FINAL REPORT

BRIDGE LENGTHS:

JOINTLESS PRESTRESSED GIRDER BRIDGES

by

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DISCLAIMER

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ABSTRACT

In 1984, the state of Alaska designed a jointless highway bridge to span the Maclaren river. The Maclaren River Bridge is on a 0.552% grade with an overall length of 361 ft - 11 in that is divided into three spans of 119 ft - 11 in, 120 ft - 7 in, and 119 ft - 11 in. The roadway width between guardrails is 28 ft. The superstructure consists of 5 prestressed concrete girders supported by elastomeric bearing pads. The prestressed girders are encased in concrete diaphragms at the piers and abutment backwalls at the ends.

A 1989 AKDOT&PF bridge inspection report shows that cracks exist in the abutment backwalls and concrete diaphragms at the piers. It was the objective of this study to conduct a literature review for these types of structures, compile the experiences from other states, and perform a thermal analysis to assess the cause.

The magnitude of the thermal stresses and induced forces in the structure due to weather are independent of length but dependent on the exposure, geometry, materials, and the substructure support restraints. It is possible that the backfill materials at the abutment may have collected moisture and frozen, thereby causing a large resistance to movements. The results from the survey show that most states set a maximum length of about 300 to 400 ft.

A literature search on jointless bridges, national survey of DOT experiences with these bridges, and a thermal analysis was used to examine this phenomena. The results of the study indicate that these types of cracks have been found in similar bridges in other states. The probable cause originates from the restraints imposed by the stiffness of the end bents.

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INTRODUCTION

A 1989 Alaska DOT&PF bridge inspection report shows that concrete cracks were observed in abutment backwalls and pier diaphragms in the Maclaren River Bridge. This structure is 8 year old, prestressed, jointless, 3 span, zero-degree skew, and is on a 0.552% grade. The width between guardrails is 28 ft. The spans are 119'-11", 120'-7", and 119'-11" for a total length of 361'-11". Five prestressed decked bulb-tee girders, resting on elastomeric bearing pads, were used for the superstructure. The ends of each girder are encased in concrete by a cast-in-place abutment backwall at the ends of the bridge and diaphragms at the piers. A portion of the plans for this structure are given in Appendix B. It is appropriate to note that a shear key exists between the abutment cap and the abutment backwall. Pictures of the distress are presented in Appendix C.

<u>It is the purpose of this report to summarize the knowledge to date for jointless prestressed girder bridges.</u> Information was gathered in the following manner: a) a survey was sent to the bridge section of 50 state DOTs; b) a literature review was prepared; and c) a thermal analysis was made for this bridge.

The survey consisted of questions to determine: the states that use this type of structure; restrictions on bridge length; suggested support details; experiences with maintenance; and assessment of this type of structure as a choice. A copy of the questionnaire is given in Appendix A. The thermal analysis was conducted to possibly help explain why backwall and diaphragms have experienced cracks. The analysis is based on the assumption that weather at the Maclaren River Bridge is similar to Fairbanks, Alaska weather. Using this assumption, the structure was subjected to 5 and 50 year exposures. Temperature distributions for these two design periods are presented. Further, thermal slab stresses and abutment induced forces are given as a function of abutment stiffness. This will allow AKDOT&PF to examine alternatives and predict resulting effects.

LITERATURE REVIEW

Two design approaches are used by bridge engineers to account for thermal effects: expansion devices and jointless decks. The conventional design approach is based on the assumption that bridge deck expansion devices and expansion bearings allow bridges to expand or contract freely without restraint. It is common to find improperly tilted and frozen bearings, inoperative expansion devices and distressed appurtenances; these are examples that free movement does not exist(12). Some states design bridges with jointless decks supported by bearings and/or flexible bents (6,12,13,14). In either case, Emanuel and Taylor(15) showed that the length between expansion joints does not influence stress inducement. Rather, thermal stresses are affected by shape and magnitude of the thermal gradient, superstructure geometry, materials, and restraints imposed by connections and substructures.

Methodology for calculating movements and stresses involves three steps: **1)** characterizing the climatic exposure; **2)** determining structural temperature changes with respect to conditions at time of construction; and **3)** calculating deformations, and induced thermal strains and stresses (2,4,9,14,16,17,18,19).

Most research to date has focused on **[step 2]**: identifying temperature profiles for different bridges of various exposures, or **[steps 2,3]** assessing stresses and movements for different bridge types. Bridge temperature distributions presented in the literature have usually been based on: experimental site data (16,17,20,21,22); calculations from limited periods of site measured exposures (19); or laboratory studies (15,18,23,24,25). In other cases, climatological data have been used to identify extremes **[step 1]** for the purpose of calculating temperatures and induced movements or stresses **[steps 2,3]** **(**2,3,4,5,6). Others suggested polynomials to approximate temperature profiles for concrete bridges (8,9,21), and composite bridges (8,26). These approximate temperature profiles do not provide for differences in climate and have no measured return period.

In summary: a) weather induced thermal stresses can be large and should be considered in design; b) there is a lack of understanding of the interaction between weather, and induced movements and stresses; and c) AASHTO gives limited guidelines to account for movements with no guidelines for thermal stresses and no provision for regional climates and design periods.

NATIONAL SURVEY

A ten question survey was sent in July, 1992, to Bridge Design Sections of each state DOT. The survey was prepared to collect information for the purpose of reviewing some of the design limitations on jointless bridges and comparing maintenance experiences with jointless bridges. A copy of the letter of transmittal and questionnaire is in Appendix A. Between July and September, forty four states responded. During the week of October 7, 1992, two more responses (Tennessee and Wisconsin) were received but are not included in the compilation. Only responses to pertinent questions (2,3,7 and 9) are presented in this report. The answers to these questions are given below.

Question 2: Does your state have jointless bridges?

2a) If yes, what percentage of new bridges are jointless?

All forty four states answered this question. Responses show that 72.73% of the states use these type of structures for new bridges. Further, 22.73% of the states use over 60% of these type of structures in new bridge construction, see Table 1.

TABLE 1. SUMMARY OF RESPONSES FOR QUESTION 2

Percent of Jointless Bridges	Number of States Responding	Percent of Sample
N/A	10	22.73
0%	2	4.54
1-20%	13	29.54
21-40%	3	6.82
41-60%	6	13.64
61-80%	6	13.64
81-100%	4	9.09
Total Response:	44	100%

Question 3: What is the maximum bridge length your state allows for jointless structures?

The length of the Maclaren River Bridge is 361.92 ft. Some states did not answer this question. However, 13.63% of the responses restrict the length of prestressed concrete girder jointless bridges to 301-400 ft and 45.94% allow bridge lengths over 300 ft, see Table 2. The shaded areas in Table 2 shows the responses to prestressed concrete girder jointless bridges with lengths between 301-400 ft (range of the Maclaren Bridge). States which allow jointless prestressed girder bridges to exceed 300 ft are shown as shaded in Table 3.

TABLE 2. RESPONSES TO QUESTION 3

	Number of States Responding							
		Question 3: Maximum Allowable Bridge Lengths, ft						
Type of Bridge Structure	N/A	< 201	201-300	301-400	401-500	501-600	> 601	Total
a) Prestressed conc. girders	7	6	7	6	4	3	4	37
b) Concrete on steel girders	5	9	12	7	2	0	0	35
c) Steel bridges (orthotropic)	29	1	1	2	0	0	0	33
d) Concrete T-beams	21	4	2	4	2	0	1	34

Question 7:

a) Do you use an integral system?
b) Do you use a nonintegral system?
c) Do you use a semi-integral system?

Thirty-three of the 44 states responded to this question. Twenty-five responding states use integral abutments, 19 use nonintegral abutments and 11 use semi-integral abutments. Six states use all three types and 9 use two of the three, see Table 3. Note, for states that allow maximum bridge lengths over 300 ft, 12 use integral abutments, 8 use nonintegral and only 3 use semi-integral. Further, for bridges over 300 ft, 5 use only integral abutments, 5 use integral or nonintegral, 2 use all three abutment types, 1 uses only nonintegral and 1 uses only semi-integral.

TABLE 3. RESPONSES TO QUESTIONS 2, 3, AND 7

State	Response Date	Q2: %	Question 3: Maximum Bridge Length, ft				Question 7: Abutment Type		
			3a	3b	3c	3d	7a	7b	7c
Alabama	7/29/92	N/A							
Alaska									
Arizona	8/10/92	0							
Arkansas	N/A								
California	8/3/92	10	200	200	N/A	200	yes	no	yes
Colorado	8/24/92	60	**400**	400	400	400	N/A	N/A	yes
Connecticut	8/14/92	2	N/A				yes	no	no
Florida	8/18/92	N/A							
Georgia	8/3/92	80	300	300	N/A	300	no	yes	no
Hawaii	8/3/92	< 10	**330**	N/A	N/A	40	N/A		
Idaho	8/12/92	40	**400**	300	N/A	N/A	yes	yes	no
Illinois	8/14/92	60	300	200	N/A	300	yes	no	no
Iowa	8/3/92	75	**500**	500	N/A	N/A	yes	no	no
Kansas	8/24/92	65	**400**	300	N/A	450	yes	no	no
Kentucky	8/14/92	75	**400**	300	N/A	400	yes	no	no
Louisiana	8/3/92	N/A	**600**	350	N/A	N/A	no	yes	no
Maine	8/8/92	N/A	150	80	N/A	150	yes	yes	yes
Maryland	8/12/92	0	**1500**						
Massachusetts	8/12/92	< 1	80	350	N/A	N/A	no	yes	yes
Michigan	8/11/92	50	N/A	N/A	N/A	N/A	no	yes	no
Minnesota	7/29/92	> 10	150	150	N/A	N/A	no	yes	no
Mississippi	7/27/92	N/A							
Missouri	8/3/92	90	**600**	500	N/A	N/A	yes	yes	no
Nebraska	7/27/92	90	**700**	350	N/A	N/A	yes	yes	no
Nevada	7/28/92	5-10	100	100	100	N/A	yes	yes	N/A
New Hampshire	8/10/92	25	100	100	N/A	N/A	no	yes	yes
New Jersey	8/17/92	N/A							
New Mexico	8/3/92	50	**450**	300	N/A	450	yes	yes	yes
New York	8/17/92	30	N/A	N/A	N/A	N/A	yes	yes	yes
N Carolina	7/28/92	< 1	N/A	N/A					
N Dakota	8/3/92	90	**400**	400	400	400	yes	no	no
Ohio	8/10/92	N/A	300	300	N/A	N/A	yes	yes	yes
Oklahoma	9/1/92	20	300	240	N/A	N/A	yes	no	no
Oregon	8/14/92	> 50	**1100**	170	N/A	1100	yes	yes	N/A
Pennsylvania	8/3/92	1	**600**	400	N/A	N/A	yes	no	no
Rhode Island	9/1/92	N/A							
S. Carolina	8/17/92	5	300	300	N/A	N/A	yes	no	no
S. Dakota	8/3/92	95	**700**	350	N/A	N/A	N/A		
Texas	8/3/92	< 10	N/A	N/A	N/A	120	N/A	N/A	N/A
Utah	8/3/92	80	300	300	N/A	N/A	yes	N/A	N/A
Vermont	7/28/92	60	N/A	90	N/A	N/A	yes	yes	yes
Virginia	8/3/92	5	**500**	300	N/A	N/A	yes	yes	N/A
Washington	8/4/92	5-10	**450**	300	300	400	yes	yes	yes
W Virginia	8/3/92	N/A							
Wisconsin	8/7/92	80	300	150	N/A	N/A	yes	yes	yes
Wyoming	8/24/92	N/A	N/A	300	N/A	N/A	yes	no	no

Question 9: Please indicate your assessment of maintaining the jointless type of bridge? Please note any problems that have been found and indicate the solutions.

A summary of the responses to this question are listed in Table 4. The results of the survey show that four states are having difficulties with end diaphragms. These are Oregon, Texas, Washington, and Wyoming. Other problems that may be of interest were presented by California, and Minnesota.

SUMMARY

The following paragraphs attempt to summarize the findings resulting from the questionnaire. Further, a table from the state of Wisconsin showing bridge length criteria is presented for consideration, see Table 5. The table shows bridge lengths, geometry and substructure considerations.

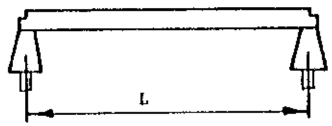
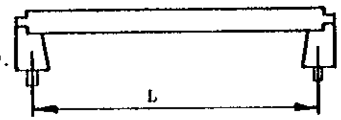
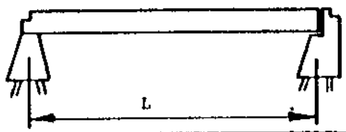
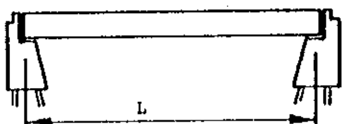
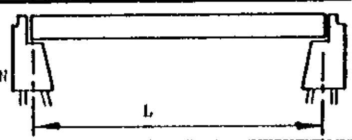
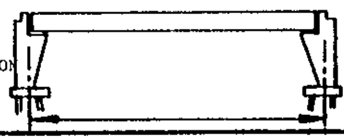
The responses to the survey show that _73% 1 of state DOT's use jointless bridges. Approximately 46% of the states allow jointless prestressed girder bridge lengths over 300 ft long. Most of the states with this type of bridge use either integral or nonintegral abutments. For bridge lengths over 300 ft, three states use semi-integral abutments with this type of bridge (Colorado, New Mexico, and Washington). The Maclaren River Bridge abutment detail is a semi-integral support.

Four states reported maintenance difficulties with concrete diaphragms. These states are Oregon, Texas, Washington, and Wyoming. According to the survey information, Texas and Wyoming no longer use jointless prestressed girder bridges. It is appropriate to point out that research by the author on composite-girder bridges show that integral abutments (rotationally restrained supports) cause large induced moments giving high thermal stresses when exposed to Fairbanks weather (5). Similar findings were also found for a Missouri climate (2).

TABLE 4. RESPONSES TO QUESTION 9

State	Question 9: List of Maintenance problems
Alabama	
Alaska	
Arizona	
Arkansas	No serious problems
California	Cracking in integral; More success with semi-integral
Colorado	Movements with approach panels
Connecticut	Cracking with fixed post-tensioned concrete frames
Florida	
Georgia	No problems at this time
Hawaii	No problem found on the few bridges built
Idaho	Cracks and bumps at ends of bridge
Illinois	Transverse cracking at about 3 to 5 feet from abutment wall
Iowa	No problems with the integral abutment design
Kansas	Upward rotation of girders; Movements of approach panels
Kentucky	Less maintenance problems
Louisiana	Limited history
Maine	Research is being conducted to evaluate the projects
Maryland	Limited history with jointless bridges
Massachusetts	Limited history with jointless bridges
Michigan	Limited experience as of this date
Minnesota	Shear blocks tend to deteriorate; Approach panels move off
Mississippi	Movements with the approach panels
Missouri	Movements with the approach panels
Nebraska	No known problems
Nevada	
New Hampshire	Rough bump at the end of the deck
New Jersey	
New Mexico	No serious problems
New York	Minor cracking in the vicinity of the formed or saw-cut joint
N Carolina	No known problems
N Dakota	No known problems
Ohio	Approach panels move off
Oklahoma	Movements with the approach panels
Oregon	Elastic shortening; Diaphragm connection spall
Pennsylvania	Less problems with jointless bridges
Rhode Island	
S Carolina	Jointless bridges are new and maintenance not needed yet
S. Dakota	No significant problems yet
Texas	Movements with the end diaphragms
Utah	No serious problems
Vermont	Pavement distress has been the biggest problem
Virginia	Bridges have been served for seven years and no problems found
Washington	Movements with the end diaphragms
Wisconsin	No known problems
Wyoming	Movements with the end diaphragms

TABLE 5. WISCONSIN JOINTLESS BRIDGE DESIGN CRITERIA

ABUTMENT ARRANGEMENTS			SUPERSTRUCTURES		
			CONCRETE SLAB SPANS	PRESTRESSED GIRDERS	STEEL GIRDERS
L = Length of continuous superstructure between abutments			L = Length and S = Skew , AL = Abutment Length		
(1) TYPE A1 WITH FIXED SEAT		TYPE A1 WITH FIXED SEAT	L ≤ 300' S ≤ 30° AL ≤ 50'	d. L ≤ 300' S ≤ 15° AL ≤ 50'	L ≤ 150' S ≤ 15° AL ≤ 50'
(2) TYPE A1 WITH SEMI-EXP. SEAT		TYPE A1 WITH SEMI-EXP. SEAT	L ≤ 300' S ≤ 30° AL > 50'	L ≤ 300' S ≤ 40°	L ≤ 200' S ≤ 40°
(3) TYPE A2 WITH FIXED SEAT		TYPE A3 WITH EXPANSION BEARING	L > 300' with flexible Piers and S ≤ 30° AL ≤ 50'	300' < L ≤ 400' AL ≤ 50' and with flexible Piers	NOT USED
(4) TYPE A3 WITH FIXED BEARING		TYPE A3 WITH EXPANSION BEARING	NOT USED	NOT USED	L ≤ 200' S > 40°
(5) TYPE A3 WITH EXPANSION BEARING		TYPE A3 WITH EXPANSION BEARING	a. L > 300' with rigid Piers and S ≤ 30°	Exceeds above Criteria for (1) and (2) and (3)	L > 200'
(6) TYPE A4 WITH EXPANSION BEARING		TYPE A4 WITH EXPANSION BEARING	NOT USED	b. Based on Geometry and Economics	c. Based on Geometry and Economics

AIR TEMPERATURE EXTREMES

It is the purpose of this chapter to provide the reader with a methodology for selecting a range of air temperatures for Fairbanks as a function of design period.

SITE CLIMATIC DATA

Historical hourly weather surface observation data for the period of 1952-1976 were obtained on tape for the Fairbanks International Airport from the National Oceanic and Atmospheric Administration (NOAA) at the National Climatic Data Center in Asheville, North Carolina (27). If irregularities are neglected, weather may be assumed to follows two trends: annual and diurnal. Annual trends account for seasonal change from winter to summer. This phenomena occurs because solar radiation increases to a maximum on the longest day as the earth's position and distance change relative to the sun. Maximum ambient air temperatures usually occur sometime later in the summer. Diurnal trends account for warming during the day and cooling at night. In this case maximum solar radiation occurs at 12:00 LST (local solar time). Minimum ambient air temperatures occur before sunrise with a maximum in the afternoon. Daily trends are altered by cloud cover, precipitation and circulating cool or warm air masses to the region.

Annual Trends and Extreme Events

Heat transfer occurs through a highway structure by conduction, convection, solar radiation and thermal long wave radiation. Over time, structures may be expected to respond to trends of the environment. Climatic boundary conditions, such as air temperatures, influence the effects of both long wave radiation and convection, wind contributes to convective cooling, and solar flux provides heat to pavements and bridge decks, see Fig. 1. Other factors, such as precipitation and changes in wind, can modify the response. If contributions of precipitation and variations in wind velocity are neglected, daily accumulated heat transfer energy for day, d, on the boundaries is a function of ambient air temperature, T_d 2, solar radiation, and wind velocity.

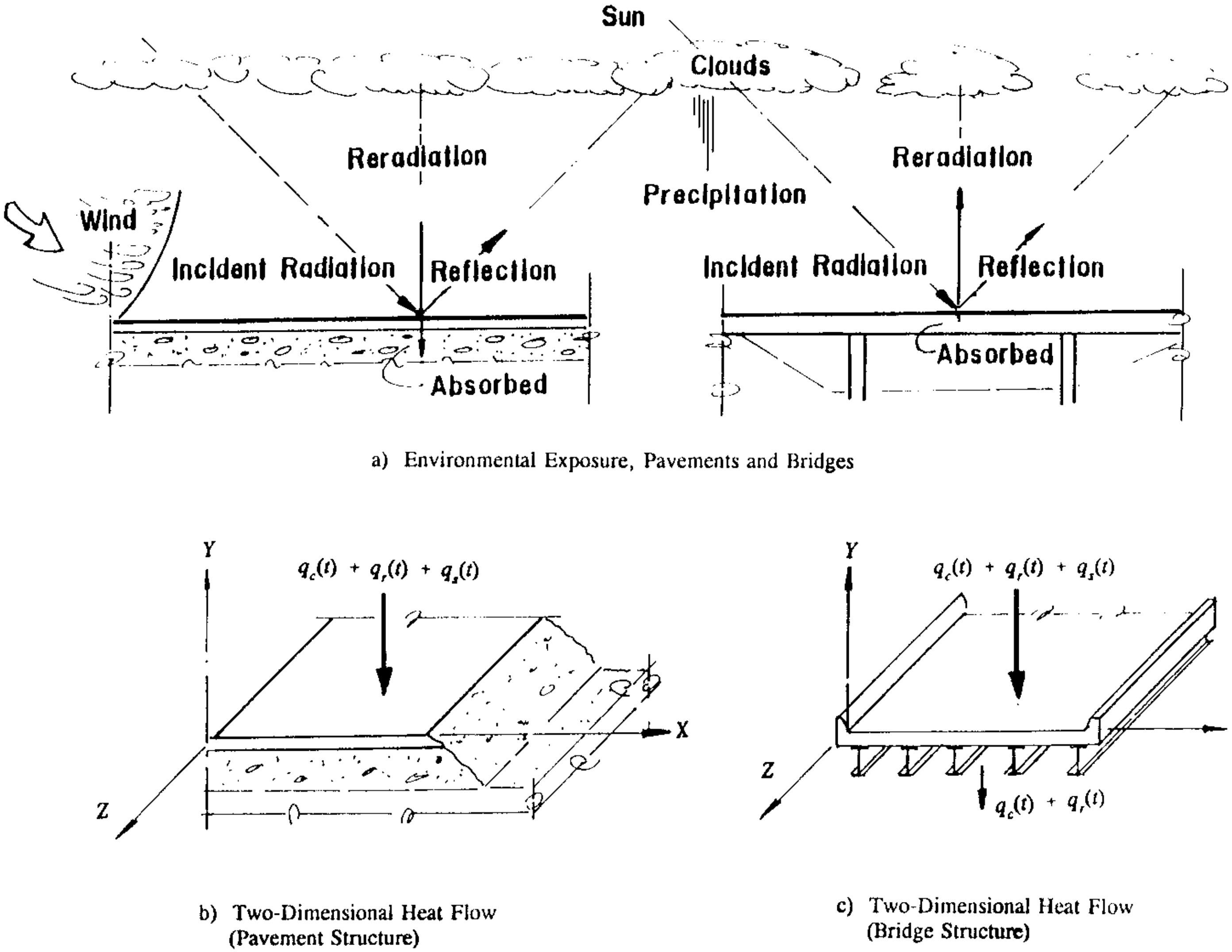


Fig. 1. Transportation Structures Exposed to Climatic Conditions [after Hulsey and Powell(60)]

Ambient Air Temperature

It is valid to assume annual trends in ambient air temperature will follow a periodic cycle of the form (2)

$$T_d = A_d \sin\left[\frac{2\pi(d-\gamma)}{365}\right] + B_d\ ;\ 0 \le d \le 365 \qquad \textbf{(1)}$$

where T_d is daily temperature; A_d is the annual temperature fluctuation about a yearly average; B_d is average yearly temperature; γ 3 is lag in days; and d is day of the year. Air temperature may be expressed as a function of the design period (2,5,39,60), see Table 6. Record maximum and minimum temperatures between 1952 and 1976 for the Fairbanks International Airport were 94°F and -62°F. The 94°F corresponds to a 30 year recurrence interval and the -62°F corresponds to a 15 year recurrence interval.

TABLE 6. AMBIENT AIR TEMPERATURE, ANNUAL TRENDS (after Hulsey and Powell (60))

Site:	A_d 4	B_d 5	γ 6	High Temperatures			Low Temperatures		
Annual Temperatures	°F	°F	(days)	°F	days	recur(yrs)	°F	days	recur(yrs)
Fairbanks, Alaska:									
Maximum, $T_{d(\max)}$ 7	29	56	100	85	191-192	1.5	27	8-9	--
Average, $T_{d(avg)}$ 8	39	26	100	65	191-192	--	-13	8-9	--
Minimum, $T_{d(\min)}$ 9	51	-4.5	104	46.5	195-196	--	-55.5	12-13	5

Air temperature extremes were found as a function of the reoccurrence period, Table 7. The temperatures range between 10 and 25°F for about 75% of the days in Fairbanks, Alaska.

Wind

Wind influences the rate of convective cooling. In Fairbanks, for the period 1952 to 1976, the dominant range of maximum wind speed varied between 0 and 20 mph with a daily average maximum of 5 mph. Wind speeds corresponding to maximum and minimum temperature days were predominately 5 mph or less.

TABLE 7. RECURRENCE COEFFICIENTS FOR AMBIENT AIR TEMPERATURE EXTREMES (60)

Recurrence Period (Years)	Maximum Temperature, Hot Days					Minimum Temperatures, Cold Days				
	Daily Maximums					Daily Minimums				
	T_d (°F)	ξ_y 10	A_d (°F)	B_d (°F)	γ (days)	T_d (°F)	ξ_y 11	A_d (°F)	B_d (°F)	γ (days)
Fairbanks:										
1	80	0.816	29	51	100	-32	0.438	51	19	104
2	86	0.878	29	57	100	-49	0.671	51	-1	104
5	89	0.908	29	60	100	-55	0.753	51	-4	104
10	91	0.928	29	62	100	-60	0.822	51	-9	104
20	93	0.949	29	64	100	-64	0.877	51	-13	104
50	96	0.980	29	67	100	-69	0.945	51	-18	104
100	98	1.000	29	69	100	-73	1.000	51	-22	104
	Note: $\phi = -1$ 12 for maximum; $\xi_y = \frac{T_{(d\text{-}yr)}}{T_{(d\text{-}100yrs)}}$ 13					Note: $\phi = 1$ 14 for minimum conditions; $\xi_y = \frac{T_{(d\text{-}yr)}}{T_{(d\text{-}100yrs)}}$ 15				

BRIDGE TEMPERATURE, DESIGN CHOICES

AASHTO (38) states that "provisions shall be made for stresses or movements resulting from variations in temperature. The rise and fall in temperature shall be fixed for the locality in which the structure is to be constructed and shall be computed from an assumed temperature at the time of erection." The AASHTO provisions provide a range of mean bridge temperatures for steel bridges of 150°F for cold climates and 120°F for moderate climates. A rise of 30°F and fall of 40°F of mean bridge temperatures are given for concrete bridges for moderate climates and a rise of 35°F and fall of 45°F in cold climates.

Bridge Temperatures for Stresses

Although codes in other countries recognize that temperature gradients are often responsible for large thermal stresses in both concrete and steel bridge structures, the AASHTO provisions have no guidelines for accommodating the effects of diurnal weather variations on bridge structures. Thus, there is a need to develop simple design guidelines to account for the affects of weather on bridge structures. *In response to this need, FHWA is in the process of funding a multi-year research project for the purpose of developing design guidelines for jointless bridges*.

Numerous approximations are available in codes of other countries for approximating temperature gradients through the cross section of prestressed concrete T-beam bridges. A summary of all of these approximations is beyond the scope of this report. Three approaches are worthy of consideration. First, Priestley (21,52) suggests that, for prestressed concrete T-beam and concrete box bridges in New Zealand, a 5th order polynomial can be used to estimate temperature gradients through the cross section of a T-beam. The Priestley suggestion is

$$T(y) = \Delta T \left(\frac{y}{1200} \right)^5 \qquad \textbf{(2a)}$$

$$\Delta T = 32 - 0.2h \; ; \; {}^{\circ}C \qquad \textbf{(2b)}$$

in which T(y) is the change in bridge design temperature at some depth y in °C, y (mm) is measured from the top of the deck to 1200 mm, and h (mm) is depth of an asphalt surface over the concrete deck. Second, PCI-PTI (58) suggests a rectangular temperature change of 18°F through the flange of a prestressed T-beam. The change in temperature through the web is taken as zero. Neither of these approximations account for design period. For example, do these represent conditions for a 50 year design period?

A third approach is suggested by the author. It involves: a) selecting a design life for the structure; b) determining the weather exposure from the first part of the chapter that corresponds to this design period; c) performing a heat flow analysis using either the finite element method or a finite difference method to estimate temperatures through the cross-section; and c) determining the maximum gradients for exposures over time. This technique enables the engineer to rationally estimate temperature distributions as a function of the selected design life. This methodology has the disadvantage of being complex and time consuming. Therefore, it is not practical in a typical design office; but a simple to use computer program could be written to give bridge engineers a design temperature gradient or future research could be conducted to develop simple temperature gradients as a function of the design period.

Mean Bridge Temperatures, Movements

A 1991 NSF report by Kuppa and Roeder (6) provides insight into movements in relation to exposure. Movements were calculated and compared with AASHTO values using mean temperatures for three types of bridges at 11 SOLMET climatic sites. Linear equations for maximum and minimum mean bridge temperature were expressed as a function of maximum and minimum air temperature in the form

$$\Theta = a + b\,T \tag{3}$$

in which Θ 16 is mean bridge temperature, a 17 and b 18 are constants and T 19 is either maximum or minimum air temperature. It should be pointed out that the Kuppa and Roeder equations do not account for rotational movements. The equations are based on 50 years of temperature extremes and clear sky solar radiation. Assuming these linear relationships are valid for return periods, Hulsey and Powell suggested replacing T 20 with the return period temperatures of Table 7 giving equations for summer and winter exposures as follows.

Summer Exposure

The maximum mean bridge temperatures, °F, from Kuppa and Roeder (6,60) and the design period air temperatures from Table 7 are

$$(\Theta_{\max})_{yr} = 6.5 + 1.015\,(T_{\max})_{yr}\ ;\ composite \quad \textbf{(4a)}$$
$$(\Theta_{\max})_{yr} = 4.6 + 0.979\,(T_{\max})_{yr}\ ;\ box\ girder \quad \textbf{(4b)}$$
$$(\Theta_{\max})_{yr} = 4.594 + 0.9526\,(T_{\max})_{yr}\ ;\ T\text{-}beam \quad \textbf{(4c)}$$

in which $(T_{\max})_{yr}$ 21 is the maximum summer air temperature from Table 7 for a given design period and $(\Theta_{\max})_{yr}$ 22 is the mean bridge temperature of the bridge cross section. Unrestrained axial movement of the bridge, relative to a point of zero movement, may be calculated by $(\Delta_{\max})_{yr} = \alpha\,[\,(T_{\max})_{yr} - T_{const}\,]$ 23. Note, α 24 is the thermal strain coefficient and T_{const} 25 is the temperature at time of construction. The results of research by Powell(39) suggest that the temperature at time of construction for Fairbanks is about 57°F.

Winter Exposure

Similarly, the minimum mean bridge temperatures, °F from Kuppa and Roeder (6) and the design period winter air temperatures of the author (60) from Table 7, are

$$(\Theta_{\min})_{yr} = 9.06 + 1.096\,(T_{\min})_{yr}\ ;\ composite \quad \textbf{(5a)}$$
$$(\Theta_{\min})_{yr} = 17.24 + 1.186\,(T_{\min})_{yr}\ ;\ Box, T\text{-}beam \quad \textbf{(5b)}$$

Mean Bridge Temperatures for Fairbanks

The proposed modifications to Kuppa and Roeder's equations (6) suggest that 50 year maximum mean bridge temperatures for a prestressed concrete T-beam bridge, exposed to a 50 year return period in Fairbanks, would be 96.04°F. The 50 year minimum would be -64.59°F. Assuming temperature at time of construction is 57°F, the temperature ranges are a rise of 39.04°F(AASHTO, 35°F) in the summer and a fall of 121.69°F(AASHTO, 45°F) in the winter. The temperature range between summer and winter is 160.63°F (96.04°F + 64.59°F). This suggests that the AASHTO provisions for the Fairbanks climate is inadequate.

SUMMARY

A rational method is proposed for estimating temperature extremes in terms of design period. This is a strength approach. Although the methodology provides a rational way to estimate thermal effects, simplification is needed for design office use.

Equations for estimating design mean bridge temperatures for a given design life are proposed. These relations may be used to estimate bridge movements. The temperature at time of construction in Fairbanks, Alaska should be taken as 57°F.

The AASHTO provisions underestimate the mean bridge temperature rise and fall for estimating bridge movements. Also, AASHTO makes no provisions to account for temperature gradients; temperature gradients can cause large thermal stresses and induced forces.

THERMAL RESPONSE OF THE MACLAREN RIVER BRIDGE

This chapter shows the results of a thermal study that was conducted for the Maclaren River Bridge. The geometry is in Appendix B. The structure is a symmetrical, 3 span, jointless, prestressed concrete, girder bridge. The overall length is 361 ft - 11 inches. The roadway width is 28 ft between the bridge rails. The superstructure consists of 5 prestressed girders with spans of 119 ft - 11 in, 120 ft - 7 in, and 119 ft - 11 in. The girders are cast together at the piers with a concrete diaphragm and at each end of the bridge with a concrete abutment backwall. Elastomeric bearing pads support the girders. In 1998, distress was observed by AKDOT&PF at the abutment backwall and at the pier diaphragms. Examples of the distress are in Appendix C.

A model of the structure was subjected to summer and winter weather extremes for Fairbanks. Weather loads for 5 and 50 year design events were imposed on the structure for a period of three days for each exposure. The temperature at the time of construction was assumed to be 57°F. Based on these exposures, a parametric study to evaluate the influence of support restraint on movements and stresses was undertaken using an elastic analysis. Temperatures through the cross section of an interior prestressed girder were calculated every 6 minutes over a 3-day period for a summer and a winter exposure. Temperature changes at 2 hour intervals resulting from these exposures were used to calculate slab stresses, bridge movements, and induced forces for summer and winter exposures for abutment stiffnesses of 0 to 2000 kips/in.

BRIDGE TEMPERATURES

An interior prestressed girder was subjected to 5 and 50 year Fairbanks, Alaska summer and winter weather extremes. A finite element model was selected to calculate temperatures every 6 minutes for a 72 hour period, see Fig 2. The model consisted of 182 nodes, 140 elements, 40 convection boundaries, 11 solar radiation boundaries, and 11 thermal radiation boundaries. The concrete thermal properties were $k = 1.0$ Btu/(ft^2-hr-°F), $c = 0.16$ Btu/(lb-°F), and $\rho = 150\ pcf$ 26.

Summer

Bridge deck maximum temperatures occurred at noon. These were 92°F for a 5 year design period and 100°F for a 50 year event. The temperature gradient for both was about 24°F, see Fig. 3. The maximum air

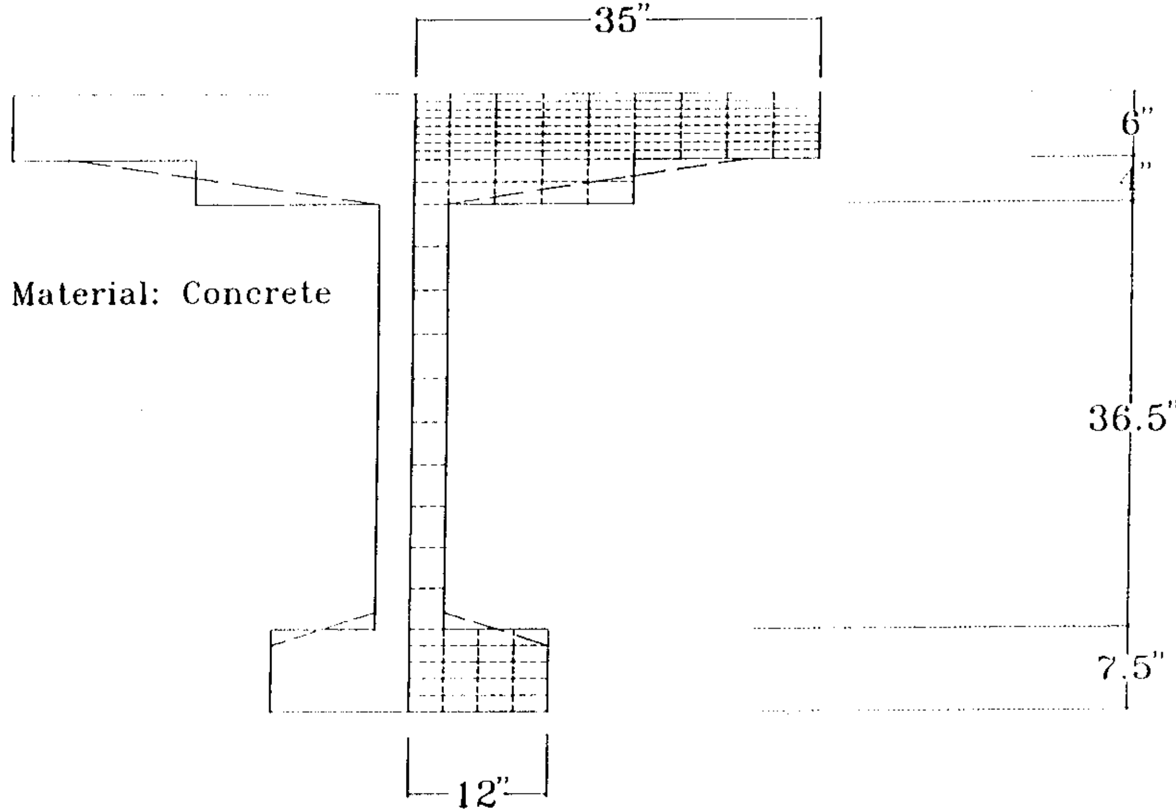


Fig. 2. Finite Element Model for Heat Flow, Interior Girder

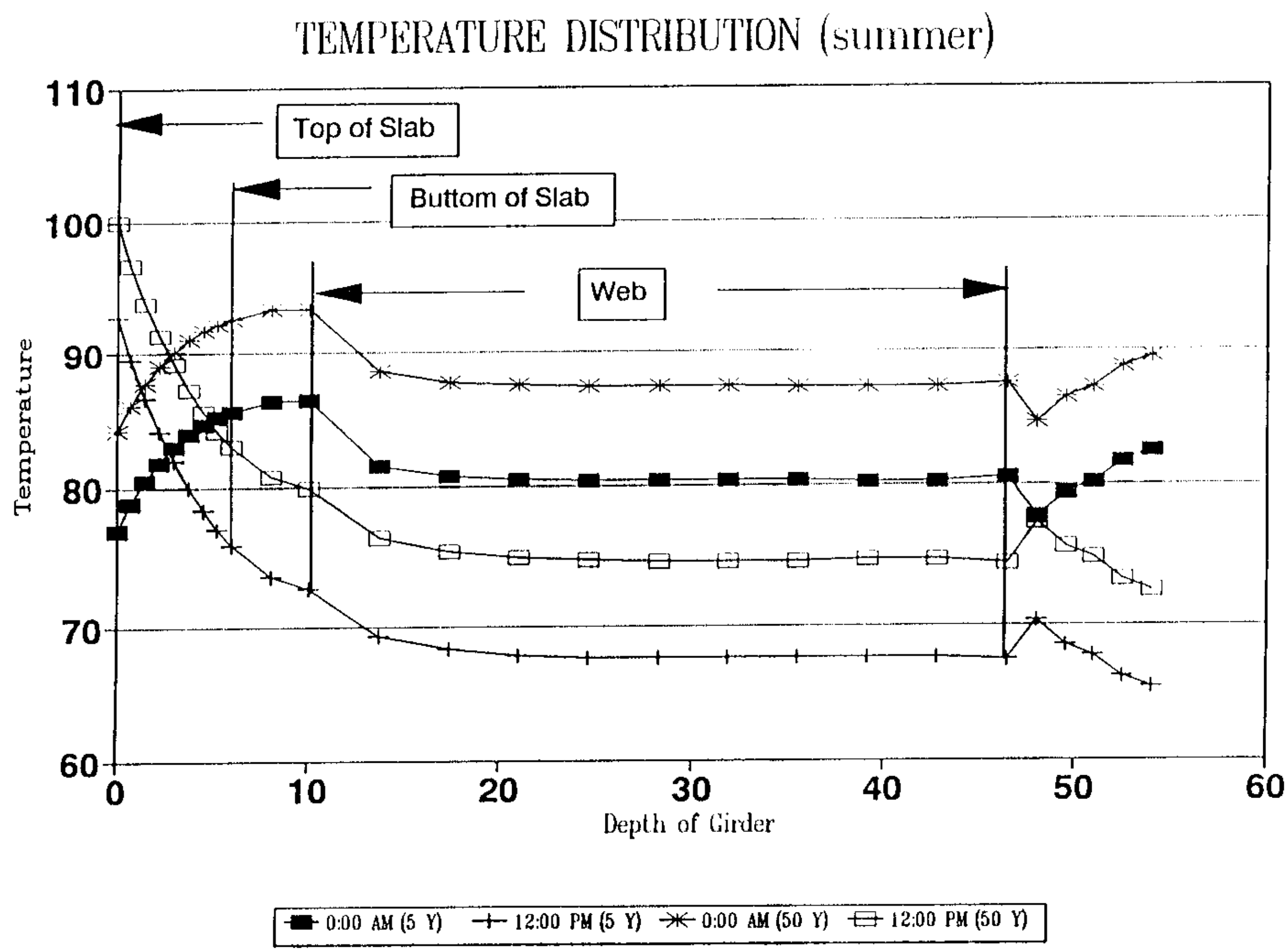


Fig. 3. Interior Girder Temperature Distribution, Summer

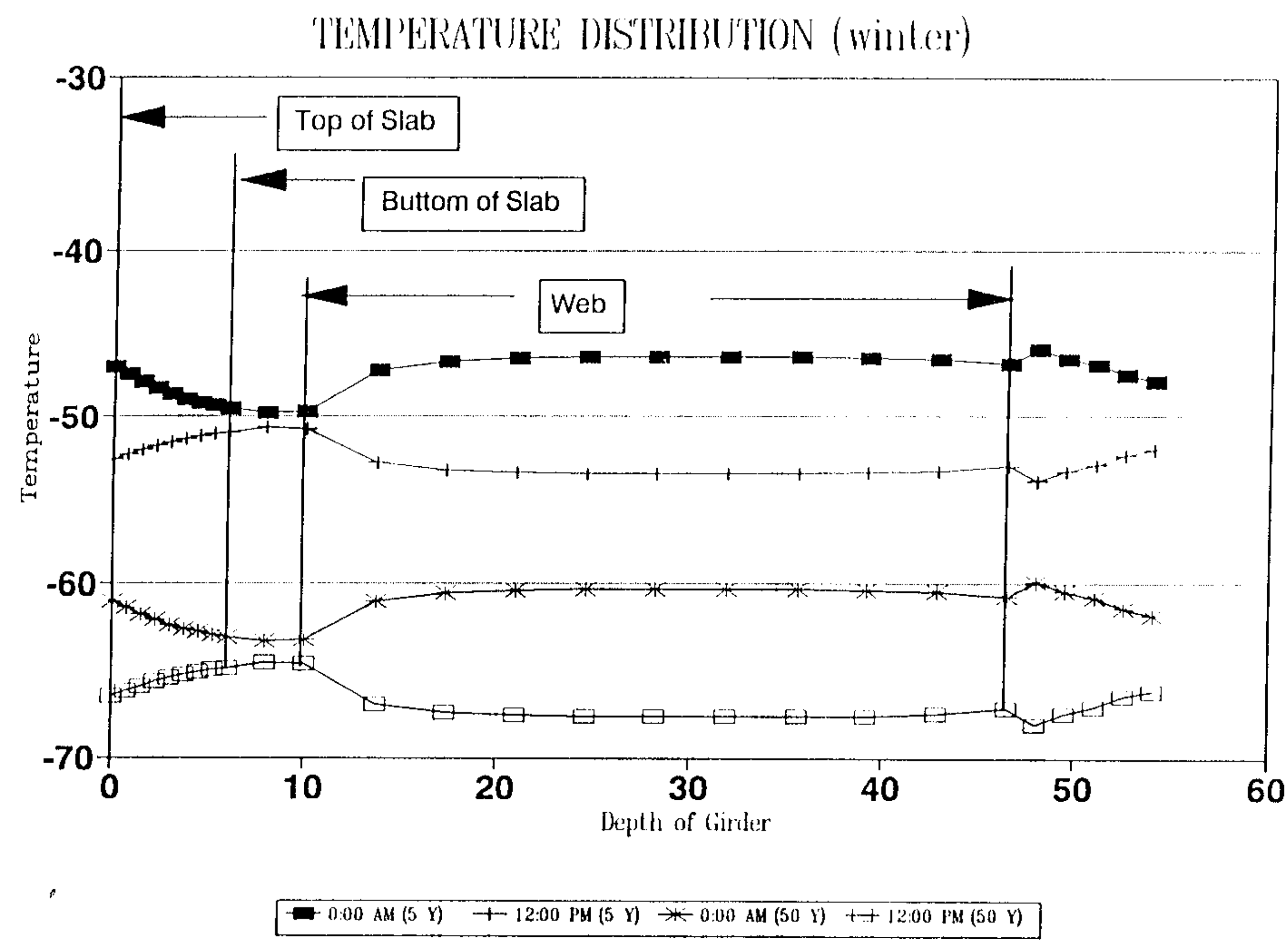


Fig. 4. Interior Girder Temperature Distribution, Winter

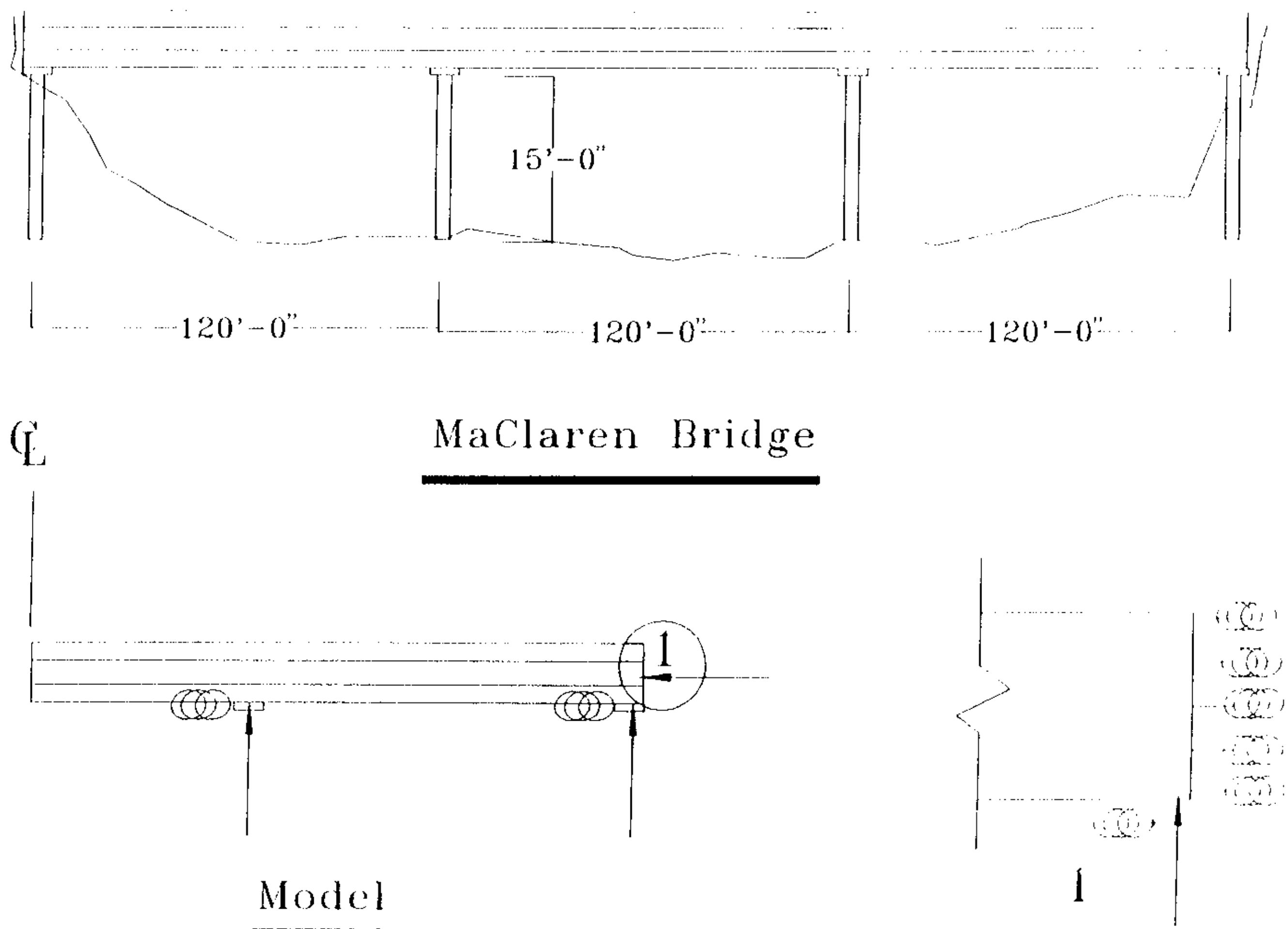
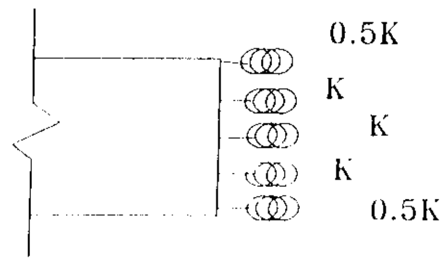
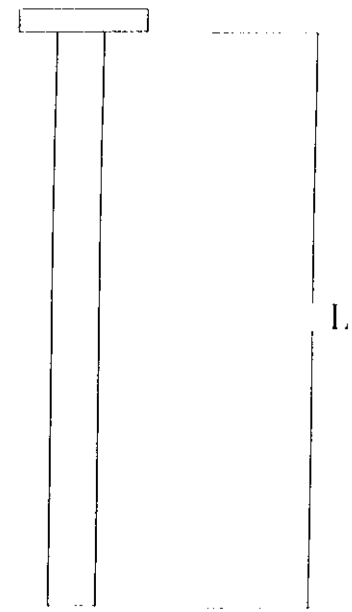


Fig. 5. Analytical Model of the Maclaren River Bridge



End Soil Stiffness

	Summer	Winter
K (k/in)	> 0	0
L	20'-0"	15'-0"



Pile Length

Fig. 6. **Maclaren River Bridge Spring Approximation for Substructure Restraints**

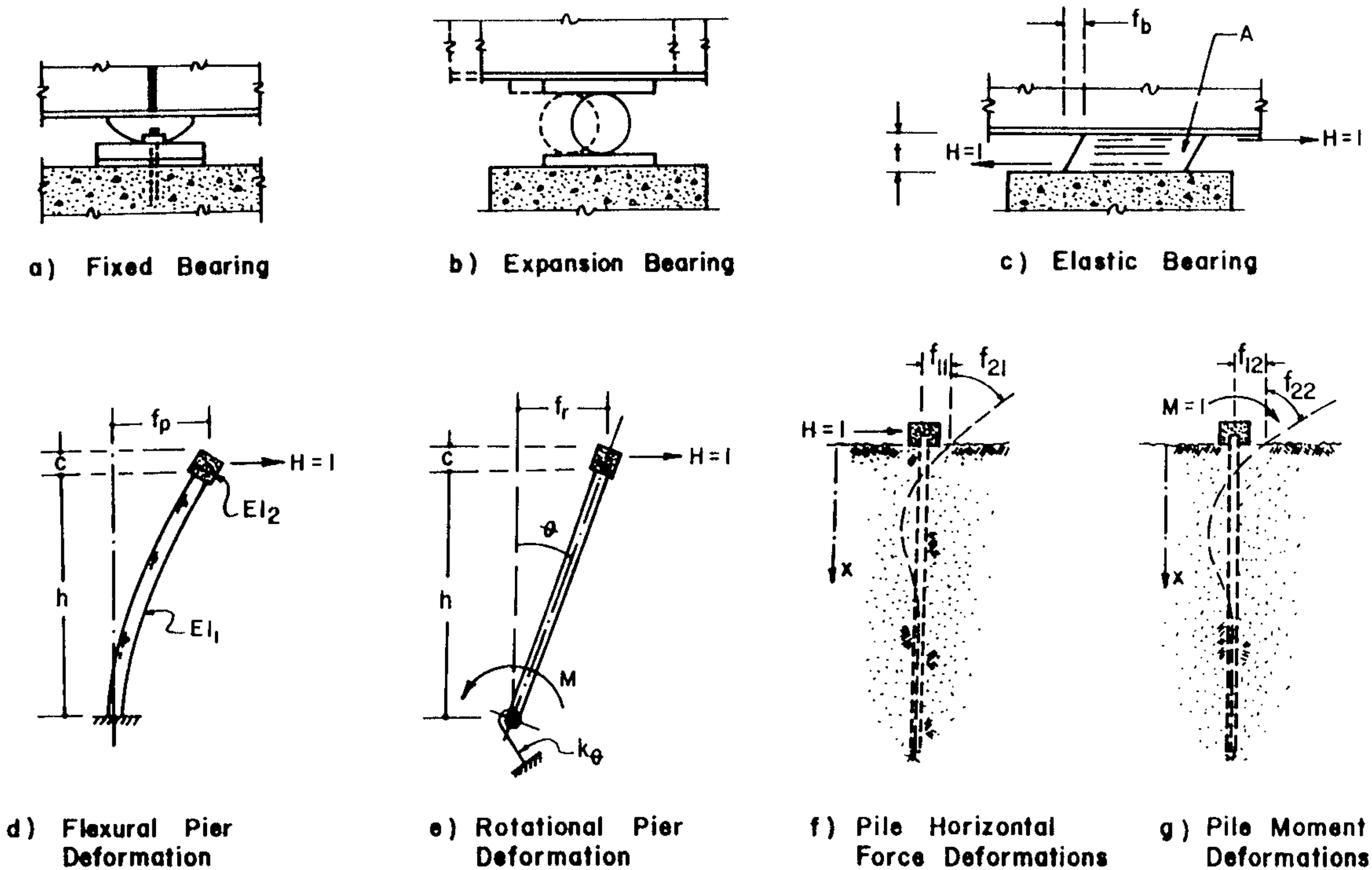


Fig. 7. Stiffness Models for Different Support Conditions

temperature is 89°F for the 5 year and 96°F for the 50 year.

Winter

Minimum temperatures in the bridge deck were between -46°F and -52°F for a 5 year event and between -60°F and -68°F for the 50 year design period. The gradient in both cases was approximately 4°F, see Fig. 4. The minimum air temperature for 5 and 50 year weather is -55°F and -69°F, respectively.

STRUCTURE, ANALYTICAL MODEL

Theoretically, the magnitude of stress induced by axial thermal strain does not increase with bridge length when the structure is free to move. But, thermal strain gradients cause curvatures and curvatures induce stress. Thus, there is a stress redistribution in indeterminate structures. Support restraints impose additional stress. A parametric study was performed to examine the affects of substructure restraints when exposed to 5 and 50 year weather extremes.

A composite-plate beam element developed by the author(2) was used, in combination with springs, to approximate the piles, and resistance to movement by the approach material behind the abutment. Using this model, induced movements and stresses in the bridge, due to the temperatures produced by the summer and winter exposures, were calculated as a function of time. The plate portion of the element accounts for bending of the slab and upper flanges of the T-beam. The beam part of the element accounts for the web and lower flange. One-half of the structure was analyzed in accordance with the model of Figs. 5 and 6. Substructure restraints were approximated by item d) for the interior bent and item f) at the abutments, see Fig. 7. Slip at the shear key was not considered in this analysis.

Superstructure

Movements, support forces, strains and stresses were calculated every two hours for three days of weather. An interior prestressed concrete T-beam (composite slab-beam element) was modeled with 6 joints and 5 elements. A concrete modulus of 3800 ksi, poisson's ratio of 0.2, thermal strain coefficient of 5.5 x 10^{-6} in/in/°F, and an initial temperature of 57°F was selected for this study.

Substructure

When support restraints are reduced, the magnitude of thermally induced stresses will be reduced. The lateral resistance of unfrozen soils are given in Table 8. In the state of Alaska, it is reasonable to assume that some of the material behind the abutment backfill and piles can be frozen. The lateral stiffness of embedded structures in frozen or partially frozen materials was not found in the literature.

TABLE 8. SUBGRADE CONSTANT, k Terzaghi (61)

Relative Density	Tons/ft^3, (lbs/in^3)		
	Loose	Medium	Dense
Dry or Moist Sand	8 (7)	24 (21)	65 (56)
Submerged Sand	5 (4)	16 (14)	40 (34)

Soil modulus is calculated by

$$E_s = \frac{kx}{B}\ ;\ pci \qquad \textbf{(18)}$$

in which k is given by Table 8, x is the depth measured from ground surface, and B is the pile width. A spring at a discrete point is calculated by

$$k_s = E_s(B)(h) \qquad \textbf{(19)}$$

in which h is the spacing of the springs.

Assuming a dense sand (see Table 8 and the soil borings in Appendix B), the stiffness of the abutment piles in an unfrozen soil was calculated by applying a horizontal load of 1000 kips at the top of the pile. It was assumed that the resistance of a frozen soil could approach ten times the resistance of an unfrozen soil. The calculated pile stiffnesses at the abutments may be approximated by:

	Pile Stiffness (kips/in	
Condition	Unfrozen	Frozen
Summer	333.1	2404.2
Winter	44.7	120.3

Summer

The calculated lateral support stiffness of the interior bents was 15 kips/in. Five springs were attached to the abutment backwall and one spring at the bearing support. These springs resist longitudinal movement of the bridge deck. The bearing support spring represents lateral resistance of the piles at the end bents. A resistance of 30 kips/in was used for this spring. The five springs at the end bent produce an effective resistance of 4 times the resistance of one. These springs approximate the resistance of the soil block behind the abutment during expansion movements. The soil block resistance springs were varied from 0 to 2000 k/in (4 springs => 8000 k/in). Note, the abutment pile stiffness is about 333.1 k/in for unfrozen soils and approximately 2400 kips/in for frozen soils.

Winter

The calculated lateral stiffness at the interior bent was 55 kips/in. Because of contractive movements, no springs were placed behind the abutment backfill. The lateral stiffness of the abutment piles were varied from 0 to 5000 k/in. The purpose was to examine the possible affect of frozen conditions. The calculated lateral abutment pile resistance in unfrozen materials was approximately 44.7 k/in. The frozen resistance could approximate 120 kips/in.

SLAB STRESSES

Figs. 8 and 9 show the affects of induced longitudinal slab stresses for winter and summer exposures. The stresses are expressed as a function of the lateral resistance of the piles and longitudinal resistance of the soils at the abutment. The results of this study shows that significant tensile stresses could be induced when the soils surrounding the piles or the approach are partially frozen. It appears the worst condition occurs during the winter.

LONGITUDINAL REACTION

The restraints imposed by the substructure supports cause longitudinal forces to develop at the ends of the structure. The magnitude of these forces are shown in Figs 10 and 11. Again, the results of this analysis show that large forces will develop at the abutment backwall during the winter. The maximum forces at the pier diaphragms

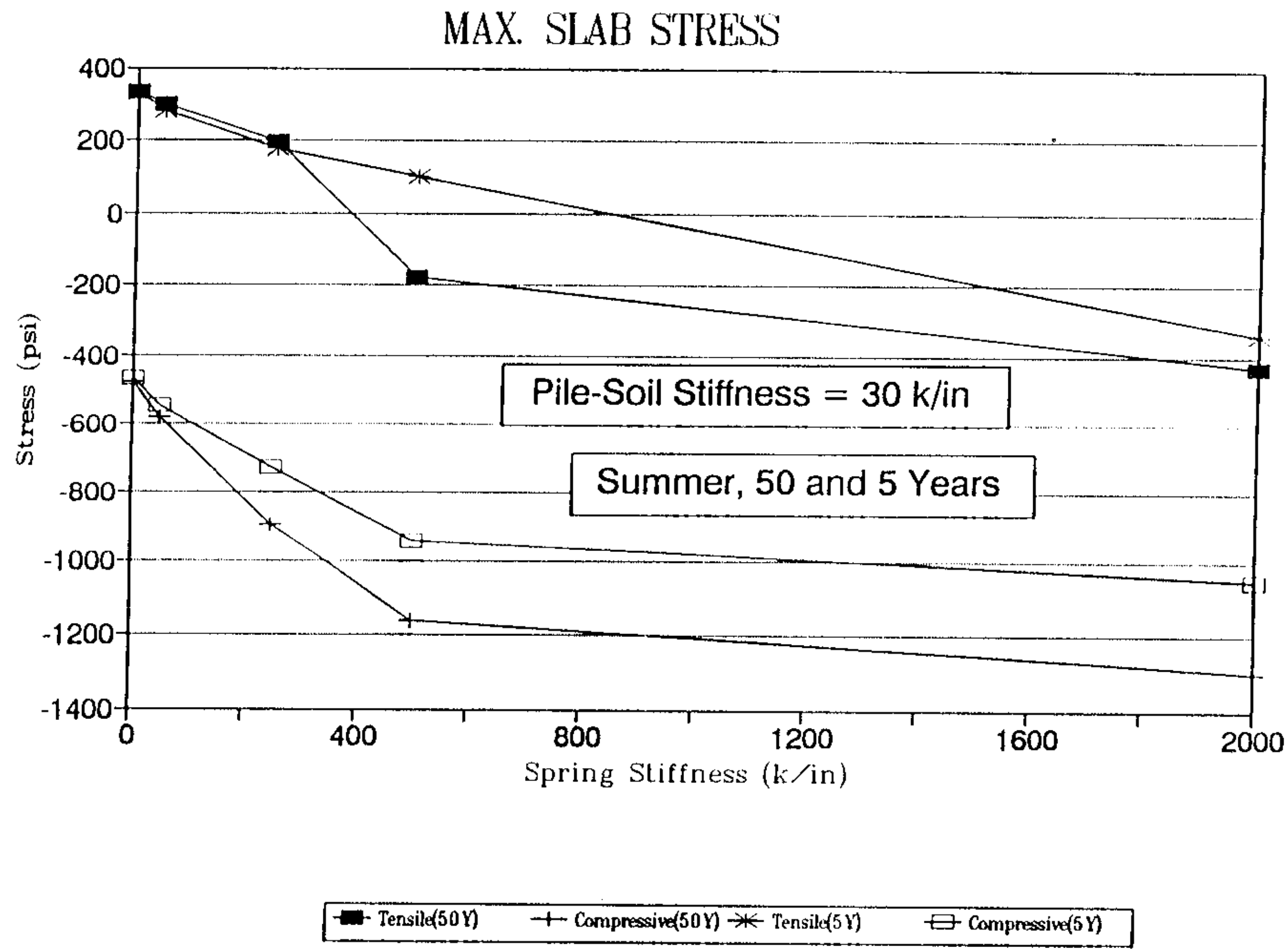


Fig. 8. **Maclaren River Bridge Longitudinal Slab Stresses, Summer**

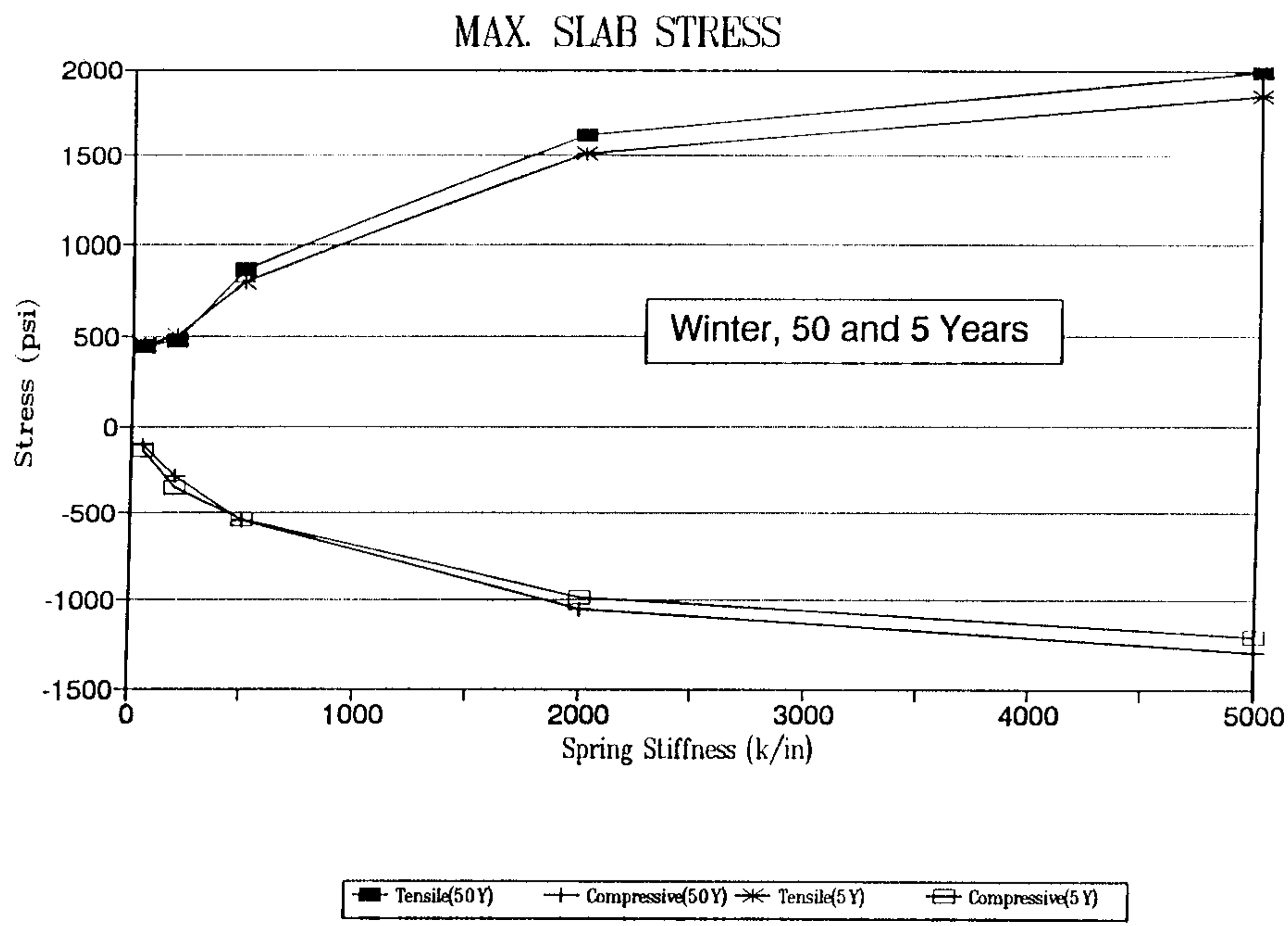


Fig. 9. **Maclaren River Bridge Longitudinal Slab Stresses, Winter**

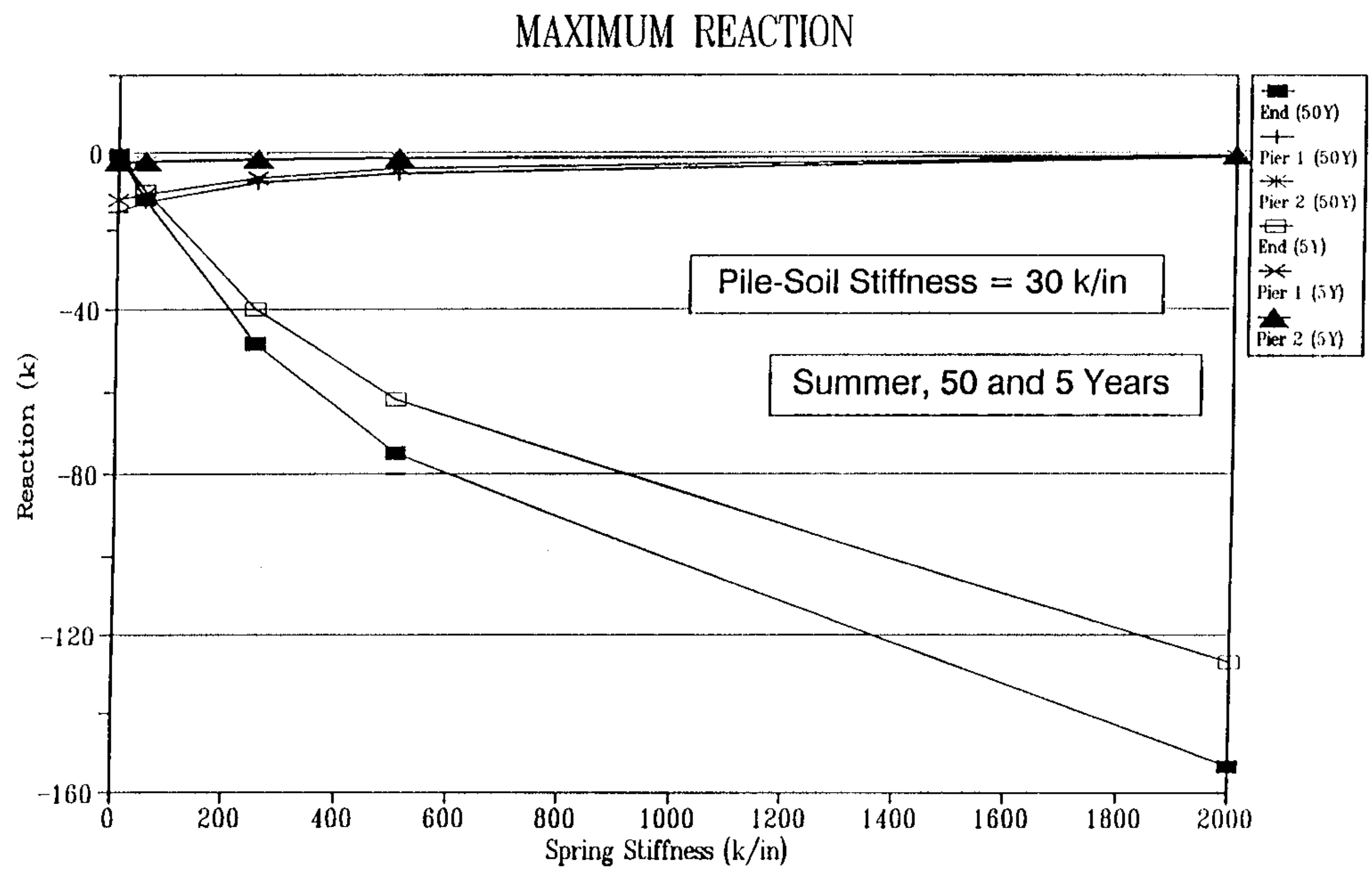


Fig. 10. Maclaren River Bridge Longitudinal Induced Support Reactions, Summer

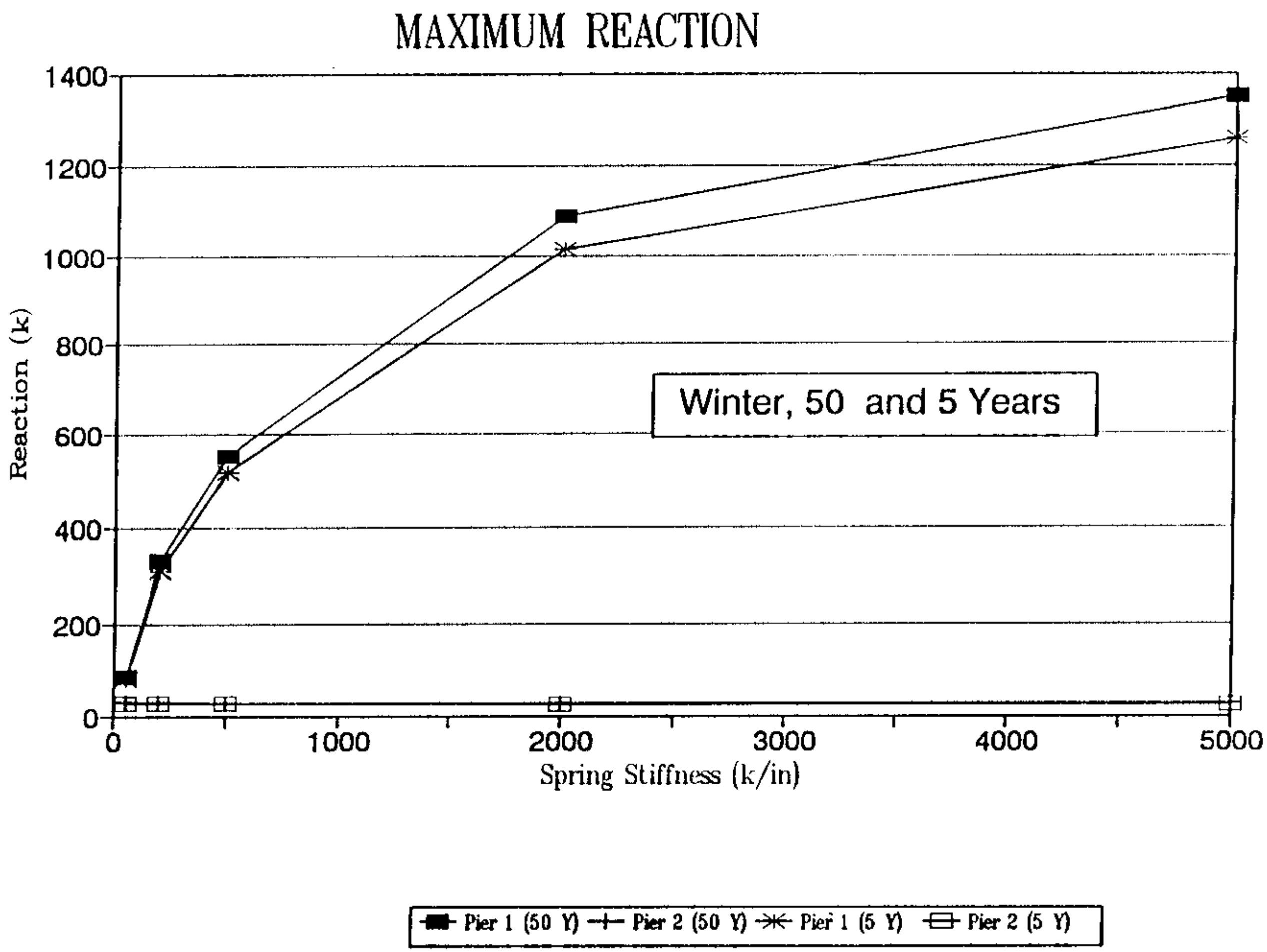


Fig. 11. Maclaren River Bridge Longitudinal Support Reactions, Winter

occur during the summer.

LONGITUDINAL ABUTMENT MOVEMENTS

Figs. 12 and 13 show movements at the ends of the bridge for both summer and winter. The maximum displacements will occur during the winter if no resistance is available at the abutment. These displacements are a maximum of 1.6 inches for a 50 year event. Maximum displacements at the ends, during the summer, are 0.55 inches for a 50 year event and no restraint.

SUMMARY

A parametric study was performed for the purpose of evaluating if the restraints imposed by the substructure were responsible for the distress found during the 1989 inspection of this structure. It is the author's opinion that the lateral resistance of the pile supports and the resistance of the soil block behind the abutment backwall may be caused by partially frozen or frozen soils and the resistance under these conditions could be extremely large. Under these conditions, slab stresses, and induced longitudinal forces were large.

It is therefore recommended for consideration that the use of jointless bridges in interior Alaska may require special details to minimize resistance to movement.

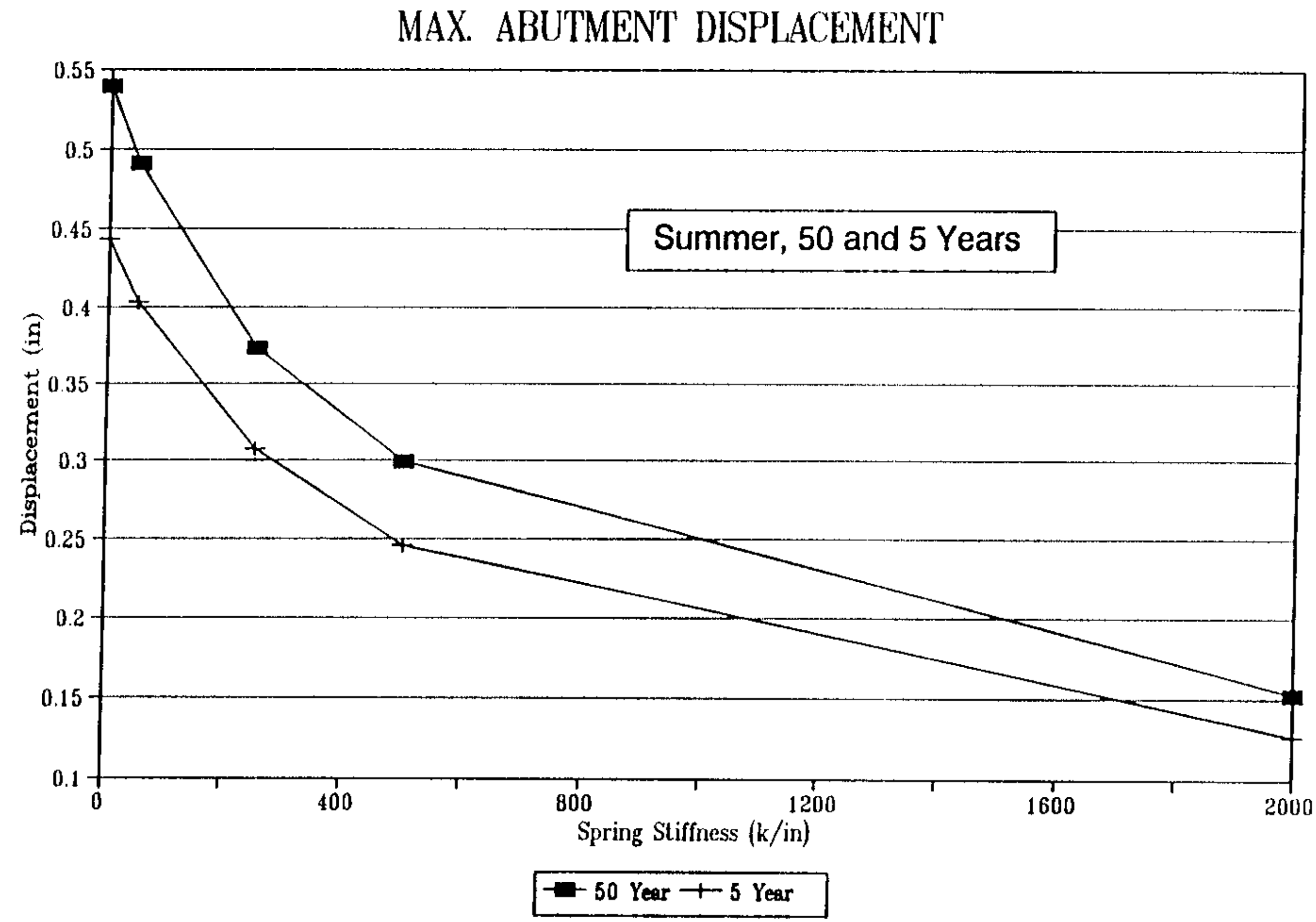


Fig. 12. Maclaren River Bridge Abutment Movements, Summer

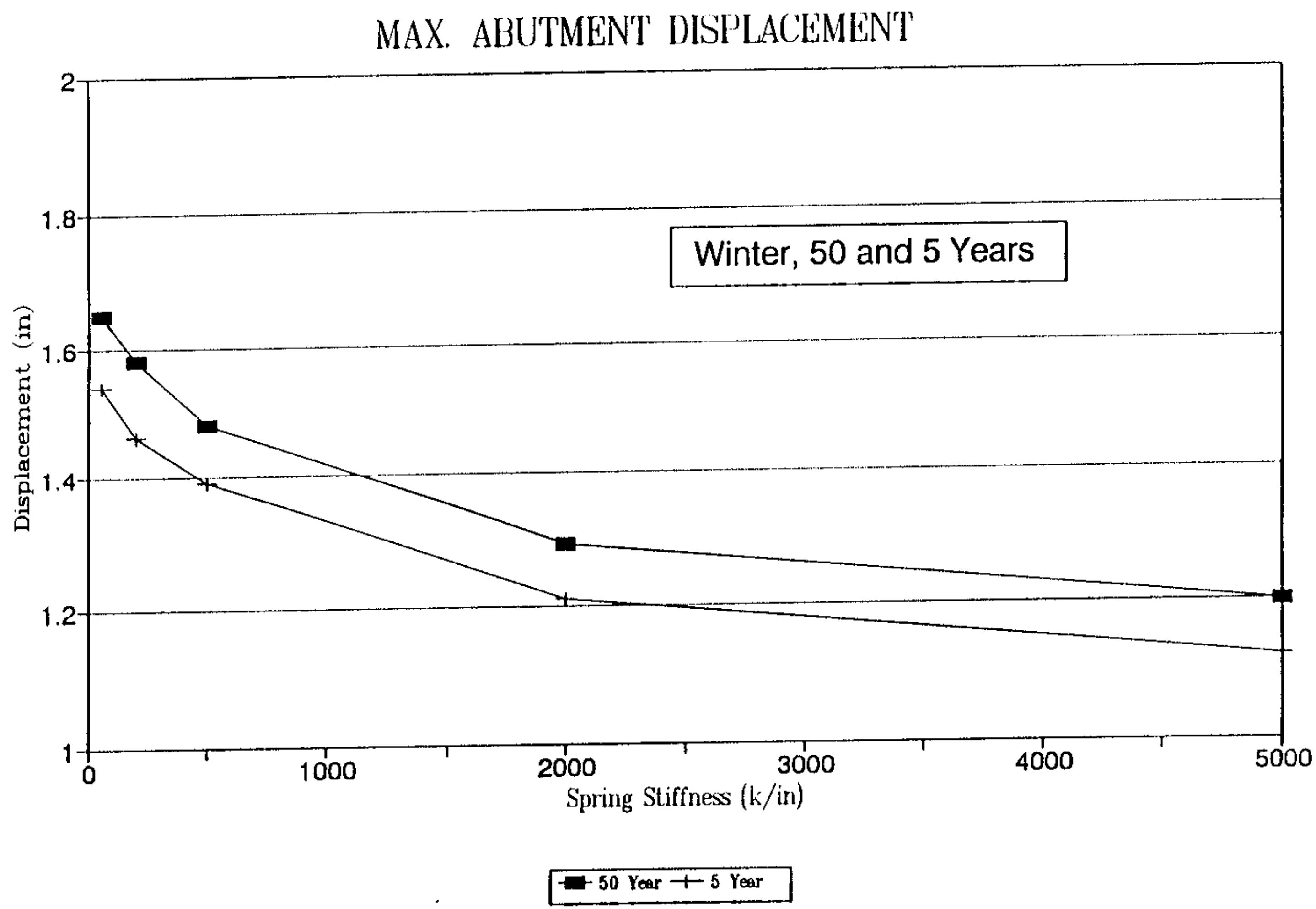


Fig. 13. **Maclaren River Bridge Abutment Movements, Winter**

SUMMARY AND RECOMMENDATIONS

A 1989 AKDOT&PF inspection of the Maclaren River Bridge showed distress at the abutment backwall and pier diaphragms. This structure is a 3-span, 361 ft-11 in, jointless, prestressed, girder bridge. The roadway width between bridge rails is 28 ft.

Jointless bridges are often used by other DOTs because it is thought that they require less maintenance than conventional bridges. A three part study was conducted to determine: a) length limitations used by state DOTs for prestressed, concrete girder, jointless bridges; b) maintenance experiences by the states with this type of structure; and c) if frozen soils will produce large substructure restraints in interior Alaska and cause large induced stresses to develop in this type of structure. The study incorporated a literature review, a national survey of state DOTs, and a parametric study of the Maclaren River Bridge.

Theoretically, the magnitude of induced stresses in jointless bridges are not dependent on length but develop due to the geometry, material properties, exposure, and substructure restraints. The national survey showed that 73% of the responding states used jointless bridges and approximately 46% of these states use jointless bridges over 300 ft in length. Only four states reported maintenance problems with concrete diaphragms.

The mean bridge temperature range provided in AASHTO for calculating movements is inadequate for Interior Alaska. Also, no provisions are available in AASHTO for calculating stresses; this should be corrected.

A model of the Maclaren River Bridge was subjected to 5 and 50 year Fairbanks, Alaska weather extremes. The temperature at time of construction was assumed to be 57°F. A thermal coefficient of expansion or contraction of 5.5×10^{-6} in/in/°F was assumed. Based on these assumptions, stresses, end movements, and longitudinal forces at the abutment backwall and pier diaphragms were calculated for various substructure stiffness restraints. The results of the analysis showed that large stresses can occur, especially if the ground is frozen.

It is recommended that a study be conducted to evaluate the stiffness of piles in frozen soils. It is recommended that special details be used for jointless bridges in Interior Alaska. These details should be designed to minimize lateral resistance.

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APPENDIX A.

LETTER, QUESTIONNAIRE

APPENDIX A.

LETTER, QUESTIONNAIRE

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Attention: Bridge Design

«SA»

Some state departments of transportation now have both jointless and conventional bridges. Conventional bridges are those in which expansion bearings and expansion joints in the bridge deck are relied upon to relieve thermal effects. The jointless bridge is based on the idea that no expansion joints and expansion bearings are necessary, provided the structure is attached to a flexible substructure.

Recently, cracking has been found at the beam abutment interface and at the pier diaphragms for one of Alaska's prestressed concrete bridges. The bridge has a roadway width of 28 feet and spans of 119.92, 120.58, and 119.92 feet with a total length of 361.92 feet.

The structure has a six-inch concrete deck supported by five AASHTO I-girders. Due to problems, Bridge Design at the Alaska Department of Transportation requested that I assemble current technology and experiences of other DOTs with jointless bridges.

Toward this end, I prepared a brief questionnaire to seek answers to questions AKDOT considers important. Please take a few minutes and help us understand current practice by DOTs with the jointless approach. It is imperative that I receive your response by the middle of August.

I thank you in advance for your time and assistance with this important data. Please send your response to:

J. Leroy Hulsey, Associate Professor
Department of Civil Engineering
263 Duckering Building
University of Alaska Fairbanks
Fairbanks, AK 99775
Phone: (907) 474-7816
FAX: (907) 474-6806

Sincerely,

J. Leroy Hulsey

BRIDGE DESIGN QUESTIONNAIRE

J. Leroy Hulsey, Associate Professor
Department of Civil Engineering
263 Duckering Building
University of Alaska Fairbanks
Fairbanks, AK 99775
Phone: (907) 474-7816
FAX: (907) 474-6806

1. Please indicate your state DOT ____________________(state)

2. Does your state have jointless bridges ____________________(Yes, no)

   a) If yes, what percentage of new bridges are jointless ____________________(%)

3. What is the maximum bridge length your state allows for jointless structures?

   a) Concrete decks on prestressed girders ______________ feet

   b) Concrete decks on steel girders (composite) ______________ feet

   c) All steel bridges (e.g. orthotropic) ______________ feet

   d) Reinforced concrete t-beams ______________ feet

   e) Other ______________ feet

4. When you have a jointless bridge with a skew, what length limitations do you use? Are other types of limitations added for this kind of structure.

5. Do you have any limitations such as flexibility of substructure requirements for:

   a) Interior bents (piers) ____

   ____

   b) End bents (abutments) ____

   ____

6. What restrictions do you have for details between the end bent and support piles?

7. This question is a continuation of 6......

a) Do you use an integral system (rotation-restrained)? _______Yes/No

b) Do you use a nonintegral system (free to rotate)? _______Yes/No

c) Do you use a semi-integral system? _______Yes/No

Other comments: ________________________________

____

____

____

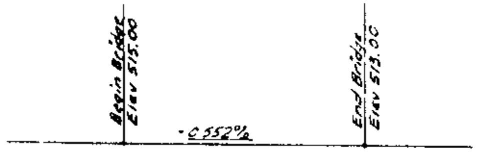
8. Do you have any special details for the pier caps connecting the structure (interior bents)?

9. Please indicate your assessment of maintaining the jointless type of bridge? Please note any problems that have been found and indicate the solutions.

10. If you had a choice, would a new design be jointless or conventional? Indicate why?

APPENDIX B.

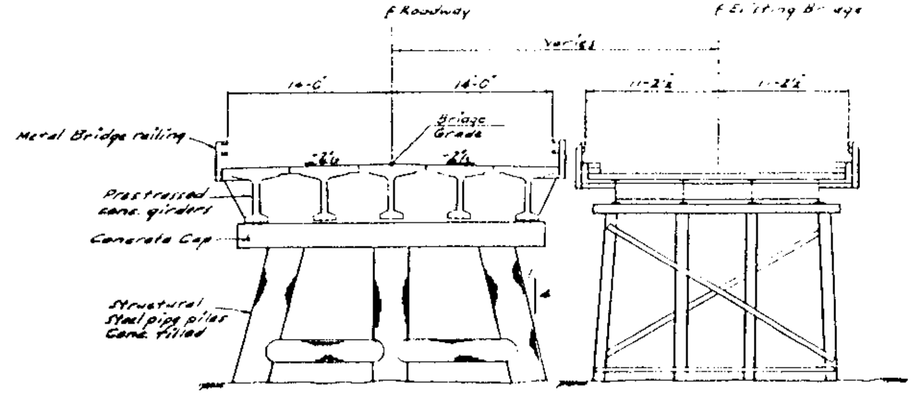
BRIDGE PLANS



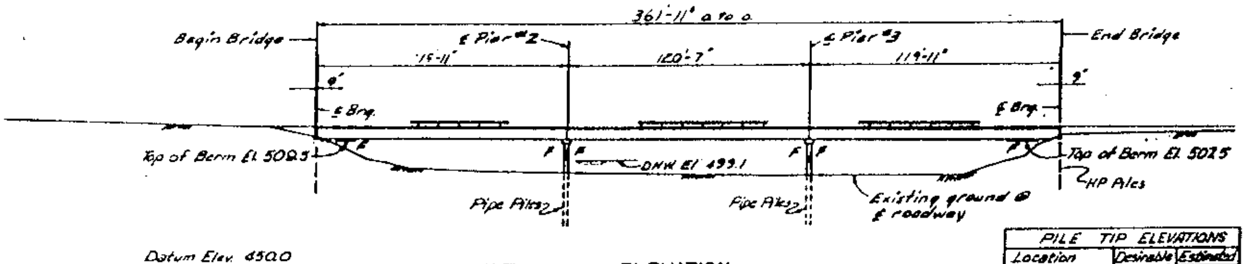
GRADE DATA

No Scale

Bench Mark: Brass Cap MC-1-Block 4
US Survey 42-42
Assumed Elev 500.0



TYPICAL SECTION

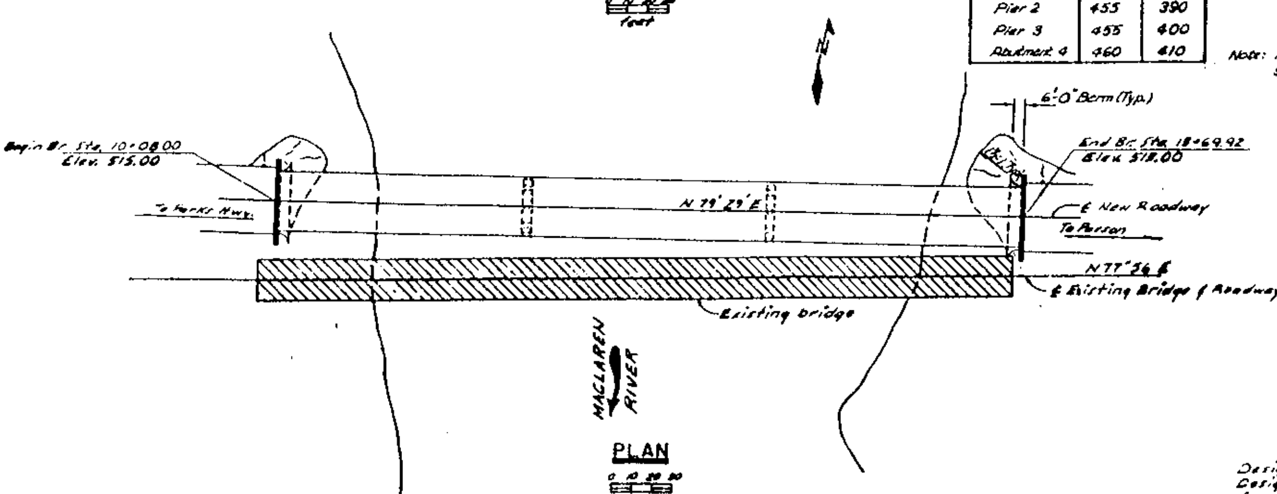


ELEVATION

feet

PILE TIP ELEVATIONS		
Location	Desirable	Estimated
Abutment 1	460	390
Pier 2	455	390
Pier 3	455	400
Abutment 4	460	410

Note: For Estimate of Bridge Quantities, see Dwg. No. 2.



PLAN

feet

DRAWING INDEX	
TITLE	DWG. NO.
GENERAL LAYOUT	1 of 8
SITE PLAN	2 of 8
ABUTMENTS AND PIERS	3 of 8
TYPICAL SECTION	4 of 8
GIRDERS	5 of 8
BRIDGE RAILING	6 of 8
LOG OF TEST HOLES	7 of 8
LOG OF TEST HOLES	8 of 8

The following Standard Drawings apply: S-04.01S, S-04.01W, G-14.01S, S-14.01W, G-18.00, G-24.01S, G-24.01W

Design Live Load = HS 25
Design load includes 25 p.s.f. for future asphalt overlay

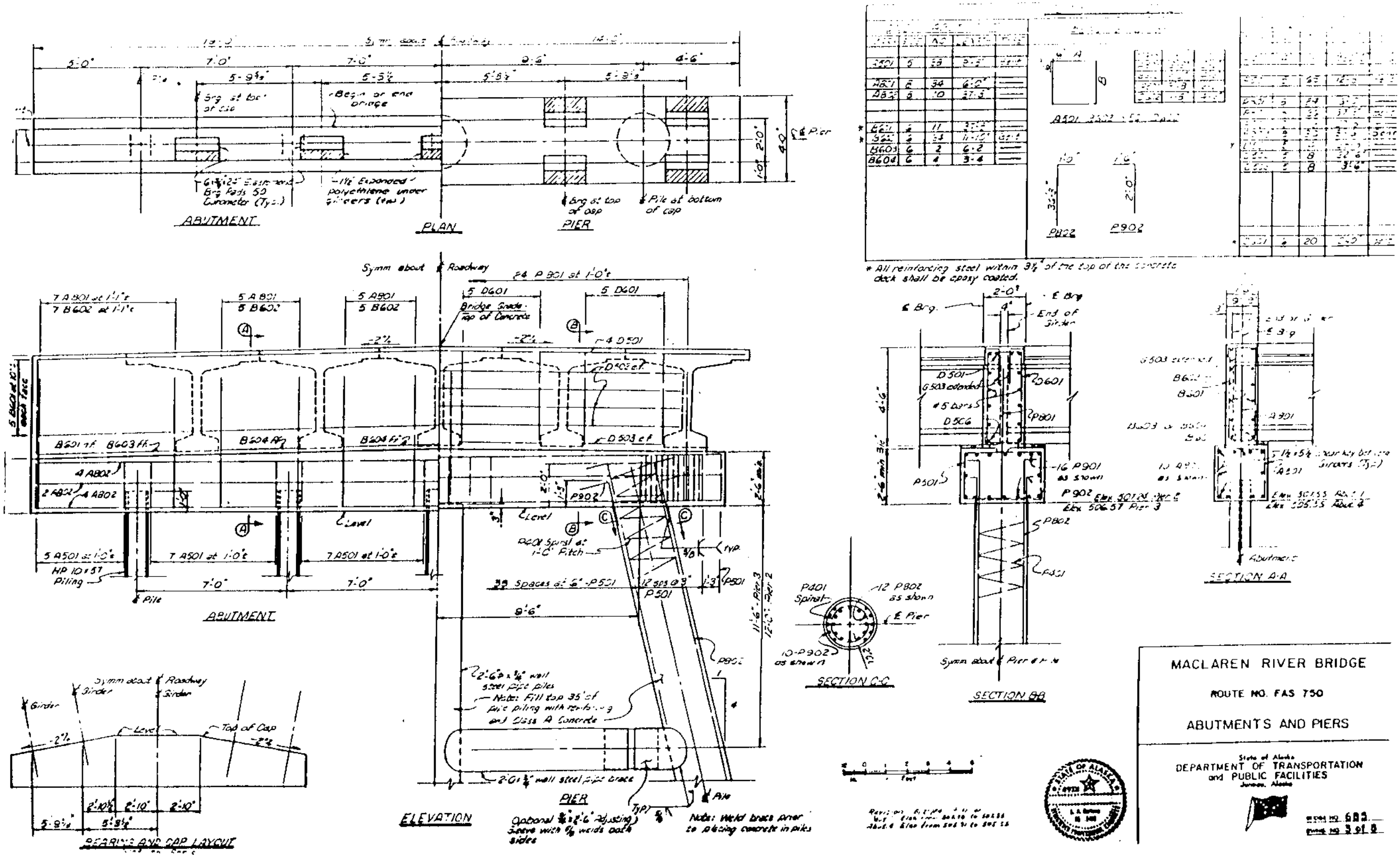
MACLAREN RIVER BRIDGE

Route No. FAS 750

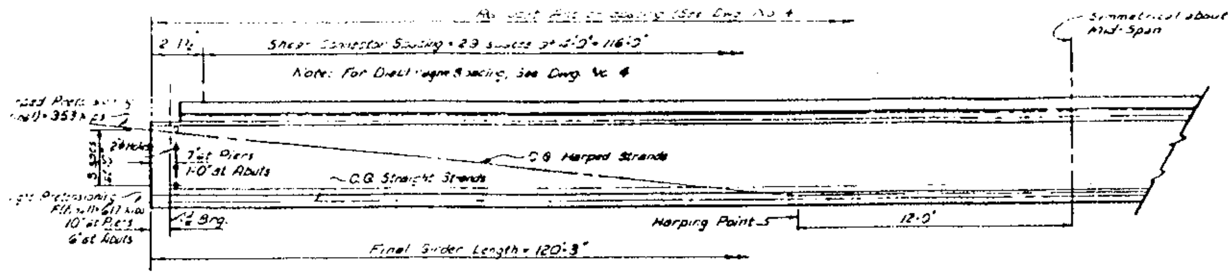
GENERAL LAYOUT

State of Alaska
DEPARTMENT OF TRANSPORTATION
and PUBLIC FACILITIES
Juneau, Alaska

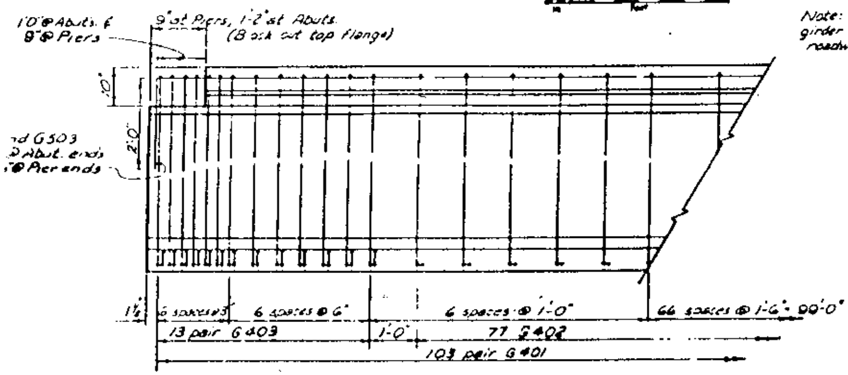
Bridge No. 685
Sheet No. 1 of 8



ABUTMENT
PLAN
PIER
Symm about ℄ Roadway
Bridge Grade-Top of Concrete
ABUTMENT
HP 10×57 Piling
ELEVATION
PIER
Note: Fill top 35' of pipe piling with reinforcing and Class A Concrete
2'-0 x ½" wall steel pipe brace
Optional ⅝"x2'-6" Adjusting Sleeve with ⅝ welds both sides
Note: Weld brace prior to placing concrete in piles
BEARING AND CAP LAYOUT
SECTION C-C
SECTION B-B
SECTION A-A
* All reinforcing steel within 3½" of the top of the concrete deck shall be epoxy coated.
MACLAREN RIVER BRIDGE
ROUTE NO. FAS 750
ABUTMENTS AND PIERS
State of Alaska
DEPARTMENT OF TRANSPORTATION and PUBLIC FACILITIES
Juneau, Alaska
BRIDGE NO. 683
SHEET NO. 3 of 8



ELEVATION

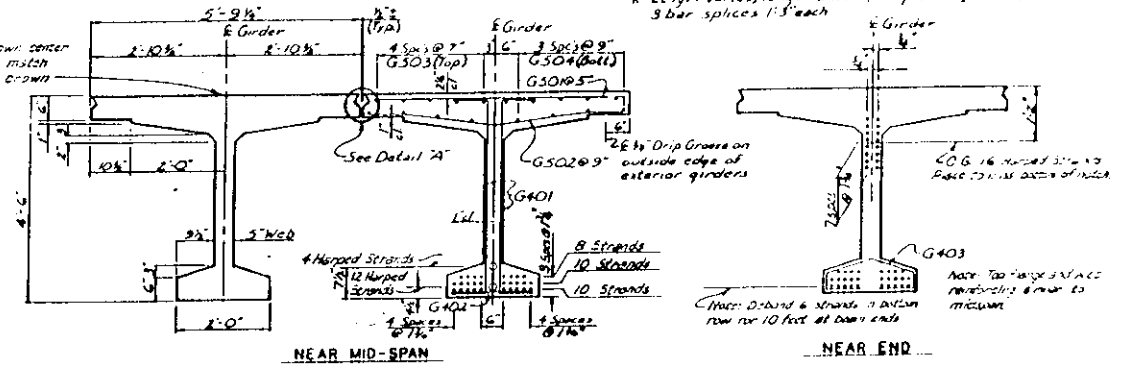


END ELEVATION

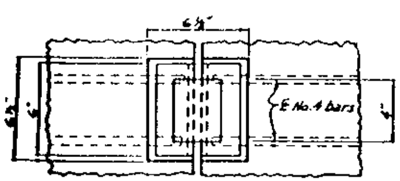
REINFORCING STEEL (ONE GIRDER)

	Mark	No.	Size	Length	Type
①	G401	[illegible]	4	[illegible]	[illegible]
	G402	77	4	[illegible]	[illegible]
	G403	[illegible]	4	[illegible]	[illegible]
①	G501	[illegible]	5	[illegible]	[illegible]
	G502	[illegible]	5	[illegible]	Bent
① *	G503	[illegible]	5	[illegible]	[illegible]
*	G504	[illegible]	5	[illegible]	[illegible]

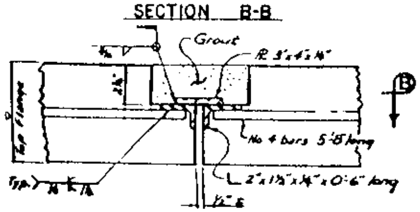
① All reinforcing within 3½" of the top of the concrete deck shall be epoxy coated.
* Length varies; length shown, longest required, includes 3 bar splices 1'-3" each



TYPICAL SECTION

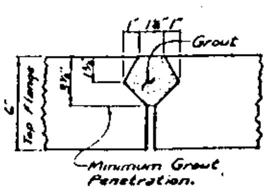


SECTION B-B



SHEAR CONNECTOR DETAIL

No Scale



DETAIL "A"

No Scale

GIRDER NOTES

Alternate designs are allowed per Section 502 of the Specifications.

Concrete for girders shall be of normal weight having the following strengths:

At Stress Transfer F'ci = 5500 psi
At 28 days F'c = 6500 psi

All prestressing shall be ½ inch round strands having an ultimate strength of 270 Ksi and an area of 0.153 sq. in.

Design is based on the following steel stresses:

Pretensioning
Jacking Stress = 189 Ksi
After Losses = 144 Ksi

One inch clear on all reinforcing except as noted.

All structural steel embedded in girders shall be A36. All structural steel shall be galvanized after fabrication.

Approximate girder weight = 120 kips

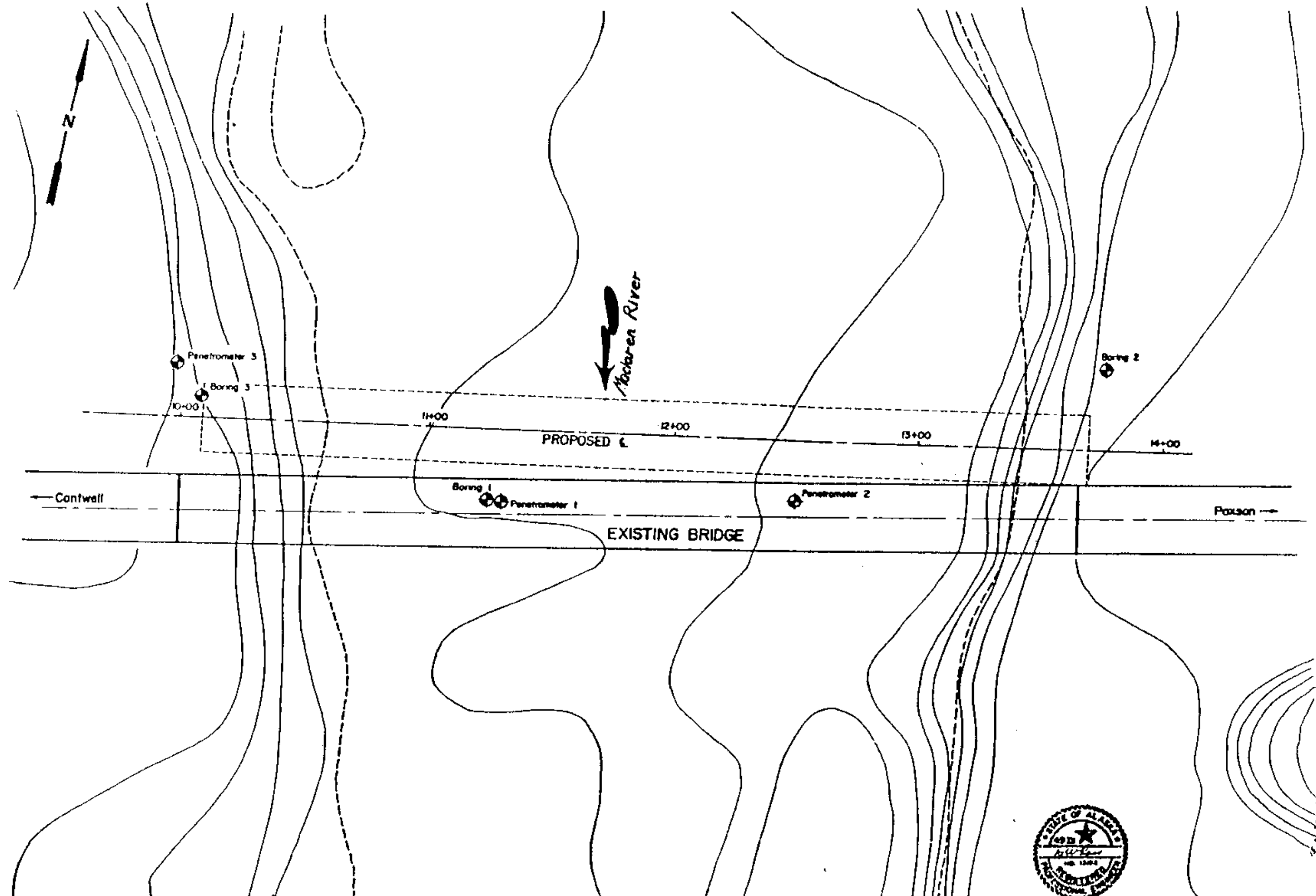
MACLAREN RIVER BRIDGE

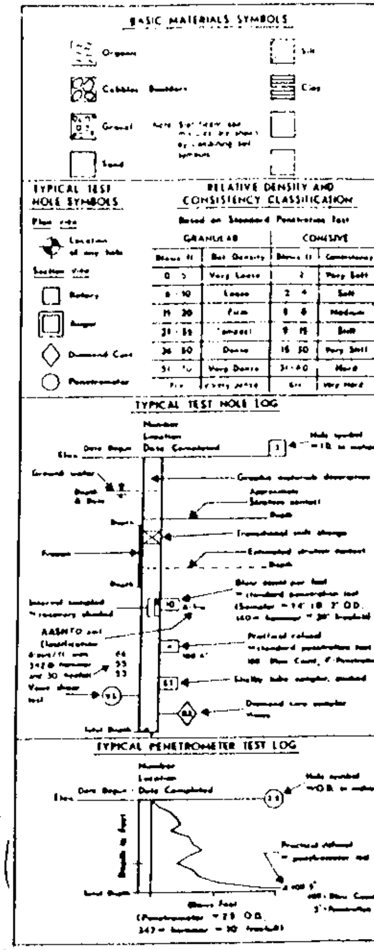
Route No. FAS-750

GIRDERS

State of Alaska
DEPARTMENT OF TRANSPORTATION and PUBLIC FACILITIES
Juneau, Alaska

Bridge No. 685
Sheet No. 5 of 8





RELATIVE DENSITY AND CONSISTENCY CLASSIFICATION

Based on Standard Penetration Test

GRANULAR		COHESIVE	
Blows/ft	Rel. Density	Blows/ft	Consistency
0-5	Very Loose	2	Very Soft
6-10	Loose	2-4	Soft
11-20	Firm	5-8	Medium
21-35	Compact	9-15	Stiff
36-50	Dense	16-30	Very Stiff
51-[illegible]	Very Dense	31-60	Hard
[illegible]	[illegible]	61+	Very Hard

SOIL GRAIN SIZE DEFINITIONS

Boulder (rounded) .... >12" Diameter
Cobble (rounded) .... 3" Diameter - 12" Diameter
Broken rock (angular) .... >3" Diameter
Gravel (rounded), Stone (angular) ...... #10 Sieve - 3" Diameter
Sand .... #200 Sieve - #10
Silt/clay .... < #200 Sieve
Note: Soil classifications are visual only unless AASHTO [illegible]

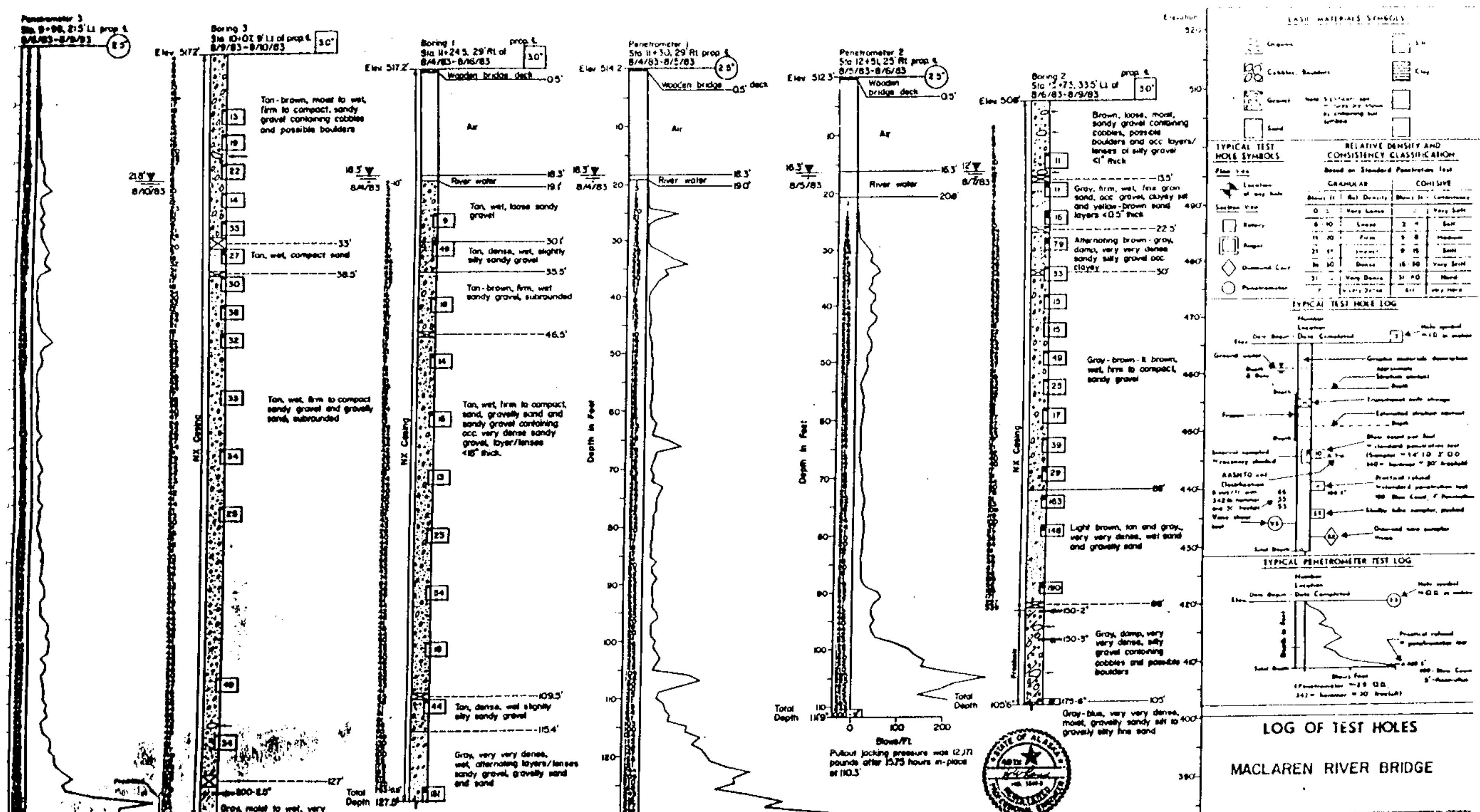
LOG OF TEST HOLES

MACLAREN RIVER BRIDGE

State of Alaska
DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES
Juneau, Alaska

Date March, 1984

Bridge No. 685
Drawing No. 7 OF 8



LOG OF TEST HOLES

MACLAREN RIVER BRIDGE

SOIL GRAIN SIZE DEFINITIONS

Boulder (rounded)	>12" Diameter
Cobble (rounded)	3" Diameter - 12" Diameter
Broken rock (angular)	>3" Diameter
Gravel (rounded), Stone (angular)	#10 Sieve - 3" Diameter
Sand	#200 Sieve - #10
Silt/clay	< #200 Sieve

Note: Soil classifications are visual only unless AASHTO soil class is shown on the log

State of Alaska
DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES
Juneau, Alaska

Date March, 1984
Approved

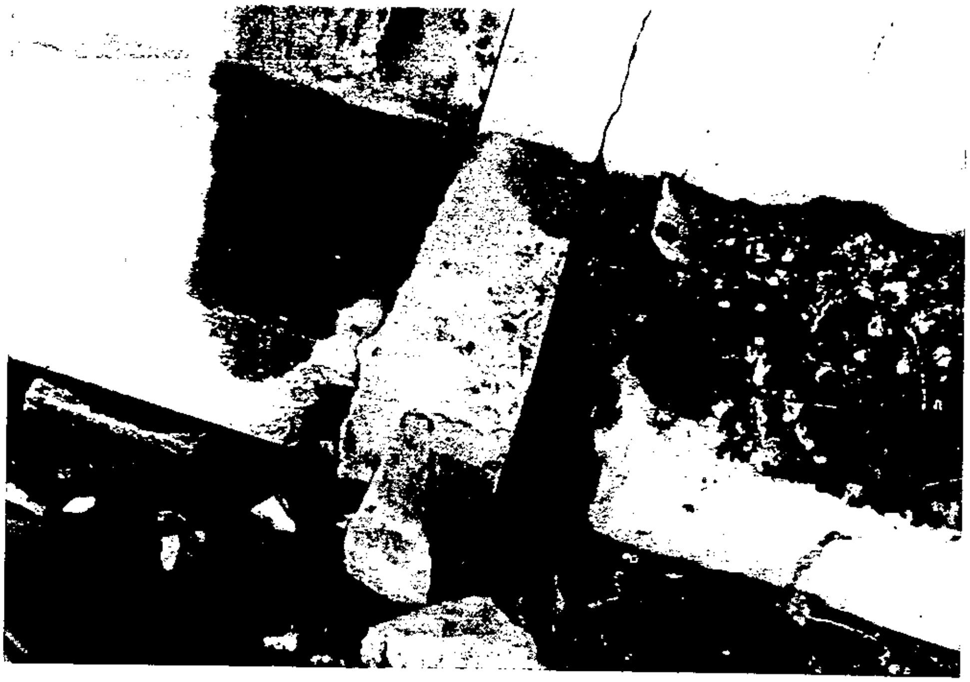
Project No. 685
Sheet No. 8 OF 8

APPENDIX C.

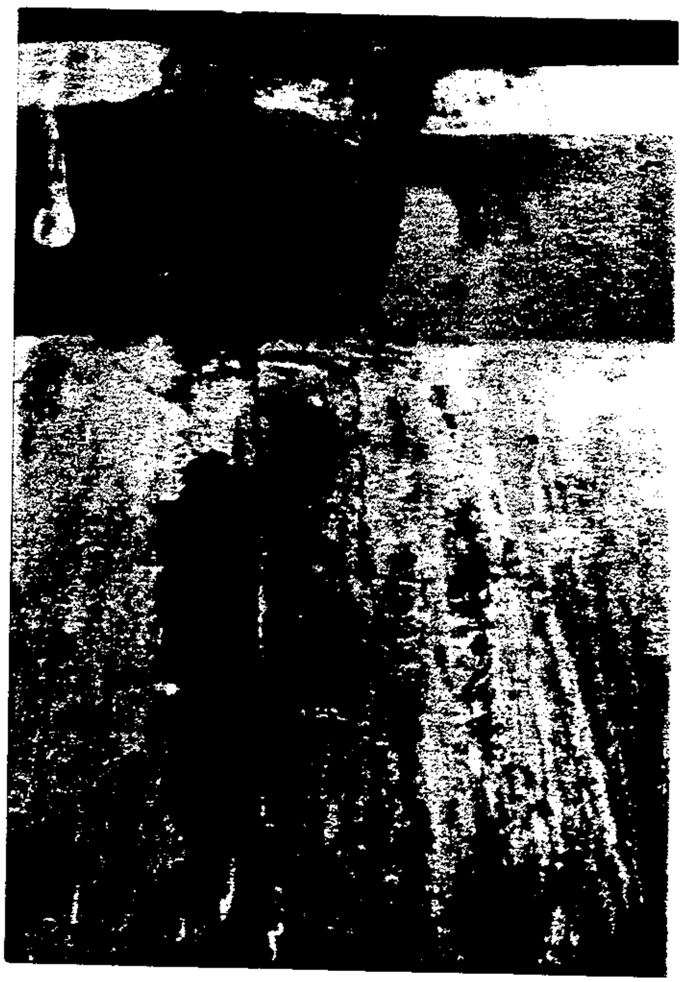
AREAS OF DISTRESS

a) Cracking-Abutment 1 Backwall

b) Cracking-Abutment 4 Backwall



c) Cracking in Pier 2 Diaphram



d) Cracking in Pier 3 Diaphram