind the anchor head to handle the stresses. Because of the potentially large splitting force, the limited space for mild reinforcement and the desire to provide an active system, the radial post-tensioning system was designed and detailed based on the above splitting force equation that



**FIGURE 11. A corner detail for the anchor chamber waterproofing system.**

was modified in order to reflect a massive section.

The mild steel reinforcement was installed by the general contractor along with the radial post-tensioning hardware. The concrete design strength was 4,500 psi.

Unit bid prices were \$0.45 per pound for the mild reinforcement, \$225 per cubic yard for the concrete and \$9,000 for each anchorage's radial post-tensioning system.

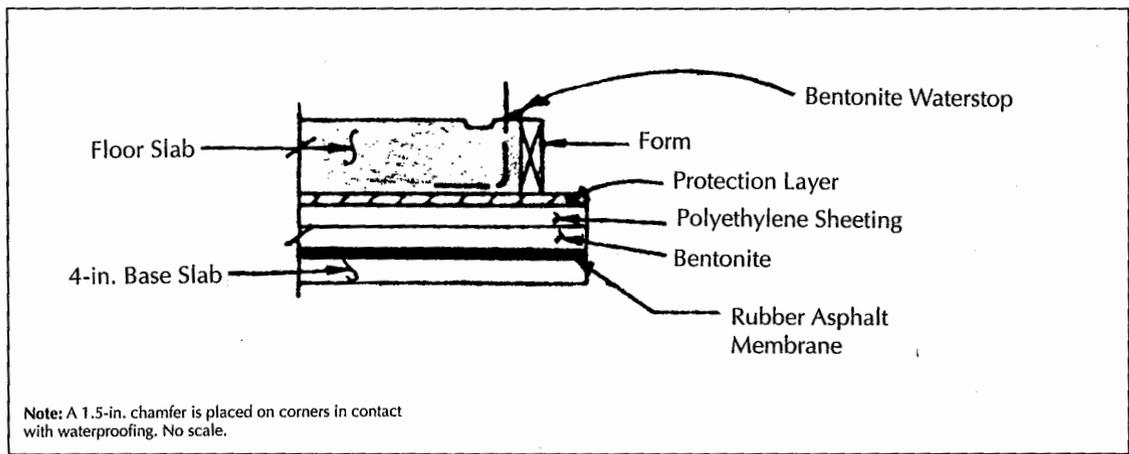
### Anchor Chamber Waterproofing Design/Construction

Buried chambers (or vaults) were required for suspension cable installation and for future inspection purposes. These chambers are located below the water table and are buried under the approach roadway with up to fifteen feet of fill

placed above them (see Figure 2). There was a need to provide a dry environment for long-term corrosion protection of the suspension cable and related hardware. Because of that requirement and because the waterproofing system would not have access to repair any leak, the designer decided to provide for two independent waterproofing systems.

After studying the desirable features of numerous available systems, the designer decided to combine the two following systems for this particular installation:

- A layer of bonded rubberized asphalt membrane that was glued to the exterior of the concrete chamber walls.
- A layer of sprayed bentonite that was covered by a polyethylene sheet and a protec-



**FIGURE 12. Waterproofing floor slab forming detail.**

tion board in order to protect the systems against damage from the backfilling operation.

While the rubberized membrane is a positive impermeable sheet of uniform thickness, it does have seams that can leak. It also does not lend itself to accommodating irregular shapes and corners.

The bentonite system is sprayed on so there are no seams. Also, it covers irregular shapes well. Nevertheless, the presence of salts (which could form in the future) can affect its swelling properties and, therefore, detract from its waterproofing ability.

Both systems can present problems if constructed using poor workmanship. The intent of providing dual systems was to prevent moisture infiltration in a submerged condition, assuming worst case scenarios.

Again, specifications were developed on a generic basis with these features required. Special details were devised for the floor and corner terminations and detailed flashing requirements were specified (see Figures 11 and 12). Manufacturers and applicators were required to provide shop drawings of all corner and termination details. The specifications mandated that the work to be completed by experienced applicators under the direction of the general contractor and of the manufacturer's representatives, as well as under the constant inspection of the project's engineer.

A de-humidification system was also pro-

vided within each chamber to keep the relative humidity at or below 35 percent, at which point the rate of steel corrosion slows to a low rate. The chambers were relatively small, so no active method of air circulation was felt to be needed. Open pipe sleeves connect the lower inspection chamber to the upper inspection chamber in order to allow passive air circulation.

The unit bid price for the entire waterproofing system was \$2.65 per square foot.

## Conclusion

A thorough field inspection program was put in place to carefully monitor and record all of the contractors' work regarding the construction of the anchorage system components. A maintenance manual was prepared to guide future engineers in monitoring the performance of these anchorages. The Hennepin Avenue Suspension Bridge is now complete and the anchorage system, as well as the rest of the structure, is functioning as intended and serving as a landmark structure in the historical district of Minneapolis.

**ACKNOWLEDGMENTS** — *The new suspension bridge was designed by Howard Needles Tammen & Bergendoff for the Hennepin County Department of Public Works. For oversight and review, staff at the Minnesota Department of Transportation, in addition to the FHWA, supplemented county personnel. The general contractor, Johnson Brothers Corp., Litchfield, Minnesota, engaged nationally experi-*

enced Nicholson Construction Company of Bridgeville, Pennsylvania, as the specialty rock anchor subcontractor. The radial post-tensioning hardware was supplied by VSL Corporation.



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Bridge anchorage system and tower structures. During the construction, Hughes served as HNTB's Resident Engineer to augment Hennepin County's on-site engineering staff. He is currently Director of Engineering in HNTB's office in Irvine, California, where he manages large bridge projects in California.

#### REFERENCE

1. Leonhardt, F., "Prestressed Concrete Design and Construction," William Ernst & Son, Berlin, 1964.