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EVALUATION OF THE DESIGN OF SIMPLE-SPAN PRECAST PRESTRESSED BRIDGE GIRDERS MADE CONTINUOUS FOR COMPOSITE DEAD AND LIVE LOADS

State of the Art

Final Report

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16. ABSTRACT

A detailed investigation was undertaken to study the time-dependent behavior and relevant design criteria for simple-span precast prestressed bridge girders made continuous. A survey of different DOTs across the country was conducted by circulating questionnaires to determine the current practice of analysis and design of this type of bridge. Creep and shrinkage tests were conducted on steam cured concrete at an early age since this type of concrete is used in most precast girders. Computer simulations were carried out using the programs PBEAM and WALL_HINGE to investigate the effects of time-dependent material behavior and design parameters on the effective continuity for live load plus impact. The results indicate that positive moment connections in the diaphragms at the piers are not required as they have no structural advantages. The results also indicate that the effective continuity for live load plus impact can vary from 0 to 100% depending on the design parameters and sequence of construction. The computer analysis was used to determine an upper limit for the amount of negative moment reinforcement over the supports to ensure full moment redistribution and strength.

Computer programs were developed based on simplified analysis to determine the time-dependent restraint moments and service moments upon application of live load. Recommendations for design procedures are included with examples. Suggestions for further research are also included.

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CHAPTER 1

RESEARCH APPROACH

INTRODUCTION

Many concrete bridges are built with simple-span precast prestressed concrete girders made continuous *in situ*. The precast girders are usually pretensioned, between 10 to 90 days old at the time of construction; the girders are placed as simply supported beams on piers and abutments. The concrete deck is cast-in-place, filling in the spaces between girders making them (partially) continuous. Thus, the degree of continuity in a structural system such as this, depends on the time dependent properties of the members as well as the stiffness of the connection provided at the intermediate supports (piers).

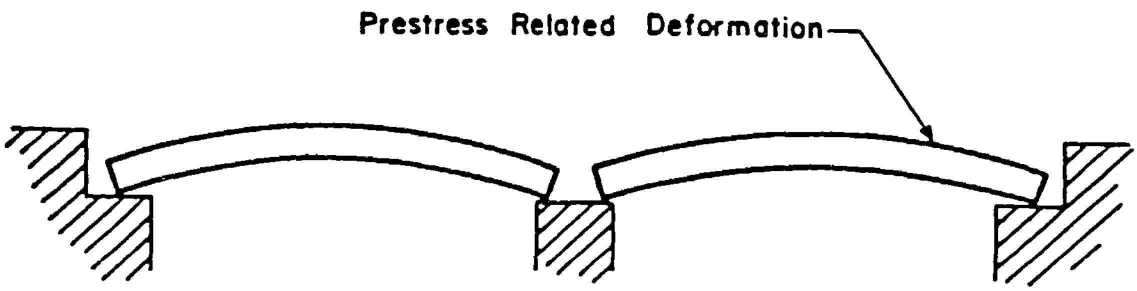
In pretensioned prestressed concrete members, the prestressing force causes the girders to deflect upwards (camber). The ends of the members tend to rotate if they are simply supported, as shown in Fig. 1. When the deck slab is poured and the girders are made continuous, the ends of the girders are restrained from rotating, and as a result, positive moment may develop at a pier, as shown in the figure. Positive moment may also develop at a pier when alternate spans are loaded.

One of the major problems associated with designing continuous bridges built with cast-in-place deck slabs is the estimation of degree of continuity. Owing to various design methods, construction sequences, connection details, and materials used in different states, the degree of continuity varies significantly. To complicate matters further, the creep and shrinkage of concrete and associated cracking, also influence the degree of continuity. The main objective of the study reported is to establish the degree of continuity of a bridge at any given age for given material properties, reinforcement details, connection types and construction types.

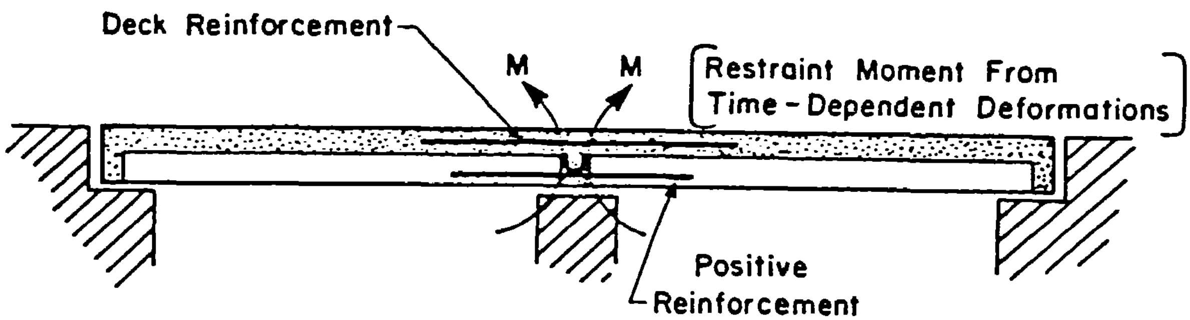
OBJECTIVES

The particular objectives of the study reported were:

(1) To investigate the behavior of precast, prestressed bridge girders made continuous



(a) Simply Supported Precast Girders



(b) Cast-In-Place Deck Slab and Diaphragm Form a Continuous Girder for Live-Load and Time-Dependent Effects

Fig. 1 Two-Span Bridge with Precast, Prestressed Girders Made Continuous by connections using cast-in-place slabs and diaphragms at the piers, and

(2) To develop design procedures and guidelines for computing elastic, inelastic and ultimate time dependent moments depending on the degree of continuity developed at the piers.

RESEARCH PHASES

The project was carried out in four phases:

Phase 1 - Review Existing Data

Literature review on creep and shrinkage prediction of steam cured concrete loaded at an early age; data on camber of composite and non-composite precast prestressed members; methods of prediction of creep and shrinkage; analytical methods to account for the time dependent behavior of prestressed continuous girders. This was accomplished by a survey of several DOTs.

The review of literature is fairly complete and up to date on methods of computing time dependent strains. However, review of previous work on analysis for creep and shrinkage effects on concrete structures is not totally complete. Of the analytical methods available for time dependent effects on concrete structures, the so called "discrete element" method alone has been discussed. Other important methods such as "creep transformed section" method, "improved Dischinger" method etc., have not been discussed.

Phase 2 - Service Moments

Procedures were applied for computing the degree of continuity taking into account the dead loads, live loads, time dependent effects, realistic geometry and accurate material properties. Efficient computer programs such as PBEAM were used to simulate precast prestressed multispan bridges made continuous *in situ*.

The exact reasons for selecting the program PBEAM are not given. Information on other programs suitable for analyzing concrete structures for time dependent effects, is also not provided.

Improved procedures for ultimate strength and defomation capacity of this type of bridge. Computer programs such as WALL_HINGE were used for flexural strengths, inelastic defomation, and failure modes for the hinging region of concrete sections.

Phase 4 - Pier Connection Design

Procedures were developed for strength and serviceability requirements for the positive and negative moment connections at the piers. Implications of the use of positive moment reinforcement were evaluated. Even though PBEAM can be used to determine service moments at the supports, the authors feel that the amount of input data preparation makes it cumbersome for practicing engineers. Two computer programs were developed for the determination of service moments at supports of continuous bridges constructed of precast prestressed girders and cast-in-place deck (BRIDGERM and BRIDGELL). Results of analyses using these programs were compared to results of analyses conducted in Phase 2. Recommendations were established conceming the need for positive moment reinforcement at piers, conditions for determination of design moments and limitations on the amount of deck reinforcement for this type of bridge to ensure sufficient strength and ductility.

CHAPTER 2

FINDINGS

INTRODUCTION

This chapter presents the results of the literature review as well as the analyses for service moments and flexural strength, using the programs PBEAM and WALL_HINGE. Design procedures as well as the implications of the analysis results along with recommendations are also presented.

SUMMARY OF LITERATURE REVIEW

Continuous Bridges

The design and construction of bridges composed of simple span prestressed girders and cast-in-place decks have become quite popular. There are several advantages of continuous bridges, as listed below;

- Elimination of joints and problems associated with joints.
- Reduction in positive live load moments at mid spans.
- Reduction in the area of prestressing steel.
- Enhancement of structural performance, especially in seismic zones.
- Overall reduction in cost of construction.
- Improvement in appearance and riding quality.

Extensive experimental and analytical investigations were conducted at Portland Cement Association. It was found that higher restraint moments were present in bridges with positive moment connection. On the other hand, when no positive moment connection was provided, extensive cracking occurred at the bottom of the diaphragm. Design procedures and recommendations were presented in 1969, by the Portland Cement Association, in the so called PCA method. Although existing bridges designed and built by this procedure are generally performing well, it is believed that this method may not accurately predict the true behavior of these structures. The uncertainty is due to different loading conditions and construction stages, time-dependent effects, and connection details, which were not accurately considered in the PCA method.

Creep and Shrinkage Prediction

While a wealth of literature exists on the time-dependent properties of normal concrete, data on creep and shrinkage of steam cured concrete (which is most commonly used in precast industry) is scarce. Existing creep and shrinkage models were reviewed for suitability in predicting time-dependent properties of steam cured concrete. The models reviewed were;

- Model of ACI Committee 209
- Model of CEB-FIP
- Model of Bazant and Panula

The ACI model, which is the simplest one, was adopted for analysis. Since the data on creep and shrinkage of steam cured concrete is scarce, tests were conducted on four batches of concrete collected from precasting plants located at different parts of the country.

SUMMARY OF FINDINGS

Results of Questionnaire

Questionnaires were sent to various DOTs, designers, fabricators and others to obtain information on the current state of practice in design and construction of bridges constructed with precast prestressed girders made continuous. The information collected was used in establishing various parameters and design procedures recommended in the report. Detailed responses to the questionnaire are shown in Appendix A of the NCHRP 12-29 report.

The questionnaire covered the following major categories;

- Bridge configuration
- Material properties

- Design procedures
- Reinforcement design
- Construction sequence
- Bridge performance
- Miscellaneous

Some of the problems encountered during the service life of simple span bridges made continuous (i.e., bridge performance), as obtained from the respondents of the survey are worth mentioning, as listed below;

(1) Positive moment reinforcement which required field adjustment due to poor fit.

(2) Misplacement of reinforcement such as extended strands inadvertently cut off.

(3) Transverse cracking of the deck in negative moment areas and throughout the bridge.

(4) Excessive girder camber, requiring adjustment of profile grade, etc.

(5) Incorrect construction sequencing.

(6) Cracking of pier diaphragms due to long term creep and shrinkage.

(7) Cracking and spalling of pier diaphragms when diaphragms are cast before the deck.

(8) Spalling of piers and abutments due to poor girder location or inadequate seat detailing.

(9) Movement of girders during construction when deck concrete is poured before diaphragms.

Creep and Shrinkage Tests

Creep and shrinkage tests were conducted on specimens from four different regions of the United States to broaden the knowledge on steam cured concrete loaded at an early age. The mix properties of concretes obtained from different parts of the country are shown in Table 1 and the test conditions for creep experiments are shown in Table 2. The test data collected from these tests were used for obtaining empirical parameters of the prediction equations. The forms of equations assumed for fitting the data are those given by ACI-209. Tables 3 and 4 show the curve fit parameters of creep and shrinkage

TABLE 1 CONCRETE MIX DESIGNS FOR CREEP TEST SPECIMENS

Component	Precaster			
	A	B	C	D
Cement, pcy	750	800	800	660
Sand, pcy	1020	1100	1320	1230
Stone, pcy	1860	1780	1670	1840
Water, pcy	270	240	230	210
W/C Ratio	0.37	0.30	0.29	0.31
Admixtures	Air-Entraining Agent, ASTM C494 Type D Water Reducer - Retarder, ASTM C494 Type F High Range Water Reducer	High Range Water Reducer	ASTM C494 Type F High Range Water Reducer	ASTM C494 Type G High Range Water Reducer, Air-Entraining Agent, Retarder

TABLE 2 PRETEST CONCRETE CONDITIONS FOR CREEP TEST SPECIMENS

	Precaster			
	A	B	C	D*
Curing conditions	Steam 12 hr	Steam "overnight"	Steam 14-1/2 hr	Steam "overnight"
Time from casting to loading for creep test	54-1/2 hr	51-1/4 hr	48 hr	26 hr (1d) 50 hr (2d)
Pretest comp. strength, psi	6570	7510	6800	4770 (1d) 4830 (2d)
Pretest Mod. of Elasticity, ksi	5850	5310	4500	4640 (1d) 4650 (2d)

*Designations (1d) and (2d) refer to cylinders loaded for the creep test at 1 day and 2 days after casting, respectively.

tests. In general, the results from the tests were between the upper and lower bounds of the ACI prediction model.

VERIFICATION OF ANALYTICAL PROCEDURES

Analysis for Service Moments

Time-dependent deformations and restraint moments induced in multispan bridges built of prestressed girders made continuous were studied by using the program PBEAM. Appendix C of the NCHRP 12-29 report gives a detailed description of this program. The program uses a step-by-step method for time-dependent analysis with a tangent stiffness method implemented for solving nonlinear responses. Depending on the stress level and time-dependent material properties, the program accounts for cracking of girder and/or deck concrete under positive or negative moments.

Comparison Between Analysis and Test Results

To confirm the validity of PBEAM, results of computer analysis were compared with to the PCA test observations conducted by Mattock. These tests were performed on prestressed girders which were monitored for long term behavior. The comparison between test results and computer analysis for long-term variations in moments and support reactions are shown in Figs. 2, 3, 4. The comparison appears to be fairly good, using the ACI creep and shrinkage model. Note that this may change if the CEB-FIP model is used for predicting creep and shrinkage strains (CEB-FIP model for time-dependent strains, decomposes the creep strains into delayed elastic i.e. reversible, and plastic, i.e., irreversible components; the computations therefore significantly change).

Analysis for Flexural Strength

Program WALL_HINGE was used to analyze the strength, inelastic deformation capacity and failure modes for the hinged regions of structures subjected to combined axial load, moment and shear. The analysis accounts for a nonlinear strain distribution within the hinged region. The important effects of aggregate interlock, dowel action, and interaction of compressive stress and shear stress in the compression zone were included.

TABLE 3 CURVE FIT PARAMETERS FOR CREEP COEFFICIENT DATA

Precaster	Days of Data	Test ν_u	ψ	d	ACI-209 ν_u
A	635	2.36	0.50	6.8	2.49
B	606	3.42	0.68	23.7	2.12
C	365	1.50	0.56	6.4	2.19
D (1d)*	334	3.11	0.65	11.1	2.33
D (2d)*	305	3.15	0.60	11.1	2.33

*Designations (1d) and (2d) refer to creep tests started at 1 day and 2 days after casting, respectively.

TABLE 4 CURVE FIT PARAMETERS FOR SHRINKAGE STRAIN DATA

Precaster	Days of Data	Test $(\epsilon_{sh})_u$, millionths	α	f	ACI-209 $(\epsilon_{sh})_u$, millionths
A	635	574	0.54†	11.6†	585
B	606	884	0.68†	12.3†	593
C	365	809	0.66†	20.2	671
D (1d)*	334	710	0.91	22.5	616
D (2d)	305	660	0.92	21.7	616

*Designations (1d) and (2d) refer to creep tests started at 1 day and 2 days after casting, respectively.

†Outside of ACI-209 Normal Range.

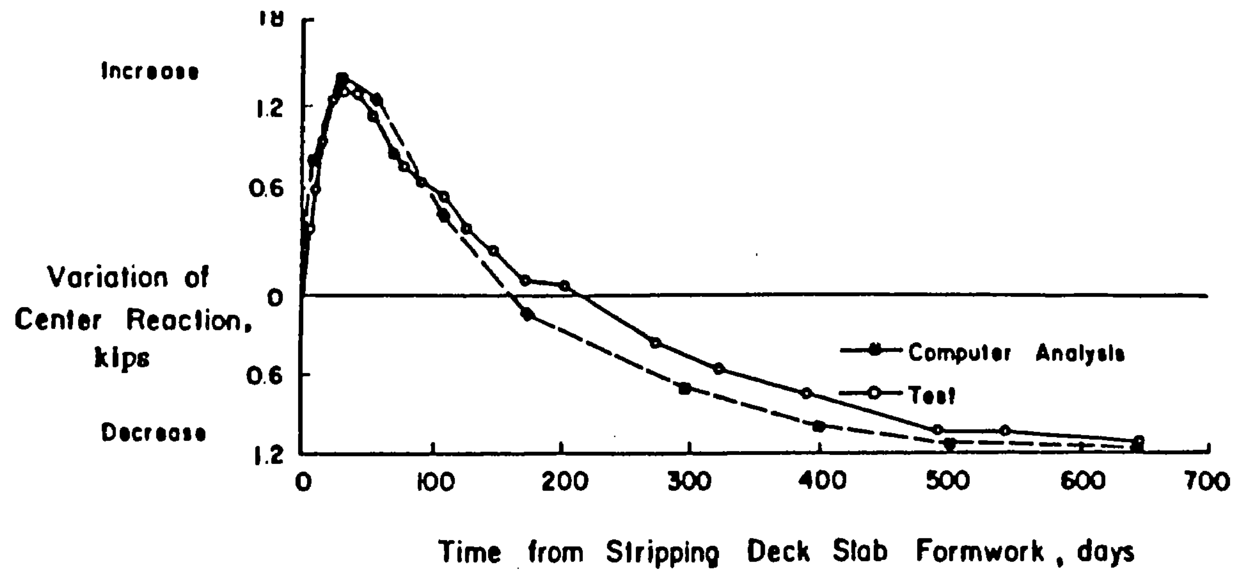


Fig. 2 Variation with Time of Center Support Reaction

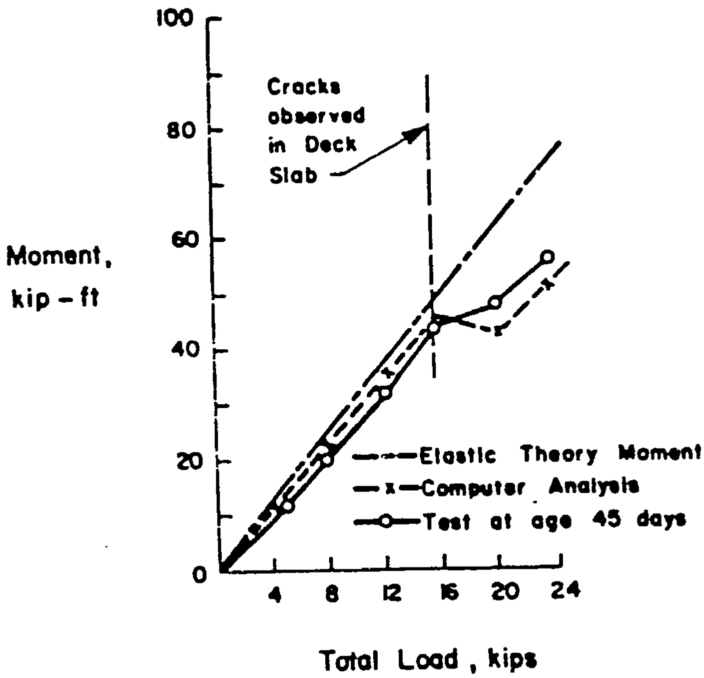


Fig. 3 Variation with Applied Load of Center Support Moment, Girder 3/4

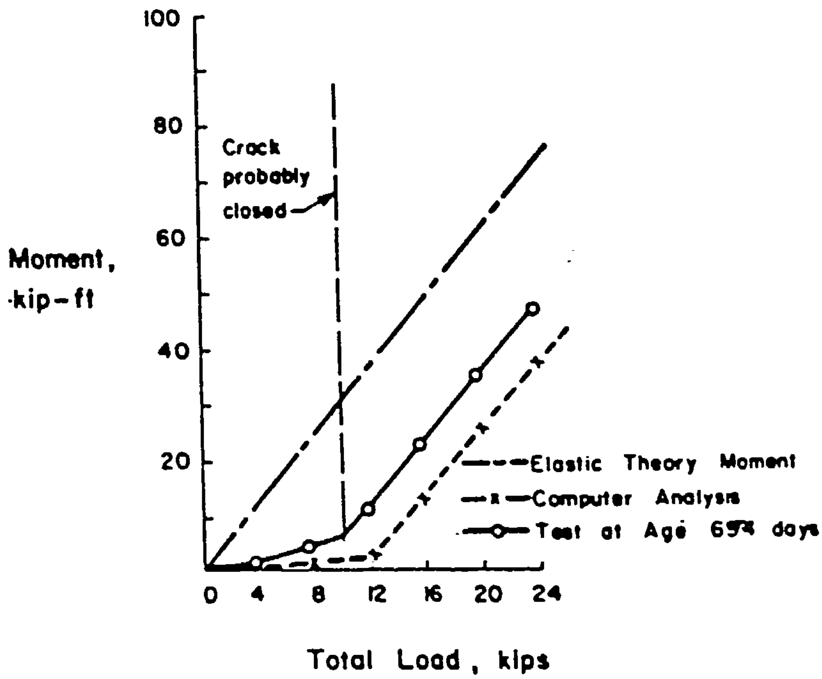


Fig. 4 Variation with Applied Load of Center Support Moment, Girder 1/2

Comparison Between Analysis and Test Results

In order to verify the applicability of WALL_HINGE, the program was used to analyze the girders tested by Kaar et al., at PCA labs in 1960 (Ref. #6 of the NCHRP 12-29 report). Tables 6, 7 show the comparison between WALL_HINGE and test results. The ratios of moments measured in test over the moments calculated are close to 1.0 in Table 5, indicating good agreement with test results.

SERVICE MOMENTS

The primary task of the project was to determine the time-dependent effect on continuity of precast, prestressed girders made continuous. To accomplish this, a parametric study was made using PBEAM to examine the range of bridge behavior as related to the range of material properties and bridge design parameters, particularly from the point of view of design. A parametric study was carried out using PBEAM. The important types of variables considered were;

(1) Girder type
(2) Span length
(3) Girder spacing
(4) Positive Moment continuity reinforcement
(5) Time-dependent materials properties such as concrete compressive strength, creep coefficient, shrinkage strain and steel relaxation coefficient.
(6) Girder age when deck and diaphragm are cast
(7) Girder age at application of live load
(8) Construction sequence

Values for these parameters were chosen based on the responses to the questionnaire circulated to different DOTs and structural design firms. Four girder types were used in the parametric study, AASHTO types IV, VI, modified bulb tee BT72/6 and a box section. The girder span used varied from 70 to 130 ft. The girder spacing was between 4.5 and 8 ft and had a minor influence on bridge behavior. Table 7 shows the girder sections used in the parametric study.

TABLE 5 - NEGATIVE MOMENT CAPACITIES FROM TESTS AND ANALYSES

Girder	Deck Steel, %	PCA Test, in.-kip	BEAM BUSTER		WALL_HINGE	
			Calculated, in.-kip	Mtest/Mcalc	Calculated in.-kip	Mtest/Mcalc
1	0.83	-2440	-2060	1.18	-2230	1.09
2	1.66	-3490	-3610	0.97	-3570	0.98
3	2.49	-4000	-4960	0.81	-4290	0.93

TABLE 6 - DECK REINFORCEMENT STRAINS NEAR MOMENT CAPACITY FROM TESTS AND ANALYSES

Girder	Deck Steel, %	PCA Test	BEAM BUSTER	WALL_HINGE
1	0.83	0.0119	0.0195	0.0320
2	1.66	0.0016	0.0065	0.0118
3	2.49	0.0012	0.0020	0.0014

TABLE 7 - GIRDER SECTIONS USED IN PARAMETRIC STUDY

Girder Type	Girder Cross Section Area, in.^{2}	Composite Section Area, in.^{2}	Midspan Composite Dead Load Moment,* kip-in.
AASHTO-IV	789	1387	21,700
AASHTO-VI	1085	1683	26,300
BT72/6	701	1299	20,300
Box	920	2180	34,100

*For simple span length = 100 ft

One of the main priorities of the parametric study was to determine the effect of varying amounts of positive reinforcements at supports on the bridge behavior. Typical values were 0.2, 3.6 and 7.2 sq. in. The 0.2 sq. in. is intended to represent an unreinforced section, 3.6 and 7.2 sq. in. of steel to represent a reinforced section.

The difference in compressive strengths between concrete deck and girders had a minor effect on the bridge behavior, as indicated by PBEAM results.

For the purposes of the parametric study using PBEAM, it was assumed that the deck and diaphragm were constructed simultaneously. The live load configuration shown in Fig. 5 was assumed for studying the bridge response. The magnitude of the loads is based on AASHTO HS20-44 lane load with a girder spacing of 8 ft and and an impact factor for a span length equal to 130 ft. For bridges with girder age at continuity of 17 days, live load was applied at 650 days or later. For bridges with girder age at continuity of 67 days or later, live loads were applied at 30 days after continuity was established.

Appendix D (of the NCHRP 12-29 report) contains the results of parametric study using PBEAM for analysis of 41 combinations of variables.

RESULTS OF PARAMETRIC STUDY FOR SERVICE MOMENTS

The program PBEAM was used to study the bridge response using several combinations of variables. The results are categorized as below;

Effect of Continuity Connection on Positive Moment

The effects of providing various amounts of positive moment connection steel on support restraint moments is shown in Fig. 6. The amount of positive reinforcement is designated by A_s in this figure. The figure also shows the effect of incremental application of live loads on the support restraint moments. For low area of steel (unreinforced), the restraint moments remain unaltered, while the restraint moment changes sign for reinforced sections. At 100% live load, the moments are negative for reinforced sections while it remains positive for unreinforced section. This is due to the fact that unreinforced sections have ends which are not sufficiently restrained, thus allowing rotations. Note that the increment in live loading is instantaneous so that time-dependent effects due to load increment can be ignored.

The time-dependent moments at the midspan of an interior span are shown in Fig. 7. The midspan moments increase due to creep effects in reinforced sections, while in unreinforced sections, the moment remains constant. The effect of incremental application of live load seems to depend on the degree of continuity at the supports. Again, the load was assumed to be incremented instantaneously, ignoring it's time-dependent effects on the midspan moments.

It is claimed in the NCHRP 12-29 report that, for any value of positive restraint reinforcement provided at the connection all girders reached the same resultant midspan moment after application of 100% live load. However, Appendix -D (of the NCHRP 12-29 report) has several plots (see for example pages D-11, D-13, D-17, D-21, D-23, D-27, D-29, D-35, D-39, D-47, D-49, D-75, and D-79) where girders with different areas of positive restraint reinforcement have different midspan moments upon application of 100% live load. Realizing that these graphs are only results of runs using a computer program (PBEAM), one has to carefully review the comments about the role of positive moment reinforcement made in the report. It is recommended in the NCHRP 12-29 report, based on the computer studies of several girders (see Table 8, Figures 6,7 and all figures in Appendix -D), that positive reinforcement has no structural advantage and that it should be ignored in the design. It seems plausible to do so only if this fact can be confirmed by another analysis, probably using a different computer program.

It was also seen that (see Table 8) higher ultimate creep coefficients result in more positive restraint moments at the supports. This table also shows that the midspan moments for an interior span are almost the same for girders with and without positive restraint reinforcement. Since providing positive reinforcement connections at the supports is a time consuming and expensive procedure, the economic factors of providing such connections should be also considered.

Effect of Continuity Connection on Negative Moment

Results of PBEAM analysis indicate that negative moments can develop in diaphragms at the supports, depending on the age of the girder when the deck and diaphragm are

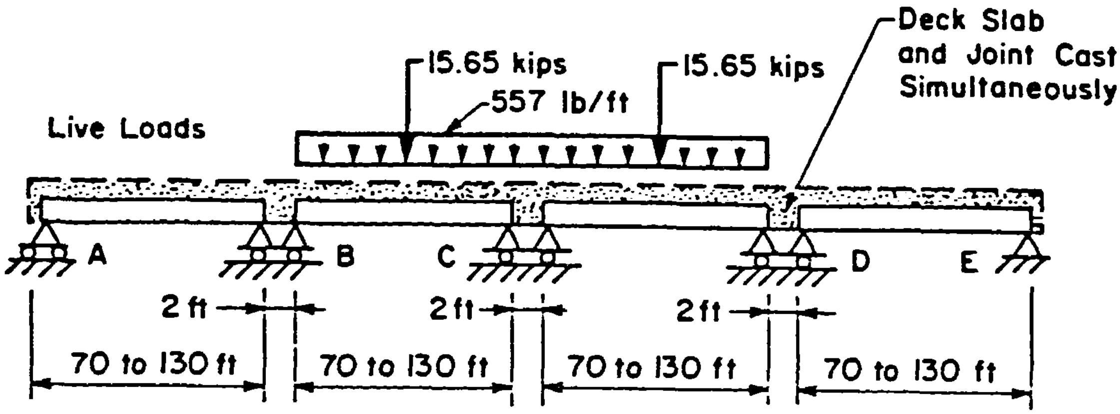


Fig. 5 Bridge with Four Spans of Various Girder Types

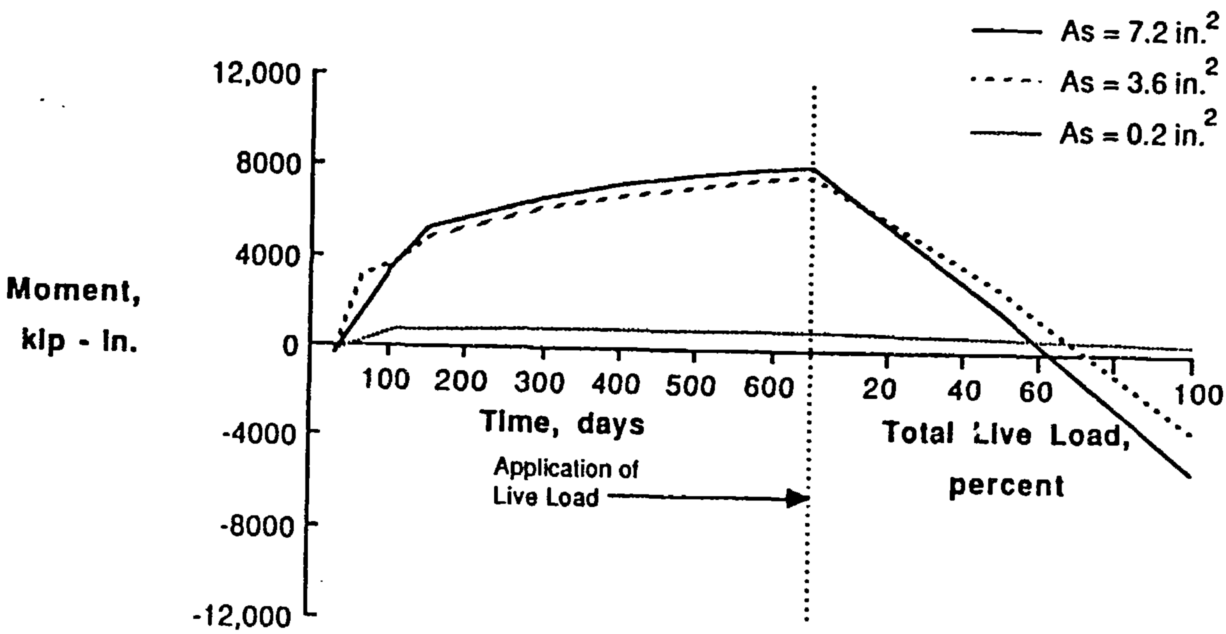


Fig. 6 Central Support Moments for Continuity at Girder Age of 17 Days, Loading at 650 Days

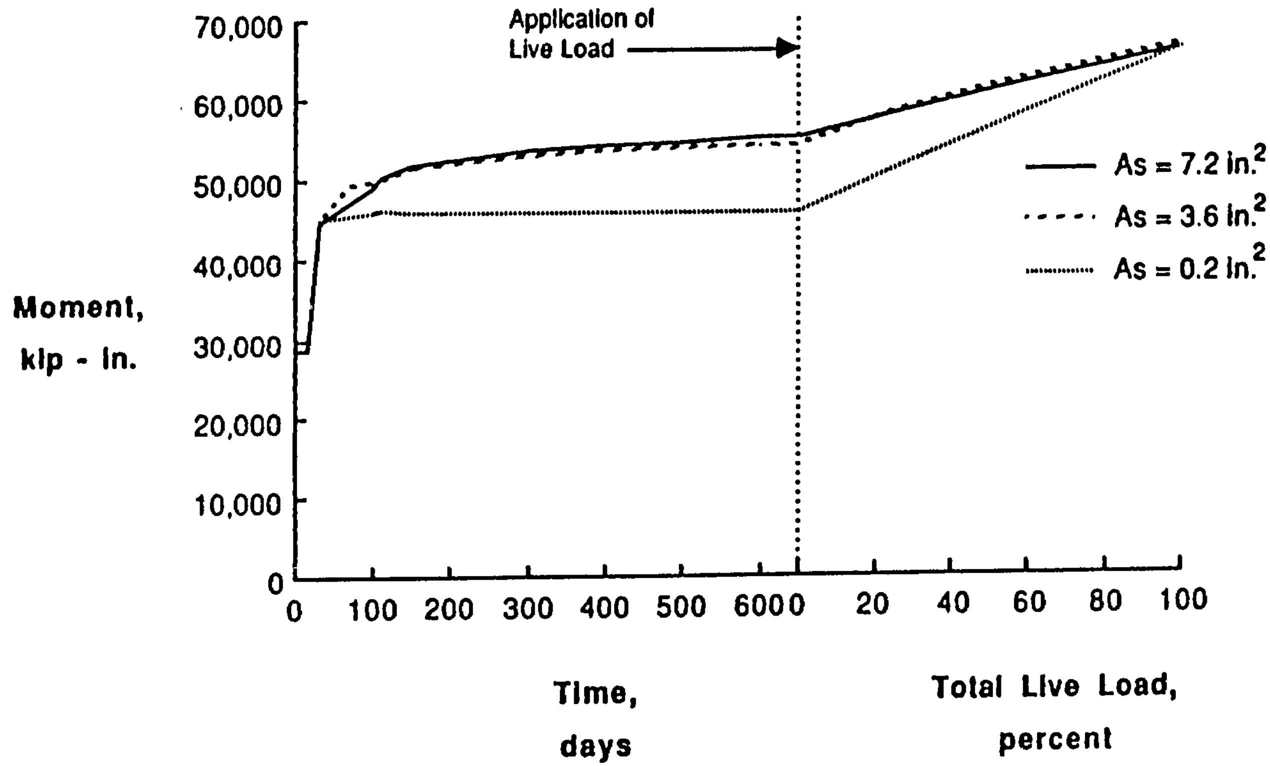


Fig. 7 Interior Span Midspan Moments for Continuity at Girder Age of 17 Days, Loading at 650 Days

TABLE 8 - RESULTANT MOMENTS FROM PARAMETRIC STUDY - 100-FT SPANS, GIRDER AGE AT CONTINUITY 17 DAYS, AGE AT LOADING 650 DAYS

Girder Type	Ultimate Creep Coefficient	Positive Reinforcement, in.^{2}	Resultant Moments, kip-in. At Support B	At Midspan	At Support C
AASHTO-VI	3.25	7.2	2940	39,200	-4750
		0.2	490	40,200	- 160
	1.625	7.2	-960	36,000	-7150
		0.2	-400	36,600	-6500
BT72/6	3.25	7.2	-100	31,100	-6750
		0.2	1100	31,900	-6310
	1.625	7.2	-3130	28,600	-8700
		0.2	-2110	29,200	-8390
AASHTO-IV	3.25	7.2	4370	35,800	-3930
		0.2	320	36,000	400
	1.625	7.2	1230	33,300	-5900
		0.2	230	33,900	-3680
Box	3.25	7.2	7230	48,800	-5790
		0.2	-640	47,700	40

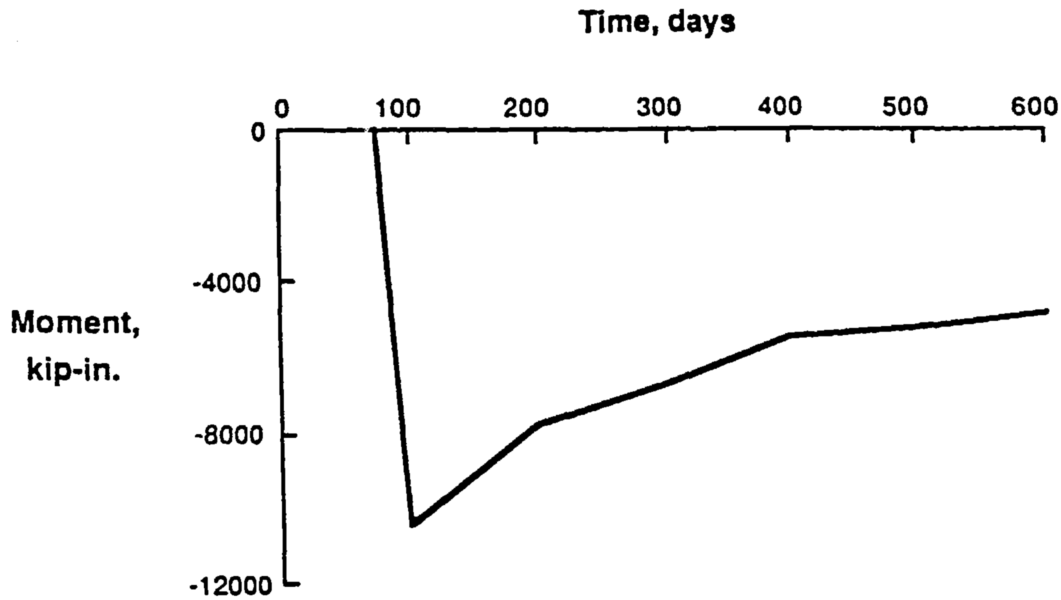


Fig. 8 Central Support Restraint Moment for Continuity at Girder Age of 67 Days

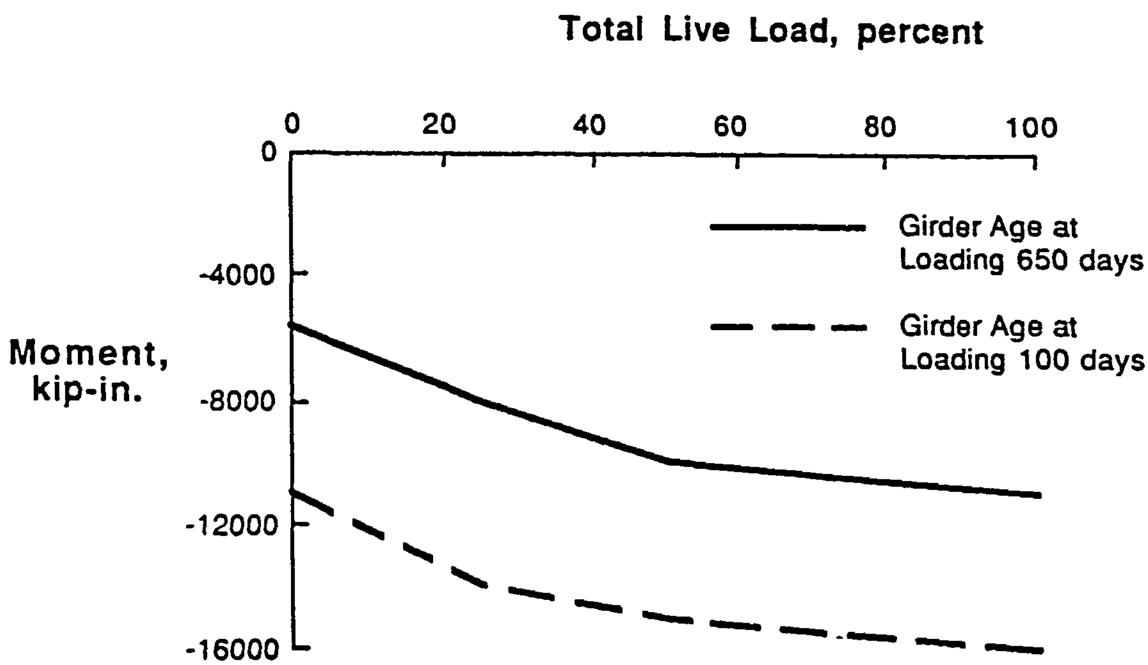


Fig. 9 Central Support Resultant Moments for Continuity at Girder Age of 67 Days, Loading at 100 and 650 Days

cast. The maximum resultant negative moment depends on the differential age between the girder and the deck, and on the age of the bridge when the live load is applied. Fig. 8 shows typical time-dependent restraint moments at the center support. Shrinkage difference between the deck and the girder directly influences the restraint moment development, immediately after continuity is established.

With increasing girder age at continuity, the negative moments increase due to difference in shrinkage between the deck and the girder. This increases the potential for transverse cracking in the deck. Also, if the live loads are applied immediately after the continuity is established, high negative moments will develop. In several of the analyses conducted, the negative cracking moment as defined by the AASHTO was exceeded. After negative cracking moment occurs, the negative moment does not increase due to creep effects. Therefore, negative moment continuity is reduced when negative moments exceed the cracking moment.

Effect of Construction Sequence

Restraint moments at supports are highly dependent on the time of construction. A Primary factor is the girder age at the time of continuity by the construction of cast-in-place deck and diaphragm. For the purposes of parametric study, PBEAM runs were conducted for deck and diaphragm cast simultaneously at girder ages of 17 or 67 days. Some comparisons were made with deck cast before the diaphragm and vice versa. When the deck is cast before the diaphragm, end rotations due to shrinkage between deck and girder is unrestrained until the diaphragm is in place. As a result, the negative restraint moment component and the potential for transverse cracking in the deck are reduced. On the other hand, if the diaphragm is cast before the deck, resultant moments are only slightly affected. In this sequence, the diaphragm has attained a higher strength at the time the deck is cast. Therefore, the diaphragm resists some negative moment induced by the deck dead load and develops higher negative restraint moment from differential shrinkage between the newly cast deck and the restrained girder. This sequence increases the probability of establishing full continuity and reducing resultant positive moments at midspan but only slightly.

RESULTS OF PARAMETRIC STUDY FOR FLEXURAL STRENGTH

In bridges constructed of prestressed concrete girders with composite cast-in-place decks, the positive moment region within the span is inherently much more ductile than the negative moment region at the supports. The large compressive area of the concrete deck in the positive moment region ensures that yielding of prestressing steel occurs before crushing of concrete. The negative moment region has only the bottom flange area and web of the prestressed girders available for compression. Because of this relatively small compression area, crushing of concrete in the bottom of the girder occurs before yielding of the reinforcement in the deck. The program WALL_HINGE was used to establish criteria to limit the amount of reinforcement in the deck to insure ductile behavior.

Results of Analytical Study for Flexural Strength

Different cross sections were analyzed using the program WALL_HINGE and compared with the method presented by Park and Paulay. Table 9 shows the required central support rotation, and the rotation capacities calculated using the program. The required amounts of rotation decrease with increasing amounts of reinforcement. Some of the results obtained using the program are shown in Tables 9 and 10. Fig. 10 shows the ductility ratio plotted against the reinforcement ratio. The data show a strong trend of decreasing ductility with increasing reinforcement ratio.

Performance of PCA Method

The accuracy of PCA method was compared with PBEAM analysis, and the results are shown in Tables 11 and 12. It can be seen that for some values of age at continuity, the discrepancies between PBEAM and PCA analyses is too big. The effect of differential shrinkage is not efficiently treated in the PCA method and this fact becomes significant as the age at continuity increases.

PROPOSED ANALYSIS PROCEDURES

In order to determine the design moments for service conditions, restraint due to creep and shrinkage should be evaluated to determine the appropriate degree of structural

TABLE 9 - ROTATIONS AT CENTRAL SUPPORT - REQUIRED AND WALL_HINGE RESULTS

Girder	Span Length, ft	Concrete Strength, psi	Deck Reinf. Yield, psi	Deck Reinforcement, in.^{2}					
				8		16		24	
				θ_r, rad.	θ_c, rad.	θ_r, rad.	θ_c, rad.	θ_r, rad.	θ_c, rad.
Cal. 66	100	6500	60000	0.0054	0.0103	0.0035	0.0060	0.0021	0.0039
AASHTO-IV	100	6500	60000	0.0075	0.0116	0.0055	0.0091	0.0039	0.0075
AASHTO-VI	130	6500	60000	0.0076	0.0185	0.0059	0.0126	0.0045	0.0096
BT72/6	130	6500	60000	0.0076	0.0097	0.0054	0.0069	0.0037	0.0028
BIV-48	90	6500	60000	0.0080	0.0180	0.0058	0.0137	0.0040	0.0067
Cal. 66	100	5000	60000	0.0061	0.0093	0.0040	0.0045	0.0026	0.0029
AASHTO-IV	100	5000	60000	0.0085	0.0096	0.0062	0.0082	--	--
Cal. 66	100	6500	40000	0.0061	0.0162	0.0049	0.0108	0.0039	0.0060
AASHTO-IV	100	6500	40000	0.0083	0.0186	0.0069	0.0109	0.0058	0.0087

TABLE 10 - COMPARISON OF ROTATION AND DUCTILITY RATIOS

Girder	Concrete Strength, psi	Deck Reinf. Yield, psi	Deck Reinf. Area, in.^{2}	Rotation Ratio, θ_c/θ_r	Ductility Ratio, ϕ_u/ϕ_y	ρ/ρ_b
Cal. 66	6500	60,000	8	1.91	5.2	0.27
			16	1.71	3.3	0.54
			24	1.86	1.4	0.81
AASHTO-IV	6500	60,000	8	1.55	5.4	0.20
			16	1.65	4.1	0.41
			24	1.92	3.1	0.61
AASHTO-VI	6500	60,000	8	2.43	8.4	0.17
			16	2.14	5.5	0.33
			24	2.13	4.2	0.50
BT72/6	6500	60,000	8	1.27	4.7	0.26
			16	1.28	3.0	0.52
			24	0.76	1.0	0.78
BIV-48	6500	60,000	8	2.25	8.3	0.21
			16	2.36	6.0	0.41
			24	1.68	2.6	0.63
Cal. 66	5000	60,000	8	1.52	4.4	0.33
			16	1.13	3.0	0.66
			24	1.12	0.9	0.99
AASHTO-IV	5000	60,000	8	1.13	4.5	0.25
			16	1.33	3.5	0.50
Cal. 66	6500	40,000	8	2.66	11.6	0.16
			16	2.20	6.2	0.33
			24	1.54	3.4	0.49
AASHTO-IV	6500	40,000	8	2.24	12.8	0.13
			16	1.58	7.5	0.25
			24	1.50	5.5	0.38

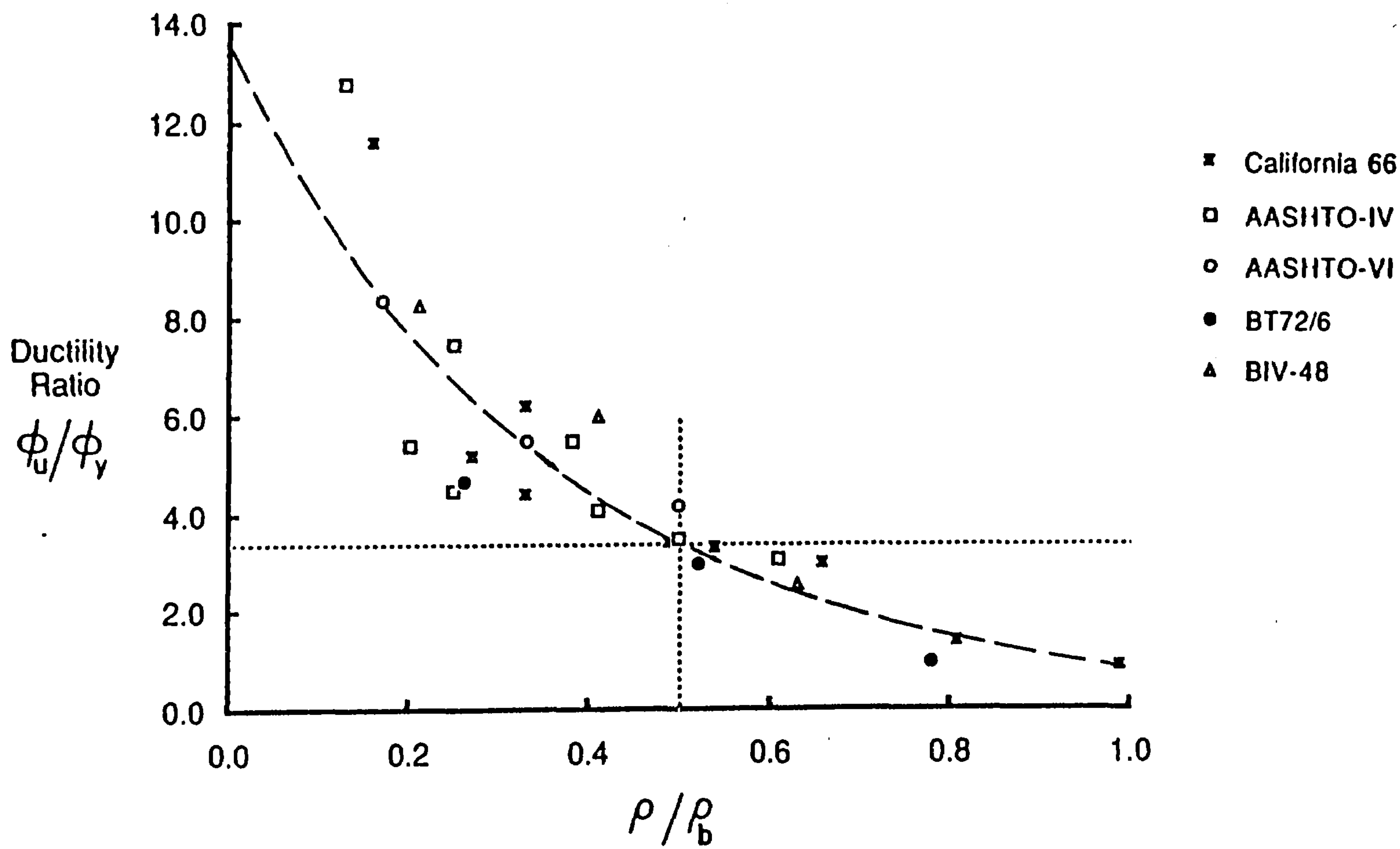


Fig. 10 Ductility Ratio vs. Percentage of Balanced Reinforcement Ratio

TABLE 11 - COMPARISON OF RESTRAINT MOMENT RESULTS USING PCA METHOD AND PBEAM ANALYSIS

Age at Continuity, Days	Restraint Moment at B, kip-ft		Restraint Moment at C, kip-ft	
	PCA	PBEAM	PCA	PBEAM
17*	849	867	588	933
37*	705	253	491	341
67*	302	-333	222	-188
17**	849	658	588	842

*Area of positive reinforcement = 7.2 sq in.
**Area of positive reinforcement = 3.6 sq in.

TABLE 12 - COMPARISON OF RESULTANT MOMENTS WITH APPLICATION OF LIVE LOADS

Age at Continuity, Days	Moment at B, kip-ft		Moment at Midspan of BC, kip-ft		Moment at C, kip-ft	
	PCA	PBEAM	PCA	PBEAM	PCA	PBEAM
17*	402	-195	5421	5458	-753	138
37*	258	-833	5301	4833	-850	-522
67*	-145	-1525	4965	4343	-1119	-809
17**	402	-16	542	5558	-753	158

*Area of positive reinforcement = 7.2 sq in.
**Area of positive reinforcement = 3.6 sq in.

continuity. The computer program PBEAM is capable of carrying out the complex analyses accounting for the time-dependent effects. However, use of PBEAM is very cumbersome, time consuming, and requires a large amount of computer memory. There was a need for an improved simplified analysis procedure for use in design.

It is difficult to reproduce the results predicted by PBEAM in a simplified analysis method. For instance, the degree of continuity which depends on opening and closing of cracks, had to be replaced by simpler assumptions. The simplest method is to assume full structural continuity for calculation of both live load plus impact and time-dependent restraint moments.

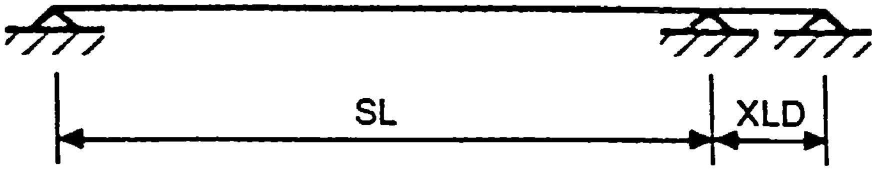
A computer program was developed as an improved method to predict restraint moments at redundant supports of bridges constructed of precast prestressed girders made continuous (program BRIDGERM). A second program (BRIDGELL) was developed to calculate maximum moments under AASHTO HS live load plus impact specifications.

Calculation of Restraint Moments

The program BRIDGERM was developed based on the PCA restraint moment calculation procedure, with some modifications. Separate time-dependent functions are provided for girder and deck time-dependent properties. The ultimate values of the creep coefficient and shrinkage strains for the deck and the girder are provided by the user. Creep and shrinkage functions are assumed.

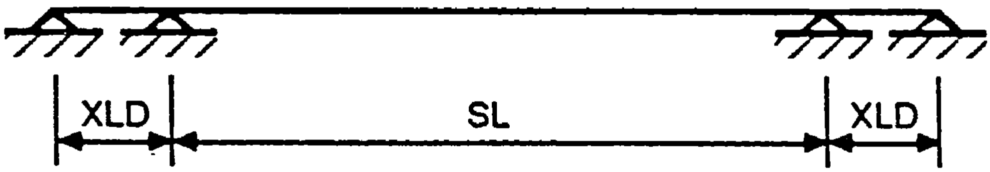
Additional modifications involve the use of a simplified bridge model for calculating the elastic restraint moments. A single interior span is supported by a double pinned support at each end, as shown in Fig. 11. The double pinned support accounts for the finite length of the diaphragm. For the purposes of elastic analyses, the ratio of sectional stiffness of the girder to sectional stiffness of the diaphragm is assumed to be equal to 1.

The analysis procedure in BRIDGERM involves superimposing restraint moment increments calculated over a series of time intervals. The user specifies the girder age at which continuity is established and the age at which the deck is in place. In general,



(a) Exterior Span

SL = Span Length
XLD = Diaphragm Length



(b) Interior Span

Fig. 11 Simplified Bridge Models for BRIDGERM Analysis these two ages should be assumed to be the same, unless there is a considerable age difference of several days. The modified procedure incorporates a time-dependent creep effect factor to account for the age at loading for the change in moment within each time step.

In order to determine the success of the simplified restraint moment calculation method, a comparison is made to the results of the PBEAM analyses. Tables 13 and 14 show comparisons of restraint moments. The BRIDGERM results show considerable improvement over PCA method, but still fall short of the accuracy range of PBEAM. Three typical restraint moment vs. time responses are shown in Fig. 12, 13 and 14. The BRIDGERM results are only marginally satisfactory. The discrepancy seems to be higher for early age of continuity. This marginal satisfaction is a compromise for having a very simple program such as BRIDGERM available to practicing engineers.

In general, the CTL method (improved PCA) method as implemented in the program BRIDGERM can be assumed to have reasonable agreement with PBEAM results. This method is an improvement in terms of the ease of use as well as accuracy of results over the PCA method.

Calculation of Live Load Plus Impact Moments

The program BRIDGELL was developed to calculate maximum live load plus impact moments for a continuous interior bridge girder loaded by AASHTO standard. An elastic continuous beam program was modified by addition of routines to generate AASHTO HS truck and lane loads. The bridge is analyzed for combinations of truck position, direction, and axle spacing. The user may specify load magnitudes other than HS20-44 by entering a multiplication factor, such as 1.25 to analyze for HS25-44 loading.

SUMMARY

This chapter presented the findings of the research for this project. Based on the literature, two programs PBEAM and WALL_HINGE were selected for evaluation of time-dependent behavior and strength of precast prestressed concrete girders made continuous. Creep and shrinkage tests were carried out to determine the time-dependent

TABLE 13 - RESTRAINT MOMENTS AT CENTRAL SUPPORT OF FOUR-SPAN BRIDGE - AGE OF CONTINUITY 17 DAYS

Ultimate Creep Coefficient,	Girder Type	Span Length, ft	PBEAM Moment @ 650 days ft-k	CTL Method Moment @ 650 days ft-k	PCA Method Moment @ 650 days ft-k
3.25	AASHTO-IV	70 100	217 403	275 392	40 131
	AASHTO-VI	100 130	313 678	340 771	-83 316
	BT72/6	100 130	275 1017	297 975	54 736
1.625	AASHTO-IV	70 100	-- 226	41 124	-185 -120
	AASHTO-VI	100 130	96 385	21 320	-352 -68
	BT72/6	100 130	102 697	26 425	-201 286

TABLE 14 - RESTRAINT MOMENTS AT CENTRAL SUPPORT OF FOUR-SPAN BRIDGE - AGE OF CONTINUITY 67 DAYS

Ultimate Creep Coefficient	Girder Type	Span Length, ft	Maximum Negative Moment ft-k		
			PBEAM	CTL Method	PCA Method
3.25	AASHTO-IV	70	-900	-668	-541
		100	-900	-655	-518
	AASHTO-VI	100	-1160	-916	-778
		130	-750	-861	-678
	BT72/6	100	-644	-785	-605
		130	-519	-706	-435
1.625	AASHTO-IV	70	-900	-738	-680
		100	-900	-727	-651
	AASHTO-VI	100	-1130	-1009	-979
		130	-750	-881	-851
	BT72/6	100	-634	-859	-761
		130	-566	-801	-622

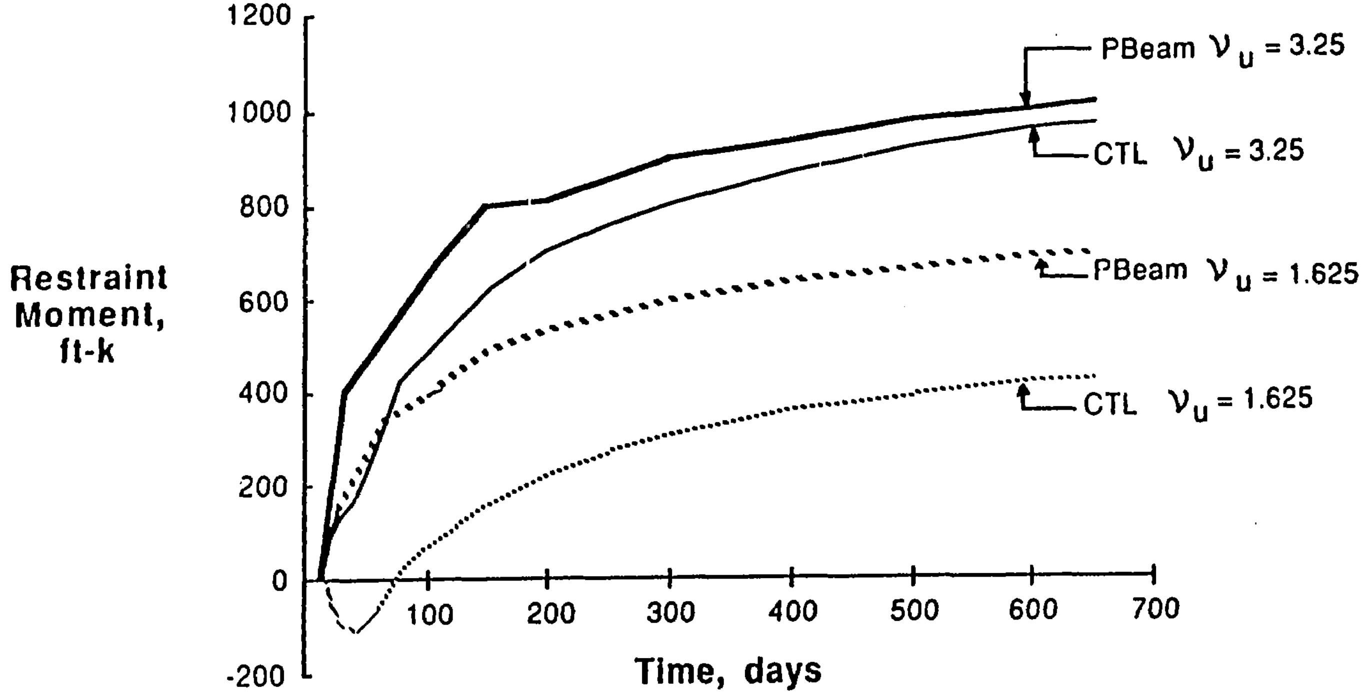


Fig. 12 Comparison of CTL Method and PBEAM Results - BT72/6, Early Age of Continuity

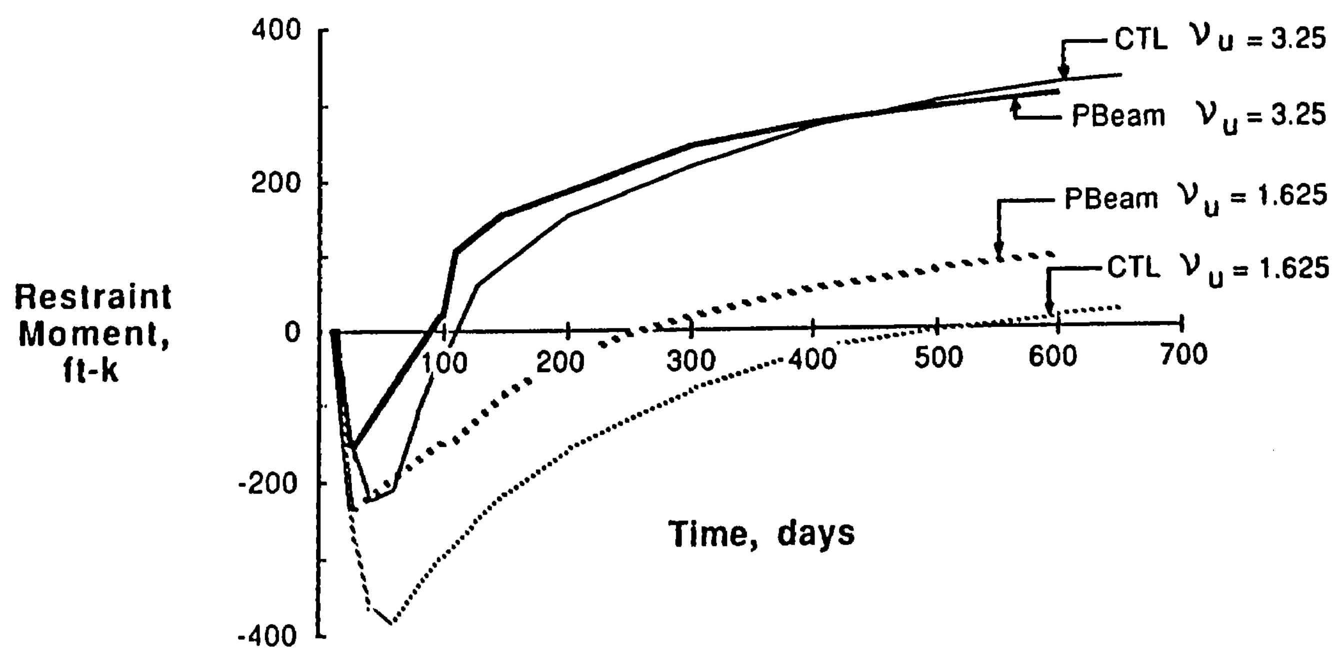


Fig. 13 Comparison of CTL Method and PBEAM Results - AASHTO-VI, Early Age of Continuity

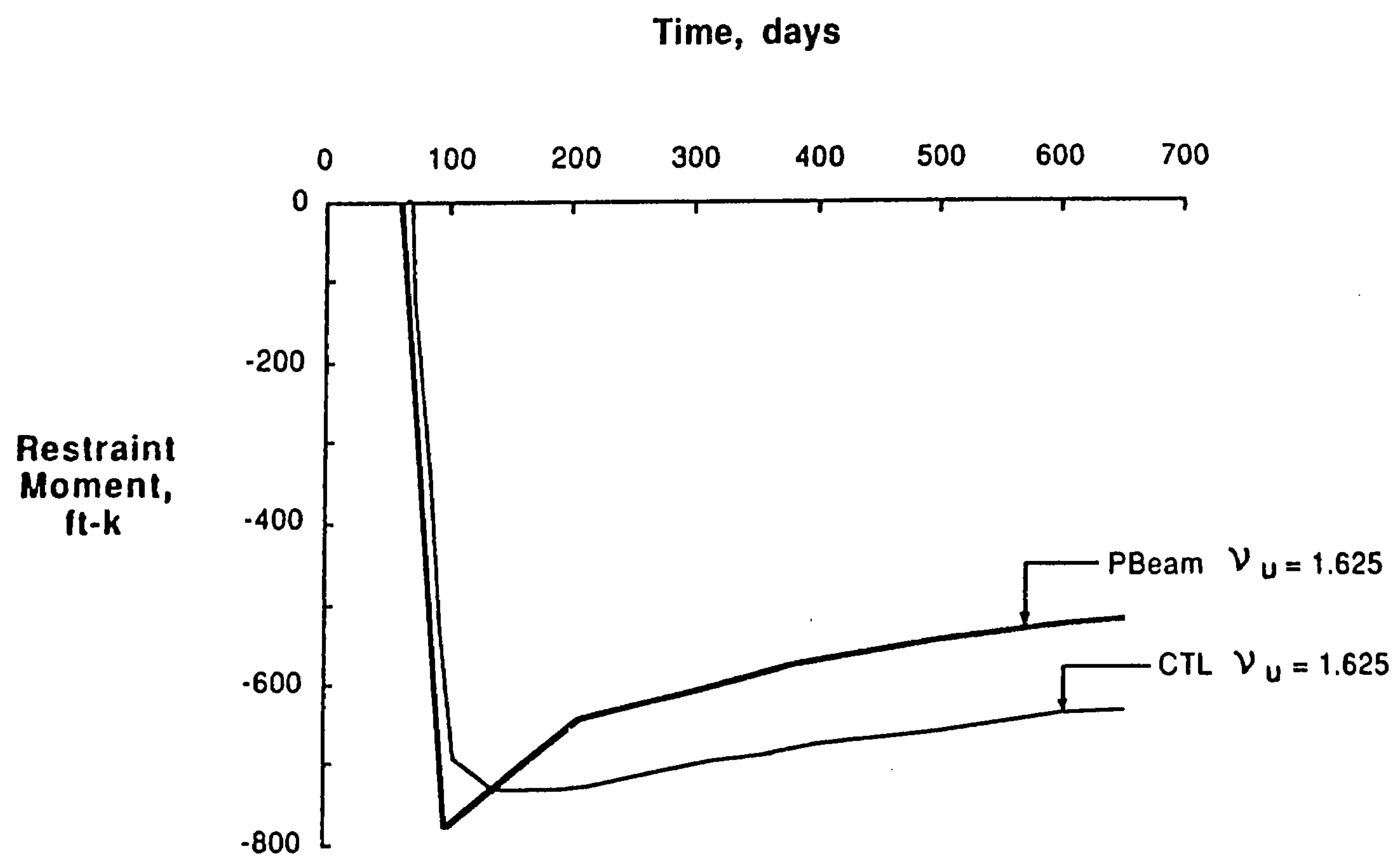


Fig. 14 Comparison of CTL Method and PBEAM Results - AASHTO-IV, Late Age of Continuity

properties of steam cured concrete.

Responses to questionnaire indicated that current practice for design and construction of simple span girders made continuous varies considerably. These responses were used to determine the range of design parameters for this type of bridge.

A parametric study using computer programs indicated that the effective continuity for live load can range from 0 to 100%. The presence of positive moment reinforcement has negligible effect on the reduction of mid span service moments. Construction timing has a major influence on the effective continuity. Maximum loss of continuity occurs with a combination of young girders at time of casting the deck and the diaphragm with late-age application of live load plus impact. Maximum negative moment and probability of transverse cracking in the deck over the supports occurs with older girders at time of casting the deck and diaphragm combined with early age application of live load.

A parametric study of the effect of amount of deck reinforcement and cross sectional shape of the girders indicated that the limit for negative ratio equal to 0.5 times the balanced reinforcement ratio. This is an important criterion to insure redistribution of moments and attainment of maximum strength of girders.

Based on the analysis using PBEAM, it was found that there is potential for a more economic design for positive midspan moments in the prestressed girder. A new simplified program BRIDGERM was developed using modifications to PCA method to calculate the time-dependent restraint moments. Another program BRIDGELL was developed to calculate the continuous moments due to applied live loads.

Based on the findings in this chapter, recommendations for analysis and design are presented in chapter 3.

CHAPTER 3

INTERPRETATION, APPRAISAL AND APPLICATION

DESIGN FOR SERVICE MOMENTS

Current AASHTO provisions for design of bridges constructed of prestressed girders made continuous do not specify how to consider the effects of creep and shrinkage. The following recommendations are based on the findings of the analytical studies presented in chapter 2.

Positive Moment at Supports

Providing positive moment reinforcement at the supports has no structural advantages. The Time-dependent positive restraint moment gradually induces a crack in the bottom of the diaphragm concrete. With the application of the live load, the positive moment crack must close prior to inducing negative moment at the continuity connection. Thus the presence of the positive moment reinforcement helps to maintain relatively small cracks, thereby increasing live load continuity. The presence of positive moment at the connections, increases the positive midspan resultant moment. Therefore, the resultant midspan moments which include moments due to dead load, restraint moments due to creep and shrinkage, and live load moments, are independent of the area of positive moment reinforcement provided in the diaphragm connections at the supports. Hence, providing positive moment reinforcement has no advantages for this type of bridge.

The primary advantage of providing positive moment reinforcement is in maintaining a smaller crack near the bottom of the diaphragm. There have been many bridges built without positive moment reinforcement and these bridges are performing well, as indicated by many respondents to the questionnaire. There were no serviceability problems associated with the lack of positive moment connections. Therefore, providing positive moment connections at supports is not recommended.

Positive Moments at Midspan

The positive moments at midspans consist of simple span moments due to girder and deck weight and moments acting on the continuous structure, including those due to additional dead load, live load plus impact, and time-dependent restraint moments at supports. Time-dependent behavior influences the continuous behavior such that the effective continuity for live load plus impact can vary from 0 to 100%. Therefore, time-dependent effects must be considered in the design if continuity is important, as discussed below.

Time-Dependent Restraint Moments

We have to first determine the restraint moments assuming full structural continuity. These restraint moments can then be added to support moments for the service load case, which causes the maximum midspan moment. Analysis for restraint moments assuming full structural continuity is applicable whether or not positive moment connections are provided at the supports.

Computer programs such as BRIDGERM can be used for calculating the restraint moments. To account for the variability of creep and shrinkage strains, the program uses the upper bound values of the time-dependent strains as a conservative measure. So, the time-dependent effects such as moments and support reactions predicted by using BRIDGERM may be higher than the observed values. It is also recommended that if the time-dependent properties of the deck and girder concrete are not known, ACI recommendations should be used for estimating the time-dependent strains.

The values of these restraint moments are added to the live load moments which produces the maximum positive midspan moment in the corresponding span. The values of these continuity moments at supports are used to determine the midspan service moments to be used for checking the allowable stresses. If the resultant moments at the supports are positive, the simple span moments at midspan for live load, dead load, and impact should be used with no reduction for effects of structural continuity. The recommended maximum negative continuity moment to be used for reduction of midspan positive moment is equal to 125% of the negative cracking moment of the section. The number 125% is obtained from results of the parametric study in which for girders with restraint moment at or above cracking, the resultant moments with application of live load reached approximately 125% of the cracking moment. The procedure described here has been used in the computer program BRIDGERM.

If no analysis is conducted to determine the time-dependent restraint moments at supports, the midspan moment should be taken as the simple span additional dead load and live load plus impact moment.

Live Load Plus Impact

One of the main recommendations of this study is that positive moment reinforcement at the supports need not be provided since its disadvantages of construction outweigh the advantages in serviceability. For the determination of live load plus impact moments on the continuous structure, the nature of continuity at supports must be considered for various configurations. If we consider the loading shown in Fig. 15, the negative moments occur at the two supports adjacent to the loaded span and positive moments occur at the next support in each direction. Since positive moment resistance is assumed to be zero, we are in effect, isolating the loaded span from the rest of the bridge. Similarly, for a loaded end span, the effective continuous structure is as shown in Fig. 15. Calculations of additional dead load and live load plus impact moments can be done using the program BRIDGELL. The program uses the procedure discussed here for computing live load plus impact moments. The continuous structure for a loaded interior span consists of three spans and two spans for a loaded end span, as shown in Fig. 15. For load configurations in which no positive moments occur, such as additional dead load, the entire bridge can be assumed to act as continuous.

Negative Moment at Supports

Negative moments at supports consist only of moments acting on the continuous structure including those due to additional dead load, live load plus impact, and time-dependent effects. At service load, AASHTO requires that the compressive stress at girder ends not exceed $0.6\ f'_c$. Since the negative moment region acts essentially as a conventional reinforced section, the reinforcement ratio should meet the strength requirements. It is recommended that the tensile stresses be checked based on restraint moments calculated at 50 days after establishment of continuity.

Negative moments from additional dead load and live load plus impact can be

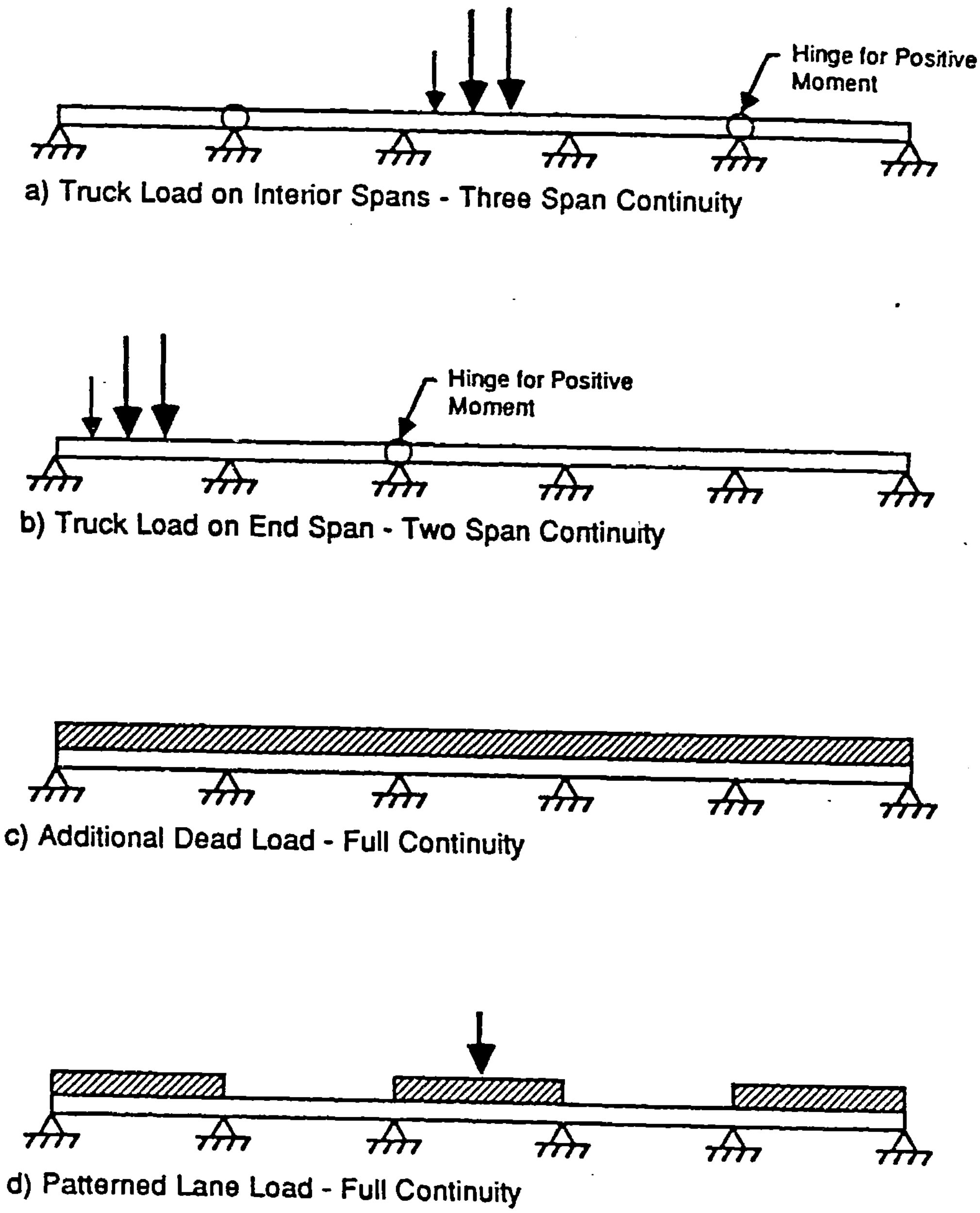


Fig. 15 Continuity for Determining Design Moments from External Loads

determined from analyses assuming full structural continuity. In general, maximum negative moments occur for patterns of AASHTO lane loading in which no positive moments occur at supports.

STRENGTH DESIGN

Time-dependent effects have no influence on the strength of a structure. Only the stresses and strains in a structure vary with time. Therefore, the design of the bridge to meet the strength provisions of AASHTO code need not include time-dependent effects. This is an additional measure of safety since one does not know the exact degree of continuity, which influences the maximum positive midspan moments.

Positive Moment at Midspan

As in any structural design, the girder strength must be greater than the maximum factored moments. The factored moments must include the dead load of the girder and the deck, additional dead load acting on the now continuous structure, live load and impact effects. The live load and impact effects must be based on AASHTO specifications. Program BRIDGELL calculates the moments this way. The restraint moments are not to be included in the positive midspan moment for strength design.

Negative Moment at Supports

Negative moment sections at supports must have flexural strengths greater than the maximum moment from appropriate factored loads. Minimum deck reinforcement shall provide strength equal to 125% of the negative cracking moment as defined in AASHTO specifications. To ensure ductile behavior to develop full strength, the deck reinforcement in negative moment regions shall not exceed 50% of the balanced reinforcement.

CONSTRUCTION SEQUENCE

Performance as a continuous structure is highly dependent on the age of the girder at at the time when the deck and diaphragm are cast. High restraint moments at supports occur when continuity is established at late girder ages. This is mainly due to differential shrinkage and creep strains that develop when deck concrete is cast on older girders.

The sequence of deck and diaphragm construction affects the development of restraint moments. Casting of the deck prior to the diaphragm increases the resultant positive moments at midspans but decreases the potential for deck cracking. Casting the diaphragm before the deck only slightly decreases the resultant midspan positive moments, but increases the potential for deck cracking. There is no advantage in casting one before the other. Simultaneous casting is the best construction procedure.

Full continuity can be established by delaying the casting of the deck and the diaphragm. However, delayed casting increases the negative moments and the the construction time.

PROPOSED REVISIONS TO AASHTO SPECIFICATIONS

Current Specifications

Current specifications for simple span prestressed girders made continuous are contained in article 9.7.2, which has been reproduced on the next page.

Proposed Revisions

Design procedures and findings from the report will be recommended to be incorporated into AASHTO "Standard Specifications for Highway Bridges", article 9.7.2. The recommendations are as follows;

(1) The first recommendation relates to the title of the article itself. *The new title for the article 9.7.2 will be "Bridges Composed of Simple Span Precast Prestressed Girders Made Continuous".*

(2) Article 9.7.2.2 (no change in title) will be revised as follows. *The new article 9.7.2.2 will state that provision of positive moment reinforcement at the support is not required.*

(3) Article 9.7.2.3 will describe the design for positive midspan moments for service and strength conditions.

Separate subsections will deal with service moment design and strength design.

The service moment design subsection will include a description of allowable stresses. The method of determining positive midspan design moments including appropriate degree of continuity will be described.

The strength design subsection will describe the determination of design moments.

(4) A new article 9.7.2.4 will consist of an expanded version of the current article 9.7.2.3, dealing with the design for negative moments at supports.

Separate subsections will deal with service moment design, strength design, and limits on quantity of deck reinforcement.

The service moment design subsection will include a description of allowable stresses and provisions for including negative restraint moment in negative service moment. The method of computing design negative support moments including the appropriate degree of continuity will be described.

The strength design subsection will consist primarily of current article 9.7.2.3. Upper and lower limits on the amount of deck reinforcement to provide sufficient ductility and strength will be specified in a third subsection of this article. The lower limit of providing reinforcement sufficient to develop strength equal to at least 125% of the cracking moment and definition of cracking moment will be described. Upper limit of $0.5\rho_b$ *and the definition of* ρ_b *will be described.*

9.5 EXPANSION AND CONTRACTION

9.5.1 In all bridges, provisions shall be made in the design to resist thermal stresses induced, or means shall be provided for movement caused by temperature changes.

9.5.2 Movements not otherwise provided for, including shortening during stressing, shall be provided for by means of hinged columns, rockers, sliding plates, elastomeric pads, or other devices.

9.6 SPAN LENGTH

The effective span lengths of simply supported beams shall not exceed the clear span plus the depth of the beam. The span length of continuous or restrained floor slabs and beams shall be the clear distance between faces of support. Where fillets making an angle of 45 degrees or more with the axis of a continuous or restrained slab are built monolithic with the slab and support, the span shall be measured from the section where the combined depth of the slab and the fillet is at least one and one-half times the thickness of the slab. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of the fillet shall be considered as adding to the effective depth.

9.7 FRAMES AND CONTINUOUS CONSTRUCTION

9.7.1 Cast-in-Place Post-Tensioned Bridges

The effect of secondary moments due to prestressing shall be included in stress calculations at working load. In calculating ultimate strength moment and shear requirements, the secondary moments or shears induced by prestressing (with a load factor of 1.0) shall be added algebraically to the moments and shears due to factored or ultimate dead and live loads.

9.7.2 Bridges Composed of Simple-Span Precast Prestressed Girders Made Continuous

9.7.2.1 General

When structural continuity is assumed in calculating live loads plus impact and composite dead load moments, the effects of creep and shrinkage shall be considered in the design of bridges incorporating simple span precast, prestressed girders and deck slabs continuous over two or more spans.

9.7.2.2 Positive Moment Connection at Piers

9.7.2.2.1 Provision shall be made in the design for the positive moments that may develop in the negative moment region due to the combined effects of creep and shrinkage in the girders and deck slab, and due to the effects of live load plus impact in remote spans. Shrinkage and elastic shortening of the pier shall be considered when significant.

9.7.2.2.2 Non-prestressed positive moment connection reinforcement at piers may be designed at a working stress of 0.6 times the yield strength but not to exceed 36 ksi.

9.7.2.3 Negative Moments

9.7.2.3.1 Negative moment reinforcement shall be proportioned by strength design with load factors in accordance with Article 9.14.

9.7.2.3.2 The effect of initial precompression due to prestress in the girders may be neglected in the negative moment calculation of ultimate strength if the maximum precompression stress is less than $0.4f'_c$ and the continuity reinforcement, p, in the deck slab is less than 0.015; where $p = A_s/bd$.

9.7.2.3.3 The ultimate negative resisting moment shall be calculated using the compressive strength of the girder concrete regardless of the strength of the diaphragm concrete.

9.7.2.4 Compressive Stress in Girders at Piers at Service Loads

The compressive stress in ends of girders at piers resulting from addition of the effects of prestressing and negative live load bending shall not exceed $0.60f'_c$.

9.7.3 Precast Segmental Box Girders

9.7.3.1 General

9.7.3.1.1 Elastic analysis and beam theory may be used in the design of precast segmental box girder structures.

9.7.3.1.2 In the analysis of precast segmental box girder bridges, no tension shall be permitted across any joint between segments during any stage of erection or service loading.

9.7.3.1.3 In addition to the usual substructure design considerations, unbalanced cantilever moments due to segment weights and erection loads shall be accommodated in pier design or with auxiliary struts. Erection equipment which can eliminate these unbalanced moments may be used.

SUGGESTED FURTHER RESEARCH

INTRODUCTION

The long term performance of concrete bridges (reinforced and prestressed) of different types of construction is of primary importance in the state and interstate highway system. These bridges, once designed and built, are expected to perform well against variations in loads and environmental conditions such as temperature and humidity. In addition to time-dependent effects of concrete (such as creep and shrinkage), other factors such as degree of enclosure, exposure to prevailing winds and sunlight, influence the performance of bridges.

Often, so much time is spent in refining the design methods to economize the amount of steel that designers tend to forget effect of thermal stresses. The NCHRP report discusses methods of calculating time-dependent moments but does not address the calculation of long term deflections at all. Frequently, design procedures developed for one part of the country do not work for others due to change in climate and other factors. One needs practical data to verify the applicability of design equations and to calibrate some of the empirical constants of these equations.

One therefore has to concentrate on the problems hitherto not addressed, with particular reference to the State of Arizona. It is worthwhile to develop analytical models for predicting long term deflections and for analyzing temperature effects in concrete bridges. Since practical data on deflections and thermal effects in bridges in Arizona are scarce, it is very important to collect these data by instrumenting several bridges. The data so collected will be very helpful in calibrating some of the empirical constants of the analytical models as well as verifying their applicability.

After reviewing the literature up to date including the NCHRP report, the following topics in the authors' opinion warrant further investigation. Considering the fact that a good number of bridges are being built or planned to be built in the near future, these research topics must be assigned higher priority as their outcome will very much be useful in the design and construction of bridges in Arizona.

(1) Development of an Analytical Model for Long-Term Deflections of Precast Prestressed Concrete Bridge Girders Made Continuous.

(2) Monitoring Several Concrete Bridges in Arizona for Long-Term Deflections.

(3) Evaluation of Temperature Effects in Concrete Bridges and Their Quantification by In situ Measurements

Briefly, the first research topic should consist of developing a mathematical model for predicting time-dependent deformations for precast prestressed concrete bridge girders made continuous on the site. The model developed should be such that the results from the NCHRP 12-29 project are utilized. This includes the computations of time-dependent moments taking into account the degree of continuity. The time-dependent effect of live loads not considered in the NCHRP 12-29 investigation should also be included in the computation of long-term deflections.

The validity of any analytical model can be ascertained by comparing its predictions with some practical observations. This is more important for analytical models predicting the bridge behavior such as deflections. Practical bridge deflections data are very scarce. These data are extremely valuable as they truly reflect the local effects on the bridge behavior. If the analytical models contain any empirical constants, the best way to determine them is by using the these data collected from different bridges.

One of the effects frequently overlooked by designers is the temperature effect on bridge behavior. A combination of high temperature and low humidity can often cause effects severe than those caused by loads. The analysis of thermal stresses and their effects on bridges is a complex topic, but is very much necessary in the state of Arizona. Analytical models need to be developed for analyzing the effects of thermal stresses. Practical observations on temperature variations in bridges are necessary to calibrate any empirical parameters of the models developed for thermal effects.

CONCLUSIONS

The following conclusions have been reached based on the analysis presented in the NCHRP 12-29 report. Comments and criticisms of the authors of this report are written in *italics.*

(1) Precast prestressed concrete girders made continuous on the site are very commonly used in many road and highway bridges. Construction of these bridges involves a positive moment connection at the piers, which is difficult, time consuming and expensive to design and construct.

This is something that needs to be confirmed with designers and construction engineers. We have to ascertain whether this problem really exists for bridge construction in Arizona.

(2) The current procedures for analysis, design and construction of this type of bridge is not very clear and varies widely within the United States. The current AASHTO specifications are not very clear. Most states use the PCA procedure which does not adequately handle some situations that exist in the current practice of construction timing and sequence.

Results from the survey of several DOTs presented in Appendix -A of the NCHRP 12-29 report, support this observation. It is generally agreed that there is indeed a need for unified design and analysis procedures for this type of bridge.

(3) Creep and shrinkage properties of steam cured concrete are not readily available. Tests conducted in this project have added significant amounts of data. Test results indicate that the ACI model predicts time-dependent strains for this type of concrete fairly accurately.

(4) The computer program PBEAM and WALL_HINGE were used for the time-dependent and live load (including impact) analyses of bridges. These programs were used after obtaining 'satisfactory' agreement with existing experimental results.

Based on the computer simulation studies, some important conclusions regarding the role of positive reinforcement have been made. However, it should be mentioned here that there is no test data on the effect of positive moment connection reinforcement to compare with PBEAM results. One can therefore question the validity of this computer program for studying the effects of positive moment reinforcement at the connections.

(5) Computer simulation studies of service moments indicated that there is no structural advantage for providing positive moment reinforcement at the supports. When the bridge is made continuous and the live loads are not yet applied, the presence of positive moment reinforcement at the supports increases the positive midspan moments in addition to maintaining a smaller crack width at the bottom of diaphragm. However, when full live load is applied, the positive moment reinforcement is in the compression zone and offers no structural advantage. Therefore, the resultant midspan moments are essentially independent of the area of positive moment reinforcement provided at the connections.

These conclusions are based on the results from the program PBEAM. It was commented earlier (chapter 2) that not all bridges analyzed using this program had the same resultant midspan moments at application of 100% live loads. The fact remains that PBEAM has not been verified with test results involving positive moment reinforcement at the connections. It is therefore not plausible to generalize the behavior of positive moment reinforcement based on a computer program study. It is also not known as to what type of theoretical treatment has been used in the program PBEAM, especially on the time-dependent effects of positive reinforcement. Full details of this program are not given in the NCHRP 12-29 report. Until another analysis using a different computer program is made, **this recommendation has to be treated with caution.**

(6) The performance of the bridge as a continuous structure is highly dependent on the age of the girder when the diaphragm and the deck are cast. By delaying the construction of the deck and the diaphragm, one increases the differential shrinkage and creep strains (between the older girder and freshly placed deck and diaphragm) which may lead to cracking. Even though this delay causes high support moments (which decrease the midspan positive moments), it is generally not

advantageous to delay the construction of the deck and the diaphragm.

This observation is a valid one. Cracking caused due to differential time-dependent strains between the girder and the deck outweighs the advantages of lower positive midspan moments obtained by delaying the construction of the deck and diaphragm. So there is no advantage of delaying the construction of deck and the diaphragm.

(7) The sequence of deck and diaphragm construction affects the development of restraint moments. Casting of the deck prior to the diaphragm increases resultant positive moments at midspan, whereas casting of the diaphragm prior to the deck slightly decreases the resultant midspan positive moments. There is no major economic or construction advantage in casting the deck and the diaphragm at different times. Simultaneous casting is the simplest solution.

(8) Computer simulation studies using the program WALL_HINGE indicated that an upper limit of 50% of ρ_b ensures sections with sufficient rotational ductility to develop the full failure mechanism as well as enough deformation to give adequate warning of failure.

(9) The PCA method originally developed in 1969, was improved so that it can deal with more practical situations and eliminate some uncertainties. Based on the improved procedure, two computer programs BRIDGELL and BRIDGERM were developed. These programs calculate time dependent restraint moments of the continuous structure due to dead loads (BRIDGERM) and resultant moments upon application of the live load (BRIDGELL).

The programs developed are based on improvements to the PCA method which was originally developed in 1969. Since then, there have been other methods developed, most notably the Creep Transformed Section Method, which can be used to analyze continuous structures. The performance of the programs developed are only marginally satisfactory. It is felt that simple yet accurate procedures can be developed to predict restraint and live load moments in continuous bridges, based on more recent literature on time dependent analysis of continuous bridges.