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Precast, Prestressed, Spliced and Post-Tensioned California Bulb-Tee Girder Bridge

DESIGN EXAMPLE USING ADAPT

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ABSTRACT:

This work presents, by way of a California Bulb-T bridge design example, the design considerations specific to precast prestressed girders made continuous through splicing. Segmental construction, splicing, continuity post-tensioning, time-dependent effects, stress losses, and the composite action between the topping and the precast girders are addressed in the design example. Modeling of the bridge, using the ADAPT software system, is discussed, and followed by samples of the analysis results. Also, considerations specific to the design of segmentally constructed bridges, in particular Bulb-T girders, are briefly reviewed.

1 - INTRODUCTION

The typical bridge in California has, for the past 30 years, consisted of a concrete, cast-in-place, post-tensioned box girder. Precast, prestressed I-girder bridges have been avoided, since they can not practically span the 40 m to 60 m clear distance typical of California bridge structures. In addition, their performance under seismic forces has been in question. However, recent developments spearheaded by other states in the US, in particular Florida, followed by research conducted at the University of California, San Diego [Holombo et al, 1997] under the joint sponsorship of the California Department of Transportation (CALTRANS) and the Precast Prestressed Concrete Manufacturer Association of California (PCMCA), coupled with new design tool capabilities [ADAPT, 1997], have opened the way to a novel I girder bridge design for California highways.

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The proposed scheme has five distinctive features:

(i) It extends significantly the clear span, traditionally covered by prestressed, precast beams, by splicing the girders within the span, instead of over the support.
(ii) It ties the precast girders to the bents (piers) using a transversely post-tensioned, cast-in-place bent cap, designed to resist high intensity seismic actions likely in California.
(iii) It provides multi-span continuity of members.
(iv) By avoiding obtrusive ground support, it minimizes interference with automobile traffic below.
(v) Its design process can be readily facilitated using new software with the ability to model and analyze the segmental construction sequence, with due considerations for the time-dependent effects of creep, shrinkage, aging of concrete, relaxation in prestressing and compŏsite action.

This paper gives an outline of the design considerations of segmentally constructed, bulb-T girder bridges and concludes with a design example, using ADAPT-ABI software. The extent of the design example is limited to the computations specific to the segmental construction and time-dependent parameters. Treatment of live loading follows standard procedure for the completed structure. Seismic design of the completed bridge is not included. The reader is referred to publications from University of California, San Diego for further information [Holombo et al 1997].

1.1 Scope and Objective

To ensure the serviceability and safety of both the individual components of the bridge, during transportation and erection, and the completed bridge, during its design life span, the focus of the structural design is on the following:

- Geometry control during construction and service life
- Crack control
- Strength safety against overload

The computation of stresses and actions in a bridge are procedures aimed at establishing the satisfactory performance of the bridge in regards to the preceding objectives.

Other important considerations, such as fire proofing and cover for durability are handled separately.

Recent developments in computational techniques and structural engineering enables the structural engineer to do away with traditional laborious procedures and their associated approximations. With expedient design tools, the bridge engineer can now place a greater emphasis on improvements in performance and economy.

Before proceeding with the design example, it is in order to present a brief outline of: (i) segmental construction in general, (ii) essential features of software design tools for segmental construction, and (iii) several aspects of the new integrated design procedure.

1.2 Segmental Construction

A segmentally constructed bridge, is built from discrete components which are assembled over a period of time. In addition, a segmentally constructed bridge has one or more of the features described below:

(i) The components are called upon to carry loading in a configuration and through a construction-phase structural system other than that of the completed structure. A good example is balanced cantilever construction, where the cantilevering structural system of the bridge is for construction phase only.

(ii) The construction loading on the bridge components, either through construction equipment or assembly procedure, results in stresses which exceed those of the completed structure. Hence the design of the components, including the prestressing, is controlled, in part, by the construction technology.

(iii) Early-age loading of concrete, oftentimes within the first 24 to 48 hours after casting, leads to high deformation values which must be carefully evaluated and accounted for in the deflection and camber control of the completed structure.

(iv) The bridge undergoes significant changes in its load carrying structural system during its erection. Spliced-precast-prestressed girder bridges are generally assembled with interim supports. As simply supported members they carry their selfweight. When spliced they take the load of freshly placed topping before the composite action sets in.

(v) The method of construction greatly influences the initial stresses in the completed structure to the extent that the analysis of the completed structure, without regard for its construction scheme, becomes irrelevant. Consider the span-by-span construction of a two span bridge made continuous over the common support. The self weight moment at the interior support is primarily governed by the method of construction.

(vi) Most retrofit projects involve the addition of fresh concrete, external or internal prestressing, and recently developed synthetic fabrics (wrapping). Mixed material properties, the interaction of shrinkage and creep strains of the freshly placed concrete with the retrofitted components in resisting the applied loading, and the subsequent redistribution of loading among the new and existing components all require time-delayed analysis specific to segmental construction.

In summary, there is need for a segmental construction analysis procedure wherever the effects of time, changes in the structural system, and high construction loads impact the performance and safety of a structure, both during construction and when completed.

Prime examples of segmentally constructed bridges are:

- Prestressed-precast-spliced girders with post-tensioning and concrete topping
- Balanced cantilever construction
- Various schemes of span-by-span construction for continuous bridge frames
- Incrementally launched bridges
- Bridges retrofitted with external tendons, concrete jackets, and synthetic fabrics
- Cable stayed bridges
- Suspension bridges

1.3 Analytical Features Essential for a Comprehensive Design of Segmentally Constructed Bridges

Depending on the specifics of the proposed bridge, the design algorithm, whether implemented in a software package or used directly, should have several or all of the following capabilities:

(i) Ability to model the construction of the bridge as it is being installed (this includes the addition of bridge components, such as brackets or strong backs, and the addition or deletion of temporary members);
(ii) Capability to model the start of construction at several independent locations and at different times, each following its own schedule, and finally to bridge the components into an integrated, completed structure;
(iii) Addition and deletion of temporary supports;
(iv) Externally applied displacements to control deformation;
(v) Inclusion of the stiffness of construction equipment, such as launching girders and form travelers, whenever attachment of the equipment to the bridge components affects the stiffness of the bridge structure;
(vi) Capability to model both pre- and post-tensioning, for bonded and unbonded construction and mixed systems;
(vii) Complete capability to model different concrete materials, based on model codes, such as ACI, FIP/CEB, AASHTO, or based on laboratory tests of concrete representative of the bridge to be designed;
(viii) Authentic modeling of creep and shrinkage;
(ix) Ability to model composite action between old and fresh concrete, or other materials, such as synthetic fabric wrapping aroundconcrete
(x) Allowance for aging of concrete;
(xi) Capability to model external tendons;

(xii)Capability to model cable stays;
(xiii)Determination of losses in prestressing due to friction, seating loss, relaxation, creep, shrinkage and aging of concrete.

Apart from the features listed in the foregoing, there are a number of convenience requirements, such as the ability to view the modeled structure and the computed solution. Graphical generation and display of both the structural model and the computed results greatly limit the potential for errors in data generation and interpretation of the results.

Other convenience features include load combinations, enveloping of the results, generation and application of moving loads in combination with other loading conditions.

1.4 Selected Features of the Integrated Design Procedure

One of the strong features of the integrated analysis procedure, such as the one developed for the ADAPT-ABI software, is the major simplification in analysis procedure coupled with increased accuracy. In most traditional designs, a simplification is associated with an approximation and a corresponding reduction in accuracy. This is not the case for the integrated analysis procedure.

1.4.1 Elimination of n=Es/Ec from the Analysis and the Associated Approximation

Concrete gains strength as it ages. In addition, its immediate response to applied loading, expressed through its modulus of elasticity (Ec), shows greater stiffness with lapse of time.

In non-segmental construction, where the instantaneous response of a composite construction to applied loading is sought, it is common practice to model the structure based on the modulus of elasticity of concrete at the time of applied loading. In segmental construction, however, where time-dependency and load history are of prime importance, selection of a constant value for the concrete's modulus of elasticity is not realistic. In the integrated analysis presented herein, this discrepancy is resolved by considering the concrete's modulus of elasticity (Ec) as a variable with time. Thus the question of determination of a modulus of elasticity ration, "n," between steel and concrete becomes redundant.

1.4.2 Elimination of Transformed Section Consideration for Composite Action Between Old and New Concrete

Since in the integrated design procedure, the age of each concrete component is traced throughout the analysis, and its modulus of elasticity is continually updated with the lapse of time, the question of assuming an effective width for the newly cast concrete does not arise. At any given time, and for any applied loading, the response of the bridge is determined with the relative stiffnesses of its components, each made current to the time of loading (See figure 1.4.2-1).

1.4.3 Simplification in Stress Loss Computation in Prestressing

In the traditional analysis, it becomes necessary to compute stress losses in prestressing,
due to creep and shrinkage of concrete, as a separate item, and then make allowance for them. In the integrated analysis procedure, the stress loss calculations are implicit in the computations. The difference between the two approaches is explained with the aid of Fig. 1.4.3-1.

Consider a representative segment of a bridge (Fig.1.4.3-1), in which only one prestressing tendon is shown for simplicity. Part (a) of the figure shows the segment in isolation, in which the prestressing is represented in the traditional manner by its replacement through the forces it exerts on the segment. The prestressing forces "P" are viewed as a set of applied loads, such as dead and live loading. The prestressing forces are then replaced by their equivalent set of actions at the centroid of the segment (d). Due to time dependent effects, such as shrinkage, applied loading or relaxation in prestressing, the segment deforms, as illustrated in (f). Since the prestressing actions are considered as a set of equivalent applied loads, they are treated in the same manner as the dead loading. In the analysis, the equivalent prestressing actions remain unchanged (f). The actions shown in part (f) are the same as those shown in part (d). Stress loss in prestressing, due to time-dependent effects, such as creep, must therefore be computed separately and accounted for.

The alternative modeling, which is adopted in ADAPT-ABI, does not assume the tendon is removed from the segment (part c). Each tendon in the segment is viewed as an independent element, subject to deformation and change in stress based on the deformation of the segment within which the tendon is housed and the stiffness of the tendon. If more than one tendon traverses the segment, there will be more than one tendon element in that segment. In other words, the prestressing tendons are not lumped into an equivalent representation. The tendon is first stressed and its stress losses due to friction are computed. Then the tendon element is assumed

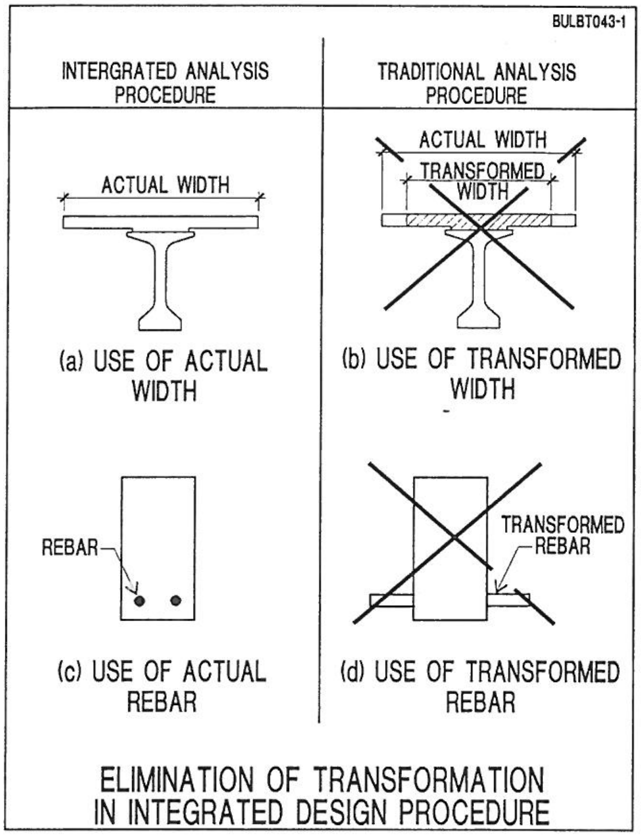


FIGURE 1.4.2-1

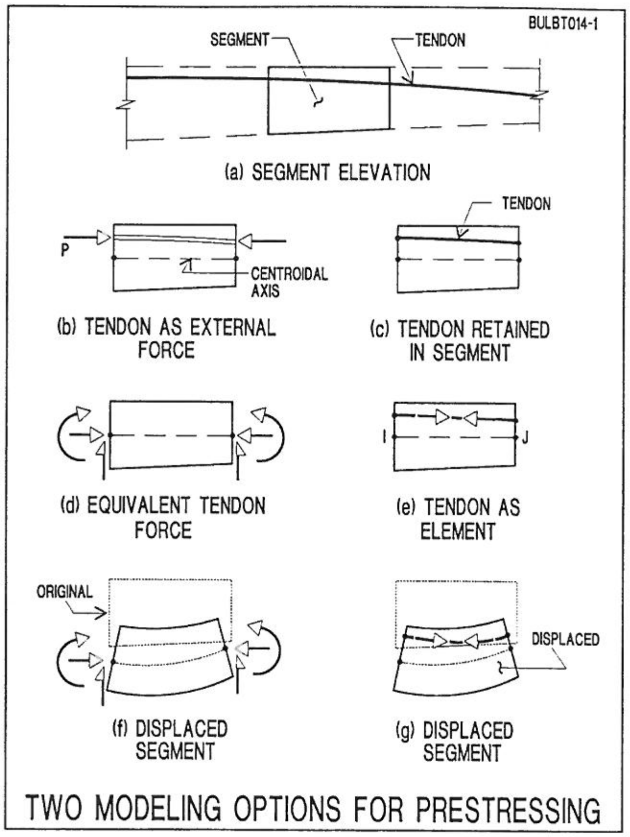


FIGURE 1.4.3-1

locked at its two ends on the opposing faces of the concrete segment in which it is housed. At the time the tendon is assumed locked to the faces of the concrete segment, the tendon force is assume to be that obtained from the friction loss computation at stressing and the elastic shortening of the concrete segment. Any subsequent deformation of the concrete segment, such as shown in part (g) of the figure, will be accompanied by a compatible displacement of the tendon element. The displacement of the tendon ends will result in a change in the tendon force. It is observed that with this scheme of modeling, there is an implicit interaction between the displacement of the concrete segment and the force in tendon. Therefore, the change in prestressing force is automatically accounted for, irrespective of the cause of displacement of the concrete segment

1.4.4 Elimination of Transformed Section Consideration for Reinforcement

Similar to the tendon representation described in the preceding section, the nonprestressed reinforcement can also be represented either as individual bars (like tendons), or as an equivalent material uniformly distributed over the cross section (transformed section). The transformed section is eliminated in the current analysis and the deformations and stresses are determined on the basis of the combined stiffness of the two materials, concrete and steel. The share of each material in resisting the applied loading is governed, among other factors, by the modulus of elasticity of that material at the time the loading is applied (see figure 1.4.2-1).

2 - CALIFORNIA BULB-T BRIDGE GIRDER DESIGN EXAMPLE

The bridge selected for the design example is a typical two-span, four lane freeway overpass constructed with precast-prestressed girders, spliced at 0.23 times span length on each side of the central support. Post-tensioning continuity tendons, together with a cast-in-place topping slab integrate the precast girders into a continuum, forming a two span continuous bridge. The girder splice and the bent cap are designed to provide full continuity of the members. The bent cap design and corresponding details follow recommendations for seismic design intended to ensure ductility and construction equivalent with conventional construction.

3 - GEOMETRY AND REINFORCEMENT

3.1 Geometry

The overall geometry and specific details implemented in the example are presented in this section.

3.1.1 Overall Geometry

The elevation and plan views of the bridge, including the overall geometry of the structure, are shown in Fig. 3.1.1-1. Each of the two equal spans is 43 m. The central column is assumed 8 m from the top of footing to the soffitt of the bent cap segment. The connection of the central pier to the bent cap, as well as the connection at the base of the central pier are fixed. The end supports at the two abutments are rollers in the longitudinal direction. At these two locations, horizontal movement is free, but vertical displacement is restrained.

The transverse cross-section of the bridge is illustrated in Fig. 3.1.1-2, showing the four precast girders spaced 3.317 m apart below the 13 m cast-in-place bridge deck. The round central column is 1.85 m in diameter (Fig. 3.1.1-3). The precast (PC) girders are supported over the cast-in-place bent cap, which extends 150 mm below the soffit of the girders. Five tendons along the bent cap, passing through holes in the PC girders (see figure 3.1.2-3), provide additional strength for the bent cap in supporting the girders.

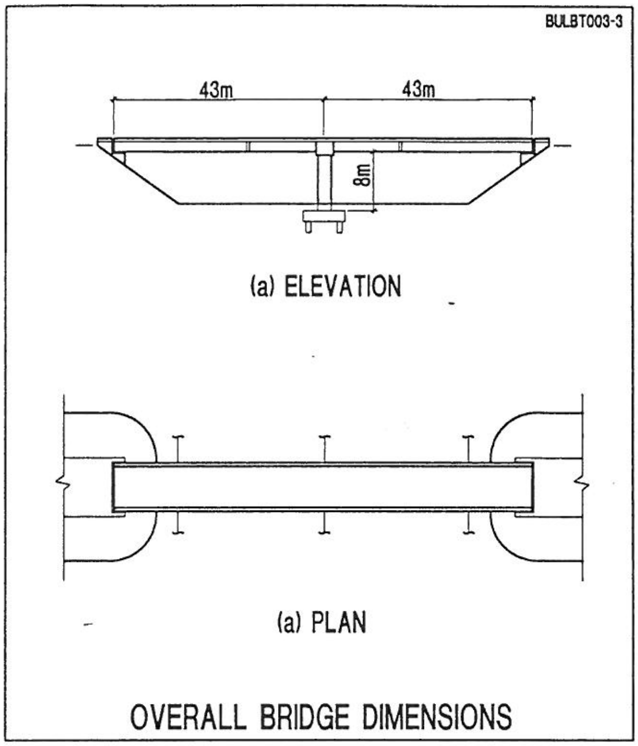


FIGURE 3.1.1-1

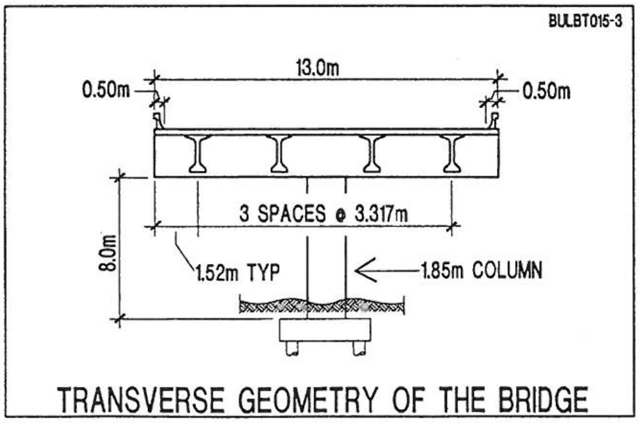


FIGURE 3.1.1-2

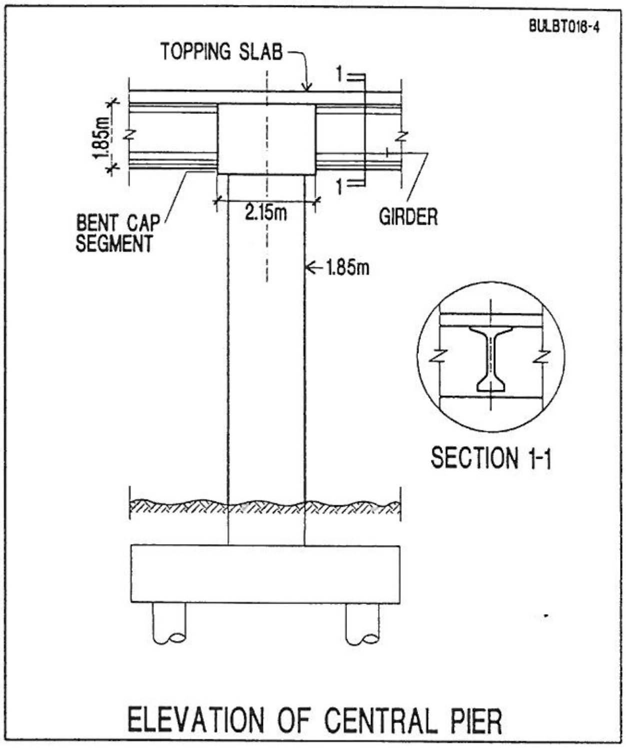


FIGURE 3.1.1-3

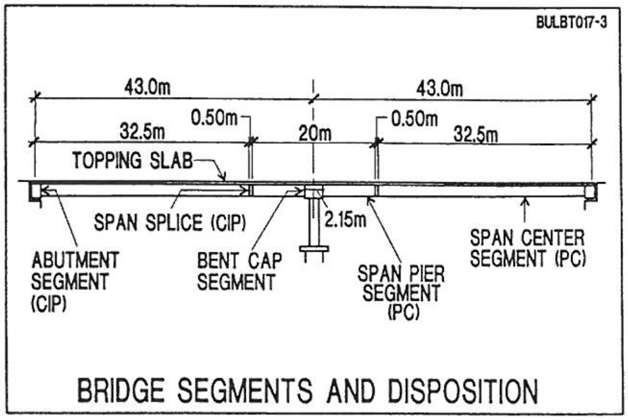


FIGURE 3.1.2-1

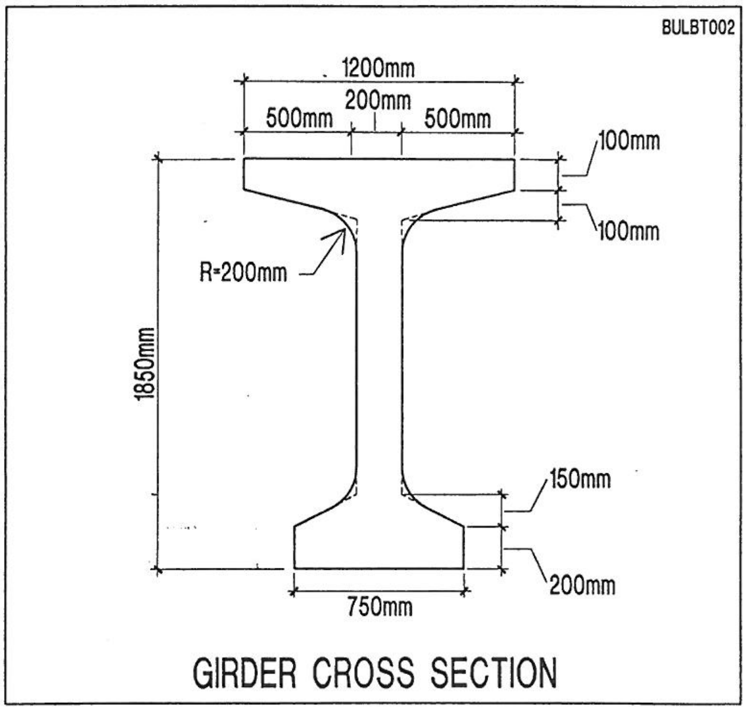


FIGURE 3.1.2-2

3.1.2 Segment Geometry

The components of the bridge used for the segmental analysis, and their disposition in the completed structure, are shown in Fig. 3.1.2-1. These are:

- Precast girder span center segments
- Precast girder span pier segment
- CIP span splices
- CIP bent cap segment
- CIP abutment segments
- CIP topping slab

There is no diaphragm connecting the stems of the girders in the transverse direction, except the bent cap and the abutment segments.

The cross-sectional geometry of the precast segments is illustrated in Fig. 3.1.2-2. The segments are of constant cross-section. There is no enlargement at the abutments. The stressing hardware for the post-tensioned tendons are placed within the cast-in-place segment of the abutment as described further on.

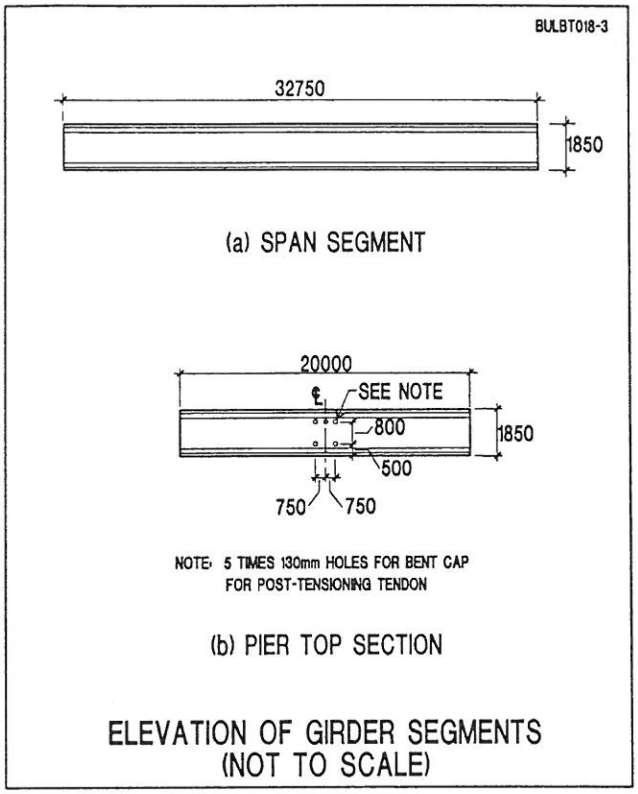


FIGURE 3.1.2-3

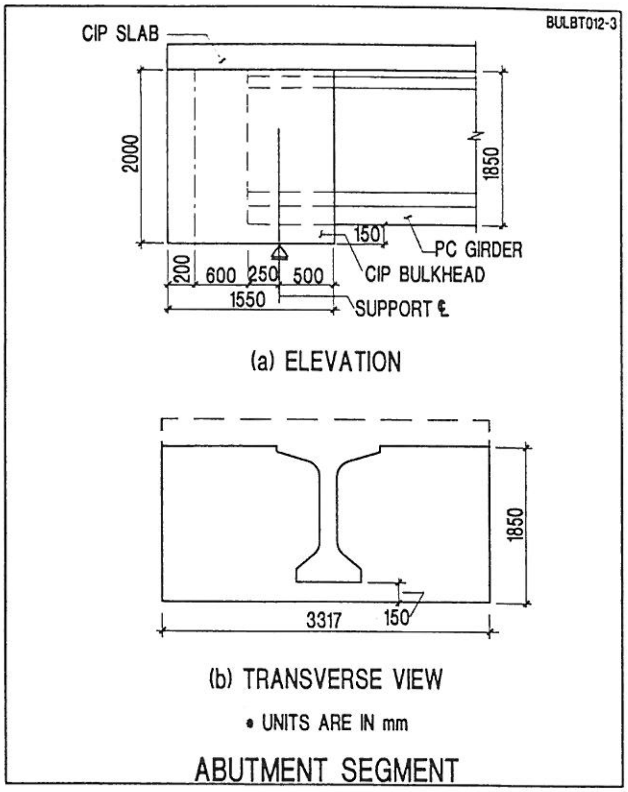


FIGURE 3.1.2-4

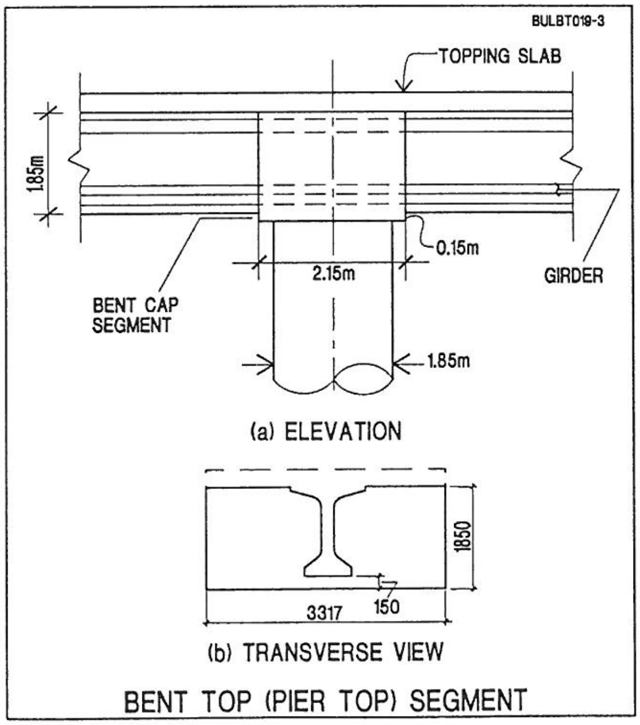


FIGURE 3.1.2-5

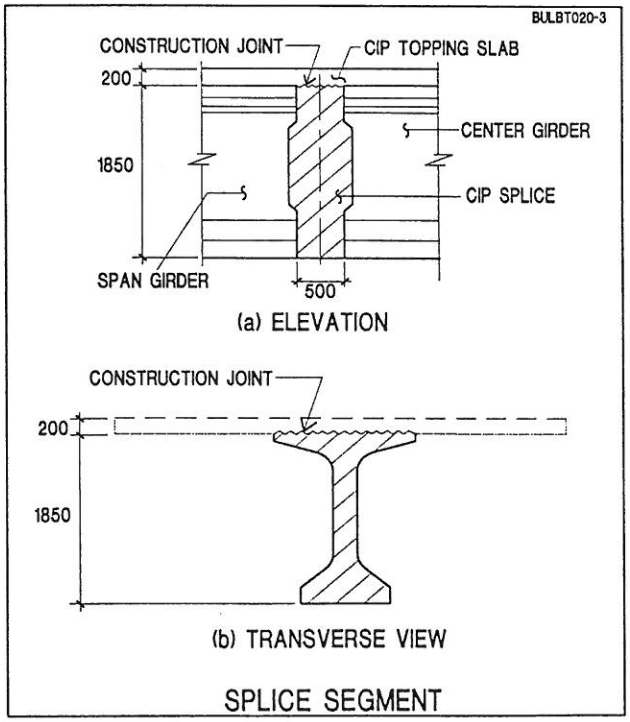


FIGURE 3.1.2-6

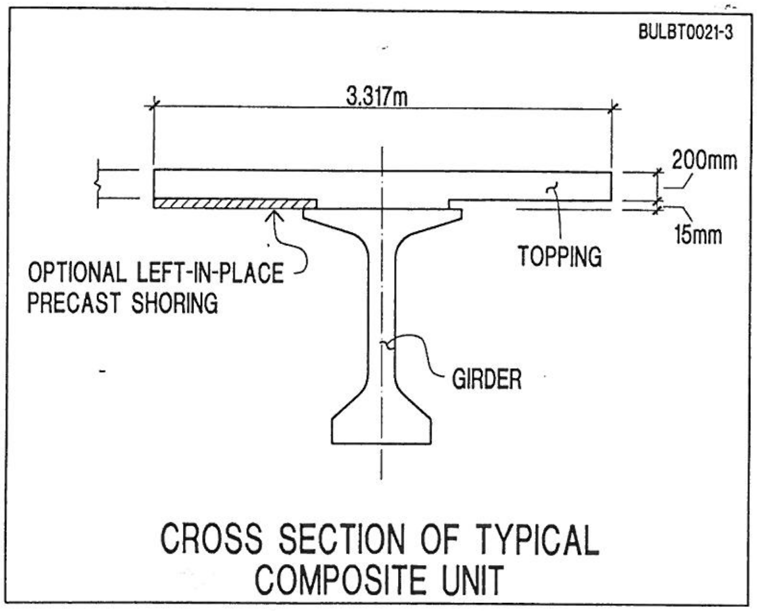


FIGURE 3.1.2-7

Fig. 3.1.2-3 shows the overall dimensions of the span and pier girder segments. The segments are of uniform cross-section. Five holes at the mid-length of the pier segment are used to install the post-tensioning in the bent cap.

The geometries of the abutment segments and the bent cap segment are shown in Figs. 3.1.2-4 and 3.1.2-5. Note that the transverse view of these sections, showing a width of 3317 mm, represents one of the four girders with its associated tributary. In the transverse view of these segments the void represents the PC girder. In modeling the abutment and bent cap segments, the cross-sectional area used is the one shown in part (b) of the figures, in which the region occupied by the girder is deducted. Further, note in Fig. 3.1.2-4 that the abutment segment consists of a solid (end of the bridge) and a hollow region (over the support).

The geometry of the splice segment is shown in Fig. 3.1.2-6. Note that the cross-section of the splice is the same as that of the precast girder. Experimental studies performed at the University of California in San Diego [Holomb, et. al., 1997] have concluded that the provision of a diaphragm at the splice does not enhance the seismic performance of the bridge, and that a design for satisfactory performance can be achieved without a diaphragm.

A representative unit of the bridge, consisting of a single PC girder and its associated topping is illustrated in Fig. 3.1.2-7. This representative unit is modeled for the design example.

3.1.3 Prestressing Amount and Geometry

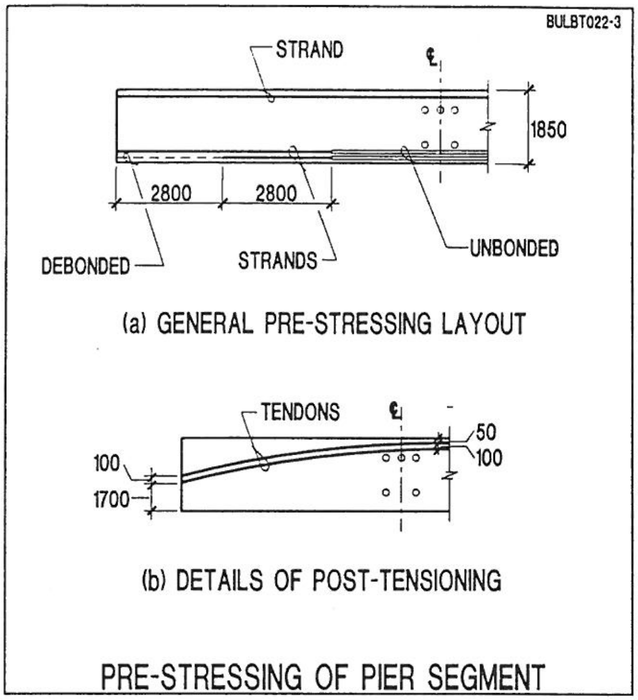
The prestressing of the bridge consists of pretensioning strands and post-tensioning tendons. The amount and layout of the longitudinal prestressing in the bridge is shown in Figs. 3.1.3-1 through 3.1.3-4.

A. Pretensioning

The pretensioning strands are of three types. Some are continuous through the length of the segment and remain in tact after the completion of the structure, to supplement the post-tensioning in resisting the applied loading. Some are debonded at the ends of the segments, in order to control the stresses at regions of low prestressing demand. In the pier segment, the prestressing at the bottom is used primarily for handling and erection. Once the girder is installed, it will be neutralized. For this reason, it is wrapped at center and is modeled as an unbonded tendon.

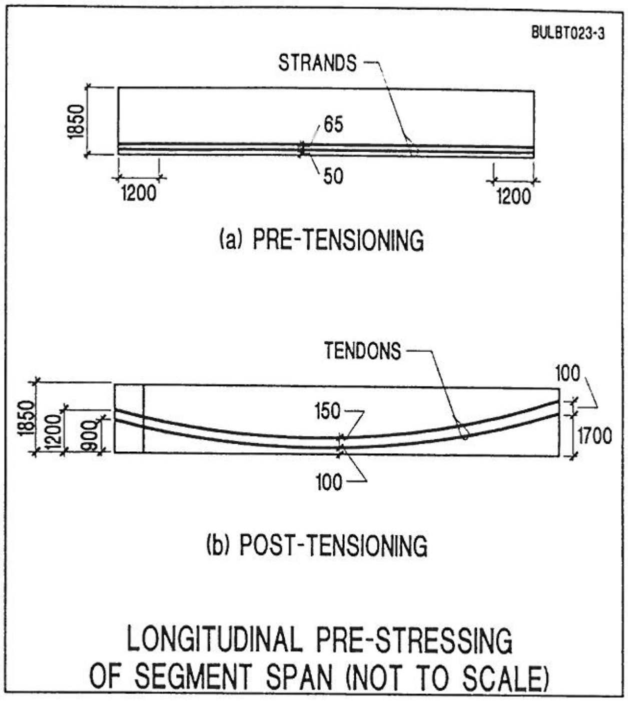
B. Post-Tensioning

There are two post-tensioning tendons. These are stressed at two stages during the construction as described further on. The geometry of the post-tensioning tendons is reversed parabola over the central column and simple parabola from its low points to the abutments. Note that the stressing hardware and the associated reinforcement is placed beyond the end of the precast girders in the abutment segment.



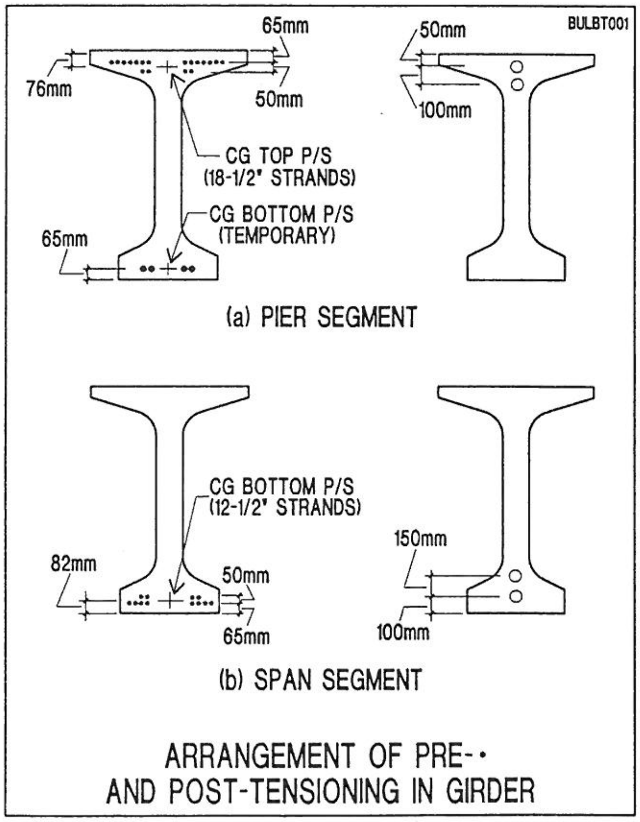
PRE-STRESSING OF PIER SEGMENT

FIGURE 3.1.3-1



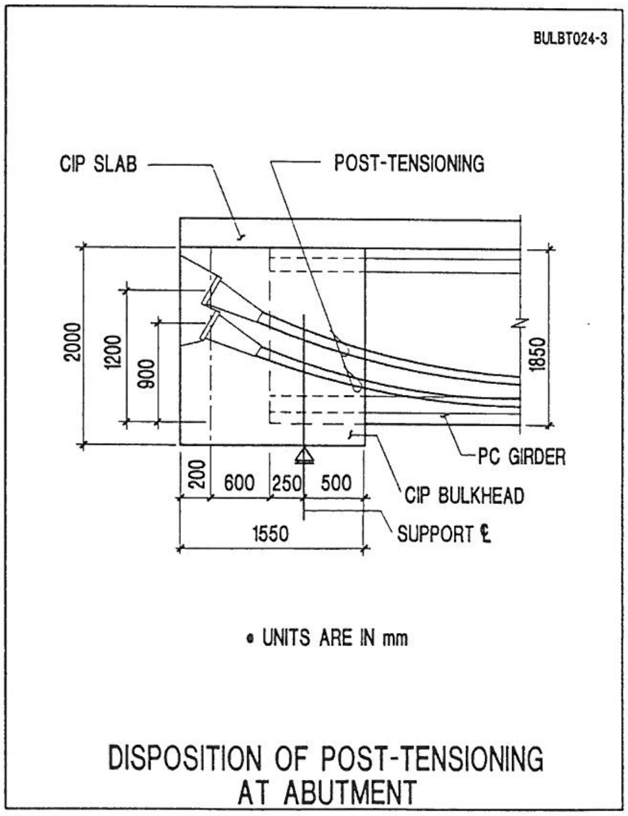
LONGITUDINAL PRE-STRESSING OF SEGMENT SPAN (NOT TO SCALE)

FIGURE 3.1.3-2



ARRANGEMENT OF PRE-• AND POST-TENSIONING IN GIRDER

FIGURE 3.1.3-3



DISPOSITION OF POST-TENSIONING AT ABUTMENT

FIGURE 3.1.3-4

4 - MATERIAL AND ENVIRONMENT

4.1 Concrete

4.1.1 Precast Girders

- Class of concrete = V
- f'c = 42 MPa
- f'ci (at release) = 35 MPa
- Unit weight = 2400 kg/m3
- Creep Characteristics = ACI 209
- Creep Coefficient = 3
- Shrinkage Characteristics = ACI 209
- Shrinkage Coefficient = 0.000278
- Thermal expansion = 10E-6 /degree Centigrade

4.1.2 Splice Concrete

- Class of concrete = V
- f'c = 28 MPa
- Unit weight = 2400 kg/m3
- Creep Characteristics = ACI 209
- Creep Coefficient = 3
- Shrinkage Characteristics = ACI 209
- Shrinkage Coefficient = 0.000278
- Thermal expansion = 10E-6 /degree Centigrade

4.1.3 Cast-in-Place Bridge Deck; Substructure; Bent Cap andAbutment Segment

- Class of concrete = V
- f'c = 28 MPa
- Unit weight = 2400 kg/m3
- Creep Characteristics = ACI 209
- Shrinkage Coefficient = 2.9
- Shrinkage Characteristics = ACI 209
- Shrinkage Coefficient = 0.00272
- Thermal expansion = 10E-6 /degree Centigrade

For the design example presented herein, the creep, shrinkage and aging of concrete are modeled using ACI 209. Other concrete modeling schemes, such as FIP/CEB and laboratory generated data can also be used. In particular, for high performance concrete, laboratory generated creep, shrinkage and aging values are recommended.

4.2 Nonprestressed Steel

4.2.1 Material

- Specification : ASTM A615M Grade 420
- Ultimate strength = 460 MPa
- Es = 200000 MPa

4.2.2 Amount

- Bulb Tee = 0.75% of gross area
- Splices = 2%
- Central pier = 2%
- Bent cap = 1.5%
- Topping slab = 1.0%
- Sheathing = 0% for unbonded tendons and ties

4.3 Prestressing Strands

4.3.1 Material

- 12.7 mm (1/2") nominal diameter strands; ASTM A416
- Ultimate strength (fpu) = 1860 MPa
- Yield stress (fpy) = 1675 MPa
- Es = 197000 MPa
- Area = 98.7 mm^2
- Type = Low relaxation
- Stressing = 0.75fpu (1400 MPa)

4.4 Post-tensioning Tendons

4.4.1 Material

- 15.2 mm nominal diameter strands: ASTM A416
- Ultimate strength (fpu) = 1860 MPa
- Yield stress (fpy) = 1675 MPa
- Es = 197000 MPa
- Area = 140 mm^2

- Type = Low relaxation

4.4.2 Tendons

- Two tendons, designated as T1, T2
- Stands per tendon = 12 strands each 15.2 mm dia
- Duct (galvanized) = 83 mm outside diameter
- Jacking force = 0.80 fpu (1490 MPa)
- Stressing = Both end
- Anchor set = 6 mm
- Friction coefficients:
  - Wobble K = $0.66\text{x}10^{-6}$ /mm
  - Angular μ = 0.25 /radian

4.5 Environment

- Relative humidity = 75%
- Ambient temperature = 20 degrees Centigrade
- Temperature rise and fall = + - 22 degrees Centigrade

5 - CONSTRUCTION

5.1 Construction Scheme

5.1.1 Precast Prestressed Girders

The precast prestressed girders are cast, stressed, cured, stored, and transported to the site for erection. These events are modeled by the program. As a result, at the time of erection, the girders have locked-in stresses associated with the history of their construction and loading.

5.1.2 Erection Sequence

Figures 5.1.2-1 and 5.1.2-2 show the schematics of the erection sequence. The erection sequence is broken into 9 stages . The construction schedule is outlined in Table 5.1.2-1.

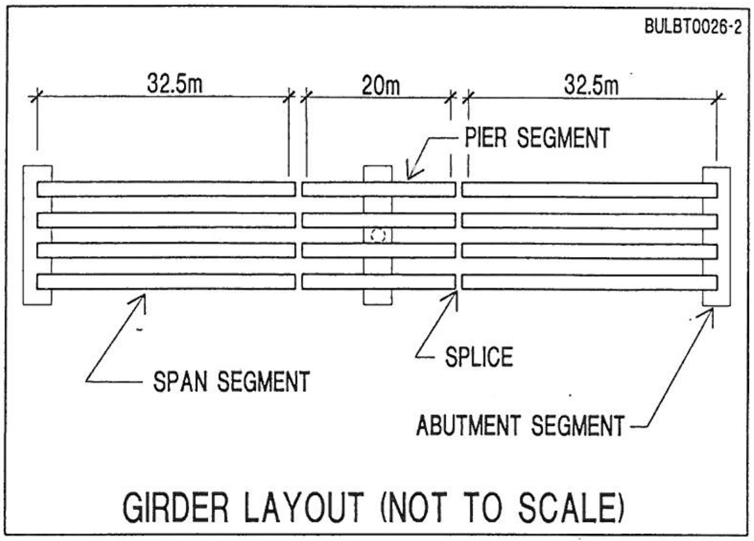


FIGURE 5.1.2-1

Stage 1	Construct central column and abutments
Stage 2	Erect temporary support on each side of the central column; Center piersegments over ground support
Stage 3	Cast bend cap, cut unbonded bottom strands and remove ground support
Stage 4	Install erection tie and stress; Position span segment on support bracket and install span segment
Stage 5	Position the second span segment; Remove tie
Stage 6	Secure strong backs over the splice segments; Cast the splice segments of both spans; Cast abutment segments
Stage 7	Install continuity tendons andstress first stage tendons
Stage 8	Cast topping slab
Stage 9	Stress remainder of continuity tendons
Stage 10	Add railing

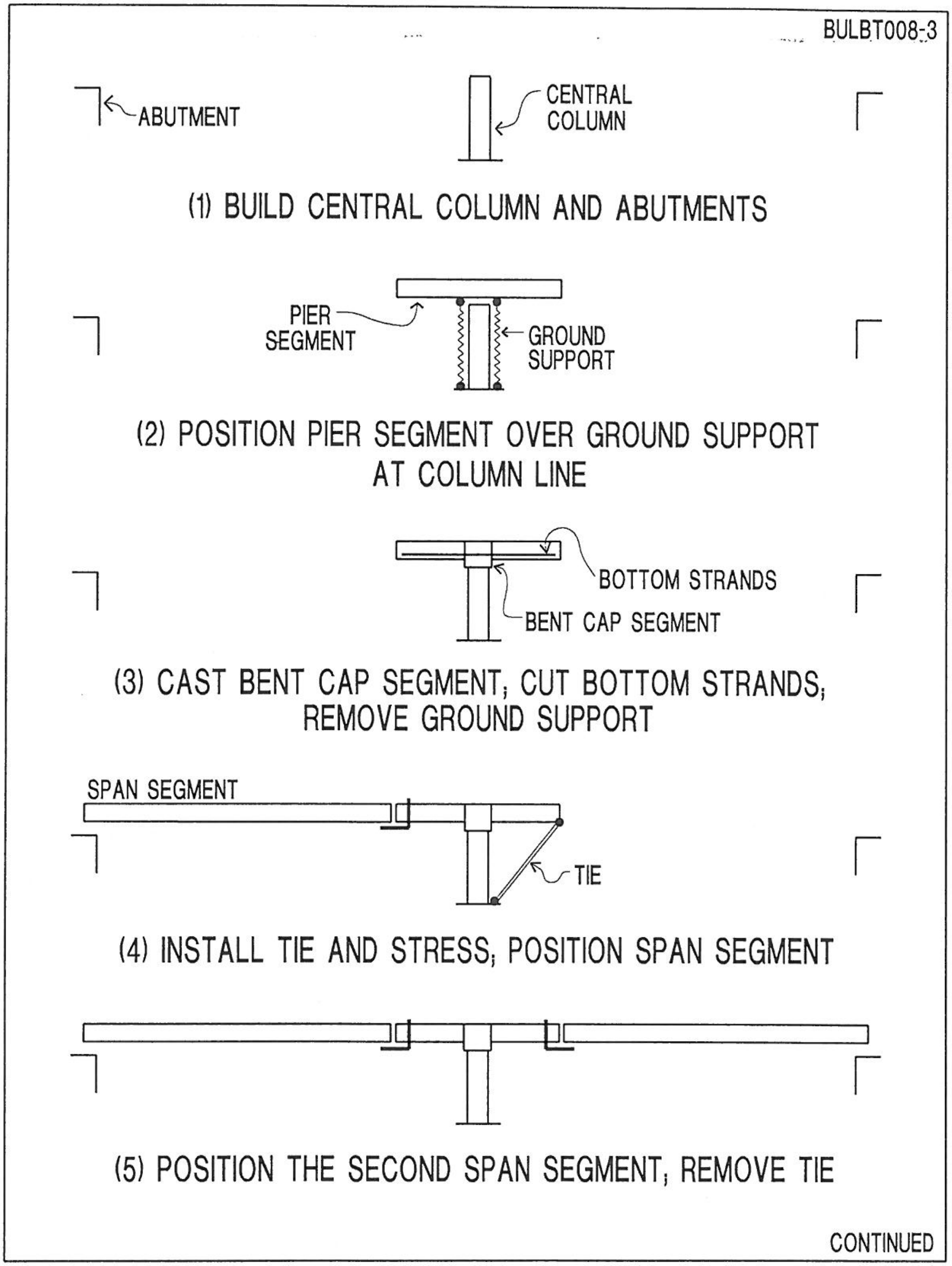


FIGURE 5.1.2-2(a)

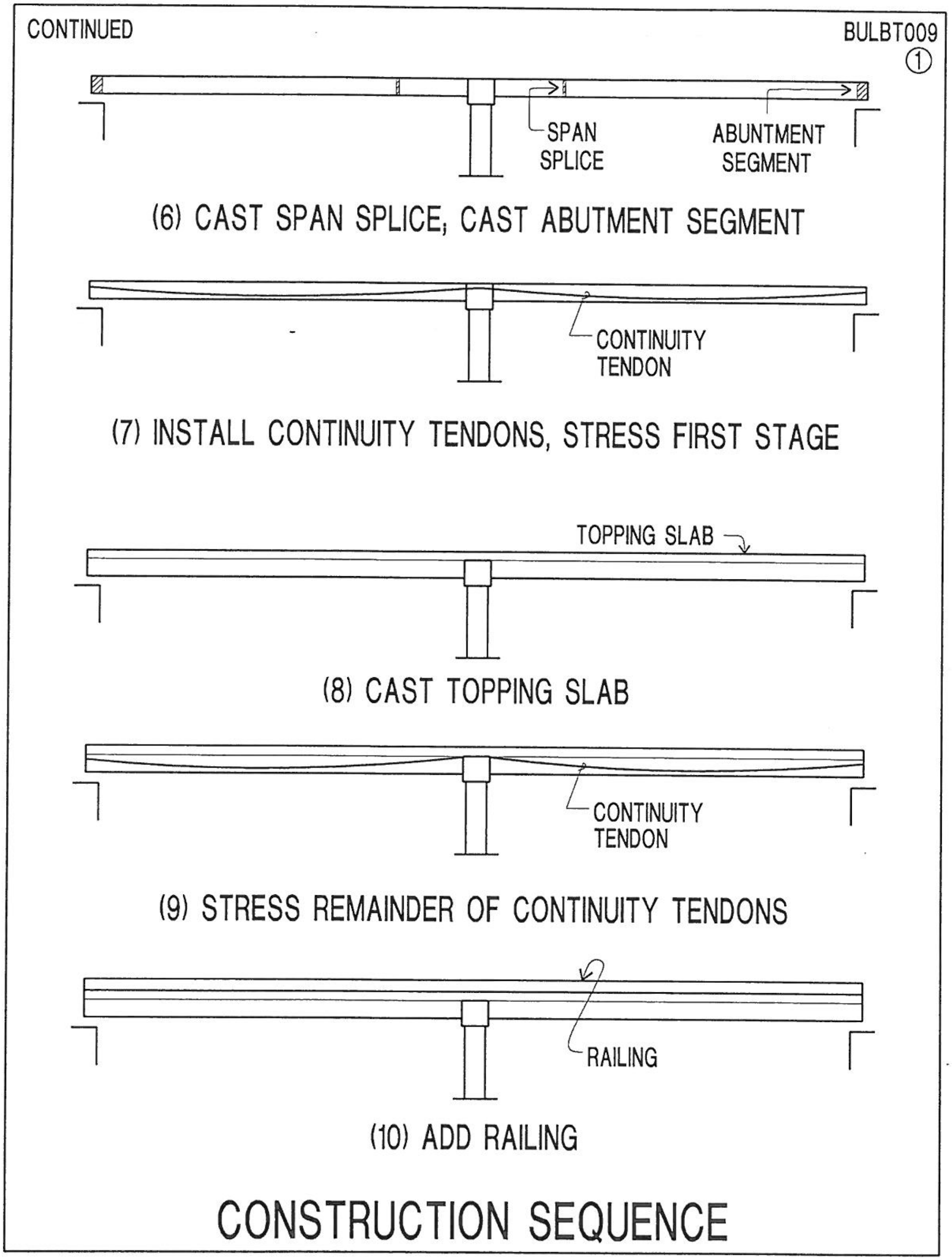


FIGURE 5.1.2-2(b)

TABLE 5.1.2-1 TIMETABLE AND SCHEDULE OF CONSTRUCTION

Event	Action	From Day	To Day	Duration(Days)	Comment
1	Stress prestressing tendons	1	1	0	
2	Cast concrete of girders	1	1	1	(1)
3	Release strands	2	2	1	
4	Remove girders from bed	4	4	1	(2)
5	Store girders in yard	5	18	14	(3)
6	Cast footings, column and abutments	-20	1	21	
7	Transport and position pier segment over ground support	19	28	10	
8	Cast pier top segment (bent cap)	29	38	10	
9	Stress transverse tendons in bent cap, remove shoring	39	41	3	(4)
10	Cut bottom strands of pier segment	42	42	1	
11	Install tie on one side and stress	42	42	0	(5)
12	Secure span segment in position on span with no tie	43	44	2	(6)
13	Position the second side's span segment	45	46	2	
14	Place tendon duct splices through the span splice and abutment segment	47	48	2	(7)
15	Cast span splice and abutment segments	49	55	7	
16	Place the continuity post-tensioned tendons; stress stage 1	56	57	2	
17	Place topping slab	58	72	15	
18	Stress the remainder of continuity tendons	73	73	1	
19	Add railing and other superimposed dead load	74	83	10	
20	Add live load	84	84	1	
21	Observe structure after 20 years	84	7300	7216	

Comments:

(1) Due to different pretensioning arrangements, span and pier segments will be cast separately

(2) Girders will be supported at two points, at a position to be determined. In the design example supports are assumed to be 600 mm from each end

(3) The design example assumes that the column is fixed to the foundation at the bottom. The footing and pile cap are not modeled.

(4) The prestressing in the bent cap transverse to the girders is not modeled. Its stressing shall be coordinated with the removal of the supports of the center girders.

5.1.3 Construction Equipment

A. Temporary Ties

Refer to Fig. 5.1.2-2. Prior to the placement of the first span segment, tension ties are installed between the free tip of the center segment and the footing. Four - 12.7 mm strands are used. These are stressed to 1400 Mpa providing a total of 550 kN of force. The provided force in the ties relieves the stress in the central column due to the weight of the span segment on one side only. Once the second span segment is installed, the ties are neutralized.

B. Support Brackets

Refer to Fig. 5.1.2-2. The support brackets are used to transfer the weight of the span segments to the center segment prior to construction of the splices. These members are assumed as rigid attachments, capable of transferring the load to the tip of the center segment without deformations of their own.

C. Strong Backs

Prior to the pouring the splice segments, the strong backs (not shown in Fig. 5.1.2-2) by way of their large stiffness, retain the relative position of the adjacent tips of the span segment and the pier segment, until the splice concrete cures, the first post-tensioning is applied and the splice is capable of providing the intended continuity. Therefore, the strong backs are modeled as rigid connections. Their weight is not included in the example presented herein. The weights, if significant, should be included in the analysis.

Other construction equipment used is assumed to be either ground supported, or to have no significant impact on the stress and displacement history of the bridge.

6 - DESIGN CRITERIA

The design is based on:

- Maintaining the stresses within permissible limits under specified loading scenarios during the construction and when complete;
- Providing adequate strength safety in the event of overload under specified overload values and load combinations

6.1 Loads

6.1.1 Dead Load

Selfweight of concrete	= 22.60 kN/m3
Superimposed dead loading	= 1,320 N/m2
Traffic barriers (2 barriers among four girders)	= 2.86 kN/m

6.1.2 Erection and Transportation Load

25% of self weight

6.1.3 Temperature Forces

Temperature rise and fall = 22 degrees centigrade

6.1.4 Live Load (not included in the design example)

HS-20-44 for service
P-Loading for ultimate

6.1.5 Seismic Loading (not included in the design example)

Zone IV seismic loading

6.2 Allowable Stresses

6.2.1 Code Stipulated Values

The allowable stress limits are listed in Table 6.2.1 in terms of the concrete strength f'c and the code stipulated load combinations

TABLE 6.2.1-1 ALLOWABLE STRESS LIMITS

Stress Type	Temporary (*Before long-term losses due to creep and shrinkage*)		Final (*After long-term losses due to creep and shrinkage*)	
	Release Strength (f'_{ci})	28-Day Strength (f'_c)	Precompressed Tensile Zone	Other Areas
Compression:				
Case I	$0.6f'_{ci}$	$0.6f'_c$	$0.6f'_c$ (1)	$0.6f'_c$ (1)
Cases II & III	$0.6f'_{ci}$	$0.6f'_c$	$0.4f'_c$	$0.4f'_c$
Tension:				
DL Only	$0.63\sqrt{f'_{ci}}$ (2)	$0.63\sqrt{f'_c}$ (2)	0.00	$0.50\sqrt{f'_c}$ (3)
DL + Other Loads	$0.63\sqrt{f'_{ci}}$ (2)	$0.63\sqrt{f'_c}$ (2)	$0.50\sqrt{f'_c}$ (3)	$0.50\sqrt{f'_c}$ (3)

(1) The allowable value per Caltrans Bridge Design Specifications is $0.4f'_c$. But the most recent releases of the PCI Handbook and the AASHTO Standard Specifications permit $0.6f'_c$. Cases I, II and III are defined as follows:

Case I.	All Load Groups except as stated in Cases II and III below.
Case II.	Prestressing Force + Permanent DL
Case III.	Live Load + 0.5(PS + DL)

(2) This value is the maximum allowable tension. However, if the tension stress exceeds the smaller of 1.38 MPa and $0.25\sqrt{f'_c}$, bonded reinforcement should be provided to resist the tension in the concrete computed based on uncracked section.

(3) Assumes bonded reinforcement, including prestressed strands.

6.2.2 Allowable Stress Values for the Design Example

For the specific condition of the design example presented herein the allowable stresses are listed in Table 6.2.2-1.

TABLE 6.2.2-1 ALLOWABLE STRESSES FOR THE DESIGN EXAMPLE

CIP	f'c	PC Girders		CIP Segments		Stage
		Tension	**Compression**	**Tension**	**Compression**	
DL	f'ci=35	3.75	21			Construction
DL	f'c= 42	4.00	25			Construction
DL	f'ci=21			2.75	12.5	Construction
DL	f'c=28			3.25	16.75	Construction
Service DL	f'c=42	0	25			Service
	f'c=28			0	12.5	Service
DL plus other loads	f'c=42	3.25	25			Service
	f'c=28			2.50	12.5	

6.3 Load Combinations

Since the focus of the design example presented is the segmental aspect of the analysis and the treatment of time-dependent effects until 20 years, the load combinations listed in AASHTO [ ] for segmental construction are not recited. These are load combinations for the following three cases.

- Construction phase
- Service Load Conditions
- Load Factor Design Conditions

It is important to note that when dealing with the service load conditions for stress and deflection computations, the Dead Load (DL) shall be substituted by (DL+PS). Where, PS is the actions due to prestressing. The impact of time dependent parameters, such as creep and shrinkage should be lumped with DL when calculating the service stress and deflections.

In the load factor design combinations for strength check, Dead Load (DL) must be substituted by (DL+HP). Where, HP is the hyperstatic actions due to prestressing. Hyperstatic actions are also referred to as "secondary actions." The impact of time dependent parameters, such as creep and shrinkage must also be included as part of DL, if such effects are not evaluated independently. If evaluated independently, they must be accounted for in the computations according to the factors specified in the code [AASHTO]

7 - Analytical Modeling

7.1 Modeling assumptions

7.1.1 Bridge Components

The components of the bridge for the segmental analysis, and their disposition in the completed structure, are shown in Fig. 3.1.2-1. These are:

- Precast girder span center segments
- Precast girder span center segment
- CIP span splices
- CIP bent cap segment
- CIP abutment segments
- CIP topping slab

Each of the above segments will be modeled and analyzed according to its own specifications and construction schedule. The railings are considered non-structural and treated as added weight.

In addition to the segment components of the bridge, the reinforcement features of the bridge to be modeled for the analysis are:

- Prestressing strands, with recognition of:
    - Debonded length,
    - Development length,
    - Unbonded lengths used for transportation and installation, and neutralized prior to completion of the bridge;
- Temporary shoring for support of the pier segments prior to their integration with the central column;
- Temporary supporting brackets at the ends of the pier segment to provide temporary support for the span segments;
- Installation and stressing of temporary ties to avoid overstressing of the central pier during the construction;
- Strong back over the splice region to allow for casting and curing of splices;
- Post-tensioning tendons, stage-stressed;
- Recognition of the added weight of the wet topping prior to its curing and prior to it becoming effective in sharing the load with the precast girders;

- Recognition of abrupt change in the position of the centroid of the bridge at the face of the bent cap and the abutment segments, and its influence on the internal actions and stresses in the bridge components.

7.1.2 Assumptions

Only one girder and its associated tributary is modeled. The tributary is assumed to be the full spacing between interior girders (3.317 m). Since the analysis accounts for the difference in the modulus of elasticity of new and old concrete, it is no longer required to determine the ration "n" and assume a transformed section.

The central pier is shared by four girders. In the analysis the entire pier cross-section is modeled, but only one quarter of its area and stiffness is included in the computations.

No allowance is made for shear lag, since the width of the topping slab relative to the span of the bridge is small. The impact of shear lag is considered to be negligible.

The ACI concrete model is used for the variation of creep and shrinkage with time. In practice, for high strength concrete, laboratory generated data might be more appropriate.

7.1.3 Discretization

The segments of the bridge are each discretized into a number of finite elements as reflected in the input data reproduced in Appendix A. Prestressing strands which are at the same level and have the same condition, such as debonding over the same length are bundled together and represented by one finite element. The cast-in-place topping is modeled separately with its own dimensions, concrete property and history of construction and loading. The changes in the centroidal axes and other features at the connection of the central pier to the bent cap and at the abutments are modeled using offset and master - slave features (see Fig. 7.1.3-1).

The debonded, temporary prestressing strands are placed in an essentially zero stiffness duct in the computer model. It is necessary to place the debonded tendons in a duct (modeled as an element), in order to avoid the interaction of the strand element and the precast girder over the debonded region.

The central pier is assumed fixed to the foundation at its base. The footing and pile cap are not modeled.

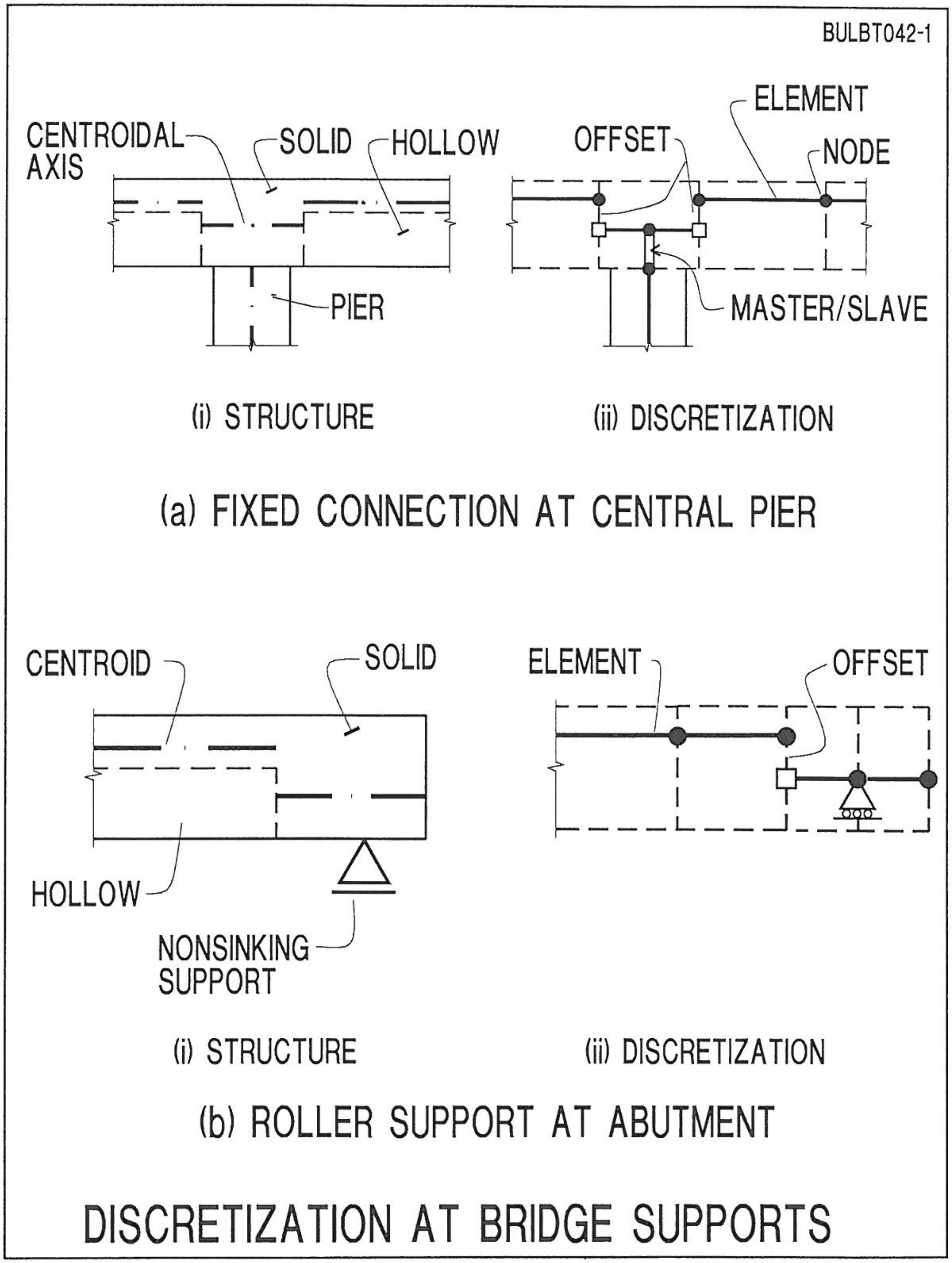

BULBT042-1
CENTROIDAL AXIS
SOLID
HOLLOW
PIER
OFFSET
ELEMENT
NODE
MASTER/SLAVE
(i) STRUCTURE
(ii) DISCRETIZATION
(a) FIXED CONNECTION AT CENTRAL PIER
CENTROID
SOLID
HOLLOW
NONSINKING SUPPORT
ELEMENT
OFFSET
(i) STRUCTURE
(ii) DISCRETIZATION
(b) ROLLER SUPPORT AT ABUTMENT
DISCRETIZATION AT BRIDGE SUPPORTS

FIGURE 7.1.3-1

Girders are supported at two points during storage, at a position to be determined. In the design example the girders are assumed supported at about 600 mm from each end while in storage.

7.1.4 Analytical Tools

The computations presented herein were carried out using the ADAPT-ABI version 2.50 computer program. This program has the capability of analyzing precast-prestressed concrete girders, that are spliced, post-tensioned and provided with a topping slab as included in this example. For the current example the computer model incorporated all important parameters related to segmental construction of the bridge. The program accounted for the composite action between the new and old concrete, as well as the influence of time dependent effects, such as aging of concrete, creep, shrinkage and relaxation in prestressing.

8 - INPUT DATA

8.1 Data Presentation

8.1.1 Tabulated Input Data

The tabulated input data is annotated and reproduced in Appendix A

8.1.2 Views from input

Computer screen views of the bulb-T design example are illustrated in the following pages. In most of the figures, either the vertical or the horizontal axis is exaggerated, in order to reveal specific details of the bridge.

The computer view of the elevation of the completed bridge is shown in Fig. 8.1.2-1. Fig. 8.1.2-2 is a presentation of the prestressing in the bridge. The vertical axis is exaggerated for clarity. Note that at the bottom of the central segment, the prestressing strands are removed in the completed structure. Figures 8.1.2-3 and 8.1.2-4 show the details of the modeling of the connections of the superstructure to the supports. The abutment segment extends beyond the end support and houses the stressing hardware of the post-tensioning tendons. The view of the splice for the completed bridge is shown in Fig. 8.1.2-5. The prestressing strands are shown terminating short of the face of the splice, due to the development length (1.2m) assumed for the prestressing strands in the computer modeling. The post-tensioning strands traverse the splice. An interim condition during the construction is reproduced in Fig. 8.1.2-6. At this

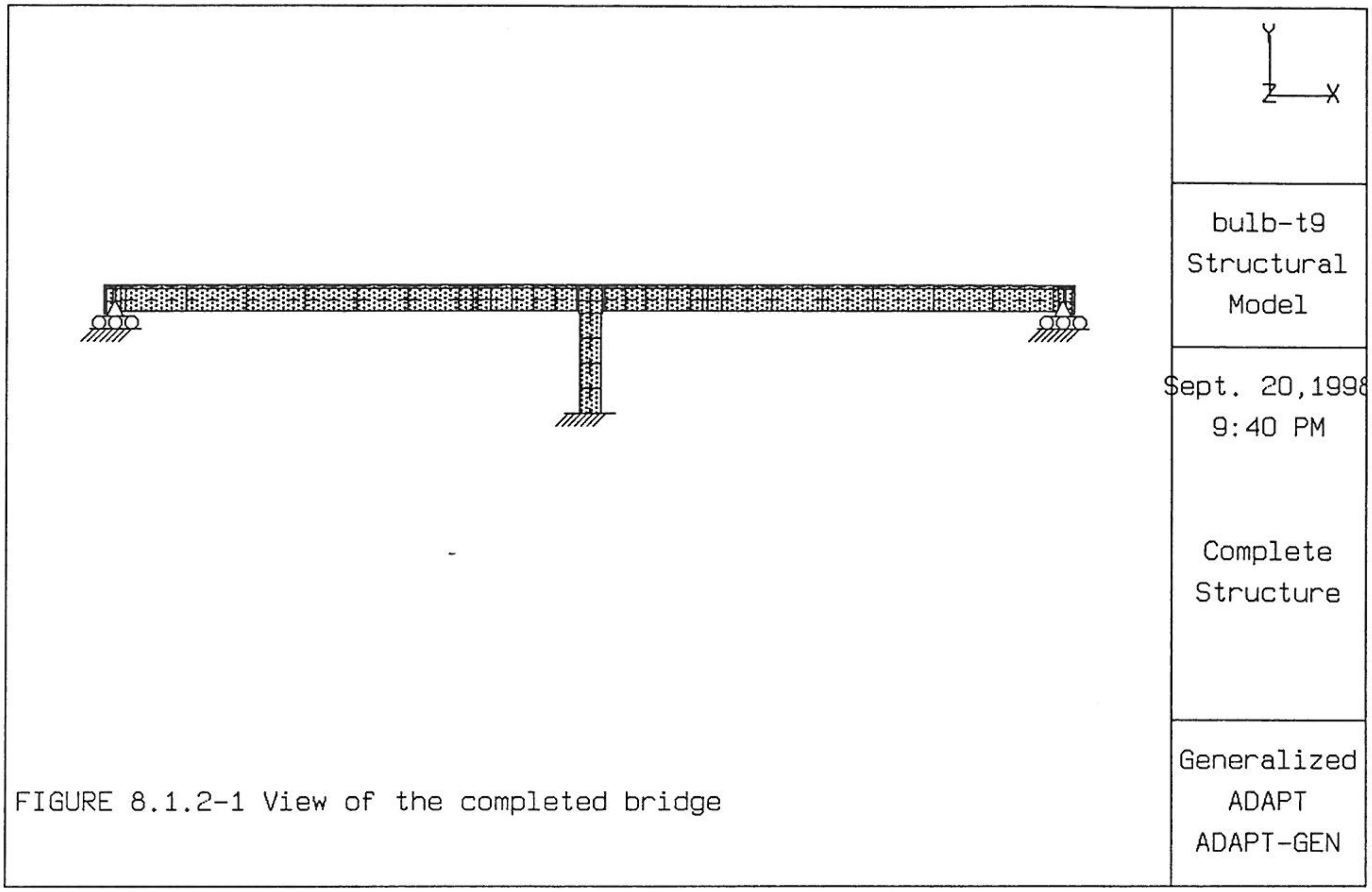


FIGURE 8.1.2-1 View of the completed bridge

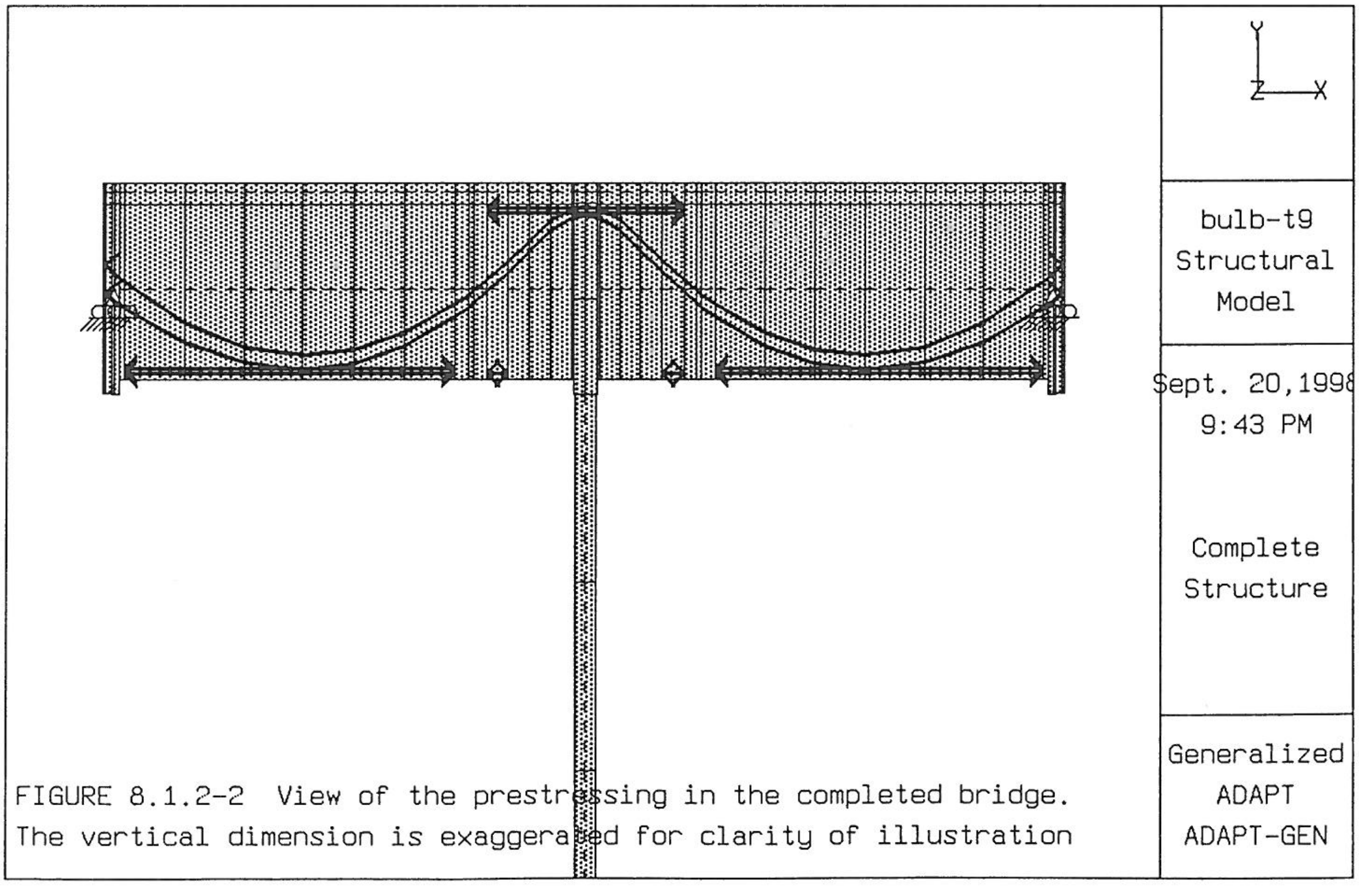


FIGURE 8.1.2-2 View of the prestressing in the completed bridge. The vertical dimension is exaggerated for clarity of illustration

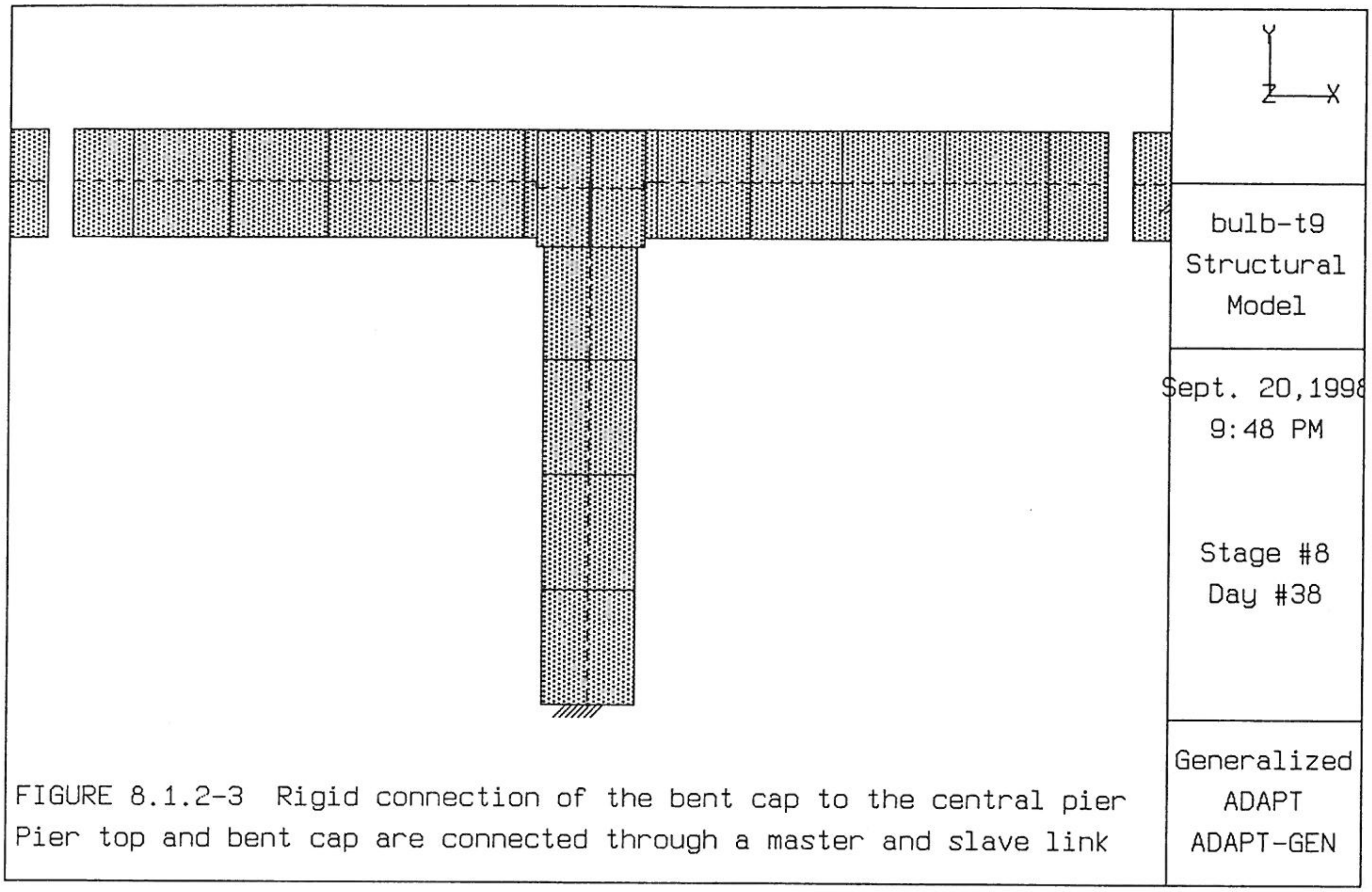


FIGURE 8.1.2-3 Rigid connection of the bent cap to the central pier
Pier top and bent cap are connected through a master and slave link

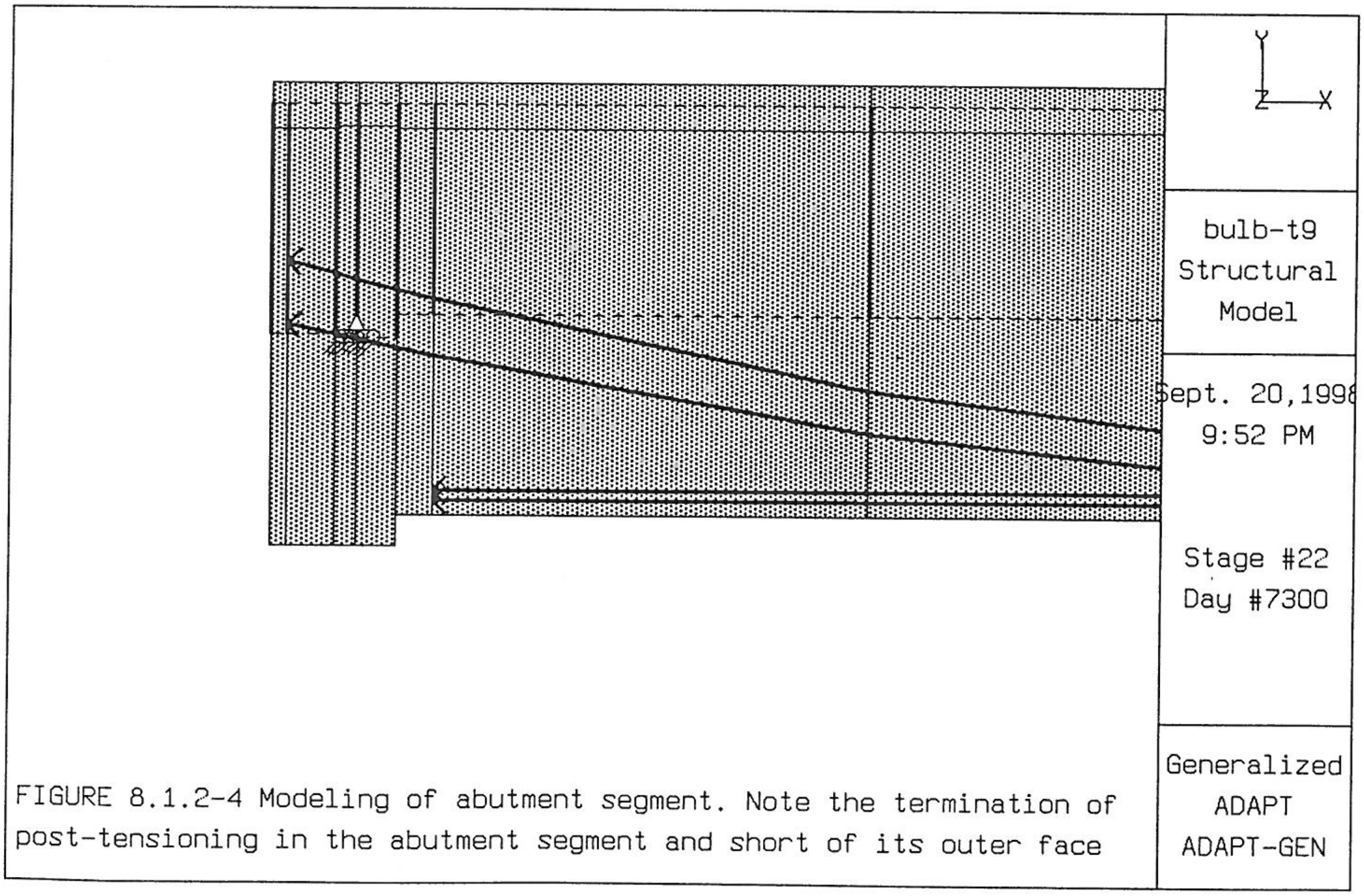


FIGURE 8.1.2-4 Modeling of abutment segment. Note the termination of post-tensioning in the abutment segment and short of its outer face

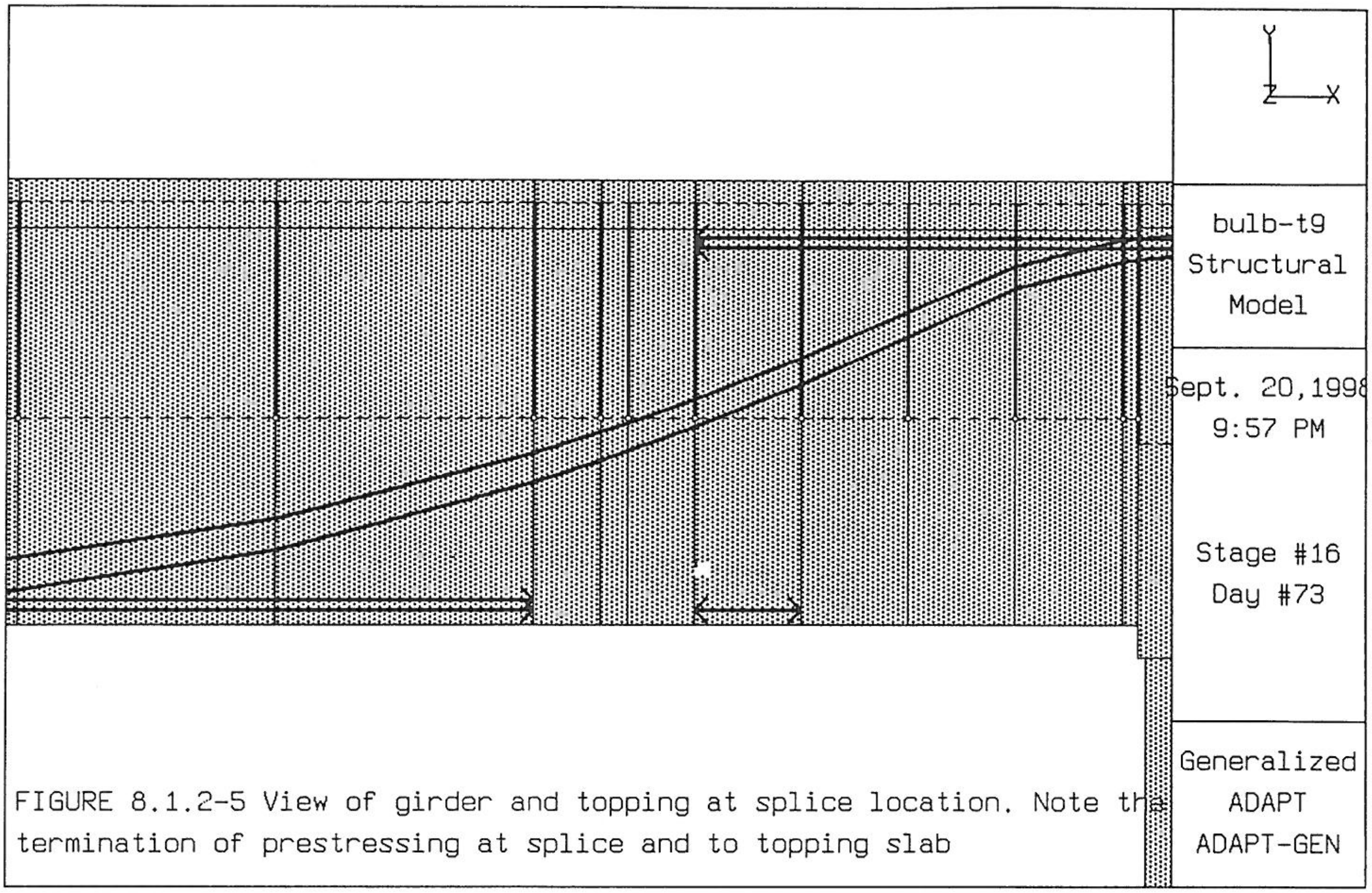


FIGURE 8.1.2-5 View of girder and topping at splice location. Note the termination of prestressing at splice and to topping slab

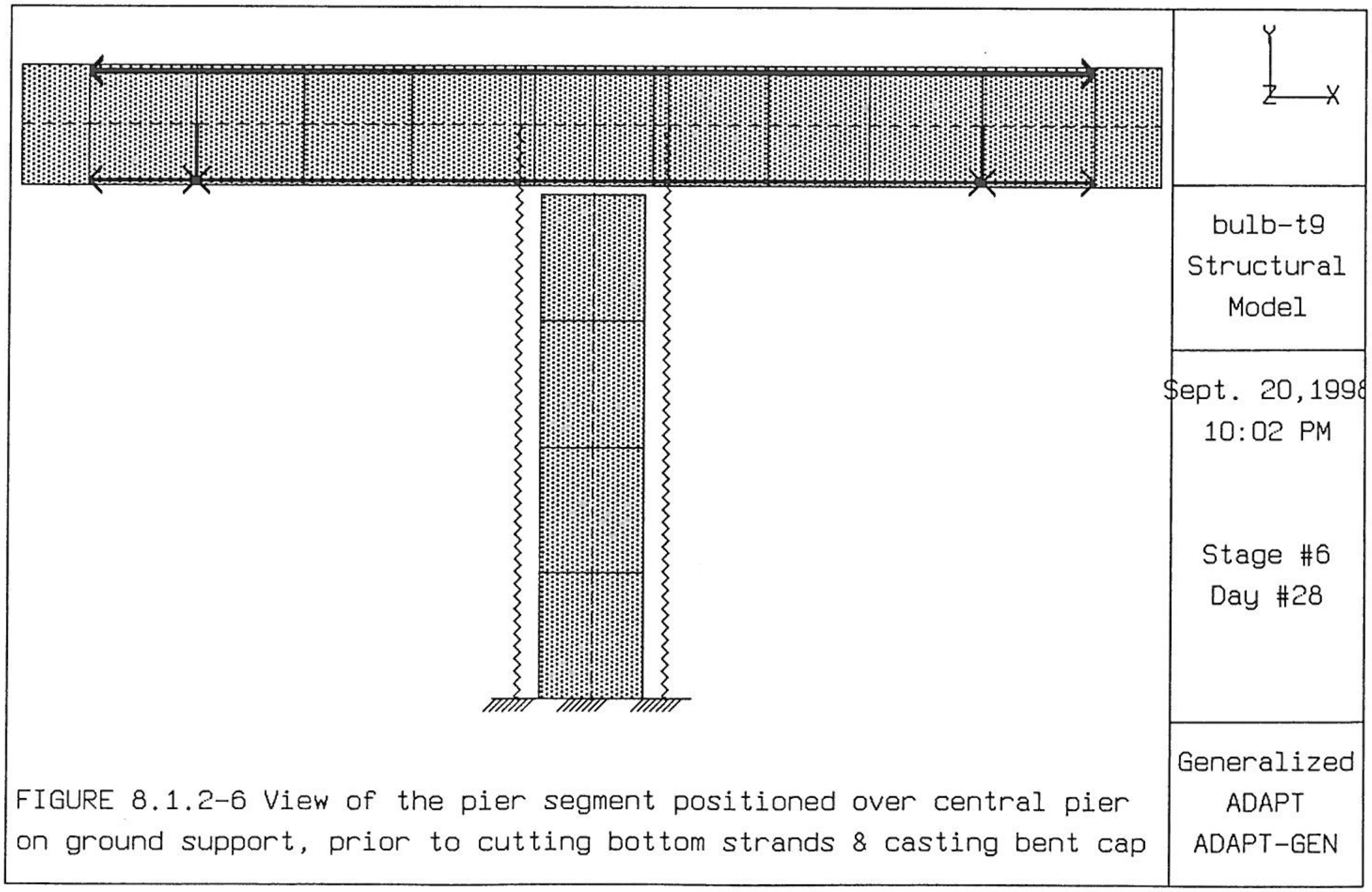
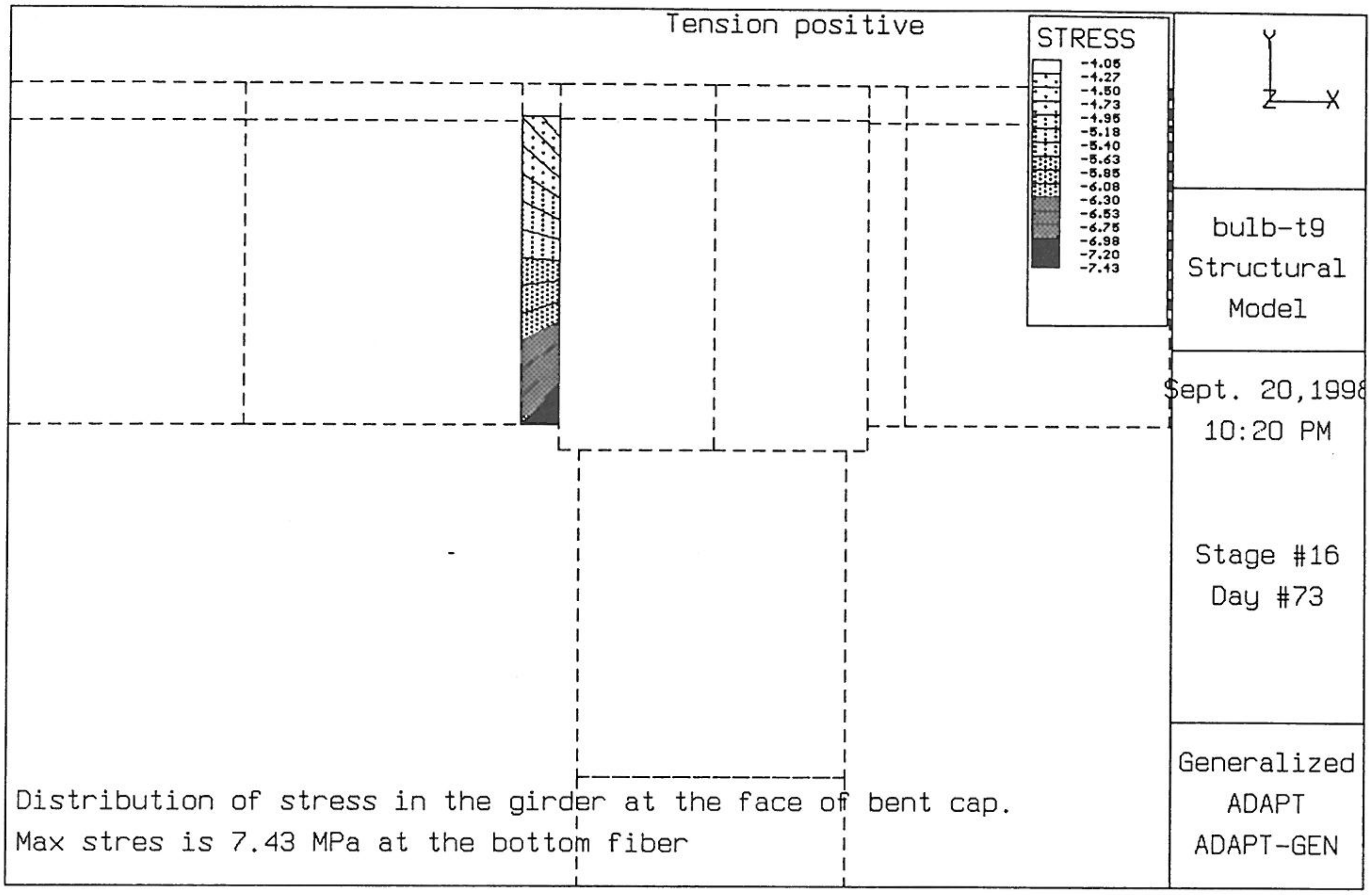
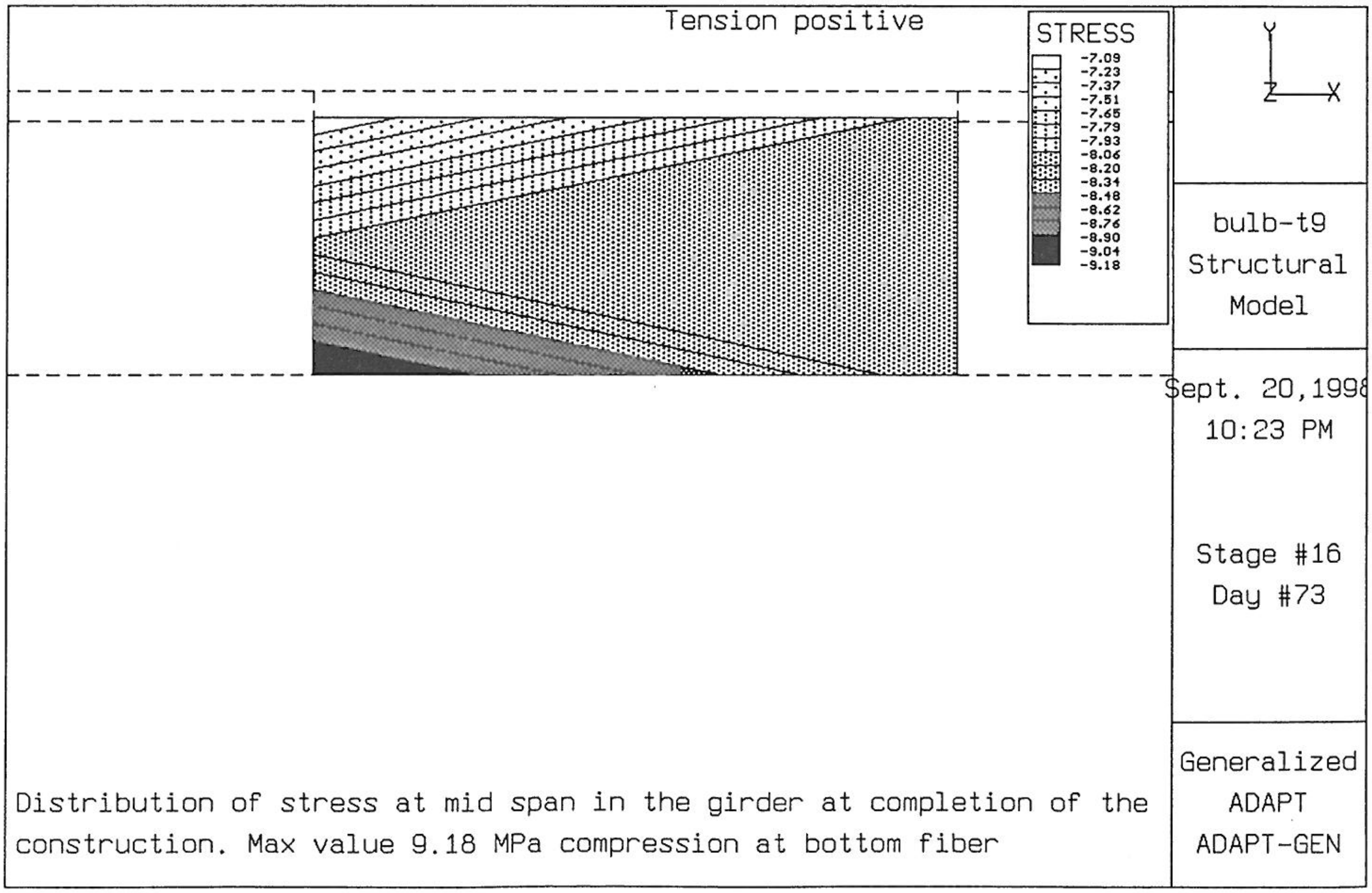


FIGURE 8.1.2-6 View of the pier segment positioned over central pier on ground support, prior to cutting bottom strands & casting bent cap



Distribution of stress in the girder at the face of bent cap.
Max stres is 7.43 MPa at the bottom fiber



Distribution of stress at mid span in the girder at completion of the construction. Max value 9.18 MPa compression at bottom fiber stage, the pier segment is held over the ground supports on each side of the central pier, prior to casting the bent cap, and cutting of the bottom prestressing strands. The distribution of stress in the girder at the face of the bent cap at the completion of the structure is shown in Fig. 8.1.2-7.

9 - SELECTED ANALYSIS RESULTS

9.1 Nature of Analysis Results

Like most other finite element programs, ADAPT-ABI generates a mound of output information, expressing the deformation, stresses and actions at all locations of the bridge and for each stage of construction. For expedient evaluation, the results can be grouped into four groups. They can be scanned, viewed and evaluated selectively for expediency in design. The four groups are:

- Information necessary to show that the design meets the requirements of the code, such as deformations and stresses. This is the information that would be reproduced and included in the submittals to the bridge authorities. It is very concise. For the design example given herein, this category of output is reproduced and discussed.
- Annotated input data: This is a re-print of input data which is broken into parts and commented extensively, with the objective to assist the plan checkers and reviewers in the validation process. The input data for the design example is reproduced in Appendix A.
- Figures reflecting the bridge response and behavior: Views and numerical values of the deformation, stresses and actions throughout the bridge during construction serve as a basic background to develop an understanding of the behavior of the structure and validate the solution. Many such graphs and views can be produced. These are not included in this report.
- Information used for validation of the solution: Data for equilibrium check, and comparisons of overall moments and shears for hand calculation are used for validation of the solution. These are not included in this report.
- The computation of the design example is outlined through discussions on modeling, tabulated input data, selected views of input data, output views / graphs necessary to illustrate the compliance of the design example with the code specifications, and information necessary for combination with live load conditions.

9.2 Presentation of Results

9.2.1 Code Compliance Results

The envelope of stresses at the top and bottom of the precast girders during the construction are shown in Fig. 9.2.1-1 along the length of the bridge. The diagrams also show the allowable stresses for construction phase. The stresses shown are due to the self weight, prestressing, long-term effects of creep, shrinkage, and stress relaxation, stress losses in prestressing, aging of concrete and construction equipment. Note that the stresses are well below the allowable values throughout the bridge. The actual stresses at any given condition are available from the analysis, but are not included in this paper. The stresses in the top slab are low and/or in compression. They are not included in the paper for lack of significance, but are available from the analysis.

Similarly, the stresses in the precast girders at 20 years are shown in Figure 9.2.1-2, together with the permissible values. Again, the computed stresses are well within the limits of allowable values.

For load combinations with live loading and for strength check, it is necessary to have the actions due to self weight (DL) and the prestressing (hyperstatic actions) expressed explicitly. These are obtained directly from the results of the analysis. For the condition of the bridge at 20 years, the moment due to self weight and the hyperstatic (secondary) moments are shown in Fig. 9.2.1-3. The hyperstatic moments presented herein include the effects of creep and shrinkage. If necessary, these can be extracted are presented separately.

9.2.2 Supplemental Output

This section includes selected output of interest for verification and appreciation of the behavior of the bridge.

The distribution of moments along the length of the bridge due to prestressing is shown in Fig. 9.2.2-1. The distribution shown includes allowance for all the time-dependent factors and loss in prestressing in both pre- and post-tensioning. These moments are to be added to the DL moments for serviceability stress checks. For strength checks, the hyperstatic moments shown in Fig. 9.2.1.3 are used.

The change in stress over time at selected points along the length of the bridge, and at selected times, is shown in Fig. 9.2.2-2, from the day of completion of the bridge up to 20 years.

The change in stress over time along the length of the bridge, including all time-dependent effects, is shown in Fig. 9.2.2-3.

The distribution of moments, and prestressing in the bridge at completion of the structure and after 20 years are shown in Figs. 9.2.2-4 through 9.2.2-7

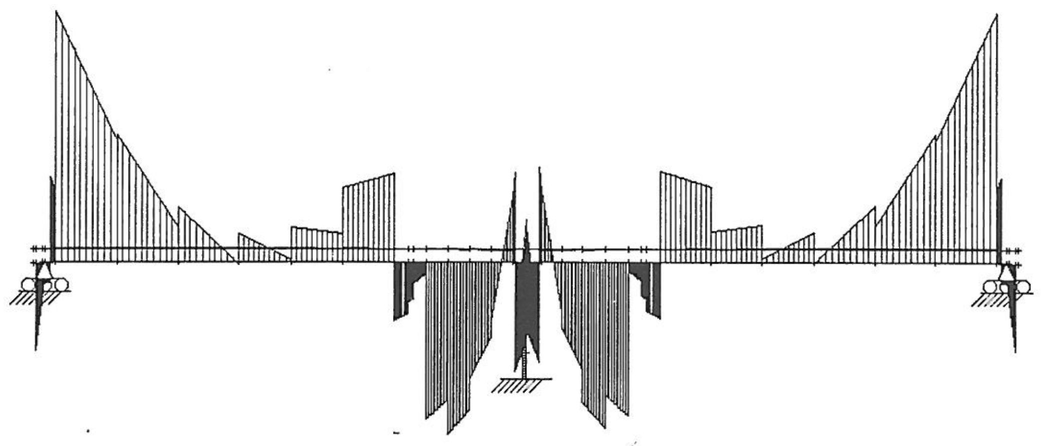


FIGURE 9.2.2-4 Moment at completion of the construction in the girders
Units are kNm

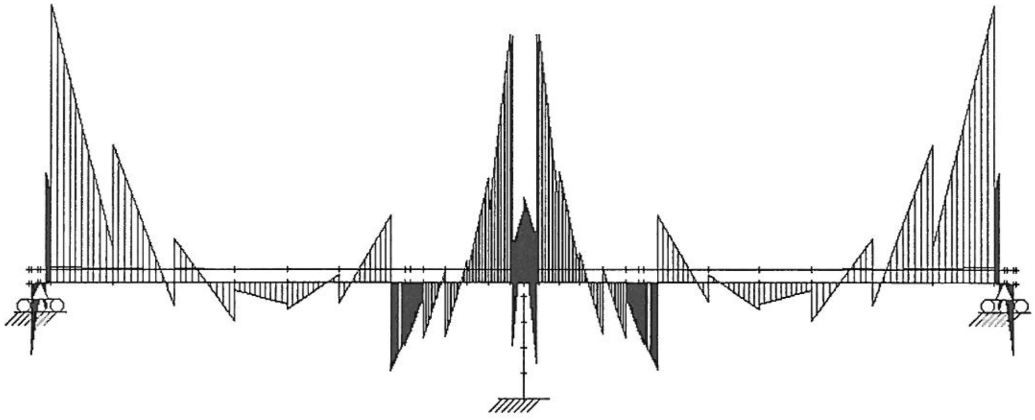


FIGURE 9.2.2-5 Moments in the girder at 20 years. Units are kNm

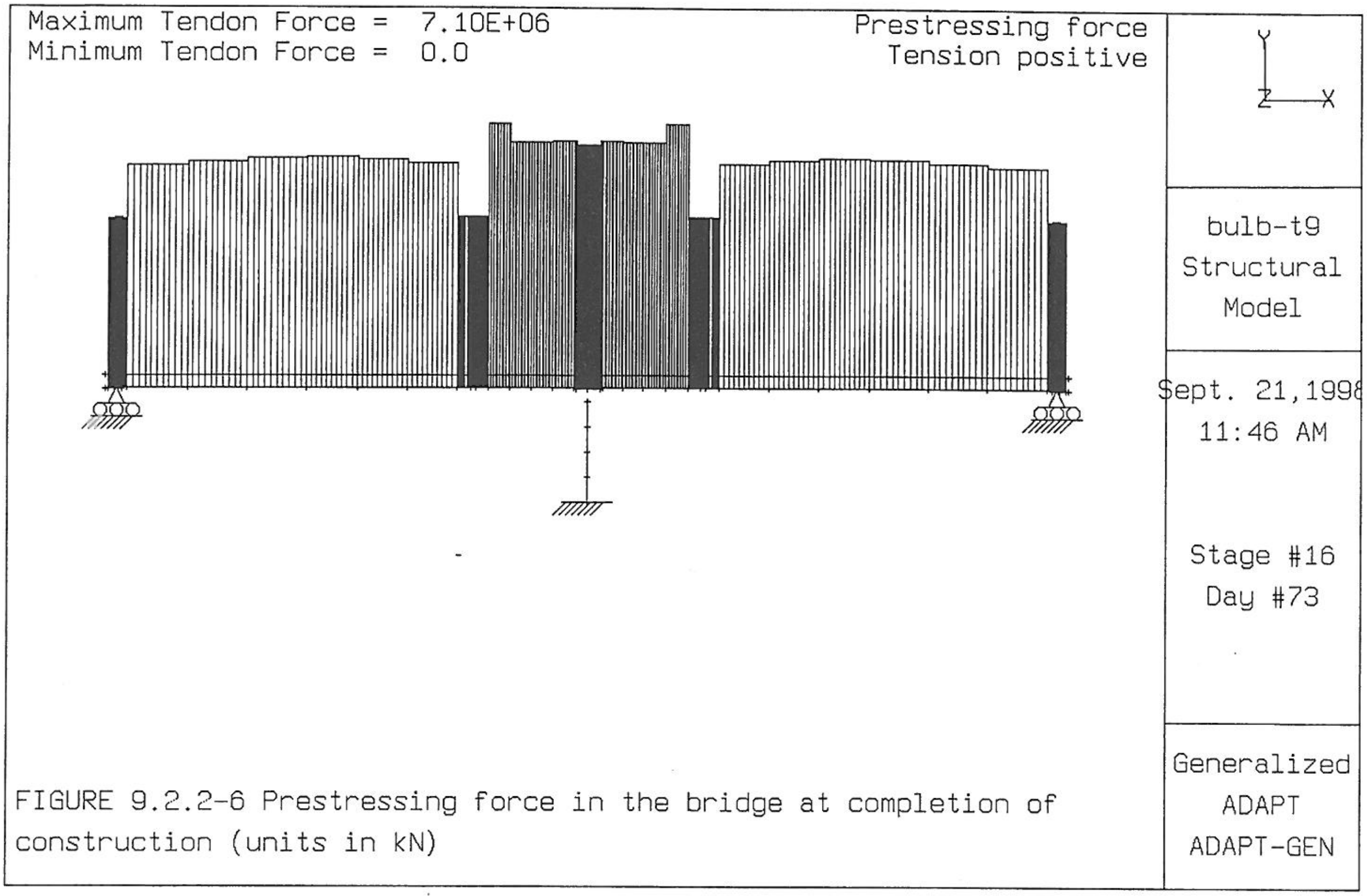


FIGURE 9.2.2-6 Prestressing force in the bridge at completion of construction (units in kN)

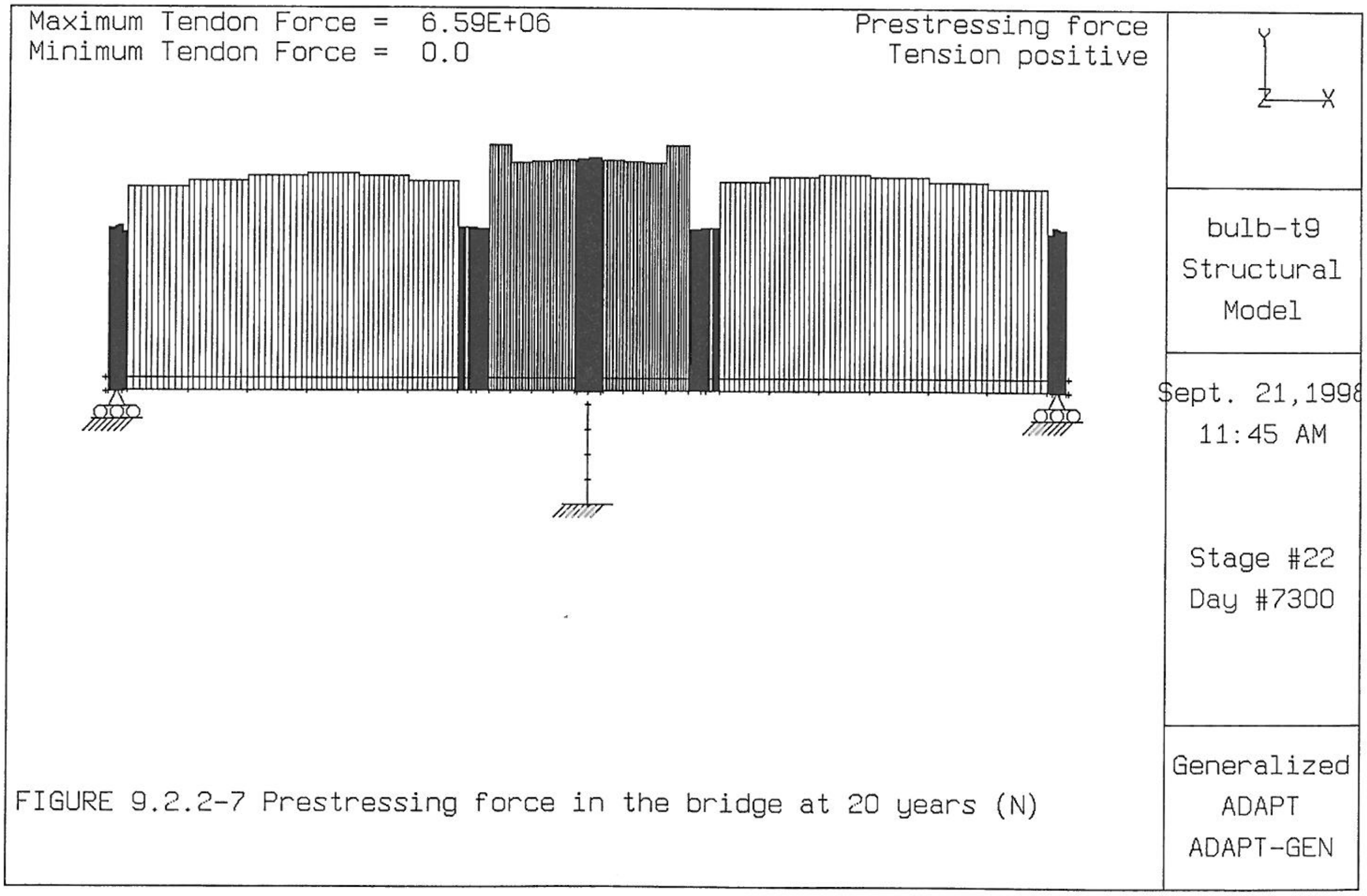
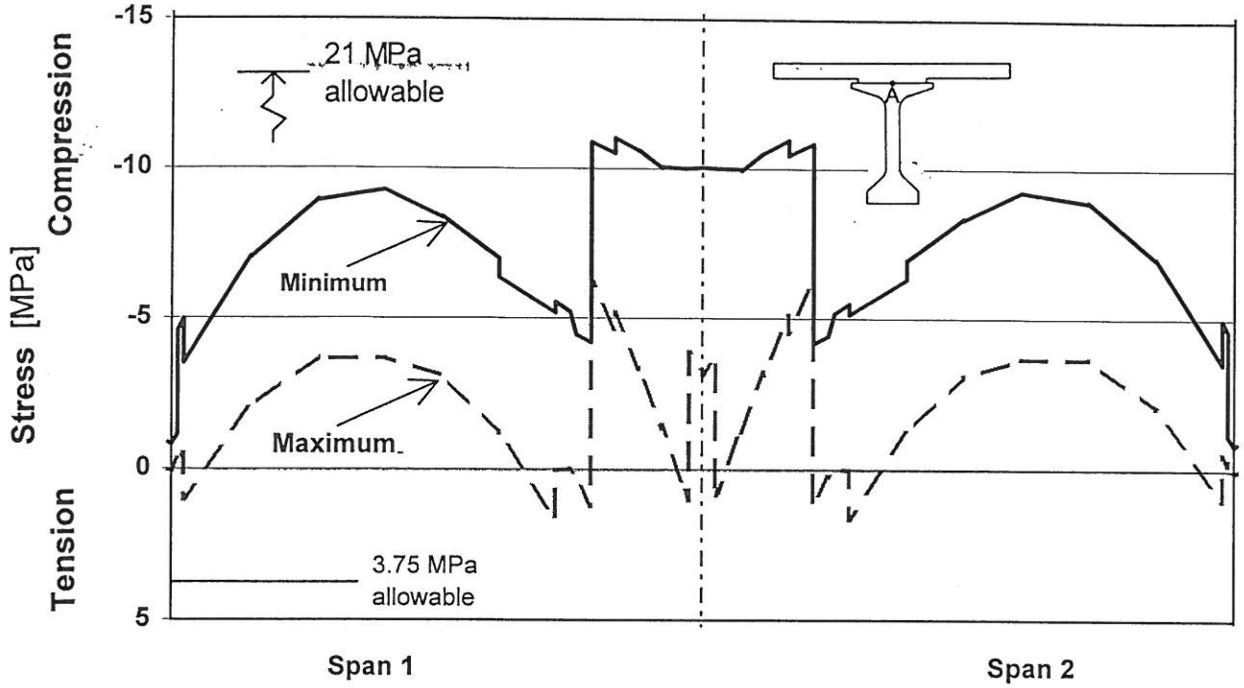
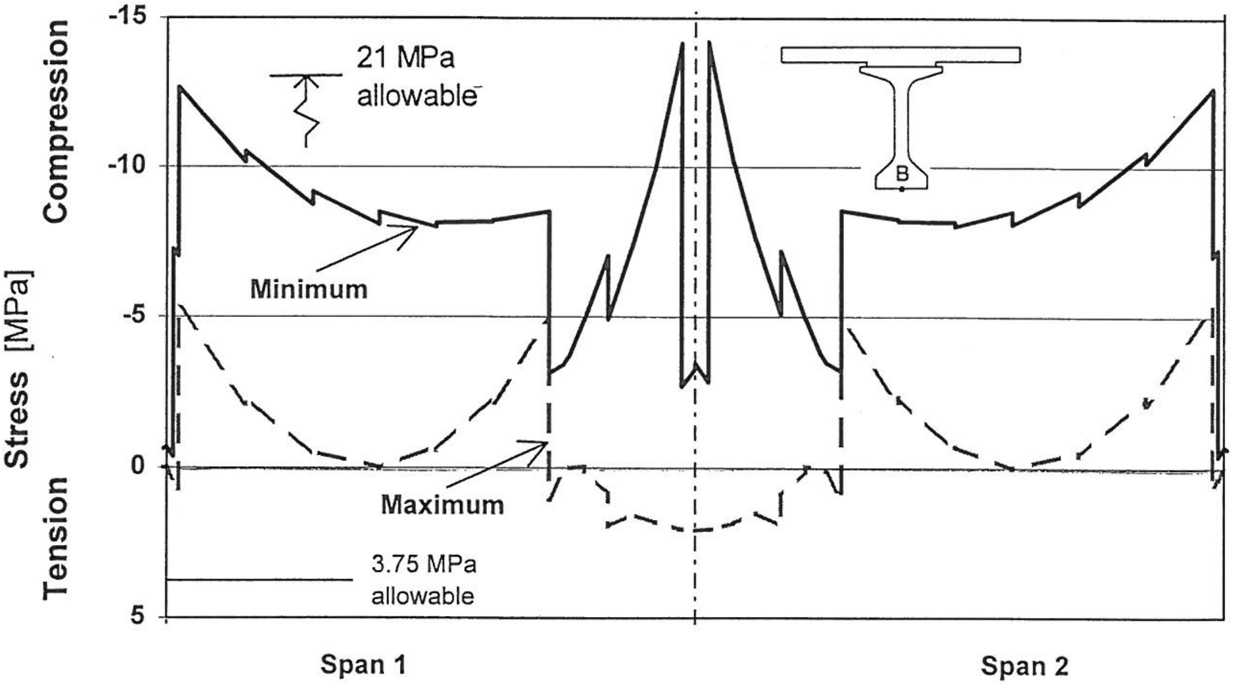


FIGURE 9.2.2-7 Prestressing force in the bridge at 20 years (N)



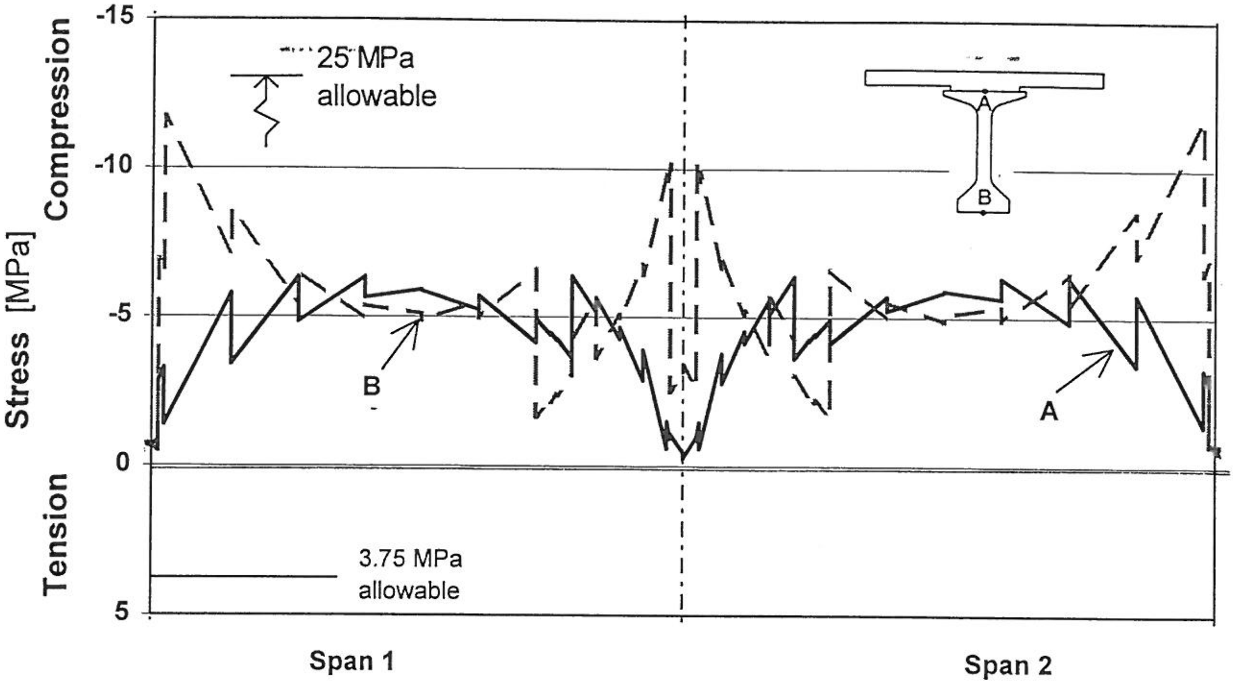
(a) Stress at Top Fiber of Girder (Point A)



(b) Stresses at Bottom Fiber of Girder (Point B)

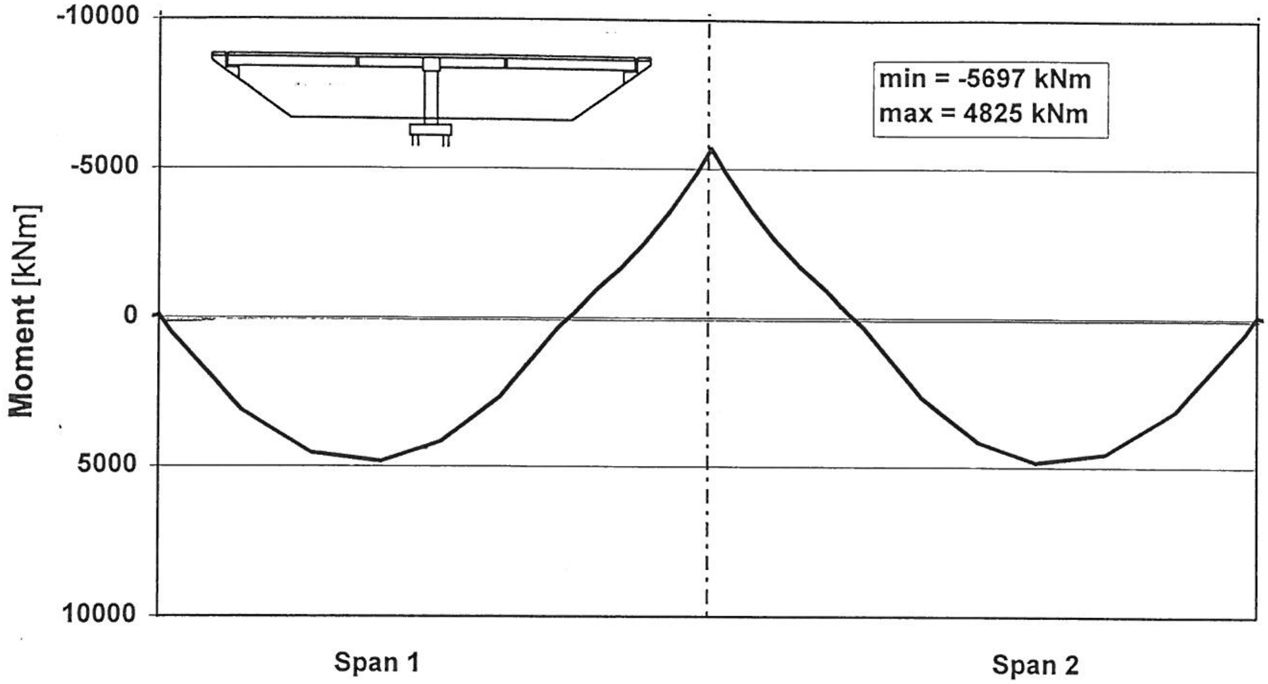
STRESS ENVELOPE IN GIRDER DURING CONSTRUCTION

FIGURE 9.2.1-1

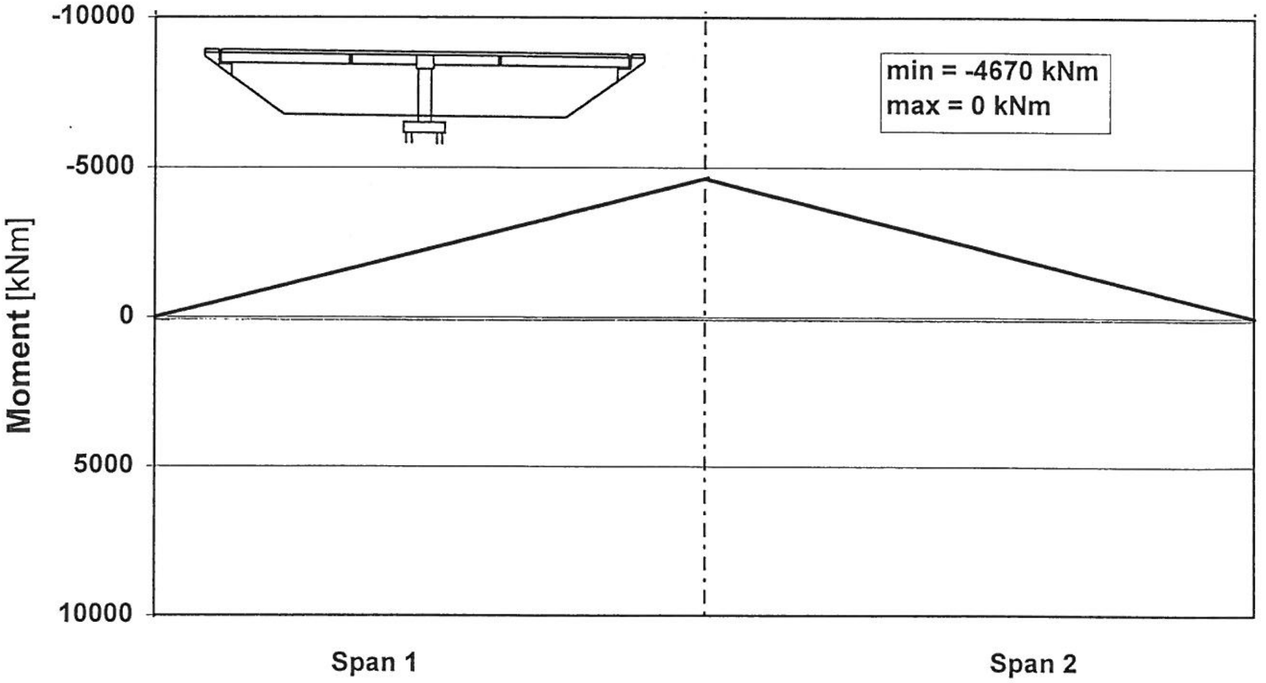


STRESS PROFILE IN GIRDER AT 20 YEARS

FIGURE 9.2.1-2



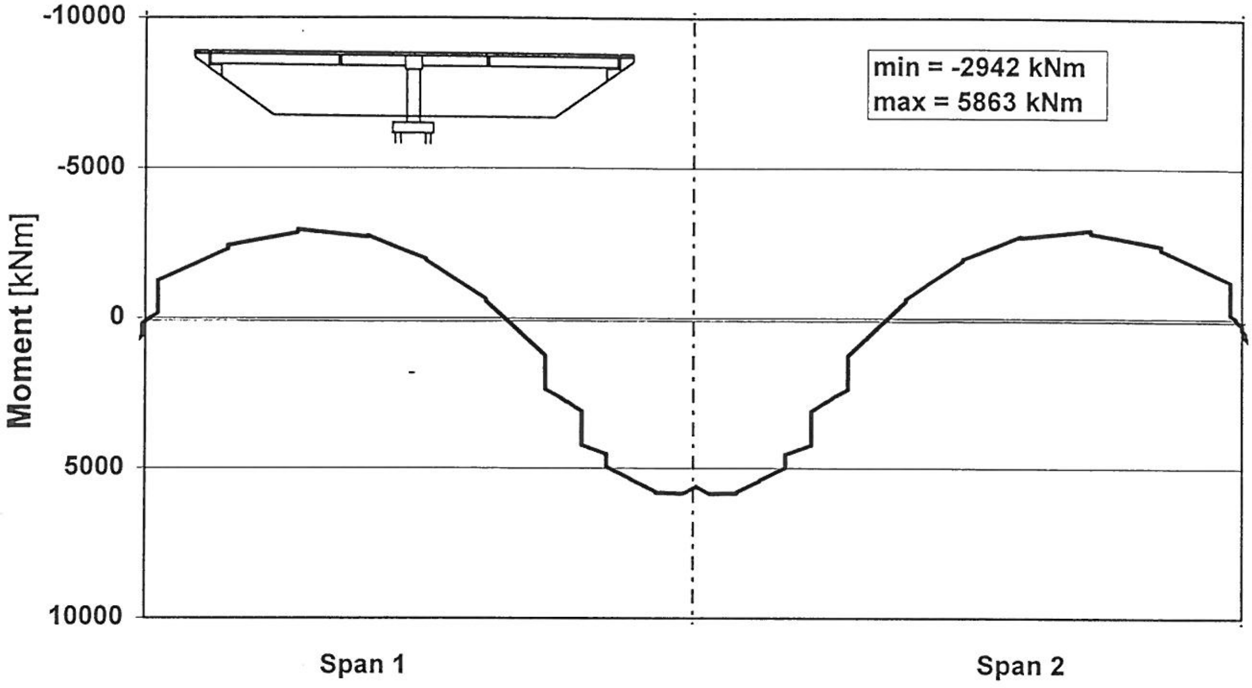
(a) Moments due to DL only



(b) Hyperstatic Moments Due to Prestressing and Time-dependent Effects

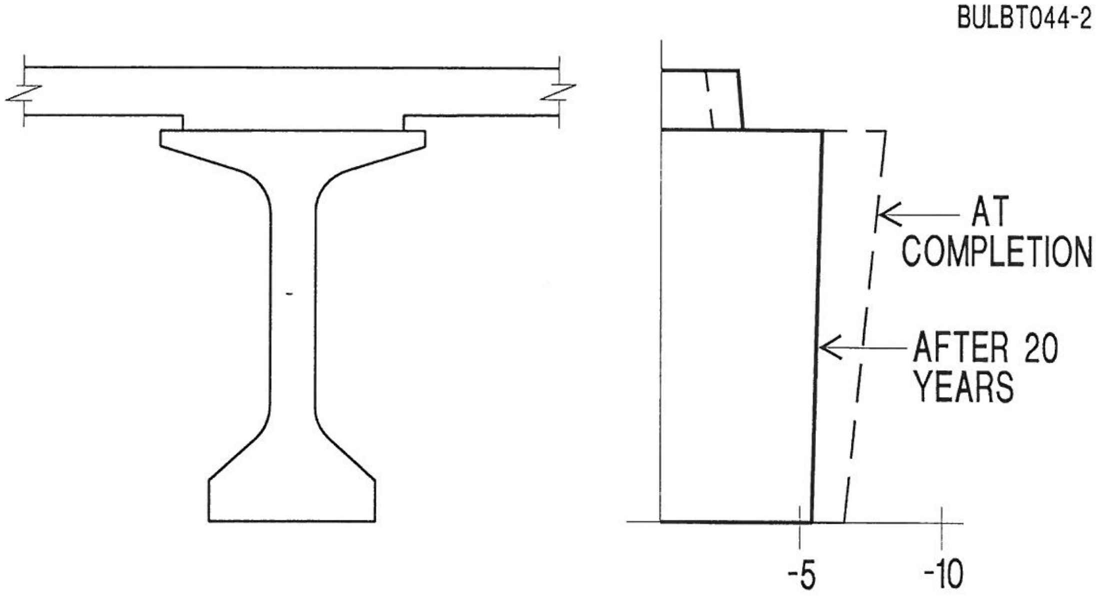
MOMENTS FOR STRRENGTH CHECK AFTER 20 YEARS

FIGURE 9.2.1-3

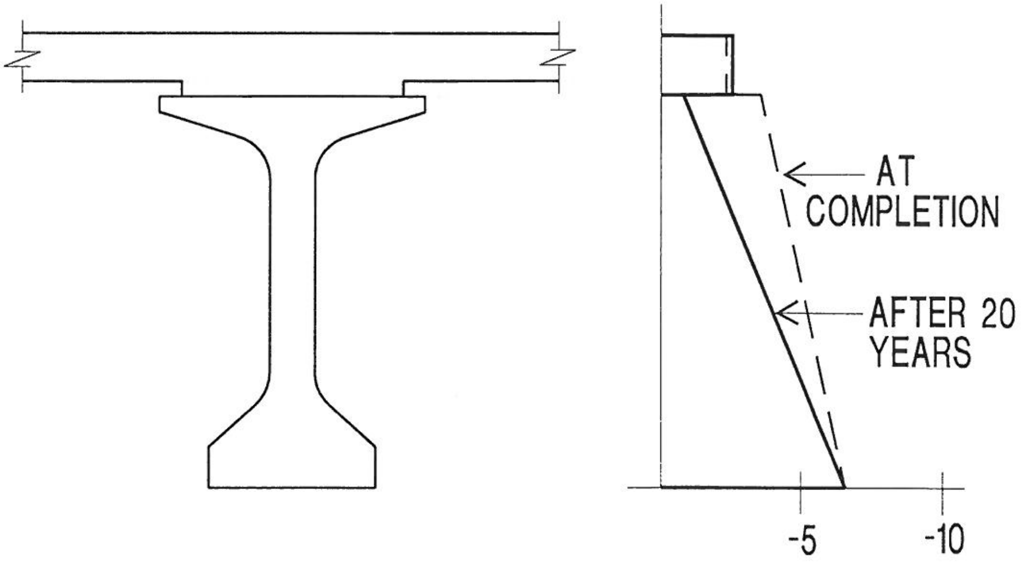


MOMENTS DUE TO PRESTRESSING ALONG THE PC GIRDER
AFTER 20 YEARS
(pre-. and post-tensioning)

FIGURE 9.2.2-1



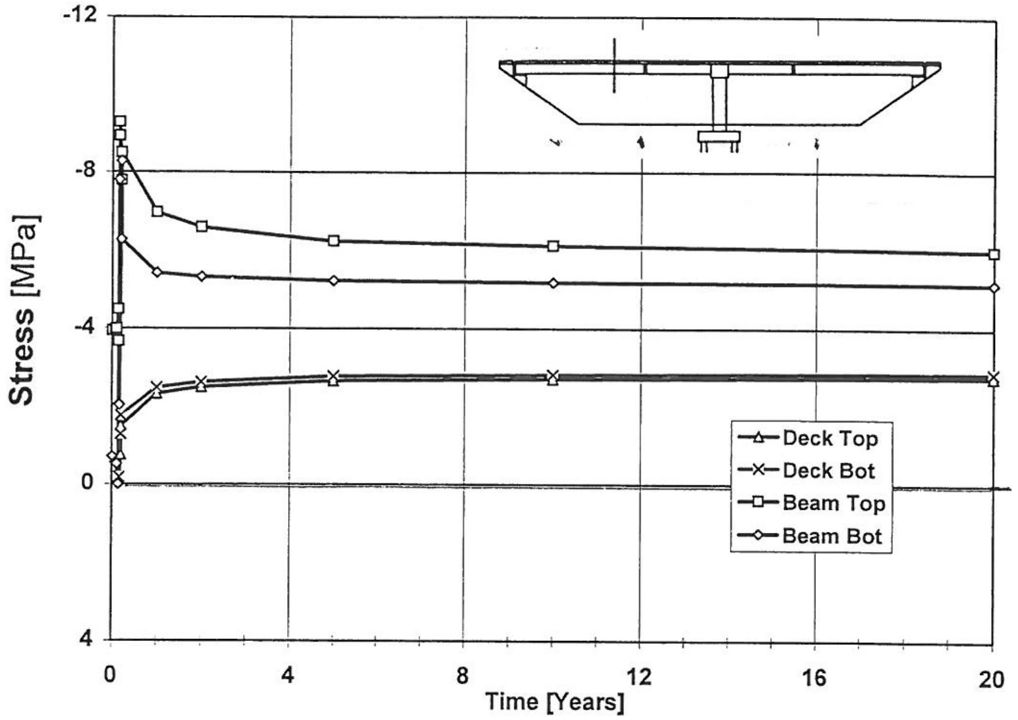
(a) AT 0.4 * SPAN



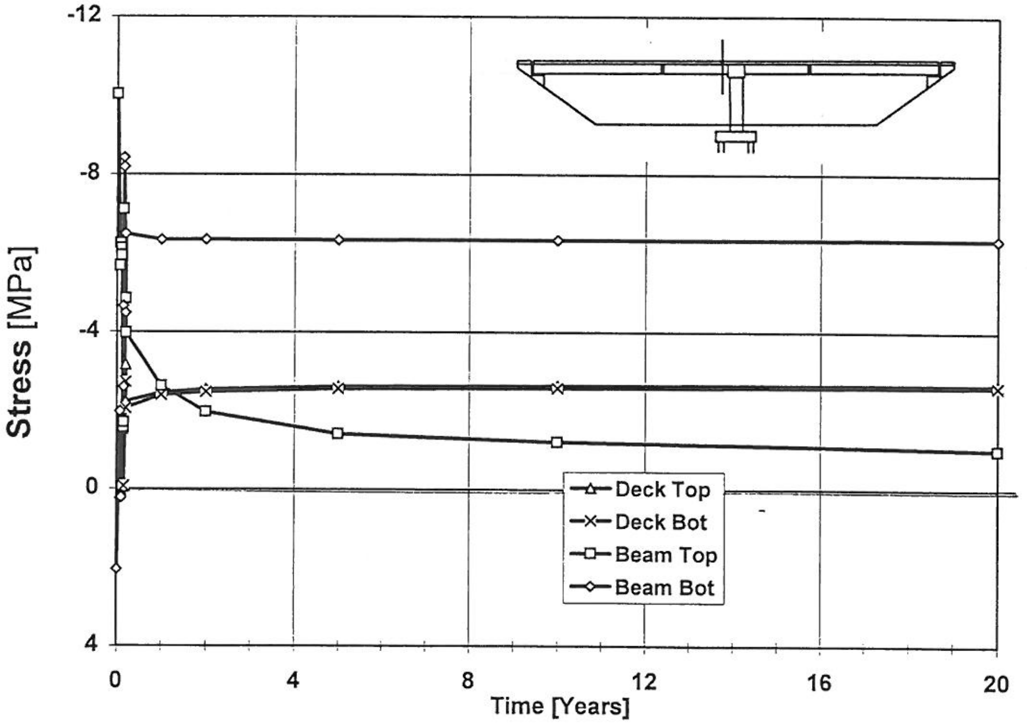
(b) AT FACE OF BENT CAP

STRESS DISTRIBUTION (MPa)

FIGURE 9.2.2-2



(a) PC Girder at 0.40 * Span Length



(b) PC Girder at Face of Bent Cap

CHANGE IN STRESS OVER TIME DUE TO SELF WEIGHT

FIGURE 9.2.2-3

10 - REFERENCES

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Pcmpap10

Precast, Prestressed, Spliced and Post-Tensioned California Bulb-Tee Girder Bridge

DESIGN EXAMPLE USING ADAPT

APPENDIX A

```
;===========================================================================
;                ADAPT SOFTWARE SYSTEM FOR STRUCTURAL CONCRETE
;                          ADAPT-ABI version  2.50
;          1733 Woodside Road, #260, Redwood City, Ca 94061
; Tel: (650) 306 2400; Fax: (650) 364-4678  E-mail: support@AdaptSoft.com
;                     web site: www: AdaptSoft.com
;===========================================================================
;  Name of this file:   BULB-T9.INP
;  Last update      :   September 15, 1998
;
;
; The following is input data for a two span bridge made up of California
; bulb tee precast prestressed girders spliced at 0.32 of span from central
; pier. The continuity is achieved through the splice, post-tensioned
continuity
; tendons and a topping slab. The connection of the deck to the central pier
; is rigid. Over the abutments, the deck rests on rollers.

; The span of the bridge is 43.0m. The deck is for two lanes and 13m wide.
; It consists of 4 prestressed precast bulb-Tee girders.

; The data generated herein represents one interior bulb-Tee. The continuity
; tendons are stressed in stages
;

START

TITLE N=3
       TWO-SPAN SPLICED BULB-TEE GIRDER WITH POST-TENSIONING AND TOPPING
       SLAB

UNITS U=SI                          ; Units are all in N, mm except for
;                                     concrete weight which is in kg
ACTIVATE EXTRACT

CONCRETE PARAMETERS N=6
 1 M=ACI W= 2.4E-06                  ; Bulb Tee
 2 M=ACI W= 2.4E-06                  ; Topping slab
 3 M=ACI W= 2.4E-06                  ; Splice concrete
 4 M=ACI W= 2.4E-06                  ; Concrete for central pier
 5 M=ACI W= 2.4E-06                  ; Bent cap
 6 M=ACI W= 2.4E-06                  ; Used to model sheathing of unbonded
tendons

MESH INPUT
 NODES N=53
  1    X= -1.050   Y= 0      Scale= 1000 ; Outer face of abutment segment
  2    X= -0.850   Y= 0                  ; Tendon end
  3    X= -0.250   Y= 0                  ; End of girder
  4    X= 0.000    Y= 0                  ; Support line (left)
  5    X= 0.500    Y= 0                  ; Inside face of abutment segment
  6    X= 0.950    Y= 0                  ; Node for strand development length
  9    X= 17.200   Y= 0      G=6,9,1     ; Node at 0.4L from support
  12   X= 31.050   Y= 0      G=9,12,1    ; Node for strand development length
```

```
  13  X= 32.250  Y= 0                  ; Left span segment
  14  X= 32.750  Y= 0                  ; Left span splice
  15  X= 33.950  Y= 0                  ; Node for strand development length
  19  X= 41.663  Y= 0      G=15,19,1   ; Spring Support
  20  X= 41.925  Y= 0
  21  X= 43.000  Y= 0                  ; Support line (center)
  22  X= 44.075  Y= 0
  23  X= 44.338  Y= 0                  ; Spring Support
  25  X= 48.000  Y= 0      G=23,25,1   ; Tie connection node
  27  X= 52.050  Y= 0      G=25,27,1   ; Node for strand development length
  28  X= 53.250  Y= 0
  29  X= 53.750  Y= 0                  ; Right span splice
  30  X= 54.950  Y= 0                  ; Node for strand development length
  33  X= 68.800  Y= 0      G=30,33,1   ; Node at 0.4L from support
  36  X= 85.050  Y= 0      G=33,36,1   ; Node for strand development length
  37  X= 85.500  Y= 0                  ; Inner face of abutment segment
  38  X= 86.000  Y= 0                  ; Support line (right)
  39  X= 86.250  Y= 0 -                ; End of girder
  40  X= 86.850  Y= 0                  ; Tendon End
  41  X= 87.050  Y= 0                  ; Outer face of abutment segment
  42  X= 43.000  Y= -8.945             ; Central column
  46  X= 43.000  Y= -1.095 G=42,46,1   ; Bottom of bent cap
  47  X= 41.663  Y= -8.945             ; Pier segment ground support
  48  X= 44.338  Y= -8.945       ; Pier segment ground support
  49  X= 32.500  Y= 0                  ; Left splice bracket
  50  X= 32.250  Y= 0                  ; Left splice bracket
  51  X= 53.500  Y= 0                  ; Right splice bracket
  52  X= 53.750  Y= 0                  ; Right splice bracket
  53  X= 46.000  Y= -8.945             ; Construction tie support


CONCRETE PROPERTIES N=6
  1 Fpc= 42 Cr=3.0 Sh=0.000278 W= 2.44E-06 Ac=1e-5 M=1 ; Bulb Tee
  2 Fpc= 28 Cr=2.9 Sh=0.000272 W= 0.0      Ac=1e-5 M=2 ; Topping slab
  3 Fpc= 42 Cr=3.0 Sh=0.000278 W= 2.44E-06 Ac=1e-5 M=3 ; Splices
  4 Fpc= 28 Cr=2.9 Sh=0.000272 W= 2.36E-06 Ac=1e-5 M=4 ; Central pier
  5 Fpc= 28 Cr=2.9 Sh=0.000272 W= 2.44E-06 Ac=1e-5 M=5 ; Bent cap
  6 Fpc= 1  Cr=0.0 Sh=0.0      W= 0        Ac=0.0  M=6 ; Unbonded sheathing

 ;slab is weightless, load applied separately

MILD STEEL PROPERTIES N=6
  1 Es= 200000 P=0.0075   As=1e-5   ; Bulb Tee
  2 Es= 200000 P=0.0100   As=1e-5   ; Topping slab
  3 Es= 200000 P=0.0200   As=1e-5   ; Splices
  4 Es= 200000 P=0.0075   As=1e-5   ; Central pier
  5 Es= 200000 P=0.0150   As=1e-5   ; Bent cap
  6 Es= 200000 P=0.0      As=1e-5   ; Unbonded & tie sheathing

SECTION PROPERTIES  N=7
  1 Area= 6.865E+05      I= 3.145E+11    C= 905, 945     ; Bulb Tee
  2 Area= 6.814E+05      I= 2.391E+09    C= 100, 115     ; Topping slab
  3 Area= 6.865E+05      I= 3.145E+11    C= 905, 945     ; Span splice
  4 Area= 0.4*2.688E+06  I=0.4*5.750E+11 C= 925, 925     ; Central pier
  5 Area= 6.634E+06      I= 2.211E+12    C= 1000,1000    ; Abutment/full
  6 Area= 5.948E+06      I= 1.890E+12    C= 1011,989     ; B.cap/Abutment/part
  7 Area= 1              I= 1            C= 1,1          ; Sheathing

OFFSET DATA N=4
 1 OI=0,1020    OJ=0,1020               ; Offset topping slab
 2 OI=0,-95.0   OJ=0,-95.0              ; Offset bent cap/abutment/full
 3 OI=0,-106.   OJ=0,-106.              ; Offset bent cap/abutment/part
 4 OI=0,65-945  OJ=0, 65-945            ; Offset for unbonded tendons

ELEMENTS  N=96
FRAME     N=94
   1,1,2     C=5 X=5 St=1 G=1,2,1,1,1    Off=2    ; Left diaphragm/full
```

```
    3,3,4     C=1 X=1 St=1 G=3,12,1,1,1              ; Left span segment
    13,13,14  C=3 X=3 St=3                           ; Left span splice
    14,14,15  C=1 X=1 St=1 G=14,27,1,1,1             ; Pier segment
    28,28,29  C=3 X=3 St=3                           ; Right span splice
    29,29,30  C=1 X=1 St=1 G=29,38,1,1,1             ; Right span segment
    39,39,40  C=5 X=5 St=1 G=39,40,1,1,1  Off=2      ; Right diaphragm/full
    41,1,2    C=2 X=2 St=2 G=41,80,1,1,1  Off=1      ; Topping slab
    81,42,43  C=4 X=4 St=4 G=81,84,1,1,1             ; Pier segment
    85,20,21  C=5 X=6 St=5 G=85,86,1,1,1  Off=3      ; Bent cap segment
    87,25,53  C=1 X=7 St=6                           ; Construction tie element
    88,50,49  C=1 X=1 St=1                           ; Left continuity bracket
    89,51,52  C=1 X=1 St=1                           ; Right continuity bracket
    90,16,26  C=6 X=7 St=6                Off=4      ; Unbonded tendon sheathing
    91,3,4    C=5 X=6 St=1 G=91,92,1,1,1  Off=3      ; Left  diaphragm/part
    93,37,38  C=5 X=6 St=1 G=93,94,1,1,1  Off=3      ; Right diaphragm/part
 SPRINGS   N=2
    95,19,47  K=1e30                                 ; Left  pier spring
    96,23,48  K=1e30                                 ; Right pier spring

 PRESTRESSING STEEL  N=2
   1 Ep= 197000 Meu=0    K=0        Fpu=1860 R=45 Ap=1e-5   ; Pre-tensioning
   2 Ep= 197000 Meu=0.25 K=0.66e-6  Fpu=1860 R=45 Ap=1e-5   ; Post-tensioning

 TENDON GEOMETRY  N=12
 1 SPANS=2 M=2 AREA= 1.68E+03           ; Post-tensioning stage 1
  1 N= 20 G= 2,21,1  B=0,-945 E=1000,-945
    R=0,0.412,0.098  S= 1200 , 250 , 1800
  2 N= 20 G=21,40,1
    R=0.098,0.588,0  S= 1800 , 250 , 1200
 2 SPANS=2 M=2 AREA= 1.68E+03           ; Post-tensioning stage 2
  1 N= 20 G= 2,21,1
    R=0,0.412,0.098  S= 900 , 100 , 1700
  2 N= 20 G=21,40,1
    R=0.098,0.588,0  S= 1700 , 100 , 900
 3 SPANS=1 M=1  AREA=8*98.9             ; Pre-tens., left span, bot outer
  1 N=7  G=6,12  B=0,65-945   E=1000, 65-945
    R=0,0,0  S= 0,0,0
 4 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., left span, bot inner
  1 N=7  G=6,12  B=0,115-945   E=1000, 115-945
    R=0,0,0  S= 0,0,0
 5 SPANS=1 M=1  AREA=8*98.8             ; Pre-tens., right span, bot outer
  1 N=7  G=30,36 B=0,65-945    E=1000, 65-945
    R=0,0,0  S= 0,0,0
 6 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., right span, bot inner
  1 N=7  G=30,36 B=0,115-945   E=1000, 115-945
    R=0,0,0  S= 0,0,0
 7 SPANS=1 M=1  AREA=14*98.8            ; Pre-tens., pier segmt, top outer
  1 N=13 G=15,27 B=0,905-65    E=1000, 905-65
    R=0,0,0  S= 0,0,0
 8 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., pier segmt, top inner
  1 N=13 G=15,27 B=0,905-115   E=1000, 905-115
    R=0,0,0  S= 0,0,0
 9 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., pier segmt, bot outer
  1 N=2  G=15,16 B=0,65-945    E=1000, 65-945
    R=0,0,0  S= 0,0,0
10 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., pier segmt, bot outer
  1 N=11 G=16,26 B=0,65-945    E=1000, 65-945
    R=0,0,0  S= 0,0,0
11 SPANS=1 M=1  AREA=4*98.8             ; Pre-tens., pier segmt, bot outer
  1 N=2  G=26,27 B=0,65-945    E=1000, 65-945
    R=0,0,0  S= 0,0,0
12 SPANS=1 M=1 AREA= 4*98.8             ; Construction Tie
  1 N=2 G=25,53,28   B=48250,0 E= 46250 , -8945
    R=0,0,0  S= 0,0,0

MESH COMPLETE
```

```
SET G=0,0 Day=1                         ; Set weight of wet concrete

CHANGE STRUCTURE

;.................. EVENT 1 ...............
                                ; Stress prestressing tendons
 STRESS N= 3,11    STRESSTO=0.75*1860 0.75*1860 Anchor=0,0

;................. EVENT 2 .............

 BUILD N=3,12,1  Day=1                  ; Left span segment
 BUILD N=29,38,1 Day=1                  ; Right span segment
 BUILD N=14,27,1 Day=1                  ; Pier segment
 BUILD N=90      Day=1                  ; unbonded sheathing

 RESTRAINTS
  6 R=1,1,1                             ; Anchor prestressing strands
 12 R=1,1,1                             ; at stressing bed bulkheads
 15 R=1,1,1
 16 R=1,1,1
 26 R=1,1,1
 27 R=1,1,1
 30 R=1,1,1
 36 R=1,1,1

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 3 ............
SOLVE Day=3                             ; Release prestressing strands
SET G=0,-1
CHANGE STRUCTURE
 RESTRAINTS
  4 R=0,1,0                             ; left  abutment support
 38 R=0,1,0                             ; right abutment support
;                                       ; Release prestressing strands
 6  R=0,0,0                             ; Left span
 12 R=1,1,0                             ; Left span
 15 R=1,1,0                             ; Pier segment
 16 R=0,0,0
 26 R=0,0,0
 27 R=0,1,0                             ; Pier segment
 30 R=1,1,0                             ; Right span
 36 R=0,0,0                             ; Right span

CHANGE COMPLETE
SOLVE ! OUTPUT

;.................EVENT 4 ..................
SOLVE  Day=4  ! OUTPUT          ; Remove Beam From Bed


;.................EVENT 5 ......
SOLVE  Day=18  ! OUTPUT         ; Storage in yard


;.................EVENT 6 ......
SOLVE  Day=20                   ; Cast central column
CHANGE STRUCTURE
 BUILD N=81,84,1 Day=20         ; Cast central column
 RESTRAINTS
 42 R=1,1,1                     ; Apply fixity at column base

CHANGE COMPLETE
SOLVE ! OUTPUT
```

```
;..................EVENT 7 .................
SOLVE Day=28                    ; Transport precast segments
CHANGE STRUCTURE                ; to site and position
 BUILD N=95,96,1                ; Ground support shoring
 RESTRAINTS
 15 R=0,0,0                     ; Transfer pier segment to shoring
 27 R=0,0,0                     ; Transfer pier segment to shoring
 47 R=1,1,1                     ; Ground Support Restraints
 48 R=1,1,1                     ; Ground Support Restraints

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 8 .................
SOLVE Day=30                    ; Cut bottom strands of pier
CHANGE STRUCTURE                ; segment
 REMOVE N=90
 DE-STRESS
 LIST=10

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 9 ................
SOLVE Day=38                    ; Cast bent cap segment
CHANGE STRUCTURE                ; bent cap
 BUILD N=85,86,1 Day=35
 RESTRAINTS
 21 R=3,3,3 M=46

 REMOVE N=95,96,1               ; Remove shoring

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 10................
SOLVE Day=41                    ; Install tie and stress
CHANGE STRUCTURE
 RESTRAINTS
 53 R=1,1,0                     ; construction tie support

 BUILD N=87 Day=40
 STRESS N=12 STRESSTO= 0.75*1860 0.75*1860 Anchor=0,0

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 11................
SOLVE Day=45                    ; Left span in position
CHANGE STRUCTURE                ; Install left bracket
 BUILD N=88
 RESTRAINTS
 49 R=3,3,3 M=14                ; install left bracket
 13 R=3,3,0 M=50                ; 49 slave of 14 and 13 hinge at 50

CHANGE COMPLETE
SOLVE

CHANGE STRUCTURE
 RESTRAINTS
   12 R=0,0,0                   ; place span left segment on bracket

CHANGE COMPLETE
SOLVE  ! OUTPUT
```

```
;..................EVENT 12.................
                                ; Place right span segment

CHANGE STRUCTURE                ; Install right bracket
 BUILD N=89
 RESTRAINTS
 51 R=3,3,3 M=28                ; 51 slave of 28
 29 R=3,3,0 M=52                ; 29 slave of 52

CHANGE COMPLETE
SOLVE

CHANGE STRUCTURE
 RESTRAINTS
   30 R=0,0,0                   ; place right segment on bracket

CHANGE COMPLETE
SOLVE

CHANGE STRUCTURE
 DE-STRESS                      ; Destress construction tie
 LIST=12
 REMOVE N=87                    ; Remove construction tie

CHANGE COMPLETE
SOLVE

;..................EVENT 13.................
SOLVE Day=55                    ; Abutment and splice segments
CHANGE STRUCTURE
 BUILD N=13                     ; cast left span splice
 BUILD N=28                     ; cast right span splice

 BUILD N=1,2,1                  ; cast abutment segments
 BUILD N=39,40,1                ; cast abutment segments
 BUILD N=91,94,1                ; cast abutment segments

CHANGE COMPLETE
SOLVE ! OUTPUT


CHANGE STRUCTURE
 RESTRAINTS
 13 R=0,0,0                     ; Release brackets restraints
 14 R=0,0,0
 28 R=0,0,0
 29 R=0,0,0
 49 R=0,0,0
 51 R=0,0,0

 REMOVE N=88,89                 ; Remove brackets

CHANGE COMPLETE
SOLVE ! OUTPUT

;..................EVENT 14.................
SOLVE Day=57

CHANGE STRUCTURE

 STRESS N=1 STRESSTO= 0.80*1860 0.80*1860 Anchor=6,6 ; Cont. tendons 1

CHANGE COMPLETE
SOLVE ! OUTPUT
```

```
;..................EVENT 15.................

   LOADING
   L=1,40,1 F=0.0,-0.6814*2.36*9.81 ; Topping slab weight

SOLVE Day=58 ! OUTPUT

CHANGE STRUCTURE
 BUILD N=41,80,1

CHANGE COMPLETE
SOLVE Day=59 ! OUTPUT              ; Topping slab hardens

;..................EVENT 15.................
SOLVE Day=73
CHANGE STRUCTURE
 STRESS N=2 STRESSTO= 0.80*1860 0.80*1860 Anchor=6,6 ; 2nd continiuity tendon
CHANGE COMPLETE
SOLVE ! OUTPUT

SOLVE Day=74    ; SIDL Wearing surface + Traffic barriers

       LOADING
       L=41,80,1 F=0.0,-4.378-2.86

SOLVE ! OUTPUT

 ; SOLVE Day=84  ; live load

SOLVE Day=1*365       ! OUTPUT
SOLVE Day=2*365       ! OUTPUT
SOLVE Day=5*365       ! OUTPUT
SOLVE Day=10*365      ! OUTPUT
SOLVE Day=20*365      ! OUTPUT
;;............................. extract actions due to DL and prestressing
CAPTURE STATUS

  LOADING Selfw               ; actions due to dead load only
     DEAD LOAD
  SOLVE
  OUTPUT

  LOADING Prestr              ; actions due to prestressing
     PRESTRESSING
  SOLVE
  OUTPUT


STOP
```