Systems Concepts for Precast and Prestressed Concrete Bridge Construction

Arthur R. Anderson

During the past 20 years, some significant changes in concrete bridge construction have taken place, mainly because of the developments in precast and prestressed concrete. The rate of change has recently intensified because of steep labor costs for job-site construction trades and the demand for longer clear spans to improve highway safety and aesthetics. Thus, the trend in highway bridge construction will be strongly influenced by economics, safety, short time schedules, and environmental impact.

During the past 50 years, the steel industry has taken the lead in the development of improved structural shapes and higher working stresses. For example, until 1923, the standard steel I-beam had a thick web and a relatively narrow flange. A Carnegie beam of this style compared with a modern wide-flange beam of equal weight is shown in Figure 1. The old-style section was rolled in depths ranging from 3 to 24 in., and a total selection of 29 beam sizes and weights was available in 1 quality of steel, with a specified working stress of 12,500 psi. During the intervening years, the steel industry developed a wide variety of very efficient beam shapes. These are available in 6 qualities of steel with yield strengths ranging from 36,000 to 100,000 psi.

In comparatively recent times, the quality of factory-produced concrete has advanced from the commonly accepted 2,500-psi compression strengths of the 1940's to today's concretes in the 7,000 to 12,000 psi range. Similarly the heavy massive concrete sections of previous times have been supplanted by the more efficient prestressed concrete sections shown in Figure 2.

The AASHO standard beams have found wide application for grade-crossing separation structures in most regions of the United States. Types III and IV have been most popular, satisfying needs for bridges in the span range of 50 to 90 ft. Concrete strengths of 4,000 psi at transfer and 5,000 psi at 28 days have prevailed. On the other hand, the Washington State practice has featured mainly the type 80, 100, and 120 beams, for spans ranging from 75 to 145 ft. Concrete strengths have been in the range of 5,000 to 8,500 psi at transfer and of 6,000 to 10,000 psi at 28 days. For average work, a transfer strength of 5,000 psi and a 28-day strength of 6,000 psi have been usual. Figure 3 shows the results of 28-day cylinder tests of concrete produced in Washington State for prestressed bridge beams. The majority of the tests ranged from 8,500 to 11,000 psi.

High-strength concrete in refined cross sections is achieved by the use of a water-cement ratio of 0.33 to 0.35, resulting in 0 to ½-in. slump. A pan mixer is preferable,

Figure 1. Standard I-beam of the early 1900's and modern wide-flange section.

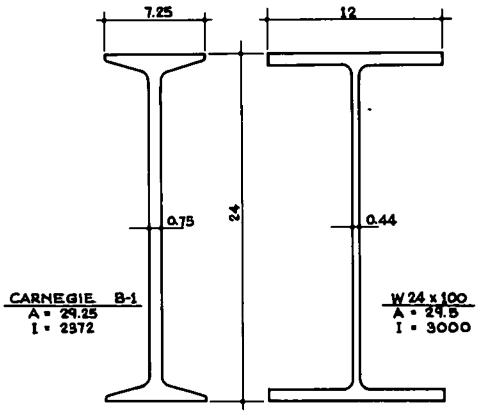
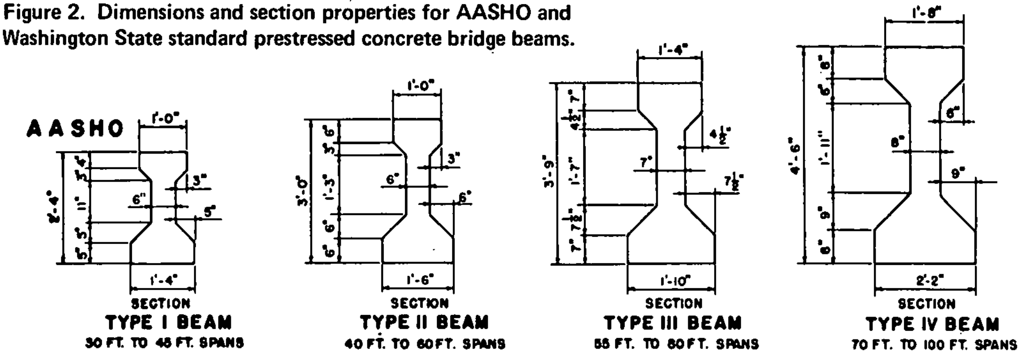
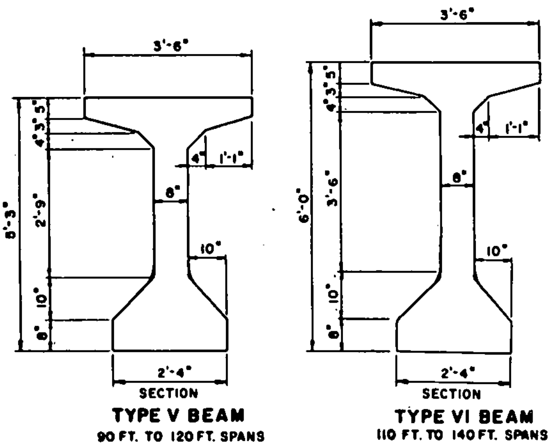


Figure 2. Dimensions and section properties for AASHO and Washington State standard prestressed concrete bridge beams.





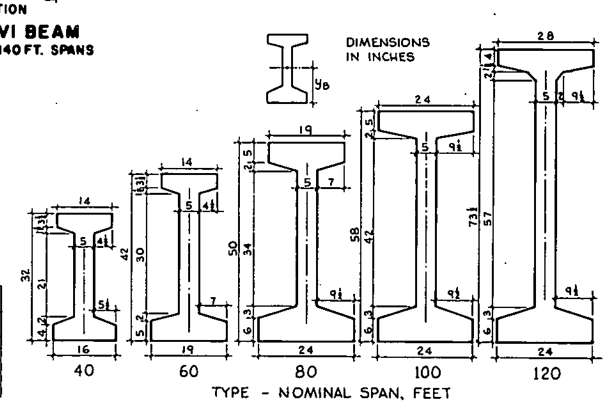
BEAM PROPERTIES

BEAM TYPE	AREA Inches²	ȳ Inches	Moment of Inertia Inches⁴
I	276	12.59	22,750
II	369	15.83	50,980
III	560	20.27	125,390
IV	789	24.73	260,730
V	1013	31.96	521,180
VI	1085	36.38	733,320

WASHINGTON

BEAM PROPERTIES

TYPE	AREA IN²	y_b IN.	MOMENT OF INERTIA-IN⁴
40	253	15.16	31,000
60	332	18.63	70,100
80	476	22.53	154,900
100	546	27.90	249,000
120	626	35.60	456,000



and compaction of the concrete requires form vibration. With automatic moisture control, uniformly high concrete quality is maintained. It is not unusual to achieve 6,000-psi strength in 16 hours, ensuring daily turnover of the stressing beds.

Because of the very low water-cement ratio and excellent compaction, concrete cover of ½ to ¾ in. over the web reinforcement has proved to be adequate for beams exposed to severe climate for 20 years. Thus, it is practical to utilize 4-in. web sections for bridge beams produced with high-quality concrete.

In 1959, Concrete Technology Corporation developed the bulb-T section shown in Figure 4. This section possesses very desirable properties in that the wide top flange ensures lateral stability for transportation and erection.

Also, the flange simplifies the placing of the cast-in-place concrete deck slab. Structurally, the flange serves as a haunch for the deck slab in resisting transverse bending due to concentrated wheel loads. Five-inch decks acting compositely with the flange have been tested with 64-kip concentrated loads, producing only hairline cracks. Contractors report a formwork saving of $1.50/sq ft of deck with the bulb-T beam. Moreover, a substantial saving in transverse deck slab reinforcement is possible because of the haunch effect of the bulb-T flange.

Several constraints to achieving high performance from factory-produced pretensioned beams should be evident to bridge designers.

Rapid turnover of the stressing beds is obtained by the prestress being transferred to immature concrete at about 50 percent of its eventual strength. For this reason, many producers favor low specified strengths (3,500 psi) at transfer. This practice is self-defeating, for it limits the stress level (0.6 f'_{ci}) conferred on the concrete. To overcome this handicap, one can design bridge beams with combinations of pretension and post-tension. A good ratio is about ⅔ pretension and ⅓ post-tension. This ratio produces enough pretension to almost counteract the beam dead-load bending, with moderate compression in concrete at transfer. The beam usually has 0 camber at this condition. Daily turnover of the stressing bed is easy to accomplish, and the beams can be stored until the concrete matures to full strength. At this time, the post-tension is introduced, which in effect means that the transfer strength f'_{ci} may also be nearly the full ultimate strength f'_c. By this means, the author has been able to achieve transfer strengths f'_{ci} = 8,500 psi, permitting 4,500-psi prestress levels in high-performance beams. This most effectively increases the span-load capacity of the beam.

Another advantage of the combination pretension and post-tension arrangement is the possibility of maximizing the eccentricity of the tendons, as shown in Figure 5.

An interesting comparison can be made regarding relative performance of the 3 bridge beam types described above. One direct measure of performance based on cross section alone is the relation between the beam's section modulus and its section area. Obviously, for a given section modulus, the least practical section area is to be desired. As with steel beams, the highest moment capacity-to-weight ratio within practical limits should be the designer's goal. Figure 6 shows graphically the relation between section modulus and section area for the AASHO, Washington State, and bulb-T standard bridge beams.

When one combines the advantages of high section modulus to section area ratio with the use of very high strength concrete and combination pretensioning and post-tensioning, the results become dramatic. Not only do the span load capabilities mount upward but also the ability to transport and erect very long beams becomes possible. Figure 7 shows graphically the maximum span capability for the AASHO, Washington State, and bulb-T standard beams for various beam spacings under HS20-44 live loadings on simple spans. To date, beams up to 147 ft in length have been delivered over the road in Washington State, and barge shipments of beams 167 ft long have been accomplished.

In 1969, the author developed a decked bulb-T bridge beam in which the bottom flange and web are cast from 160-pcf, 8,500-psi concrete.

A deck flange of 120-pcf, lightweight 6,000-psi concrete is cast monolithically to the web. This bridge beam, designated DBT (decked bulb-T), is practical for simple spans up to 190 ft, with depths ranging from 29 to 77 in. Deck widths of 4 to 10 ft are offered. Figure 7 also shows graphically the span-spacing capacity for the decked bulb-T beams. Since the beam depths given represent the total depth of section from finished roadway to beam soffit, the high span-to-depth ratio for this section is noteworthy.

Figure 3. Strength of 28-day cylinders for bridge-beam prestressed concrete.

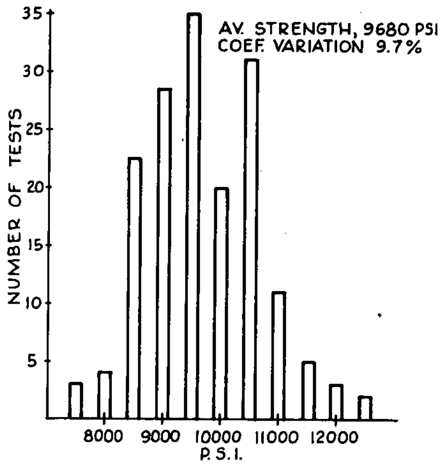
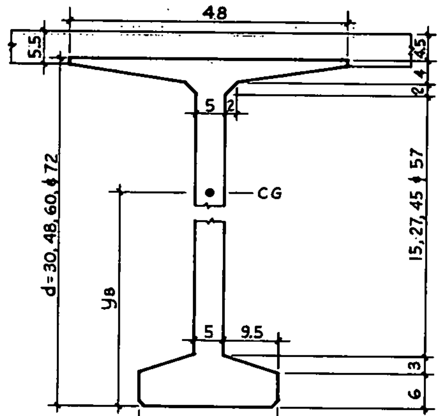


Figure 4. Bulb-T beam properties.



Depth d, in	Area in^2	y_B in	Moment of inertia in^4
24	380	12.14	28,900
30	410	14.95	51,100
36	440	17.78	80,700
48	500	23.51	164,000
60	560	29.29	283,200
72	620	35.12	442,800

Figure 5. Comparison of tendon eccentricity of beams.

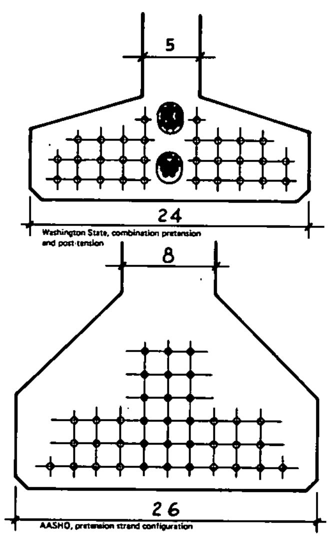


Figure 6. Section modulus and section area relations for prestressed concrete bridge beams.

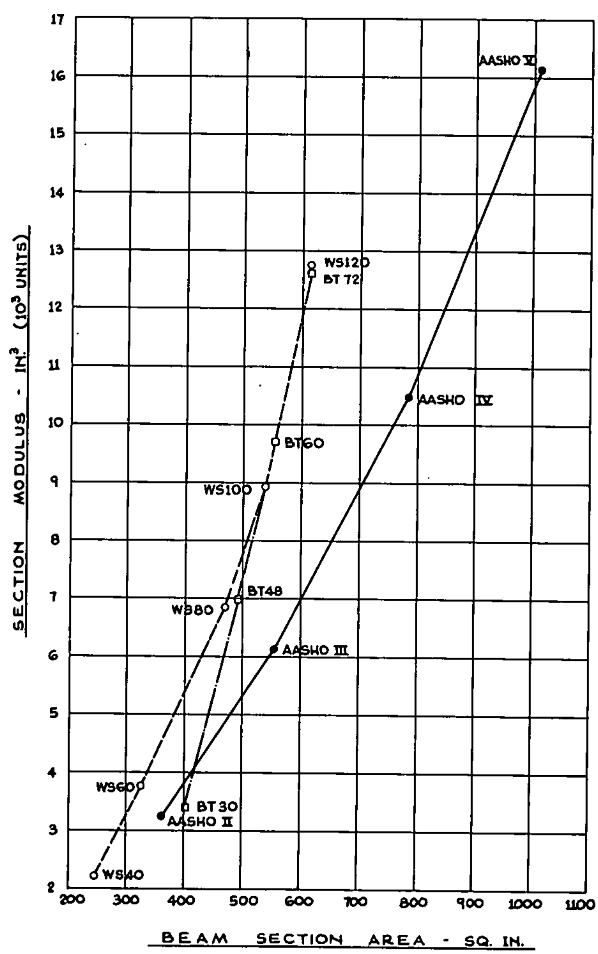


Figure 7. Span-spacing capacity for standard beams.

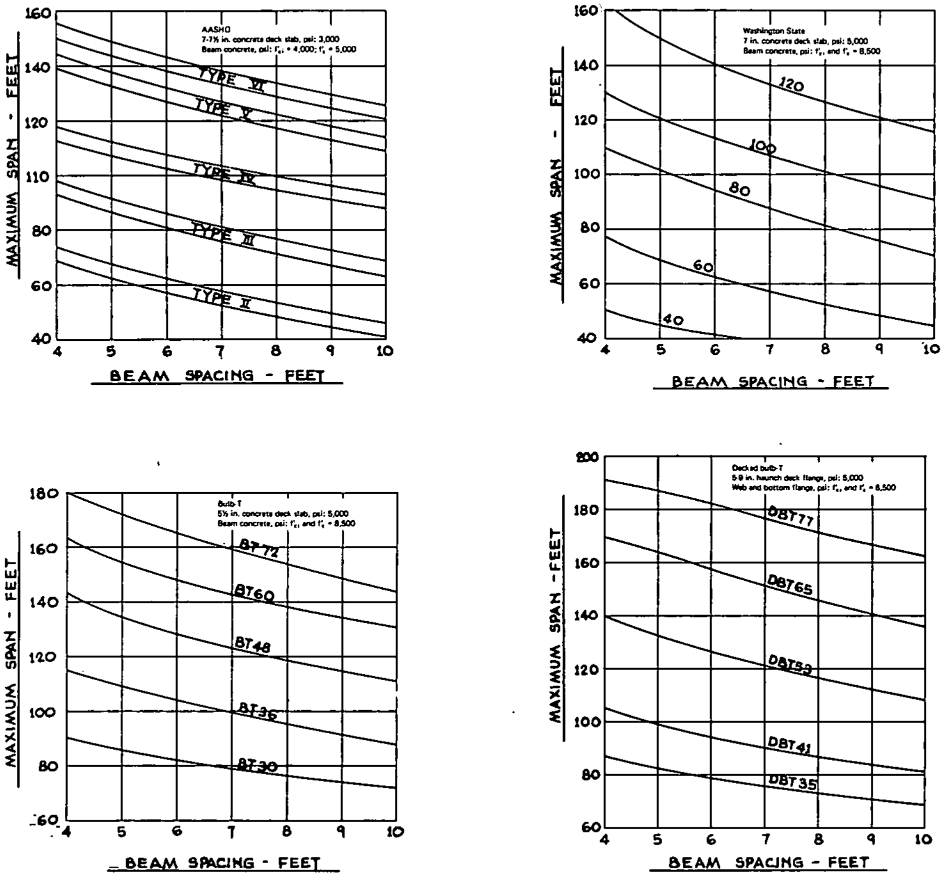
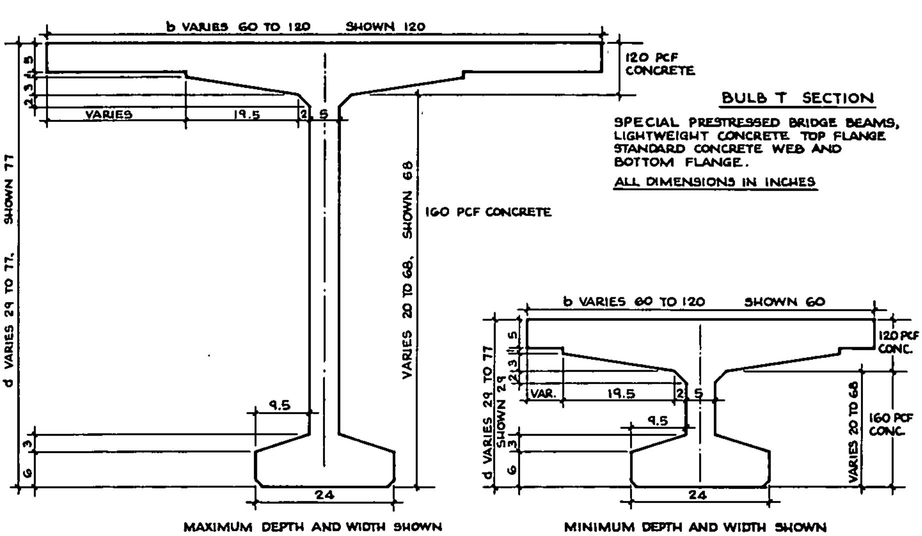


Figure 8. Decked bulb-T beam sections.



The cross sections for the DBT beam are shown in Figure 8. In the calculation of the properties for design, the lower elastic modulus of the deck flange lightweight concrete is compensated by transforming the section as shown in Figure 9. The elastic modulus of the top flange is 50 percent of the modulus for the web and bottom flange. Thus, the top flange is assumed to be half width in the calculation of the section centroid, moment of inertia, and section modulus. Of course, since plane sections remain plain during bending, the true top flange stress is actually one-half the value calculated for the transformed top flange.

The more efficient utilization of concrete with Washington State and bulb-T standard bridge beams has been clearly demonstrated. But the saving in prestressing steel for the longer spans is also impressive, as shown in Figure 10. For example, the bulb-T 72 spaced at 6-ft centers can reach a 160-ft span with 48 one-half in. 270k strands, whereas the AASHO VI beam spaced 6 ft requires 68 strands to reach a 140-ft span. It is also noteworthy that the bulb-T 72 and the decked bulb-T 65 and 77 reach out to spans of 160 to 180 ft with no more than 50 one-half in. 270k strands. The span-depth ratio can sometimes exceed 30, which is attractive when restricted clearance is a problem.

Thus, on the basis of materials and structural dead load alone, the savings offered by lighter, high-performance beams are impressive. From the standpoint of transportation and erection, economic implications are also evident. Highway truck-loading regulations pose serious contraints, for they place definite limitations on maximum weights transportable over highways. Freight tariffs escalate rapidly as the weight of haul increases. For example, the state of Washington has 6 tariff steps by weight categories. Step 1 is the lowest tariff, free of surcharge, and applies to prestressed beams below 52,000 lb. Above step 1, surcharges mount rapidly for overweight permits, extra axles, flagmen, and pilot cars. Costs per kip in the 6 tariff steps for 50-mile hauls in Washington State are as follows:

Tariff Step	50-Mile Haul Cost ($/kip)	Maximum Load (kip)
1	1.80	52
2	2.51	67
3	3.54	77
4	3.84	83
5	4.11	107
6	4.11	154

Table 1 gives the longest standard beam permissible within the weight limit of each tariff step.

Construction techniques play an important part in the economy of bridges. For example, the elimination of expansion joints over the piers of grade-crossing structures by introducing continuity for live load is attractive. In Washington State, longitudinal bars have been placed in the deck slab over intermediate piers to develop negative moments. The beams are placed on 4-in. wood blocks (Fig. 11d). The cast-in-place diaphragm over the pier fills the 4-in. space behind the block to provide the bearing. When the diaphragm concrete has matured, the deck slab is placed continuously over the intermediate piers. This concept not only eliminates costly expansion joints and bearing materials but also extends the span capacity of the beams beyond their simple span limits.

In the interest of safety and aesthetics, the trend for bridges over Interstate highways is to eliminate the intermediate piers (Figs. 11a and 11b). The author has proposed the elimination of all intermediate piers within the right-of-way by the introduction of inclined struts, as shown in Figure 12. This structure, with a 200-ft clearance at freeway grade, can be constructed with bulb-T 60 beams. The 120-ft beams, anchored to the exterior strut, reach beyond the interior strut, the cantilever arm supporting 120-ft drop-in beams. After assembly, the three 120-ft beams are post-tensioned together, creating a 3-span continuous structure.

Figure 9. Transformed sections of decked bulb-T beams.

TRANSFORMED SECTION BEAM PROPERTIES				
b/2	d	AREA	y_B	MOMENT OF INERTIA
30	29	456	15.24	48,200
	35	486	18.35	78,900
	41	516	21.46	118,400
	53	576	27.63	225,700
	65	636	33.77	374,400
	77	696	39.89	568,900
36	29	486	15.93	51,800
	35	516	19.17	84,600
	41	546	22.39	126,700
	53	606	28.76	240,600
	65	666	35.07	398,100
	77	726	41.32	603,400
42	29	516	16.54	55,100
	35	546	19.91	89,700
	41	576	23.23	134,100
	53	636	29.79	254,200
	65	696	36.25	419,800
	77	756	42.63	635,200
48	29	546	17.09	58,000
	35	576	20.56	94,300
	41	606	23.99	140,800
	53	666	30.72	266,600
	65	726	37.33	439,700
	77	786	43.85	664,600

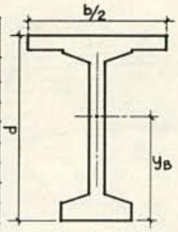


Figure 10. Strand requirements for prestressed bridge beams spaced at 6-ft centers.

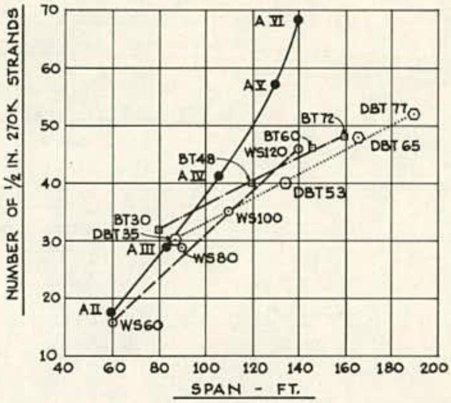
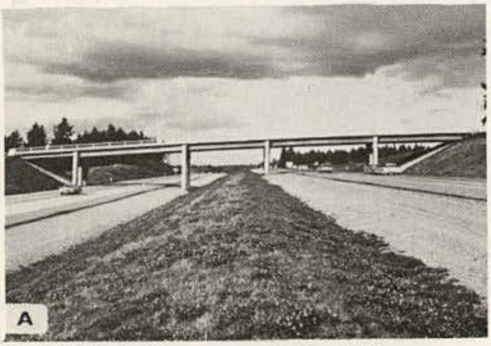
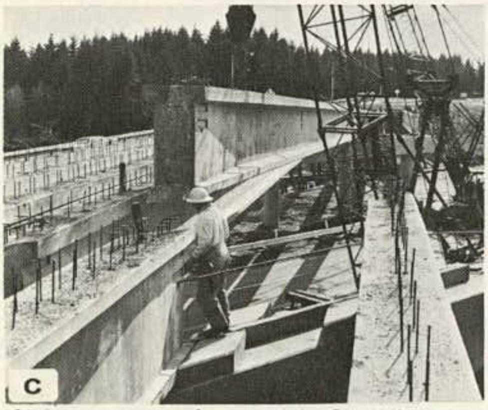


Figure 11. Washington State bridges.



50- to 80-ft simple spans erected in 1960

135-ft WS 100 beam delivered by truck and steering trailer (1966)



135-ft beam erected from truck by 2 mobile cranes (lifting weight, 21 tons per crane)

Prestressed beams on 4 x 4 in. wood blocks; space under beam ends filled with concrete for bearing

Figure 12. Bulb-T beam bridge with 200-ft clear opening at grade.

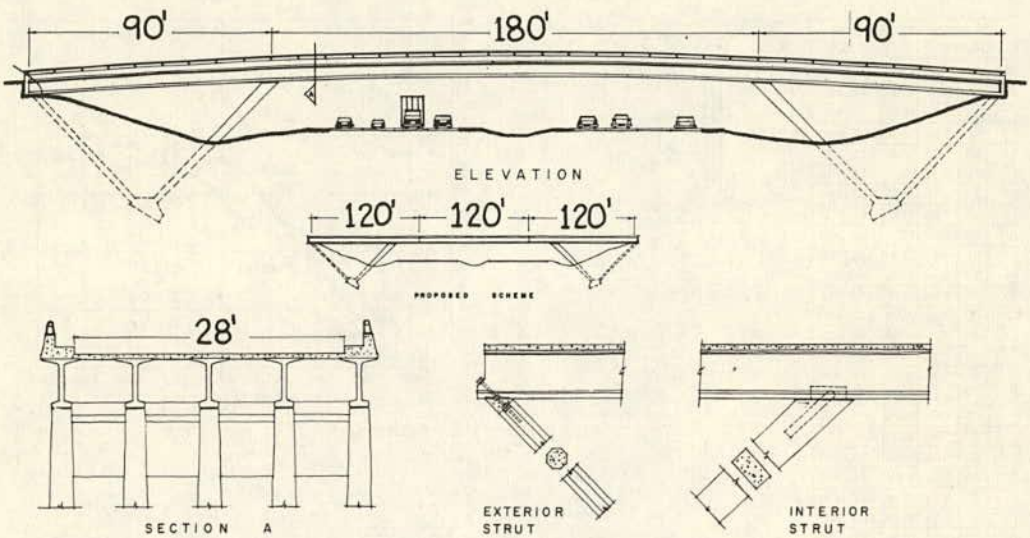
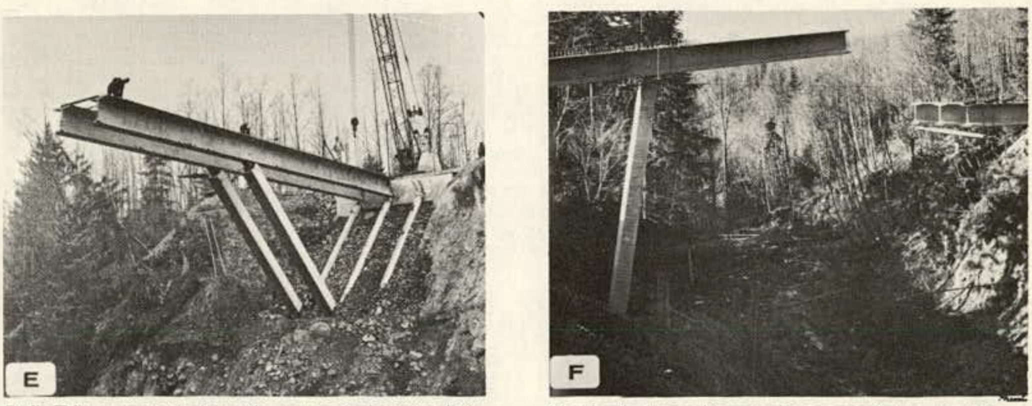


Figure 13. Inclined strut bridges.



Bulb-T beams erected on pin-connected struts that are prestressed and first connected to footing

Vertically braced beam pin-connected to top hinge, walked out over canyon, and secured to abutment

Table 1. Limiting beam length by tariff step for 50-mile haul in Washington State.

Beam Type	Unit Weight per Foot (lb)	Step 1	Step 2	Step 3	Step 4	Step 5	Step 6
AASHO III	580	88					
AASHO IV	820	63	82	94	101	115	
AASHO V	1,050	49	64	73	79	102	140
AASHO VI	1,130	46	59	68	73	95	136
WS 80	528	98					
WS 100	605	86	111	127			
WS 120	695	75	96	111	120	154	
BT 36	490	106					
BT 48	550	94	120	139			
BT 60	620	84	108	124	134	160	
BT 72	690	75	97	111	120	155	180

A number of inclined-strut bridges have been constructed for forest road bridges, such as the example shown in Figure 13. These bridges are designed for extra-heavy loads, approximately 3 times AASHO HS20-44, i. e., HS60 to HS70 loadings.

The organization of structure greatly simplified the construction, eliminated costly falsework over deep ravines and box canyons, enabled rapid assembly of the precast and prestressed elements, and resulted in substantial economy. The structure shown in Figure 13 was completed for the Weyerhaeuser Timber Company 6 weeks after receipt of the order to proceed with design.

During the past 5 years, construction costs have skyrocketed because of phenomenal escalation of site labor wages. Not only have construction hourly rates doubled in some regions but, even more serious, the availability of skilled construction journeymen has steadily declined. This trend will continue, which challenges bridge designers to resort to industrialized construction techniques. In other words, construction will turn toward manufacturing and assembly.

A good example of a totally prefabricated, prestressed, and precast concrete bridge construction is shown in Figure 14. Since this bridge was erected in the Olympic Peninsula of Washington State, 80 miles from the nearest ready-mix concrete source, and long-distance travel pay was required for construction labor, a high degree of prefabrication was indicated. On the other hand, logging operations in the vicinity of the project made cranes and bulldozers economically available.

A 66-ft span for a logging road was required to cross a river subject to sudden floods. This called for rapid construction during a short low-water period. The river banks were sound rock covered by 2 to 3 ft of overburden. Conditions suggested precast concrete abutments supported on 4 short columns, with a rock rip-rapped spill-through embankment. After the banks were cleared of overburden, holes were drilled into the rock to receive reinforcing bars projecting from the abutment columns.

Four DBT 35 beams constituted the superstructure. The beams were fabricated to identical camber, and the top surfaces were accurately finished to provide a uniform riding surface. Combination pretension and post-tension was applied to achieve high span-load capability with uniformity of camber.

When the site was ready, with favorable low stream flow, the 2 abutments were delivered, and in 1 day those pieces were erected. The reinforcing bars stubbed out of the columns were inserted into grout-filled holes in the rock. The abutments were completely installed on 1 day, and the 4 superstructure beams were delivered and erected in 8 hours a few days later.

Shear transfer between beams was developed by a galvanized steel K-shaped diaphragm at midspan. These were bolted to plates embedded and anchored to the concrete. The edges of the beam flanges were connected by weld-connecting steel inserts located at 6-ft intervals. The keys in the edges of the flanges were then filled with a stiff high-strength cement mortar. A few days later, the approach fill was placed and compacted, and the guardrails were installed. The bridge was then opened for traffic.

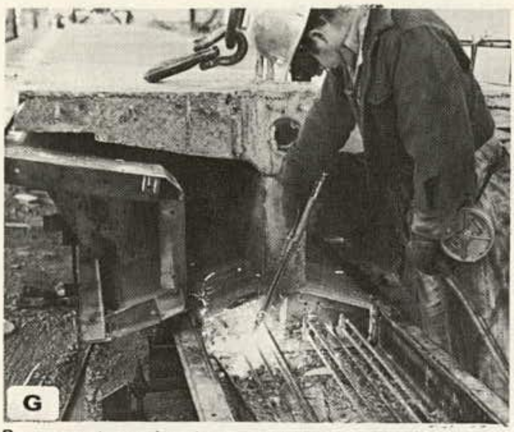
The contract price for all site work was $7,500, or $4.50 per square foot of bridge. The client, whose budget was $12,500, was naturally pleased.

Precast and prestressed concrete beams and girders for straight highway bridges are now commonplace throughout all of the United States. Considerable standardization has taken place, and design procedures have become routine. Much of the ultimate potential for prestressed concrete yet remains latent, waiting for discovery by bridge engineers everywhere.

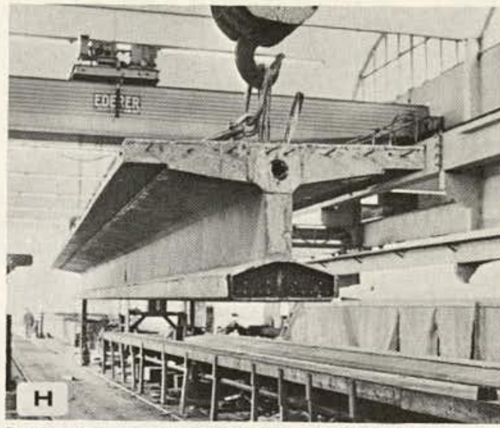
A recent project that has exploited some of the latent potential of prestressed concrete is the monorail for Disney World in Orlando, Florida. This structure in effect is a bridge for a rubber-tired vehicle, with fairly heavy wheel loads, shown in section in Figure 15.

Some 7 miles of 6-span continuous prestressed concrete box girder sections, supported on precast concrete columns, were constructed according to details shown in Figure 16. The girders, about 350 in number, spanned from 90 to 110 ft. Half of the girders were straight, and half were on vertical and horizontal curves to radii from 350 ft upward. Figure 17 shows that the hollow girders were of variable sections, 26 × 48 in. at midspan and 26 × 80 in. at the ends. The girder soffits were set to a parabolic configuration, whose curvature varied inversely with girder length.

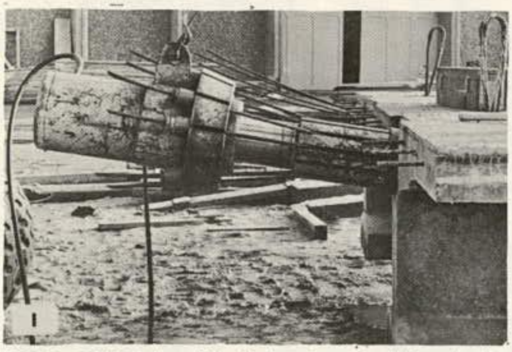
Figure 14. Totally precast decked bulb-T bridge.



Beam prestressed and removed from stressing bed; tension released on hydraulic rams; strands then cut with torch



Beam removed from 0 camber to storage until strength reaches 8,500 psi



Beams post-tensioned to design stress level (camber in all beams identical)

Precast abutment with stub columns anchored in solid rock; spill-through backfill placed

Girders complete with curbs erected; superstructure placed in 8 hours

Steel diaphragm bracing bolted to embedded steel plates to provide shear transverse between neighboring beams

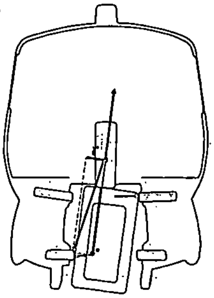


Figure 15. Girder loading of Disney monorail vehicle.

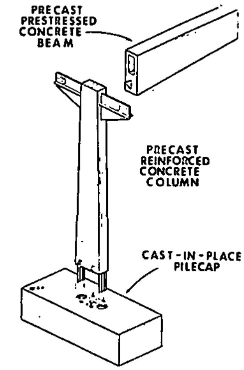


Figure 16. Main structural elements of Disney monorail.

Figure 17. Structural details of Disney monorail guideway (all dimensions in in.).

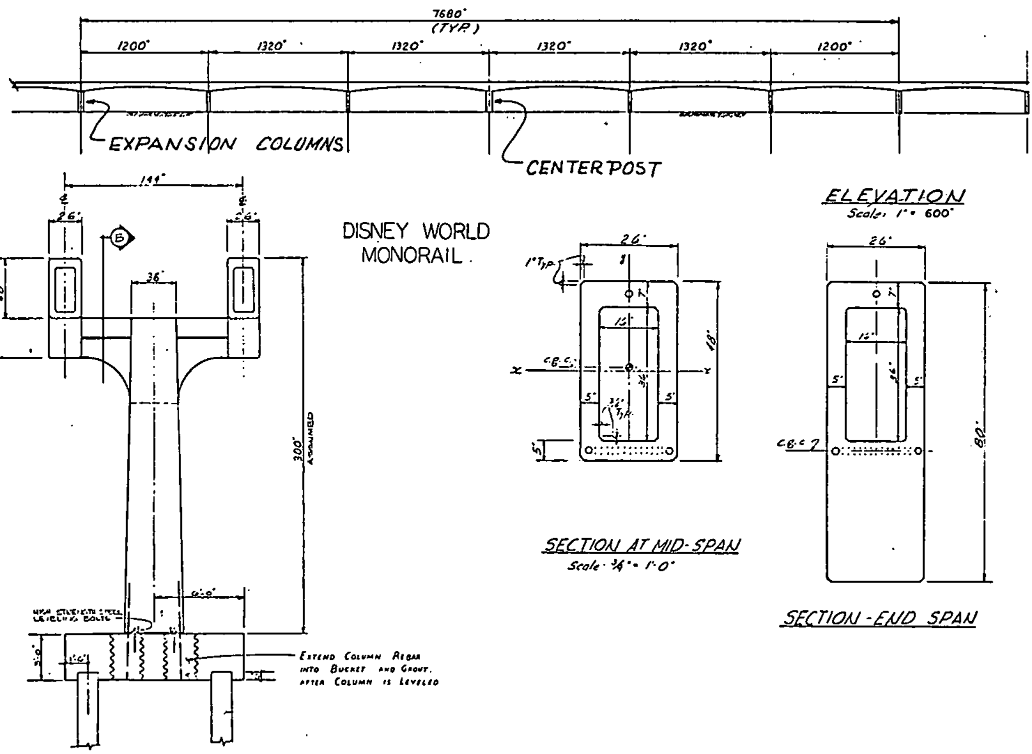


Figure 18. Strength of Disney monorail 28-day cylinders (avg strength, 8,600 psi, and coefficient variation, 5.67 percent).

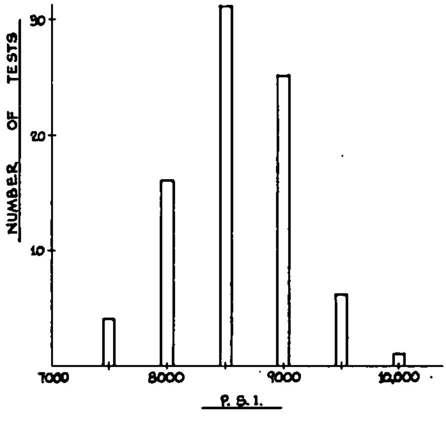


Figure 19. Geometrics for monorail system.

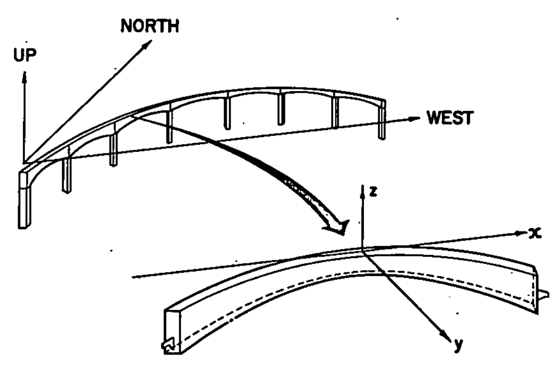
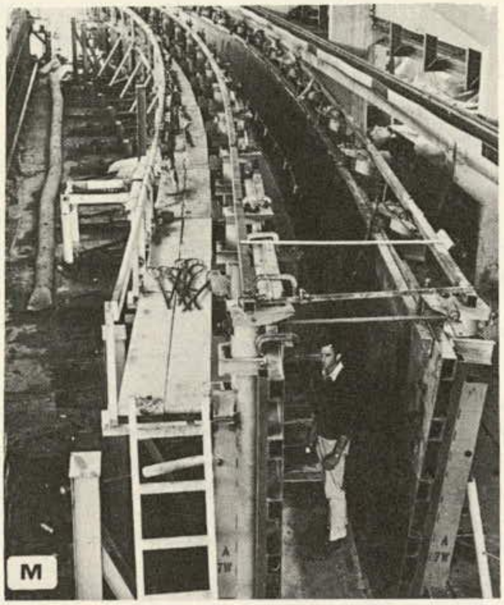
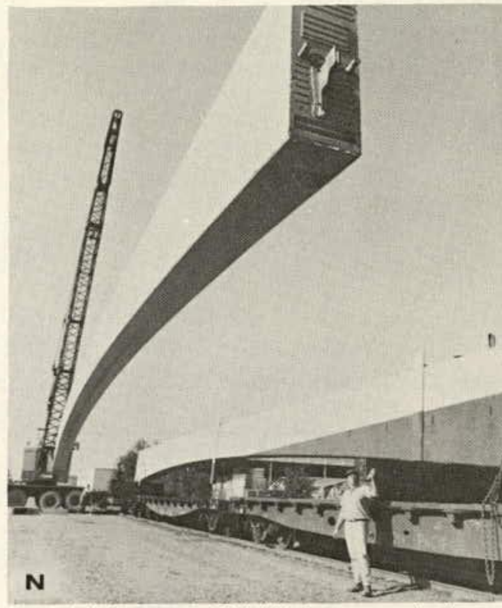


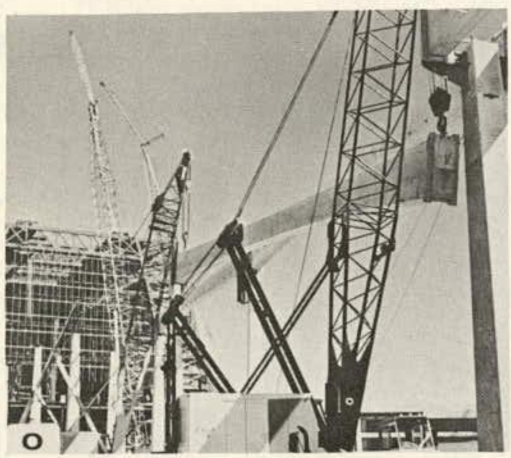
Figure 20. Construction of monorail at Disney World.



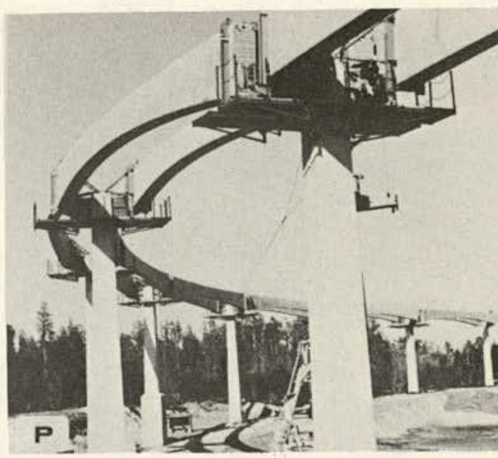
Variations in horizontal, vertical, and superelevation permitted by adjustable forms for curved sections of monorail



Curved girders stressed initially by post-tensioning at factory to compensate for dead-load bending (ducts for field post-tension cables project from girder end)

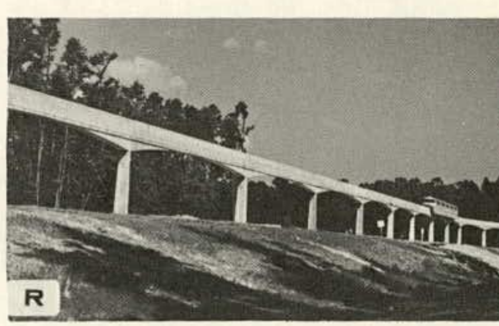


Girders erected and temporarily supported on projecting steel lugs



Cable ducts coupled by sleeves and gap between girders filled with cast-in-place concrete (6 girders connected into continuous spans)

Strands pulled through ducts; tendons 640 ft long post-tensioned and grouted



80 percent of expansion joints eliminated by continuity through post-tensioning; smooth guideway ensured by camber and deflection control

Both curved and straight girders were partially prestressed sufficiently at the factory to compensate for the dead-load bending of the girder. Concrete strength for the girders was 4,500 psi at transfer and 7,000 psi at 28 days. The actual strengths achieved are shown in Figure 18. Noteworthy is the fact that more than 90 percent of the tests exceeded 8,000 psi. The 5.76 percent coefficient of variation for production concrete represents first-class quality control.

To meet Disney's criteria for passenger comfort and smooth riding qualities required that the beamway dimensional tolerance be set at 0.1 in. True position for the columns was set at 0.1 in. laterally and 0.25 in. longitudinally and vertically.

The geometry of the monorail was defined by a system of north-south, east-west, and vertical coordinates that were established. All dimensioning was in inches and decimals. The X, Y, and Z coordinates of the beamway centerline at 100-in. stations were defined by mathematical equations. Mathematical labor was reduced by computerizing the geometrics. From the generalized coordinates of the system in space, a transformation was developed to define the geometry of the formwork for each girder, as shown in Figure 19. Horizontal and vertical curvature and superelevation angles up to 8 deg were required for the curved girders. Since no 2 curved girders were alike, the cost of a separate form for each girder would have been prohibitive. It was found more economical to build an elaborate, flexible steel form capable of bending to the shape of each and every curve in the project. The adjustable form is shown in Figure 20m.

The straight beams, usually 100- and 110-ft spans, were built in straight side forms, with an adjustable soffit, which accommodated the changes in the parabola curvature for each girder length. Zero-slump concrete was placed and compacted with powerful form vibration. One girder per day was produced in each form, with 16-hour strengths reaching 5,000 psi. Stage 1 prestress for balancing girder dead-load bending was by pretension in the straight girders and by post-tension in the curved members. When delivered and erected, the girders had virtually no camber from stage 1 prestress. Figure 20n shows girders being loaded on rail cars for delivery from the plant to the job site.

For erection, temporary support on the steel crosshead was provided by a projecting 1.75-in. thick steel plate at each end of the girder. The girders were placed to 0.1-in. true position tolerance on the crosshead by using precision survey instruments, including laser-beam equipment. Adjustable steel bracing hardware (Figs. 20p and 20q) was provided to fix the girders in true position. The protruding post-tension ducts (Fig. 20n) were coupled together, and the spaces between girder ends at all interior piers of each 6-span continuous girder series were filled with concrete.

For the typical series of 6 continuous spans, the post-tension cables, 650 ft long, were pulled into the ducts. They were stressed with hydraulic jacks (Fig. 20q) at the expansion ends, and then pressure grouted. For straight spans, no friction during stressing was experienced, and stressing was done from one end only. On the other hand, the effects of friction in the curved girders were minimized by jacking the tendons from both ends.

Although the Disney World monorail was a complex and challenging project, demanding the highest quality and perfection in workmanship, the project was executed to Disney's standards, and it promises to give the public a very smooth and comfortable ride. We hope the monorail system may be the answer to urban public transportation in many cities where existing land and rights-of-way may permit elevated structures over existing streets and highways. The application of prestressed concrete is an ideal solution to the problem.