Approach Spans to the West Seattle Bridge

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Synopsis

Some of the largest precast prestressed concrete girders used to date in the United States are currently being installed on the West Seattle Freeway Bridge in Washington. This report describes the design/construction system adopted for the project's Harbor Island, West Interchange, and East Interchange units.

Construction efficiencies inherent in some of the largest precast prestressed concrete girders used to date in the United States are speeding the West Seattle Freeway Bridge towards its scheduled 1985 completion. This complex \$87 million urban freeway project consists of four major units as illustrated in Fig. 1.

The West Interchange connects the existing West Seattle Viaduct and surface streets to the main span crossing the West Waterway (main navigation channel). The Harbor Island unit carries the roadway across the industrial area and East Waterway. The East Interchange unit connects the project to the existing Spokane Street Viaduct, Alaskan Way Viaduct, and surface streets.

The segmentally constructed, post-

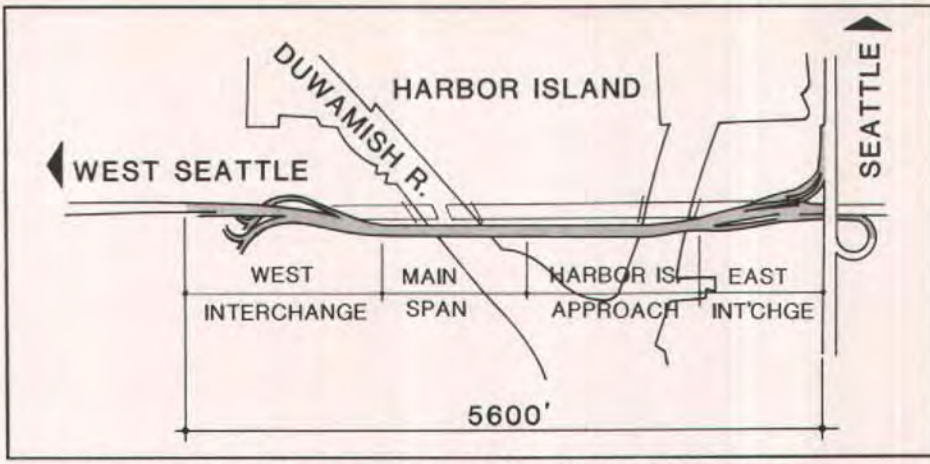


Fig. 1. Map of project area showing location of West Seattle Bridge approach.

Table 1. Summary of approach contracts.

Contract Number	Description	Deck Area (sq ft)	Girders Number	Girders Lin. ft
5	Harbor Island Approach	168,125	161	24,183
6	West Interchange	245,719	242	30,216
7	East Interchange	222,730	294	27,291

Note: 1 sq ft = 0.09 m²; 1 ft = 0.305 m.

tensioned concrete box girders comprising the three-span West Waterway Crossing have been described elsewhere.[1,2] This paper focuses on the precast prestressed concrete girder system used for the Harbor Island, West Interchange, and East Interchange units. The magnitude of the project is indicated in Table 1, which lists the number and lengths of girders and roadway areas for the main units.

Girder Design

Several site constraints combined to result in a clear need to maximize the span length while still preserving the economy inherent in precast prestressed girder construction. For example:

1. Soil conditions required expensive and heavy pile foundations.

2. Surface traffic conditions and public desires favored a layout which minimized the number of columns and provided an "open" appearance.

3. Construction phasing to accommodate traffic routing through the site during construction severely restricted the possible pier locations.

The principal design issues faced in the development of the approach units were aesthetics, economy, construction staging, and design for good seismic response. The solution, which evolved from these considerations, is a series of rigid frames made up of precast prestressed concrete girders and cast-in-place reinforced concrete columns, cap beams, and roadway slab.

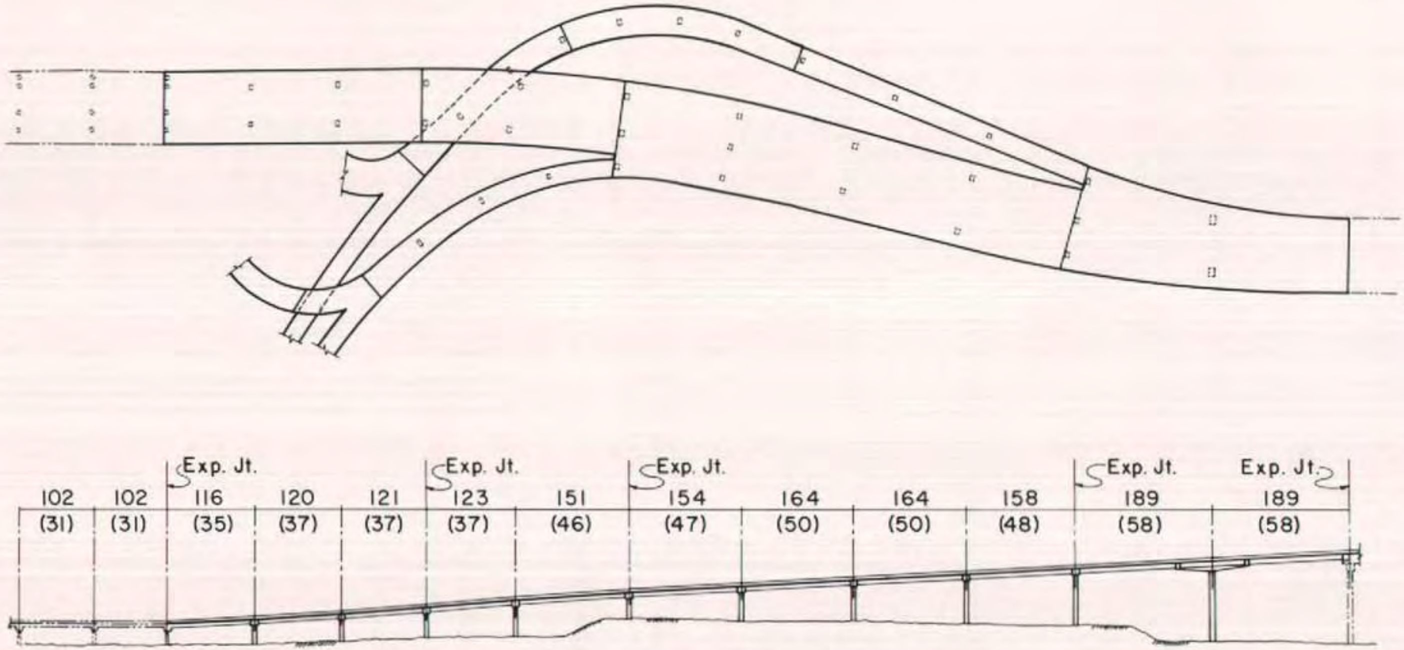


Fig. 2. West Interchange plan (top) and elevation (bottom). Span lengths given in feet (m).

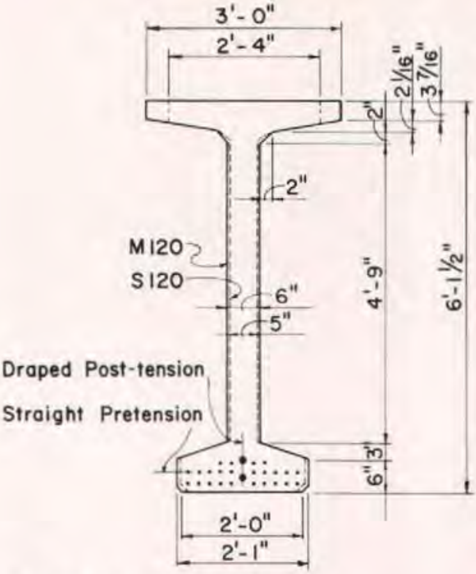


Fig. 3. Girder section showing modifications (M120) to standard section (S120).

The construction sequence was planned so as to maximize the development of dead load continuity moments at the piers. The rigid frames were an essential feature of the design for proper seismic response. Fig. 2 shows the plan of the West Interchange, which consists of four separate frames and two ramp structures which frame into the main roadway.

Variations of the basic design theme for the units were (1) different girder sections for shorter spans in areas of sharp curvature; (2) shorter frame lengths; and (3) the insertion of simple spans where required to decouple seismic response motions between frames.

The largest standard girder section available was the Washington State Department of Transportation 120 Series girders which had previously been utilized for spans up to about 145 ft (44 m). To allow the longer spans needed for this project, two modifications to this section were made which allowed easy adaptation of the existing standard forms (Fig. 3).

1. The web width was increased from 5 to 6 in. (127 to 152 mm), thus increasing the shear capacity of the section and allowing more room for concrete placement around the post-tensioning tendons.

2. The flange width and thickness were increased to provide better stability during handling and erection.

Using the modified section, the maximum girder length of 154 ft (47 m) was established after consultation with local precasters. Longer and heavier girders would have been beyond the capabilities of existing transportation equipment and would have made erection more difficult. These limitations established an upper limit on span length for this structure type. Horizontal clearance requirements over surface streets and railroad required a 185-ft (56 m) span adjacent to the main span unit which was achieved by widening the typical 10-ft (3 m) cast-in-place cap beam into a 70-ft (21 m) long cast-in-place reinforced concrete box girder over the pier.

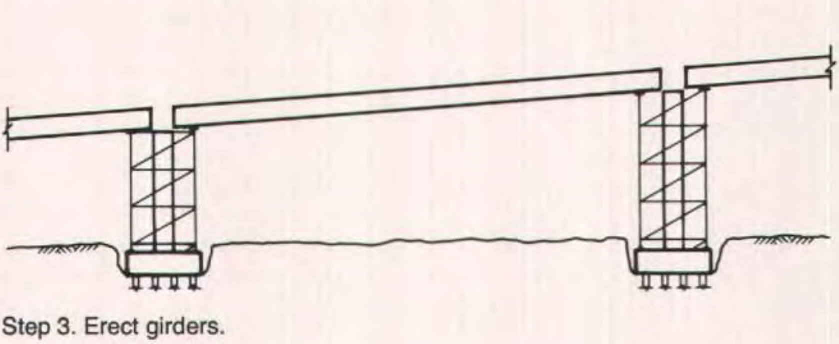
Prestressing of I-girders is provided by a combination of straight pretensioned strands in the bottom flange and draped post-tensioned tendons. The straight pretensioned strands furnish adequate prestress for removal from the pretensioning bed and handling in the yard. Some of the straight pretensioned strands were debonded by plastic sleeves near the end for stress control. Release strengths required were less than would have been required for a fully pretensioned design. Camber growth is better controlled since post-tensioning can be delayed until higher strength and modulus of elasticity are achieved. The turnover time for the beds was thus held to 24 hours. Post-tensioning operations were off the critical path and were performed at the yard storage locations.

An essential element in achieving the utmost structural efficiency and in satisfying the seismic design requirements was the construction sequence. This sequence was designed to obtain significant dead load negative moments at the piers through the sequencing of the slab pours. Staging the slab casting sequence so that the sections over the piers were cast before the sections over the central portions of the slab resulted in a larger dead load negative moment than would result if the slab were cast in one pour. This additional negative moment reduced the required positive moment at the midspan of the girders, but more importantly it prevented moment reversal (i.e., positive moments at the piers) under seismic response conditions. It is difficult to provide constructable, efficient details to resist positive moments at the piers in precast girder construction.

The construction sequence used is illustrated in Figs. 4 and 5. Poor foundation conditions made it necessary to

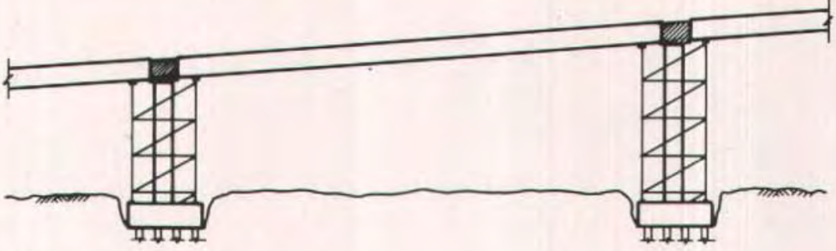
Step 1. Construct pier up to construction joint at cap beam soffit.

Step 2. Erect girder and cap beam falsework.

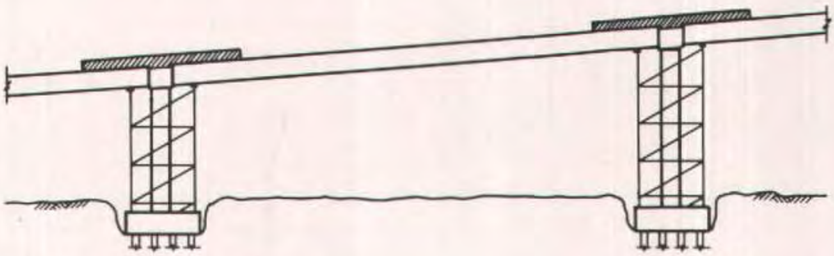


Step 3. Erect girders.

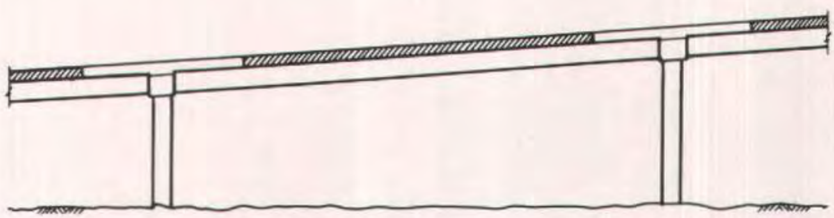
Fig. 4. Construction sequence.



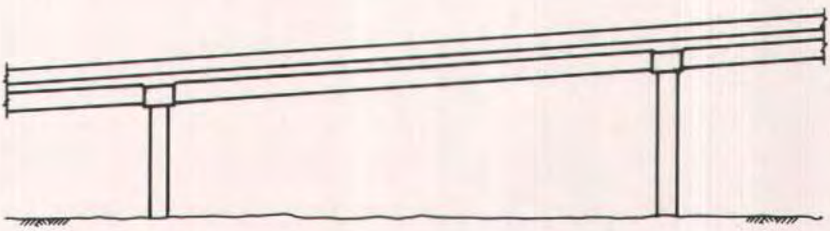
Step 4. Cast cap beams.



Step 5. Cast slab over piers.



Step 6. Cast slab over midspan portion.



Step 7. Cast barrier curbs and complete.

Fig. 4 (continued). Construction sequence.

Fig. 5. Girders erected on falsework prior to casting cap beams and deck slab.

Fig. 6. Shear-friction reinforcement in girder ends.

support the erected girders on steel falsework founded on the permanent pile caps. This specification requirement prevented premature overloading of the shear-friction connections between the girders and the cast-in-place cap beam due to falsework settlement.

Close attention was given to the detailing of reinforcing steel projecting from the ends of the girders. This reinforcement had to be closely coordinated with cap beam reinforcing to avoid interference. The resulting detail is the overlapping 180-degree hooks shown in Fig. 6. The reinforcement required for positive moment (due to seismic loads) where the girders framed into a pier with an expansion joint at the far side usually required additional mild steel at the bottom flange. The bottom straight pretensioned strands were extended in the cap beam in all girders. Fig. 6 also shows the shear keys which were developed to meet design shear requirements. Dapped-end girders at expansion joints had additional shear keys on the sides where the end diaphragm framed in.

Plans of detailing for the cap beam and girder interface included a scale model of the cap beam and all required reinforcement (Fig. 7). This model proved to be a helpful aid in preventing reinforcing steel conflicts. As a result of this prior planning, the project has experienced a minimum of field problems of reinforcing steel conflicts.

Construction

Concrete strength for the girders was specified at 7000 psi (49 MPa). This strength requirement is met routinely by precasting plants in the area. Transfer strength for pretensioning was 4000 to 5500 psi (28 to 38 MPa) depending upon the girder details. Strength at application of the post-tensioning was required to be 6000 psi (42 MPa).

The mix designs used for girder manufacture were predicated upon the

Fig. 7. Cap beam scale model.

Table 2. Typical concrete mix designs.

Material	Amount per Cubic Yard		
	Contract 5, 7		Contract 6
Cement, Type III	605 lbs	605 lbs	752 lbs
Water, total	249 lbs	212 lbs	300 lbs
Dry, fine aggregate (F.M. 3.1)	1190 lbs	1435 lbs	1176 lbs
Dry, coarse aggregate (⅞ in.)	—	—	1743 lbs
Dry, coarse aggregate (½ in.)	2105 lbs	1985 lbs	—
Water-reducing admixture	42 oz	30 oz	40 oz
Superplasticizer	—	60 oz	—
Slump	3-5 in.	3-5 in.	3-5 in.

Note: 1 lb = 0.45 kg; 1 oz = 28.35 g; 1 in. = 25.4 mm.

pretensioning transfer strength, since this strength was the critical requirement for girder removal from the casting bed. Some of the typical mix designs used are shown in Table 2. Weather conditions prevalent at the time of casting were the final determinant in mix design selection.

Erection of the 75-ton (68 t) girders required two 200-ton (181 t) cranes, as shown in Fig. 8. Girders were transported by tractors with steerable rear trailers, as shown in Fig. 9.

Summary

The precast girder approach structures on this project were successful in overcoming the traditional aesthetic objections to precast girder structures. This success was due to the attention given to (1) achieving a harmonious pattern and arrangement of girders in the structure as viewed from below; (2) maximum span lengths; and (3) the details providing cap beam soffits flush with the girders (Fig. 10).

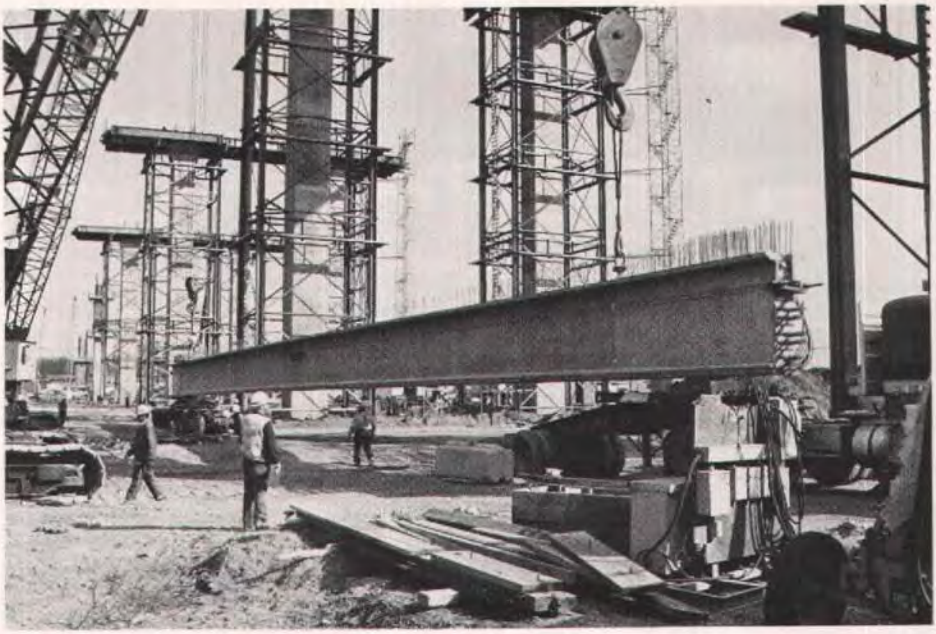


Fig. 8. Erection of girders.

Fig. 9. Girder transportation using steerable rear trailers.

Fig. 10. View from beneath structure.

Preliminary design studies clearly indicated the economy of precast girder construction over competitive structure types and these studies were reflected in the bids received. Precast girder construction also facilitated the maintenance of traffic through the area during construction by minimizing the falsework required.

REFERENCES

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2. "Girder Duo Cantilevers in Union," *Engineering News Record*, November 25, 1982, pp. 26-27.
3. "Record Girders Trucked to Site," *Engineering News Record*, March 24, 1983, p. 69.

Credits

West Seattle Design Team: Andersen Bjornstad Kane Jacobs, Inc., Seattle, Washington; Parsons Brinckerhoff, Seattle, Washington; Tudor Engineers, Seattle, Washington; Kramer, Chin and Mayo, Seattle, Washington; and Contech Consultants, Seattle, Washington.

Special Consultant: Dr. A. H. Mattock, University of Washington, Seattle, Washington.

Girder Manufacturers: Concrete Technology Corporation, Tacoma, Washington, and Prestressed Concrete Products, Woodinville, Washington.

General Contractors: Kiewit-Grice: (a joint venture), Seattle, Washington, and Moseman Construction, Seattle, Washington.

Owner: City of Seattle.

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