PRECAST CONCRETE PIER CAP SHELLS FOR THE NEW NEW YORK (TAPPAN ZEE) BRIDGE - DESIGN FOR 100 YEAR SERVICE LIFE

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ABSTRACT

With only 62 months to build the substructure for the 3.1 mile long New NY (Tappan Zee) Bridge, precast substructure elements were widely used to speed construction, improve safety, and provide a durable final product meeting 100 year service life requirements. Precast concrete pier cap shells – 92' long, 10.5' wide, 13' deep and weighing 280 tons each – are used for 59 approach piers. These tub-shaped precast elements minimize over-water formwork and provide a safe work space for tying of reinforcement. The design features of the precast pier cap shells are chosen to minimize pick weights, standardize details, and to provide fully-composite behavior with the infill concrete.

In order to ensure superior performance for the 100 year service life of the bridge, a detailed time-dependent analysis of the staged construction sequence was performed to evaluate the effect of differential creep & shrinkage and locked in stresses between the precast shell and the cast-in-place infill concrete. Prestressing is provided in the precast pier cap shells to eliminate concrete tension during the second stage cast-in-place concrete infill placement, and to minimize cracking due to long-term differential shrinkage.

Keywords: Accelerated Construction, Concrete, Connections, Creative/Innovative Solutions, Design-Build, Substructure

INTRODUCTION

On a bridge project as large as the Tappan Zee Bridge replacement, there are significant opportunities for saving time and cost through innovative precast concrete elements. The repetitive nature of the approach substructure bents for the New NY Bridge, which include nearly 80 bridge piers, were ideally suited for unique custom-designed precast elements[1]. Precast pier cap shells were developed for most of these piers (59 total, see Fig. 1) to act as forms during construction, becoming composite substructure elements when additional reinforcement and a concrete infill are placed.

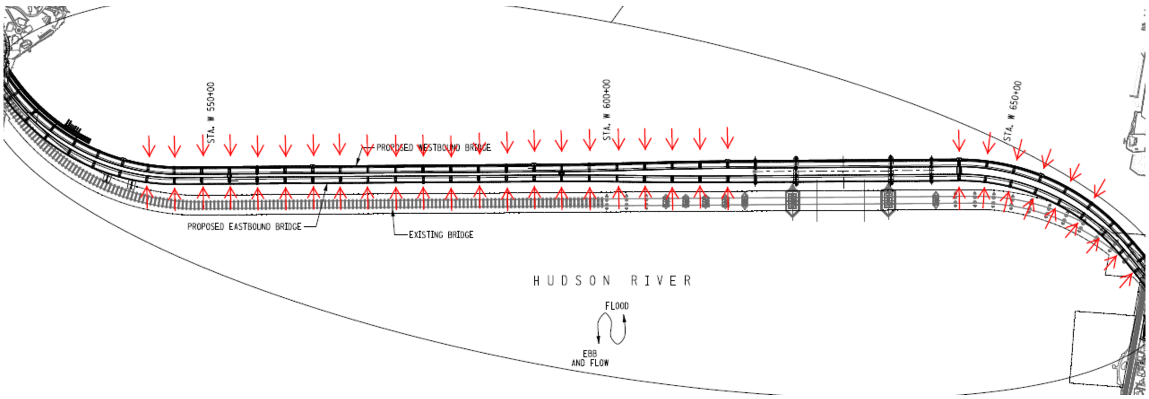


Fig. 1 – Approach Structure Layout (arrows indicate precast pier cap locations)

PRECAST PIER CAP SHELLS

The approach structures consist of steel girders supported on concrete pier caps (see Fig. 2), which are located up to 100 feet over the water. Precast pier cap shells were utilized to take advantage of the design-build team's floating equipment, to minimize formwork over the water, and to speed construction. The precast shells weigh between 235 tons and 280 tons and are 13 feet deep, 10.5 feet wide, and up to 92 feet long. The shells, shown in Fig. 3, are designed to be erected on top of the cast-in-place concrete columns and filled with concrete to complete the cap.

The precast pier cap shells are prestressed to minimize tension and long-term cracking, and are detailed to act compositely with the cast-in-place infill concrete. The 6000-psi concrete shells use prestressing (0.6-inch diameter strands) in the walls and in the soffit slab – up to 18 strands in each wall, and 10 strands in the soffit slab – in order to provide a zero-tension condition when the infill concrete hardens, so that the differential stresses between the precast shell and the 5000-psi infill concrete are minimized. The shell is made composite with the infill using a roughened inside surface and mechanical couplers for shear and torsion reinforcing steel.

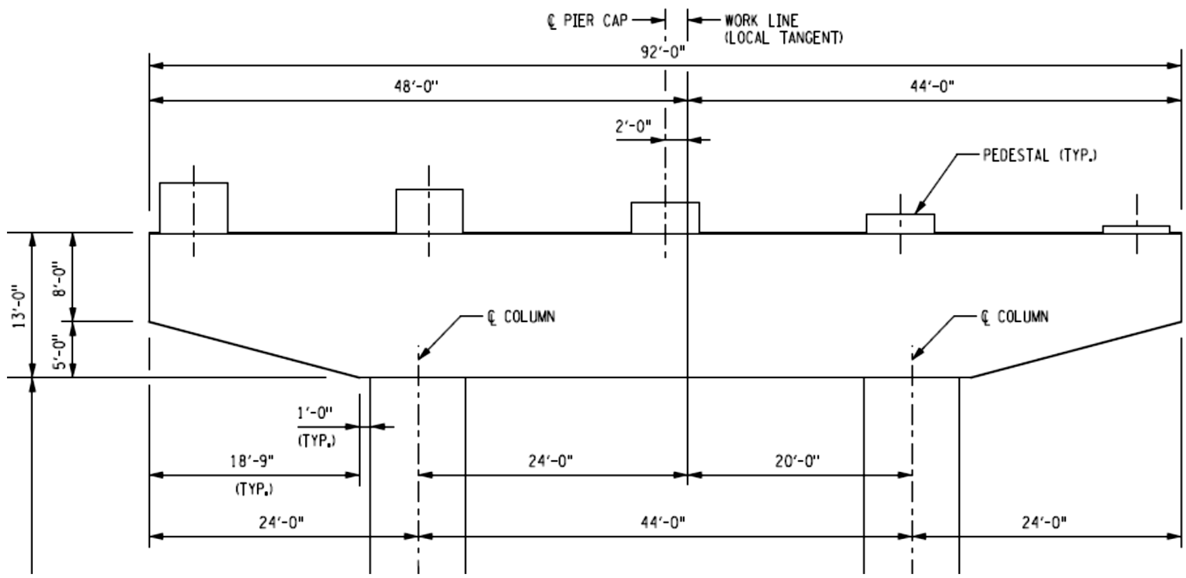


Fig. 2 – Approach Structure Pier Cap

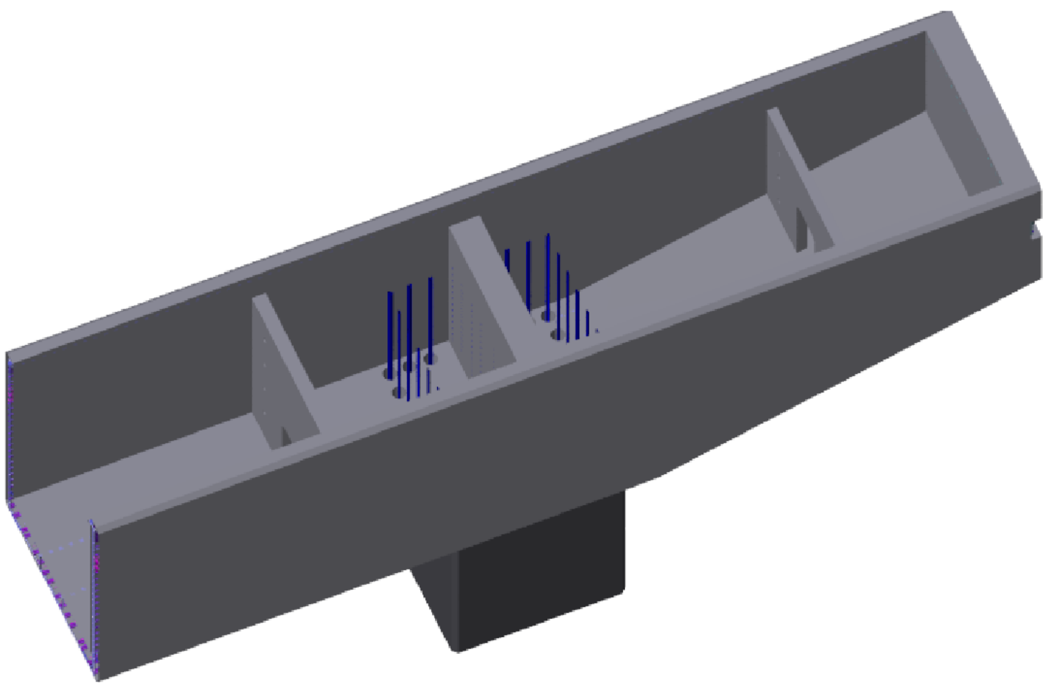


Fig. 3 – Precast Pier Cap Shell

Construction sequence for the pier caps is as follows:

1. Precast shell is cast in the precast yard with internal diaphragms for handling stresses, and prestressing force is transferred to the shell.

2. Column is constructed to top of column level, with column reinforcement bundled to maximize bar spacing. Bundling of the column bars minimizes the number of ducts required to be cast into the bottom slab of the shell, and increases the placement tolerances; thus simplifying the connection with the precast shell.

3. Precast shell, with ducts cast in the soffit slab to accept column reinforcing, is erected on top of columns supported by shims under diaphragms located at the center of each column. See Fig. 4.

4. Infill reinforcing may be placed inside the shell prior to picking, or after erection on top of the columns.

5. The column/cap joint is grouted, including the ducts around the column reinforcing.

6. Once column/cap grout reaches strength, the infill concrete is placed, at a rate not exceeding 2-ft vertically per hour, to the top of the pier cap walls.

The goal for production design was to have the primary pier cap reinforcement (negative moment and shear) designed using an analysis based on monolithic behavior of the section, and thus detailed staged time-dependent and sectional analyses have been performed to verify the composite behavior of the pier cap and these design assumptions.

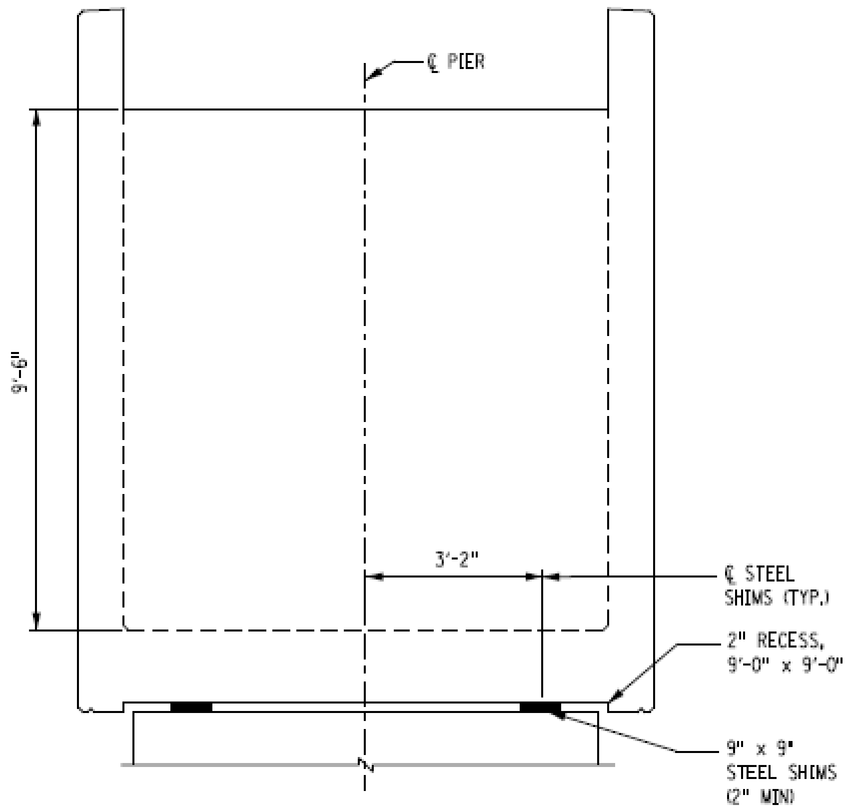


Fig. 4 – Shell Supported on Shims, Top of Column

PRECAST PIER CAP ANALYSIS & DESIGN

The design of the 10-inch thick shell walls considers the load effects of the construction sequence. The walls are checked for concrete pour pressure and handling stresses; form pressure from the 11.5-ft infill pour is significant, and the internal diaphragms are used to reduce bending moment in the walls (see Fig. 5). Handling stresses are checked assuming 20% impact during lifting; because the lifting points coincide with the precast shell support points, the lifting stresses do not control the shell design.

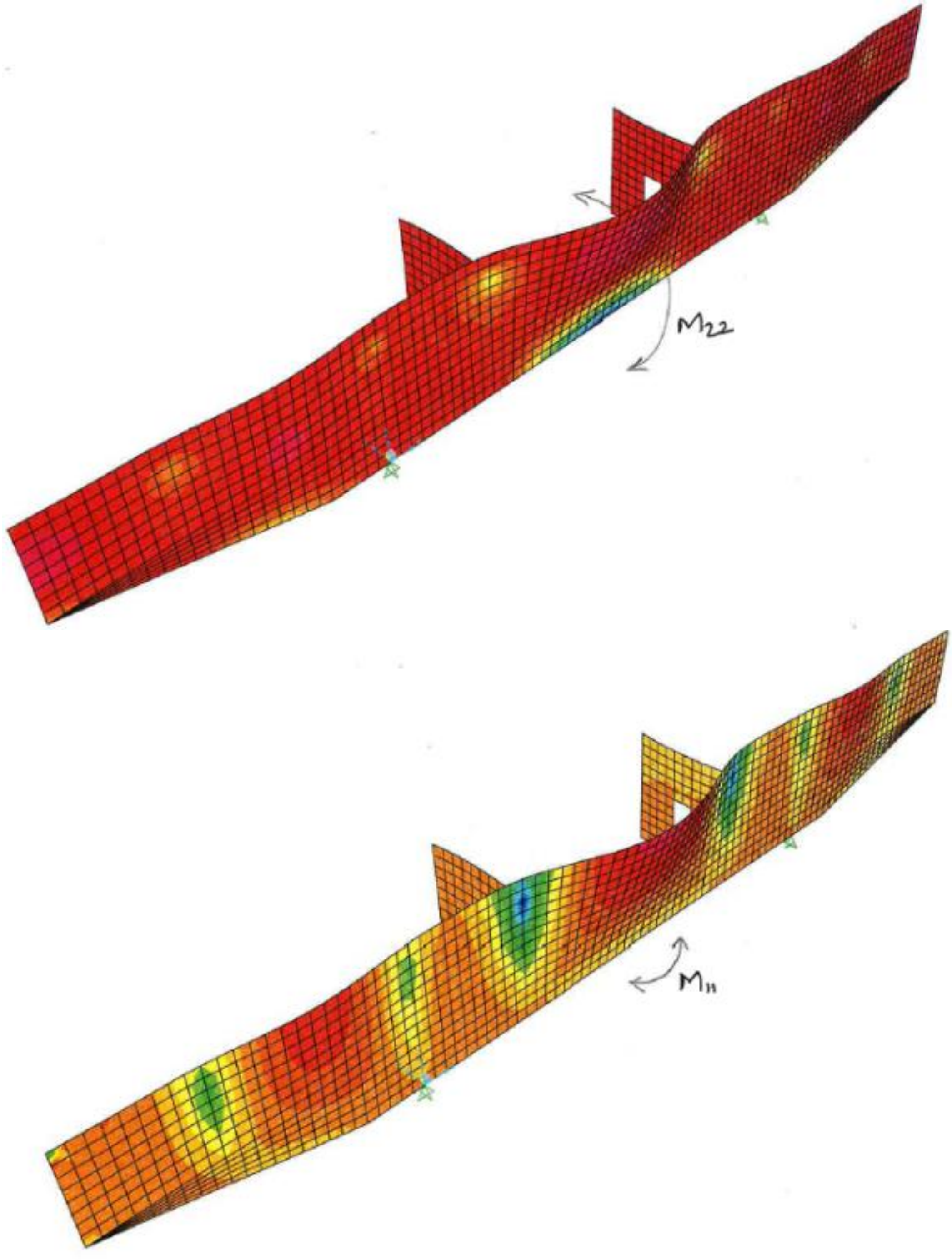


Fig. 5 – Wall Bending Due to Form Pressure

Prestressing is provided to resist tension from the global negative bending moments in the shell caused by the weight of the infill concrete. The prestressing is also a means to control the long-term shrinkage cracking in the shell. When the pier cap section is made composite, the precast shell carries the wet concrete weight of the infill. The resulting forces are locked in when the infill concrete hardens. More importantly, after the section becomes composite, the difference in initial stress, age, and exposed perimeter between the shell and the infill result in force redistribution due to differential creep & shrinkage. To analyze the redistribution of forces in the pier cap due to long-term creep and shrinkage, a staged time-dependent SAP2000 analysis model was used. In the analysis model, the precast shell and the infill concrete section are each modeled at their center-of-gravity, connected by links to model the composite section behavior. Elements are added to the model in stages, with appropriate material properties and age, in accordance with the assumed construction sequence.

The stages analyzed are as follows:

1. Transfer prestress to precast shell in yard (day 3)

2. Creep & shrinkage from day 3 to day 60 on precast shell

3. Cast infill concrete (day 60)

4. Creep & shrinkage from day 60 to End of Construction (day 365) on precast shell and infill concrete

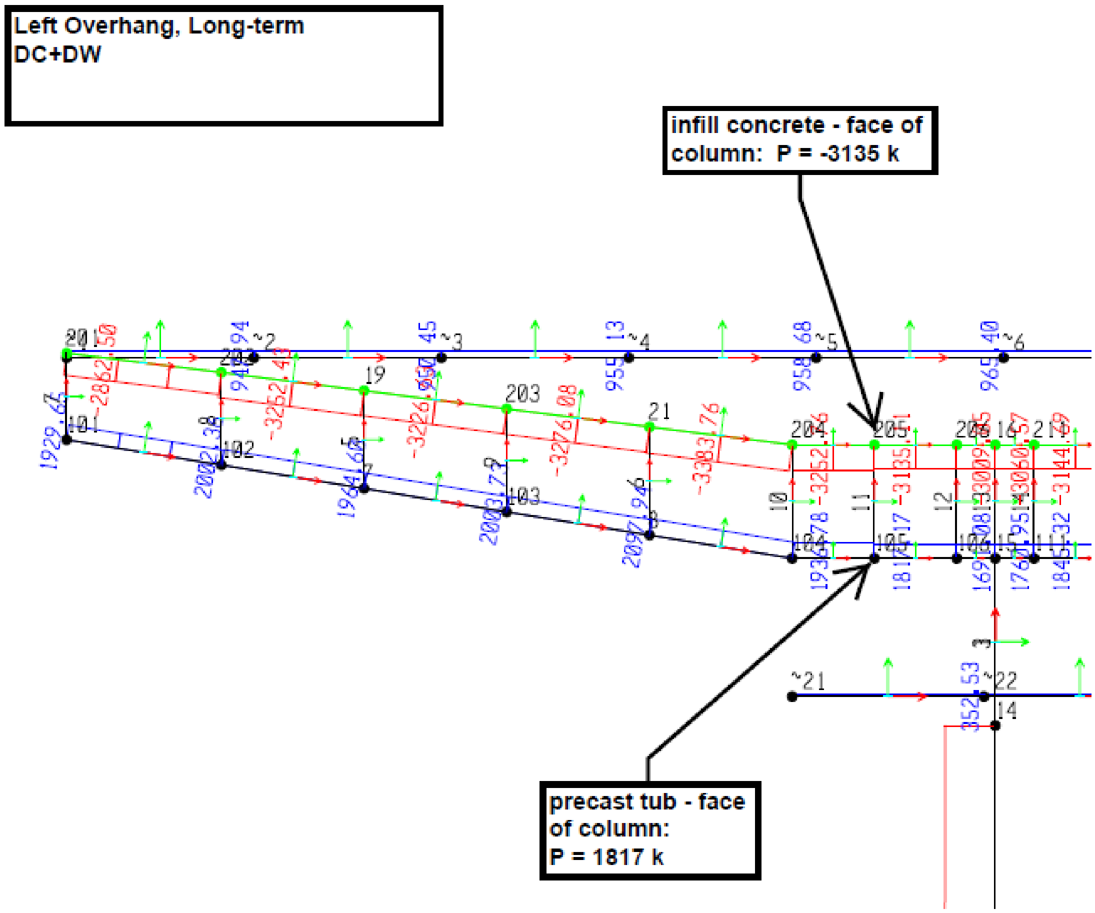
5. Added dead load at End of Construction (day 365)

6. Service I loading at End of Construction (day 365)

7. Service I loading removed

8. Creep & shrinkage from End of Construction (day 365) to End of Service Life on precast shell and infill concrete

9. Service I loading at End of Service Life



End of Construction (1 year):

DC+DW:		Axial (kips)	Moment (kip-ft)
Precast Tub	Left OH	-1318	-6354
Infill Conc	Left OH	-103	-32064

Long-Term (30 years):

DC+DW:		Axial (kips)	Moment (kip-ft)
Precast Tub	Left OH	1817	-6671
Infill Conc	Left OH	-3135	-41007

Fig. 6 – SAP2000 Analysis - Force Redistribution Due to Cr+Sh

Differential creep & shrinkage was evaluated using the provisions of the CEB-FIP Model Code 1990[2]. The large notional thickness (volume/surface ratio) of the infill relative to the precast shell results in significant differential shrinkage forces as the shell shrinkage greatly exceeds the infill shrinkage, and the shell shrinkage is resisted by the infill concrete (see Fig. 6). The resulting tension in the shell developed over time, combined with the negative service load moments due to live load, may cause cracking in the top of the shell wall later in the service life of the pier cap. This behavior is verified using both hand calculations and the time-dependent model.

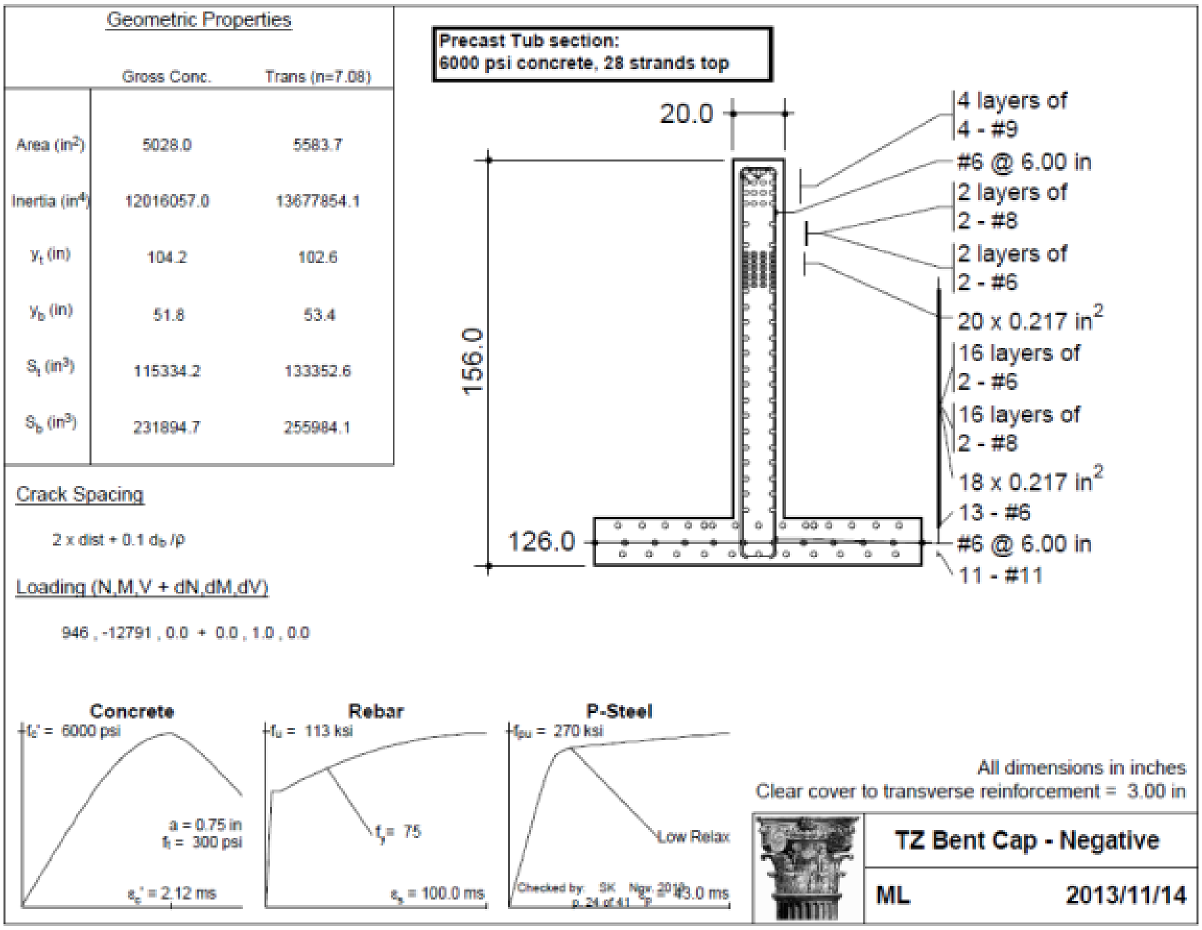


Fig. 7 – Response-2000 Analysis Section – Precast Shell

The crack widths in the shell walls at the service limit state are assessed using the reinforced concrete sectional analysis program Response-2000 (see Figs. 7 and 8), and the loads from the time-dependent staged analysis. The sectional analysis considers the loads in each component (shell and infill concrete) individually, using the results of the time-dependent staged analysis, at both end of construction and end of service life. The prestressing is used to control the crack widths to the limiting value of 0.012 inches provided by the project Corrosion Protection Plan.

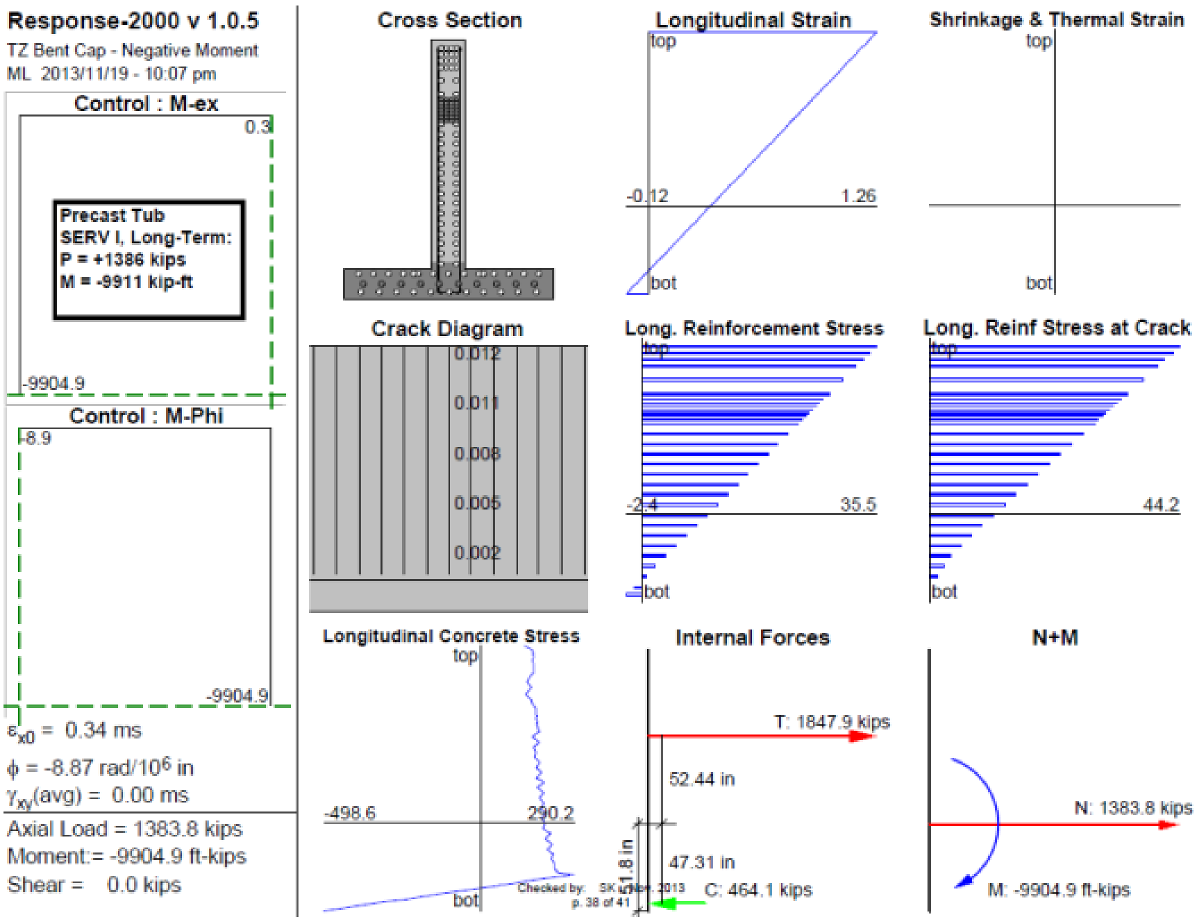


Fig. 8 – Response-2000 Analysis Results – Precast Shell

To facilitate production design of nearly 60 pier caps, strength design of the section is based on the pier cap acting as a monolithic section, and does not rely on the prestressing in the precast portion of the cap to resist ultimate bending loads. The conservatism of this approach has been verified using detailed moment-curvature analysis of the composite section.

PRECAST PIER CAP DETAILS

The composite behavior of the section is achieved by providing a roughened surface on the inside of the shell, as well as mechanical couplers for shear and torsion reinforcing steel. See Figs. 9 and 10. Shear flow on the interface is combined with the differential creep and shrinkage forces to determine the required interface shear transfer capacity.

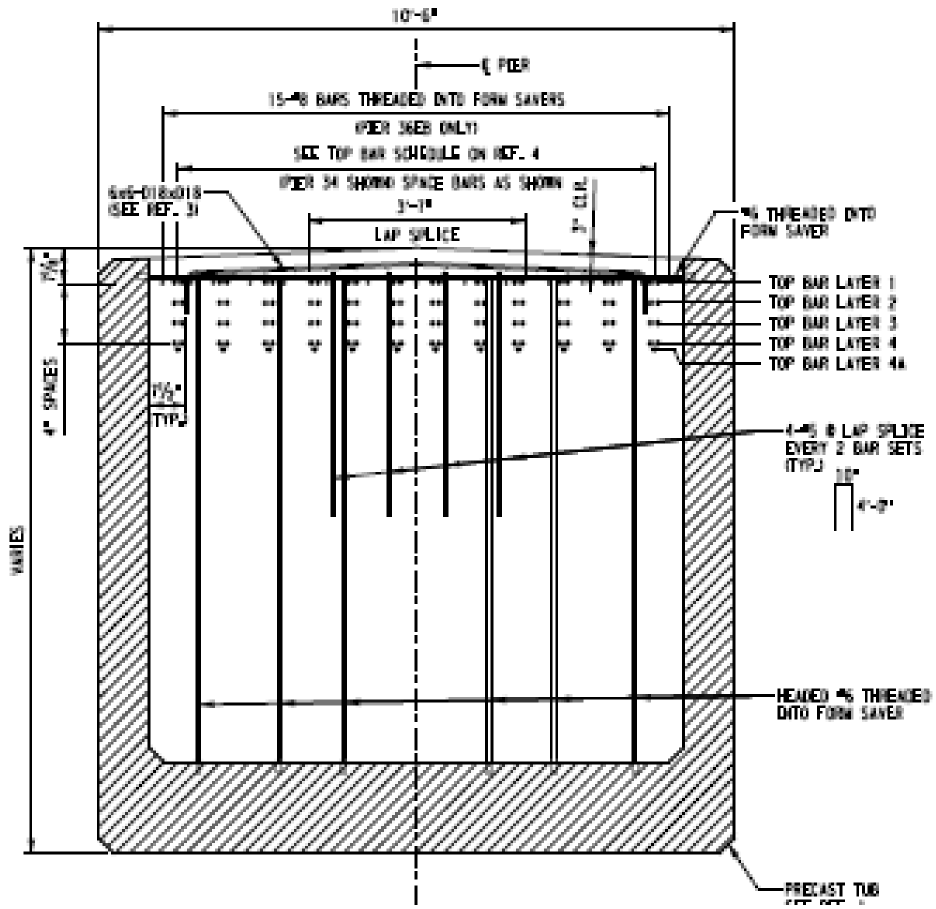


Fig. 9 – Cross Section with Infill Reinforcement

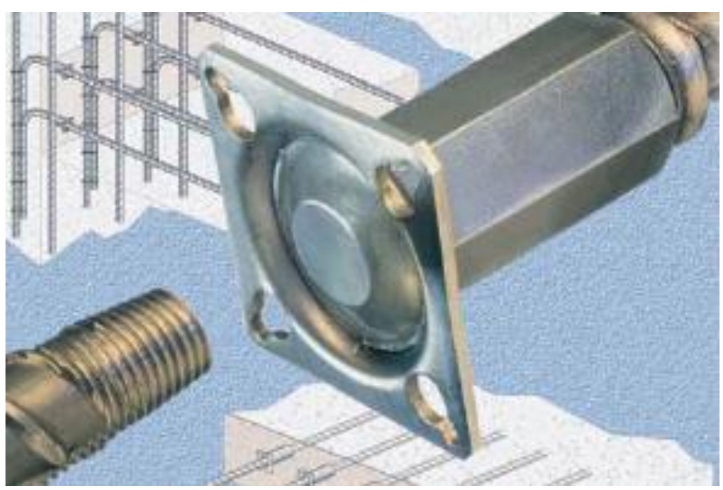


Fig. 10 –Mechanical Coupler

Details of the column connection are shown in Fig. 11. Galvanized ducts, 8-inches in diameter, are cast into the soffit of the precast shell to accept the column bars. The column vertical steel is extended into the pile cap a distance of 1.25 times the development length of the bundled column bars. To meet the requirement of continuing the confinement steel into the cap, transverse reinforcing is provided by threading bars into the diaphragm located over the column.

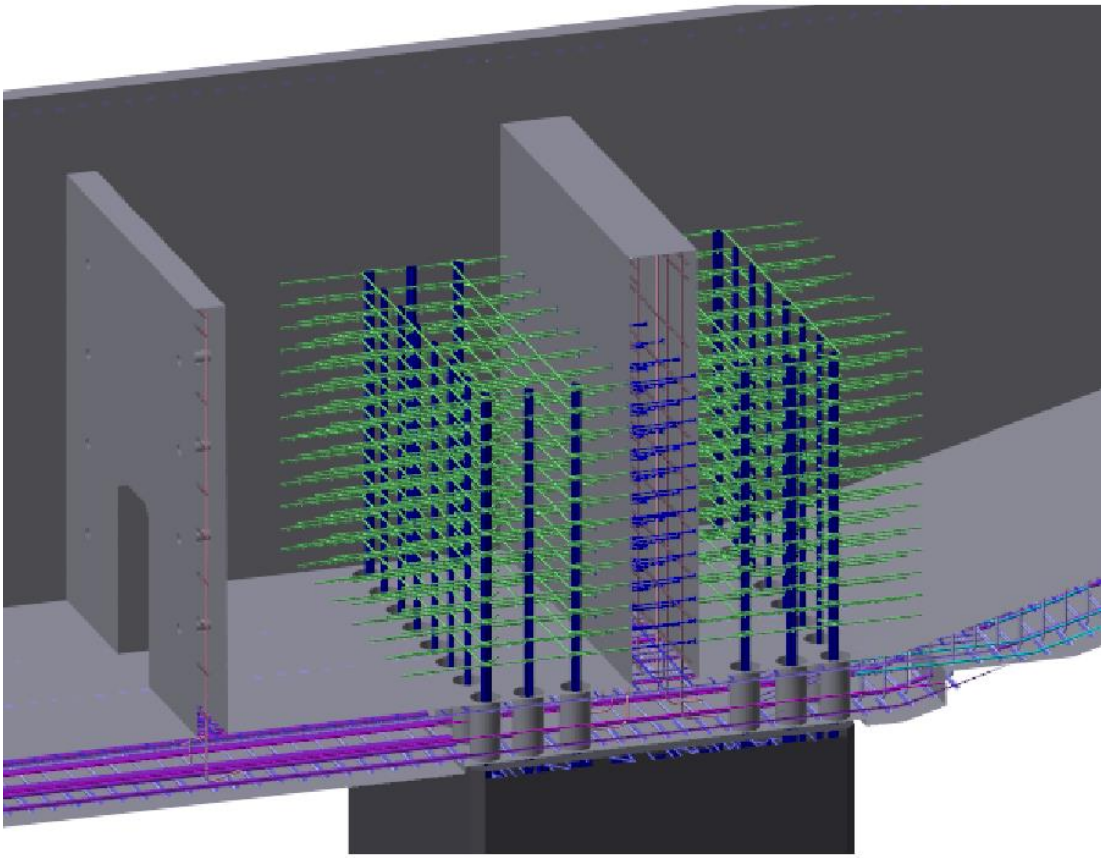


Fig. 11 – Column Connection

CONCLUSION

Precast pier cap shells were used on the Tappan Zee Bridge replacement to speed construction, improve quality, improve safety, and reduce labor costs. The precast pier cap shells served as forms to be filled with cast-in-place concrete, and were designed to perform compositely as part of the final structural section using rigorous analyses and careful detailing. Design of these components required consideration of intermediate construction stages and locked-in construction forces, using staged construction analyses. The 100-year service life requirements of the project necessitated consideration of crack widths, service level rebar stresses, and time-dependent effects.

While the cost of the precast substructure components is a more demanding design process, the size of the job and the repetitive nature of the approach structures will allow the design-build team to reap the benefits of a high-quality final product and greatly enhanced construction speed.

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