1999 Harry H. Edwards Industry Advancement Award Winners

The Harry H. Edwards Industry Advancement Award is presented each year to honor precast/prestressed concrete projects that show superior creativity and innovation, thereby advancing the technological expertise of the industry and providing new ideas with great potential for additional applications. Four projects were selected this year as worthy of special recognition for their innovation and ground-breaking technology.

Brief summaries of the four projects are given below:

- **Tennessee State Route 50 Bridge Over Happy Hollow Creek —** This nine-span precast, prestressed concrete bulb-tee bridge is the longest totally jointless, integral abutment bridge constructed in the United States. The bridge is virtually maintenance free with neither expansion bearings or joints to maintain.
- **San Mateo-Hayward Bridge —** An innovative precast concrete frame design was used in this bridge seismic retrofit project to transfer loads from the existing foundation to cast-in-steel-shell piles and decrease pier displacements during seismic activity.
- **Benaroya Hall —** Precast concrete structural components and flawlessly cast architectural panels were used in a "box within a box" design to provide sound isolation and performance-quality acoustics for this concert hall situated in a noisy urban environment.
- **Arched Precast Wall System, University of Minnesota —** Curved precast concrete wall panels provide the interior surface for a sandstone cave surrounding a new underground library where limited construction space and environmental conditions made other construction alternatives impossible.

More detailed descriptions of the above projects are given in the following pages.

Tennessee State Route 50 Bridge Over Happy Hollow Creek

Edward P. Wasserman, P.E.
Civil Engineering Director, Structures
State of Tennessee Department
of Transportation
Nashville, Tennessee

At 1175 ft 2½ in. (358 m), the structure carrying State Route 50 over Happy Hollow Creek is the longest totally jointless, integral abutment bridge constructed in the United States. The structure is a nine-span precast, prestressed concrete AASHTO 72 in. (1829 mm) bulb-tee girder bridge with composite concrete deck. The bridge spans are designed as simply supported for the dead loads of the girder and deck slab but the structure is continuous for live loads and composite dead loads. As a result of the design, the bridge is virtually maintenance free, containing neither expansion bearings nor joints to maintain. This article describes the major features of the bridge and discusses the design concept used in attaining a jointless bridge deck with integral abutments.

The structure on Tennessee State Route 50 over Happy Hollow Creek is a nine-span AASHTO 72 in. (1829 mm) bulb-tee girder bridge with composite concrete deck (see Fig. 1). With an overall length of 1175 ft 2½ in. (358 m), it is the longest totally jointless, integral abutment bridge in the United States. Situated in Hickman County, Tennessee, the bridge was constructed as part of a major highway improvement project.

The bridge is designed to be virtually maintenance free, containing neither bearings or joints to maintain or replace. Special details at the end of the 25 ft (7.62 m) approach roadway pavements accommodate an elastomeric concrete/silicone sealing joint that protects the approach roadway from bridge thermally induced movements.

The bridge spans are designed as simply supported for the dead loads of the girder and deck slab but the structure is continuous for live loads and subsequent composite dead loads.

The structure has a 44 ft (13.4 m) wide roadway with spans ranging in length from 129 to 140 ft (39.3 to 42.7 m). By multiplying the total length and width of the bridge, the total deck area is found to be about 54,000 sq ft (5020 m^2).

This article describes the key features of the structure, especially concentrating on the conceptual design that led to a totally jointless integral abutment bridge.

Fig. 1. Bridge carrying Tennessee State Route 50 over Happy Hollow Creek.

The 46 ft (14 m) wide, 8¼ in. (210 mm) thick composite concrete slab conforms to a 4-degree 45-minute curve for approximately 976 ft (297 m) of its length while the remaining 199 ft (60.7 m) conforms to a spiral curve. A plan and elevation of the bridge is shown in Fig. 2.

Supporting the six-girder cross section, shown in Fig. 3a, are two-column bents varying in height from approximately 51 to 91 ft (15.5 to 27.7 m) as shown in Fig. 4. The superstructure is pin-connected to the bent caps which rest on plain 60 durometer neoprene pads. These bents vary in skew such that they are arranged to allow all beams in all spans, but one, to be of equal length. Because the beams are chorded, the slab overhang on each side of the bridge varies from 3½ to 5½ ft (1.07 to 1.68 m) along the span.

There are no expansion bearings under the bridge. Instead, thermal expansion and contraction for the 1175 ft 2½ in. (358 m) long curved stucture is accommodated solely through the deflection of its supporting piers and translation of the abutments.

To accomplish continuity, a common diaphragm, shown in Figs. 3b and 3c, joins both the ends of the girders in adjacent spans and the cast-in-place slab. Because the dead load slab deflections of the 72 in. (1829 mm) deep bulb-tees are relatively large and their depths significant, the Tennessee Department of Transportation requires one of several options to be used by the contractor.

In the first option, the contractor could choose to pour the entire deck in one operation, concurrently pouring the diaphragms. On the other hand, if the deck cannot be poured in one operation, then no construction joint may be located closer than 10 ft (3.05 m) nor further than 15 ft (4.57 m) from an interior support. Also, no diaphragm at an interior support may be poured unless the slab in the positive movement area of the adjacent spans has been poured. This prevents cracking of the common diaphragms at the supports.

To steady the girders in the interim, permanent intermediate diaphragms, composed of galvanized steel angles in an X-brace configuration, are placed at one-third points in the span and temporarily near the supports.

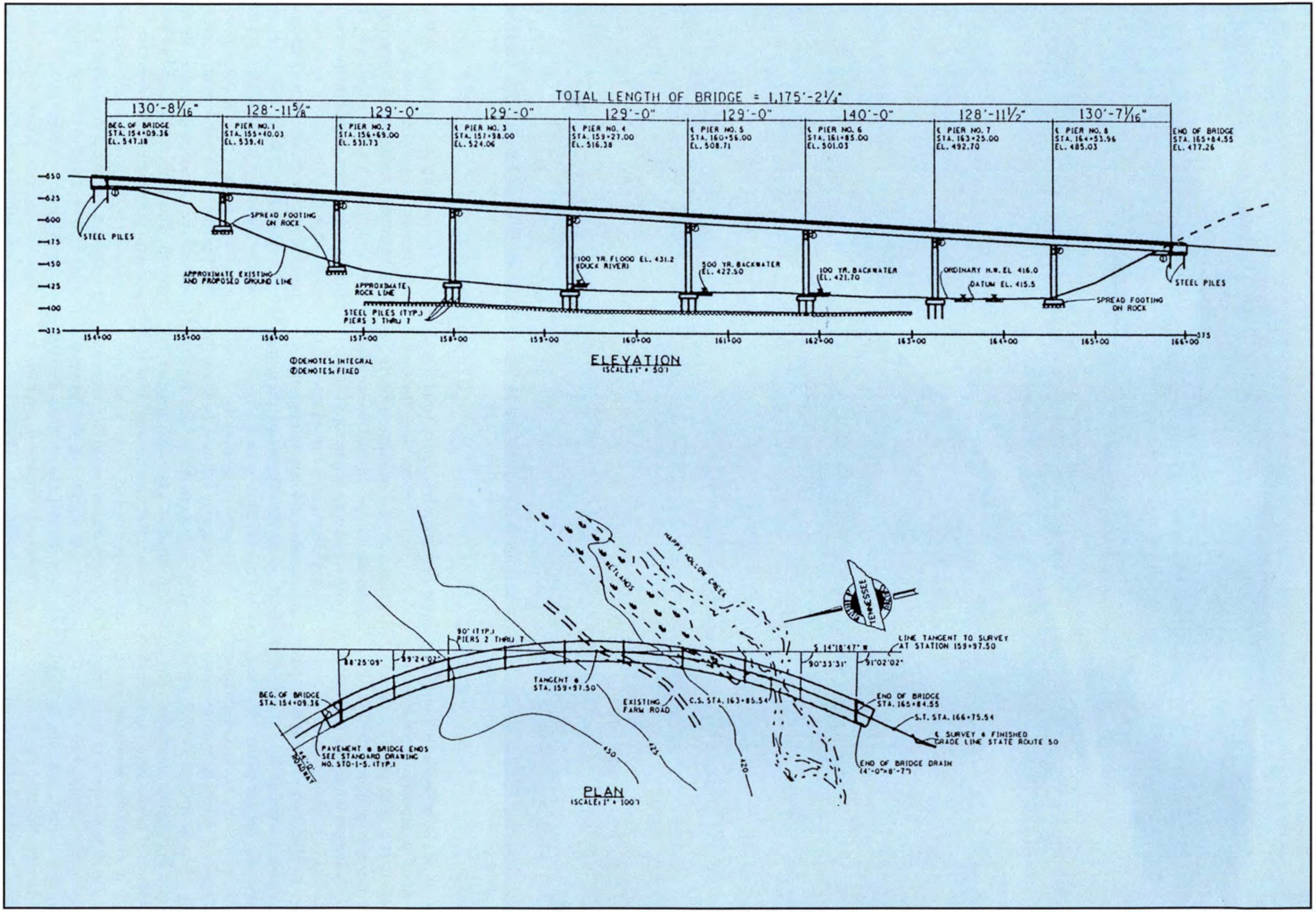


Fig. 2. Plan and elevation of bridge.

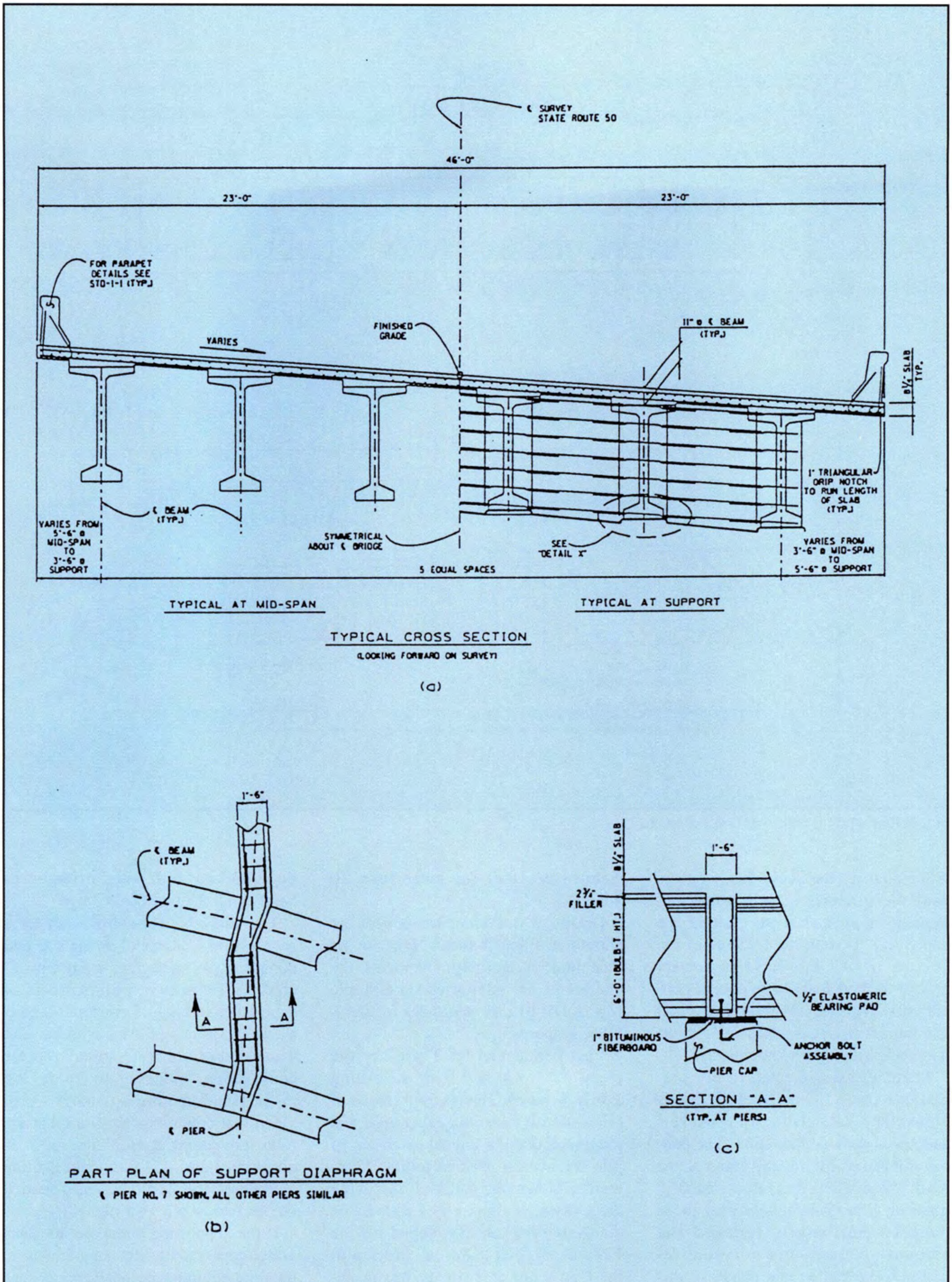


Fig. 3. Typical cross section of bridge at midspan and support together with partial plan and section of support diaphragm.

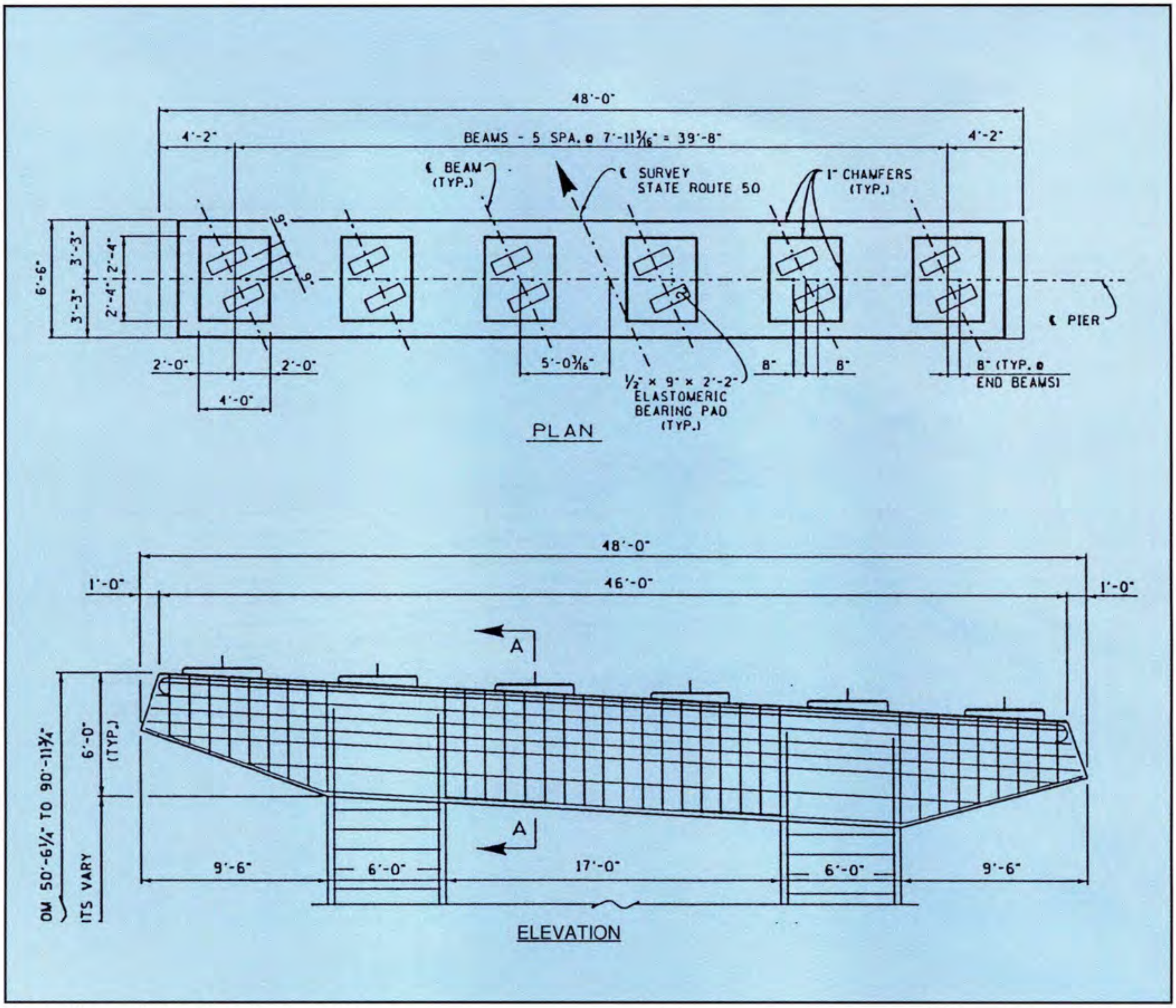


Fig. 4. Plan and elevation of column bents.

After pouring the deck, the X-braces near the supports may be removed. Pouring sequences require either the end 3 to 4 ft (0.91 to 1.22 m) of the slab, or all of the positive movement area of the end span, to be poured concurrently with the abutment back wall and wing walls, thus achieving a jointless deck with integral abutments.

The contractor elected to use precast, prestressed concrete deck panels to provide a safe platform for workers and speed deck construction. The panels comprised 3½ in. (89 mm) of the total 8¼ in. (210 mm) deck. Laid in nominal 8 ft (2.44 m) widths, these panels significantly reduced the amount of ready-mix concrete required to be hauled and placed at this remote site. Precast, prestressed panels have been successfully used by the Tennessee DOT for more than 20 years.

Details of the integral abutment are shown in Figs. 5 and 6. Due to the magnitude of thermal movements, the abutments are supported on a single row of HP 10 x 42 steel piles for flexibility purposes.

The Tennessee DOT prefers the piles to be oriented with the strong axis in bending. Tennessee's choice of orientation is based on experience and judgment, being a logical extension of pile orientation used in jointed abutments. However, calculations using the COM624P Laterally Loaded Pile Analysis Program, developed for the Federal Highway Administration at the University of Texas at Austin, indicates a slightly higher vertical load capacity in the deflected piles than can be achieved with piles oriented to bend about the weak axis.

The analysis procedures used for a jointless bridge depend on the size and complexity of the bridge being considered. In Tennessee, with 30 years of experience in integral construction, little analysis is performed on routine bridges. For concrete superstructure construction, bridges up to 800 ft (244 m) in length require no analysis provided that conditions are such that the total movement at an abutment does not exceed 2 in. (51 mm), and that the abutments are stub-type and founded on one row of piles for flexibility.

If the supporting bents are not integrally connected to the superstructure, then the columns are analyzed as cantilever beams to identify the force required to deflect the free end the re-

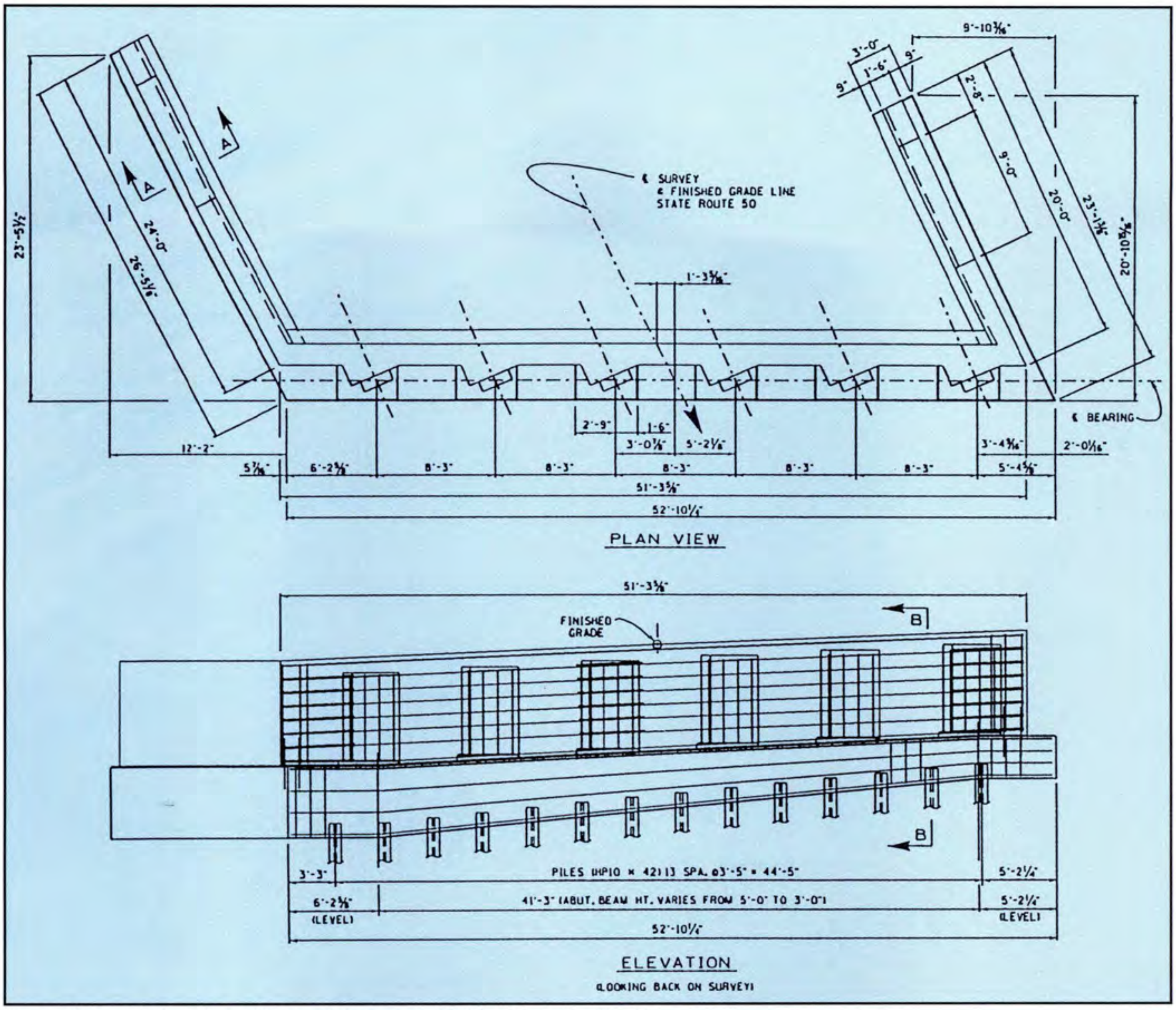


Fig. 5. Plan and elevation of integral abutment of bridge (see also Fig. 6).

quired distance in order to accommodate thermal effects. A free end condition at the top is assumed in the longitudinal direction. In calculating this moment, experience has shown that substituting the long-term modulus of elasticity of 1,000,000 psi (6895 MPa) gives satisfactory results and adequately models the actual cracked column behavior without the need for rigorous computations.

Where the resulting thermal moments combined with other appropriate longitudinal moments become too large to conveniently provide for reinforcement, expansion devices under the superstructure can be added to reduce the applied moments. However, in the latter case, the designer must verify that the force required to cause the expansion bearings to move does not exceed the force required to cause the bent to deflect. Should the force to move the bent be less, then the expansion bearing cannot function and alternate arrangements to accommodate thermal movements in the bridge must be made.

Because the State Route 50 Bridge over Happy Hollow Creek exceeded Tennessee's standards of practice, special considerations had to be made. First, it was desirable to eliminate expansion joints and expansion bearings, not only because of their high initial cost and future maintenance costs, but because of the skew of the substructures and curvature in alignment.

In general, it is difficult to predict the path of movement in curved structures. For example, it could be along the radial axis of the deck or the chord line of the end span girders or even along a chord struck from abutment to abutment. Another question to consider is what effects do column stiffness and skew have in influencing the path of expansion. Unfortunately, the wrong choice in orientation of the expansion joint and bearings can lead to their destruction or structural damage to the girders and abutments.

To arrive at a logical decision in not using joints at the abutments in the bridge, the designer can consider two options that appear to be the boundaries for the behavior of the bridge due to thermal effects, i.e., to consider the bridge to be either curved or straight.

First, the bridge could be considered to act as a curved bar, fixed at one end, i.e., an abutment. Fig. 7 represents this model. If the radius of cur-

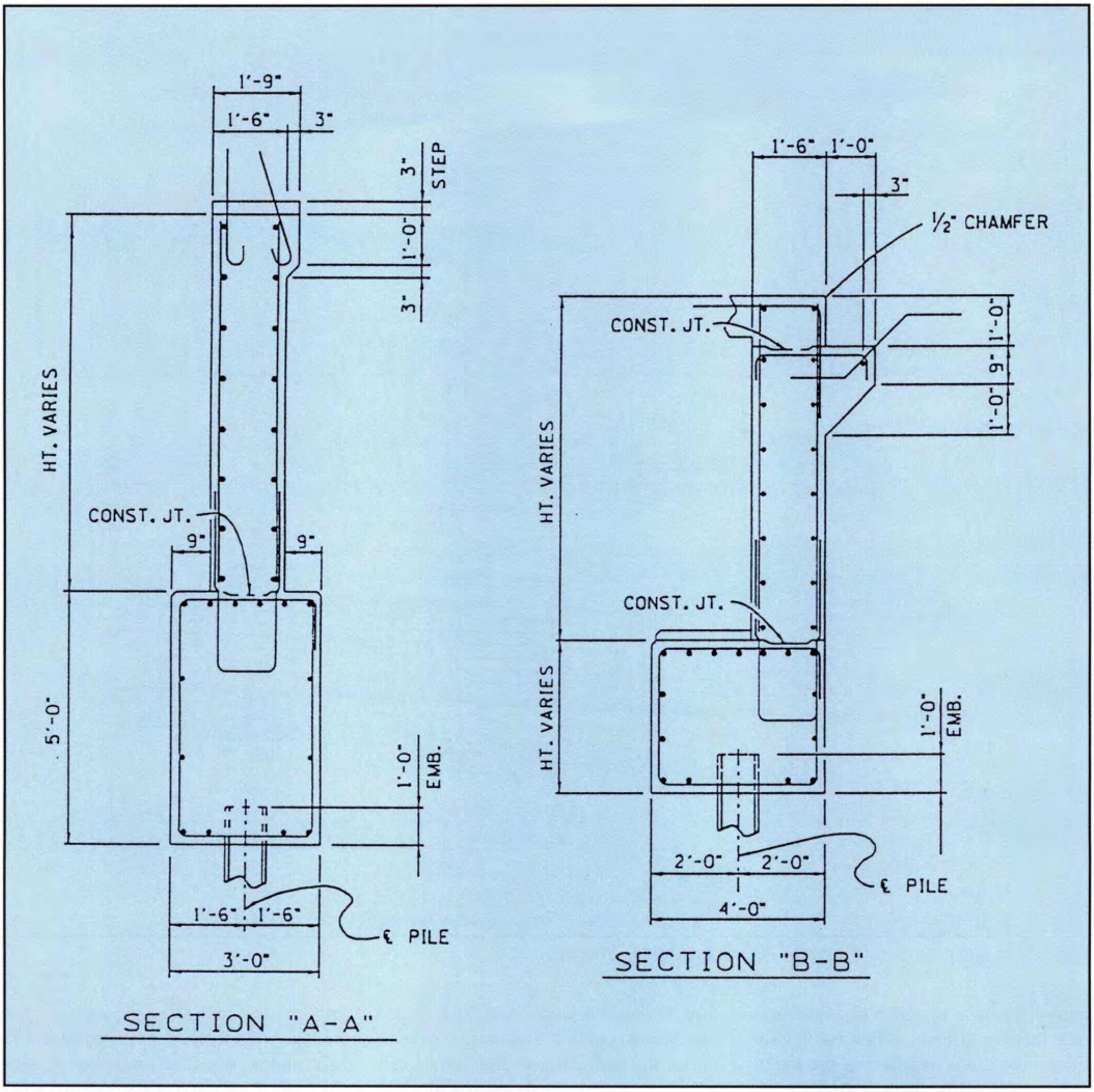


Fig. 6. Sections of integral abutment of bridge (see also Fig. 5).

vature of the bar is large compared to its cross-sectional dimensions, conventional beam deflection formulas may be used to calculate the lateral deflection of the curved bar under the influence of a concentrated load P, acting at the free end.

If the free end is considered to be at a distance equal to the bridge length and the lateral deflection is identified as equal to the total thermal expansive movement at the free end, then the concentrated load P causing the deflection can be quantified. This force can then be visualized as the reaction force needed to be exerted by the abutment to cause the bridge to bow outward should the abutment remain stationary.

Because the designer has no control over the temperature at which the completed structure will be made fully integral at the abutments, it can be assumed that the required lateral deflection must be equal to the maximum movement expected at the abutment. In this particular case, the movement is 2.97 in. (75 mm). From the geometry of the structure, the deflection, Δ, can be derived as follows:

$$\Delta = \frac{PR^3}{EI}(0.03918) \qquad (1)$$

where

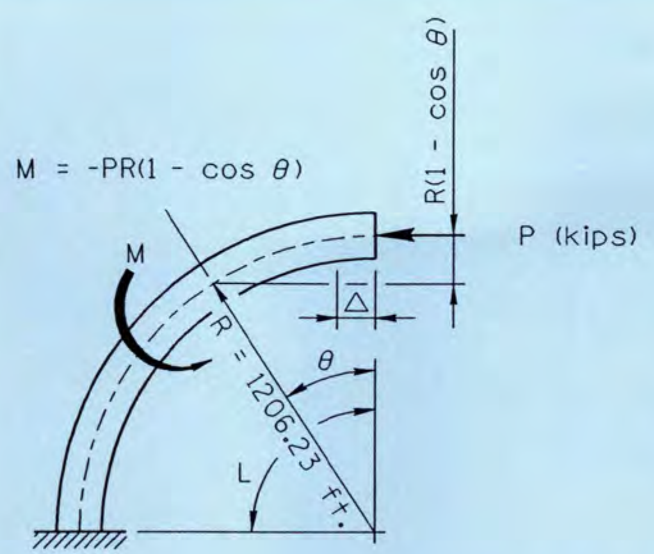
P = concentrated load at beam end or resisting force at abutment of bridge

R = radius of curvature of bridge

E = modulus of elasticity of concrete deck

I = transverse moment of inertia of bridge superstructure

Note that the factor "0.03918" is a product of integration. See the calculations in Fig. 7.



To compute the horizontal displacement of the curved bar, Δ, for a given applied force, P:

$$\Delta = \int_0^{L} \frac{-R(1-\cos\theta)\left[-PR(1-\cos\theta)R\right]d\theta}{EI}$$

where L is the angle of arc subtended, in radians

Symplifying:

$$\Delta = \frac{PR^3}{EI}\int_0^{L}(1-\cos\theta)^2\,d\theta$$

$$L = \frac{\text{Bridge Length}}{\text{Radius}} = \frac{1175.19}{1206.23} = 0.9742 \text{ radians}$$

$$E = 3{,}834{,}000 \text{ psi}$$

$$I = 115{,}600{,}000 \text{ in.}^4$$

$$\Delta = 0.03918\frac{PR^3}{EI}$$

$$P = \frac{EI\Delta}{0.03918R^3}$$

If $\Delta = 2.967$ in., the force required to cause or restrain this displacment is:

$$P = \frac{2.967(3{,}864{,}000)(115{,}600{,}000)}{0.03918(1206.23)^3(12)^3} = 11 \text{ kips}$$

Fig. 7. Model showing principle of curved bar fixed at abutment.

By re-arranging the terms in Eq. (1) and inserting the appropriate values for the symbols, the required resisting force P can be obtained:

$$P = \frac{\Delta EI}{(0.03918)R^3} = \frac{(2.97)(3{,}834{,}000)(115{,}600{,}000)}{(0.03918)(1206\times 12)^3} = 11 \text{ kips (49 kN)} \quad (2)$$

This force is much less than the force required to overcome the passive pressure behind the abutment, namely, 2606 kips (11590 kN). Therefore, the bridge will bow laterally under thermal expansion rather than mobilize the abutments.

This simplified solution ignores the stiffness of the bents and is far from being an exact analysis. However, the forces required to deflect the tall columns of this structure are relatively small and the large difference between the forces to deflect the structure vs. the force to move the abutment makes further refinement unnecessary.

The other boundary condition is to consider the bridge as being straight. In this case, each abutment would be required to move a total amount of 2.97 in. (75 mm). Recent tests conducted by the University of Tennessee for the DOT indicate that HP 10 x 42 piles with an embedment of 12 in. (305 mm) into the abutment beam, as shown in Fig. 5, can sustain this amount of movement repeatedly without detriment to the serviceability of either the piles or abutment beam.

With these boundary conditions identified, the Tennessee DOT decided to construct the State Route 50 bridge as jointless with integral abutments.

To account for the possibility of movement at the abutments, approach pavements equipped with silicone expansion joints adjacent to the asphalt pavement of the roadway were installed. The details of the approach pavements, standard for Tennessee's jointless bridges, are shown in Figs. 8 and 9.

A total of 6946 linear ft (2117 m) of 72 in. (1829 mm) bulb-tee girders were used. In addition, 25,000 sq ft (2325 m^2) of prestressed deck panels were used. The precast, prestressed concrete girders were manufactured by Standard Concrete Products, Inc. while the precast panels used in the project were fabricated by CPI Concrete Products, Inc.

The total cost of the project was about \$2.45 million with the precast concrete work amounting to about \$988,000.

The bridge carrying State Route 50 over Happy Hollow Creek was completed in October 1997 (see Figs. 10 and 11). During the last two years, the structure has been performing with total satisfaction. As a result of this design, the bridge is virtually maintainance free, containing neither expansion bearings or joints to maintain.

The experience in Tennessee has been that jointless integral abutment prestressed concrete bridges have been performing excellently for many years. Indeed, most bridges in Tennessee are now being constructed using jointless integral abutments.

The design solution used in this structure extends the present boundaries with regard to integral abutments/jointless bridge technology.

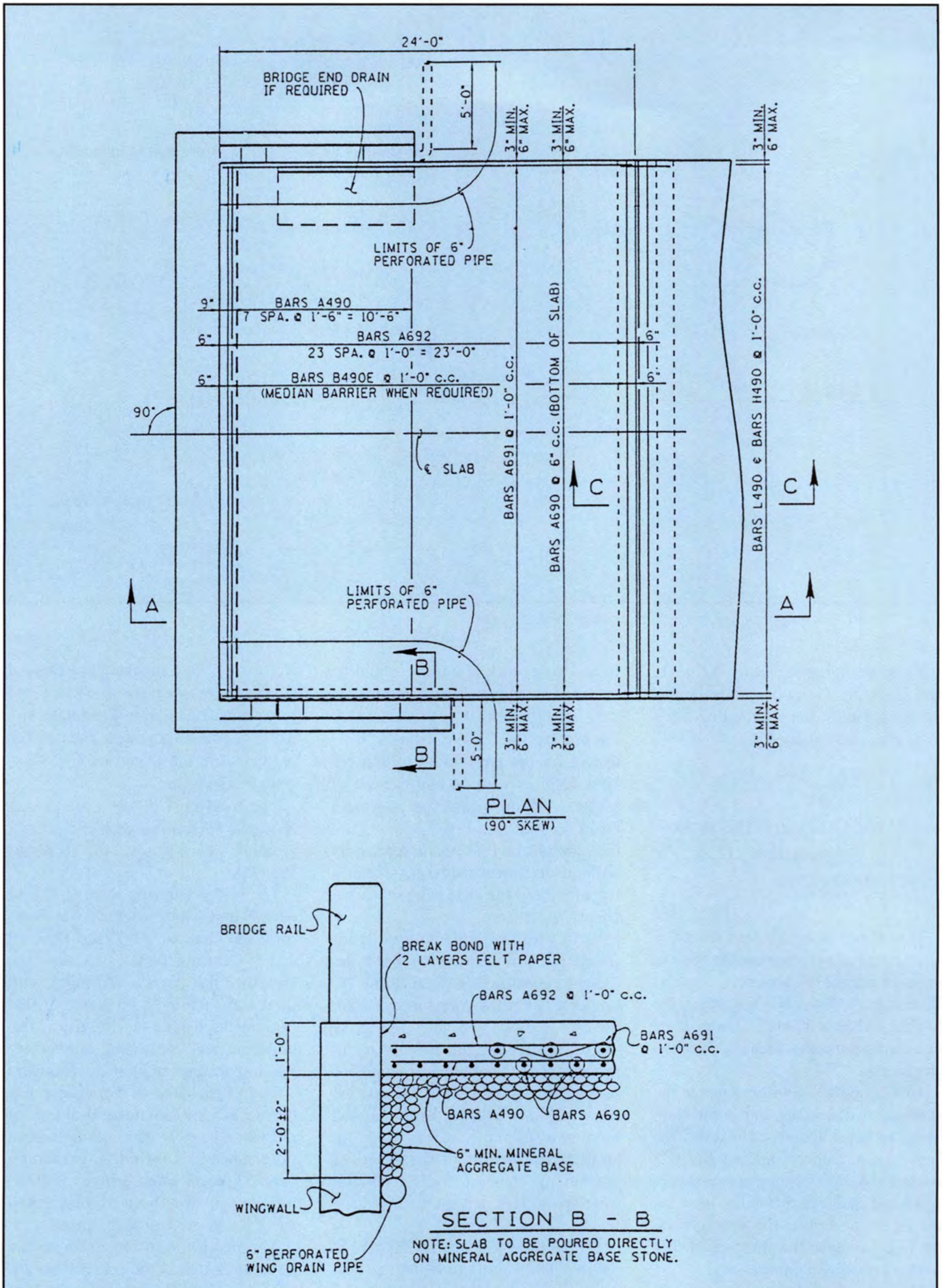


Fig. 8. Plan and section of approach roadway pavements (see also Fig. 9).

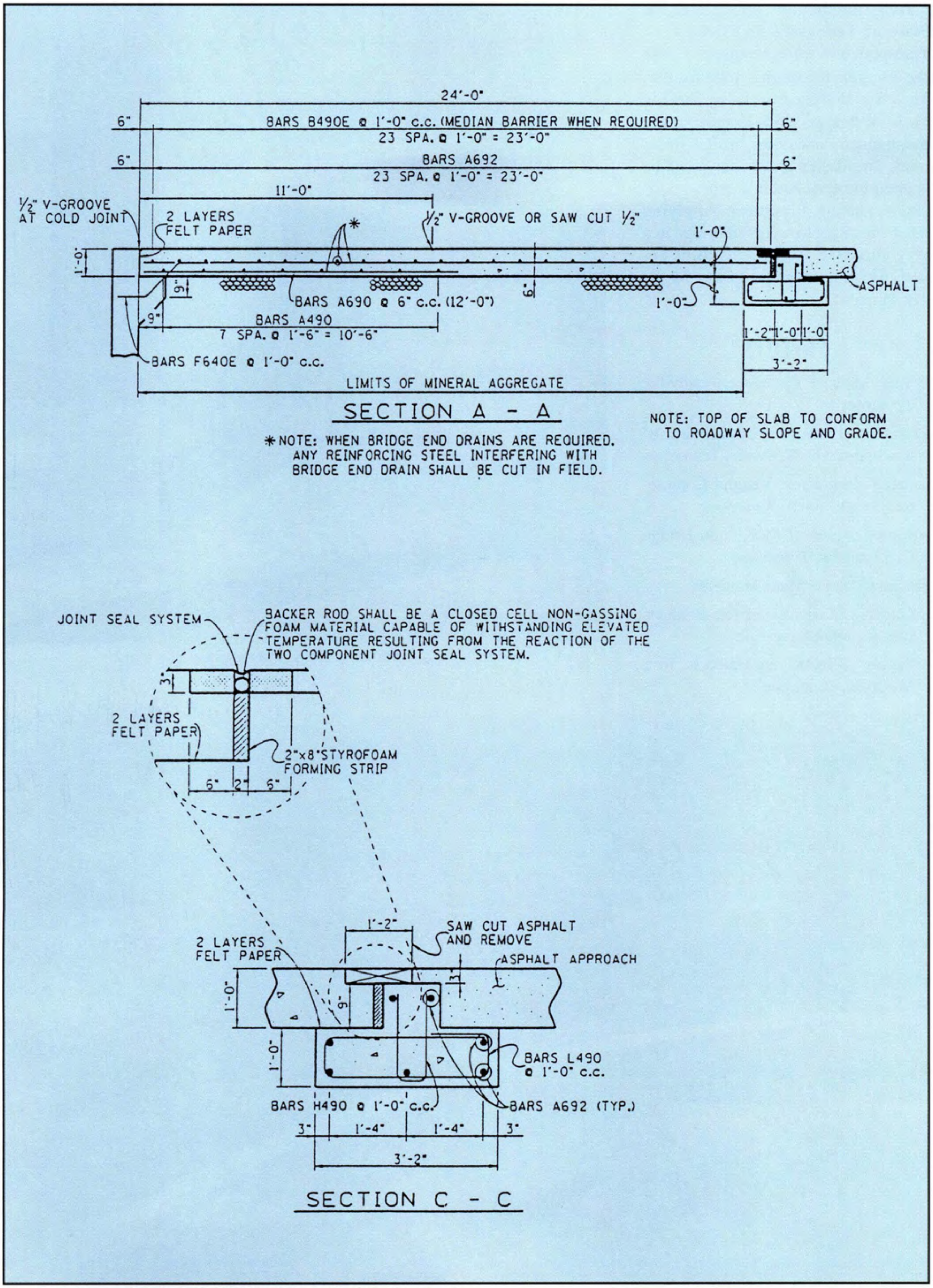


Fig. 9. Sections of approach roadway pavements (see also Fig. 8).

Jury Comments: "We applaud the State of Tennessee Department of Transportation for continuing to push the boundaries of this technology. They are sharply increasing the life cycle of bridges and decreasing the maintenance needs for bridge structures. This design philosophy could be applied to other building types, especially parking structures. Applying this technology to a curved bridge in a very attractive setting adds to its appeal, which is enhanced further by the slender piers."

CREDITS

Owner: State of Tennessee, Nashville, Tennessee

Engineer: Tennessee Department of Transportation, Nashville, Tennessee

General Contractor: Vaughn Contractors, Inc, Waverly, Tennessee

Bridge Contractor: McKinnon Bridge Co., Franklin, Tennessee

Precast Concrete Manufacturers:

- Girders: Standard Concrete Products Inc., Columbus, Georgia
- Panels: CPI Concrete Products, Inc., Memphis, Tennessee

Fig. 10. Underside view of completed bridge showing bent and piers.

Fig. 11. Aerial view of completed bridge.

San Mateo-Hayward Bridge San Francisco, California

An innovative precast concrete frame design was used in this bridge seismic retrofit project to transfer loads from the existing foundation to cast-in-steel-shell piles and decrease pier displacements during seismic activity.

The seismic retrofit of the 2-mile (3.5 km) long main span portion of the San Mateo-Hayward Bridge spanning San Francisco Bay made innovative use of precast concrete frames to encase the existing piers, allowing the transfer of a portion of the seismic loads to large steel piles situated outside the bridge outline. The precast frame system provides a foundation suitable for the local soil conditions that will decrease the piers' displacements during seismic activity.

The use of precast concrete frames for this project stemmed from the need to transfer forces from the existing foundation to large retrofit piles placed some distance from the existing pier. The system had to be constructed underwater in extremely soft and liquid local soils, making pumped caissons with cast-in-place seals prohibitively expensive. This is because the very deep seals would have to be removed in order to minimize mass during possible future seismic events.

Contract bidders were given the option of casting the frames in place but the precast alternate was ultimately chosen. Casting the frames away from the bridge site allowed speed of construction and less exposure to bad weather during vulnerable construction operations. Production of the components at the precasting plant also minimized on-site construction in the environmentally sensitive bay setting.

The precast concrete frames were designed to transfer the loads from the existing foundation to four large diameter [8 to 12 ft (2.4 to 3.7 m)] cast-in-steel-shell (CISS) piles that limit the drift and uplift of the existing foundation. The precast frames were engineered to take advantage of the existing foundation capacity. The frames do not restrict the foundation's vertical downward movement because it would induce large moments on them.

The encasement frames were placed around the existing piers so casting in two segments was required. The plan and elevation of a typical frame are shown in Fig. 1. The frames were designed with angular transfer beams at each corner of the footing. These beams limit the allowable uplift of each footing corner and, thus, minimize seismic rocking. The frame also has a pair of grout bags at each corner to transfer horizontal forces and movements to the large diameter steel piles that are provided to limit lateral movement and rocking at the pier.

Each precast unit consists of a rectangular frame with four outrigger beams connecting to the large diameter CISS piles. Longitudinally and transversely post-tensioned tendons ensure the ability of the assembly to transfer large forces through the different sections. The two long beams, transverse to the bridge deck, have an expanded polystyrene core to minimize the weight of the finished assembly. The two short beams, parallel to

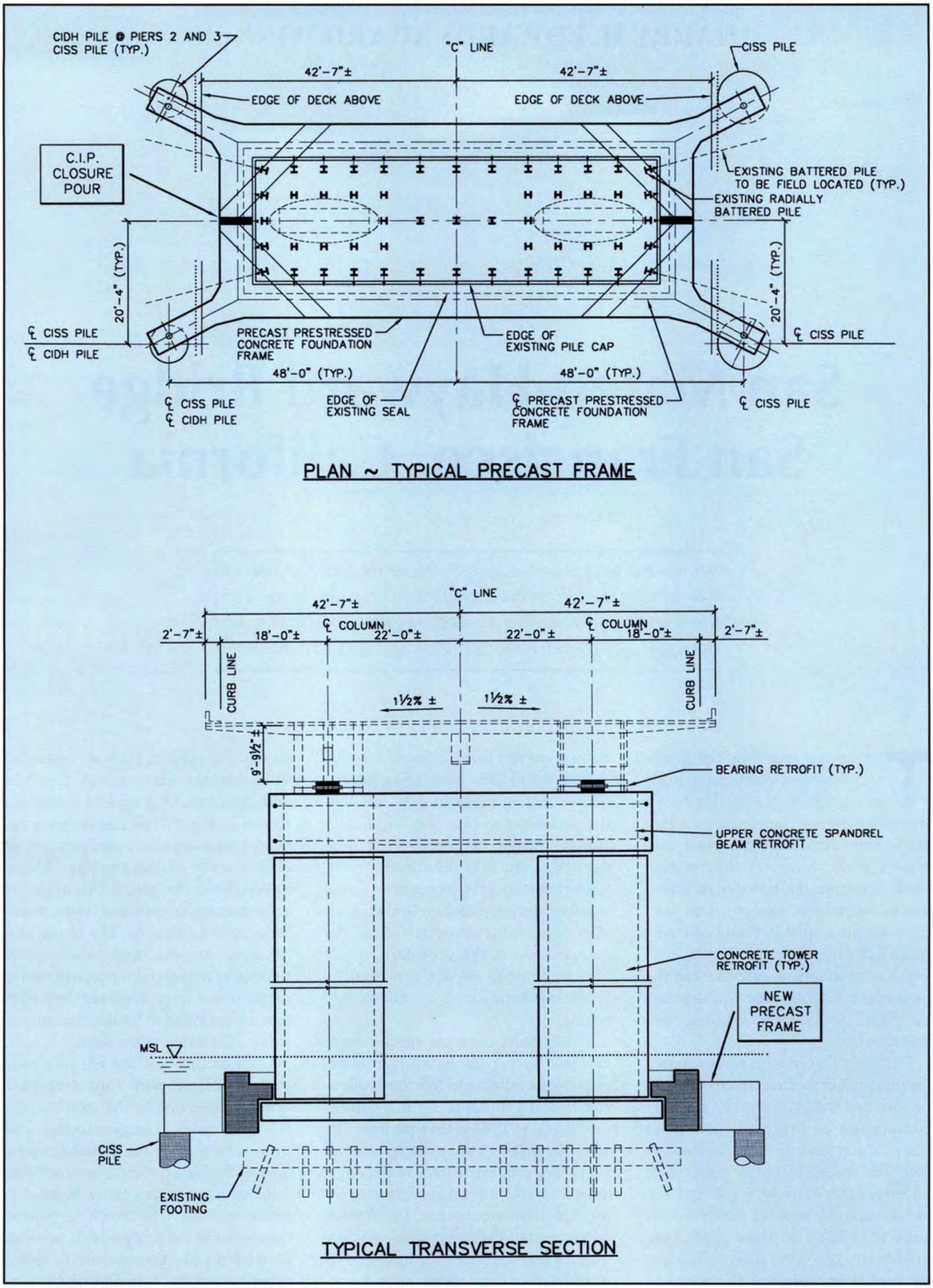


Fig. 1. Typical plan and elevation of precast frame unit.

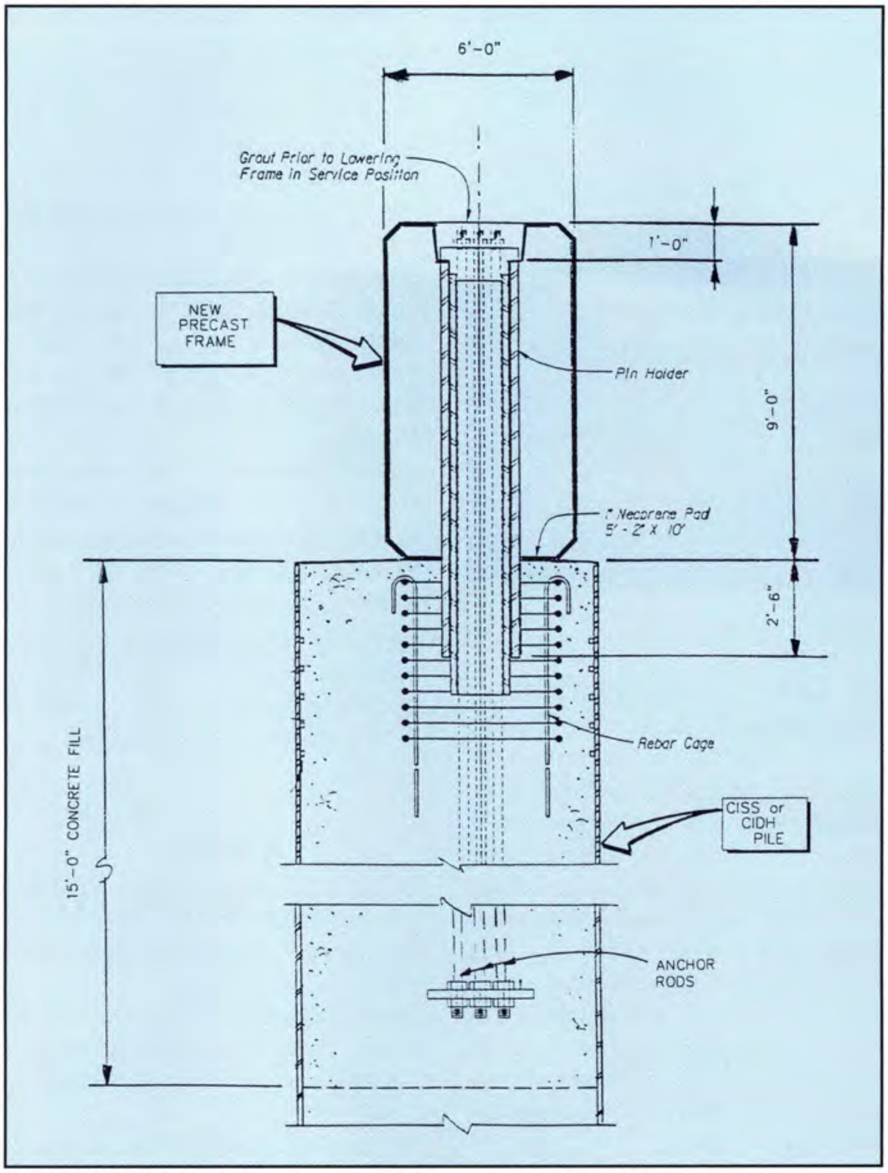


Fig. 2. Typical section showing precast frame-to-pile connection.

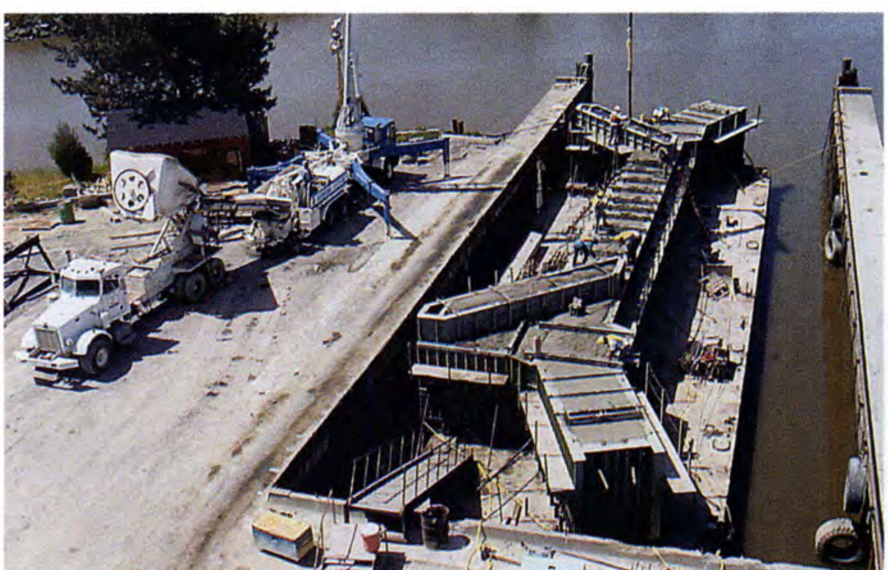


Fig. 3. Casting of segment on barge at precasting plant.

the bridge deck, are solid and contain a cast-in-place closure pour. This closure pour contains a series of grouted splice sleeves to connect longitudinal post-tensioning to transfer torsion through the cast-in-place closure pours.

The horizontal and vertical forces used in the design of the precast frames were derived from a bridge global model with approximately 3000 nodes. Seismic motions were developed using a seismic event magnitude of eight, from the adjacent San Andreas Fault, as required by the California Department of Transportation (Caltrans). The forces transmitted to each CISS pile, depending on the pier location, reach as much as 760 kips (3380 kN) in the longitudinal direction, 1850 kips (8229 kN) in the transverse direction, and 2000 kips (8896 kN) in the vertical uplift direction.

The horizontal translational drifts are transferred through neoprene bumper pads and grout filled bags at eight locations near the inside corners of the precast frames. The vertical uplift forces are transferred by means of a short transfer beam located at the inside corners of the precast frame. The short beam straddles over the existing foundation and has a neoprene pad to mitigate the impact when the two structures come into contact.

Forces are transferred from the rectangular frame to the outrigger beams and then to the CISS piles. Due to the geometry of the assembly, bending and torsional moments developed at the outrigger beams are transferred into the rectangular frame beams. The longitudinal and transverse design of the frames required consideration of superimposed load cases and reinforcement requirements addressing the need to provide for tension, bending and torsional effects.

The outrigger beams of the segment fit over the tops of the large steel piles and an 18 in. (457 mm) diameter steel pin containing multiple anchor rods is used to secure the two together. The piles were cleaned out to a depth of 15 ft (4.57 m) and tremied full of concrete after the pins were put in place. The remainder of the pile was left filled with local soil. The pins are detailed to develop their capacity in both the pile

Fig. 4. Completed segment on barge ready for shipping.

and the outrigger beam and provide both vertical and horizontal capacity. The pin details are shown in Fig. 2. The CISS piles support the frame.

The precast segments were manufactured by Pomeroy Corporation in Petaluma, California. The large size of the segments led to the strategy of casting them directly on barges at the precasting plant (see Fig. 3). The irregular plan of the segments required formwork allowing dimensional variation with the casting barges set up to accommodate this. The segments have unusually dense reinforcement consisting largely of #11 reinforcing bars. Many of the individual bars required four men to lift them into place.

For corrosion protection, all of the reinforcement is epoxy coated. The segments are compressed with 24 post-tensioning tendons in each segment. Each tendon contains 18 strands. The completed cage alone weighed 120,000 lbs (54432 kg). A travel lift gantry crane with a picking beam was used to transport the cage from its assembly jig to the casting barge. Steel forms were placed around the cage and approximately 230 cu yd (176 m^3) of concrete was pumped into the form over the course of 5 hours.

The segments were barged 50 miles (80 km) to the transfer site (see Fig. 4) where they were transferred in pairs to special erection barges. The erection barges were winched around a pier and then carefully positioned and anchored with spud piles. Jacks were used to slide the segments together around the pier on temporary Teflon sliding pads. The segments were then joined with a 2 ft (0.61 m) long closure pour using 52 grouted splice sleeves and final post-tensioning. A jacking frame, supported by the existing foundation, was used to lift the completed frame off the erection barge and then lower it into final position underwater (see Fig. 5). Inflatable grout bags at bearing pad locations allowed the required tolerances for setting the frame.

The San Mateo-Hayward Bridge rehabilitation project is scheduled for completion in January 2000. The use of precast concrete encasement frames provides a novel solution for seismically retrofitting the bridge that has not been used before in this part of the United States. The erection of the first few pier encasements has proven to be successful, providing unexpected efficiencies and promising a timely completion for the project. The total cost of the project was bid at $102 million with the precast production and shipping amounting to about $9 million.

Jury Comments: "This bridge uses an innovative approach to solve a complex problem. It is commendable because of its importance for underwater applications. Even with some very large precast elements, the formwork was kept fairly simple. The way the load is supported is quite unique. It is an extremely clever and elegant solution."

CREDITS

Owner: California Department of Transportation, Sacramento, California

Engineer: Carter & Burgess, Inc., Oakland, California

General Contractor: Morrison Knudsen/Traylor Brothers/Weeks Marine, a joint venture, Foster City, California

Precast Concrete Manufacturer: Pomeroy Corp., Petaluma, California

Fig. 5. Lowering completed frame after barge is removed.

Benaroya Hall Seattle, Washington

Precast concrete structural components and flawlessly cast architectural panels were used in a "box within a box" design to provide sound isolation and performance-quality acoustics for this concert hall situated in a noisy urban environment.

Urban planners chose a most unlikely location as home for the world-class Benaroya Hall concert facility in Seattle, Washington. Surrounded by noisy city streets, the full-block urban site is alive with unwanted noise and vibration. A "box within a box" concept was developed for the auditorium using precisely formed precast concrete components within a shell of cast-in-place concrete to isolate the facility from ambient noise and to tune the hall for maximum acoustical performance (see Fig. 1).

The City of Seattle selected the location for the 183,000 sq ft (17000 m^2) Benaroya Hall with the intent of revitalizing a neglected downtown area. Unfortunately, a busy underground railroad tunnel runs diagonally under the steeply sloping site and a public transit bus station lies 60 ft (18.3 m) deep under the entire width of an adjacent city street, producing an abundance of ambient noise both within and outside of the normal hearing range.

Most concert halls are designed simply for isolation from audible sounds. The standards for the Benaroya Hall, which comprises a 2500-seat concrete hall and a 540-seat recital hall, required isolation from audible sounds and from sounds outside the normal hearing range, such as the low frequencies generated by the diesel train engines running beneath the site. The inherent mass of concrete could be used effectively to isolate the hall from outside noise, but would be difficult to cast with the precision required to accentuate the acoustic properties of the auditorium.

A massive concrete foundation mat, 6½ ft (2 m) thick and 60 ft (18 m) across, was cast diagonally from one end of the site to the other, serving to dampen and reflect noise and vibrations from frequent train traffic. Each side of this foundation mat is supported on 6 ft (1.8 m) diameter reinforced concrete piers, drilled approximately 100 ft (30 m) deep to bearing stratum.

With foundation isolation provided by the concrete mat, a structural frame was needed to provide interior sound isolation and acoustic tuning. Through creative partnering of the structural engineer, construction manager, architect and precast concrete manufacturer, a low cost yet acoustically superb structural solution was developed using meticulously fabricated precast concrete components within a cast-in-place shell to provide both the needed isolation and the fine degree of acoustical response sought by the owners and acoustical consultant. In all, 560 structural precast concrete components, including walls panels, hollow-core slabs and balcony components, were used for the hall.

Inside the hall, the acoustical consultant dictated a complex pattern of wall surfaces, necessary to scatter sound throughout the hall and create the sensation of sitting close and far away at the same time. Given the unusual shapes of the interior wall surfaces, an economical, repetitive form-

Fig. 1. Panoramic view of Benaroya Hall in Seattle, Washington.

work geometry was needed to allow the desired acoustical surfaces to be cast in concrete.

To achieve the desired surfaces, the art of tessellation — creating irregular surfaces with repetitive interlocking shapes — was put into practice. Through an intensive study of possible formwork surfaces, a uniquely irregular (yet repetitive) acoustical wall surface with 40 different panel configurations was developed that could be cast using just two sets of forms. By blocking out the formwork in different ways for each panel, no two of the cast panels are identical (see Fig. 2).

The precast concrete wall panels provided flexibility, inherent mass and strength in a seamless, precisely formed wall surface that is necessary for maximum acoustical performance. The panels also prevent air, which carries unwanted sound waves, from penetrating the wall surfaces into the hall — a crucial design constraint required by the acoustical consultant.

The geometric interior concrete walls provide a massive acoustic substrate for the wood paneling inside the auditorium. Because no shims could be used on the stripping behind the paneling due to acoustical constraints, the precast walls were fabricated with virtually no tolerances.

A precast concrete system was also chosen to replace the cast-in-place concrete balcony tier after the construction schedule was shortened by eight months to accommodate an opening performance date. Balcony seating at the rear of the hall uses precast concrete riser units supported on precast raker beams adapted from modern sports arena construction.

Tessellation studies were again employed to efficiently design the framing for the cantilevered side tier balconies. Precast concrete beam and slab units were integrated into repeating three-dimensional patterns. Stepping down towards the stage, each seating section contains one precast beam unit and one slab unit.

Cast-in-place concrete columns and slabs were used sparingly. They tie the precast concrete walls to the precast beams and slabs. The balcony and side tier walls and framing — precast concrete joined by cast-in-place joints — create a massive barrier to unwanted noise intrusion. The seamless concrete surfaces form a solid base from which the acoustical finishes can reverberate and reflect the music.

The auditorium space is nested within an outer box of cast-in-place concrete comprising the surrounding structure. There are no connections between the inner and outer boxes except at the auditorium base, where special rubber isolation bearings take gravity loads and additional prestressed rubber bearings lock the hall's position to resist seismic loads. A 7 to 9 in. (178 to 229 mm) joint surrounds the hall's perimeter, isolating the inner box.

The exterior of the structure is clad

with 150 architectural precast concrete panels. Not only did the panels provide an attractive, flawless finish for the building but they also furnish additional mass to block extraneous sounds.

The architectural precast concrete panels were manufactured by P. Kruger Concrete Products Ltd. in Edmonton, Alberta, Canada, while the structural precast concrete panels were manufactured by Con-Force Structures Ltd. in Vancouver, British Columbia, Canada. The precast products were shipped to the project site by truck-trailer.

The total cost of this project was about $118 million with the precast concrete work amounting to about $2 million.

Completed in September 1998, the Benaroya Hall has exceeded the designer's expectations in acoustic performance (see Figs. 3 and 4). Only 0.3 percent of sound energy enters the auditorium from outside the hall. Many accolades have been received, including the praise of Seattle Symphony Conductor Gerard Schwartz, who said "This is the greatest hall I've ever conducted in, and I've been in some great halls."

Jury Comments: "Sitting over a train station is a difficult building problem, particularly for an auditorium. This solution represents an interesting use of precast components as acoustical members to provide an inner acoustical framework that is soundproofed from the substantial outside noise. The design is elegant and well placed."

CREDITS

Owner: City of Seattle, Seattle, Washington

Engineer: Skilling Ward Magnusson Barkshire, Inc., Seattle, Washington

General Contractor: Baugh Construction Co., Seattle, Washington

Precast Concrete Manufacturers:

- Structural Precast Concrete: Con-Force Structures Ltd, Vancouver, British Columbia, Canada
- Architectural Precast Concrete: P. Kruger Concrete Products Ltd., Edmonton, Alberta, Canada

Fig. 2. Interior view showing tessellated precast panels.

Fig. 3. Concert hall during a performance with a capacity crowd in attendance.

Fig. 4. Concert hall lobby during a performance intermission.

Arched Precast Wall System University of Minnesota Minneapolis, Minnesota

Curved precast concrete wall panels provide the interior surface for a sandstone cave surrounding a new underground library where limited construction space and environmental conditions made other construction alternatives impossible.

Faced with the monumental task of constructing 90,000 sq ft (8361 m^2) of one-sided, curved cast-in-place concrete walls for an underground library at the University of Minnesota, engineers devised an innovative precast concrete design alternate that simplified the construction process, saved time, reduced costs and provided superior quality. At the heart of the system are curved precast concrete panels ranging from 15 to 25 ft (4.6 to 7.6 m) in height.

Construction of the $35 million library involved mining caverns and tunnels with a total footprint of 106,000 sq ft (9847 m^2) and an excavated volume of 100,000 cu yd (76460 m^3). Each of the two caverns is 600 ft long, 25 ft high and 70 ft wide (183, 7.62 and 21.3 m) and was constructed in easily excavated St. Peter sandstone with an overhead limestone ceiling (see Fig. 1).

Geologic conditions required that the concrete wall construction closely follow the mined excavation of the cavern face in order to provide the required support to the sandstone at the perimeter of the excavation. Casting the walls in place would have required the installation and subsequent removal of heavy steel forms needed to withstand the very high concrete pressures. The low overhead clearance would complicate handling the formwork, reinforcing steel and concrete placement.

Due to the nature of the sandstone, form ties could not be feasibly secured and horizontal shoring to resist concrete pressures would project out into the cavern, obstructing cavern excavation and rock bolting operations. The cool cavern temperatures would retard concrete placement rates and formwork cycle time.

Considering the serious challenges to be overcome with a cast-in-place system, the general contractor's consultant and the owner's design engineer worked together to devise an alternate method of construction. Merging modern construction with mechanical, hydraulic and structural engineering concepts, the team and their associates devised a way to line the caverns with precast concrete panels. The 400 panels were 10 ft (3.0 m) wide and 8 in. (203 mm) thick with heights ranging from 15 to 25 ft (4.6 to 7.6 m). Each panel weighed up to 25,000 lbs (11340 kg). In all, 90,000 sq ft (8361 m^2) of precast panel components with concave radiuses of 30 ft (9.1 m) were fabricated for the project.

A special panel handling machine was devised by modifying a Caterpillar backhoe to move the panels through the 15 ft (4.6 m) cavern entrance and then tilt the panels up into place within 2 in. (51 mm) of vertical clearance (see Fig. 2). The ability to set fully cured panels near the cavern face allowed faster excavation. A three-man crew was able to transport and set these huge wall panels, within tolerances of ⅛ in. (3 mm), in about 30 minutes.

After the wall panels were set in place, a 4 in. (102 mm) thick, high slump pea grout was placed behind

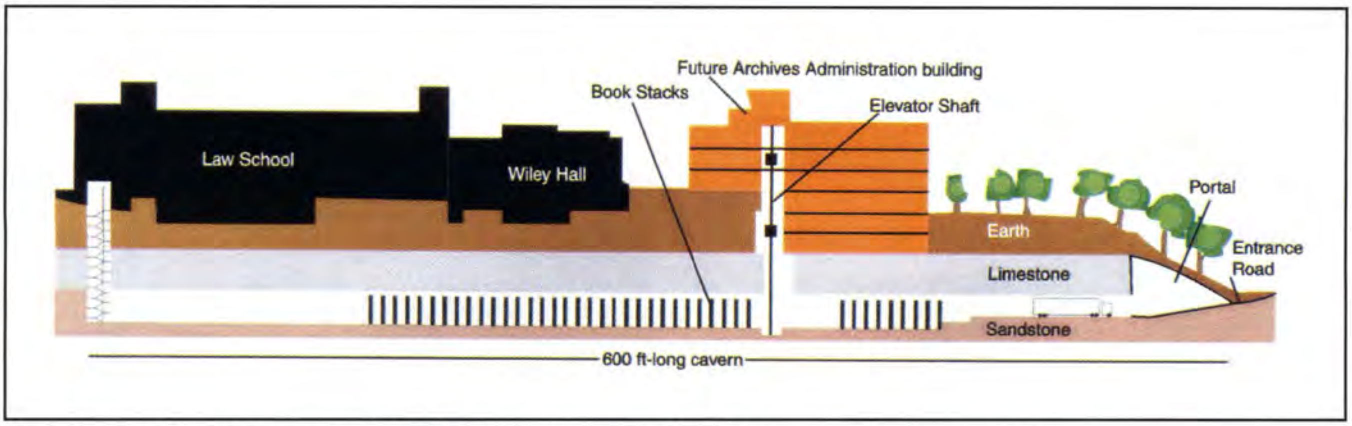


Fig. 1. Section view of library access center showing cavern, existing buildings and new construction.

them to provide full contact support to the sandstone supporting the overhead limestone layer and the existing law school buildings. Engineering calculations showed that the hydrostatic pressure from 25 vertical ft (7.6 m) of grout would exceed 430,000 lbs (1912 kN) of horizontal thrust per panel. To withstand this tremendous pressure, the top and bottom edges of the curved panels were dry packed in place, forcing the concave panels to function as arches (see Fig. 3).

To ensure a void-free filling of the space behind each panel, the grout was pumped up from the base. To counteract the bending stresses from the unbalanced pressures during the grouting operation, a very simple but effective detail was used: two coil bolt inserts were cast into each panel near the upper one-quarter point. After panel erection, two 1 in. (25.4 mm) round coil bolts were screwed through the panel inserts until contact with the excavated sandstone wall was achieved.

The end result was that these coil bolts, or deflection blocker bolts, prevented the panels from deflecting due to grout-induced bending stresses, maintaining the panel in its true arch form. These arches were capable of withstanding the pressures from rapid filling of a 25 ft (7.6 m) head of grout without any added ties, shores, or bracing (see Fig. 4). Following the grouting operation, the coil bolts were removed and reused.

The contractor was able to grout up to five panels [50 ft (15.2 m) total width] to their full 25 ft (7.6 m) height with a single hose connection in one continuous operation. The resultant forces served to stress the panels and to impart a permanent pressure for long-term stability of the sandstone. The pressure grouted arched precast panels resulted in a higher quality structure, speeded the excavation process, and reduced the on-site crew size by thousands of man hours compared to that required for cast-in-place walls. They also provided a higher quality wall system, with all panels manufactured under controlled factory conditions.

Fig. 2. Precast panel positioned on handling machine (top) and being tilted into position (bottom).

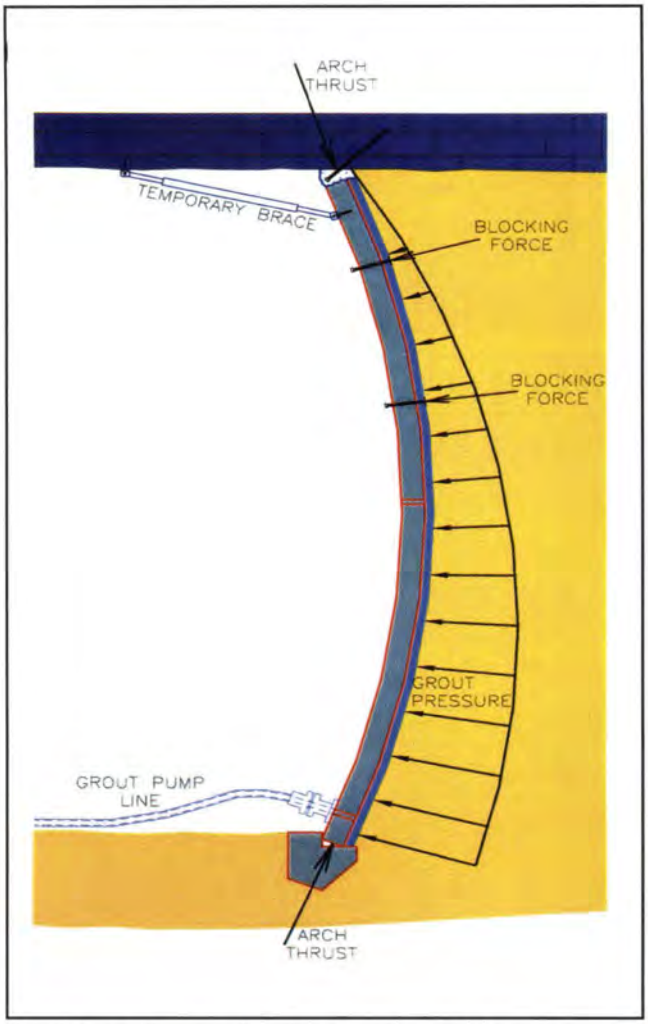


Fig. 3. Force diagram showing curved panel interaction.

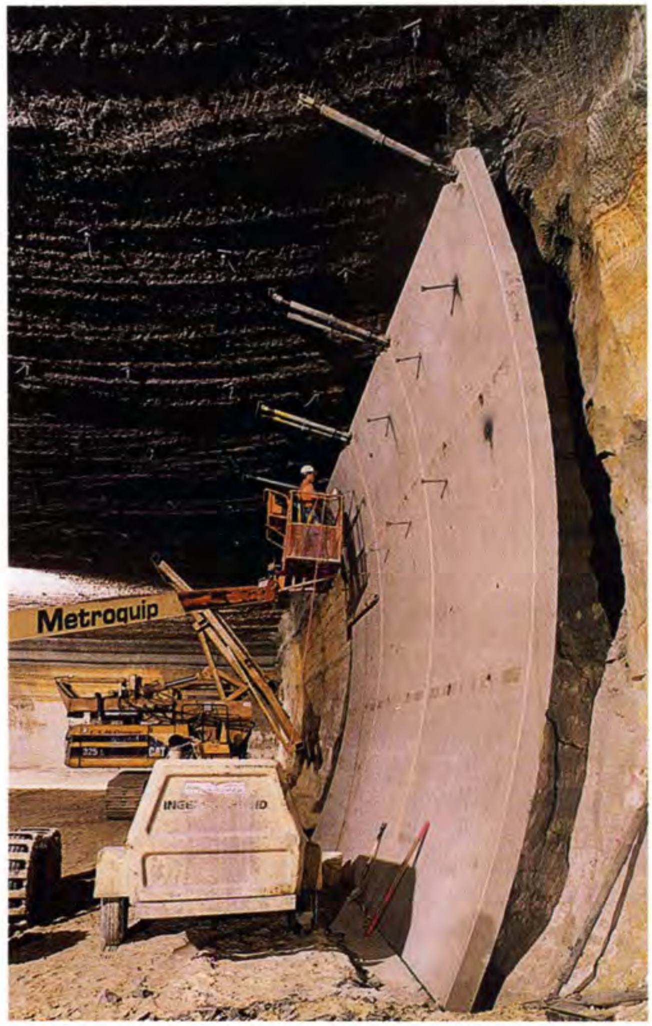


Fig. 4. Precast panels set in place and prepared for grouting.

The construction innovations developed for this project simplified the building process, saved time, reduced costs and provided higher overall quality. These wall construction concepts promise to have widespread applicability in future mined excavations for developing underground real estate that could be used for multiple purposes.

The precast concrete panels were manufactured by County Prestress Inc. at their plant in Maple Grove, Minnesota. The panels were shipped by truck trailer the short 12 miles (20 km) to the project site. The precast concrete contract was just under $1 million.

Jury Comments: "This project shows a wonderful engineering solution to a problem of spatialization. The method for bringing in the panels and making them work as walls to create this inner space is remarkable. Using the mass of precast elements to create an environment in which they could build another structure for storage was very innovative."

CREDITS

Owner: University of Minnesota, Minneapolis, Minnesota

Engineer: CNA Consulting Engineers/Middleton Engineering, a joint venture, Minneapolis, Minnesota

General Contractor: C.S. McCrossan Inc., Maple Grove, Minnesota

Precast Concrete Manufacturers:

- County Prestress Corp., Maple Grove, Minnesota
- Hanson Spancrete Midwest, Inc., Maple Grove, Minnesota