MISSOURI COOPERATIVE HIGHWAY RESEARCH PROGRAM
FINAL REPORT

72-9

AN INVESTIGATION OF DESIGN CRITERIA FOR STRESSES INDUCED BY SEMI-INTEGRAL END BENTS: PHASE I--FEASIBILITY STUDY

MISSOURI STATE HIGHWAY DEPARTMENT
UNIVERSITY OF MISSOURI - ROLLA
FEDERAL HIGHWAY ADMINISTRATION

AN INVESTIGATION OF DESIGN CRITERIA FOR STRESSES INDUCED BY SEMI-INTEGRAL END BENTS: PHASE I--FEASIBILITY STUDY

Study 72-1

Prepared for

MISSOURI STATE HIGHWAY DEPARTMENT

by
Jack H. Emanuel
John L. Best
J. Leroy Hulsey
Joseph H. Senne
LeRoy E. Thompson

CIVIL ENGINEERING DEPARTMENT
UNIVERSITY OF MISSOURI-ROLLA
ROLLA, MISSOURI

in cooperation with

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

5-15-84

ABSTRACT

This investigation explores the feasibility of the development of a rational design criteria for bridges with Semi-Integral end bents.

The study was divided into five major parts: 1) A review of the state of the art as evidenced by the literature and a qualitative evaluation of the factors influencing bridge behavior; 2) A survey by questionnaire of the practical applications in current design practice; 3) Observation of bridge behavior patterns as indicated by inspection of selected bridges; 4) A theoretical analysis of the potential magnitudes of movements and stresses induced by temperature differentials; and 5) A discussion of instrumentation techniques and recommendations for instrumentation of a prototype bridge.

By virtue of their interaction the factors influencing bridge behavior are very complex. Although prior studies have been hampered by a paucity of usable and meaningful data, due generally to complications beyond the control of the investigator, reasonable progress has been made in individual areas such as temperature differentials and temperature distribution.

The survey of design practice established that: 1) There is no simple rational design criteria currently available which takes into account the stresses induced by environmental conditions; 2) There is a wide variance in methods used for consideration, if any, of such stresses; and 3) Bridge design engineers would welcome a simple, rational design criteria and specific recommendations as to design details.

The field observations showed trends of irregularities, but not to any degree of predictability.

From the review of the literature, it becomes apparent that bridge behavior is influenced to varying degrees by a large number of variables. However, the effect of many of these variables can be considered as negligible, and prior studies identify thermal effects, humidity, creep, shrinkage, and backfill movement and settlement as the major parameters to be considered.

For the restrained structure studied, the theoretical thermal stresses induced by assumed temperature distributions approached 20 percent of the dead and live load stresses, and when combined with other factors influencing behavior, could cause a very significant increase in stress at specific locations. These stresses are significant and warrant further study.

The investigation concludes that development of a simple and usable rational design criteria for bridge superstructures supported by flexible substructures is feasible subject to experimental substantiation of the theoretical approach. Instrumentation techniques are discussed and a detailed procedure for instrumentation of a prototype bridge for further experimental study of the major factors as mentioned above is presented.

CONTENTS

LIST OF TABLES

LIST OF FIGURES

INTRODUCTION

Safety, economy, maintenance, riding quality, esthetics, and simplicity of design and construction are factors of primary concern to the bridge design engineer. In order to economically utilize materials and yet design safely, the design engineer must be able to predict bridge behavior within acceptable known limitations. Thus, inconsistencies between design assumptions and field behavior become important.

The parameters affecting bridge behavior are innumerable and involve a complex interdependence of such factors as material properties; structure location; geometry and orientation; structure flexibility; superimposed load effects; construction procedures; soil and foundation conditions; and environmental influence (27, 34). Thus, it is understandable that opinions vary among design engineers as to the relative significance of the factors influencing behavior; the magnitudes and types of movements and/or stresses induced; and methods of analysis to be used for consideration of these effects. In the hope of providing for movement without induced stress, engineers use supporting and expansion devices ranging from simple pads or plates to elaborate rocker or roller arrangements and suspended spans. Some bridges are constructed without any supporting or expansion devices whatsoever, and such methods are receiving increased attention. Continuous girder bridges with spans approximately twice the length of those in the United States without suspended spans and interior expansion joints are being used in some foreign countries with no behavioral problems reported (69).

Field observations such as reported by Emanuel and Ekberg (27) show that in many cases the supporting and expansion devices do not function as anticipated by the design engineer. Common observations include inconsistent rocker movements, frozen rockers, and spalling and cracking of the abutments. Such observations have lead to more extensive use of the simpler and less expensive devices used primarily for precast concrete bridges and an increased interest toward the use of superstructures supported by flexible substructures.

A survey of design practice for bridge bearing and expansion devices conducted by Ekberg and Emanuel (26) confirmed that there is a wide variance in design methods and limitations for bridges of this type. Design engineers have expressed a need for additional information and rational design procedures. Thus a study was initiated by the Department of Civil Engineering, University of Missouri-Rolla, in cooperation with the Missouri State Highway Department and the Federal Highway Administration. The long range goal of the study is to develop criteria for bridges whose superstructures are supported by flexible substructures. The study is to be completed in phases, the first consisting of a feasibility study relative to recommendations toward the conduct of subsequent phases. The work reported herein is the final report of the first phase.

Scope of Phase I

The initial objectives of Phase I were: 1) To review the state of the art as evidenced in the literature, 2) To determine and qualitatively evaluate the factors influencing bridge behavior, 3) To conduct observations and review the comparative behavior of selected bridges, and 4) To evaluate the relationship of data obtainable through instrumentation toward development of design criteria and make recommendations toward subsequent phases of the study. A survey by questionnaire of current design practice was also conducted to supplement the literature review and establish the basis for current design practice. The study reported herein is divided into five major parts as follows:

1. A review of the literature;
2. A survey by questionnaire of the design practice of state highway departments and some governmental agencies;
3. Field observations of selected bridges; and
4. A comparison of design and theoretical induced stresses.
5. A discussion of instrumentation techniques and recommendations for instrumentation of a prototype bridge.

CONCLUSIONS

This study was initiated as a result of an increased interest in and usage of bridge superstructures supported by flexible substructures, and the apparent lack of information concerning bridge behavior and induced stresses associated with bridges of this type. Specifically, the investigation was to explore the feasibility of the development of a rational design criteria for bridges with Semi-Integral end bents.

The initial objectives of the study were: 1) to review the state of the art as evidenced in the literature; 2) to qualitatively evaluate the factors influencing bridge behavior; 3) to conduct field inspections of the comparative behavior of selected bridges; and 4) to evaluate the relative significance of data obtainable through instrumentation of a model or prototype bridge, and make recommendations as to the feasibility of such an investigation. As the investigation progressed it became apparent that both a survey of current design practice and a theoretical analysis of a Missouri State Highway bridge, designed in accordance with standard procedures and specifications, for potential magnitudes of movement and stresses induced by thermal effects, a major parameter, would be useful to the study. Thus, the initial objectives were altered slightly.

The conduct of the investigation was divided into five parts as follows:

1. A review of the state of the art as evidenced by the literature and a qualitative evaluation of the factors influencing bridge behavior.
2. A survey by questionnaire of the practical applications in

current design practice.

3. Observation of bridge behavior patterns as indicated by inspection of selected bridges.
4. A comparison of design dead and live load stresses with theoretical induced thermal stresses.
5. A discussion of instrumentation techniques and recommendations for instrumentation of a prototype bridge.

Survey of Current Design Practice

Some of the primary objectives of the survey were:

1. To determine if there exists a rational design criteria which accounts for time-dependent effects and structural geometry for bridges with superstructures supported by flexible substructures and the extent to which this type of construction is used.
2. To identify the factors considered significant to bridge behavior or which might be indicated through problems encountered and objections to usage.
3. To establish the maximum allowable design length between positive expansion devices and the maximum permissible length of bridge deck when partially or fully restrained end bents are used.
4. To determine the interest in and need for future research in the area of restrained structures.

A questionnaire and cover letter was sent to 50 State Highway Departments and 5 Governmental agencies. From the replies of 43 State Highway Departments and 3 Governmental agencies, resulting in a total response of almost of 84 percent, the following conclusions were apparent.

1. Although differences of opinions remain, the use of superstruc-

tures supported by flexible substructures is becoming generally accepted.

2. Current design practice is more restrictive for composite steel bridges than for concrete structures.
3. There is no simple rational design criteria currently available which takes into account environmental effects in conjunction with structural type and geometry.
4. Maximum expansive lengths of 300 ft for steel structures and of 400-450 ft for concrete structures are generally recognized by design engineers utilizing superstructures supported by flexible substructures. However, expansive lengths of 671 and 736 ft have been reported for Non-Integral and Semi-Integral steel structures, respectively, and lengths of approximately 500 ft have been reported for Non-Integral, Semi-Integral and Integral concrete structures.
5. Induced stresses resulting from thermal effects, creep, shrinkage, backfill movement and settlement, etc., are recognized by bridge design engineers as potentially significant. There is a wide variance in methods used for consideration, if any, of such stresses.
6. Some problems were reported for both steel and concrete structures for the three types of construction. It would appear that, in general, the problems reported are neither more prevalent nor of greater magnitude than those experienced when movable supporting and expansion devices are used.
7. Bridge design engineers are extremely interested in induced stresses and associated problems. Most design engineers are

uncertain as to the magnitude of induced stresses in both unrestrained and restrained bridges; and would welcome a simple, rational design criteria and specific recommendations as to design details.

Field Observations of Selected Bridges

This phase of the study consisted of field observations throughout the State of Missouri of the behavior of 35 selected composite steel bridges having Semi-Integral end bents.

Some of the primary objectives of the field observations were:

1. To determine if induced stresses resulting from environmental effects cause apparent structural distress in bridges constructed with Semi-Integral end bents.
2. To provide a comparison of the behavior of bridges with Semi-Integral end bents and conventional bridges with frozen supporting devices.
3. To provide an insight to the factors influencing the behavior of bridges with Semi-Integral end bents and the relative magnitudes of their effect.

Although not observed to any degree of predictability, the following trend of irregularities became to be expected:

1. Cracking of abutments under the girders;
2. Parapet cracking near the abutments--often at the first handrail bracket;
3. Curb and edge deck cracking at the inflection points;
4. Edge deck cracking at the piers; and
5. Parapet cracking approximately 3 to 6 ft each side of the mid-

point of the center span.

It should be noted, however, that the mere occurrence of these irregularities is not indicative of the structural integrity and safety of the bridge, although they may permit ingress of moisture and lead to subsequent deterioration. Also, abutment cracking under the girders is the only irregularity which may be considered as peculiar to Semi-Integral end bents--as compared to conventional methods of construction. Some of the most extensive abutment cracking under the girders, as well as parapet, curb and edge slab cracking, was observed in bridges which were not yet open to traffic, including a set of twin bridges painted in 1970 but without approach slabs.

Review of Literature

The primary objectives of this portion of the study were:

1. To establish the relative significance of the parameters influencing bridge movement and stresses.
2. To identify the state of the art and rational methods of analysis, if any.
3. To examine prior research in the areas of movement, strains, stresses, and instrumentation of bridge structures.
4. To evaluate the feasibility of future research toward establishing a simple, rational design criteria for such structures.

Secondary objectives especially directed to bridge instrumentation were:

1. To review related investigations, the parameters studied, and the relative successes and/or failures.

2. To establish causes for problems encountered in prior studies that they might be avoided in future studies.

3. To relate the success of past research with the need for future study and the feasibility of development of a design criteria for bridges with Semi-Integral end bents.

The following conclusions from the review of literature are summarized below.

Although a large number of factors are considered to affect bridge behavior, many have a negligible effect on overall movements and/or induced stresses. The factors considered most significant and those proposed for further study are: creep, shrinkage, temperature, humidity, and backfill movements and settlements. Temperature may normally be considered the primary cause of movements and induced stresses following subsidence of creep and shrinkage. Even though large overall movements occur with seasonal changes, daily fluctuations of temperature and differences in the thermal coefficient of expansion are considered to be the major causes of induced stresses due to a non-uniform temperature distribution throughout the depth of the bridge.

Although prediction procedures to account for the effects of such items as creep and shrinkage exist, only limited data are available in the area of thermal movements which are applicable to restrained effects in structures; as for example, bridges with Semi-Integral end bents. There appears at this time to be no complete theory or rational approach which takes into account all of the major parameters affecting bridge behavior, either restrained or unrestrained.

Since bridges are subjected to a large number of influences during construction and usage, special techniques are required in order to obtain meaningful and useful data. Factors such as instrumentation noise, atmospheric conditions, vandalism, power failures, measurement of stress without strain and strain without stress are some of the items which must be considered during instrumentation of a structure for field testing. In addition, the type of data to be obtained must be considered. Long-term stability gages, such as the Carlson strain meter, vibrating wire gages, and sophisticated sensing devices are extremely important to testing of environmental effects. Fluctuations in cyclic data are often so rapid that meaningful data cannot be recorded by hand recording methods.

However, some researchers have reported successful techniques and give indications as to the range of magnitudes of the effects of the major parameters affecting bridge behavior. Utilization of these techniques yield useful data which may be combined with theoretical concepts for development of a feasible simple design criteria accounting for the major parameters affecting bridge behavior.

Prediction of Induced Bridge Stresses

The intent of this portion of the study was to establish the relative order of magnitude of induced stress caused by one of the major parameters affecting bridge behavior. The parameter selected for illustration was that of thermal effects. The bridge studied was evaluated as a longitudinally restrained structure; in this instance as a Semi-Integral end bent bridge. Although general conclusions cannot be made for all bridges, it was shown that, based upon the assumed temperature distribution, thermal effects may induce stresses of significant magnitude. For the restrained structure studied, the induced thermal stresses approached 20 percent of the dead and live load stresses, and when combined with stresses induced by other factors such as creep, shrinkage, and humidity could cause a very significant increase in stress at specific locations. Thus, it may be concluded that these stresses are significant to both unrestrained and restrained structures and warrant further study.

As bridge structures get longer between restraints, axial effects increase and should therefore be considered when designing the abutment shear key and for the effects of induced stresses in the piling, which act as part of the restraints to superstructure movement. Also, other effects such as soil pressures at the abutment may act as additional restraints and consequently increase the resulting longitudinal forces.

Summary of Conclusions

From the literature survey, the survey by questionnaire, field inspection of selected bridges, and the theoretical calculations of induced stresses the following is concluded:

1. Although bridges with superstructures supported by flexible substructures are being successfully designed and used, the general lack of information concerning bridge behavior and associated stresses is acknowledged by bridge design engineers and a rational design criteria would be welcomed.
2. The magnitude of stresses induced in bridge superstructures supported by flexible substructures may become a significant percentage of the allowable stress and are of sufficient magnitude to warrant further study of actual structures and comparison with theoretical values.

3. Prior investigators have identified temperature, humidity, creep, shrinkage, and backfill settlement and movement as the major parameters affecting bridge behavior.
4. Although many investigators have experienced difficulty in obtaining meaningful field data, others through proper instrumentation techniques have identified some parameters affecting bridge behavior and, within reasonable limits, relative magnitudes.
5. Usable data for evaluation of bridge behavior and induced stresses can be obtained, provided long-term stability gages and sophisticated sensing devices are utilized--e.g., the Carlson strain meter.
6. Laboratory testing and evaluation of sensors and instrumentation procedures preliminary to a field study is of major importance to any investigation; especially to a long-range study of a prototype bridge.
7. It is recommended that field investigations be made in order to check and simplify the theoretical approach developed in this study for partially or fully restrained end bent bridges subjected to environmental effects.

A discussion of instrumentation techniques and a detailed procedure for instrumentation of a prototype bridge are presented in the Recommendations for Future Study. Based upon this investigation, it is concluded that the development of rational design criteria for bridges with Semi-Integral end bents is feasible.

REVIEW OF LITERATURE

Uncertainties as to the magnitudes and types of bridge movements and the effects of the stresses subsequently induced continue to be of major concern to bridge design engineers. Not only are the variables affecting bridge movements, such as temperature, humidity, creep, shrinkage, concrete growth, and soil movements, extremely numerous, but the magnitudes of their effects have not been established.

Current design practice generally ignores induced strains and stresses in simple span and continuous structures and assumes that expansion joints, should they exist, relieve induced stress. Field observations of expansion devices which have not performed as anticipated have caused an increased interest in the usage of bridges constructed with superstructures supported by flexible substructures. However, apparently there is no simple rational method taking into account movements and stresses induced in such structures.

Thus, in this phase of the investigation a review of the literature related to movements and stresses induced in bridge superstructures with Semi-Integral end bents was conducted. The objectives of this phase were:

1. To establish the relative significance of the parameters influencing bridge movement and stresses.
2. To identify the state of the art and rational methods of analysis, if any.
3. To examine prior research in the areas of movement, strains, stresses, and instrumentation of bridge structures.
4. To evaluate the feasibility of future research toward establishing a simple, rational design criteria for such structures.

The factors which, under given conditions, may influence bridge behavior are myriad. Some of the variables affecting bridge movement behavior are reported to be (27, 34).

A. Properties of Materials and Structural Elements
   1. Coefficient of thermal expansion
   2. Thermal diffusivity
   3. Coefficient of thermal conductivity
   4. Substructure flexibility
   5. Ductility
   6. Strength
   7. Resistance to chemical action
   8. Corrosion resistance
   9. Porosity and moisture absorption
   10. Concrete--materials affecting mix

B. Environmental Effects
   1. Fluctuation and range of ambient temperature
   2. Solar radiation
   3. Precipitation
   4. Humidity
   5. Wind
   6. Earthquake

C. Geometry
   1. Bridge orientation
   2. Degree of skew
   3. Allowance for expansion
   4. Physical arrangement of bridge deck, abutments and approaches
   5. Reinforcing
   6. Composite vs. noncomposite action
   7. Span length
   8. Overall structure length

D. Other Effects
   1. Type of bridge
   2. Type and arrangement of devices used
   3. Relationship between dead and live load
   4. Traction of live load
   5. Frequency and direction of traffic
   6. Speed of vehicles
   7. Type and condition of wearing surface
   8. Type and condition of approaches
   9. Type and height of abutment fills
   10. Method of compaction
   11. Stability of soil
   12. Types of soil strata
   13. Fluctuation of water table
   14. Horizontal and vertical movement of piers and abutments
   15. Accumulation of debris
   16. Construction tolerance
   17. Maintenance

Any attempt to account for all of the possible variables becomes virtually impossible. Thermal effects, shrinkage, creep, wind, braking forces, and movements resulting from abutment fill are considered to be major contributors to longitudinal and vertical movements, and thus have received more attention over the years. The following review of literature gives an insight into the parameters studied to date.

Thermal Behavior

An unrestrained structure composed of a homogeneous isotropic material will experience axial elongation without stress when subjected to a uniform temperature change, and will experience curvature and axial elongation without stress when subjected to a linearly varying temperature change. If completely restrained against movement, the structure will experience stress without strain when subjected to temperature change (33, 49). If the structure is unrestrained and subjected to a nonuniform temperature distribution throughout its depth, both stress and strain will occur (33). The thermal behavior of composite bridge structures is further complicated due to the use of dissimilar materials. It is customary to assume the thermal coefficient of expansion for steel and concrete to be 6.5×10^{-6} and 6.0×10^{-6} in./in./° F, respectively. However, contrary to popular belief the thermal coefficient of expansion of concrete ranges between 1.2×10^{-6} to approximately 9.3×10^{-6} (8, 11, 20, 23, 54, 81, 83) depending on material constituents, moisture conditions and temperature. Imposed restraints result from the difference in coefficients of the steel and concrete. Bearing friction and substructure flexibility also impose additional restraints and stresses.

Concrete shortens as a result of movements due to elastic shortening, shrinkage and cooling. Zederbaum (88, 89) has shown that a bridge structure normally shortens or elongates by both ends moving inward or outward and, thus, that some point on the bridge deck does not move. Knowing the point of zero movement--sometimes referred to as the stagnation point--the designer may evaluate the induced forces in the structure as a result of superstructure and substructure movements. The stagnation point is determined by locating the center of gravity of horizontal stiffness of the supports (see procedure in Appendix C). Deck movements relative to the point of zero movement and associated stresses due to shrinkage, creep, temperature, and elastic shortening effects may be calculated (86).

Thermal Coefficient of Expansion and Contraction

The variance of the thermal coefficient of expansion for concrete has received little, if any, attention in the codes. However, thermal stresses in simple composite girder bridges resulting from temperature changes have been calculated to be large in magnitude as a result of the different material properties (90).

Chow found the average thermal coefficient of expansion for limestone and gravel aggregates to be 55 and 70 percent of the coefficient of expansion of steel ($6.5 \times 10^{-6}/°F$), respectively. He also found that there is a certain amount of residual expansion after a complete freeze-thaw cycle (23).

Walker, et al., (83) found that coarse and fine aggregates having thermal coefficients different than the mortar caused changes in the thermal coefficients in proportion to the volume of aggregate. The degree of saturation was found to have an effect on the thermal coefficient. Saturated and oven dry samples had similar coefficients of expansion, whereas partially dry samples had considerably higher coefficients of expansion.

In his tests of thermal coefficients for concrete, Callen (20) found that when the difference between the coefficients of expansion for mortar and for aggregates is large, the durability decreases under freeze-thaw cycles. Limestone aggregate used with a natural fine, siliceous aggregate showed signs of lowered durability. In addition, Callen reported that limestone-sand mortars normally have coefficients in the range of 4 to 5 x $10^{-6}/°$ F while most natural siliceous-sand mortars have coefficients of about 6 x $10^{-6}/°$ F.

Tables 1 through 5 illustrate the variations in thermal coefficients and are included as a reference for selection of values for various aggregates.

Table 1. Comparison of Thermal Coefficients of Concrete[a]

Coarse aggregate	Fine aggregate	Moisture condition of concrete	Coarse aggregate, percent volume of concrete: 0	20	40	50	100
			Coefficient of expansion per deg F x 10^{-6}				
Quartz gravel	Quartz sand	Saturated	6.14	6.14	6.24	6.22	
		Partially dry	8.53	7.81	7.30	7.24	6.3
		Oven dry	6.79	6.59	6.35	6.32	
Crushed limestone	Quartz sand	Saturated	6.14	5.13	4.33	4.02	
		Partially dry	8.53	6.20	4.53	4.28	2.45
		Oven dry	6.79	5.23	4.12	3.71	
Quartz gravel	Mfd. limestone	Saturated	3.98	4.47	4.84	5.08	
		Partially dry	5.35	5.61	5.76	5.81	6.3
		Oven dry	4.38	5.06	5.40	5.60	
Crushed limestone	Mfd. limestone	Saturated	3.98	3.78	3.58	3.42	
		Partially dry	5.35	4.57	4.04	3.67	2.45
		Oven dry	4.38	4.00	3.62	3.51	

[a]From Walker, et al. (83).

Table 2. Comparison of Temperature Change Cycle[a]

		Thermal coefficient per deg F x 10^{-6}					
		Quartz sand and gravel			Trap rock + Calc. sand		
Temp. cycle	Direct. of change	Normal conc.	Air-Entr. conc.	Avg.	Normal conc.	Air-Entr. conc.	Avg.
40° F water 0° F air	Expan.	7.41	7.69	7.55	5.99	6.88	6.44
	Contr.	7.39	7.92	7.65	5.75	6.90	6.32
	Avg.	7.40	7.80	7.60	5.87	6.89	6.38
40° F air 0° F air	Expan.	7.16	7.45	7.30	5.25	5.51	5.38
	Contr.	7.12	7.39	7.26	5.59	5.44	5.52
	Avg.	7.14	7.42	7.28	5.42	5.48	5.45
140° F water 40° F water	Expan.	6.49	6.40	6.44	4.54	4.48	4.51
	Contr.	5.59	5.53	5.56	4.13	4.10	4.12
	Avg.	6.04	5.96	6.00	4.34	4.29	4.32
140° F air 40° F air	Expan.	5.97	6.09	6.03	4.56	4.71	4.64
	Contr.	5.90	5.93	5.92	4.75	4.82	4.78
	Avg.	5.94	6.01	5.98	4.66	4.76	4.71
40° F H_2O to 40° F air to 140° F air to 140° F water and back	Expan.	6.18	6.19	6.18	4.50	4.53	4.52
	Contr.	6.04	6.24	6.14	4.46	4.47	4.46
	Avg.	6.11	6.21	6.16	4.48	4.50	4.49
160° F to 60° F in water	Expan.	6.60					
	Contr.	6.63					
	Avg.	6.62					

[a]From Walker, et al. (83).

Table 3. Thermal Coefficient of Expansion of Various Materials[a]
(Nominal Temperature Range, 77-82° F)

Material and Source	Coef. of exp. x 10^{-6} /° F	Material and Source	Coef. of exp. x 10^{-6} /° F
Pyrex Glass...........	1.94	Dolomite, Wis.........	5.75
Marble, Georgia "Tate"--Gray........	2.99	Gravel, Eau Claire, Wisconsin...........	5.94
Marble, Georgia "Tate"--White.......	3.02	Sand, Beloit, Wis.....	5.99
Trap Rock, Penn.......	4.31	Slag, Ensly, Ala......	6.25
Granite, Camak, Ga....	4.47	Limestone, Thornton, Illinois............	6.41
Granite, Lithonia, Ga.	4.68	Slag, Woodward, Alabama.............	6.49
Trap Rock, Wis........	4.74	Sand, Standard, Ottawa, Ill.........	6.58
Slag, Birmingham, Ala.................	5.12	Sand, Roquemore, Montogomery, Ala....	6.61
"Sand-Gravel", Platte River, Schuyler, Nebr................	5.20	Gravel, Roquemore, Montgomery, Ala.....	6.71
Limestone, New York Dolomitic...........	5.25	Sand, Victoria, Texas.	6.72
"Sand-Gravel," Platt River, S. Bend, Nebr................	5.49	Sand, Chattanooga, Tenn................	6.73
Limestone, Elmhurst, Illinois.....	5.71	Gravel, Chattanooga, Tenn................	6.73
"Sand-Gravel," Florena Switch, Kansas.	5.72	Gravel, Chert, Grand River, Okla.........	7.02
Sand, Elgin, Illinois.	5.73	Gravel, Victoria, Texas...............	7.13
"Sand-Gravel," Republican River, McCook, Nebr..........	5.73	Copper................	9.32

[a]From Verbeck and Hass (81).

Table 4. Thermal Coefficients[a]

Aggregate	Temperature °F	Coef.[b] of contractions 10^{-6}	Coef.[c] of expansion 10^{-6}	Average thermal coef. 10^{-6}
Limestone	T^d - 32	4.00	----	----
	32 - 0	3.65	----	----
	0 - T^e	3.18	----	----
	T^f - 0	----	4.02	----
	0 - 32	----	4.73	----
	32 - T^g	----	4.67	----
	average	3.60	4.47	4.04
Gravel	T^d - 32	5.03	----	----
	32 - 0	4.56	----	----
	0 - T^e	4.07	----	----
	T^f - 0	----	4.80	----
	0 - 32	----	5.57	----
	32 - T^g	----	4.67	----
	average	4.55	5.01	4.78

[a]From Chow (23).

[b]Coefficient of contraction as determined for an average of 9 limestone specimens and 5 gravel specimens under decreasing temperature cycles.

[c]Coefficient of expansion as determined for an average of 9 limestone specimens and 5 gravel specimens under increasing temperature cycles.

[d]Initial temperatures for this series of tests ranged between 74.8° F and 44.4° F.

[e]Final temperatures ranged between -35.3° and -27.5° F.

[f]Initial temperatures range between -36.8° and -27.5° F.

[g]Final temperatures ranged between 73° and 51° F.

Table 5. Thermal Coefficient of Rock Minerals From Various Sources[a]

Classification--Description and Source	Coef of exp. x 10^{-6} /° F
LIMESTONE	
Limestone pebble from aggregate, Hungry Horse Dam, Mont.	3.7
Limestone from tunnel at Hungry Horse Dam, Mont.	4.5
Quarried limestone (Cedar Bluff) from Fort Riley, Kan.	3.0
Limestone from Pike View, Colo.	2.4
Limestone from Pike View, Colo.	3.8
Limestone from Pike View, Colo.	1.2
Quarried "Cottonwood limestone," Manhattan, Kan.	2.8
Quarried "Cottonwood limestone," Manhattan, Kan.	2.6
Quarried "Cottonwood limestone," Manhattan, Kan.	2.2
Pebble from Republican River gravel, Colo.	4.2
Specimens are three pebbles of	
chalky limestone from Republican River gravel	1.9
opaline limestone from Republican River gravel	1.9
argillaceous limestone from Republican River gravel	1.4
Quarried limestone from Angostura Dam, S. Dak.	2.0
Small slivery samples of crushed siliceous magnesium limestone from California	6.5
	5.2
	5.2
Sandy limestone pebble from gravel, Palisades Dam, Idaho	5.3
Kaibab limestone from near Glen Canyon, Ariz.	4.5

[a]From Mitchell (54).

The values presented in the tables illustrate the effect of the constituent materials upon the thermal coefficient for concrete. It will be noted that the coefficient varies with temperature and moisture. Generally, the thermal coefficient and thermal conductivity for lightweight aggregate concrete is lower than that for normal weight concrete (39). Also, the thermal coefficient for reinforced concrete is different than that for plain concrete. Berwanger (11, 12) presented formulas for determination of an effective thermal coefficient for reinforced concrete sections taking into account both symmetrical and non-symmetrical reinforcement. The following formulas were given for determination of effective thermal coefficients for symmetrically reinforced concrete slabs and for slabs with only one layer of reinforcement.

$$C_s, C_{ss}, C_{sc} = K_c + \frac{(K_s - K_c)(1/A_cE_c + ey/E_cI_c)}{(1/A_cE_c + 1/A_sE_s + e^2/E_cI_c)} \tag{1}$$

where

b, t = Width and thickness of slab, respectfully, in.

A_s = Area of reinforcement, sq in.

$A_c = bt - A_s$ = concrete area, sq in.

d' = concrete cover measured from the center of the bars, in.

E_c, E_s = Modulus of elasticity of the concrete and reinforcing, respectfully, psi.

$\bar{y} = A_s(t/2 - d')/(bt - A_s)$ = the location of concrete centroid, measured from the reinforcing steel.

$e = (t/2 + \bar{y} - d')$ = Eccentricity of temperature induced force, in.

K_c, K_s = Thermal coefficient of concrete and steel, respectfully, in./in./° F.

C_s = Coefficient of expansion of symmetrical reinforced concrete slab, in./in./° F.

C_{ss} = Coefficient of expansion of unsymmetrical reinforced concrete slab, concrete face at steel bars, in./in./° F.

C_{sc} = Coefficient of expansion of unsymmetrical reinforced concrete slab, concrete face remote from steel bars, in./in./° F.

$I_c = (bt^3/12 + bt\bar{y}^2 - A_se^2)$ = Moment of inertia, in.^{4}

$y = t/2 + \bar{y}$ for C_{ss}

$y = -(t/2 - \bar{y})$ for C_{sc}

Air Temperature

Air temperature as well as other weather data such as humidity, precipitation, wind, and solar ratiation are recorded for most regions of the world. With proper interpretation, local weather data may be used in design for provision of movements and/or stresses induced by temperature.

Both the daily cycle and the yearly temperature cycle are important to the engineer. The daily cycle may be responsible for large temperature differentials, which may result in high induced stresses even in simply supported composite steel-girder bridges and the yearly cycle provides data for the maximum movements during the year. Daily temperatures basically follow two cycles (10). The normal minimum daily temperature is reached at or shortly before sunrise, when the structure temperature approaches or reaches equilibrium with the air temperature, followed by a general warming trend until the maximum temperature is reached at or shortly before sunset. These temperature cycles may be altered by air masses and precipation such as rain or snow or cloudiness.

British researchers (21, 28, 78) have used weather bureau records in an attempt to define the response of bridges to changes in the environment. Berwanger (10) used the Winnipeg International Airport weather data to select three possible temperature distributions for use in bridge analysis.

Steward (74) in his study of 231 expansion joints in 80 structures obtained temperature data from local Government gage stations for use in calculating coefficients of expansion.

Zuk (92) modified Barber's formulas (5) for determination of bridge temperatures in Virginia based on measured ambient air temperature. It was found that the experimental data correlated well with the calculated values. Both bare concrete decks and decks with thin asphaltic overlays were considered.

Koslov and Desai (44), using Louisiana Highway Department field observation data of air temperatures and bridge deck concrete temperatures, established relationships based on Zuk's formulas for time-temperature-depth. Phase lags, the phase angle difference between the assumed sine curves of surface temperatures between the top and the bottom of the slab, were found to increase with time approximately in a linear manner.

Bridge Temperatures and Stresses

Composite steel girder bridges are subject to a complex temperature distribution throughout the structure due to the use of bi-materials, the heterogeneity of concrete, differences in thermal diffusivity and thermal conductivity of the materials used, and exposure conditions of the top and bottom surfaces of the bridge deck and beams. However, even with this complexity the bottom elements of a composite girder bridge will generally have the same temperature as the air due to the high conductivity of the steel (92). The temperature of the upper elements of the bridge and the exterior girders will vary due to such factors as solar radiation, wind, and precipitation (10). The top of the deck when exposed to the sun is warmer than the bottom elements, the variance depending on material properties, color and solar radiation. When subjected to precipitation such as rain or snow the deck cools more rapidly than the girders resulting in a differential temperature distribution through the structure. A uniform temperature distribution may exist at or just before sunrise if the air temperature conditions have been constant for several hours and the structure has reached equilibrium with the air temperature.

Several temperature distributions have been assumed by investigators. Berwanger in his study of a three-span continuous composite girder bridge (11) concluded that three temperature distributions are likely to occur.

A uniform temperature is possible just before sunrise when the air temperature and the structure have reached equilibrium. In the other two cases, one due to warming and the other due to cooling, non-linear temperature distributions would occur throughout the slab. In all cases, the bottom of the slab was assumed to have the same temperature as the girder. Maher (49) in a study of continuous prestressed concrete box bridges assumed a linear temperature differential through the top slab based on observations of the Medway Bridge and the Western Avenue Bridge in England and the Victoria Bridge in Australia.

Narouka, et al., (55) took measurements on the interior of a composite girder bridge when it was constructed in Japan in 1955. The tests showed that the slab did not experience a uniform temperature throughout the slab during two days of testing for 10-hour periods. The slab had an asphalt surface which varied between 1-in. and 2-in. thick. The highest surface temperature was 122° F when the highest air temperature was 95° F above the bridge and 86° F below the bridge. The concrete slab reached its highest temperature approximately 2 hours later at 4:00 p.m. and the temperature of the upper and lower parts of the slab was 108° F and 91° F, respectfully. The maximum temperature differential in the concrete slab was about 16° F and the maximum differential between the upper and lower flanges of the steel girder was about 5° F.

Emerson (28) ran long term measurements on three bridges in southern England in an attempt to establish the range of temperatures and movements for which a bridge should be designed.

Measurements of extreme values of shade temperature and mean bridge temperature were conducted on the Medway bridge, 70 percent concrete beam and slab, 30 percent variable depth concrete box; the Hammersmith Flyover, concrete box spine beam; and the Beachley Viaduct/Wye bridge, steel box in southern England. The mean bridge temperature was given by the sum of the products of isotherms and their mean temperatures divided by the total area of the cross-section of the bridge, which equals the total bridge movement divided by the product of the thermal coefficient and the total length of bridge. It was reported that the results of these tests, based on the two concrete bridges, showed the maximum mean bridge temperature on a hot day to be only a few degrees above the mean shade temperature. The tests further indicated that the lowest mean bridge temperature occurred at 9:00 a.m. ± 1 hr (B.S.T.) and agreed closely with the mean shade temperature; however, no direct correlation was possible between the mean bridge and mean shade temperatures because of lag time.

Test results on the Beachley Viaduct/Wye bridge showed that the temperature range in the steel box section was greater than the range of shade temperature in the same area. Based on the test results it was concluded that on cold days during the winter, the minimum mean bridge temperature would fall from 5° to 7° F below the minimum shade temperature and during the summer the maximum mean bridge temperature would be approximately 1.5 times the mean shade temperature based on degrees centrigade. It was concluded that the ranges of bridge temperatures quoted in current British Standards were inadequate, with particular reference to the minimum temperatures--which were shown to be not sufficiently low.

Capps and Emerson (21) measured a maximum temperature differential of 42° F between the top and bottom surfaces of the steel box (Beachley Viaduct/Wye bridge) when covered with a $1\frac{1}{2}$-in. asphaltic overlay.

Barber (5) presented formulas to predict pavement surface temperatures based on observed weather data. The formulas show a close correlation between calculated and predicted values. The formulas take into account the relationship between pavement temperature and wind, precipitation, air temperature, and solar radiation controlled by the thermal properties of the pavement.

Zuk (92) modified formulas introduced by Barber (5) to calculate surface temperatures of concrete bridge decks for the Virginia area. The following formulas were proposed for prediction of maximum surface temperature, in degrees farenheit, for the middle Atlantic States' based on the assumption of a sinusoidal effective daily temperature cycle, and assuming average values for the secondary parameters. Whether subjected to hot or cold temperatures, bridge temperatures generally decay rapidly with depth and the temperature at middepth approximates that at the bottom of the slab.

$$T_m = T_a + 0.018L + 0.667(0.50T_r + 0.054L) \qquad (2)$$

where T_a equals the average daily temperature (° F), T_r equals the daily range of air temperature in ° F, and L is the solar radiation received on a horizontal surface in cal/sq cm/day; thus the maximum bridge surface temperature is T_m.

The effect of asphaltic overlays on bridge decks can raise the temperature of the deck 15° F or more above that of the bare concrete deck. If an asphaltic surface is anticipated in the future, proper provisions must be considered. This can be an important factor in the differential temperature between the top and the bottom of bridge decks, which in turn may increase the induced thermal stresses.

The following equation gives the maximum bridge surface temperature for a concrete deck with a thin asphaltic overlay.

$$T_m = T_a + 0.027L + 0.65(0.50T_r + 0.018L) \tag{3}$$

Zuk also presented the following simplified formula for predicting the maximum temperature differential between the top and bottom of a composite steel girder bridge.

$$\Delta T_m = T_m - T_a \lambda T_r \tag{4}$$

where λ is the lag factor and was assumed to vary from one-fourth in the summer to one-half in the winter; an average value of three-eighths was used for the middle Atlantic States.

A simple span composite steel girder bridge was instrumented in Charlottesville, Virginia, over the Hardware River. The maximum temperature differential throughout the depth of the bridge at an interior beam was 37° F and was observed when the slab was warm and the beam was cool. Exterior beams were observed to have maximum temperature differentials of 42° F on a sunny but cold afternoon. The presence of insulation for de-icing purposes had the effect of increasing the maximum temperature differentials by 25 percent.

Zuk also reported that German specifications require provisions for temperature stresses in composite members for a temperature differential of ± 27° F between the concrete and steel girders.

Hendry (38) has shown that a 2-in. asphalt topping is necessary before the insulating effect of surfacing begins to offset the increase in temperature due to the darker surface.

Antoni and Beal (4) investigated stresses induced in a concrete bridge pier due to shrinkage and temperature differentials. The pier was instrumented to verify AASHO 1964 requirements for the thermal coefficient, temperature variation, and shrinkage. Two days after pouring the pier cap (day 43), a 66° F temperature differential between the cap and the footing was found to occur. No differential existed eight days later. From then until the end of the test (day 300), the temperature differentials between the cap and footing ranged from a maximum of +11° F to a minimum of -8° F. The general conclusion was that the effective temperature differential causing bending moment in the frame was of the order of 20° ± 5° F.

Krishnamurthy (45) observed temperature effects on a three-span continuous test bridge of reinforced concrete for three days in 1969. The slab was 6-1/4-in. thick and the supporting concrete beams varied in depth from approximately 34 in. to 56 in. at the supports. The test data indicated that, prior to sunrise, the top surface of the bridge starts out at a lower temperature than the bottom and rises at a faster rate. At approximately 8:00 a.m. the two surfaces reached the same temperature. It was shown that the temperature continued to rise at the top surface until a maximum occurred between 1:00 and 3:00 p.m., with the maximum at the bottom surface occurring sometime later. During a normal day, the maximum temperature differential of 20° F occurred around 1:00 p.m.

Thermal Stresses and Movements

An unrestrained homogenous isotropic material subjected to a uniform temperature distribution experiences axial deformation. A linear temperature variation produces flexural deformation, whereas a nonlinear temperature distribution throughout the depth produces both axial and flexural deformations with associated induced stresses. Composite girders use shear connectors to prevent movements between the interface of the concrete deck and the steel girder. This restraint and the difference in the coefficients of expansion for the steel and concrete cause induced thermal stresses as the two materials expand and contract (10).

Internal thermal stresses are normally affected more by large temperature differentials than by the large overall seasonal temperature changes which produce the maximum expansion or contraction of the bridge (92).

Two options are available to design engineers for consideration of thermal movements and induced stresses in bridges. A sufficient number and size of expansion joints may be provided to accomodate the movements, or the structure may be designed so as to resist the movements and induced stresses. The first method is widely used for simple span and continuous bridges. However, due to construction and maintenance costs and because field observations show that expansion devices quite often do not function as intended, thus producing induced stresses, many designers are adopting the second method.

Thermal behavior of bridges was discussed in a report on the state of the art in 1972 (66). Thermal stresses, movements, joint sealants, expansion joints, bearings, and some current design specifications were reviewed.

Procedures for analysis of homogenious isotropic plates subjected to temperature differentials throughout the depth have been available for some time (16, 33, 77) but their application to bridges is very limited.

A rigorous theory of elasticity approach was used by Aleck (1) to determine the interface stresses in plates restrained on an interface edge and subjected to a uniform temperature. It was shown that interface stresses are limited to the plate ends and act over a distance equal to approximately one-half the plate depth. This concept is important for bridge structures where shear connectors act as restraints to the concrete slab. Typical interface forces and moments are shown in Appendix C.

Thermal stresses in a multiple layer beam of different materials and thickness were investigated using a minimum potential energy approach (30). The concept takes into account an arbitrary temperature distribution across the thickness and length.

In 1961, Zuk (90) presented a paper which accounted for thermal stresses and shrinkage effects in simple span composite bridges. An elastic theory was developed for stresses and strains due to various linear thermal gradients, considering the concrete deck as a homogenous material. In the analysis the beam and the slab were first separated to determine the stresses, strains, and curvatures of each, and then recombined in accordance with boundary and compatibility conditions for strains and curvature at the interface. Stresses and deflections were calculated for a 66 ft 3 in. simply supported composite steel girder bridge using a 25° F temperature differential between the steel girder and the concrete deck. Interface shears and moments concentrated at the ends were found to be as high as 410.63 kips and 769.38 in.-kips, respectfully. The corresponding steel girder stress was calculated to be 23.7 ksi. Shrinkage of the concrete was found to produce a compressive stress of 8.5 ksi in the steel girder.

In 1963, Liu and Zuk (47) extended Zuk's earlier work dealing with composite sections (90) to simply supported prestressed-concrete sections. More complex equations were developed and shears and moments were computed for four different cases of prestressed members. Interface shears and moments concentrated at the ends of the beam, based on an assumed parabolic 25° F temperature differential, were determined to be as high 30.43 kips and 122.88 in.-kips, respectfully. Transverse slab stresses were calculated to range from 800 psi compression to 1000 psi tension. Thus, if only the minimum reinforcement required by AASHO were used some cracking might still occur.

Zuk (92) in 1965 presented equations for temperature distribution in bridge slabs based on heat conduction relationships developed by Carslaw and Jaeger (22); and equations for calculation of thermal stresses and strains in simple span composite bridges subjected to a general temperature distribution. It was stated that even though a bridge may have adequate provision for over-all expansion and contraction, there could still exist large internal stresses due to non-uniformity in temperatures and material properties.

In 1965 Zuk (91, 93) also presented a simple emperical formula intended for use as a design check for simple span composite highway bridges. Based on field tests of various bridges with spans ranging from 47 ft 3 in. to 71 ft 5 in., the formula relates the thermal stresses in the bottom flange of the girder to the temperature difference between the top and bottom of the slab and the bridge depth as follows:

$$f_b = 2500T_s/h \tag{5}$$

where

f_b = thermal stress in bottom flange of girder (+ equals tension), in psi; T_s = temperature difference between the top and bottom of the slab (+ when top of slab is warmer than bottom), in °F; h = total depth of bridge (slab and girder), in inches. The author suggests that modifications may be needed for different geographic areas.

Berwanger (10, 12) developed equations based on an extension of Zuk's work which more fully account for the factors affecting thermal stresses in composite reinforced concrete-slab and steel-beam bridges. For example, the effect of slab reinforcement on the thermal coefficient was included. Consideration was also given to the thermal stresses in continuous composite girder bridges. Thermal stresses were calculated for a simple span and a continuous composite girder bridge. These stresses were found to be large enough to warrant further study.

Theoretical thermal stress and strain relationships for multiple span Semi-Integral end bent bridges, based on an extension of Zuk's and Berwanger's work, were developed during this study as shown in Appendix C.

In 1969, Wah and Kirskey (82) reported on an exhaustive study of thermal behavior which included a theoretical treatment, an experimental model, and field testing of a bridge superstructure. The equations developed were based on a fourier series expansion. One set of equations was related to the in-plane thermal expansion of beam-slab bridges, and another set was related to the flexure of beam-slab bridges. The equations developed for deflection of the slab and beam were similar to those for deflection due to lateral loading, except that the thermal loads were allowed to vary both vertically and horizontally. The general equations take into account any temperature variation. A computer program was developed for use in the solution.

A study of thermal behavior for a three-span continuous reinforced concrete test bridge subjected to daily temperature variations was reported by Krishnamurthy (45) in 1971. Using the surface temperature, transient temperature variations were calculated by means of a finite difference technique. Changes in reaction and moment due to temperature differentials were calculated by a matrix formulation (computer program) of the slope deflection equations. Comparisons were made of experimental and theoretical values. Even though unforeseen complications arose, the reaction changes for the three days of testing appeared to have satisfactory agreement with the theory.

Steward (74) studied the movements of 231 expansion joints in 80 bridges throughout California during a three-year period. From this study it became obvious that the use of expansion joints at the abutment had a significant effect on the total bridge movement. An increase in movement per unit length for bridges with expansion joints at the abutments over bridges without expansion joints at the abutments ranged from 31 percent in the valley area to 58 percent in the desert.

The apparent coefficient of expansion for concrete and steel were 5.3×10^{-6} in./in./° F and 6.5×10^{-6} in./in./° F, respectively. Box girder structures moved less during thermal changes than other concrete structures. It was further observed that the type of expansion bearing had very little effect on the joint movement caused by thermal changes. Steel girder bridges were found to have greater vehicle impact movements than did concrete structures. Uniformly spaced expansion joints on long structures did not necessarily move the same. There appeared to be no significant change in movements as a result of skews.

Black and Adams (15) observed expansion joint movements on five different bridges in England. The study was conducted to evaluate expansion joint and bearing design problems that have become evident with the increased use of bridges with span lengths of 100 to 200 ft. The report, based on four different bridges, showed that the five-day mean bridge temperature generally followed the five-day mean air temperature. Thus, the probable five-day movement could be estimated from known five-day mean air temperatures. The shortest free length reported was 112 ft for a reinforced concrete T-beam bridge and the longest was 2043 ft for a prestressed concrete box-section bridge. The magnitude of movements and effectiveness of various expansion and bearing devices were discussed.

Berks (7) found that thermal effects were the primary factors influencing movement in bridges. Annual movement cycles were 1/2 in. and 3/4 in. per 100 ft of span for concrete and steel bridges, respectively.

Zuk (94, 95) conducted end movement studies on four bridges for approximately a one-year period in the Virginia area. Three basic types of cyclic end motion were reported. The first, a short-duration transient motion due to vehicular traffic, decays rapidly and is of small magnitude producing movement in the order of a few thousandths of an inch. The second is a daily cycle due to solar effects in the order of a hundreths of an inch. The third is a yearly cycle caused by seasonal environmental effects and in the order of tenths of an inch. These three effects could be superimposed. The results, based on measurements of these movements taken at the moveable end of the bridge and ambient air temperature readings, indicated that there was an imprecise relationship between girder movement and air temperature. Also there appeared to be a lag time between air temperature and movements of from one to seven hours. End movements were found to be different at the top and the bottom of the girders, resulting in girder end rotations. It appeared that air temperature was the major parameter affecting length changes. Other factors noted which might affect behavior were:

1. Actual temperature inside the structure
2. Solar radiation
3. Volumetric changes caused by moisture and chemical action
4. Creep and shrinkage
5. Bearing restraints
6. Pier and abutment movements
7. Loading and changes in vibration

Influence lines are a valuable tool to the design engineer, and are commonly used for various conditions of load, settlement, and deflections. Many engineers are less familiar with influence lines for thermal effects. However, such influence lines could be of use in design if thermal effects are considered (14, 75).

Temperature differentials may lead to rather large bending stresses in long slender columns. A procedure for calculating theoretical values of curvature and stress for long slender columns, taking into account such factors as solar radiation, location and orientation of structure, and material properties, is given by Stephenson (72).

Shrinkage and Creep

Volume changes resulting from shrinkage and creep have been of concern to engineers for many years and are the subject of more than 1500 research papers. Engineers are faced with either the problem of what effect these volume changes may have upon the structure if they are neglected or the problem of which methods should be used if they are considered in the design. Roll reported (68) that differential shrinkage and creep between concrete slabs and non-shrinking steel beams in composite construction may cause excessive deflections, slab cracking, and overstressing of the concrete and steel. To emphasize the point, Roll described the second floor of an office building in New York which had been unoccupied for 16 months after completion of construction and yet experienced slab cracking and large deflections.

Shrinkage, a volume decrease of the concrete, occurs when the free water evaporates. When submerged in water, the concrete regains much of the volume lost due to shrinkage experienced earlier (85). Shrinkage occurs at a decreasing rate over several months; as opposed to creep which normally takes years to complete.

Hatt (37) in 1926 described many of the variables associated with volume changes in a study of road slabs. The variables discussed were shrinkage and expansion, temperature changes, and plastic flow under load. The factors then thought to affect shrinkage and expansion were the following:

1. Composition of cement
2. Aggregate (fineness and quality)
3. Wetness of mix and proportioning
4. Size of specimens and forms (whether absorptive or not)
5. Exposure (humidity and temperature)
6. Age of specimens and duration of test
7. Time base for measurement (before or after initial set)

In 1958, Ross (70) evaluated three methods of predicting creep under variable stress conditions. The rate of creep method, effective modulus method, and the superposition method were discussed. It was found that the effective modulus method could be used with confidence for members under a gradual decline of concrete stress, such as a reinforced concrete column. However, under variable or abrupt changes of stress the effective modulus method would probably give serious errors, and one of the other methods should be used.

Frudenthal and Roll (32) in 1958 reported the results of studies made on concrete cylinders 3 in. and 4 in. in diameter and 10 in. high under sustained stresses of 15 to 65 percent of the 28-day compressive strength. The response of various concrete mixes, creep-recovery, and the response to a history of loading-unloading cycles were studied. A mathematical model was developed to predict creep and creep-recovery of various concretes subjected to any arbitrary stress sequence. The temperature and relative humidity were under controlled conditions of 70 ° F and 60 percent, respectively, during all tests. The test results indicated that creep was a linear function of stress for a stress up to approximately 20 to 26 percent of ultimate, after which a nonlinear behavior occured. Based on a small specimen study, creep recovery was found to be complete within a few days after stress removal. The recovery appeared to be a linear function of the previous sustained stress, regardless of magnitude. Other investigators have reported linear relationships up to 75 percent of ultimate.

Birkeland (13) presented a procedure in 1960 for predicting deflections and stresses in simple-span composite sections due to shrinkage effects. The slab and beam were first imagined to be separated and shrinkage was allowed to occur. Equal tensile forces, of such magnitude that their resultant elongation was equal to the differential shrinkage, were applied at the cross-sectional centroid. The slab and beam were then superposed and a compressive force equal to the previously applied tensile force was assumed to act on the composite section at the slab centroid. Thus, stress, strain and deflection relationships were obtained. Equations for interface forces and moments acting at the ends of the section were also discussed.

Two beams were tested and the results discussed. Actual and theoretical deflections were shown to compare satisfactorily. However, a rather large value of differential shrinkage (450×10^{-6} in./in.) was assumed for the calculations. The author reported that the effective

Young's modulus for the slab may be approximately 1/40 that of steel.

Lyse (48) in 1960 reported the major factors affecting creep and shrinkage to be the amount of the cement paste, physical and rheological properties of the concrete, temperature and relative humidity of the environment during curing, and type and intensity of sustained stress in percentage of strength at time of loading. The effect of cement paste and sustained stress only were investigated. The general conclusions of the investigation were that shrinkage of concrete is approximately proportional to paste content and that creep, under a sustained stress, is approximately proportional to paste content. The relative humidity of the environment has a definite effect on shrinkage and creep.

Zuk (90) in 1961 presented a procedure for predicting stresses and deflections of composite sections, in which some of the theoretical factors omitted by other investigations were included. The approach considers both thermal and shrinkage effects. Shrinkage was considered by introducing an equivalent temperature distribution. The concept as proposed by Zuk may be seen in more general terms in Appendix C.

In 1964, Branson (17) summarized two methods of predicting stresses, strains, and deflections caused by differential shrinkage in composite beams. One method, composite section method, was that proposed by Birkeland and others whereby a differential shrinkage force is applied to the slab alone and then to the composite section. Both forces are assumed to act at the slab centroid. The other method, separate section method, considers a differential shrinkage force applied at the interface of both the slab and beam, resulting in an eccentrically applied force acting on each. It was shown that, in general, the composite section method predicted higher stresses and deflections; sometimes twice as high. The separate section method was recommended.

Concrete structures are subject to creep and shrinkage with time (2, 56). The amount depends on such factors as mix constituents, member geometry, and atmospheric conditions. Creep also depends upon age of concrete at time of loading, applied stress and strength, and the state of applied stress. Methods have been developed for including the effects of creep and shrinkage in design and analysis of reinforced concrete members (62).

In order to predict ultimate creep the relevant factors and their respective magnitudes must be evaluated. Neville and Staunton (57) in 1965 proposed a method of estimating creep for various stress-strength ratios. A hyperbolic equation was used to determine a standard creep-time time curve (for constant stress and strength). As the concrete stress or strength varied, a correction factor was applied to the standard curve, based on the principle that creep is a linear function of the stress-strength ratio. A numerical procedure was also discussed in which the strength-time relationship was used. The method was verified by experimental data of the authors and other investigators.

In 1965, England (29) developed a solid model, which simulated the heterogeneous nature of concrete, to predict creep and shrinkage strains for any mix proportions and any aggregate. The model was a two-phase material consisting of unit cubes of aggregate surrounded by a cement or mortar paste. It was concluded that the model could be used successfully to predict time-dependent strains of many mix proportions. and with a minimum of experimental data.

Branson and Metz (18) in 1965 published a comprehensive study of instantaneous and time-dependent deflections of simple and continuous reinforced concrete beams. Included were an evaluation of methods of estimating these deflections; the results of an investigation using 4 by 5-in. by 9-ft simply supported and continuous reinforced concrete beams with 9-ft spans; and proposed methods of estimating deflections. The interrelated effects of cracking, shrinkage warping, creep, tensile and compressive steel percentage, continuity and moment redistribution in indeterminate beams were discussed. The proposed methods of predicting deflections were found to agree satisfactorily with the experimental data obtained.

In 1966, Evans and Kong (31) reported the effects of creep on reinforced and prestressed concrete and presented a simple approach for creep prediction based on the major parameters affecting creep as determined by past research. The effects of stress, strength, and duration of loading; age and maturity of concrete at first loading; relative humidity; temperature; cement-paste content; water-cement ratio; aggregate-cement ratio; type and fineness of cement; character, size, grading, and shape of aggregates; and vibration and revibration of the concrete, and their relative magnitudes are discussed.

In 1966, Hanson and Mattock (36) reported studies of the influence of the size and shape of member on shrinkage and creep based on 4 years of test data. Concrete cylinders ranging from 4 to 24 in. in diameter and I-shaped members with depths from 11-1/2 in. to 46 in. were tested at 70° F and 50 percent relative humidity. Measurements were also made on sealed 6-in. diameter cylinders stored in a fog room. Two aggregates, Elgin sand and gravel and a porous sandstone, were used in the tests.

The general conclusions of the study, based on Elgin gravel, were that creep and shrinkage are dependent on the size and shape of members at all ages and that both creep and shrinkage decrease as member size is increased. There was a satisfactory correlation, for practical design purposes, between creep and shrinkage versus the volume-surface ratio. The member geometry appeared to affect creep only for the first 3 months of testing. A similar behavior appeared to occur for the porous sandstone aggregate.

In 1968, Bazant (6) reported that cyclic creep, a nonlinear phenomena, is irreversible and causes an accumulation of creep with the number of cycles of load, N. After approximately 10^6 cycles the creep will approximate the creep deformation which would occur after several years under sustained stress. Bazant outlined the effects of cyclic creep and showed that they can become significant especially for slender prestressed concrete bridges. Cyclic creep may be considered as an accelerated creep.

In 1968, Wallo and Kesler (84) presented a comprehensive review of the literature on creep and shrinkage of concrete and listed 166 references. Procedures for prediction of creep were developed based upon the research reviewed.

Equations for predicting creep and shrinkage based on an accumulated summary of past research data were proposed in 1970 (3, 19). The following equations may be used for predicting creep and shrinkage:

<u>Shrinkage</u>

$$\varepsilon_{sh} = (\varepsilon_{sh})_u S_t S_H S_T S_r S_c \qquad (6)$$

where

ε_{sh} is the final shrinkage strain, in./in.

The suggested average value for ultimate shrinkage strain, $(\varepsilon_{sh})_u$, for moist cured concrete at 40 percent ambient relative humidity is given by (3,50)

$$(\varepsilon_{sh})_u = 800 \times 10^{-6} \text{ in./in.} \tag{7}$$

If the concrete under these conditions is to be placed in parts, the basic ultimate shrinkage may be taken as one half the above value resulting in

$$(\varepsilon_{sh})_u = 400 \times 10^{-6} \text{ in./in.} \tag{8}$$

A moist cured concrete with a 4-in. slump or less, 40 percent ambient relative humidity, and a minimum member thickness of 6-in. or less may have a shrinkage at any time after the age of 7 days as determined by the following equation

$$(\varepsilon_{sh})_t = \frac{t}{35 + t} (\varepsilon_{sh})_u \tag{9}$$

where

$(\varepsilon_{sh})_u$ = ultimate shrinkage strain

t = time in days after initial shrinkage is considered.

To estimate the differential shrinkage in composite beams it is suggested that a correction factor of 1.20 be used which is based on the shrinkage of moist cured concrete from day one (3).

Thus

$$S_t = 1.20 \tag{10}$$

<u>Humidity</u> The equation for shrinkage may be corrected for humidity by using the following equations recommended by ACI Committee 209.

$$S_H = 1.40 - 0.010H \qquad 40\% \le H \le 80\% \qquad (11\text{-a})$$

$$S_H = 3.0 - 0.03H \qquad 80\% \le H \le 100\% \qquad (11\text{-b})$$

where

S_H = correction factor for humidity

H = relative humidity in percent

For example, when $H \le$ 40, S_H = 1.0

H	S_H
40	1.0
50	0.9
60	0.8
70	0.7
80	0.6
90	0.3
100	0.0

Member Thickness   It has been established that member size may effect shrinkage depending on size and volume-surface ratios. The following equation, as proposed by Branson and Christiason (19) establishes correction factors for member thickness.

$$S_T = 1.23 - 0.038T \qquad \text{for} \le 1 \text{ yr of drying} \qquad (12\text{-a})$$

$$S_T = 1.17 - 0.029T \qquad \text{for the ultimate value} \qquad (12\text{-b})$$

where

S_T = correction factor for member thickness

T = minimum thickness, in.

Branson reported that for most design purposes this effect may be neglected for members up to 8 or 9 in. in thickness but that for thicker members the volume-surface ratio should be taken into account. The thickness correction factor may be offset by the effect of slump greater than 4 in. and air entrainment greater than 6 percent.

Reinforcing   Martin (50, p. 282) reported that the CEB (41) has established the following expression for the determination of the correction factor for effects of reinforcing.

$$S_r = \frac{1}{1 + np} \tag{13}$$

where

S_r = correction factor for effect of reinforcement

n = 20 to 25, say use 20

p = percent of reinforcement, expressed in decimal form

using $n = 20$,

$$S_r = \frac{1}{1 + 20p} \tag{14}$$

Concrete Mix The concrete mix correction factor depends on water-cement ratio, aggregate, cement paste, and additives. Taking these factors into account, Branson and Christiason (19) recommended the following for correction factors;

Slump

$$f_s = 0.89 + 0.041S \tag{15}$$

where

f_s = correction factor for slump

S = slump, in.

This factor may normally be neglected except for high slumps. Thus, f_s = 0.97 for a slump of 2 in.; 1.0 for 2.7 in.; 1.01 for 3 in.; 1.05 for 4 in.; and 1.09 for 5 in.

Cement Content

$$f_c = 0.75 + 0.034N \tag{16}$$

where

f_c = correction factor for cement content

N = number of 94 lb sacks of cement per cu yd of concrete.

No correction factor is required for 5 to 8 sacks of cement per cu yd.

Percent Fines

$$f_F = 0.30 + 0.014F \qquad \text{for } F \leq 50 \text{ percent} \qquad (17\text{-a})$$

$$= 0.90 + 0.002F \qquad \text{for } F \geq 50 \text{ percent} \qquad (17\text{-b})$$

where

f_F = correction factor for fine aggregate

F = percent of fine aggregate by weight.

The correction factor f_F is 0.86 for 40 percent; 1.0 for 50 percent; and 1.04 for 70 percent.

f_F may be considered negligible.

Air Content The following equation is recommended to correct for the effects of air entrainment in concrete:

$$f_A = 0.95 + 0.008A \qquad (18)$$

where

f_A = correction factor for percent air

A = air content, percent

A = 0.98 for 4 percent; 1.00 for 6 percent; and 1.03 for 10 percent air.

f_A may be considered negligible for most cases.

The correction factor for the concrete mix is obtained by combining the above effects. Thus,

$$S_c = f_s f_c f_F f_A \qquad (19)$$

For average design conditions, these affects may be neglected and $S_c = 1.0$

Utilizing all of the above factors affecting shrinkage it can be shown that, assuming a concrete thickness of 6 in., the equation under average conditions becomes

$$\varepsilon_{sh} = (800 \times 10^{-6}) S_t S_H S_r \qquad (20\text{-a})$$

or if the concrete is placed in parts

$$\varepsilon_{sh} = (400 \times 10^{-6}) S_t S_H S_r \tag{20-b}$$

Martin (50) reported the average relative humidity in Missouri for Kansas City, St. Louis, and Springfield to be 65, 68 and 70 percent, respectfully.

For an average relative humidity of 68 percent and 2 percent reinforcing steel, the following values for shrinkage in composite members are obtained

$$\varepsilon_{sh} = 490 \times 10^{-6} \text{ in./in.} \tag{20-c}$$

or if the concrete is placed in parts

$$\varepsilon_{sh} = 245 \times 10^{-6} \text{ in./in.} \tag{20-d}$$

which compares within 22.5% of the strain, 0.0002 in./in. proposed by AASHO.

<u>Creep</u>

$$C = (C)_u C_{LA} C_H C_T C_r C_c \tag{21}$$

where C is the final creep coefficient, defined as the ratio of creep strain to initial strain.

The suggested average value for the creep coefficient at any time and the ultimate creep for a slump of 4 in. or less, 40 percent ambient relative humidity, a minimum member thickness of 6 in. or less, and a loading age of 7 days is (3):

$$(C)_t = \frac{t^{0.6}}{10 + t^{0.6}}(C)_u \tag{22}$$

where

$(C)_t$ = creep coefficient at any time

t = time, days

$(C)_u$ = ultimate creep coefficient, 2.35 for H = 40 percent.

Loading Age For loading ages later than 7 days for moist cured concrete, ACI Committee 209 recommends the following correction factor:

$$C_{LA} = 1.25t_{LA}^{-0.118} \tag{23}$$

where

C_{LA} = correction factor for loading age, for moist cured concrete

t_{LA} = loading age, days

For example, when

t_{LA}	C_{LA}
10	0.95
20	0.87
30	0.83
60	0.77
90	0.74

Humidity ACI Committee 209 recommended the following relationship to correct for humidity effects on creep.

$$C_H = 1.27 - 0.0067H \qquad H \geq 40 \text{ percent} \tag{24}$$

where

C_H = correction factor for humidity

H = humidity, percent

For example, when

H	C_H
40	1.0
50	0.94
60	0.87
70	0.80
80	0.73
90	0.67
100	0.60

Member Thickness Branson and Christiason (19) proposed the following to account for member thickness.

$$C_T = 1.14 - 0.023T \qquad \text{for} \leq 1 \text{ yr loading} \tag{25-a}$$

$$1.10 - 0.017T \qquad \text{for ultimate value} \tag{25-b}$$

where

C_T = correction factor for member thickness

T = the minimum concrete thickness, in.

Reinforcing ACI Committee 209 recommends the following correction factor for taking into account the presence of compression steel in reinforced members and the resulting movement of the neutral axis. The approximate effect of progressive cracking under creep loading and repeated load cycles is also included in the correction factor.

$$C_r = 0.85 - 0.45(A_s'/A_s) \qquad \text{but not less than } 0.40 \tag{26}$$

where

A_s' = compressive reinforcement, sq in.

A_s = tensile reinforcement, sq in.

Concrete Mix The concrete mix has been shown to affect the creep properties of concrete. The following correction factors were proposed by Branson and Christiason (19) to account for concrete mix.

Slump

$$C_{fs} = 0.82 + 0.067S \tag{27}$$

where

C_{fs} = correction factor accounting for concrete slump

S = slump of the concrete, in.

It is reported that the effect of slump tends to be offset by member thickness. Thus, C_{fs} may normally be neglected, except for high slumps.

Cement Content

$$C_{fc} = 1.0 \tag{28}$$

Percent Fines

$$C_{fF} = 0.88 + 0.0024F \tag{29}$$

where

C_{fF} = correction factor for fine aggregate

F = percent of fine aggregate by weight

C_{fF} can normally be approximated as unity and thus, neglected.

Air Content

$$C_{fA} = 1.0 \qquad \text{for } A \leq 6 \text{ percent} \qquad (30\text{-a})$$

$$= 0.46 + .090A \qquad \text{for } A > 6 \text{ percent} \qquad (30\text{-b})$$

where

C_{fA} = correction factor for air content

A = percent air

The correction factor is 1.0 for up to 6 percent; 1.09 for 7 percent; and 1.17 for 8 percent air content. The correction factor may be affected by member thickness and can normally be neglected for an air content of less than 7 percent.

Thus, the correction factor for concrete mix becomes

$$C_c = C_{fs} C_{fc} C_{fF} C_{fA} \qquad (31)$$

For average conditions, and using a relative humidity of 68 percent as for the shrinkage expression, the equation for the creep coefficient becomes

$$C = (C)_u C_H \qquad (32\text{-a})$$

$$C = 2.35\ (.81) = 1.90 = \text{creep strain/initial strain} \qquad (32\text{-b})$$

Concrete Growth

Concrete growth may be caused by either chemical or physical phenomena. Growth due to chemical causes such as high alkali cements and deleterious aggregates has been the subject of extensive field and laboratory study for years. David and Meyerhof (24) reported that stresses in composite structures caused by concrete growth are comparable to those produced by differences in the thermal coefficients of steel and concrete at low temperatures. Expansive movements due to concrete growth may become serious to the point of structural failure, and AASHO (71) warns against the use of composite construction when expansive concretes are used. In Massilon, Ohio, an eleven-span 748-ft highway bridge built in 1948 was closed in December, 1964, due to excessive movements of the concrete deck thought to be caused by cement-aggregate reaction (35). The bridge has a concrete-filled-deck with steel stringers and concrete piers and abutments. One 210-ft span was reported to have moved in excess of 3 inches. The bridge also experienced permanent tilting of some of the expansion rockers, a cracked abutment backwall, and cracking of the deck. Some of the deck was sheared away from the supporting beams.

In 1963, Oleson (60) reported a comprehensive continuing field and laboratory study, initiated in 1947, on abnormal cracking in highway structures in Georgia and Alabama. A total of 294 bridges with known combinations of materials were studied; 142 in Georgia, and 152 in Alabama. The results indicated that abnormal cracking was associated with cements having an alkali content in excess of 0.6 percent. A continuation of the study showed that, regardless of aggregate, structures in Georgia and Alabama with less than 0.6 percent alkali content did not experience any evidence of alkali-aggregate reaction. However, other investigators have indicated that the 0.6 percent value may not necessarily be a sufficiently safe criterion in all cases.

Because of the excessive movements associated with expansive cement-aggregate reactions, many state highway departments have discontinued the use of high alkali cements and deleterious aggregates.

Physical causes of concrete growth such as freezing-thawing, wetting-drying, and cooling-heating must also be considered by the design engineer. Chow (23) reported in a study of thermal coefficients and moisture creep that concrete increases in length with each alternate wetting-drying cycle resulting in a residual expansion. Old specimens which had been exposed to conditions of freezing and thawing had a tendency to grow more than newly prepared specimens. Concrete with limestone aggregates experienced larger growth than concrete with gravel aggregates. The water-cement ratio, proportions of the mix, size of the aggregate, and size and shape of the specimen also affect growth. The reinforced specimen experienced smaller growth than the plain concrete specimens which indicates that additional induced stresses may occur in reinforced concrete structures.

Concrete growth due to moisture does not require visible wetting; it is also influenced by air humidity. Davis (25) reported that for throughly dried concrete bars, air of relative high humidity is nearly as effective as immersion in causing expansion. Matsumoto (52) reported that concrete expands and contracts with absorption and drying of the specimen, and that richer mixes absorb more water and thus expand more than leaner ones. The rate of absorption due to immersion was found to be less for older specimens. However, the total absorption did not change appreciably with age.

The amount of expansion was small until an absorption of 2 percent was reached. Above 2 percent the rate of expansion was proportional to absorption until a sudden decrease in absorption occured after which additional absorption gave large increases in expansion.

Lea and Davey (46) reported that drying shrinkage and reversible wetting movements are of the same order of magnitude as thermal movements; approximately 1/2 in. in 100 ft. Mercer (53) referred to a paper presented by Professor A. H. White in 1915 which reported that a small bar cut from a sidewalk after 20 years of service elongated 0.175 percent by successive immersions at room temperature, and that successive long immersions with intermediate dry periods caused progressive expansions much greater than movements due to temperature. Mercer also warned against consideration of moisture change as an equivalent thermal change, since thermal influence can be vastly different from the effects due to permeability and moisture.

Other Effects

Thermal effects, creep, shrinkage and concrete growth are generally considered to be the primary factors influencing bridge behavior. However, other factors such as braking forces, substructure flexibility, skew, and soil stability may also cause significant movements and induced stresses.

Braking forces and vehicular vibratory movements are short term loading effects as opposed to the movements and induced stresses caused by thermal effects, creep, shrinkage, concrete growth and soil stability.

Soil stability continues to be the source of many problems. Backfill settlements causing rough rides, traffic noise, and structure movement are some of the resulting effects. Peck and Ireland (63) reported that the cause for these effects may be contributed to improperly placed and compacted backfills. Jones (42) studied the approaches of a number of bridges in California. The investigation revealed that more approach patching was required for closed-abutment bridges than for spill-through structures. This was believed to be the result of better compaction of the approach fill for open-end structures by construction equipment and consolidation of the underlying material by the weight of the approach fill, usually completed before the bridges are built. There was little difference in the amount of required approach patching needed for open-end structures on spread footings as compared with those on piles. Significant end movements can occur as a result of subsurface conditions and this effect should be treated with caution by the utilization of positive design procedures. Excessive abutment settlements and movements were reported on three bridges built in Canada and some movement and settlement occurred on four others (73). The bridges were designed with abutments supported on point bearing H-piles through a soft compressible clay layer. The abutments, contrary to usual movements, moved outward from the bridge, away from the batter, thus causing excessive rocker tilting. These movements were believed to be due to the settlement of the backfill, resulting in a dish shape at the top of the embankment with a consquent lose of horizontal restraint and development of a net outward force. The abutment then tilted toward the depression, causing movement of the structure. Normal maintenance practice would only complicate the problem with the addition of more material (weight) to fill the settled approach. The following recommendations for avoiding a recurrence of these problems in future structures were given: (1) staging the approach embankment and bridge construction (most strongly recommended); (2) battering piles in the opposite direction; or (3) providing the abutment with parallel wingwalls in conjunction with the batter away from the bridge, which would provide a rigid frame supported by piles battered in both directions.

Observed abutment movements for an abutment on Interstate Route I-80 in New Jersey were reported (43, 59). The investigation was conducted to clarify the factors which cause some abutments, supported on piles driven through plastic clays, to tilt outward towards their backfill. These movements can result in extensive maintenance and/or repairs.

It was suggested that preloading should be completed approximately one year or more in advance of pile driving. Only piles with adequate resistance to bending should be used under bridge abutments in danger of backward tilting. It was further stated that the piles should be of a non-displacement type; for example, steel H-piles, or open end pipe piles kept open by washing during driving. Pile layout and direction of their batter should be governed by three-dimensional considerations of soil deformations at all levels and of their interaction with likely abutment displacements. The "spill-through" type of abutment with no wingwalls can be economically and easily adapted to meet these general requirements.

Bridge Instrumentation

Time dependent effects as related to movements have been of concern to bridge engineers for years. The need for a rational approach, taking these factors into account, becomes increasingly important as bridge structures become longer and more flexible. Additional guidelines are needed for consideration of such items as: economical and feasible maximum bridge lengths between positive expansion devices; the effect of skew; the most effective expansive and bearing device for a particular design; and the relative magnitude of movements and/or induced stresses. In order to accomplish these guidelines further research is needed.

In a study of bridge behavior over long periods of time, there is no substitute for instrumentation and testing of bridges under actual use. However, field testing, unlike laboratory testing is complex and requires special instrumentation techniques and care to avoid problems associated with environmental exposure. The success and/or failure of an investigation will depend to a large extent on the procedures developed and the instrumentation used to account for and isolate the variables and their respective magnitudes. Thus, the objectives of this section of the literature review were:

1. To review related investigations, the parameters studied, and the relative successes and/or failures.
2. To establish causes for problems encountered in prior studies that they might be avoided in future studies.
3. To relate the success of past research with the need for future study and the feasibility of development of a design criteria for bridges with Semi-Integral end bents.

Only those investigations related to this study are discussed. Although instrumentation procedures are emphasized, in some cases duplication of prior discussion is unavoidable, and in other cases instrumentation techniques were not reported.

Zuk (94, 95) observed girder end rotations, movements, air temperatures, and time of day on four bridges in Virginia. Data was recorded for approximately a year by the use of a movie camera positioned to take pictures of dial readings, thermometer readings and time. The general conclusions, based on this rather inexpensive instrumentation, indicated that there was an imprecise relationship between air temperature and end movement. The objective of the study was not to separate and explain the variables affecting movements but to simply measure the magnitude of movements. Thus, in order to isolate variables associated with end movements additional instrumentation would be required.

Steward (74) observed 231 expansion joints in 80 bridges in California under various climatic conditions over a three year period. The study was to determine the influence of weather at time of construction on joint width requirements. Measurements were taken by the use of an inexpensive scribe mounted on the railing. Impact movements, longitudinal movements, and end rotations, were either measured or estimated for the structures tested. Temperatures used to calculate apparent coefficients of expansion were obtained from local Government gage stations. The effect of daily average temperatures at time of construction and concrete shrinkage effects were recommended as subjects for further study.

As reported earlier in this report, bridge movements have been observed in England and compared to meterological data (15, 28). However, the exact nature in which the measurements were taken and the data recorded was not clear.

The Hammersmith Flyover, a bridge in England, is a precast prestressed-concrete continuous four-lane viaduct, 2043-ft long between abutments, and with 16 spans mostly of 140 ft (65). The main structural element of the flyover is a continuous hollow-spine beam 26-ft wide and varying in depth from 6 ft 6 in. at midspan to 9 ft at the supports. The spine beam is supported by concrete columns with roller bearings at the base to accomodate a 10-in. range of movement. The longitudinal superstructure movement is provided for by one expansion joint located approximately 820 ft from one end. Site measurements were taken to evaluate prestressing strand behavior, superstructure movement, and roller bearing behavior under service conditions (87).

Values for bearing friction were calculated from the jacking force required to reposition the bearings (required by a change in the planned construction sequence). Since the section weighed almost 10 kips, it was necessary to apply the jacking force in two stages. The jacking operation was performed by three Freyssi flat jacks, each 13-3/4 in. in diameter, placed vertically in the narrow gap between the end anchor block and the plinth within the abutment. The bearing friction was found to vary with movement and the stages of jacking. The first jacking operation resulted in a peak value of bearing friction of approximately 1.25 percent, whereas the second jacking operation produced a peak value of just over 2.5 percent.

A bearing friction of 1.3 percent under ordinary daily temperature movements was determined from data obtained by using vibrating wire strain gages cast into one of the columns.

Air temperatures inside the spine-beam were used as an indicator of concrete temperatures. The temperature was measured by means of a sensitive thermometer graduated to 0.1° F. Movements at one of the pier columns closely followed the inside temperature changes without a time lag. A linear relationship of the interior air temperature and the corresponding longitudinal movements at a pier column bearing were shown in a plot of three days' readings.

The actual air temperature was found to vary from 20° F to 92° F over a 7 month period whereas the internal air temperature was found to vary from 31.8° F to 81.5° F, respectively. Based on a 50° F temperature range in the structure, subtracting the effects of creep and shrinkage, and neglecting humidity, an effective thermal coefficient of 6.7×10^{-6} /° F was determined. The instrumentation used in measuring these movements was not reported.

Future research planned for long term testing includes the efficiency of a road heating system; temperature distributions in the cross section; and longitudinal movements.

Three types of investigations were conducted to evaluate the temperature effects in beam-slab bridges by Wah and Kirksey (82); theoretical, experimental model, and field tests on a bridge instrumented during construction (De Zavala Overpass, San Antonio, Texas). An experimental model 8-ft by 6-ft by 7-in. thick was constructed to simulate a mathematical model developed during the study and the test bridge slab. However, due to its stiffness, the deflections were found to be too small, even under large thermal gradients. The initial model was later abandonded and replaced with an 8-ft by 6-ft by 3-in. slab with two layers of welded-wire fabric to simulate top and bottom reinforcement. The slab was placed on a single steel beam (with shear connectors) so as to form a simulated simple-span composite girder section. Temperature effects were introduced by the use of heat banks of infrared bulbs. Deflections, strains, and temperature distributions were measured by Ames dial gages; foil gages, applied to the reinforcing and to the steel beam, and concrete embedment gages; and thermocouples, respectively. Strains were recorded by the use of a 100 channel recorder. Creep effects were found to have a significant influence on the behavior of the heated slab. The slab arched up when the heat was applied due to the top heating up faster than the bottom. However, as time progressed the upward deflection gradually decreased in magnitude and in some cases actually reversed in direction, while the temperature difference between the top and the bottom of the slab remained nearly the same. Creep deflections tended to offset thermal deflections when the top of the slab was warmer than the bottom. The thermal stresses as calculated from the measured strains were quite high and erratic.

The De Zavala Overpass is a 46-ft simply supported bridge with 14 pan-type evenly spaced reinforced concrete beams acting as supporting girders. The roadway width is 38 ft 2 in. and the skew is zero. During construction, 390 thermocouples and 14 embedment gages were installed. One bank of 13 thermocouples was lost during concreting, leaving a total of 377 active thermocouples. Strains, bridge temperatures and ambient air temperatures were recorded periodically for two days in August and one 22-hour period in December, 1967. Deflections were also measured during this time by means of a transit. There was a significant difference between the measured deflection and calculated values based on the mathematical model. This reportedly was due primarily to the deviation between the bridge and the mathematical model and the presence of different boundary conditions. Stresses as calculated from measured strains were found to be as high as 1500 psi in the concrete. The creep effects would not be as significant for the bridge as for the model due to the lower temperature differentials experienced. It was reported that due to the rapid change of atmospheric conditions hand recording of temperatures may lead to erroneous and erratic results. Thus, adequate instrumentation and sensors in conjunction with automatic recording devices becomes extremely important.

Temperature distributions and movements of the Beachley Viaduct/Wye bridge were reported by Capps and Emerson in 1968 (21). Data were taken to evaluate the temperature range and movement experienced by the structure during the period January, 1966, to August, 1967. The structure is a continuous hollow steel box section 3784 ft long. In cross section the box is approximately 10 ft deep at the center and tapers to form a trapezoidal section with cantilevered hollow sections on each side giving a total superstructure width at the top of approximately 100 ft 6 in. The viaduct is supported at approximately 200-ft intervals on trestles which can rock to provide movement of the superstructure. The Wye bridge, which has the same cross section as the Beachley Viaduct, has a center span of approximately 770 ft supported on piers pinned at the base. The structure is fixed at the west end and at the Beachley anchorage of the Severn Bridge. The superstructure movements are provided by one expansion joint located approximately half way between the fixed ends.

Thermocouples were used to measure temperatures in the box section of the structure and the shade temperature of the surrounding air. A modified self-balancing potentiometric recorder was used to record both temperature and movement. The movement was measured by the use of a linear potentiometer at the expansion joint. The solar radiation was measured by means of a Kipp solarimeter equipped with a battery recorder. Since the bridge is orientated approximately south-east to north-west, thermocouples were placed on both the north and south sides of the box to evaluate whether there was a temperature difference across the width.

It was found that on the hottest day the top flange plate on the south side was 4° F warmer than on the north side, and the bottom flange plate on the south side was 2° F warmer than on the north side.

Temperatures and movements were observed for both the unsurfaced steel deck and the deck when surfaced with 1-1/2 in. of asphalt.

The success of the instrumentation was not reported; it can only be conjectured that any problems with the instrumentation or techniques used would have been reported. It was reported, however, that the coefficient of expansion for the steel was $6.1 \times 10^{-6}/°F$ as determined from the measured data and the length of the bridge.

Bridge piers, like any exposed structure element, are exposed to complex environmental effects and material behavior, combined with superimposed short term loadings. A concrete bridge pier was instrumented and observed for approximately 300 days for the purpose of evaluating temperature and shrinkage stresses (4). The pier observed is a three column pier with a cap beam 3-ft wide by 4-ft deep and 46-feet long supported by three columns 3-ft in diameter and 10 ft 8 in. in length and resting on a continuous footing supported by cast-in-place piles. The footing is 8-ft wide by 3-ft deep and 47 ft long. A number of measuring devices were located in the cap beam, footing and outside column to provide a duplication of data to aid in the isolation of variables associated with field investigations. Compressible spacers were located at the top and bottom of each column to break the bond and so that loading would be resisted only by the longitudinal reinforcement. Electric strain gages and thermocouples were provided for measuring reinforcing strains and internal and ambient air temperatures, respectively. Invar bars connected to linear motion potentiometers were placed in the footing and cap beam to record total length changes. Predrilled brass plugs, cemented to the concrete cap beam, and Whittemore strain gages were used to obtain concrete strain readings. Vertical movements were measured by means of a precise level. Gages, thermocouples and potentiometers were monitored by an automatic recorder.

Some unforseen difficulties were reported. Factors such as cold weather required the cement used for application of electric strain gages to be cured by heat lamps, and in some cases gages were necessarily applied prior to placing the reinforcement in the forms. Other factors such as electrical interference and circuitry problems were reported to have caused gage failures, erratic gage operation, recording difficulties, and unexplained data shifts. Some further problems stemmed from the construction sequence, environmental effects, change of personnel and the loss of the reference benchmark midway through the testing period.

Erratic Whittemore strain gage readings were thought to be caused by the influence of diurnal effects and changes in personnel taking the readings.

It was concluded that the instrumentation system used did not give a satisfactory long term performance.

A 26-ft wide by approximately 66-ft long composite girder concrete slab bridge with a 12° skew was instrumented in Virginia (92). The superstructure was composed of a 7-1/2-in. concrete deck supported on four 36-in. wide-flange beams. The structure was 2 years old when testing was initiated. The objective was to study the thermal behavior with and without insulation. Consquently, approximately half way through the test period the middle 20-ft of the bridge was sprayed with urethane foam to a thickness of one inch. Strains, temperatures, solar radiation, moisture, and vertical movements were recorded. Twenty-two thermocouples were used to measure the temperature distribution throughout the depth of the structure. Two ambient air temperature gages were also installed. Thermocouples were monitored at hourly intervals by an automatic recorder. Whittemore strain gages were used to insure long term stability. Concrete moisture was measured with a nuclear moisture probe using a radium-beryllium source. All readings were taken under dead-load constant-moisture conditions in order to isolate the induced strains. Unforeseen complications in strain and deformation measurements arose due to interface slip between the concrete and steel. There were also axial end restraints induced as a result of bridge expansion to such an extent that several of the 1-in. anchor bolts were bent and a 12-in. concrete backwall cracked from pressure transmitted through a 1-in. expansion filler.

After application of the insulation, only temperature measurements were observed because of associated instrumentation problems.

The primary emphasis of the publication was a presentation of theoretical equations to predict bridge temperatures and stresses due to atmospheric conditions rather than a review of the field investigation. Recommendations relative to instrumentation techniques were not made.

An investigation of a three-span continuous reinforced concrete bridge built near Auburn University for the sole purpose of testing was reported by Hudson in 1967 (40). A half-width bridge was constructed according to Alabama standards. The spans are 44-54-44 ft resulting in a bridge length of 132 ft between end bearings. The concrete deck is 6-1/4 in. thick and supported by haunched reinforced concrete girders. Reactions, deflections, surface strains, steel strains, longitudinal movements, and internal and ambient temperatures were observed. The results of both dead and live load tests were reported.

The objective was to study the bridge behavior over a period of time under dead-load conditions, and to evaluate the effects of factors such as creep, shrinkage, and temperature. Bridge behavior due to live load effects were also to be investigated.

Instrumentation for the dead-load study utilized a 100-kip load cell for reaction studies; dial gages; etched foil resistance strain gages and

Whittemore gages for strain measurements; and three thermistors for internal temperature measurements.

Approximately a year after completion of the bridge, dial gages were mounted at the bridge ends to measure expansion and contraction. Lucite boxes were used to protect the gages. Internal temperature readings were read at two-hour intervals the first day, twice daily until the readings stabilized, and then daily.

It was concluded that thermal changes exerted the greatest influence; however, no direct relationship between force changes and temperature was established. It was observed that the sum of the four girder reactions (half of the bridge) was subject to a continuous variation, increasing with rising temperature, and vice versa.

The live load study utilized foil gages; special aluminum cantilever beams for deflection measurements; and a 100-kip load cell for reaction measurements. Measurements were recorded by three 18 channel oscillographs, resulting in a total of 54 channels.

It was concluded during the live load study that the instrumentation, while adequate, left much to be desired. It was reported that both more vertical and horizontal gages would have been desirable. Gage malfunctioning was very low; only one failed to perform out of a total of forty. It was suggested that more cross-sectional strain readings should have been taken. A tertiary objective of the experimental program was to develop a mathematical approach to the problem of stress distribution for the type of bridge investigated. It was stated that from the data gathered the objective was unfeasible.

Response to daily temperature variations was investigated by Krishnamurthy (45) during 1968 and 1969 on the same reinforced concrete test bridge used by Hudson (40). Measured effects were compared with theoretical considerations for the observed temperature variations.

Reactions, strains, temperatures, and temperature variations were observed over a three day period in 1969. Some of the instrumentation used by Hudson was also used in this investigation. Four 100-kip load cells were used to measure reaction changes. Like Hudson's study only one-half of the bridge (length) was instrumentated. Thirty-two thermocouples were used to measure temperature variations, and electrical resistance strain gages at 40 locations were used to measure strains.

It was reported that the observations began before sunrise and continued hourly until late afternoon, and that each set of readings took approximately 20 to 30 minutes. Complications arose in the reaction measurements when malfunctions occurred in one load cell and a jacking unit failed. The reactions were measured by jacking up the point in question, removing the bearing and inserting the load cell. It was reported that the reactions behaved erratically; however, strain readings were found to be consistent and stable.

The theoretical study utilized a finite difference technique to describe the transient temperature; moments and reaction changes due to temperature differentials were calculated by a matrix formulation of the slope deflection equations. A computer program was especially written for the study.

It was reported that:

> The accuracy of the results and the utility of their evaluation are limited by the relative paucity of measured data, and insufficient information on the mechanical and thermal properties of the concrete. However, the study demonstrates the feasibility of the analysis technique and the validity of the mathematical model used. Further more exhaustive research is necessary and will be fruitful.

It was also indicated that better load cells, or other superior devices for measuring reaction changes, would be essential and that the procedure used for reaction measurements would necessarily need to be changed to include precise monitoring devices. Precise measurements of thermal and mechanical properties would be necessary.

Field measurements on instrumented piles were taken to study the factors causing some abutments to tilt towards their backfill when supported on piles driven through plastic clays (43, 59). Measurements of and the results obtained for an overpass abutment on Interstate I-80 across the Lehigh and Hudson Railroad in northwest New Jersey were discussed. Generally, the embankment height was 30 ft, underlain with a medium compact silty sand, followed by approximately 44 ft of soft varved clay and then a very compact sand and gravel.

Measurements were taken on the steel H-piling with Carlson strain meters and slope indicators. Eight piles were fitted with pipes extending to within 5 ft of the tip to receive Wilson slope indicators. The pipe was cut at 1-ft intervals in order to maintain constant effective section properties. The slope indicators were installed for determination of deflections and bending moments along the pile length. Eight Carlson strain meters (four of which were on piles receiving the slope indicators) were attached just below the footing to determine axial loads and moments of fixation transmitted from the concrete footing. Two steel plates 30-in. by 48-in. by 1-in. thick, each supported by four Carlson stress meters, were installed on the rear vertical face of the concrete footing to measure soil pressures at that elevation. In addition, nine settlement platforms were installed to measure settlement, and lateral displacements were measured by means of a triangulation technique.

The Carlson stress meters were found to be fully reliable in every respect. The Carlson strain meter was also found to be consistent and apparently reliable. It was recommended that for future studies the instrumentation used be supplemented by stress-meter plates placed along the lower horizontal surface of the footing between the piles, and that strain or stress measuring devices be placed along the pile length.

The standard Wilson inclinometers gave reasonable results; however, it was considered essential that a miniature inclinometer (1 to 2-in. in diameter) be used for future studies.

It was further suggested that more accurate triangulation of abutment displacements would be required in order to obtain usable data. Seemingly, the monuments used were affected by vibration due to passing trains and other effects. Settlement measurement problems resulted from inaccurate reference bench mark data and imprecise markings on the pipes used for settlement measurement.

Computed theoretical deflections of a continuous five-span reinforced concrete box girder bridge in Missouri were compared with actual observed values and the designers estimate (61). The bridge has span lengths of 64-80-105-105-84 ft and a roadway width of 40 ft. The depth of the box is 5 ft 6 in. and the concrete deck is 7 in. thick. Deflections were measured by means of nine stainless steel inserts permanently installed in the main spans. The top of the inserts were cupped to accomodate a Philadelphia rod with a stainless steel ball attached to the base. A vernier graduated to 0.001 ft was attached to the rod. Elevations were measured with a dumpy level and referenced to a permanent benchmark.

It was found that elevations were significantly affected by temperature and/or humidity fluctuation and differential expansion and contraction of the pier columns. A total of eight observations were made over a period of 438 days following form and shoring removal.

It was concluded that levels are not recommended for measurement of time dependent deflections, due to the difficulty of separating temperature effects from possible instrumentation errors.

The acoustic or vibrating wire strain gage has long been accepted as a reliable gage for use in studying time dependent effects. The gages are used extensively in Europe, but only to a limited extent in this country.

An extensive amount of work in the development and use of these gages as well as other measuring devices for use in long term studies on bridges has been conducted by the Road Research Laboratory in England as reported by Tyler (78). Application techniques, relative accuracy, and the cost of measuring devices for both strain and stress are discussed and described in some detail. The devices discussed are: the acoustic gage (both buried and surface mounted), recording equipment, demountable gages, a creep rig, Glotzl (Gloetzl) pressure cells and photoelastic stress plugs.

The Glotzl (Gloetzl) pressure cells and photoelastic stress plugs are used for direct measurement of stress in the structure. Although the pressure cells are rather expensive, they were found to behave satisfactorily during testing of three major structures.

The creep rig has been used by the Laboratory for use in predicting stresses due to creep. Basically, a test specimen is subjected to the same environmental conditions as the prototype and loaded in the creep rig to establish a strain history identical to that of a gage located in the real structure. The jacking force in the creep rig is then an indication of stress.

Further discussion of the creep rig usage and testing procedures as applied to actual bridges is presented by Tyler (80). It was reported that the method is simple in application to uniaxial loaded structures but experimentally more difficult for biaxial or triaxial stress states.

Studies of strains in fresh concrete for two laboratory specimens and three full-scale site investigations using vibrating wire strain gages were reported by Tyler (79). The purpose of the study was to establish a zero reading from which gage readings can be taken to obtain meaningful estimates of creep and shrinkage. Strain changes were investigated for the different acoustic gages used by the Laboratory. Studies were made of the behavior and durability of unprotected gages and gages installed in briquettes for protection.

The briquetted gages generally showed greater expansion than the unbriquetted ones. It was reported that when the coefficients of expansion of the steel gage wire and the concrete are equal the only strains recorded are elastic strains, creep, and shrinkage. It was suggested that a zero strain reading should be used when the concrete has cooled to ambient temperature. It was found that contraction movement due to temperature in the first few weeks is far greater than that due to shrinkage.

A survey of instrumentation developed for studying the behavior of bridge structures was conducted preliminary to selection of instrumentation for a long-term field investigation of a four-span prestressed concrete bridge (67). The evaluation of short-time dynamic effects were not included in the study; however, the study is applicable to measurement of the effects of static live loads. The parameters to be measured were:

> (a) stress and strain in the prestressing steel, (b) strain in the concrete as indicated by internal and external gages, (c) temperature gradients across beam sections, (d) deflections of the structure, and (e) relative humidity in the concrete.

The criteria used for selection of the various instruments were that they should:

> (a) be sturdy in order that they would withstand common fabrication and handling procedures, (b) be stable over long periods of time, (c) be temperature compensated from 0° F to 135° F, (d) be waterproof, (e) have sufficient sensitivity, and (f) be as uncomplicated and inexpensive as possible.

Deflection measuring systems discussed included a taut wire technique, a truss system mounted below the bridge with either dial gages or electrical gages, a water level system, and surveying instruments. A precise level and a specially fabricated level rod were chosen for the investigation.

Devices discussed for measuring internal strains included electrical resistance strain gages, the Carlson strain meter, and internal vibrating wire gages. The Carlson strain meter was selected for the field-phase of the investigation and was reported to perform satisfactorily.

Included in the discussion of measurement of surface strains were the Whittemore and the Munich mechanical strain gages, electric resistance gages, and optical strain gages. The Whittemore strain gage was used for the field-phase of the investigation.

Temperature measuring devices reviewed included thermocouples, thermometers, and electrical resistance gages (such as the SR-4 gage and the Carlson strain meter) or semi-conductors. Temperatures found with the Carlson meter during the field-phase of the investigation were in good agreement with the thermocouple readings.

Four methods for measuring the relative humidity at internal points were discussed. However, it was noted that all of the devices seem to have serious drawbacks concerning temperature compensation, making their use in a field investigation difficult, and none were used in the field-phase of the project.

The report also discusses laboratory instrumentation tests conducted with a 6 by 12-in. by 10-ft reinforced concrete test beam and the conclusions and recommendations resulting from these tests.

As a by-product of a study of the literature on the use of electric resistance strain gages on concrete, a bibliography of 801 items covering the period 1940 to 1964 was compiled by Berwanger (9) and published in 1968.

An investigation entitled "Analysis of Integral Abutments" conducted by Shoukry and Sarsam is presently in progress at South Dakota State University. The project was visited in November, 1971, as a part of this study. At that time the effects of thermal movements on the integral abutment were being studied.

The test bridge is a full-scale single-span structure with an integral abutment incorporating steel H-piling. The superstructure is of composite design with a concrete deck and two stringers. The thermal forces are simulated by means of hydraulic jacks acting at one end of the bridge. Displacements, strains, and rotations at critical locations are measured using electric strain gages and theodolites. Earth pressures in the backfill are being measured with pressure cells located on the back side of the abutment.

SURVEY OF CURRENT DESIGN PRACTICE FOR BRIDGE SUPERSTRUCTURES SUPPORTED BY FLEXIBLE SUBSTRUCTURES

This phase of the study of design criteria for stresses induced by Semi-Integral end bents consisted of a survey by questionnaire of current design practice for superstructures supported by flexible substructures.

Objectives of the Survey

The primary objectives of the survey were:

1. To determine the present design criteria used for bridge superstructures supported by flexible substructures, and the extent of current usage of this type of construction.
2. To establish if there exists a rational design criteria which takes into account the effects of such factors as shrinkage, creep, temperature, substructure flexibility, etc.
3. To identify the factors considered by bridge engineers to be significant to bridge behavior.
4. To identify potentially significant parameters which might be indicated through problems encountered or by objections to usage.
5. To establish a maximum workable length between positive expansion devices, or in the case of restrained structures a practicable design length, based on past performance.
6. To establish whether or not there is a need for future research in the area of restrained structures, taking into account time dependent factors.
7. To determine the field behavior of bridges in service and under

construction with superstructures connected to flexible substructures.

8. To better establish the feasibility and value of further research in the area of restrained structures.

Description of the Survey and Questionnaire

The cover letter and questionnaire shown in Appendix A were sent to the 50 state highway departments and to 5 governmental agencies. Replies were received from 43 state highway departments and 3 governmental agencies, resulting in a total response of almost 84 percent and a response from state highway departments of 86 percent.

As explained in the cover letter, the study of design criteria for stresses induced by Semi-Integral abutments is limited to continuous composite steel structures. However, in order to compare similarities and differences of current design practice for Non-Integral, Semi-Integral, and Integral types of construction for both steel and concrete structures, the questionnaire encompassed design practice for the three types of construction.

The replies reflect a wide range of design practice and limitations--not readily tabulated by simple arithmetic summation or graphical representation. In some cases, interpretation of the response to one question is dependent upon continuity with the response to prior questions. Thus, to provide a continuity of individual respondents and to aid in comparison of design practice among respondents, the states were grouped into six geographical areas and each of the respondents was assigned an identifying two digit number. The first digit represents the geographical area in which the respondent is located, and the second digit represents the particular state.

The six geographical areas, as shown in Fig. 1, are as follows:

Area 1 - Lower Middle States

Area 2 - Southeastern States

Area 3 - Northeastern States

Area 4 - Upper Midwestern States

Area 5 - Northwestern States

Area 6 - Southwestern States

The replies are tabulated and listed sequentially by questions in Appendix A. The continuity of replies by a given respondent is identifiable by the assigned respondent number. Questionnaires are naturally subject to differences in interpretation as to meaning and intent. Thus, a few replies, e.g., some replies to items 3(k) and (1), seemingly may not relate to their respective question. It will be noted, however, that, in an effort to avoid a compounding of misinterpretations, editorial license of revision and interpretation of the replies of the respondents has been reserved as the reader's prerogative.

Some respondents included additional comments or explanatory remarks of interest to design engineers, and a few answers to specific questions were too detailed for tabulation under their item number. These comments and replies have been included under Item 7, Additional Comments and Suggestions.

Fig. 1. Geographical areas

Summary and Conclusions

From the response and the replies of the respondents, the following conclusions became apparent:

1. Although differences of opinion remain, the use of superstructures connected to flexible substructures is becoming generally accepted.
2. Current design criteria, or rather limitations, continue to be more restrictive for composite steel structures than for concrete structures.
3. There is no simple, rational design criteria currently available which takes into account the effects of such factors as shrinkage, creep, temperature, humidity, substructure flexibility, etc.
4. Maximum overall expansive lengths of up to 300 ft for steel structures and of 400-450 ft for concrete structures are generally recognized by design engineers utilizing superstructures connected to flexible substructures. However, overall expansive lengths of 671 and 736 ft have been reported for Non-Integral and Semi-Integral steel structures, respectively, and lengths of approximately 500 ft have been reported for Non-Integral, Semi-Integral and Integral concrete structures.
5. Induced stresses resulting from thermal effects, creep, shrinkage, backfill movement and settlement, etc., are recognized by bridge design engineers as potentially significant. However, there is a wide variance in methods used for consideration, if any, of such stresses.
6. Some problems were reported for both steel and concrete structures for the three types of construction. It would appear that,

in general, the problems reported are neither more prevalent nor of greater magnitude than those experienced when movable supporting and expansion devices are used. One respondent reported that asphalt concrete approach fills appear to settle more than at conventional structures.

7. Bridge design engineers are extremely interested in induced stresses and associated problems; are generally uncertain as to the significance of and suitable methods for consideration of these stresses; and would welcome a simple, rational design criteria and specific recommendations as to design details.

FIELD OBSERVATIONS OF SELECTED BRIDGES

This phase of the study consisted of field observations of the behavior of selected composite steel bridges having Semi-Integral end bents.

The primary objectives of the field observations were:

1. To determine if induced stresses resulting from environmental effects cause visibly apparent structural distress in bridges constructed with Semi-Integral end bents.
2. To provide a possible basis for comparison of stresses induced in bridges with Semi-Integral end bents and stresses induced by frozen bearing and inward abutment movement in conventional bridges.
3. To provide an insight to the factors influencing the behavior of bridges with Semi-Integral end bents and the relative magnitudes of their effect.

The oldest Missouri bridge of this type was completed in 1967. Another six were completed in 1968, and more than thirty have been completed since that time. The seven oldest bridges were selected as priority points and additional bridges were selected to complete itineraries for sequential inspections.

After preliminary trial observations, the bridge inspection form shown in Appendix B was prepared and 26 bridges were selected for initial inspection and observed during the period March 16-31, 1972. Nine additional bridges were observed during this study and are included to the extent of limited data currently available. The structures observed included four sets of "twin" bridges used for divided highways. The locations of the bridges observed are shown in Fig. 2.

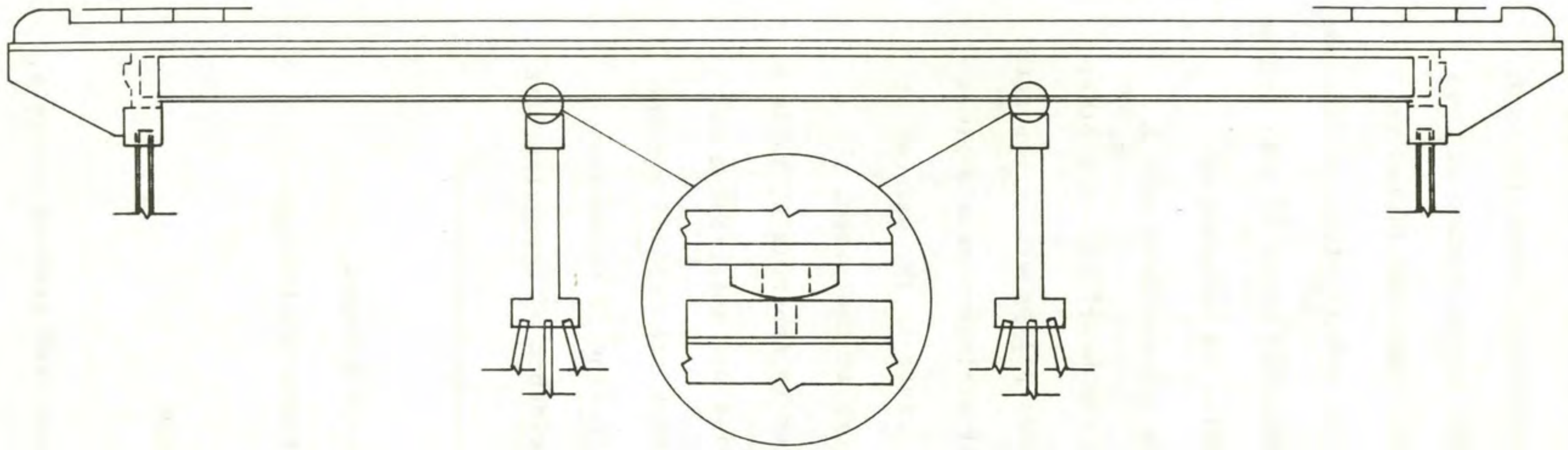
Fig. 2. Location of bridges observed

With the exception of one four-span, 226-ft bridge and one five-span, 290-ft bridge, the bridges were three-span, continuous, composite with steel girders or stringers. The over-all bridge length varied between 118 and 200 ft. Span lengths and other geometric data are listed in Appendix B, Table B. 1, and values of comparative substructure stiffnesses are shown in Appendix B, Table B. 2. In general, two types of pier bents were observed as indicated in Figs. 3(a) and (b). In the case of Fig. 3(a), pier bents were composed of concrete column piers with a pier cap and curved steel plates were used for the bearings. The other type, shown in Fig. 3(b), utilized a row of steel piling with a concrete cap and a concrete diaphragm encasing the steel stringers at the piers. Most of the bridges observed had construction joints in the parapet at the inflection points and in the parapet and curb at the piers.

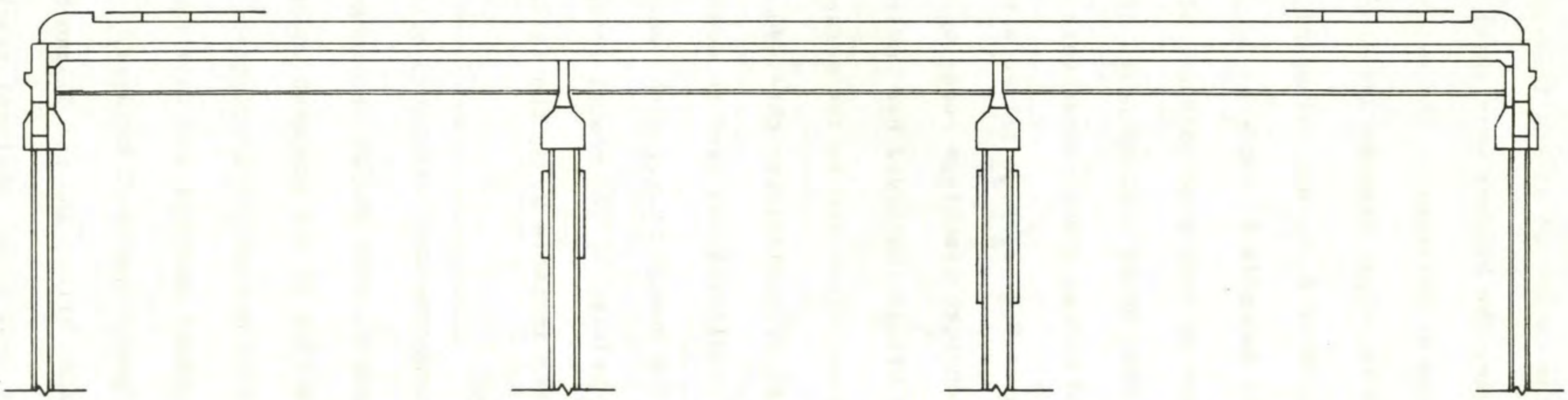
Types of irregularities observed are listed in Appendix B, Table B. 3. The term irregularity was used to denote behavior not anticipated or desired in the design (e.g., slab, parapet or curb cracks) and are not necessarily related to structural integrity. For the 32 three-span bridges listed in Table B. 3, the following incidence of irregularities was observed:

1. Transverse deck cracks--3 bridges.
2. Parapet, curb and/or deck edge cracks--24 bridges.
3. Cracking of the abutment under the girders--24 bridges.
4. Girder corrosion--2 bridges.
5. Abutment movement and rotation--1 bridge.
6. Wingwall cracks--3 bridges.

Both the 4-span bridge and the 5-span bridge displayed cracked parapets, curbs and/or deck edges, abutment cracks below the girders, and



a) Concrete column pier bents



b) Steel piling pier bents

Fig. 3. Types of pier bents

girder corrosion.

Summary of Observations

Although not observed to any degree of predictability, the following trend of irregularities became to be expected:

1. Cracking of abutments under the girders;
2. Parapet cracking near the abutments--often at the first handrail bracket;
3. Curb and edge deck cracking at the inflection points;
4. Edge deck cracking at the piers; and
5. Parapet cracking approximately 3 to 6 ft each side of the midpoint of the center span.

It should be noted, however, that the mere occurence of these irregularities is not indicative of the structural integrity and safety of the bridge, although they may permit ingress of moisture and lead to subsequent deterioration. Also, abutment cracking under the girders is the only irregularity which may be considered as peculiar to Semi-Integral end bents--as compared to conventional methods of construction. Some of the most extensive abutment cracking under the girders, as well as parapet, curb and edge slab cracking, was observed in bridges which were not yet open to traffic, including a set of twin bridges painted in 1970 but without approach slabs.

PREDICTION OF INDUCED BRIDGE STRESSES

Slab and beam movements occur whenever the ambient temperature rises or falls from a base temperature. This base would normally be selected as the average temperature occurring during the construction phase of the bridge. These movements take the form of direct elongation or shortening and a phenomenon generally referred to as warping or curling. The latter effect occurs because of a temperature gradient throughout the slab and beam system and because of unequal thermal coefficients of expansion in the steel and concrete sections.

Other causes of elongation and warping would be the shrinkage and creep characteristics of the concrete slab. It is probable that the effect of shrinkage is sufficient to cause cracking of bridge decks within a very short time after construction. Other factors which would affect movements are those of support conditions, dead load, soil stability and many other factors over which the designer has little or no control.

The general theory for strains and stresses in composite bridges due to thermal behavior has been developed by others (10, 92) and was extended during this study to include the effects of end bent and intermediate flexible support restraints. The procedure, presented in detail in Appendix C, is a theoretical approach not intended for design use.

To illustrate the theory and provide an insight to the potential magnitudes of environmental effects, the procedure is used to calculate the thermal stresses that might occur in the deck and girders of a continuous composite bridge designed by the Missouri State Highway Department in accordance with their standard procedures and specifications.

The bridge configuration and composite beam properties are shown in Figs. 4 and 5. The approach could be used to illustrate shrinkage or other effects provided the proper assumptions for strain and an equivalent temperature distribution were used.

Temperature Distribution

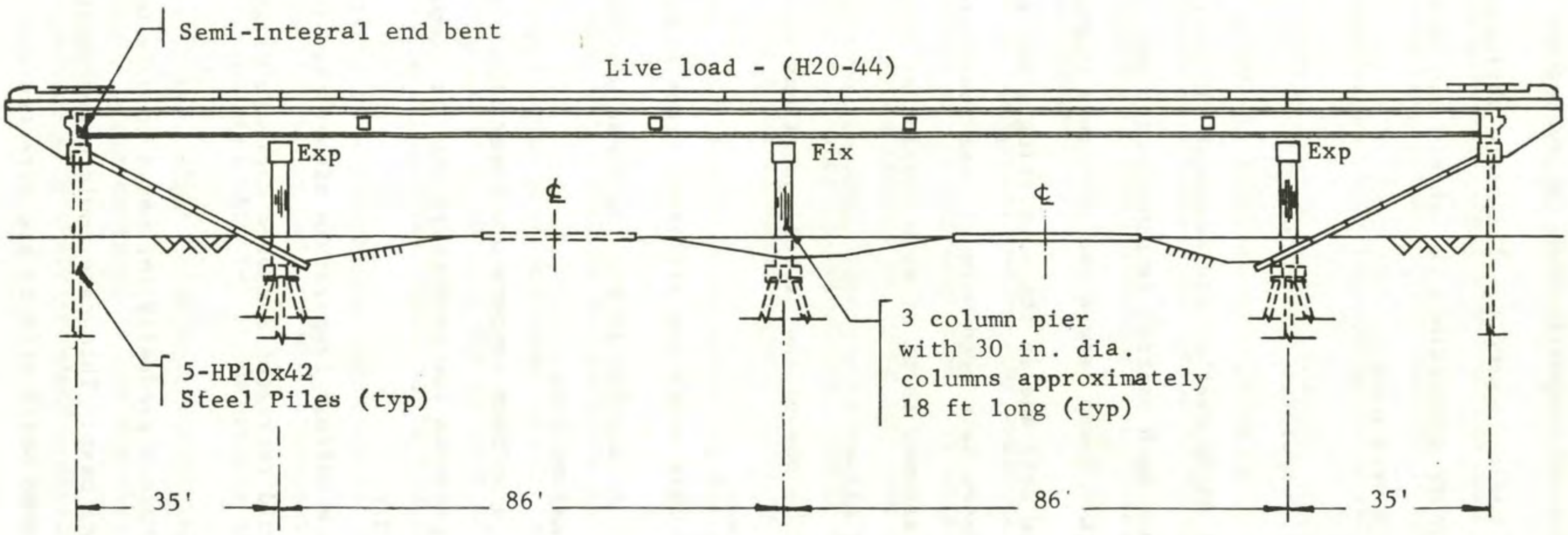
It is evident that daily weather and seasonal climatic temperature differentials can produce both overall length changes and curvature changes in a bridge deck. These length and curvature changes induce stresses in the concrete deck and in the supporting steel girders. The magnitude of these stresses have been more or less considered negligible by design engineers or assumed to be of such a value as to be within the safety tolerance of allowable stresses. Whether or not these assumptions are correct is one of the overall questions to which this investigation is directed.

For the purpose of this study six different temperature distributions have been investigated. As labeled in Fig. 6, these distributions are based on the following assumptions.

a) The slab is at a uniform temperature lower than that of the beam. This represents the case where the air temperature increases rapidly.

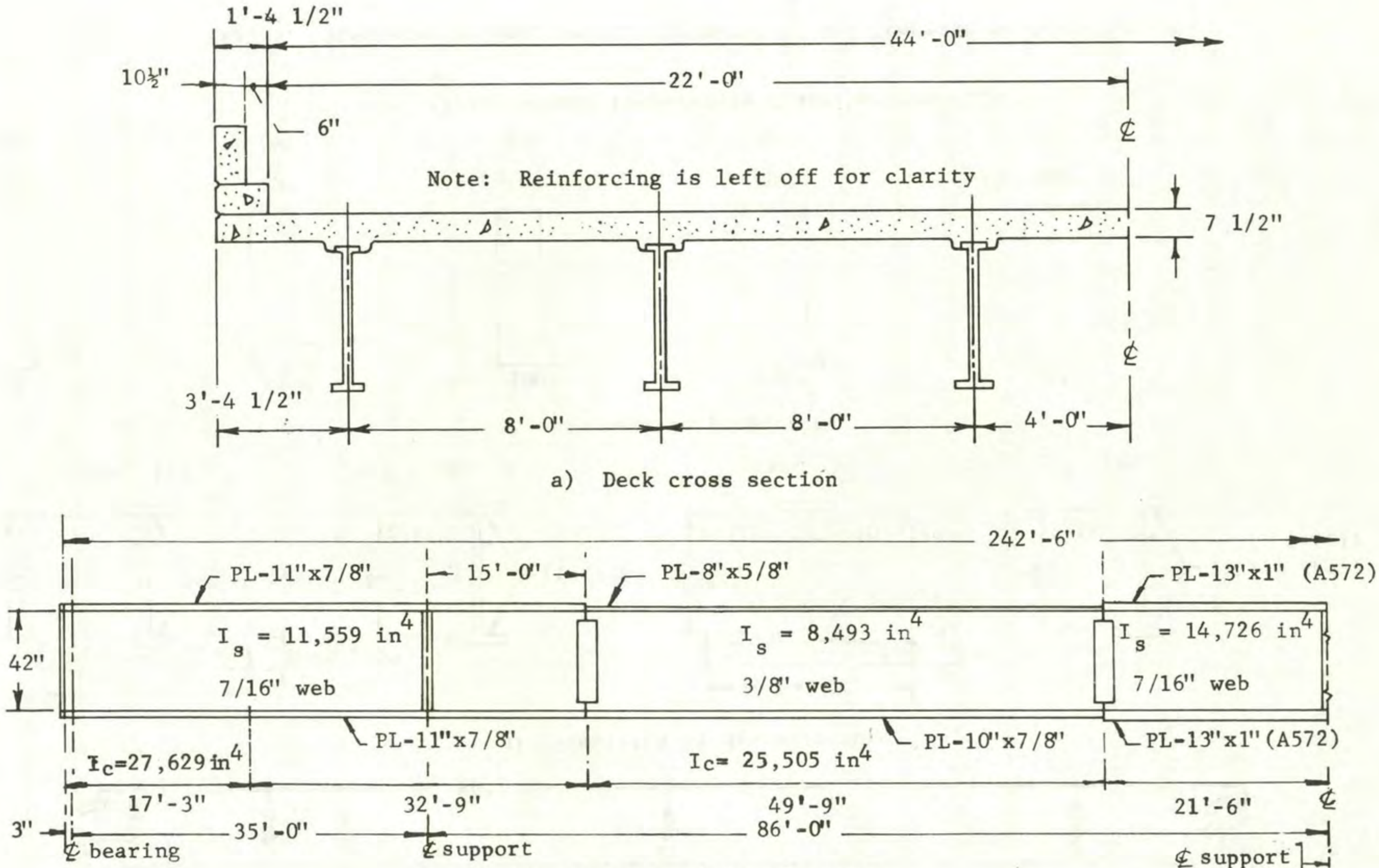
b) The slab is at a uniform temperature higher than that of the beam. This would represent a sudden decrease of ambient temperature.

c) The slab temperature gradually increases to a value greater than that of the beam. This case could occur during early morning hours when solar effects are directed upon the upper



(0° Skew)

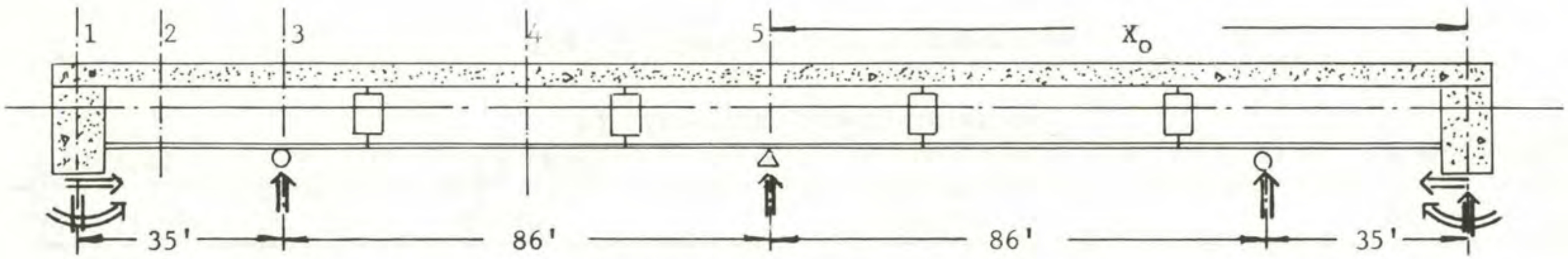
Fig. 4. Elevation of bridge reviewed



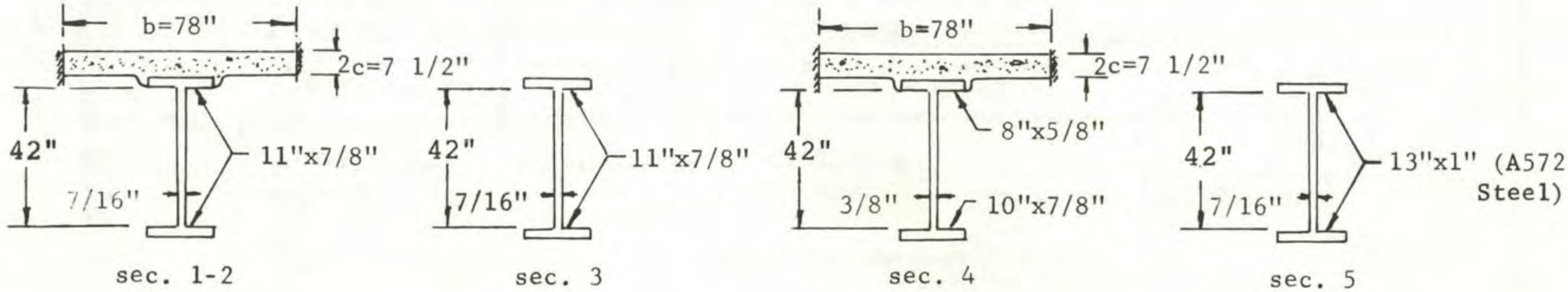
a) Deck cross section

b) Interior girder elevation

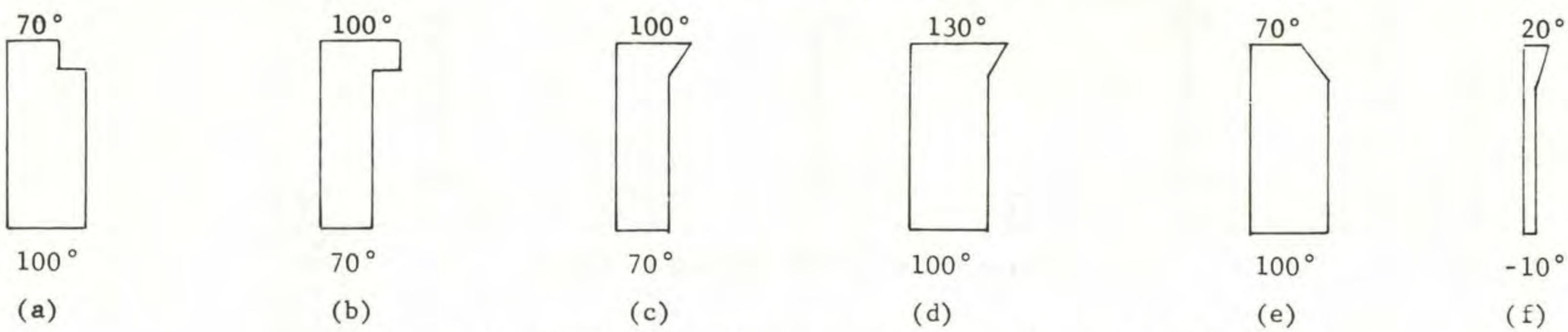
Fig. 5. Superstructure properties



a) Simulated bridge structure



b) Section properties



c) Final assumed temperature distributions (T_f)

Fig. 6. Simulated bridge, section properties, and temperature distributions

surface of the slab.

d) The upper surface of the slab receives the full solar effects of mid-summer sun.

e) The upper surface of the slab is cooled by a summer shower without any significant decrease in air temperature.

f) The extreme mid-winter condition with the slab surface receiving some solar effects.

A base temperature of 70° F has been selected for this study. It is also assumed that the beam is at a uniform temperature throughout its depth and that a maximum temperature differential of 30° F exists between the beam and the top of the slab. This may be represented in equation form by

$$T_{sy} = T_{sf} - T_i \tag{33-a}$$

and

$$T_{by} = T_{bf} - T_i \tag{33-b}$$

where

T_i = base temperature (70° F)

T_{sf} = daily or seasonal slab temperature

T_{bf} = daily or seasonal beam temperature

T_{sy} = slab temperature above or below the initial base temperature

T_{by} = beam temperature above or below the initial base temperature

Unrestrained Thermal Stresses

Unrestrained stresses have been calculated for the six listed temperature cases. The procedure for calculation of these stresses, which assumes free movement of the simply supported case, is presented in

Appendix C. While these stresses would not be a final stress case, they are indicative of the magnitude of stresses which may occur and of the temperature distribution cases which may produce the highest stresses.

In general, the procedure followed is to solve for the beam and slab interface boundary forces from Equations C.34 and C.39 of Appendix C. These forces are then used to solve for the stress distribution as given by Equations C.40 and C.41 of Appendix C.

Average values of the beam and slab properties, shown in Table 6, were used in order to simplify the calculations for the thermal stresses.

Table 6. Average Beam and Slab Properties[a] -- Typical Interior Girder

Beam	
Moment of Inertia, I_b	11,500 in.^{4}
Area, A_b	29 sq in.
Modulus of Elasticity, E_b	29,000,000 psi
Centroidal Depth	21.75 in.
Thermal Coefficient of Expansion, α_b	6.5×10^{-6} in./in./°F
Slab	
Width, b	78 in.
Depth, 2c	7.5 in.
Modulus of Elasticity, E_s	$E_b/8$ psi
Poisson's Ratio, μ	0.2
Thermal Coefficient of Expansion, α_s	4.0×10^{-6} in./in./°F

[a]Notation as in Appendix C.

The unrestrained stresses obtained by using the above properties are listed in Table 7.

Table 7. Simple Span Unrestrained Stresses--Typical Interior Girder

Temperature Distribution (Figure 6)	Location	Stress
a	slab top	+ 38 psi
	slab bottom	+ 150 psi
	beam top	-4470 psi
	beam bottom	+ 680 psi
b	slab top	- 25 psi
	slab bottom	- 114 psi
	beam top	+3310 psi
	beam bottom	- 210 psi
c	slab top	0
	slab bottom	+ 55 psi
	beam top	+1675 psi
	beam bottom	- 440 psi
d	slab top	+ 10 psi
	slab bottom	- 10 psi
	beam top	+ 505 psi
	beam bottom	- 360 psi
e	slab top	- 155 psi
	slab bottom	+ 227 psi
	beam top	-1820 psi
	beam bottom	+ 400 psi
f	slab top	- 240 psi
	slab bottom	+ 45 psi
	beam top	+5720 psi
	beam bottom	- 945 psi

Restraint Effects

An iterative procedure was used to evaluate the stiffness effects of abutments and intermediate supports. A numerical technique was employed in solving for these effects (58). This procedure essentially consists of 1) determining the elongations and rotations of the simply supported beam subjected to an induced curvature, 2) correcting for intermediate reactions assuming that frictional effects at the intermediate bents are nil, and 3) correcting for abutment stiffness.

Corrected values of elongation and rotation are then computed and if the values differ appreciably from those obtained in 2) then the procedure is repeated until the convergence criteria is met.

Abutment Piling Flexibilities

Procedures have been developed (51, 64) for evaluating the behavior of a pile when subjected to lateral loads. An investigation based on these procedures was made to determine the flexibility coefficients of the piling at the abutments of the bridge under study.

The assumed pile depth was sixty-five feet below the pile cap with sixteen feet of pile being in a pre-bored hole through compacted fill and forty-nine feet being in undisturbed clay. The subgrade modulus for dry sand around the pre-bored portion was assumed to be seven tons per cubic foot and seventy-five tons per cubic foot for stiff clay (76). The flexibility coefficients obtained by computer analysis were 0.00354 ft and 0.000486 radians for deflection and rotation due to a one kip horizontal load applied at the top of the pile; and 0.000486 ft and 0.000110 radians for deflection and rotation due to a one kip-foot moment applied at the top of the pile

Restrained Stresses

Restrained stresses were calculated for two temperature distributions. The two distributions studied were (a) and (f) as shown in Fig. 6-c since these were the apparent conditions which would cause the maximum tension and compression stresses.

The restrained stresses were calculated from the indeterminate joint moments which were determined as outlined above. The actual stress calculations were made on the basis of the composite section in regions where the dead load and live load moments indicate positive curvature and on the non-composite section where a negative curvature is indicated.

An additional effect which was included is that of axial stresses. The maximum computed longitudinal compressive force was 10.7 kips for temperature distribution (a) of Fig. 6-c. A longitudinal tensile force of 33.0 kips was obtained for temperature distribution (f) of Fig. 6-c. The magnitude of these forces are too small to cause any significant secondary moment or stress amplification. However, it is obvious that these forces should be considered in the design of the abutment shear key and piling.

Summary of Thermal Stresses

A summary of thermal induced stresses is presented and compared with dead load and live load stresses in Table 8. It would appear that this effect is of the order of twenty percent of the calculated dead load and live load stresses. This calculated increase in stress could perhaps become critical especially in regions near the intermediate supports because of the increased compression flange stress.

All thermal stress calculations were based on the assumption of an elastic medium. For seasonal temperature changes it is quite likely that the actual thermal stresses may be increased in some regions and decreased in others because of creep effects, shrinkage and soil pressures.

Table 8. Thermal Stresses and Design Stresses--Typical Interior Girder

Beam[a] Section	Stress Location	Thermal Stress[b], Psi (a)[a]	Thermal Stress[b], Psi (f)[a]	(DL + LL) Design Stress[c], Psi
(1)	slab top	+ 80	- 350	---
	slab bottom	+ 170	+ 20	---
	beam top	-4320	+5550	---
	beam bottom	- 740	+3320	---
(2)	slab top	+ 120	- 350	- 173
	slab bottom	+ 190	+ 20	- 65
	beam top	-4270	+5550	- 520
	beam bottom	-1650	+3320	+ 4096
(3)[d]	beam top	- 360	+1650	+18808
	beam bottom	-4140	+5380	-18808
(4)	slab top	+ 150	- 390	- 810
	slab bottom	+ 180	+ 10	- 310
	beam top	-4190	+5460	- 2470
	beam bottom	-2430	+4030	+19406
(5)[d]	beam top	-1350	+2580	+25926
	beam bottom	-3120	+4440	-25926

[a]See Fig. 6 for beam section location and temperature distribution cases.

[b]Based on the average values shown in Table 6.

[c]Based on the actual section properties shown in Fig. 5.

[d]Non-composite sections.

RECOMMENDATIONS FOR FUTURE STUDY

The investigation of design criteria for stresses induced by semi-integral end bents was originally planned to be conducted in three phases. Phase I was to be essentially a review of the literature of related studies; Phase II involved selection, installation and testing of instrumentation components in a test model; and Phase III was to consist essentially of instrumentation during construction and sequent testing of a four-span grade separation structure built with semi-integral end bents; and reduction of data toward development and presentation of design data.

Due partly to economic considerations and also to the possible early construction of the anticipated prototype bridge, it was deemed necessary to forgo the model study and substitute such laboratory studies as might be possible preliminary to field instrumentation of the prototype.

The development of the design criteria will require a long range field study. The environmental factors considered most significant to bridge behavior are: creep, shrinkage, temperature, humidity, and backfill movements and settlement. The parameters to be measured to account for these effects are: internal and ambient air temperatures; ambient humidity; concrete moisture; solar radiation; wind speed and direction; creep and shrinkage; backfill pressures and settlement; horizontal and vertical deflections; mechanical and thermal properties of the concrete; coefficient of friction for the expansion bearings; unit strains; reactions at the expansion bearings; abutment pile deformations; and shear and moment capacity at the abutment shear key.

The instrumentation techniques and sensors recommended herein were selected on the basis of past performance as reported in the literature and the following criteria. Sensors for long term field studies must be durable in order to withstand placement and handling operations; moisture proof; have adequate sensitivity and remain stable over long periods of time; and be temperature compensated from -20° to 150°F. These requirements eliminate many sensing devices and routine testing procedures. It is recommended that the final selection be made on the basis of economics, simplicity and laboratory tests of the sensors.

Instrumentation for long range studies involving environmental effects requires special techniques and only a few related long term field investigations have been reported. Thus, it is extremely important that the experimental techniques be refined in the laboratory prior to use in a field investigation. The advantages of a laboratory study are obvious, and would give the investigator an opportunity to modify procedures; allow the personnel to become totally familiar with the sensors, instrumentation techniques, placement procedures, and use of equipment; and would permit a change of sensors and techniques in preparation for the most efficient, usable and economical procedure. It would be extremely dangerous to attempt a study of such magnitude as a field study of a prototype bridge without adequate preparation beforehand.

Proposed Measurements

Other investigators have found in related field studies that within a cycle of observation changes occurred so rapidly from point to point that data obtained by hand recording was useless. Thus a data acquisition system becomes necessary to monitor cyclic data. It has also been found that symmetrical behavior does not necessarily occur and sensors should ideally be placed over the entire width and length of the structure. However, this often is unfeasible due to economic considerations.

An instrumentation program to obtain reliable data and data reduction for a number of parameters may become expensive. In addition to the data acquisition system, which provides the necessary automatic recording, the instrumentation program would benefit by the use of a minicomputer in the system. This has several advantages; for example, it can be programmed to check the system so the investigator will know if the sensors are providing reliable data. With this device the erratic sensor can be traced and evaluated for the cause of the problem. It also, to a certain extent, provides bounding on the sensors during placing so that damaged sensors can be replaced where it is possible to do so. The minicomputer adds flexibility to the system, a set of checks and balances, realtime analysis and data compression, and a means of taking raw data from the system and storing it on magnitic tape in engineering units. These tapes would then be the source of recorded data to be used for final reduction.

It is believed that the following approach for instrumenting a full-scale bridge will yield the data required. It should be emphasized, however, that these procedures are not necessarily the only means of obtaining the required data for induced stresses in restrained composite-girder bridges subjected to environmental effects, but that most of the procedures are among those that have been proven successful over the years by other investigators on related studies.

Superstructure Properties

It is recommended that the concrete used in the structure be tested under actual field conditions for both strength and stress-strain characteristics, as well as thermal behavior and creep and shrinkage characteristics. For example, the stress-strain characteristics can be studied by exposing the cylinders to actual field conditions and periodically testing in the laboratory for stress-strain and strength. The coefficient of linear expansion for the concrete deck may be evaluated by subjecting sections similar to the deck to heat in the laboratory and measuring elongation and contraction versus temperature change and moisture. It is also recommended that sections similar to the concrete deck and composite beam be subjected to these same conditions and the strain changes versus temperature and moisture elvaluated.

Concrete specimens having the same percentage of reinforcement and volume-surface ratios as the bridge deck should be subjected to actual field conditions and studied for shrinkage behavior as a function of time, humidity, and concrete moisture. These tests can be done quite simply by measuring the volume change versus these factors. A similar test can be performed by placing specimens in a creep rig under constant pressures to evaluate the creep behavior of the deck. These pressures would then be plotted versus humidity, moisture, time, and volume change. A creep rig can be built and the measurements made rather inexpensively. These tests would provide a basis for evaluation of the approximate magnitude of creep and shrinkage in the bridge structure under various load changes.

Substructure Properties

The total induced stress in a restrained structure is influenced by the substructure stiffness. Expansion bearings are designed to move with little or no restraint. However, corrosion, and/or debris can increase the restraint even to the point of approaching a fixed bearing condition. It is recommended that a bearing identical to the expansion bearings used for the prototype and exposed to the same field conditions at the bridge site be tested for the friction coefficient periodically in the laboratory. A laboratory test for the coefficient of friction versus a vertical reaction is relatively simple by use of a testing machine and a jack. The testing machine gives a measure of the vertical reaction and the jack can be used to initiate horizontal movement and measure the corresponding horizontal force.

Additional restraints are imposed by the stiffness of the piers, the effective stiffness of the soil-pile interaction at the abutments and the shear key at the abutment. In order to evaluate the shear-key it is recommended that a series of sample abutment sections be placed under actual field conditions. The specimens would be used for a laboratory study of the shear capacity of the key--by subjecting the upper and lower cap sections to opposite and equal horizontal forces until failure--as well as to evaluate the moment resistance capacity of the cap--by subjecting the caps to a couple.

The effective stiffness of the pile-soil interaction at the abutment is very difficult to evaluate. However, it is recommended that prior to placing the abutment caps on actual bridge abutments, that each pile be subjected to horizontal loading of fixed increments and the corresponding end rotations and deflections measured. Likewise each pile should be subjected to end couples of fixed increments and the corresponding end rotations and deflections measured.

These tests should establish the approximate effective stiffness of each pile and the corresponding average effective stiffness of each abutment support. Then knowing the measured bridge elongation, the induced abutment forces can be evaluated by using the following expression:

$$\begin{bmatrix} P \\ M \end{bmatrix} = [k] \begin{bmatrix} \delta \\ \theta \end{bmatrix} \tag{34}$$

Prototype Bridge Measurements

It is believed that data relative to bridge behavior can be obtained for environmental effects and evaluated and compared provided the data is recorded periodically by the use of a data acquisition system. This allows all sensors (except those feasible to be read by hand) to be read nearly simultaneously and thus compared for induced strains, temperature distributions, and resultant induced stresses. Comparative strains should be obtained by mechanical gages, for example the Whittemore strain gage, as a check on the instrumentation.

Strains

As previously stated, it is necessary for studies of this type, that the gages be durable, so as to avoid damage during construction operations; stable; moisture proof; have adequate sensitivity; and demonstrate virtually no drift over long periods of time--for example, in excess of one year. In addition, prior to construction precautionary procedures need to be developed in the laboratory so as to protect the gages during construction. Other investigators have used wire cages to protect the gages and others have placed the gages in concrete briquets. The latter method has the disadvantage of possible errors resulting from differences in material properties of the briquets and the field concrete. It is recommended that adequate time be allocated for a laboratory study to develop, modify and/or improve techniques to avoid loss of gages.

Gages that have demonstrated satisfactory long-term stability are the Carlson strain meter, the vibrating wire strain gage, and mechanical gages such as the Whittemore strain gage. Other gages considered are the standardizing strain gage (invented by G. E. Monfroe), weldable strain gages, Valore gage, SR-4 gages, and specially designed semi-conductor transducers. Although it currently appears that these generally are the only gages worthy of consideration, other gages may be manufactured in the near future and warrant investigation and comparison.

The Carlson strain meter was originally developed to measure internal strains in massive concrete dams. Meters installed prior to 1940 were still operating in 1953[a]. The meter can be supplied by Terrametrics, Golden, Colorado. The gage is an unbonded-wire gage; that is, the resistance elements are not continuously attached to the material being strained. The meter consists of a long cylinder with anchors at the end to engage the surrounding concrete. Within the cylinder are two equal coils of tensioned music wire connected in such a manner that when the concrete is strained one of the coils is stretched

---

[a]Raphael, J.M., Carlson, K.W., "Measurement of Structural Action in Dams," Part 2, Terrametrics, Golden, Colorado, 1965.

and the other shortened. The gage must be preset for the maximum strain range in the required direction. The ratio of the resistance of the expansion coil and the resistance of the contraction coil is used as a measure of length change in the strain meter. Temperature can also be measured with the meter by taking the sum of the resistances of the expansion and contraction coils. This meter has the advantage of being able to obtain internal temperatures at the point where strain is measured. The coils are encased in a brass tube filled with corrosion-resistant oil to prevent corrosion and moisture damage. Each meter is connected with a 3-conductor cable.

The body of Carlson strain meters vary in length from 3 in. to 20 in. Size of the gage is an important consideration and thus the meters considered for this study are the miniature strain meter (SM-3), with a 1/2-in. diameter by 3-in. long cylinder, for the concrete deck and perhaps either the SM-3 or the SA-10 for the concrete piers. The SA-10 has a body length of 10 in. Saddle mounts are available for attachment to steel structures.

Vibrating wire strain gages have been used extensively in Europe and England for long range studies. The long term stability and sensitivity characteristics are reported to be good. The measuring element of the gage is a tensioned wire attached to the two ends of the cylinder. The wire is plucked electromagnetically and its frequency is measured. Changes in frequency are related to changes in strain. Vibrating wire strain gages can be supplied by Terrametrics, Golden, Colorado, or the Slope Indicator Co., Seattle, Washington. The sensitivity of the vibrating wire gage is comparable to that of the Carlson strain meter; however, most of the gages do not measure temperature. There is one vibrating wire gage presently being developed that will measure temperature as well as strains; however, a thermocouple will be used for the temperature sensing element.

Mechanical strain gages such as the Whittemore or Munich gage, which use dial gages to mechanically measure movements between reference points attached to a structure, are widely used. They have good long term stability and, when temperature compensation is provided, are excellent for measuring long-term effects. It is recommended that mechanical gages be used for comparison and as a check on the instrumentation.

Care in the design of reference points and their attachment to the structure, and in the procedures required in reading the device are needed for consistent results (67). Thus, it is necessary to establish procedures and develop reference points in the laboratory prior to field use of mechanical gages.

The standardizing strain gage, invented by G. E. Monfroe of the Portland Cement Association, is a gage designed for strain measurements for long range studies. This gage, however, is not commercially available and will not be considered further in the course of the investigation.

Weldable strain gages such as those furnished by Ailtech, Cutler-Hammer Co., (formerly Microdot, Inc.) are electric resistance gages. Weldable gages consist of a short length of material such as nickel-chromium wire which has been electroformed or etched so that a sensitive element is formed. The wire is insulated by a highly sensitive compacted magnesium powder encased in a small diameter tube made of stainless steel or Inconel alloys. This tube is then welded to an exterior flange which can then be welded to almost any metal. Past applications have included piling installations, bridges, and missile skins. It is reported that lead lengths in excess of 100 ft have been used successfully. These gages have been designed for severe environments and are inexpensive. It would seem that weldable gages would be the ideal gage due to the small size, durability, and the reasonable cost of the gages. Drift has been reported to be 50 to 100 micro units of strain per year, thus requiring adequate dummy gage installations[a]. It has been reported, that weldable gages give poor results for long range studies. In one case gages became erratic after two weeks use[b].

The Valore gage manufactured by Baldwin-Lima-Hamilton, is a SR-4 gage mounted on a piece of brass for embedment in concrete. It was reported that these gages can give erratic results during long range studies[c].

The technique of pencil bars has been used for measuring concrete strains. These are small steel bars with strain gages such as the SR-4 type. It has been reported that long term stability of the gages and of the electrical measuring circuits is generally a problem with SR-4 bonded wire gages (67). However, these gages can often be used satisfactorily for short term transient movements.

Specially designed semi-conductors, as manufactured by DSC Incorporated, give high output in the order of 50 or 100 mv/v, infinite resolution, high linearity and are temperature stable. The sensors provide both strain and temperature measurements. Their primary use has

---

[a]Private communication to J.L. Hulsey, Oct., 1972.

[b]Private commumication to J.L. Hulsey, Oct., 1972.

[c]Private communication to J.H. Emanuel, Dec., 1971.

been as load transducers and load cells. However, the information and past performance records available during this study were insufficient to support confident recommendations.

Temperatures

It has been established that internal temperature gradients may induce significant stresses in composite girder bridges with semi-integral end bents. It is also well known that temperature lags occur between the ambient air and internal temperatures and are dependent among other things, upon the materials themselves. For this reason evaluation of internal and ambient air temperatures are requisite to development of a design criteria.

Sensors considered for long range temperature measurements include thermometers, thermocouples, resistance temperature detectors (RTD's), thermistors and resistance strain gages.

The common mercury thermometer is inexpensive, easily accessible, but is not adaptable to automatic recording coincident with other data. This problem can partially be solved by measuring and recording ambient air temperature and time at specified intervals with a time-lapse movie camera or by the use of recording thermometers as furnished by companies such as Honeywell, Inc., or Weather Measure Corporation. A thermometer may also be used in concrete for internal temperature by drilling an access hole in the concrete. However, the points to be measured must be accessible. The major disadvantage is that the boundary of air surrounding the thermometer around the walls of the hole may cause temperature discrepancies of 10° F or more (67).

Thermocouples, RTD's, thermistors, and resistance strain gages all have the advantage of being compatible with automatic recording systems for both ambient and internal temperature measurements.

Thermocouples have demonstrated their reliability over long periods of time and are inexpensive. The common materials used for thermocouples are iron-constantan, copper-constantan, and chromel-alumel. Thermocouples are easily obtainable from companies such as Omega Engineering and Honeywell, Inc. Care is required in splicing lead wires to cables leading to automatic recording devices. Splices can be made without error, however, with proper splicing techniques and by the use of silver solder for the common thermocouple materials such as copper-constantan.

Resistance temperature detectors (RTD's), such as nickel elements as furnished by RDF Corp. or Honeywell, Inc., have the desired temperature range; durability; stability, with less than $\pm$ 0.25% shift in calibration in one year; an accuracy of 0.25° F or $\pm$ 1/2% of a reading, whichever is greater; and a repeatability of $\pm$ 0.1% of range. RTD's may experience non-linearity depending on the material used for the sensing element. Nickel elements, experience very little non-linearity except in the lower temperature ranges. This can be corrected by the minicomputer properly programmed in the system.

Thermistors are thermal resistors or resistors with a high negative temperature coefficient of resistance; that is, as the temperature increases, the resistance goes down and as the temperature decreases, the resistance goes up. Thermistors are semi-conductors of ceramic material made by sintering mixtures of metallic oxides such as manganese, nickel, cobalt, copper, iron, and uranium. Thermistors are very sensitive, stable, and reliable over long periods of time; however, they are more expensive than thermocouples or RTD's and thus will not be considered further for field measurements in the course of this investigation.

Electrical resistance gages considered for temperature measurements in this study are the Carlson strain meter, semi-conductor gages and SR-4 gages.

The Carlson strain meter is especially recommended because of its proven stability, long term reliability over the years, and its capability of measuring both strain and temperature. Readings can be taken of both the ratio of the resistances of the two coils and of the total resistance of the two coils in series. Using the fact that the total resistance is a function of temperature it is possible to measure the temperature of the meter. The calibration constants for temperature measurements are linear and are 8.1° F/ohm for the SA-10 and 12.2° F/ohm for the SM-3 meter. It is reported that the Carlson strain meter readings are in good agreement with thermocouple readings (67).

SR-4 gages can be mounted on unbonded bars to measure internal concrete temperatures. However, their reliability for a long range study is questionable since SR-4 gages often become unstable over long periods.

The semi-conductor has a temperature sensitivity of high magnitude. Its major disadvantage is its moisture sensitivity and expense.

It is recommended that thermocouples or RTD's be used in conjunction with the Carlson strain meter for measuring temperature. The final selection should be based on a laboratory evaluation prior to field testing.

Ambient Humidity and Concrete Moisture

It is well known that ambient humidity affects creep and shrinkage of structural concrete. Thus, humidity should be one of the parameters measured during the proposed long range study. Humidity sensors, such as the PRC-11 of Phys-Chemical Research Corp. provide sensing for humidity ranges of 0 to 100% and can be connected to the data acquisition system for automatic recording coincident with other sensing devices.

At this time there appears to be no reliable reported means of measuring concrete moisture under actual field conditions, over long periods of time, and by an automatic sensing unit. It is recommended that a humidity probe, such as the PRC-11, be placed in a sealed well in the lower face of the concrete deck and connected to the data acquisition system. This technique was checked with the manufacturer and deemed feasible. However, it is recommended that this procedure be investigated in the laboratory prior to usage on the prototype bridge.

Solar Radiation

Solar radiation has a significant effect on the internal structural temperature distribution. For example, a bare concrete deck will have a different temperature distribution than a deck with an asphalt overlay. Solar radiation can be measured by the use of spectral pyranometers. The Eppley precision spectral pyranometer as manufactured by the Eppley Laboratory, Inc., is expected to measure this parameter with the required sensitivity and stability, and the output is linear. This device can also be connected to a data acquisition system for automatic recording. Once solar radiation effects can be predicted the design criteria may be extended to other geographic areas.

Date-Time

The evaluation of recorded data must be investigated from trends due to daily and seasonal changes. The recorded data must therefore be taken coincidental with the date and time of day. Date and time may be recorded by the use of a digital clock or the minicomputer programmed to develop and record the data.

Wind Speed and Direction

Wind speed and direction not only apply forces to the bridge but may affect time lags between ambient air and internal temperatures. Wind speed and direction can be recorded by the use of a wind speed and direction transmitter such as the Belfort Instrument Co. Type L Wind Speed and Direction Transmitter. This device can be connected to the data acquisition for automatic recording.

Vehicles

Bridge behavior due to environmental effects is difficult to evaluate if data is recorded while the structure is subjected to vehicular (short term) loading. Thus, a device should be placed at each end of the bridge to sense these vehicles and stop data recording while vehicles are on the bridge.

Superstructure

Techniques for measurement of strain, temperature distribution, concrete moisture, mechanical properties and the thermal coefficient of expansion for use in evaluating environmental effects upon the structure have been previously discussed. In addition the following should be observed.

Deflections

The prediction of composite-girder bridge behavior must include not only the expected induced stresses but the vertical deflections as well. Instrumentation for vertical deflection measurements should have sufficient accuracy to measure the deflection to the nearest 0.001 ft.

The bridge under consideration has limited clearances and this consequently eliminates structural systems under the girders. It has been reported that one of the most practical and straightforward methods is the use of a precise level and a precision level rod (67). Reference points on the bridge deck and permanent bench marks should be established to insure that true deflections are always measured.

Bridge Elongation

Induced stresses in restrained bridges are also influenced by the bridge length, the point of zero movement, the substructure stiffness and the bearing type and condition. Measurements of horizontal movement at the abutment are needed to develop the interaction among these factors. These data may be measured by the use of dial gages attached to a fixed reference point and a time-lapse camera synchronized with the data acquisition system. This procedure has been used successfully by other investigators and is believed to be a rather inexpensive and reliable method. Horizontal movement may also be recorded by means of a vertical cantilevered column, isolated in a vertical hollow pipe to avoid soil movement effects, instrumented to give strain readout and connected to the data acquisition system.

Substructure

Soil Movement

Soil movement behind the bridge abutments may cause significant induced stresses in the structure. To better understand and predict these movements, measurements should be taken and evaluated for this effect. Reliable soil pressures have been measured by both Carlson soil stress meters and Gloetzl pressure cells in related studies. Both are available from Terrametrics, Golden, Colorado. The Carlson stress meter measures the pressure, or compressive stress, of the soil acting on the exposed face of the meter. The pressure is transmitted by a mercury film to an internal diaphragm whose deflection is measured by a Carlson strain-meter type sensing element which is also capable of measuring temperature as explained previously in the discussion of the Carlson strain meter. The soil stress meter has an effective area of 42 square inches. Care is required to protect the sensor during installation and construction. It has automatic readout features which makes it compatible with the data acquisition system.

Gloetzl earth pressure cells are completely hydraulic, have long term reliability and a measuring range of 0-3750 psi. The cells consist of a pressure sensing pad and a hydraulic bypass valve in a hydraulic system. To measure the pressure, the hydraulic pressure in the cell delivery line (90 percent kerosene and 10 percent 10-weight non-detergent oil) is slowly increased at a constant rate. When the delivery pressure becomes equal to the pressure acting on the cell, the valve system opens, bypassing the hydraulic fluid to the cell return lines. The pressure is indicated by a precise manometer in the delivery line. This device is not readily adaptable to automatic readout. The costs of both units are approximately equal.

It is recommended that pressure transducers such as the Carlson stress meter or similar laboratory fabricated devices be placed on the approach side of the abutment caps and near the top of the piles to obtain horizontal soil pressure. It is believed that similar transducers can be economically fabricated by the investigator to give reliable long term data. However, they should be proven by laboratory investigation prior to actual instrumentation of a prototype bridge.

Abutment Piles

The actual field behavior of abutment piles would ideally be verified during the study by placing gages along the full length of the pile prior to driving. It has been reported that the technique has been used successfully for short term studies. Data has been taken successfully during pile driving, during loading, and after unloading[a]. Adequate protection must be provided around the gages such as a small angle or channel. It is recommended that consideration be given to the Carlson strain meter (long-term gage) and to weldable strain gages for this use. This approach should be investigated prior to study of the prototype in order that refinements might be made prior to actual field studies.

Another procedure for evaluation of pile behavior is the use of an inclinometer traveling within a piece of tubing attached to the pile. The tubing is placed on the web of the pile prior to driving and then cleaned prior to instrumentation. One to two-inch diameter inclinometers have been reported to give reliable results for long term studies. The Slope Indicator Co. manufactures a miniature inclinometer (Digitilt Inclinometer model 50301) with a 1.6875-in. O.D. sensor. It is designed for automatic readout with a digital voltmeter and may be used with the data acquisition system. The accuracy is $\pm$ 0.025 foot per 100 feet of casing for a deviation of $\pm$ 51.0 seconds of arc. It is reported to have the necessary linearity, sensitivity, and repeatability.

Should strain gage placement prior to driving prove impractical, it

---

[a]Personal communications to J.L. Hulsey, Sept. and Oct., 1972.

is recommended that strain-gages be placed along the upper 15 ft of the pile (the depth of prebore) after driving. This can be accomplished by drilling an oversize prebore, enabling gages to be mounted prior to the final sand placement.

Reactions

As shown earlier, the amount of restraint imposed at friction bearings depends upon the coefficient of friction of the bearing, and thus $H = \mu R$, where μ is the coefficient of friction of the bearing, R is the reaction, and H is the restraining force, provided impending motion occurs. It is possible for the restraining force to be less than H, which then is a function of the pier stiffness. It has been recommended that the value of μ be determined in the laboratory, and thus the reaction also needs to be evaluated. It is recommended that the reactions at friction bearing locations be measured by means of load cells placed at the bearings. Due to the expense of large capacity load cells and the required sensitivity needed for a study of this type, it is believed that reliable cells can be fabricated in the laboratory utilizing a high sensitivity sensing device such as a semi-conductor gage (G.F. $\simeq$ 150). Linearity conversion would be accomplished by means of the minicomputer in the system. DSC, Inc., has semi-conductors specially designed and used primarily for load cell purposes without non-linearity difficulties. It is recommended that these cells be designed and laboratory tested for adaptation to the prototype bridge.

Abutment Cap

The behavior of the abutment cap provides a basis for evaluation of shear and moment transfer capacity. This data when compared with the laboratory tests of sample cap sections should give an insight to the performance of the key and joint. It is recommended that the relative movement of the top and bottom of the semi-integral cap sections be measured by the same techniques proposed for measurement of longitudinal elongation. The cap rotation when combined with the laboratory data should provide information concerning the moment transfer across the joint. It is also recommended that pencil bars be placed vertically near each edge of the cap across the joint, enabling the rotation to be measured. Although these may not give information for the period of the long term study they should provide reliable short term comparative data. These sensors would be connected to the data acquisition system for automatic recording.

Pier Stresses and Temperatures

A study of the total substructure stiffness is not complete without a determination of the behavior of the piers. This data can be obtained through measurement of strains, temperature, and concrete moisture by the techniques discussed above in this section. This data would provide a basis for evaluation of the induced pier forces as a result of movement of the substructure, as opposed to the strains developed due to the effects of temperature and other environmental effects.

Support Settlement

Support settlement may be significant to studies of this type and therefore should be considered. It is believed that movement considered to be significant can be hand recorded periodically by means of a precise level and a precision level rod.

Summary of Recommended Instrumentation

Selection of sensors was based upon past performance as reported in the literature; personal communication with other investigators; and the criteria that they be durable, moisture proof, sensitive, stable, and temperature compensated. The parameters to be measured and the sensors recommended are summarized in Table 9.

Table 9. Recommended Instrumentation

Parameters to be measured	Recommended sensors[a]
Internal and ambient air temperature	RTD's; miniature Carlson strain meter (SM-3) at points of strain measurement[b] or Thermocouples and miniature Carlson strain meter (SM-3) at points of strain measurement[c]
Ambient humidity and concrete moisture	Humidity probe--PRC-11, Phys-Chemical Research Corp.
Creep and shrinkage	Laboratory tests on field exposed specimens
Backfill pressures	Carlson stress meter or laboratory fabricated pressure transducers
Settlement	Precise level
Vertical deflections	Precise level
Horizontal deflections	Dial gages and/or an instrumented cantilever column for automatic recording
Mechanical and thermal properties	Laboratory tests

[a] All sensors are compatible with both System Block Diagrams I and II (Figs. 9 and 10) except as noted.

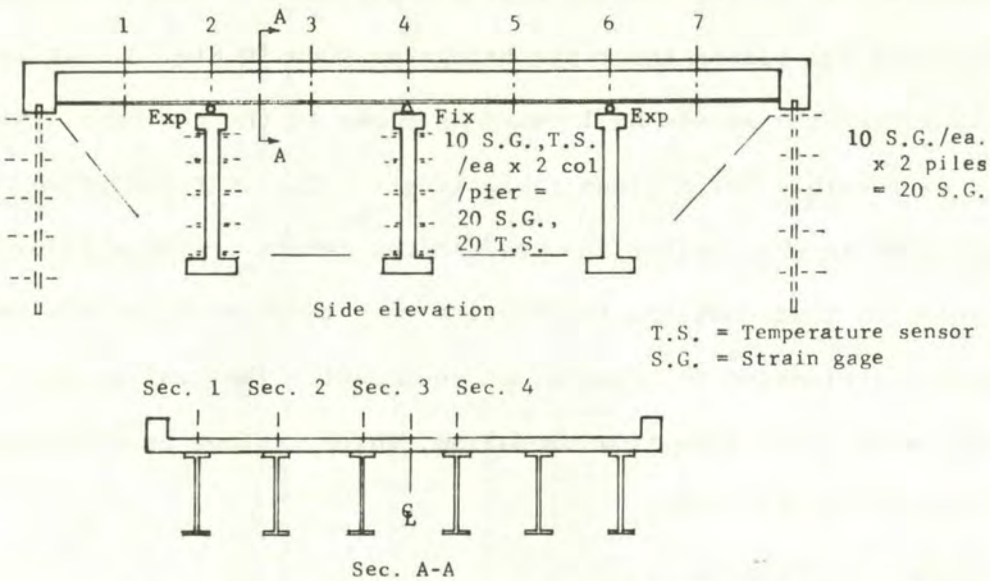
[b] Refers to a system of instrumentation compatible with the data acquisition system of Fig. 9, System Block Diagram I.

[c] Refers to a system of instrumentation compatible with the data acquisition system of Fig. 10, System Block Diagram II.

Table 9 (Continued).

Parameters to be measured	Recommended sensors[a]
Coefficient of friction of cap bearings	Laboratory test on field exposed specimen
Unit strains	Carlson strain meters (SM-3) supplemented with pencil bars (SR-4 gages); mechanical strain gages; and evaluation of use of weldable strain gages for field conditions
Reactions at expansion bearings	Load cells (laboratory fabricated utilizing semi-conductors)
Abutment pile stiffness	Hand recorded data of field load tests supplemented with performance tests using strain data (weldable or Carlson strain meters) or miniature slope indicators, Digitilt Inclinometer Model 50301, Slope Indicator Co.
Shear and moment capacity of abutment key	Laboratory tests supplemented with field performance tests
Solar radiation	Eppley Precision Spectral Pyranometers, Eppley Laboroatory, Inc.
Wind speed and direction	Type L Wind Speed and Direction Transmitter, Belfort Instrument Co.

The recommended instrumentation was generally selected for automatic readout using a data acquisition system. Alternative distribution of the proposed sensors for automatic recording are shown in Fig. 7. Alternate No. 1, with approximately 700 data points, is considered the ideal distribution for obtaining the required data and alternate No. 3, with approximately 350 data points, the minimum required for a study of this type. It should be noted that although a study might be conducted with fewer data points than alternate No. 3, and perhaps some useful data obtained, there is a minimal point at which the data obtained is insufficient toward the development of a design criteria.



Misc.

1) Humidity 2) Ambient air temp. 3) Moisture 4) Solar radiation 5) Time
6) Date 7) Vehicle conversion 8) Wind velocity and direction
9) Reactions at exp. bearings 10) Longitudinal movement
11) Possible pile inclinometer readings

Alt. #1 (all 7 loc.)

I. Abut. piles
  1) 20 S.G. x 2 = 40 S.G.

II. Piers
  1) 20 S.G. x 2 = 40 S.G.
  2) 20 T.S. x 2 = 40 T.S.

III. Slab (loc. 1-7) (located @ top, center, bottom)
  1) 3 S.G. x 4 sec x 7 = 84 S.G.
  2) 5 T.S. x 4 sec x 7 = 140 T.S.

IV. Girders (loc 1-7) (located @ 2 top 2 center, 2 bottom)
  1) 6 S.G. x 4 sec x 7 = 168 S.G.
  2) 4 T.S. x 4 sec x 7 = 112 T.S.

Subtotal 292 T.S.
332 S.G.
misc. 76
Data points 700

Alt #2 (4 loc.)

I. Abut. piles
  1) = 40 S.G.

II. Piers
  1) = 40 S.G.
  2) = 40 T.S.

III. Slab (loc. 1-4)
  1) 4 x 12 = 48 S.G.
  2) 4 x 20 = 80 T.S.

IV. Girders (loc 1-4)
  1) 4 x 24 = 96 S.G.
  2) 4 x 16 = 64 T.S.

Subtotal 184 T.S.
224 S.G.
misc. 42
Data points 450

Alt. # 3 (3 sec., 4 loc.)

I. Abut. piles
  1) = 40 S.G.

II. Piers
  1) = 40 S.G.
  2) = 40 T.S.

III. Slab (loc. 1-4)
  1) 4 x 9 = 36 S.G.
  2) 4 x 15 = 60 T.S.

IV. Girders (loc. 1-4)
  1) 4 x 18 = 72 S.G.
  2) 4 x 12 = 48 T.S.

Subtotal 148 T.S.
188 S.G.
misc. 14
Data points 350

Fig. 7. Distribution of proposed sensors for automatic recording

The data acquisition system should be housed in a controlled environment. It is recommended that a small house trailer be used for this purpose and placed under the bridge as seen in Fig. 8. It is important that the system be located as close to the center of the bridge as possible for minimum cable length. The instrumentation can be installed in the trailer and used at the laboratory site prior to construction, thus enabling familiarity with the system and the required laboratory evaluation of sensors and equipment. The trailer also provides some protection from vandalism, which can not be overemphasized in a project of this type.

Data Acquisition System

A data acquisition system is an electro-mechanical machine designed to sample and record data automatically. Two recommended data acquisition system configurations with a minicomputer are illustrated in Figs. 9 and 10, System Block Diagrams I and II, respectively. The system of Diagram I utilizes RTD's and Carlson strain meters to sense temperature and the system of Diagram II utilizes thermocouples and Carlson strain meters. Briefly, the operation of the system and the function of the system components may be explained as follows.

Referring first to System Block Diagram I, a Carlson strain meter or other strain gage senses a parameter by causing a change in resistance, ΔR. This ΔR is converted to an analog voltage by the Strain Gage Signal Conditioning unit. The analog voltage is then fed through the Analog Multiplexer to the measuring unit where it is converted to digital data.

The Minicomputer inputs the digital data and, as desired, makes performance checks, checks data for consistency, performs realtime analysis, provides data compression, converts data into engineering

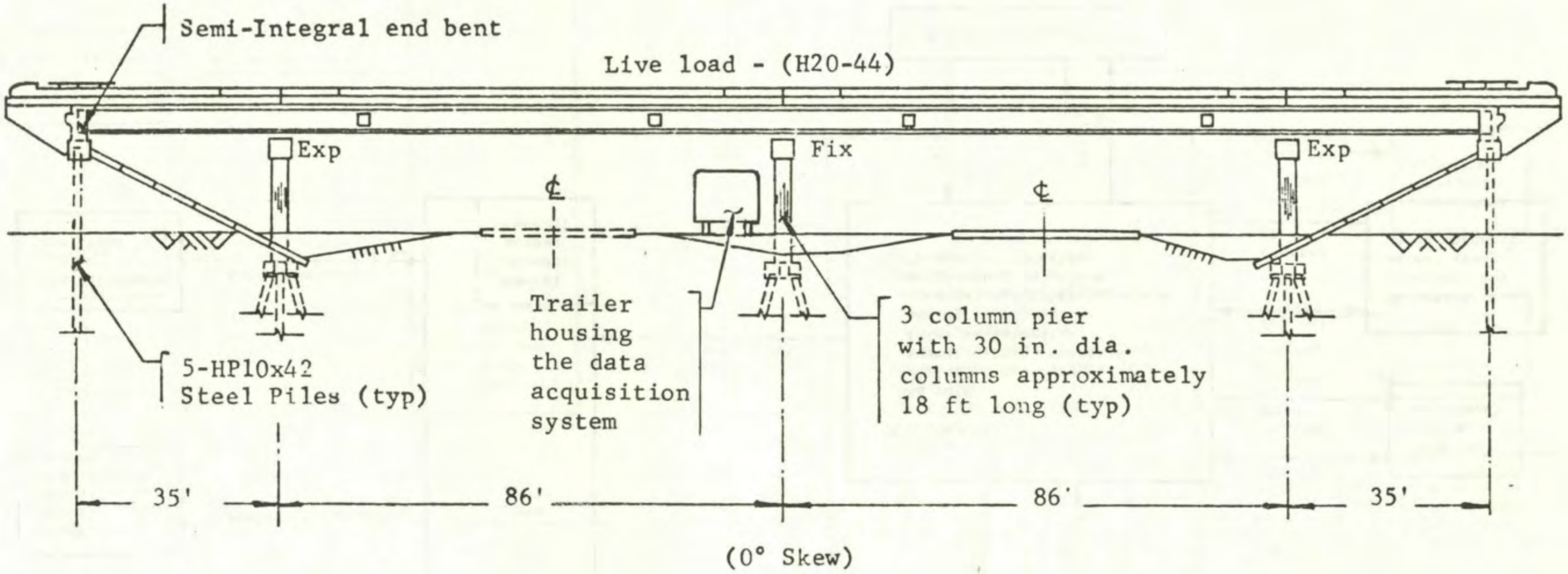
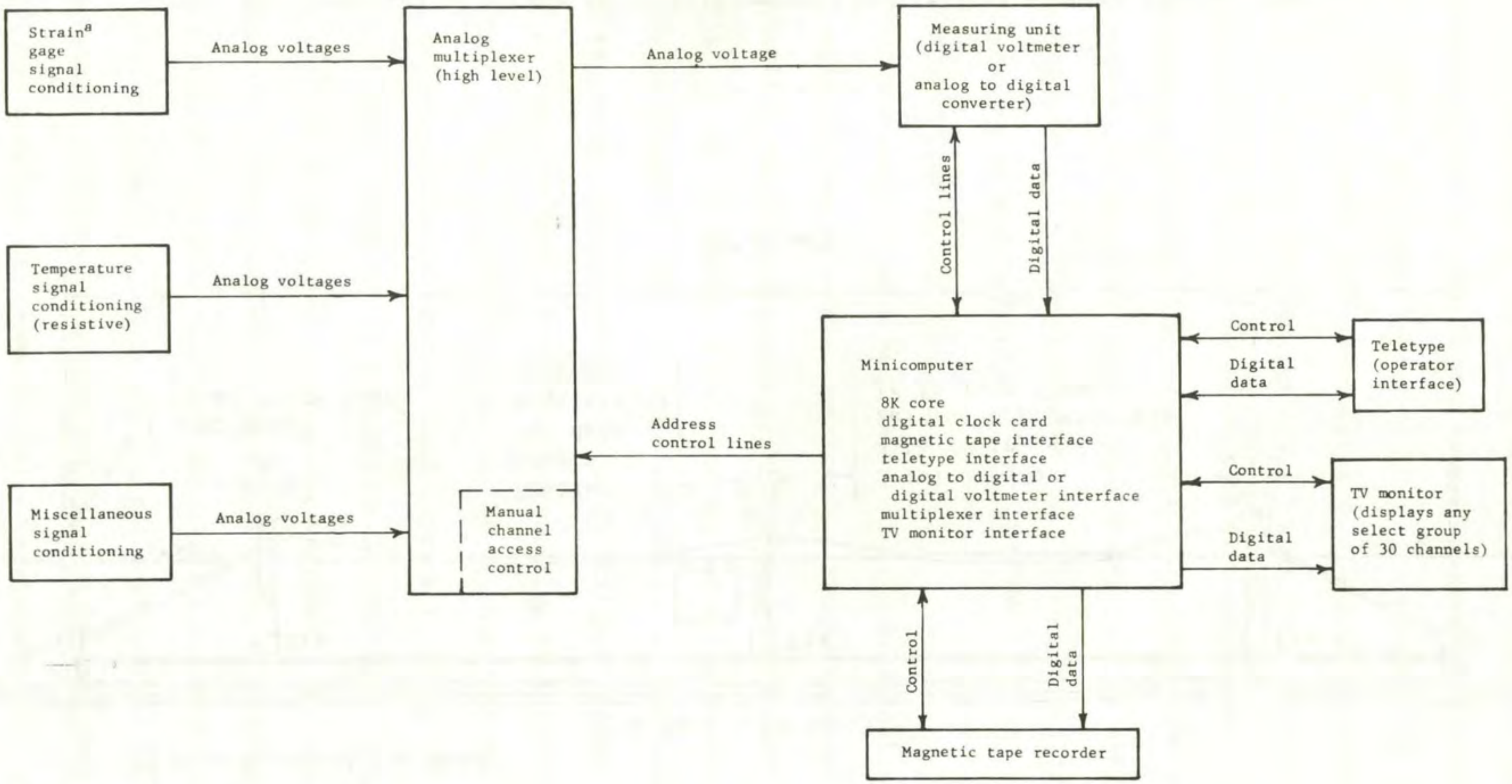
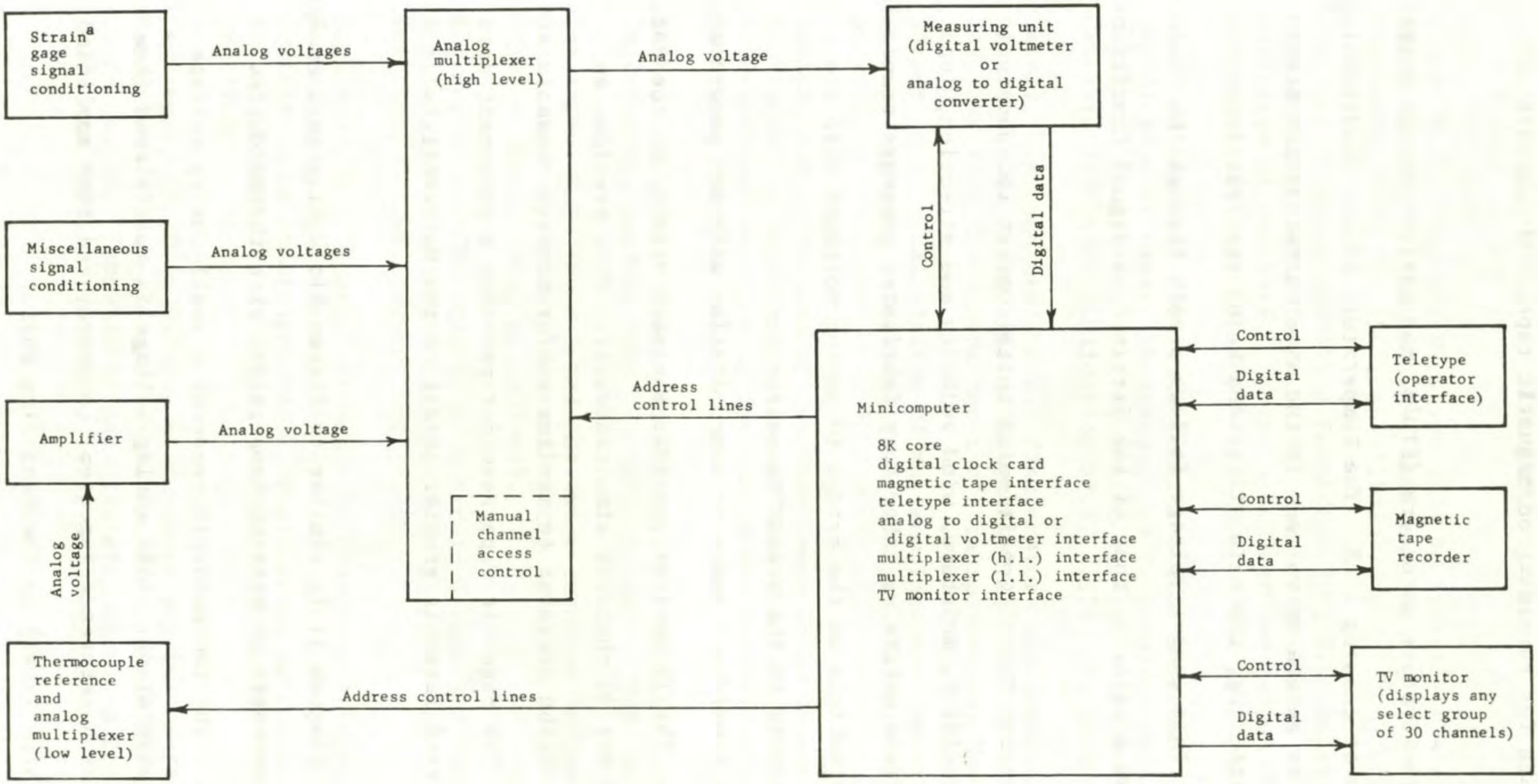


Fig. 8. Elevation of bridge to be instrumented and trailer housing instrumentation



[a]When used, Carlson strain meters measure both strain and temperature.

Fig. 9. System block diagram I



[a]When used, Carlson strain meters measure both strain and temperature.

Fig. 10. System block diagram II

units, formats data for recording on magnetic tape, and controls all system functions.

Resistance temperature detectors (RTD's) and Carlson strain meters sense temperature by causing a ΔR. The Temperature Signal Conditioning converts this ΔR to an analog voltage in the same manner as the Strain Gage Signal Conditioning, the main difference being the relative magnitude of ΔR. This analog voltage follows a path through the acquisition system similar to that of the Strain Gage Signal Conditioning unit.

The Miscellaneous Signal Conditioning units convert the physical measurements of humidity, moisture, wind velocity and direction, load cells, Carlson stress meters or laboratory fabricated pressure transducers, and presence of a vehicle on the bridge to analog voltages that are multiplexed and passed to the measuring unit.

The Teletype provides a means of communicating with and programming the minicomputer. The TV Monitor provides a visual display of the data being measured by any 30 channels simultaneously. This provides an excellent means for the operator to monitor and/or compare channels at the bridge site. The Magnetic Tape Recorder provides a permanent data record to be analyzed later in greater detail on the University's IBM 360/50.

System Block Diagram II is similar to System Block Diagram I except for the changes necessary to measure temperature with thermocouples rather than RTD's. The thermocouple produces a small analog voltage proportional to temperature. This analog voltage is multiplexed (low level) to a precision reference junction; compared; and then amplified and multiplexed (high level) to the Measuring Unit.

REFERENCES CITED

1. Aleck, B.J., "Thermal Stresses in a Rectangular Plate Clamped Along an Edge," *Transactions*, American Society of Mechanical Engineers, Vol. 71, 1949, pp. 118-122.

2. American Concrete Institute Committee 209, Subcommittee 1, "Effects of Concrete Constitutents, Environment, and Stress on the Creep and Shrinkage of Concrete," *Creep, Shrinkage and Temperature*, American Concrete Institute Publication, SP-27, 1971, pp. 1-42.

3. American Concrete Institute Committee 209, Subcommittee 2, "Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures," *Creep, Shrinkage and Temperature*, American Concrete Institute Publication SP-27, 1971, pp. 51-93.

4. Antoni, C.M., and Beal, D.B., "Temperature and Shrinkage Stresses in a Concrete Bridge Pier," *Research Report 69-6*, Research Project 321, New York State Department of Transportation, May, 1971.

5. Barber, E.S., "Calculation of Maximum Pavement Temperatures From Weather Reports," Highway Research Board, *Bulletin 168*, 1957, pp. 1-8.

6. Bazant, Z.P., "Effects of Concrete Creep under Repeated Stresses Superposed on Sustained Stresses," 8th Congress of the International Association for Bridge and Structural Engineers, New York, 1968, pp. 741-746.

7. Berks, C.J., "New Mechanical Bridge Joint System," *Civil Engineering*, London, Vol. 68, No. 741, Apr., 1968, pp. 439-440.

8. Berwanger, C., "Thermal and Shrinkage Stresses in Composite Beams," Discussion, by W. Zuk, *Journal of the American Concrete Institute*, Vol. 58, Mar., 1962, pp. 833-835.

9. Berwanger, C., "Bibliography on Strain Gauges and their Applications," *National Research Council*, Division of Building Research, Ottowa, Canada, Oct., 1968.

10. Berwanger, C., "Thermal Stresses in Composite Bridges," *Proceedings*, American Society of Civil Engineers, Specialty Conference on Steel Structures, Engineering Extension Series, No. 15, University of Missouri-Columbia, Columbia, Mo., Jun., 1970, pp. 27-36.

11. Berwanger, C., "The Modulus of Concrete and the Coefficient of Expansion of Concrete and Reinforced Concrete at Below Normal Temperatures," *Temperature and Concrete*, American Concrete Institute Publication SP-25, 1971, pp. 191-233.

12. Berwanger, C., "A Continuous Composite Steel-Concrete Bridge Prestressed by Deformations at the Interior Supports," *Second International Symposium on Concrete Bridge Design*, American Concrete Institute Publication SP-26, 1971, pp. 818-873.

13. Birkeland, H.W., "Differential Shrinkage in Composite Beams," *Proceedings*, American Concrete Institute, Vol. 56, No. 11, May, 1960, pp. 1123-1136.

14. Biswas, M., "Influence Lines for Thermal Loads in Structures," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 95, No. ST10, Proc. Paper 6806, Oct., 1969, pp. 2329-2334.

15. Black, W., and Adams, H.C., "Survey of Expansion Joints and Bearings in Britain," 7th Congress of the International Association for Bridge and Structural Engineers, Rio de Janeiro, Preliminary Publication, Vol. 1, 1964, pp. 1029-1039.

16. Boley, B.A., and Weiner, J.H., *Theory of Thermal Stresses*, John Wiley and Sons, Inc., New York, 1960.

17. Branson, D.E., "Time-Dependent Effects in Composite Concrete Beams," *Journal of the American Concrete Institute*, Vol. 61, No. 2, Feb., 1964, pp. 213-230.

18. Branson, D.E., and Metz, G.A., "Instantaneous and Time-Dependent Deflections of Simple and Continuous Reinforced Concrete Beams--Part I & II," *HPR Report No. 7*, State of Alabama Highway Department, Jun., 1965.

19. Branson, D.E., and Christiason, M.L., "Time Dependent Concrete Properties Related to Design-Strength and Elastic Properties, Creep and Shrinkage," *Creep, Shrinkage and Temperature*, American Concrete Institute Publication SP-27, 1971, pp. 257-277.

20. Callan, E.J., "Thermal Expansion of Aggregates and Concrete Durability," *Proceedings*, American Concrete Institute, Vol. 48, No. 6, Feb. 1952, pp. 485-505.

21. Capps, M.W.R., and Emerson, M., "The Thermal Behavior of Beachley Viaduct/Wye Bridge," *RRL Report No. 234*, Road Research Laboratory, Ministry of Transport, London, England, 1968.

22. Carslaw, H.S., and Jaeger, J.C., *Conduction of Heat in Solids*, 2nd ed., Oxford University Press, London, 1960, pp. 105-106.

23. Chow, D.Y.F., "Thermal Contraction and Moisture Creep in Concrete," Unpublished Master of Science thesis, McGill University Library, Montreal, Quebec, 1953.

24. David, R., and Meyerhof, G. G., "Composite Construction of Bridges Using Steel and Concrete," *Engineering Journal*, Canada, Vol. 41, No. 5, May, 1958, pp. 41-47.

25. Davis, R.E., "A Summary of the Results of Investigations Having to do With Volumetric Changes in Cements, Mortars and Concretes, Due to Causes Other Than Stress," *Proceedings*, American Concrete Institute, Vol. 26, 1930, pp. 407-443.

26. Ekberg, C.E., Jr., and Emanuel, J.H., "Current Design Practice for Bridge Bearing and Expansion Devices," *Final Report*, Project 547-S, Iowa State University, Engineering Research Institute, Aug., 1967.

27. Emanuel, J.H., and Eckberg, C.E., Jr., "Problems of Bridge Supporting and Expansion Devices and an Experimental Comparison of the Dynamic Behavior of Rigid and Elastomeric Bearings," *Special Report*, Project 547-S, Iowa State University, Iowa Engineering Experiment Station, Sep., 1965.

28. Emerson, M., "Bridge Temperatures and Movements in the British Isles," *RRL Report LR 228*, Road Research Laboratory, Ministry of Transport, London, England, 1968, pp. 1-25.

29. England, G.L., "Method of Estimating Creep and Shrinkage Strains in Concrete from Properties of Constituent Materials," *Journal of the American Concrete Institute*, Vol. 62, No. 11, Nov., 1965, pp. 1411-1420.

30. Eringen, A.C., "Thermal Stresses in Multiple Layer Beam," *Proceedings*, 1st Midwestern Conference on Solid Mechanics, University of Illinois, Apr., 1953, pp. 69-73.

31. Evans, R.H., and Kong, F.K., "Estimation of Creep of Concrete in R.C. and P.C. Design," *Civil Engineering and Public Works Review*, May 1966, pp. 593-596.

32. Freudenthal, A.M., and Roll, F., "Creep and Creep Recovery of Concrete under High Compressive Stress," *Proceedings*, American Concrete Institute, Vol. 54, 1958, pp. 1111-1142.

33. Gatewood, B.E., *Thermal Stresses*, McGraw-Hill, Inc., New York, 1957.

34. Gortz, W., Agnew, F.S., and Palmer, M.F., "Bridge Deck Expansion Joints," *Consulting Engineer*, London, Vol. 31, No. 5, May, 1967, pp. 76-78, 81.

35. "Growth Destroys a Bridge Deck," *Engineering News-Record*, Vol. 174, No. 7, Jun. 17, 1965, pp. 163-165.

36. Hanson, T.C., and Mattock, A.H., "Influence on the Size and Shape of Member on the Shrinkage and Creep of Concrete," *Journal of the American Concrete Institute*, Vol. 63, No. 2, Feb., 1966, pp. 267-290.

37. Hatt, W.K., "The Effect of Moisture on Concrete," *Transactions*, American Society of Civil Engineers, Vol. 89, 1926, pp. 270-307.

38. Hendry, A.W., and Page, J.K., "Thermal Movements and Stresses in Concrete Slabs in Relation to Tropical Conditions," Rilem *International Symposium on Concrete and Reinforced Concrete, Part 2*, Jul., 1960, pp. 1-26.

39. Holm, T.A., "Thermal Stresses Due to Unequal Thermal Coefficients of Concrete and Reinforcing Steel," *Civil Engineering*, American Society of Civil Engineers, Vol. 35, No. 2, Feb., 1965, pp. 63-65.

40. Hudson, F.M., "Investigation of a Full-Size Continuous Concrete Bridge--Part I & II," *HPR Report No. 24*, State of Alabama Highway Department, Jun., 1967.

41. *International Recommendations for the Design and Construction of Concrete Structures*, Comité Européen du Béton/Federatión Internationale de la Precontrainte, Paris, 1970. (English translation, Cement and Concrete Association, London, 1970, 80 pp.)

42. Jones, C.W., "Smoother Bridge Approaches," *Civil Engineering*, American Society of Civil Engineers, Vol. 29, No. 6, Jun., 1959, pp. 407-409.

43. King and Gavaris, "Movement Towards Its Backfill of Pile Supported Bridge Abutment," *Research Report*, New Jersey State Department of Transportation, Sep., 1970.

44. Kozlov, G.S., and Desai, D., "Preformed Elastomeric Bridge Joint Sealers: Thermal Characteristics of Bridge End Movements," *Highway Research Record No. 302*, Highway Research Board, 1970, pp. 38-48.

45. Krishnamurthy, N., "Temperature Effects on Continuous Reinforced Concrete Bridge," *HPR Report No. 58*, Research Project 930-047, State of Alabama Highway Department, Jul., 1971.

46. Lea, F.M., and Davey, N., "The Deterioration of Concrete in Structures," *Journal*, Institution of Civil Engineers, Vol. 32, No. 7, May, 1949, pp. 248-275.

47. Liu, Y.N., and Zuk, W., "Thermoelastic Effects in Prestressed Flexural Members," *Journal of Prestressed Concrete Institute*, Vol. 8, No. 3, Jun., 1963, pp. 64-85.

48. Lyse, I., "Shrinkage and Creep of Concrete," *Proceedings*, American Concrete Institute, Vol. 56, No. 8, Feb., 1960, pp. 775-782.

49. Maher, D.R.H., "The Effects of Differential Temperature on Continuous Prestressed Concrete Bridges," *Civil Engineering Transactions*, The Institution of Engineers, Australia, Vol. CE12, No. 1, Paper 2793, Apr., 1970, pp. 29-32.

50. Martin, I., "Effect of Environmental Conditions on Thermal Variations and Shrinkage of Concrete Structures in the United States," *Creep, Shrinkage and Temperature*, American Concrete Institute Publication SP-27, 1971, pp. 279-300.

51. Matlock, H., and Reese, C., "Foundation Analysis of Offshore Pile Supported Structures," *Proceedings*, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. II, 1961, pp. 91-97.

52. Matsumoto, T., "A Study of the Effect of Moisture Content Upon the Expanson and Contraction of Plain and Reinforced Concrete," Engineering Experiment Station, *Bulletin 126*, Illinois University, 1921.

53. Mercer, L.B., "Volume Changes Resulting from Moisture Variations in Set Concrete," *Commonwealth Engineer*, Vol. 37, No. 11, Jun., 1950, pp. 453-456.

54. Mitchell, L.J., "Thermal Expansion Tests on Aggregates, Neat Cement, and Concretes," *Proceedings*, American Society for Testing Materials, Vol. 53, 1953, pp. 963-977.

55. Naruoka, M., Hirai, I., and Yamaguti, T., "The Measurement of the Temperature of the Interior of the Reinforced Concrete Slab of the Shigita Bridge and Presumption of Thermal Stress," *Proceedings*, Symposium on the Stress Measurements for Bridge and Structures, Japanese Society for the Promotion of Science, Tokyo, Japan, 1957, pp. 106-115.

56. Neville, A.M., and Meyers, B.L., "Creep on Concrete--Influencing Factors and Prediction," *Symposium on Creep of Concrete*, American Concrete Institute, Publication SP-9, Paper 1, 1964, pp. 1-33.

57. Neville, A.M., and Staunton, M.M., "Method of Estimating Creep of Concrete When the Stress-Strength Ratio Varies with Time," *Journal of the American Concrete Institute*, Vol. 62, No. 10, Oct., 1965, pp. 1293-1312.

58. Newmark, N.M., "Numerical Procedure for Computing Deflections, Moments, and Buckling Loads," *Transactions*, American Society of Civil Engineers, Vol. 108, pp. 1161-1234.

59. Nieu, N.D., Antes, D.R., and Kessler, R.S., "Field Measurements on Instrumented Piles Under an Overpass Abutment," *Highway Research Record No. 354*, Highway Research Board, 1971, pp. 90-102.

60. Oleson, C.C., "Abnormal Cracking in Highway Structures in Georgia and Alabama," *Journal of the American Concrete Institute*, Vol. 60, No. 3, 1963, pp. 329-353.

61. Pauw, A., "Time Dependent Deflections of a Box Girder Bridge," *Creep, Shrinkage, and Temperature*, American Concrete Institute Publication SP-27, 1971, pp. 141-158.

62. Pauw, A., and Meyers, B.L., "Effect of Creep and Shrinkage on Behavior of Reinforced Concrete Members," *Symposium on Creep of Concrete*, American Concrete Institute, Publication SP-9, Paper 6, 1964, pp. 129-158.

63. Peck, R.B., and Ireland, H.O., "Backfill Guide," *Journal of the Structural Division*, American Society of Civil Engineers, Vol. 83, No. ST4, Proc. Paper 1321, Jul., 1957, pp. 1-10.

64. *Pile Foundation Know How*, American Wood Preservers Institute, Feb., 1969.

65. Rawlinson, J., and Stott, P.F., "The Hammersmith Flyover," *Proceedings*, Institute of Civil Engineers, London, Vol. 23, Dec., 1962, pp. 565-600.

66. Reynolds, J.C., "Thermal Stresses and Movements in Bridges," Master of Science thesis, University of Missouri-Rolla, Rolla, Mo., 1972.

67. Reynolds, R.J., and Gamble, W.L., "Field Investigation of Prestressed Reinforced Concrete Highway Bridges," Department of Civil Engineering, *Structural Research Series 327*, Illinois University, Oct., 1967.

68. Roll, F., "Effects of Differential Shrinkage and Creep on a Composite Steel-Concrete Structure," *Creep, Shrinkage and Concrete*, American Concrete Institute Publication SP-27, 1971, pp. 187-214.

69. "Room For Improvement," *Engineering News-Record*, Vol. 164, No. 5, Feb. 4., 1960, p. 100.

70. Ross, A.D., "Creep of Concrete Under Variable Stress," *Proceedings*, American Concrete Institute, Vol. 54, 1958, pp. 739-758.

71. *Standard Specifications for Highway Bridges*, 10th ed., American Association of State Highway Officials, Washington, D.C., 1969.

72. Stephenson, D.A., "Effects of Differential Temperature on Tall Slender Columns," *Concrete and Constructional Engineering*, Vol. 56, No. 5, May, 1961, pp. 175-178, and correspondence by A.J. Ashdowin on this paper and the author's reply, *Concrete and Constructional Engineering*, Vol. 56, No. 11, Nov., 1961, pp. 401-403.

73. Stermac, A.D., Devata, M., and Selby, K.G., "Unusual Movements of Abutments Supported on End-Bearing Piles," *Canadian Geotechnical Journal*, Vol. 5, No. 2, May, 1968, pp. 69-70.

74. Steward, C.F., "Annual Movement Study of Bridge Deck Expansion Joints," California Division of Highways, *R & D Report No. 2-69*, Jun., 1969.

75. Szidarovszky, J., "Examination of the Effect of Temperature Gradients by Means of Influence Lines," *Acta Technica*, Budapest, Vol. 56, No. 1-2, 1967, pp. 145-154.

76. Terzaghi, Karl, "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, Vol. 5-6, 1955-56, pp. 297-326.

77. Timoshenko, S., and Goodier, *Theory of Elasticity*, McGraw-Hill, New York, 1951.

78. Tyler, R.G., "Developments in the Measurement of Strain and Stress in Concrete Bridge Structures," *RRL Report LR 189*, Road Research Laboratory, Ministry of Transport, London, 1968, pp. 1-20.

79. Tyler, R.G., "An Interpretation of the Results from Vibrating-Wire Strain Gauges in Fresh Concretes," *Highway Research Record No. 218*, Highway Research Board, 1968, pp. 1-17.

80. Tyler, R.G., "Determining Stress in Concrete Structures," *Civil Engineering and Public Works Review*, London, Vol. 64, No. 755, Jun., 1969, pp. 588-591, No. 756, Jul., 1969, pp. 679-682.

81. Verbeck, G.J., and Hass, W.E., "Dilatometer Method for Determination of Thermal Coefficient of Expansion of Fine and Coarse Aggregate," *Proceedings*, Highway Research Board, Vol. 30, 1951, pp. 187-193.

82. Wah, T., and Kirksey, R.E., "Thermal Characteristics of Highway Bridges," Southwest Research Institute, *Final Report, No. HR 12-4*, Jul., 1969.

83. Walker, S., Bloem, D.L., and Mullen, W.C., "Effects of Temperature Changes on Concrete as Influenced by Aggregates," *Proceedings*, American Concrete Institute, Vol. 48, No. 8, Apr., 1952, pp. 661-679.

84. Wallo, E.M., and Kesler, C.E., "Prediction of Creep in Structural Concrete," Engineering Experiment Station, *Bulletin 498*, Illinois University, 1968.

85. Winter, G., *et al.*, *Design of Concrete Structures*, 7th Ed., McGraw-Hill, Inc., New York, 1964.

86. Witecki, A.A., and Raina, V.K., "Distribution of Longitudinal Horizontal Forces Among Bridge Supports," *1st International Symposium on Concrete Bridge Design*, American Concrete Institute Publication SP-23, 1969, pp. 803-815.

87. Wroth, C.P., "The Hammersmith Flyover-Site Measurements of Prestressing Losses and Temperature Movement," *Proceedings*, Institution of Civil Engineers, Vol. 23, Paper No. 6632, Dec., 1962, pp. 601-624.

88. Zederbaum, J., "The Frame Action of a Bridge Deck Supported on Elastic Bearings," *Civil Engineering and Public Works Review*, London, Vol. 61, No. 714, Jan., 1966, pp. 67-69, 71-72.

89. Zederbaum, J., "Factors Influencing the Longitudinal Movement of a Concrete Bridge System With Special Reference to Deck Contraction," *1st International Symposium on Concrete Bridge Design*, American Concrete Institute Publication SP-23, 1969, pp. 75-95.

90. Zuk, W., "Thermal and Shrinkage Stresses in Composite Beams," *Journal of the American Concrete Institute*, Vol. 58, No. 3, Sept., 1961, pp. 327-340.

91. Zuk, W., "Simplified Design Check of Thermal Stresses in Composite Highway Bridges," Highway Research Board (to be published as HRB Bulletin), Jan., 1965.

92. Zuk, W., "Thermal Behavior of Composite Bridges--Insulated and Uninsulated," *Highway Research Record No. 76*, Highway Research Board, 1965, pp. 231-253.

93. Zuk, W., "Simplified Design Check of Thermal Stresses in Composite Highway Bridges," *Highway Research Record, No. 103*, Highway Research Board, 1965, pp. 10-13.

94. Zuk, W., "End Movement Studies of Various Type Highway Bridges," *Virginia Research Council*, Charlottesville, Aug., 1968.

95. Zuk, W., "End Movement Studies of Various Type Highway Bridges," *Highway Research Record, No. 295*, Highway Research Board, 1969, pp. 1-4.

APPENDIX A

QUESTIONNAIRE AND SUMMARY OF REPLIES TO QUESTIONNAIRE

University of Missouri - Rolla

CIVIL ENGINEERING

Civil Engineering Bldg.
Rolla, Mo. 65401

Telephone
314 341-4461

A study of design criteria for stresses induced by semi-integral end bents is being conducted as a Missouri Highway Department Cooperative Research Study by the Department of Civil Engineering, University of Missouri-Rolla. The long-range goal of the study is to develop design criteria for bridges whose superstructures are supported by flexible substructures.

The use of flexible piers and abutments without expansion type supporting devices has become generally accepted and is being used for quite long superstructures. However, opinions vary among bridge design engineers as to design limitations and how to determine and provide for the stresses induced in the structure as a result of this method of construction. Apparently, there is no rational method for handling this problem and it is believed that perhaps no allowance is made for these stresses in some cases.

For practical knowledge and to enlarge our findings, we are sending the enclosed short questionnaire to state highway bridge engineers. The purpose of the questionnaire is three-fold: to help define the parameters of the problem; to review and clarify current design practice; and to aid the course of the investigation.

Although we are concentrating on continuous composite steel superstructures with end diaphragms semi-integral with the abutment, any additional comments concerning procedures used by your organization for handling induced stresses, design details used, and suggestions toward our investigation will be greatly appreciated. Sketches defining the terms non-integral, semi-integral and integral as used in this study are enclosed.

Sincerely yours,

Jack H. Emanuel
Associate Professor of
Civil Engineering

JHE:pjs

Enclosure

QUESTIONNAIRE: Design Criteria for Stresses Induced by Semi-Integral End Bents

1. Do you approve the use of flexible stub abutments and piers without expansion or rocker type supporting devices; i.e., tying the girders directly to the abutments and piers

   a) for steel structures Yes___ No___ b) for concrete structures? Yes___ No___

2. Are you using, or have you used, the above method

	I. Non-Integral		II. Semi-Integral		III. Integral	
a) for steel structures	Yes___	No___	Yes___	No___	Yes___	No___
b) for concrete structures?	Yes___	No___	Yes___	No___	Yes___	No___

3. If any of the answers to 2 above was <u>Yes</u>, what limitations, if any, are imposed (I, II, or III)

   a) for steel structures______________________________

   b) for concrete structures?______________________________

   What is the maximum overall expansive length used (without positive expansion devices)

	I. Non-Integral	II. Semi-Integral	III. Integral
c) for steel structures	________	________	________
d) for concrete structures?	________	________	________

   When used for one, two, three, and four-span structures, what are typical span lengths

		one	two	three	four
e) for steel structures	I	___;	___, ___;	___, ___, ___;	___, ___, ___, ___
	II	___;	___, ___;	___, ___, ___;	___, ___, ___, ___
	III	___;	___, ___;	___, ___, ___;	___, ___, ___, ___
f) for concrete structures?	I	___;	___, ___;	___, ___, ___;	___, ___, ___, ___
	II	___;	___, ___;	___, ___, ___;	___, ___, ___, ___
	III	___;	___, ___;	___, ___, ___;	___, ___, ___, ___

   Are you using the method for both non-skewed and skewed structures, and, if using for skewed, are additional limitations imposed

   g) for steel structures Non-skewed only___ both, 0 to 15^{o} skew___ both, 15 to 30^{o} skew___
   Additional limitations______________________________

   h) for concrete structures? Non-skewed only___ both, 0 to 15^{o} skew___ both, 15 to 30^{o} skew___
   Additional limitations______________________________

   In your design do you take <u>induced</u> <u>stresses</u> into consideration--e.g., due to thermal effects, creep, shrinkage, backfill movement and settlement, etc.,

   i) for steel structures Yes___, No___, if <u>Yes</u>, what types of stresses--i.e., due to___________
   ______________________________ How? ______________________________

   j) for concrete structures? Yes___, No___, if <u>Yes</u>, what types of stresses--i.e., due to___________
   ______________________________ How? ______________________________

In your design (non-integral construction) do you provide for (allow) girder end rotation

k) for steel structures Yes___, No___, if Yes, how? (e.g., curved steel plates)______________

l) for concrete structures? Yes___, No___, if Yes, how? (e.g., curved steel plates)______________

If girder end rotation--separate from joint rotation--is not allowed (integral or semi-integral construction) do you assume and take into consideration joint rotation due to flexing of the abutment piling and pier piling

m) for steel structures ______________

n) for concrete structures? ______________

4. What limitations do you impose with respect to flexibility and type of substructure--e.g., pile materials, L/r ratios, cast in place substructures on spread footing, backfilling techniques, preboring, etc., for integral or semi-integral type of construction

a) for steel structures ______________

b) for concrete structures? ______________

5. What objections, if any, do you have to the above method referred to in questions 1 and 2.

a) for steel structures ______________

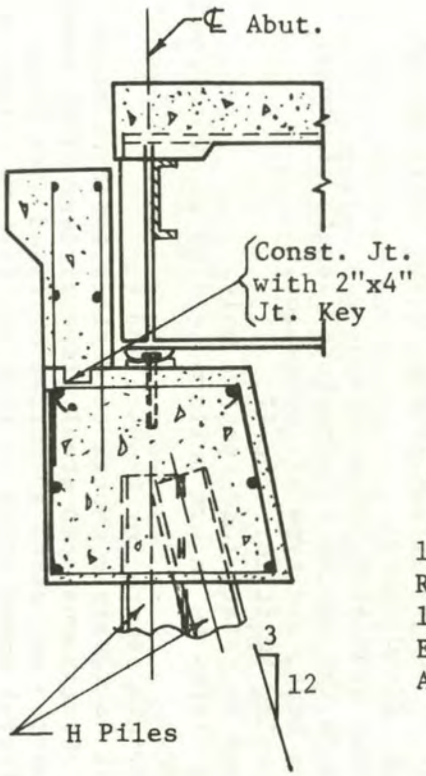
b) for concrete structures? ______________

6. What problems, if any, have you encountered when the method referred to in questions 1 and 2 was used

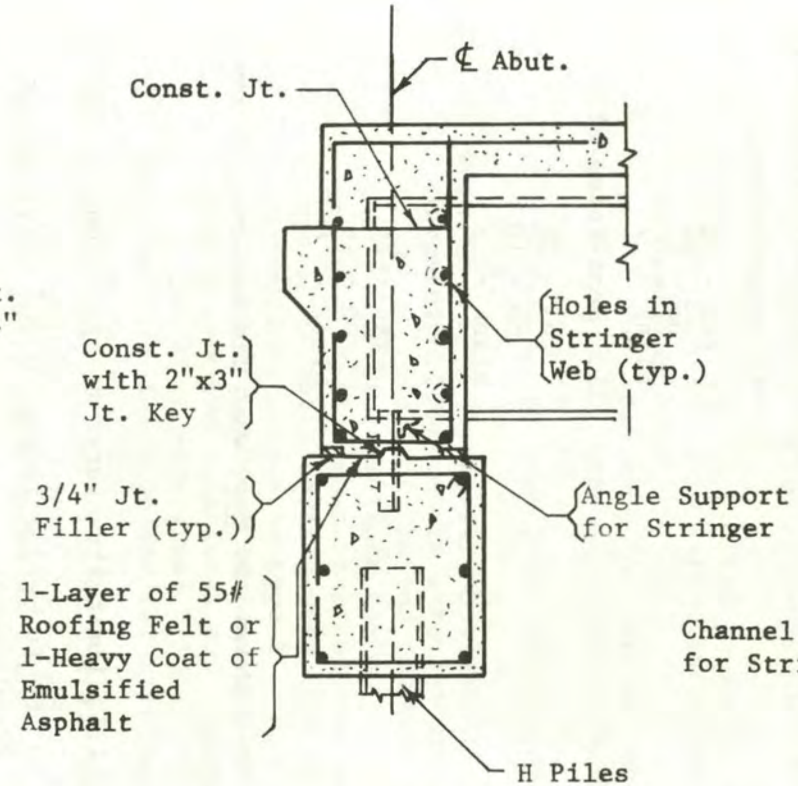
a) for steel structures ______________

b) for concrete structures? ______________

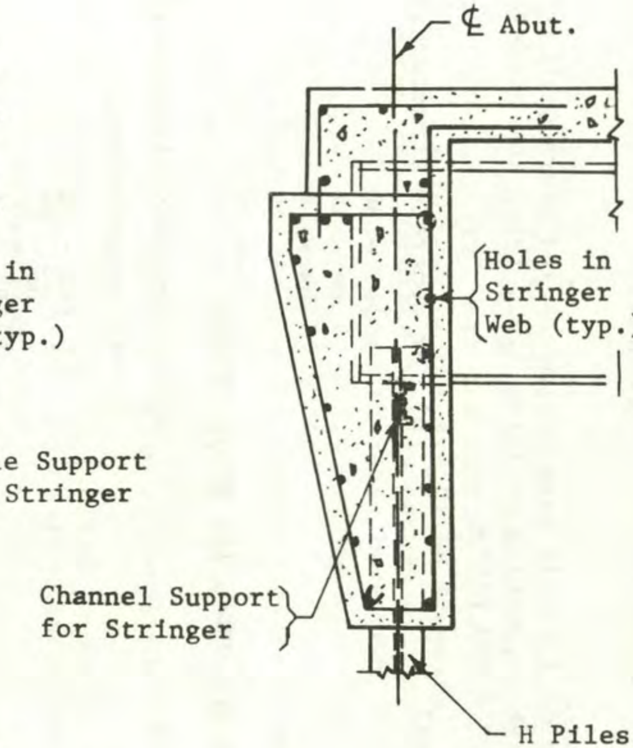
7. Additional comments and suggestions



NON-INTEGRAL



SEMI-INTEGRAL



INTEGRAL

Note: Slab reinforcement and shear connectors have been left off for clarity.

Summary of Replies to Questionnaire

1. Do you approve the use of flexible stub abutments and piers without expansion or rocker type supporting devices; i.e., tying the girders directly to the abutments and piers

a) for steel structures?

yes - 23
no - 16
blank - 7

b) for concrete structures?

yes - 30
no - 10
blank - 6

2. Are you using, or have you used, the above method

a) for steel structures

I. Non-Integral

yes - 28
no - 10
blank - 8

II. Semi-Integral

yes - 17
no - 20
blank - 9

III. Integral

yes - 13
no - 25
blank - 8

b) for concrete structures?

I. Non-Integral

yes - 26
no - 13
blank - 7

II. Semi-Integral

yes - 28
no - 10
blank - 8

III. Integral

yes - 24
no - 16
blank - 6

3. If any of the answers to 2 above was <u>Yes</u>, what limitations, if any, are imposed (I, II, or III)

a) for steel structures?

12 For types II and III, limit structures to approximately 200 ft length.
14 Type II limited to 200 ft bridge length, 0-20° skew; 160 ft bridge length, 20-30° skew.
16 Type II must have provision for rotation.
20 Limitations as to the length of structure, length of wings, height of end bent, and type of foundation.
25 Spans not exceeding 40 ft for type II.
26 The beam is free to rotate (use type II only).
27 We use bearing plates to insure uniform bearing and rotation for dead load.
35 250 ft total bridge length for types I and II.
37 Pier bents must be flexible enough to deflect for continuous structures. Continuous span steel plate girders or WF beams with cantilevered end spans supported on fixed pier bents with non-integral connections to them are free to rotate on

Question 3(a) continued.

the curved plate bearings. Piers are designed to deflect due to thermal and shrinkage stresses from the superstructure. Curtain walls at the ends of the cantilevered end spans are integral with the superstructure and are not supported by a foundation but are "free floating". Earth backfill pressure and concrete pouring sequence is considered. To date, no flexible abutment bents have been used, but they are being considered on a project under design.

41 Spans less than 45 ft.

42 Type I used for two adjacent piers fixed (part of a 3 to 7-span structure).

44 At present we are trying balanced 2-span continuous bridge $\leq$ 250 ft.

46 150 ft of expansion to abutment (300 ft maximum bridge length), 10° maximum skew.

47 Types I, II, and III limited to 300 ft with 30° skew.

48 Type III limited to a maximum structure length of 300 ft and a maximum skew of 30°.

51 Type III used only once for special application.

52 None for type I. Expansion device required at intermediate bents for type II.

53 No limitations on type I for short spans or one span of a series.

54 When this type (integral) abutment is used, care is taken to assure that temperature stresses are not excessive. For type I, the stiffness of the end abutment must be low enough so that temperature motions can occur at the abutment or else expansion motion must be accounted for in the adjacent piers. Type II is not generally used, but is subject to the same general limitations as type I. Type III has not been used for steel structures, largely because of problems associated with girder end rotations, and also because this State uses very few short span steel bridges.

55 Two rows of piles required for type I. Types II and III limited to 150 ft.

56 Types II and III not used at piers. Type I used on flexible piers only.

62 Used on short spans where deflections are small. Provide for elastic shortening due to post tensioning in the abutment design for spans approximately 115 ft and greater. Consider rotation when abutment span exceeds 160 ft. Assume a design longitudinal force of 15 to 25 kips per pile applied at the base of the diaphragm, depending upon pile type.

63 Types II and III used for short single spans only--50 ft $\pm$ maximum.

67 Type II limited to low skew bridges only.

Question 3 continued.

b) for concrete structures?

11 Type II limited to short spans (< 40 ft) and no shear key used.
12 Use type III on structures up to 500 ft length, then use type I.
14 For prestressed, use type I if over 30° skew; otherwise somewhat longer spans than for steel (limits given for steel were that type II is limited to 200 ft bridge length, 0-20° skew; 160 ft bridge length, 20-30° skew).
15 We use this type construction on rigid frame structures only.
16 Type II must have provision for rotation. Type III is used for slab spans only.
20 Limitations as to the length of structure, length of wings, height of end bent, and type of foundation.
24 Foundations bearing on piles for types II and III.
25 Spans not exceeding 40 ft for type II.
26 The beam is free to rotate (use type II only).
27 Structure length and foundation conditions.
41 Slab construction less than 100 ft.
42 Types II and III limited to thermal movement less than 1 in.
43 Used for prestressed concrete beam bridges and for slab bridges.
47 Types I, II, and III limited to 300 ft with 30° skew.
48 For type III, no limitations within practical length limits for type of bridge. Type II is no longer used.
49 Overall structure length and skew angle.
51 Types II and III used only where total bridge length is less than 300 ft.
52 Type II limited to 300 ft of structure without expansion devices.
54 When this type (integral) abutment is used, care is taken to assure that temperature stresses are not excessive. For type I, the stiffness of the end abutment must be low enough so that temperature motions can occur at the abutment or else expansion motion must be accounted for in the adjacent piers. Type II is not generally used, but is subject to the same general limitations as type I. Type III is largely restricted to flat slab or box girder construction of short to medium span. Care is taken to investigate the effect of the pile fixity on the structure.
55 Types II and III limited to structures less than 150 ft.
61 Types II and III not used with skew angles over 30°.
62 Provide for elastic shortening due to post tensioning in the abutment design for spans approximately 115 ft and greater. Consider rotation when abutment span exceeds 160 ft. Assume a design longitudinal force of 15 to 25 kips per pile applied at the base of the diaphragm, depending upon pile type.
63 Type III limited to 300 ft $\pm$ for 2 and 3-span structures; somewhat longer structures used with type II.
64 Length of structure and area in which structure is located.
65 Type III generally limited to short bridges (less than 100 ft).
66 Skews limited to 30°.
67 Type II limited to low skew bridges only.
68 For type III, superstructure length allowed is based upon flexibility of substructure.

Question 3 continued.

What is the maximum overall expansive length used (without positive expansion devices)

c) for steel structures?

I. Non-Integral	II. Semi-Integral	III. Integral
0- 40 - 0	0- 40 - 1	0- 40 - 0
41-100 - 6	41-100 - 4	41-100 - 2
101-200 - 5	101-200 - 3	101-200 - 7
201-300 - 1	201-300 - 2	201-300 - 2
>200 - 1	<200 - 1	<200 - 1
250 & 522 - 1	736 - 1	blank - 34
671 - 1	blank - 34	
blank - 31		

d) for concrete structures?

I. Non-Integral	II. Semi-Integral	III. Integral
0- 40 - 0	0- 40 - 2	0- 40 - 0
41-100 - 4	41-100 - 4	41-100 - 4
101-200 - 4	101-200 - 3	101-200 - 6
201-300 - 3	201-300 - 8	201-300 - 7
>500 - 1	325-350 - 2	350-400 - 2
blank - 34	450 - 1	450 - 2
	<500 - 1	<500 - 1
	blank - 25	blank - 24

When used for one, two, three, and four-span structures, what are typical span lengths

e) for steel structures?

I.	one	two	three	four
12	50;	80, 80;	100, 125, 100;	100, 125, 125, 100
14		75, 75;	35, 45, 35;	35, 45, 45, 35[a]
20	100;			
26		120, 120;		
37				155, 225, 165, 126
42	100;			
44		125, 125;		
47		117, 117;	77, 100, 77;	37, 95, 95, 37
49	150;			
51	137;	105, 105;	112, 127, 112;	59, 133, 133, 59
53	100;	Expansion device used.		
55		100, 100;	51, 66, 51;	91, 100, 100, 91
62	Various combinations--within allowable overall maximum.			
65	85;	Use positive expansion devices for multiple spans.		
67	100;	100, 100;	75, 100, 75;	75, 100, 100, 75

[a]For any skew.

Question 3(e) continued.

II.	one	two	three	four
12	50;	80, 80;	60, 80, 60;	40, 60, 60, 40
14		100, 100;	60, 80, 60;	44, 56, 56, 44[a]
14		80, 80;	48, 64, 48;	35, 45, 45, 35[b]
16		Expansion provided--usually at first interior bent.		
25			40, 40, 40;	40, 40, 40, 40[c]
37				184, 184, 184, 184
41	45;			
55			43, 57, 43;	
56	<90;			
62	Various combinations--within allowable overall maximum.			
63	50;			
65		Use positive expansion devices for multiple spans.		
67	100;	70, 70;	75, 100, 75;	55, 70, 70, 55

[a] 0-20° skew (all values).

[b] 20-30° skew (all values).

[c] Constructed as simple spans (all values).

III.	one	two	three	four
12	50;	80, 80;	60, 80, 60;	40, 60, 60, 40
16		Expansion provided--usually at first interior bent.		
37			34, 132, 34;	
44				Please see below[d]
46	100;	100, 100;	70, 80, 70;	60, 90, 90, 60
48	100;		87, 112, 87;	55, 70, 70, 55
52			59, 124, 59;	
55		45, 45;	43, 57, 43;	
56	<90;			
62	Various combinations--within allowable overall maximum.			
63	50;			
65		Use positive expansion devices for multiple spans.		

[d] Have used nine spans at 58 ft each.

f) for concrete structures?

I.	one	two	three	four
12	50;	80, 80;	100, 125, 100;	100, 125, 125, 100
13			65, 65, 65;	
24	50;	70, 70;	65, 70, 65;	
26		65, 65;	70, 70, 70;	70, 70, 70, 70
42	45;		30, 40, 30;	
44			28, 35, 28;	
47			67, 66, 67;	47, 68, 68, 47
51	79;	130, 130;		
62	Various combinations--within allowable overall maximum.			
65	50;	Use positive expansion devices for multiple spans.		
66		89, 107;	35, 85, 35;	
67	100;	100, 100;	75, 100, 75	75, 100, 100, 75

Question 3(f) continued.

II.	one	two	three	four
11	40 ft maximum.			
12	50;	80, 80;	100, 125, 100;	100, 125, 125, 100
13			50, 50, 50;	
20			90, 130, 90;	46, 57, 57, 46
24	50;	70, 70;	65, 70, 65;	
25			40, 40, 40;	40, 40, 40, 40[a]
41		30, 30;	25, 35, 25;	
42	20;		30, 40, 30;	
49	90;	80, 80;	70, 70, 70;	
51		130, 130;	41, 51, 41;	
52	100;	60, 60;	35, 70, 35;	50, 100, 100, 50
53	100;	100, 100;	100, 100, 100;	80, 100, 100, 80
54	45;		30, 44, 30;	
55			37, 56, 37;	
56	70;	70, 70;	70, 70, 70;	70, 70, 70, 70
62	Various combinations--within allowable overall maximum.			
63	80;	120, 120;	80, 100, 80;	60, 80, 80, 60
65		Use positive expansion devices for multiple spans.		
66			30, 40, 30;	
67		75, 75;	75, 100, 75;	60, 75, 75, 60

[a]Constructed as simple spans (all values).

III.	one	two	three	four
12	50;	80, 80;	100, 125, 100;	100, 125, 125, 100
14				44, 66, 66, 44[b]
14			50, 50, 50;[c]	
14		41, 41;	52, 68, 52;	56, 72, 72, 56[c]
15	50;	50, 50;		
16			30, 45, 30;	
20			55, 104, 55;	50, 60, 60, 50
24	50;	70, 70;	65, 70, 65;	
42	60;	45, 45;	30, 40, 30;	50, 50, 50, 50
43				40, 90, 90, 40
48	50;		60, 75, 60;	84, 84, 84, 84
51			36, 120, 51;	
53	100;	100, 100;	100, 100, 100;	80, 100, 100, 80
54	33;		61, 105, 61;	24, 30, 30, 24[d]
55		50, 50;	21, 31, 21;	
56		125, 135;		
61	150;	125, 125;		
62	Various combinations--within allowable overall maximum.			
63	80;	120, 120;	80, 100, 80;	60, 80, 80, 60
65	60;	Use positive expansion devices for multiple spans.		
68	50;	60, 60;	60, 80, 60;	30, 35, 35, 30

[b]0-30° skew and prestressed.

[c]Any skew and other than prestressed (all values).

[d]Have used six spans--37, 48, 48, 48, 48, 37.

Question 3 continued.

Are you using the method for both non-skewed and skewed structures, and, if using for skewed, are additional limitations imposed

g) for steel structures?		h) for concrete structures	
Non-skewed only	3	Non-skewed only	2
Both, 0 to 15° skew	5	Both, 0 to 15° skew	6
Both, 15 to 30° skew	15	Both, 15 to 30° skew	19
Blank	23	Blank	19

	Additional limitations (or comments)	Additional limitations (or comments)
14	30° maximum skew.	30° maximum skew for integral prestressed; otherwise no limitation.
16		Type III to about 15° only.
20	Lateral component of earth pressure considered.	Matter of judgment
26	Slots in bearings for expansion.	Slots in bearings for expansion.
42		Substructures limited to pile bents.
46	10° maximum skew.	
48	300 ft maximum length.	Skews to 45° on slab bridges; to 30° on prestressed.
49		Used only where skew is 20° or less.
51		Used only for skewed structures less than 80 ft.
62	In skewed structures having internal hinges, the piling is battered.	In skewed structures having internal hinges, the piling is battered.
63	Single spans only, 50 ft ± maximum.	
67	Bridge width vs. span ratio, number of piers, size, etc.	Bridge width vs. span ratio, number of piers, size, etc.

In your design do you take <u>induced stresses</u> into consideration--e.g., due to thermal effects, creep, shrinkage, backfill movement and settlement, etc.,

i) for steel structures?		j) for concrete structures?	
Yes	18	Yes	19
No	11	No	12
Blank	17	Blank	15

Questions 3(i) and (j) continued.

	If Yes, what types of stresses--i.e., due to (a) How? (b)	If Yes, what types of stresses--i.e., due to (a) How? (b)
11	a) Provide rockers.	
12	a) Primarily thermal and fill movement; b) use additional reinforcing steel.	a) Primarily thermal and fill movement; b) use additional reinforcing steel.
13	a) Temperature and shrinkage; b) column deflection.	a) Temperature and shrinkage; b) column deflection.
14	a) Thermal movement; b) bearing design and possibly column design.	a) Thermal, creep, shrinkage, and settlement; b) refer to PCA "Design of Continuous Highway Bridges with Precast, Prestressed Concrete Girders".
20	a) Passive earth pressure from expansion; pier deflection; b) reinforce accordingly.	a) Passive earth pressure from expansion; pier deflection; b) reinforce accordingly.
27	a) Thermal effects; b) moments induced by thermal effects are accounted for based on "E" of concrete equal to one-thirtieth that of steel.	a) Thermal effects; b) moments induced by thermal effects are accounted for based on "E" of concrete equal to one-thirtieth that of steel.
37	a) Thermal effects, shrinkage, and backfill pressures.	
42	a) Thermal effects; b) assume pier takes full deflection.	
44	a) Thermal effect; b) 12 in. C.I.P. piles are designed to take bending due to superstructure moment.	a) Thermal effect; b) allow flexing in bearing pad.
47	a) Thermal effects; b) using 1000 psf on abutments and wingwalls.	a) Thermal effects; b) using 1000 psf on abutments and wingwalls.
48	a) Temperature and backfill pressure; b) limit maximum stress in bottom flange to 90 percent of allowable.	
52	a) Thermal effects; b) expansion devices.	a) Thermal effects; b) expansion devices.
53	a) Thermal; b) calculate forces involved.	a) Thermal; b) calculate forces involved.
54	a) Thermal, creep, live load, shrinkage (deck); b) assume pile depth for fixity.	a) Thermal, creep, live load, shrinkage; b) assume pile depth for fixity.
55	a) Thermal effects; b) bearings and expansion devices, columns.	a) Thermal effects; b) columns.
56	a) All, if applicable.	a) All, if applicable.

Questions 3(i) and (j) continued.

62	a) Use an assigned value for restraining force--up to 25 kips per pile.	a) Use an assigned value for restraining force--up to 25 kips per pile.
64	a) Thermal effects, creep, backfill movement and settlement, etc.	a) Thermal effects, creep, backfill movement and settlement, etc.
66		a) Shrinkage, temperature, creep; b) provide in design.
67	a) Thermal effects, side creep, settlement; b) Earth fill strain vs. passive pressure build-up acting on bridge.	a) Thermal effects, side creep, settlement; b) Earth fill strain vs. passive pressure build-up acting on bridge.
68		a) Thermal stresses; b) add to DL and LL as per AASHO.

In your design (non-integral construction) do you provide for (allow) girder end rotation

k) for steel structures?	l) for concrete structures?
Yes - 26	Yes - 26
No - 0	No - 3
Blank - 20	Blank - 17

	If Yes, how? (e.g., curved steel plates)	If Yes, how? (e.g., curved steel plates)
11		Only on long spans (40 ft or greater).
12	Curved plates, rockers and elastomeric pads.	Curved plates, rockers and elastomeric pads.
13	Curved plates or neoprene pads.	Curved plates or neoprene pads.
14	Bearings.	Bearings.
16	Rocker shoes or curved steel plates or neoprene pads.	Neoprene pads. No for slab spans.
20	Rocker plates, pins or elastomeric pads.	Rocker plates, pins or elastomeric pads.
24		Elastomeric bearing pads, curved steel plates.
26	Curved sole plates.	Curved sole plates.
27	Dead load only.	
35	Curved steel plates and rockers.	Elastomeric pads.
37	Curved steel plates or rocker bearings.	
41	Curved plates or neoprene pads.	Curved plates or neoprene pads.
42	Curved steel bearing plates.	Elastomeric bearing pads.

Questions 3(k) and (l) continued.

44	Curved steel plates and rudimentary hinge (crossed re-bars) in backwall.	Sliding steel plate.
46	Curved plates, rockers, rollers.	Curved plates, neoprene pad.
47	Curved steel plates.	Elastomeric pads.
48	Curved steel plates.	Curved steel plates, neoprene pads.
51	Elastomeric bearing pads or pinned bearings.	Elastomeric bearing pads.
52	Curved plates and pinned type shoes.	Curved plates.
53	Neoprene pads or suitable bearing devices.	Neoprene pads or suitable bearing devices.
54	Pin.	Elastomeric pads.
55	Curved rocker bearings.	Elastomeric bearing pads.
56	Curved steel plates, when required by AASHO.	
61	Curved steel plates.	Curved steel plates.
63	Elastomeric pads.	Elastomeric pads.
64	Curved steel plates, neoprene pads.	Curved steel plates, neoprene pads, keyed joints with expansion material for clearance.
65	Curved plates.	Curved plates or neoprene pads.
67	Elastomeric bearing pads; short spans--expansion felt or 90-lb paper.	Elastomeric bearing pads; short spans--expansion felt or 90-lb paper.

If girder end rotation--separate from joint rotation--is not allowed (integral or semi-integral construction) do you assume and take into consideration joint rotation due to flexing of the abutment piling and pier piling

m) for steel structures?	n) for concrete structures?
Yes - 7	Yes - 14
No - 9	No - 7
Blank - 30	Blank - 25

Comments

	Steel	Concrete
14		For voided slab spans on occasion.
16	By judgment.	By judgment.
20	If end bent is too stiff, a hinge is provided; similar hinge top and bottom for very short end columns.	If end bent is too stiff, a hinge is provided; similar hinge top and bottom for very short end columns.
26	Bearings allow for rotation.	Bearings allow for rotation.

Questions 3(m) and (n) continued.

37	Pier or pile bents designed to deflect.	
44	Used symmetrical span arrangement and elastomeric bearing at pier.	Consider joint rotation due to flexing in bearing pad.
47	Assume flexing of the piling.	Assume flexing of the piling.
49		Piling is assumed to rotate at abutments.
54	Moments and rotations are calculated based on an assumed pile fixity position.	Moments and rotations are calculated based on an assumed pile fixity position.
61		Girder end rotation is assumed to occur by flexing of piling.
62	Use on steel is very infrequent.	Unusual deck thicknesses and pier heights regulate consideration.
63	Neglect for the small spans used.	Assume abutment rotates on single row of piles or concrete key.
67	Generally not; we have with deep curtain walls.	Generally not; we have with deep curtain walls.

4. What limitations do you impose with respect to flexibility and type of substructure--e.g., pile materials, L/r ratios, cast in place substructures on spread footing, backfilling techniques, preboring, etc., for integral or semi-integral type of construction

	a) for steel structures?	b) for concrete structures?
11		L/D = 20 maximum.
12	Use limited number of pile bent piers.	Use limited number of pile bent piers.
13	Have not used them for abutments or short, stout columns.	Have not used them for abutments or short, stout columns.
14	Pile cap bents--15 ft maximum exposed pile length.	Pile cap bents--15 ft maximum exposed pile length; insure pin connection at bottom of interior bent (voided slab).
16	Use battered piling or drilled shafts for lateral stiffness for both steel and concrete structures (except slab spans).	Use battered piling or drilled shafts for lateral stiffness for both steel and concrete structures (except slab spans).
20	For integral abutments, we would require piling, or flexible hinged columns--some flexibility.	For integral abutments, we would require piling, or flexible hinged columns--some flexibility.

Questions 4(a) and (b) continued.

24		Minimum of 20 ft piles for end bents.
27	Limited to column bents and abutments on single row of piles.	Limited to column bents and abutments on single row of piles.
42	When piers of sufficient height to cause minimal horizontal forces due to thermal movement.	Cast in place substructure on spread footing limited to square structures where integral construction is used.
43		Use prebored holes for piling.
44	Flexibility of pile material--we use 12 in. CIP, 40 ton capacity. Piles prebored to bottom of fill at abutment, granular material backfill (compacted in place); piers are either pile supported or on spread footing.	Flexibility of pile material--we use 12 in. CIP, 40 ton capacity. Piles prebored to bottom of fill at abutment, granular material backfill (compacted in place); piers are either pile supported or on spread footing.
46	Keep down resistance to movement (earth pressure) by use of corrugated sheet metal and styrofoam on abutment backwall.	
47	Steel and timber piling only; prebore through embankment; backfill after deck is in place.	Steel and timber piling only; prebore through embankment; backfill after deck is in place.
48	Select granual backfill; preboring for 10 ft minimum and backfill with sand. We use steel piling oriented as shown in your detail, although we do allow timber piles also.	Select granual backfill; preboring for 10 ft minimum and backfill with sand. No restriction on pile type.
49		Single row of piling at abutments--column type piers.
53	Integral and Semi-Integral types not used for steel structures.	Single row of piling used.
54	Structural integrity must be maintained, taking into account all of the above factors.	Structural integrity must be maintained, taking into account all of the above factors.
55	Abutments on steel piling only.	Abutments on steel piling only.
56	Vertical piles, columns, and spread footings designed for movement.	Vertical piles, columns, and spread footings designed for movement.

Questions 4(a) and (b) continued.

61		Integral end bents to date have been pile supported for reasonable flexibility. Supporting data to justify the use of integral end bents for long spans are not extensive enough to formulate any definite standards for future construction.
62	Infrequently used.	No limitations established.
63		If pile footing, use single row embedded 1 ft into concrete. Use concrete key if on spread footing.
65		AASHO Specification--stress limitations.
67	H piles or 15-in. maximum piles; one row of piles or two rows with small torsion arm; piles 20 ft or longer ordinary backfill (some yielding).	H piles or 15-in. maximum piles; one row of piles or two rows with small torsion arm; piles 20 ft or longer; ordinary backfill (some yielding).
68		Substructure flexible enough to satisfy AASHO Group IV loading.

5. What objections, if any, do you have to the above method referred to in questions 1 and 2.

	a) for steel structures?	b) for concrete structures?
14	None at this time when within skew limits.	
20	None for appropriate cases; some judgment is required.	None for appropriate cases; some judgment is required.
21	We use expansion bearings at abutments to eliminate earth pressure stresses being induced into girders.	We use expansion bearings at abutments to eliminate earth pressure stresses being induced into girders.
26	Does not allow for expansion or rotation.	Does not allow for expansion or rotation.
27	None when used within limitations established by above answers.	None when used within limitations established by above answers.
33	Areas of structural distress may be more prevalent.	Areas of structural distress may be more prevalent.
36	Not enough freedom for structure to "breathe".	Not enough freedom for structure to "breathe".

Questions 5(a) and (b) continued.

42	Our present method of using rollers has not been a source of trouble.	No objections for moderately short spans (thermal movement less than 1 in. for types II and III).
44	None--except for the expense of approach slab currently used.	
45	Induced stresses.	Induced stresses.
46		Excessive movement due to creep of prestress beams.
49	Actual movement of steel structures is normally much greater than movement of concrete structures.	
51	Method not usually considered advantageous.	None, if bridge is of moderate length.
53	Suitable for short structures or one span of series only (Type I).	Type I not often used with concrete structures.
54	None, so long as stiffnesses are properly accounted for.	None, so long as stiffnesses are properly accounted for.
61		Integral end bents to date have been pile supported for reasonable flexibility. Supporting data to justify the use of integral end bents for long spans are not extensive enough to formulate any definite standards for future construction.
64	Thermal effects in most of the state are too great.	
65	Uneconomical and approach surfacing problems.	Uneconomical and approach surfacing problems.
67	On skew bridges, ends of bridge creep sidewards; approach embankment settlement or movement; generally do not use approach slabs.	On skew bridges, ends of bridge creep sidewards; approach embankment settlement or movement; generally do not use approach slabs.

6. What problems, if any, have you encountered when the method referred to in questions 1 and 2 was used

	a) for steel structures?	b) for concrete structures?
12	Non-Integral abutments tend to move. Also, maintenance of expansion device.	We try to use Integral abutments.
16	Fill settlement causes distress.	Fill settlement causes distress.

Questions 6(a) and (b) continued.

25	Concrete cracking around beams from live load deflection.	
26	Joint between approach slab and end of bridge soon leaks and washes out fill under approach slabs.	Joint between approach slab and end of bridge soon leaks and washes out fill under approach slabs.
37	Short piers present design problems due to their lack of flexibility for Semi-Integral or Integral connections to continuous superstructures, or for multiple simple spans using these connections at both ends.	
42	No trouble since ends are allowed to rotate and/or expand. In a location which involved a 200 ft span and a very stiff pier, (low l/r) the pier stem cracked badly. This involved both ends of the span becoming fixed. A conclusion would be to limit the l/r not to maximum, but rather to a minimum value.	None for moderately short spans.
45	Spalled concrete in end-beam web.	
47	When Semi-Integral, dowel bars should be placed in center of abutment wall. Dowel bars did crack the wall when placed only 3 in. from face.	When Semi-Integral, dowel bars should be placed in center of abutment wall. Dowel bars did crack the wall when placed only 3 in. from face.
48	None with steel piling. Some trouble in construction with timber piling (please see Item 7, Geographical Area 4).	
49		Cracking of end diaphragms on structures skewed more than 20°.
51		Some cracking in endwalls; nothing serious.
53	Movement of end bent--forcing closure of expansion joints at other locations.	
56	Cracking of wingwalls.	

Questions 6(a) and (b) continued.

62	Asphalt concrete approach fills appear to settle more than at conventional structures.	Asphalt concrete approach fills appear to settle more than at conventional structures.
64		Some settlement of roadway embankment and continual cracking of roadway surfacing.
67	Skewed bridges have rotated, and are rotating; abutment movement.	Skewed bridges have rotated, and are rotating; abutment movement; girder web vertical cracks or checks (reason uncertain at present).
68		Soil erosion where superstructure contracts away from backfill.

7. Additional comments and suggestions.

Geographical Area 1.

11 We have heard of the suggestion of using the Integral cap for long span steel structures. We wonder about the problem of movement of the earth fill. It seems that this would present a problem.

12 As indicated, it has been our practice to use monolithic construction on nearly all of our reinforced concrete structures until they get of such length we consider expansion too great. This includes monolithic construction between the superstructure and abutments, and the piers. We have used rocker type expansion devices when overall length gets beyond approximately 500 ft.

While we normally use some type of bearing devices in our steel construction, we are moving more and more to trying to cast the abutment in with the ends of the girders. This is an attempt to try to reduce maintenance costs resulting from abutment movement and backwall failures. However, structures much over 200 ft in length are normally placed on some type of bearing devices.

It is hoped your study will give us additional information relative to stresses created due to expansion and just how critical these are. We realize monolithic construction does carry sometimes relatively high moments; however, these do not seem to give us any structural failure problems.

Whenever possible we use Integral abutments on concrete structures. This gives us a maintenance saving. We have

Question 7 continued.

experienced no settlement problems, but occasionally a wing wall will crack.

Separated abutments means maintenance problems. Cracking of backwalls is common. Having to re-set bearing devices is common. De-icing agents leaking through expansion joint is always a trouble generator.

13 The fixed pier is new with us and as yet none have been built. We are building some at the present time and will know more about the problems involved in a couple of years.

14 Integral type of construction is not used for steel structures. Use of Semi-Integral construction for concrete structures was discontinued in January, 1972. Non-Integral type of construction for concrete structures is not used currently.

15 We have not used this type of construction because of the unknowns as to restraint in the bridge. We do not believe the stresses predicted in the bridge are very reliable. We have used the Integral type construction on concrete rigid frames only.

16 Practically all of our abutments look like the "Non-Integral" section except piling are battered opposite directions 3:12 in pairs or 30-in. diameter straight drilled shafts are used. Continuous steel units are usually expansion with rocker shoes at abutments. Continuous concrete units are usually expansion with neoprene bearings at abutments. Simple spans are usually fixed but not Integral at abutments with expansion joint and bearing at the first interior bent. Concrete slab bridges usually rest directly on the cap, either fixed or expansion with roofing felt and graphite--some are even Integral. These abutments, being short of height, usually have vertical piling. Our abutments are designed for vertical superstructure loads and lateral soil pressure with no horizontal resistance counted for the superstructure--except for slab spans which are designed for vertical loads only. Fill settlement is not accounted for numerically in the abutment design, but does in fact cause distress in many of our abutments.

Geographical Area 2.

20 In general, we prefer Integral abutments and piers whenever feasible in continuous concrete or prestressed concrete structures, and this practice is now the rule rather than the exception. We have less experience with this type of construction in connection with steel superstructures, but we are considering this alternative in connection with various structures in the planning stages. We have

Question 7 continued.

occasionally used Integral backwalls and wingwalls supported on steel girders where free cantilever end spans are used. This type of construction has to be used with good judgment. We have used fixed bearings (permitting rotation) for continuous steel girder bridges on tall piers, with spans up to 220 ft and units up to 780 ft between expansion bearings. We have used piers Integral with concrete box girders with spans up to 100-115 ft and in unit lengths up to about 400 ft between expansion joints. We would not use Integral abutments of low height founded on rock, without positive provision for expected movements. We give consideration, without attempting rational analysis, to such forces as friction between earth and wingwalls, inertia of long wings, etc. We need criteria for passive earth pressure against backwalls when the bridge expands. Need better criteria for effects of shortening of long post-tensioned bridges due to prestressing. Would find useful the publication of suggested details for Integral and Semi-Integral construction, including cantilever end spans.

21 It is our feeling that any deflection of the abutment due to earth pressures would induce additional stresses in the girders of the superstructure. This may be insignificant but this is the method we prefer.

22 We have used this on only a very limited number of structures. Not enough experience to answer this questionnaire. Basically they are built without considering any added stresses.

23 Our Bridge Design Division has never used this type of construction, so we are unable at this time to complete your questionnaire.
We are studying the problems of induced stresses which result from this type of construction and plan to design three continuous steel superstructures with Semi-Integral or Integral abutments in the near future.

24 We have presently used all three conditions referred to on both poured-in-place concrete and prestressed, precast concrete bridges continuous over interior supports.
We have designs under way with steel bridges with Semi-Integral end bents, but we have not constructed any at this date.

25 We primarily use the Semi-Integral type with a 40-ft simple span prestressed beam placed on a neoprene pad. The other end of the beam is placed on a neoprene pad which provides for expansion. On our continuous structures we provide a joint and bearing assembly to provide for rotation and translation on a Non-Integral type end abutments. Our normal end abutment for spans above 40 ft is Non-Integral with provision made for translation and rotation.

Question 7 continued.

26 In recent years we have constructed both steel and concrete beam structures up to 300 ft in length with no provisions made for expansion in the bridge between superstructure and substructure. We have always provided for rotation of the beams with the curved sole plates as indicated on our sketch. We have also constructed structures up to 300 ft in length with expansion slots in the beams at the end bent bearings allowing the superstructure to expand against the approach fills. We are now trying to evaluate the effects of both of these types of construction after a few years of service under those conditions.

For continuous structures over 300 ft in length we have always used the Non-Integral condition similar to that indicated on your sketches. We are also presently using this type detail for most continuous structures regardless of length.

For simple span bridges our standard practice is to fix the beam or abutment and allow expansion at the opposite end of the beam.

27 We have been using this method of construction for some time and are well satisfied with the performance of the structure. Without question, our own design philosophy could be challenged; however, the structure meets the best test we've been able to come up with: it works.

28 I find it difficult to fill out the questionnaire itself because we have made very limited use of this procedure. We have occasionally used flexible piers with continuous spans but not flexible abutments except for a few concrete frames which are in a different category.

I see no objection to the use of either concrete or steel spans continuous being anchored to the abutments without provision for expansion provided the crossings are approximately square (no skew). We have actually built a few such bridges using continuous slab spans with initial expansion joints at abutments which, being small, quickly closed leaving no expansion provision within the structures.

With respect to steel expansion spans and especially those shown in your sketches as Semi-Integral or Integral, we do have serious objections. We have had too many cases of structural steel members projecting into and embedded in concrete walls and in such cases have had very considerable rusting of the steel at the point where it enters the concrete. If we were to use either of these designs, we feel we would have to provide for adequate protection against corrosion at this point and this detail we have not yet solved.

In general, I believe that the use of spans without interior provision for expansion can be successfully used with lengths up to several hundred feet provided they are not skewed. If the structures are skewed, provision would have to be made

Question 7 continued.

for preventing sidewise movement and related high stresses due to the transverse component between bridge and approach roadway slab.

Geographical Area 3.

31 In many instances, we have used flexible pile bent piers with no expansion type devices. However, we have not, to date, used abutments of this type.

32 We do not use any of the types noted in this Questionnaire. At present we are dubious of their efficiency stress-wise and in prevention or containment of water seepage at the fill side of backwall.

Geographical Area 4.

41 In our state, we have used this type of construction very sparingly--only on steel spans no longer than 45 ft and on continuous concrete slab structures up to 100 ft over-all. We therefore have limited applicable experience in their design.

42 We have not constructed any steel beam structures with the beam ends Integral or Semi-Integral with the abutments. However, we have constructed continuous reinforced concrete slab bridges with all pile bent caps Integral with the slab.

44 At each end of our structure we have incorporated a special approach slab (13 ft $\pm$ long) which terminates at a reinforced neoprene expansion joint. The road pavement also terminates at this joint.
We have used Bituminous Pavement on the approaches for flexibility because of settlement due to movement at abutment.
Some short structures have been constructed without special approach slabs.

48 We had some trouble in construction with timber piling. In this case we poured a sill similar to your Semi-Integral with anchor bolts for the beams. Once the beams were set the remainder of the abutment was poured and the abutment was integral. One problem we had was that the anchor bolts were set so that the beams would readily fit between temperatures of 30° F and 100° F. The contractor tried to erect at -20° F and needless to say the beams wouldn't fit. In another anchor bolt type design by a consultant a temperature drop broke the concrete in front of the anchor bolts as they had very little concrete cover. We are working on new details for Integral

Question 7 continued.

Abutments for timber pile situations which we feel will eliminate the problems we have experienced.

49 Steel superstructures are rarely used in the span range where the fixed type of structures are used. The greater expansion and contraction of steel would restrict their use to overall length of approximately 150 ft, instead of the 300 ft we use with concrete. We use a temperature range of 150° for steel but only 85° for concrete.

Geographical Area 5.

51 In general, I am not opposed to the use of Type II or III abutment configurations for concrete bridges of moderate total length (say 300 ft). We seldom have occasion to use these abutment configurations for steel bridges because most of our steel bridges are of relatively long (100 ft or over) span and requirements for taking the expansion of steel bridges are more rigorous than for concrete bridges.

54 Many concrete slab structures are built in this state utilizing the Type III detail. A number of composite plate girder spans have used a detail similar to Type I. On many structures, however, we are using a back wall attached to the girders but an elastomeric pad between the girder and the supporting pile cap to allow some expansion motion.

56 The use of structures without specific provision for expansion is subject to considerable discussion and care is necessary in certain details such as wingwalls on abutments. The reduction in maintenance by elimination of expansion joints and bearings is welcome.

When designing locked-up superstructures every effort is made to keep substructures flexible. Backwalls are designed for passive earth pressure due to various movements. After the . . . earthquake in 1964 massive soil movements at abutments locked-up many of our bridges including some long span trusses. To date we have not noted distress in these structures. Wingwall cracking on wings parallel to roadway was experienced on first designs. Rotation soil friction and possibly freezing was more severe than expected. More reinforcement seems to have solved this problem.

Geographical Area 6.

62 We have not used the Non-Integral and Semi-Integral abutment types as detailed. Our use of these types has been limited to abutment footings with more than one row of piles to

Question 7 continued.

provide rotational stability. On expansion end abutments some of the piles are battered.

65 We very seldom use this type of structure (Integral construction); we use positive expansion devices for multiple spans.

67 For short to moderate length bridges with no or moderate skews we have not seen any problems. On one bridge, approach has settled 3 in. ± in 5-10 years after construction with resultant settlement of corners of bridge. Some low spots in approaches at end of bridge. Possibly worse on skewed bridges. Curb lines out of line from 1 in. on 125-ft 2-span skewed 4-year old bridge to 3 in. ± on 400-ft long 5 to 6-span skewed bridge.

APPENDIX B

SUMMARY OF FIELD OBSERVATIONS OF SELECTED BRIDGES

BRIDGE INSPECTION FORM

I. <u>Required equipment</u>:

1. Camera with high speed black and white film (Tri-x ASA 400) or with Ektachrome high speed color (ASA 160).
2. 100' tape.
3. 8' or 10' tape.
4. Hand level to spot possible settlements or construction busts.
5. Marking crayon for aid in making measurements.

II. <u>Special Instructions</u>:

1. Picture taking--we should follow a systematic picture taking procedure in order to have useful information upon return from the trip. Each picture taken should be assigned a roll no. and the corresponding no. on the film for easy reference to time of picture taking. All pictures <u>must</u> be referenced as to locations on the bridge and direction of sight. The following is a suggested minimum no. of picture types to be taken for each bridge.

   Picture type 1--side view of entire bridge (specify direction of sight)

   Picture type 2--longitudinal view looking at abutment girder joint detail, (specify direction of sight)

   Picture type 3--picture of joint key at abutments (specify which abutment and direction of sight)

   Picture type 4--longitudinal view showing pier type (specify which pier and direction of sight)

   Picture type 5--longitudinal view showing pier type (specify which pier and direction of sight)

   Picture type 6--cracks and their location, magnitude, and assigned no. together with direction should be given. A convient means of showing crack width in a photo is to mark it with a no. e.g. with a crayon and put a pencil or ruler across the crack to show the magnitude.

III. Required General Information: (Actual Inspection).

Inspectors:__________________, __________________, ________________.

Date:________________ Time____________ Temp.__________ Weather______.

Bridge No.________ Bridge location (County)_________________________.

Year Built________ Name of Road highway bridge is on_______________.

Crossing: Stream or River Name______________ Road Name_____________.

Pictures:

Roll No. _____ Film No. _____ Location_____________ Direction______.

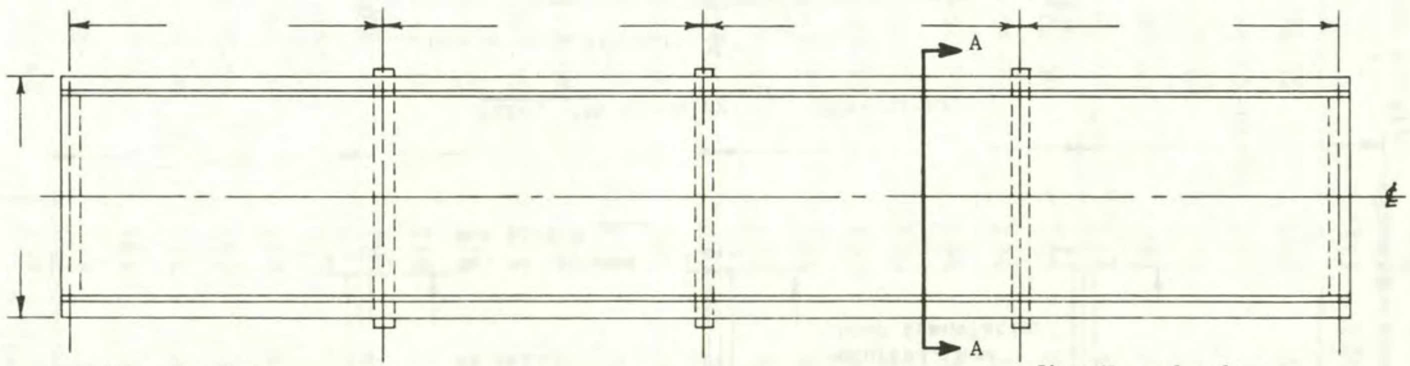
Roll No. _____ Film No. _____ Location_____________ Direction______.

IV. Superstructure: (show abuts. and pier caps rotated to the proper skew)

Show any parapet cracks (with spacing, locations; and inside face, outside face, or both; crack width, e.g., hairline, or width in in.)

Note: Show any transverse or longitudinal slab cracks in the picture below. Show location, width (hairline, --inches), length; specify whether top of slab, bottom of slab, or both.

Show North arrow here.



Show No. of piles and spacing

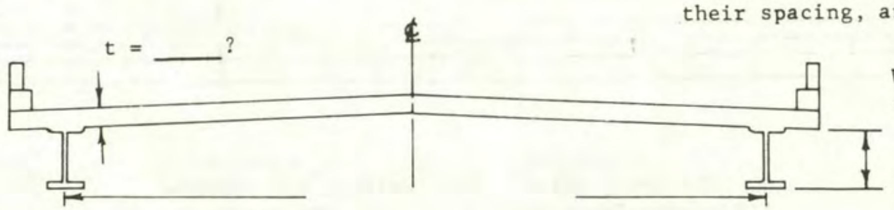
Show No. of columns and diameters

○ = ______, □ = ______?

PLAN VIEW

Note: If 3 span, or spans change from what is shown, make appropriate changes on picture.

Show No. of girders and their spacing, and size.



What is the girder depth?

SEC. A-A

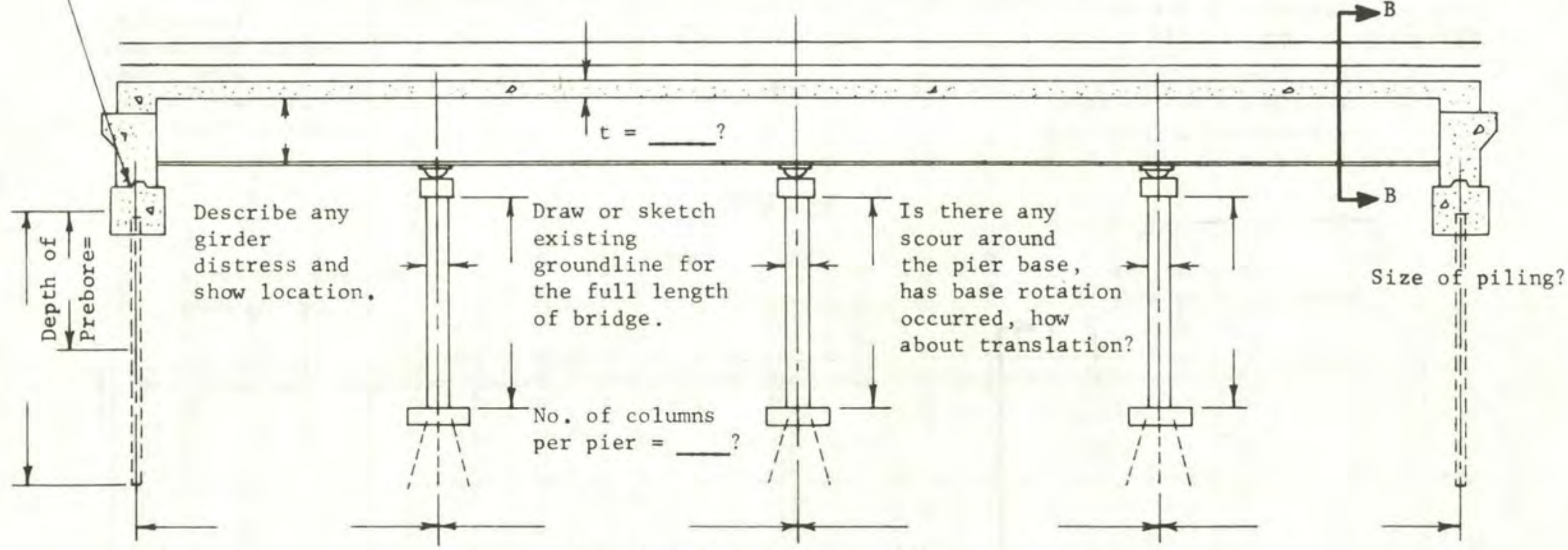
V. Substructure:

Has there been any movement here? What does the end look like, any mov., conc. spalling, etc.?

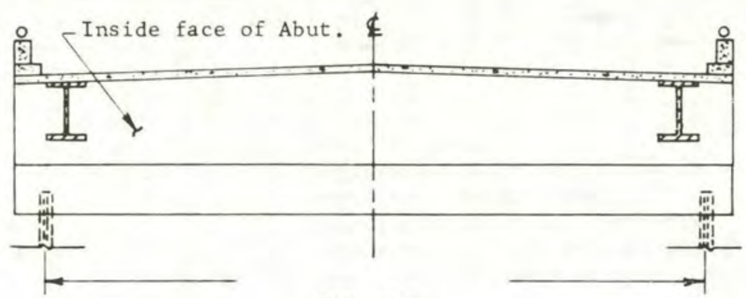
NOTE: If 3 spans or spans change from what is shown make appropriate changes on picture.

Have any cracks occurred in the pier columns, describe crack location and type.

Are the bearings centered on the pier, are they acting properly, look at rotated positions, etc.?



ELEV. VIEW (Looking ___ (Direction))



Show No. of girders and if there are any cracks in this area, describe direction, width and location.

SEC. B-B

Table B. 1. Geometric Properties of Bridges Observed

Bridge No.	No. of girders	Span lengths (ft)	Bridge length (ft)	Road-way (ft)	Loading	Skew	Deck thickness (in.)
A-1150	5	52-74-52	178	38	HS20-44	---	7.5
A-1375	4	49-64-64-64-49	290	32	HS20-44	---	7.5
A-1916	4	36-47-36	119	28	H15-44	10°	6.75
A-1973	4	60-80-60	200	26	H15-44	---	6.75
A-1982	4	49-64-64-49	226	32	HS20-44	---	7.5
A-2001	4	50-65-50	165	26	H15-44	---	6.75
A-2019	5	39-56-39	134	38	HS20-44	13°	7.5
A-2019	5	39-56-39	134	38	HS20-44	13°	7.5
A-2020	5	35-80-35	150	38	HS20-44	---	7.5
A-2020	5	35-80-35	150	38	HS20-44	---	7.5
A-2069	5	37-49-37	123	38	HS20-44	---	7.5
A-2069	5	37-49-37	123	38	HS20-44	---	7.5
A-2293	4	37-49-37	123	28	H15-44	---	6.75
A-2340	4	40-52-40	132	28	H15-44	30°	6.75
A-2350	10	43-58-43	144	70	HS20-44	5°	7.5
A-2358	4	48-62-48	158	28	H15-44	10°	6.75
A-2361	4	52-67-52	171	28	H15-44	---	6.75
A-2385	5	38-50-38	126	38	H20-44	---	7.5
A-2462	6	36-47-36	119	44	H20-44	26°	7.5
A-2463	6	36-47-36	119	44	H20-44	---	7.5
A-2465	4	37-48-37	122	28	H15-44	---	6.75
A-2525	6	36-46-36	118	44	H20-44	---	7.5
A-2531	5	56-70-56	182	34	H15-44	---	6.75
A-2565	4	46-59-46	151	28	H15-44	20°	6.75
A-2573	5	43-55-43	141	38	HS20-44	19°	7.5
A-2581	5	34-45-34	113	38	H20-44	---	7.5
A-2595	4	48-62-48	158	28	H15-44	---	6.75
A-2596	4	50-65-50	165	28	H15-44	10°	6.75
A-2616	6	52-74-52	178	46	HS20-44	---	7.5

Table B. 2. Comparative Substructure Stiffness

Bridge No.	Bent 1 No. of Piles	Bent 1 L/r [a]	Bent 2 No. of Columns	Bent 2 L/d	Bent 3 No. of Columns	Bent 3 L/d	Bent 4 No. of Piles	Bent 4 L/r [a]	Bent 5 No. of Columns	Bent 5 L/d	Bent 6 No. of Columns	Bent 6 L/d
A-1150	5	119	2	8	2	8	5	128				
A-1375	5	36[b]	8	34	8	34	8	33[b]	8	34	5	32
A-1916	4	64	4	64[c]	4	64[c]	4	64				
A-1973	4	131	4	95[c]	4	107[c]	4	145				
A-1982	5	37[b]	8	34	8	30	8	30[b]	5	32		
A-2001	4	38[b]	2	9	2	9	4	39[b]				
A-2019	5	262	2	6	2	6	5	290				
A-2019	5	326	2	7	2	7	5	328				
A-2020	5	232	2	5	2	5	5	246				
A-2020	5	251	2	5	2	5	5	272				
A-2069	5	78	5	93[c]	5	93[c]	5	87				
A-2069	5	78	5	93[c]	5	93[c]	5	84				
A-2293	4	96	2	9	2	9	4	81				
A-2340	4	58	2	7	2	7	4	58				
A-2350	10	218	4	9	4	9	10	218				
A-2358	4	96	2	11	2	10	4	73				
A-2361	4	180	2	11	2	12	4	180				
A-2385	5	113	5	99[c]	5	110[c]	5	110				
A-2462	6	73	3	9	3	9	6	64				
A-2463	5	76	3	9	3	8	5	67				
A-2465	4	49	2	7	2	7	4	64				
A-2525	5	38	2	4	2	3	5	38				
A-2531	5	38	2	4	2	5	5	38				
A-2565	4	108	2	13	2	13	4	105				
A-2573	5	52	2	9	2	9	5	55				
A-2581	5	122	5	134[c]	5	116[c]	5	105				
A-2595	4	87	2	7	2	7	4	87				
A-2596	4	73	2	6	2	6	4	73				
A-2616	6	118	2	6	2	6	6	118				

[a] L/r for a pinned-end column; does not take into account soil pressure, inflection points, and effective length.

[b] L/d; precast concrete pile or cast-in-place concrete pile.

[c] L/r; steel pier column.

Table B. 3. Bridges and Items Observed

Bridge No.	County	Year completed	No. of spans	Bridge length (ft)	Skew	Transverse deck cracks	Parapet cracks	Abutment cracks below girder	Girder corrosion	Abutment movement	Wingwall cracks	Comments
A-1150	Bates	---	3	178	---	---	x	x	---	---	---	Seepage at abutment joint.
A-1375	New Madrid	11-68	5	290	---	---	x	x	x	---	x	Corrosion of bottom flange of exterior girder. Seepage at abutment cracks.
A-1916	Lincoln	6-68	3	119	10°	---	x	x	---	---	x	Appeared in good condition
A-1973	Vernon	9-67	3	200	---	---	x	x	---	---	---	
A-1982	New Madrid	11-68	4	226	---	---	x	x	x	---	---	Corrosion of bottom flange of exterior girder. Seepage at abutment cracks.
A-2001	Buchanan	7-68	3	165	---	---	x	x	---	---	---	Cracks in concrete diaphragm.
A-2019	Platte	9-71	3	134	13°	x	x	---	---	---	---	Curved girder bridge.
A-2019	Platte	9-71	3	134	13°	---	x	---	---	---	---	Curved girder bridge.
A-2020	Clay	9-71	3	150	---	x	x	x	---	---	x	Leaching through slab cracks at inflection points and piers.
A-2020	Clay	9-71	3	150	---	x	x	x	---	---	---	Leaching through slab cracks at inflection points and piers.
A-2069	Cass	3-71	3	123	---	---	---	x	---	---	---	Curb cracks, no parapet cracks; inside surfaces floated.
A-2069	Cass	3-71	3	123	---	---	x	x	---	---	---	Parapet cracks, none on "twin".
A-2286	Harrison	---	3	---	---	---	x	x	---	---	---	Approach slabs not placed.

Table B. 3 (Continued).

Bridge No.	County	Year completed	No. of spans	Bridge length (ft)	Skew	Transverse deck cracks	Parapet cracks	Abutment cracks below girder	Girder corrosion	Abutment movement	Wingwall cracks	Comments
A-2286	Harrison	---	3	---	---	---	x	x	---	---	---	Approach slabs not placed.
A-2293	Cass	3-71	3	123	---	---	---	x	---	---	---	Curb cracks, no parapet cracks; inside surfaces rubbed; a few abutment cracks below girders.
A-2340	Osage	7-70	3	132	30°	---	x	x	---	---	---	
A-2350	Jackson	9-71	3	144	5°	---	x	x	---	---	---	Inside parapet surfaces rubbed; seepage at abutment joints.
A-2358	Monteau	9-69	3	158	10°	---	x	x	---	---	---	Abutment cracks one end only.
A-2361	Callaway	8-70	3	171	---	---	x	x	---	x	---	Cracks in pile cap; numerous parapet cracks; settlement and rotation of abutments; abutment pile cap cracked.
A-2385	Lafayette	5-71	3	126	---	---	x	x	---	---	---	
A-2462	Jackson	---	3	119	26°	---	---	x	---	---	---	Seepage at abutment joint.
A-2463	Jackson	---	3	119	---	---	---	---	---	---	---	Not completed.
A-2465	Maries	4-71	3	122	---	---	x	---	---	---	---	
A-2525	Osage	9-71	3	118	---	---	---	---	---	---	---	Appeared in good condition.
A-2531	Maries	7-71	3	182	---	x	x	x	---	---	---	Two transverse deck cracks approximately 4 ft apart at the centerline of the two end spans. Extensive parapet and curb cracks.
A-2565	Buchanan	11-70	3	151	20°	---	x	x	---	---	---	

Table B. 3 (Continued).

Bridge No.	County	Year completed	No. of spans	Bridge length (ft)	Skew	Transverse deck cracks	Parapet cracks	Abutment cracks below girder	Girder corrosion	Abutment movement	Wingwall cracks	Comments
A-2573	Lincoln	---	3	141	19°	---	x	x	---	---	---	Approach slabs not placed.
A-2581	Buchanan	9-71	3	113	---	---	x	x	---	---	---	
A-2595	Lincoln	8-71	3	158	---	---	---	x	---	---	---	
A-2596	Lincoln	8-71	3	165	10°	---	x	x	---	---	---	
A-2616	Bates	---	3	178	---	---	x	---	---	---	---	
A-2636	Mercer	---	3	---	---	---	x	---	---	---	---	Cracks in asphalt approaches.

APPENDIX C

A THEORETICAL APPROACH TO INDUCED BRIDGE STRESSES

Notation

A = cross sectional area of elastic bearing, sq in.

A_b, A_s = area of beam and gross area of concrete bridge slab, respectively, sq in.

b = effective width of concrete bridge slab, in.

b_f = flange width of steel beam, in.

c = half-depth of bridge slab, in.

d_s = distance from the center of action of the resultant passive soil pressure force to the centroid of the composite section, in.

d_1, d_2 = distance from the beam centroid to the top and bottom of the beam, respectively, in.

$\bar{d}$ = distance from beam centroid to centroid of composite section, in.

$\bar{d}_2$ = distance from bottom of beam to centroid of composite section, in.

E = modulus of elasticity, ksi

E_b, E_s = modulus of elasticity of the beam and concrete bridge slab, respectively, ksi

E_c = modulus of elasticity of the composite section, ksi

EI = flexural stiffness, kip-in.^{2}

F = interface shear between concrete bridge slab and beam, kips

f = effective pier flexibility, in./kip

f_b, f_p, f_r = bearing, pier shaft, and pier base rotation flexibility, respectively, in./kip

f_{y_1}, f_{y_2} = lateral pile flexibility, in./kip, in./in.-kip

f_{θ_1}, f_{θ_2} = rotational pile flexibility, rad/kip, rad/in.-kip

G = modulus of rigidity, ksi

H = horizontally applied force, kips

H_a = total induced horizontal force at the abutment, kips

H_i = induced horizontal force at support being considered, kips

H_n = total distributed horizontal force resulting from induced movements or externally applied forces through the deck, kips

H_s = induced horizontal force due to passive soil pressure, kips

h = height of pier, in.

I_b, I_s = moment of inertia of the beam and gross moment of inertia of concrete bridge slab, respectively, in.^{4}

I_c = moment of inertia of the composite section, in.^{4}

K = effective pier stiffness, kips/in.

K_i = effective stiffness of support being considered, kips/in. or in.-kips/rad

K_θ = elastic angular restraint stiffness, in.-kips/rad

k = subgrade modulus, ksi

L = length of end span, in.

L_i = distance from support being considered to left end of structure, in.

n = number of supports to the right or left of the point of zero movement or total number of supports, as applicable

M = moment, in.-kips

M_a = total induced moment at the abutment, in.-kips

M_i = induced moment at support being considered, in.-kips

M_{zs} = moment about the z axis in the concrete bridge slab, in.-kips

NL = length of interior span, in.

P_{xs} = axial force in the concrete bridge slab in the x direction, kips

Q = interface moment between concrete bridge slab and beam, in.-kips

q = number of friction bearing supports to the right or left of the point of zero movement or total number of supports, as applicable

R = vertical reaction, kips

R_i = vertical reaction of support being considered, kips

s = number of restrained supports, excluding friction supports, to the right or left of the point of zero movement or total number of supports, as applicable

T = temperature, ° F

T_{by} = temperature distribution through the beam, ° F

T_f, T_i = final and initial temperature, respectively, ° F

T_{sy} = temperature distribution through the concrete bridge slab ° F

t = depth of elastic bearing, in.

t_w = web thickness of steel beam, in.

V_a = shear at the abutment, kips

w_y = width of beam at distance y from beam centroid, in.

x, y, z = coordinate axes

y_b = vertical distance from the beam centroid, in.

y_s = vertical distance from the slab centroid, in.

X_i = distance from point of zero movement to support being considered, in.

X_o = distance from the left support to the point of zero movement, in.

Z = point of zero movement

α = thermal coefficient of expansion, in./in./° F

α' = apparent thermal coefficient, in./in./° F

α_b, α_s = thermal coefficient of the beam and effective thermal coefficient of the concrete bridge slab, respectively, in./in./° F

Δ_i = longitudinal deformation at the support being considered, in.

Δ_T = total longitudinal deformation, in.

Δ_t = longitudinal deformation at the centroid due to temperature, in.

Δ_θ = longitudinal deformation due to the unrestrained curvature, in.

ε_c, ε_e, ε_s = creep, elastic, and shrinkage strain, respectively, in./in.

ε_T = total unit strain at any point in time, in./in.

ε_t = temperature strain, in./in.

ε_x, ε_y, ε_z = unit strain in the x, y, and z directions respectively, in./in.

ε_{xb}, ε_{xs} = total temperature induced strains in beam and slab, respectively, in./in.

ε'_{xb} = longitudinal unrestrained beam strain in the x direction due to temperature, in./in.

ε''_{xb} = unit strain in the beam in the x direction due to interface shear and moment, in./in.

ε'_{xs} = longitudinal unrestrained slab strain in the x direction due to temperature, in./in.

ε''_{xs} = unit strain in concrete bridge slab in the x direction due to interface shear and moment, in./in.

ε'_{zs} = longitudinal unrestrained slab strain in the z direction due to temperature, in./in.

μ = coefficient of friction or Poisson's ratio, as applicable

μR = friction force, kips

ρ = radius of curvature, in.

ρ_b, ρ_s = radius of curvature of the beam and the concrete bridge slab, respectively, in.

σ = unit stress, ksi

σ_x, σ_y, σ_z = unit stress in the x, y, and z directions, respectively ksi

σ_{xb}, σ_{xs} = total temperature induced stress in the x direction in the beam and slab, respectively, ksi

σ'_{xb} = unit stress in the beam in the x direction due to temperature, ksi

σ''_{xb} = unit stress in the beam in the x direction due to interface shear and moment, ksi

σ'_{xs} = longitudinal unrestrained slab stress in the x direction due to temperature, ksi

σ''_{xs} = unit stress in the concrete bridge slab in the x direction due to interface shear and moment, ksi

σ'_{ys}, σ''_{ys} = unit stress in the concrete bridge slab in the y direction, ksi

σ''_{zs} = unit stress in the concrete bridge slab in the z direction due to interface shear and moment, ksi

θ = angular rotation of pier base, rad

θ_i = unrestrained curvature at the support being considered, rad

θ_b, θ_s = curvature of the beam and slab, respectively, rad

Introduction

Bridge structures, whether restrained or free to move, may experience significant induced stresses due to time-dependent effects. Longitudinal movements and induced stresses may lead to extensive maintenance costs and repairs if proper precautions are not taken in design and construction. In order to design for these effects, equations must be easily understood, and easy and simple to use. The primary objective of this section is to outline theoretical concepts and relationships which may be applied to any restrained composite girder bridge; for example, a Semi-Integral end bent bridge. These relationships will thus establish some of the background necessary for future simplification, when supplemented by empirical data as determined by field investigations.

The following development is divided into two phases; Phase one is concerned with procedures for locating a point of zero movement on bridge structures when expansive and contractive movements occur. The point is a function of the substructure stiffness and thus it is possible, when a massive abutment is used and other supports are free, that the point will occur at the end of the bridge or near the abutment. Phase two presents formulas to account for temperature effects in restrained or unrestrained composite girder bridges. The development is general in nature and is applicable to any time-dependent effect provided an equivalent temperature distribution is used.

Point of Zero Movement

Bridges experience longitudinal movements due to elastic deformations, creep, shrinkage, temperature, etc., and superimposed externally applied forces such as wind on the superstructure, wind on live load, braking forces, etc. The distribution of longitudinal horizontal forces caused by deck movements is a function of the stiffness of the supporting elements. As expansive and contractive movements occur, some point on the deck, Z, will remain stationary at a given point in time, which for future reference will be called the point of zero movement as in Fig. C. 1.

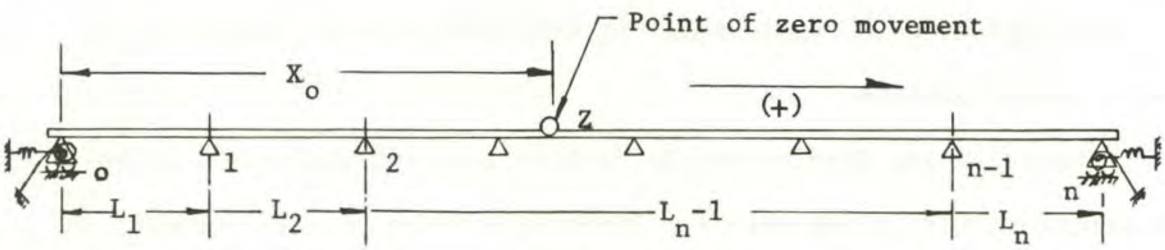


Fig. C. 1. General bridge structure--location of point of zero movement

The general theory for the location of point Z (zero movement) has been developed by other investigators.[a, b] The following development is general in nature and will apply to any continuous composite girder bridge restrained or unrestrained by the abutments against movement.

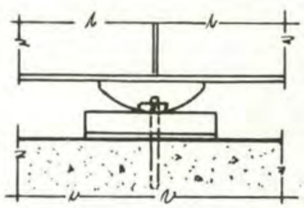
---

[a] Witecki, A.A., and Raina, V., "Distribution of Longitudinal Horizontal Forces Among Bridge Supports," *1st International Symposium on Concrete Bridge Design*, American Concrete Institute Publication SP-23, 1969, pp. 803-815.

[b] Zederbaum, J., "The Frame Action of a Bridge Deck Supported on Elastic Bearings," *Civil Engineering and Public Works Review*, London, Vol. 61, No. 714, Jan., 1966, pp. 67-69, 71-72.

Three basic types of bearings will be considered: Type 1) a fixed bearing; Type 2) a friction bearing such as a rocker or a roller; and Type 3) an elastic bearing. The translational stiffness of a bridge pier depends on the bearing stiffness, pier shaft stiffness, and the foundation restraint, whereas the abutment stiffness is dependent upon the bearing stiffness, the soil modulus, and the section properties and length of the supporting piles.

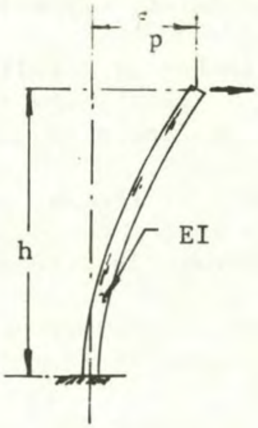
Considering the translational stiffness of a pier, if the pier support is considered to have a fixed bearing (with pintle) as shown in Fig. C. 2, the entire movement is transferred through the bearing to the pier. The bearing flexibility, f_b, is zero, and thus the flexibility and corresponding stiffness becomes that of the pier shaft and the foundation restraint effect.



$$f_b = 0 \tag{C. 1}$$

Fig. C. 2. Fixed bearing

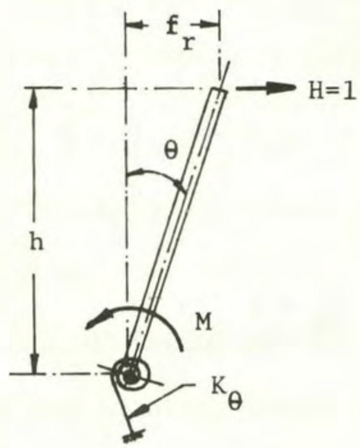
The pier flexibility may be evaluated by considering a unit load applied laterally at the top of the pier. The resulting deflection (or flexibility), f_p, is found as follows (see Fig. C. 3):



$$f_p = \frac{Hh^3}{3EI} = \frac{h^3}{3EI} \qquad \text{(C. 2)}$$

Fig. C. 3. Pier stem in bending

The rotation of the pier base may be accounted for by applying a lateral unit load at the top, and the resulting deflection at the pier top, f_r, becomes (from Fig. C. 4):



Let K_θ = Elastic angular restraint, $\frac{\text{in.-kips}}{\text{rad}}$

$M = Hh$

$\theta = f_r/h$

$K_\theta = M/\theta$

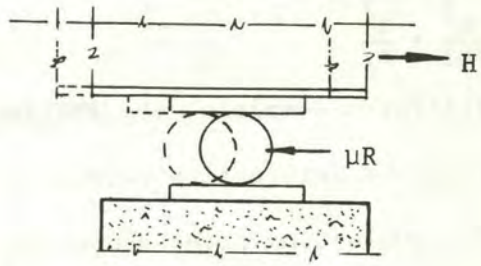
Thus equating $K_\theta \theta = Hh$

then

$$f_r = \frac{Hh^2}{K_\theta} = \frac{h^2}{K_\theta} \qquad \text{(C. 3)}$$

Fig. C. 4. Pier base rotation

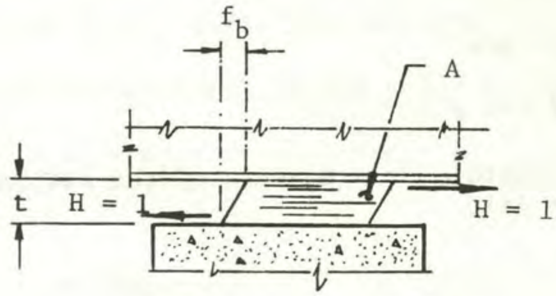
When a friction bearing as shown in Fig. C. 5 is considered in conjunction with a pier, the translational force is transmitted directly to the pier until such time as the magnitude of the force equals μR (at time of impending motion), where R is the reaction at the support. Of course, if the pier is flexible enough the force would always be some value less than μR. The friction factor, μ, is dependent on type of bearing, corrosion of the bearings, etc. Thus it is possible that friction bearings may act as fixed bearings. However, impending motion is assumed for this development, and consequently only the μR force is considered to resist movements. Some designers may choose at this stage to assume μR as negligible.



$$H = \mu R \tag{C. 4}$$

Fig. C. 5. Friction bearing (impending motion)

Elastic bearings, unlike the other two types, restrain movement and deform at the same time. As seen in Fig. C. 6, if a unit load is applied at the top of the bearing and the pier is considered restrained against movement, the resulting flexibility coefficient, f_b, is as follows:



$$f_b = \frac{Ht}{AG} = \frac{t}{AG} \tag{C. 5}$$

Fig. C. 6. Elastic bearing

Now, summing the individual flexibilities for the total deflection at the top of pier, the final effective pier stiffness is

$$K = \frac{1}{f} = \frac{1}{f_b + f_p + f_r} \tag{C. 6}$$

where

1. For a fixed bearing pier the total flexibility becomes

$$f = f_p + f_r = \frac{h^3}{3EI} + \frac{h^2}{K_\theta} \tag{C. 7}$$

2. For a friction bearing the restraining force is

$H = \mu R$ and is independent of movement once motion impends

3. For an elastic bearing support pier the flexibility becomes

$$f = f_b + f_p + f_r = \frac{t}{AG} + \frac{h^3}{3EI} + \frac{h^2}{K_\theta} \tag{C. 8}$$

In order to make the substructure stiffness complete, the abutment stiffness is determined by considering the interaction between the soil and the pile.[a, b] Design methods for evaluating piles subjected to lateral loads normally assume an elastic representation of the embedded pile and surrounding soil and are derived from the governing differential equation.

$$\frac{d^4y}{dx^4} + \frac{k}{EI}\,y = 0 \tag{C. 9}$$

where

y = the lateral pile deflection

x = the depth below ground surface

EI = the flexural stiffness of the pile

k = subgrade modulus, force/length^2, or a measure of the subgrade stiffness surrounding the pile.

---

[a] Matlock, H. and Reese, C., "Foundation Analysis of Offshore Pile Supported Structures," *Proceedings*, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. II, 1961, pp. 91-97.

[b] *Pile Foundation Know How*, American Wood Preservers Institute, Feb., 1969.

Hand solutions are available; however, when k varies with depth a computer solution is normally required. In order to evaluate the flexibility of the pile supports, it is convenient to apply a unit lateral force at the pile top and find the deflection; the same method is used when a unit moment is applied at the top of the pile (see Fig. C. 7). Thus, taking into account the support conditions between the abutment and the superstructure and applying the principles used for piers, the effective stiffness can be evaluated.

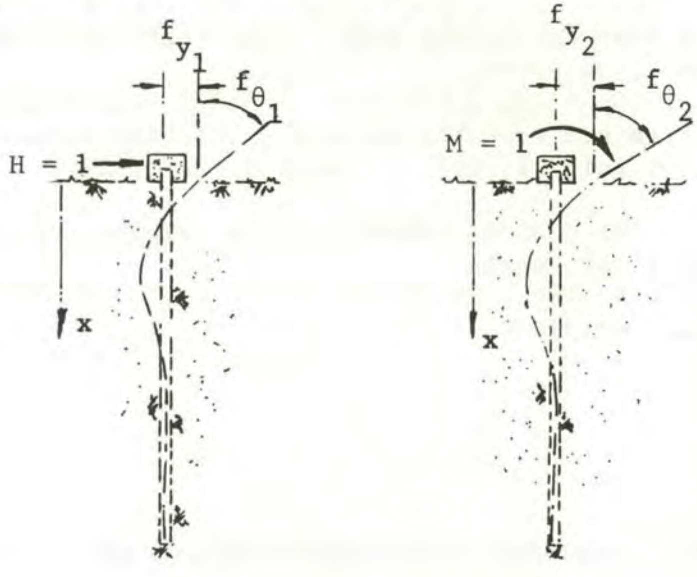


Fig. C. 7. Abutment pile flexibilities

Expansive and contractive movements of the deck are caused by such factors as creep, shrinkage, elastic deformations, temperature, etc. Thus, the total strain at any point in time, ε_T, is given by

$$\varepsilon_T = \varepsilon_t + \varepsilon_c + \varepsilon_e + \varepsilon_s \tag{C.10}$$

where

ε_t = temperature strain, $\alpha'(T_f - T_i)$

α' = may be considered as an apparent thermal coefficient or rigorously evaluated by an elastic analysis as seen in the following section

T_f, T_i = final and initial temperatures, respectively

ε_c = creep strain

ε_e = elastic strain (if applicable)

ε_s = shrinkage strain.

The location of point Z (point of zero movement in Fig. C. 1) is found by first assuming its location, determining the number and type of supports and their respective stiffnesses, equating the resulting induced horizontal forces on the structure due to movements, and then checking the assumed location.

Let n = the number of supports of any type to the right of point Z, left of point Z, or total

s = number of restrained supports excluding friction supports to the right of point Z, left of point Z, or total

q = number of friction bearing supports to the right of point Z, left of point Z, or total

i = support being considered

Then

$$n = s + q \tag{C.11}$$

Assuming that secondary axial and rotational movements due to the imposed support restraining forces acting upon the superstructure are negligible, the movement at any support to the left of point Z is given by

$$(\Delta_i)_L = \varepsilon_T(X_o - L_i)_L \tag{C.12-a}$$

where L_i is the distance from the support being considered to the left end of the structure. The movement at any support to the right of point Z is given by

$$(\Delta_i)_R = \varepsilon_T(L_i - X_o)_R \tag{C.12-b}$$

For equilibrium the total force to the left of point Z must equal the total force to the right, with due regard for signs. The sign convention shown in Fig. C. 1 assumes positive to the right. Thus, the total force to the left of point Z is

$$(H_n)_L = \overset{s}{\Sigma}(K_i\Delta_i)_L + \overset{q}{\Sigma}(\mu R_i)_L \tag{C.13-a}$$

and, likewise, the force to the right is

$$(H_n)_R = \overset{s}{\Sigma}(K_i\Delta_i)_R + \overset{q}{\Sigma}(\mu R_i)_R \tag{C.13-b}$$

Combining equations (C.12 and (C.13) and simplifying ,

$$\overset{s}{\Sigma}K_i\varepsilon_T(L_i-X_o) + \overset{q}{\Sigma}(\mu R_i)_R - \overset{q}{\Sigma}(\mu R_i)_L = 0 \tag{C.14}$$

Thus the point of zero movement Z measured from the left end of the structure is

$$X_o = \frac{\overset{q}{\Sigma}(\mu R_i)_R - \overset{q}{\Sigma}(\mu R_i)_L + \varepsilon_T\overset{s}{\Sigma}K_iL_i}{\varepsilon_T\overset{s}{\Sigma}K_i} \tag{C.15}$$

The calculated and assumed values of X_o will soon converge. Once the location of X_o is known, the movement of any point on the deck with reference to X_o can then be calculated as the product of unit strain and distance from X_o. The above procedure yields a rational approach to expansion joint design and induced forces.

The location of the point of zero movement for the structure of Fig. C. 8 may be calculated as follows. Calculated and assumed values required for Equation C.15 are listed in Table C. 1.

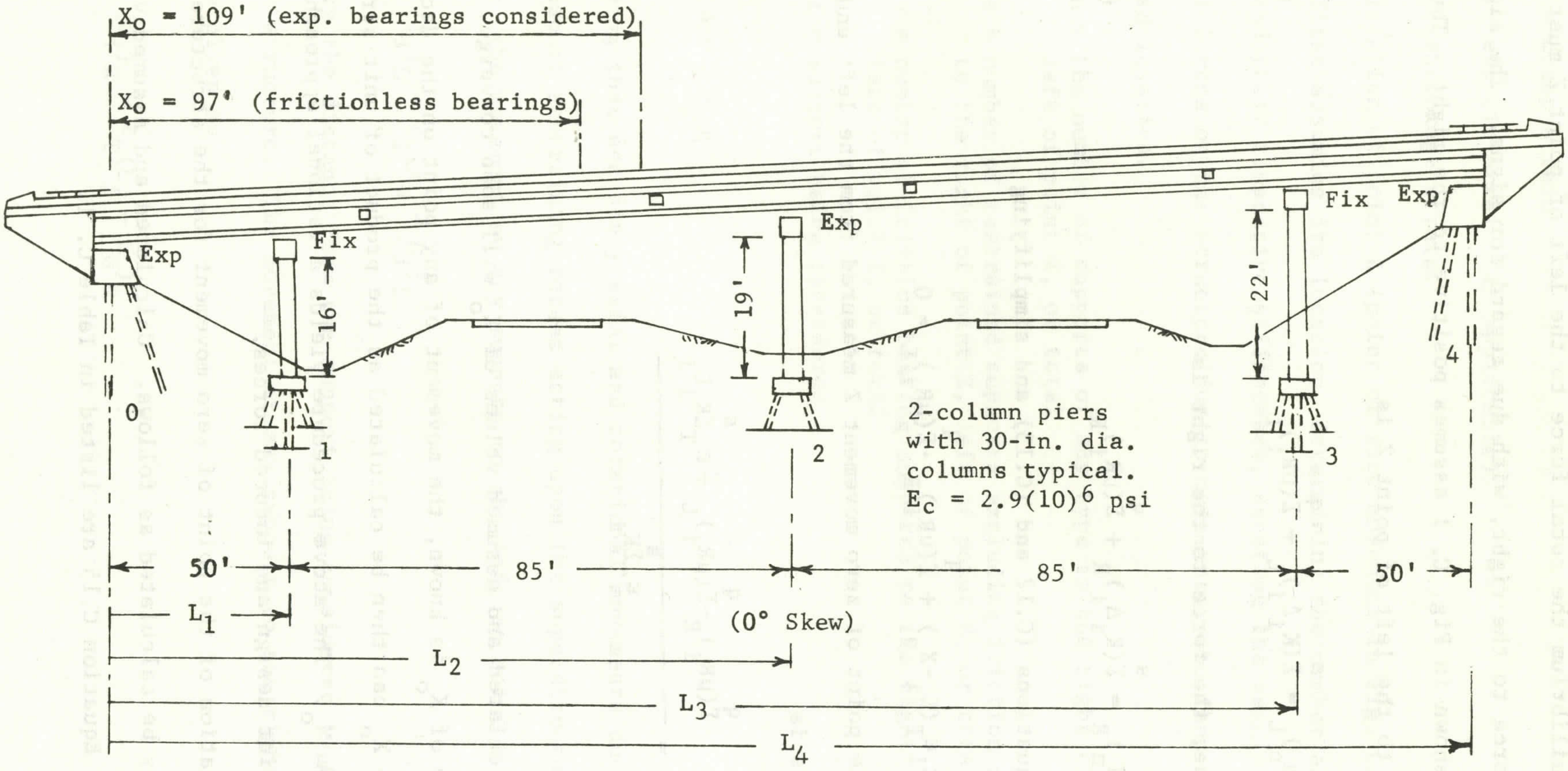


Fig. C. 8. Bridge elevation--illustrative example, determination of point of zero movement

Table C. 1. Calculated and Assumed Values for Example Problem, Location of Point of Zero Movement

Moment of Inertia of Piers	79,522 $in.^4$
Modulus of Elasticity, E_c	$2.9(10)^6$ psi
$K_1 = 3EI/L^3$, 16' Pier	97.7 k/in.
K_2, 19' Pier	58.4 k/in.
K_3, 22' Pier	37.6 k/in.
Coefficient of Friction	0.2
Dead Load Reactions	
Abutments	14 k
1st Interior Pier	66 k
Center Pier	79 k
μR--Abutments	2.8 k
Center Pier	15.8 k
Total Strain, ε_T	0.0002

As a first trial for X_o, assume frictionless bearings and Equation C.15 becomes

$$X_o = \frac{\overset{q}{\Sigma}(\mu R_i)_R - \overset{q}{\Sigma}(\mu R_i)_L + \varepsilon_T \overset{s}{\Sigma} K_i L_i}{\varepsilon_T \overset{s}{\Sigma} K_i}$$

$$= \frac{0 + 0 + 0.0002[97.7(50) + 37.6(220)](12)}{0.0002(97.7 + 37.6)(12)}$$

$$= \frac{13,157}{135.3} = 97' \text{ to the right of the left abutment.}$$

The first trial solution indicates that the point of zero movement will fall within the span to the left of the center pier. Thus two expansion bearings would be to the right of the stagnation point and one expansion bearing to the left.

For the structure shown it may be assumed that the abutment stiffnesses are of such magnitude that impending motion of the bearings will occur. However, before calculating the second trial solution, the magnitude of the hoizontal force that would be developed at the center pier due to the assumed ε_T should be checked, and

$$\begin{aligned} H_{\text{center pier}} &= \varepsilon_T K_2 (L_2 - X_o) \\ &= 0.0002(58.4)(135 - 97)(12) \\ &= 5.2\ \text{k} < (H = \mu R = 15.8\ \text{k}) \end{aligned}$$

Therefore impending motion does not occur and for the second trial solution consider the center pier as a "fixed" pier.

The second trial solution becomes

$$\begin{aligned} X_o &= \frac{2.8 - 2.8 + 0.0002[97.7(50) + 58.4(135) + 37.6(220)](12)}{0.0002(97.7 + 58.4 + 37.6)(12)} \\ &= \frac{21{,}041}{193.7} = 109' \text{ to the right of the left abutment.} \end{aligned}$$

Since the bearing locations did not change position (to the left or to the right) with respect to the assumed point of zero movement, and also inasmuch as the horizontal force induced in the center pier for $X_o = 109'$ would be less than for $X_o = 97'$, the second trial value for X_o is the true value and no further calculations are required.

The distribution of substructure resistance to externally applied superstructure forces, such as braking forces, wind, etc., is dependent upon the substructure and the bearing friction at expansion bearings.

The following takes into account substructure resistance to an applied longitudinal force. The applied force, H, shown in Fig. C. 9 is assumed positive to the right.

For the design of restrained supports, it is conservative to assume that all the resistance is provided by the restrained supports and that

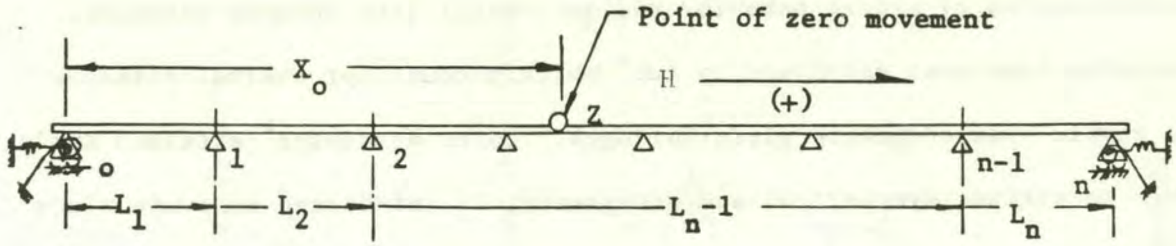


Fig. C. 9. General bridge structure

none is distributed to friction supports. Assuming that the horizontal movement of the deck due to an applied force H is equal at each support, the resulting force at each support can be found by the following expression

$$H_i = \frac{K_i H}{\overset{s}{\Sigma} K_i} \qquad \text{(C.16)}$$

where

H_i = the induced force at the restrained support being considered

K_i = the effective stiffness at support i

$\overset{s}{\Sigma} K_i$ = total substructure stiffness excluding friction bearing supports.

Induced Stresses

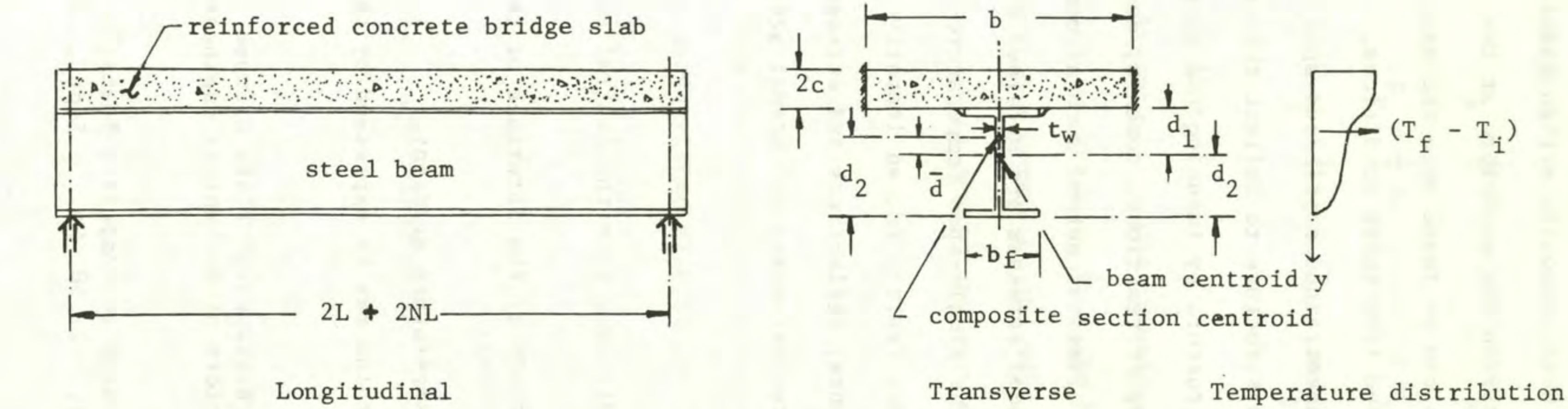
The need for a <u>simple</u> design criteria for restrained structures increases as more bridges are built with superstructures connected to flexible substructures. Additional information is needed concerning: the maximum feasible bridge length between positive expansion devices; the effect of skew; the effect of substructure stiffness on bridge behavior; the major factors influencing bridge movements and the resultant induced stresses. Theoretical relationships are necessary to an understanding of bridge behavior and an insight into induced stresses. Theories have been developed by Zuk[a] which account for thermal stresses in simple span composite girder bridges. Later Berwanger[b] extended Zuk's work to include symmetrical and nonsymmetrical reinforced concrete slabs and indeterminate structures. The development included herein accounts for both restrained and unrestrained simple span and indeterminate composite girder bridges based on an extension of Zuk's and Berwanger's work. The theory is completely general in nature; and even though it is directed towards thermal behavior, other internal effects may be accounted for provided an equivalent temperature distribution is used. (As a simplified example, for a shrinkage strain, ε_s, of say $200(10)^{-6}$ and a coefficient of thermal expansion, α, of $4(10)^{-6}$, equating $\varepsilon_s = \varepsilon_t = \alpha(T_f - T_i)$, an equivalent concrete temperature change for the shrinkage strain becomes $T_f - T_i = \varepsilon_s/\alpha = 50°$ F.

The following theory treats any composite girder bridge subjected to any temperature distribution. The intermediate supports, if any, are first removed, giving the simple span bridge of Fig. C.10. The slab and beam are then assumed separated and free to deform independently in accordance with the imposed temperature distribution. The separated slab and beam strains are easily found from the theory presented. Unknown shears and couples, the only forces possible, exist at the inter-

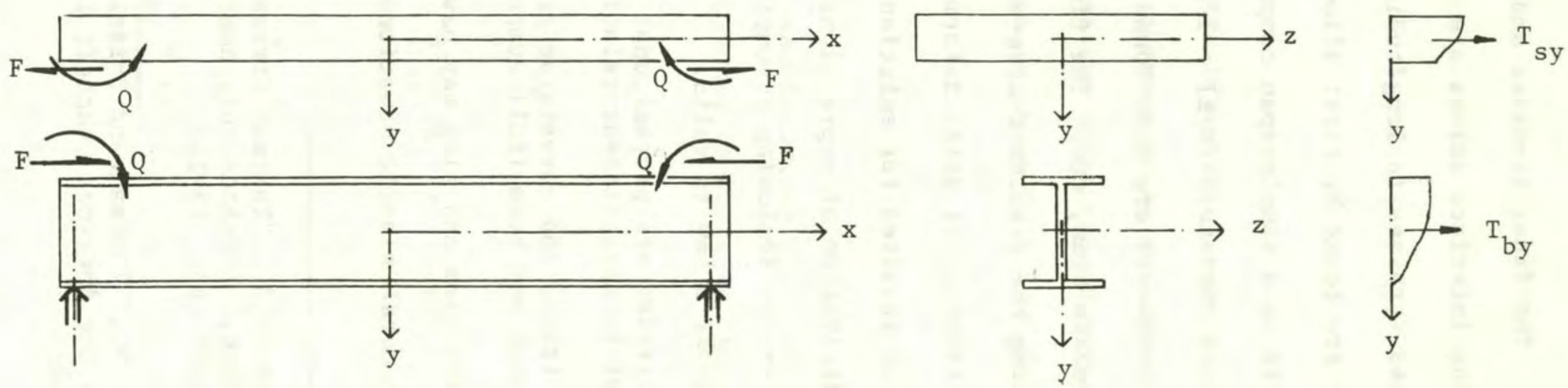
---

[a] Zuk, W., "Thermal Behavior of Composite Bridges--Insulated and Uninsulated," *Highway Research Record No. 76*, Highway Research Record, 1965, pp. 231-253.

[b] Berwanger, C., " A Continuous Composite Steel-Concrete Bridge Prestressed by Deformation at the Interior Supports," *2nd International Symposium on Concrete Bridge Design*, American Concrete Institute Publication SP-26, 1971, pp. 818-873.



a) Simple span composite beam section



b) Separated slab-beam

Fig. C.10. Simple span composite bridge beam subjected to a temperature distribution face between the slab and beam ends.[a, b] When composite action exists, the slab and beam must deform with equal strain and curvature at the interface. The final stresses and strains can be found once the magnitudes of the interface forces are determined from these conditions.

Induced stresses in longitudinally-unrestrained continuous-span structures are found by first allowing the structure to deflect through its supports as a simple-span composite structure by imposing the temperature induced curvature ($M/EI = 1/\rho$). Support reactions, moments, and structure movements are then found by any classical method such as moment area, conjugate beam, etc. The final induced stresses are obtained by superimposing the resultant stresses and the simple-span temperature-induced stresses. If axial deformations are restrained, an iterative procedure is required for solution of moments, deflections and stresses.

The derivation of expressions for stresses caused by thermal action are based on the following assumptions:

1. Hooke's Law is valid.
2. Strains are proportional to the distance from the neutral axis of bending (linear relationship).
3. Strains and curvatures are compatible at the interface of the slab and beam (full composite action).
4. The beam and slab may have any temperature gradient.

The general elastic equations for strains may be expressed by the

---

[a]Aleck, B.J., "Thermal Stresses in a Rectangular Plate Clamped Along an Edge," *Transactions*, American Society of Mechanical Engineers, Vol. 71, 1949, pp. 118-122.

[b]Zuk, W., "Thermal and Shrinkage Stresses in Composite Beams," *Journal of the American Concrete Institute*, Vol. 58, No. 3, Sept., 1961, pp. 327-340.

following:

$$\varepsilon_x = \frac{1}{E}\,[\sigma_x - \mu(\sigma_y + \sigma_z)] + \alpha T \tag{C.17-a}$$

$$\varepsilon_y = \frac{1}{E}\,[\sigma_y - \mu(\sigma_x + \sigma_z)] + \alpha T \tag{C.17-b}$$

$$\varepsilon_z = \frac{1}{E}\,[\sigma_z - \mu(\sigma_x + \sigma_y)] + \alpha T \tag{C.17-c}$$

Slab Strains

Assume the composite bridge structure is a simply supported structure. The slab is separated from the beam and assumed to be restrained against transverse movement by the adjacent beams and unrestrained vertically. The coordinate system and sign convention is shown in Fig. C.10.

$$\varepsilon'_{zs} = 0 \tag{C.18-a}$$

$$\sigma'_{ys} = 0 \tag{C.18-b}$$

Then from the elasticity Equations C.17 and Equations C.18 the longitudinal unrestrained slab strain, when subjected to an actual temperature distribution becomes

$$\varepsilon'_{xs} = \frac{(1 - \mu^2)\sigma'_{xs}}{E_s} + (1 + \mu)\alpha_s T_{sy} \tag{C.19}$$

The longitudinal unrestrained stress σ'_{xs} may be expressed in the form

$$\sigma'_{xs} = \frac{-\alpha_s E_s T_{sy}}{(1 - \mu)} + \frac{P_{xs}}{A_s} + \frac{M_{zs}\, y_s}{I_s} \tag{C.20}$$

where

$$P_{xs} = \int_{-c}^{c} \sigma dA = \int_{-c}^{c} \frac{\alpha_s E_s T_{sy}\, b\, dy}{(1 - \mu)}$$

$$A_s = 2bc$$

and

$$M_{zs} = \int_{-c}^{c} \sigma\, y dA = \int_{-c}^{c} \frac{\alpha_s E_s T_{sy}\, b\, y\, dy}{(1 - \mu)}$$

$$I_s = \frac{2bc^3}{3}$$

Substituting these relationships into Equation C.20 and combining with Equation C.19, gives

$$\varepsilon'_{xs} = \frac{\alpha_s (1 + \mu)}{2c} \int_{-c}^{c} T_{sy}\, dy + \frac{3\alpha_s y_s (1 + \mu)}{2c^3} \int_{-c}^{c} T_{sy}\, y\, dy \tag{C.21}$$

which represents the unrestrained longitudinal strain distribution through the slab. At the interface

$$\varepsilon'_{xs}\Big|_{y=c} = \frac{\alpha_s (1 + \mu)}{2c} \int_{-c}^{c} T_{sy}\, dy + \frac{3\alpha_s (1 + \mu)}{2c^2} \int_{-c}^{c} T_{sy}\, y\, dy \tag{C.22}$$

Since the beam and slab are tied together by mechanical anchors (shear connectors) there must exist restraining forces and moments at the interface (see Fig. C.10). These forces (F and Q) will induce stresses in the slab which can be expressed as

$$\sigma''_{xs} = F/A_s + \frac{(Fc - Q) y_s}{I_s} \tag{C.23}$$

Simplifying

$$\sigma''_{xs}\Big|_{y=c} = \frac{2F}{bc} - \frac{3Q}{2bc^2} \tag{C.24}$$

From theory of elasticity

$$\varepsilon''_{xs} = \frac{1}{E_s} [\sigma''_{xs} - \mu(\sigma''_{ys} + \sigma''_{zs})] \tag{C.25}$$

but $\sigma''_{ys} = 0$ and $\sigma''_{zs} = \mu\sigma''_{xs}$

Combining Equations C.24 and C.25 gives

$$\varepsilon''_{xs}\Big|_{y=c} = \frac{(1-\mu^2)}{E_s bc}\left(2F - \frac{3Q}{2c}\right) \tag{C.26}$$

The total slab strain at the interface is obtained by adding the free slab strain (Eq. C.22) to the boundary restraint strain (Eq. C.26). Thus:

$$\varepsilon_{xs}\Big|_{y=c} = \frac{\varepsilon_s(1+\mu)}{2c}\int_{-c}^{c} T_{sy}dy + \frac{3\alpha_s}{2c^2}(1+\mu)\int_{-c}^{c} T_{sy}y\,dy + \frac{(1-\mu^2)}{E_s\,bc}\left(2F - \frac{3Q}{2c}\right) \tag{C.27}$$

Beam Strains

Again consider the beam separated from the slab such that the stress in the "y" and "z" directions are zero. Then

$$\varepsilon'_{xb} = \frac{1}{E_b}\sigma'_{xb} + \alpha_b T_{by} \tag{C.28}$$

From the analysis presented for the slab (Eq. C.20)

$$\sigma'_{xb} = -\alpha_b T_{by} E_b + \frac{\alpha_b E_b}{A_b}\int_{-d_1}^{d_2} w_y T_{by}\,dy + \frac{\alpha_b E_b y_b}{I_b}\int_{-d_1}^{d_2} T_{by} w_y\, y\, d_y \tag{C.29}$$

For a symmetrical wide-flange beam the width of beam, w_y, is the flange width, b_f, for $y \geq d/2 - t_f$ and the web thickness, t_w, for $y \leq d/2 - t_f$. Combining Equations C.28 and C.29 and evaluating at the interface gives the free beam strain

$$\varepsilon'_{xb}\Big|_{y_b=-d_1} = \frac{\alpha_b}{A_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, dy - \frac{\alpha_b d_1}{I_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy \quad (C.30)$$

Now impose the interface boundary conditions to the beam.

$$\sigma''_{xb} = \frac{-F}{A_b} + \frac{[Q - F(-d_1)]y_b}{I_b} \quad (C.31)$$

since

$$\varepsilon''_{xb} = \frac{\sigma''_{xb}}{Eb}$$

$$\varepsilon''_{xb}\Big|_{y=-d_1} = -\frac{F}{E_b}\left(\frac{d_1^2}{I_b} + \frac{1}{A_b}\right) - \frac{Qd_1}{E_b I_b} \quad (C.32)$$

Again the total interface beam strain is the sum of Equations C.30 and C.32. Thus

$$\varepsilon_{xb}\Big|_{y=-d_1} = \frac{\alpha_b}{A_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, dy - \frac{\alpha_b d_1}{I_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy$$

$$- \frac{F}{E_b}\left(\frac{d_1^2}{I_b} + \frac{1}{A_b}\right) - \frac{Qd_1}{E_b I_b} \quad (C.33)$$

To achieve compatibility the slab strain must be equal to the beam strain at the interface. Equating these strains (Equations C.27 and C.33) gives

$$F\left[\frac{2(1-\mu^2)}{E_s bc} + \frac{d_1^2}{E_b I_b} + \frac{1}{E_b A_b}\right] + Q\left[\frac{d_1}{E_b I_b} - \frac{3(1-\mu^2)}{2\, E_s bc^2}\right]$$

$$= -\frac{\alpha_s(1+\mu)}{2c}\left\{\int_{-c}^{c} T_{sy}\, dy + \frac{3}{c^2}\int_{-c}^{c} T_{sy}\, y\, dy\right\} \quad (C.34)$$

$$+ \alpha_b\left\{\frac{1}{A_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, dy - \frac{d_1}{I_b}\int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy\right\}$$

Curvature

In order to complete the conditions of compatibility it is convenient to compute the curvatures of the slab and beam. The accompanying sketches and elementary moment-curvature relationships are used to establish the following expressions.

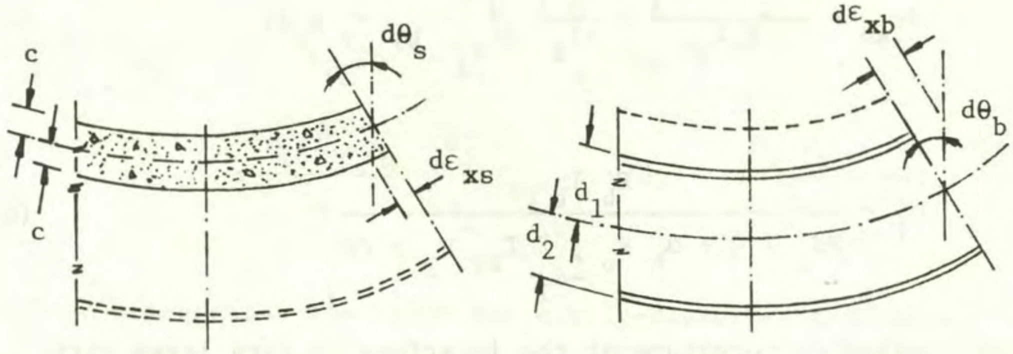


Fig. C.11. Beam-slab curvature

Consider a unit element of the slab, then

$$\frac{c}{\rho_s} = d\varepsilon_{xs}$$

where

$$d\varepsilon_{xs} = [\varepsilon_{xs}\Big|_{y=c} - \varepsilon_{xs}\Big|_{y=0}]$$

Combining Equations C.21, C.25, C.27

$$d\varepsilon_{xs} = \frac{3\alpha_s}{2c^2}(1+\mu)\int_{-c}^{c} T_{sy}\, y\, dy + \frac{3(1-\mu^2)}{2E_s bc^2}(Fc - Q) \qquad (C.35)$$

Then the curvature $\rho_s = \dfrac{c}{d\varepsilon_{xs}}$

or

$$\rho_s = \frac{2c^3 b E_s}{3(1-\mu^2)(Fc - Q) + 3\alpha_s E_s b(1+\mu)\int_{-c}^{c} T_{sy}\, y\, dy} \qquad (C.36)$$

Similarly for the beam curvature relationship

$$\rho_b = \frac{d_1}{d\varepsilon_{xb}}$$

the value of $d\varepsilon_{xb}$ is calculated by combining Equations C.28, C.29, C.31 and C.33 and is given by

$$d\varepsilon_{xb} = \frac{Fd_1^{\,2} + Qd_1}{E_b I_b} + \frac{\alpha_b d_1}{I_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy \qquad \text{(C.37)}$$

then

$$\rho_b = \frac{E_b\, I_b}{Fd_1 + Q + \alpha_b\, E_b \int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy} \qquad \text{(C.38)}$$

The actual radius of curvature at the interface is very large with respect to the centroidal distances of the beam and slab, respectively. Thus, $\rho \simeq \rho_s \simeq \rho_b$ and curvature compatibility may be obtained by equating Equations C.36 and C.38. This gives the second necessary equation for F and Q.

$$F\left[\frac{d_1}{E_b\, I_b} - \frac{3(1-\mu^2)}{2bc^2\, E_s}\right] + Q\left[\frac{1}{E_b\, I_b} + \frac{3(1-\mu^2)}{2bc^3\, E_s}\right]$$
$$= \frac{3\alpha_s(1+\mu)}{2c^3} \int_{-c}^{c} T_{sy}\, y\, dy - \frac{\alpha_b}{I_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy \qquad \text{(C.39)}$$

Equations C.34 and C.39 may be solved for the restraining forces and moments between the concrete slab and girder sections. These forces "F" and "Q" in turn are used to determine the temperature induced stresses in the slab and beam. For the slab

$$\sigma_{xs} = \frac{-\alpha_s E_s T_{sy}}{(1-\mu)} + \frac{\alpha_s E_s}{2c(1-\mu)} \int_{-c}^{c} T_{sy}\, dy$$

$$+ \frac{3\alpha_s E_s y_s}{2c^3(1-\mu)} \int_{-c}^{c} T_{sy}\, y\, dy + \frac{F}{2bc} + \frac{3(Fc - Q)y_s}{2bc^3} \qquad (C.40)$$

and for the beam

$$\sigma_{xb} = -\alpha_b T_{by} E_b + \frac{\alpha_b E_b}{A_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, dy$$

$$+ \frac{\alpha_b E_b y_b}{I_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy + F\left(\frac{d_1 y_b}{I_b} - \frac{1}{A_b}\right) + \frac{Qy_b}{I_b} \qquad (C.41)$$

These expressions are valid for simply-supported composite beams with frictionless bearings. When support restraints are considered, it is necessary to superimpose the additional axial and flexural stresses induced by resistance to elongation. Neglecting elastic shortening due to curvature, the longitudinal strain at the centroid of the composite section is given by (see Fig. C.10):

$$\varepsilon_{xb}\Big|_{y_b=\bar{d}} = \frac{\alpha_b}{A_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, dy - \frac{\alpha_b \bar{d}}{I_b} \int_{-d_1}^{d_2} T_{by}\, w_y\, y\, dy$$

$$+ \frac{F}{E_b}\left(-\frac{d_1 \bar{d}}{I_b} - \frac{1}{A_b}\right) - \frac{Q\bar{d}}{E_b I_b} \qquad (C.42)$$

then

$$\Delta_i = \varepsilon_{xb}\Big|_{y_b=\bar{d}} X_i = \varepsilon_t X_i \qquad (C.43)$$

where X_i represents the distance from the support considered to the point of zero movement, ε_t is as shown in Equation C.10, and Δ_i thus becomes the axial deformation at that support.

Continuous Spans

Economy dictates the use of intermediate supports for bridges over stream crossings and overpasses for dual lane highway systems. The restraining forces introduced at the piers and at the abutments, such as Semi-Integral or Integral end bents, may be calculated by any convenient method of elastic analysis. The resultant stresses are then superimposed upon the temperature stresses induced in the simply-supported composite beam with frictionless bearings discussed previously.

The general procedure for determining the vertical reaction effects in an indeterminate bridge structure is to apply an induced curvature to the longitudinally-unrestrained simply-supported structure. The curvature may be calculated by inverting Equation C.38. The curvature induces deflections and rotations along the bridge.

Vertical reactions must be introduced to maintain consistent deformations at the supports. The moment of inertia of both the steel girder and the composite section is likely to vary in a continuous structure. Such problems may be solved by classical methods; however, numerical techniques utilizing superposition are extremely useful and adaptable to computer solutions. The superposition principle as it might apply to a symmetric four-span composite structure is illustrated in Fig. C.12. Once the reactions and moments are known, the induced bending stresses may be calculated and added to the unrestrained stresses obtained from Equations C.40 and C.41.

When subjected to field loadings, bridge structures with longitudinally unrestrained supports will exhibit movements induced by such factors as elastic shortening, creep, shrinkage, temperature, and externally applied forces. However, structures with longitudinally restrained

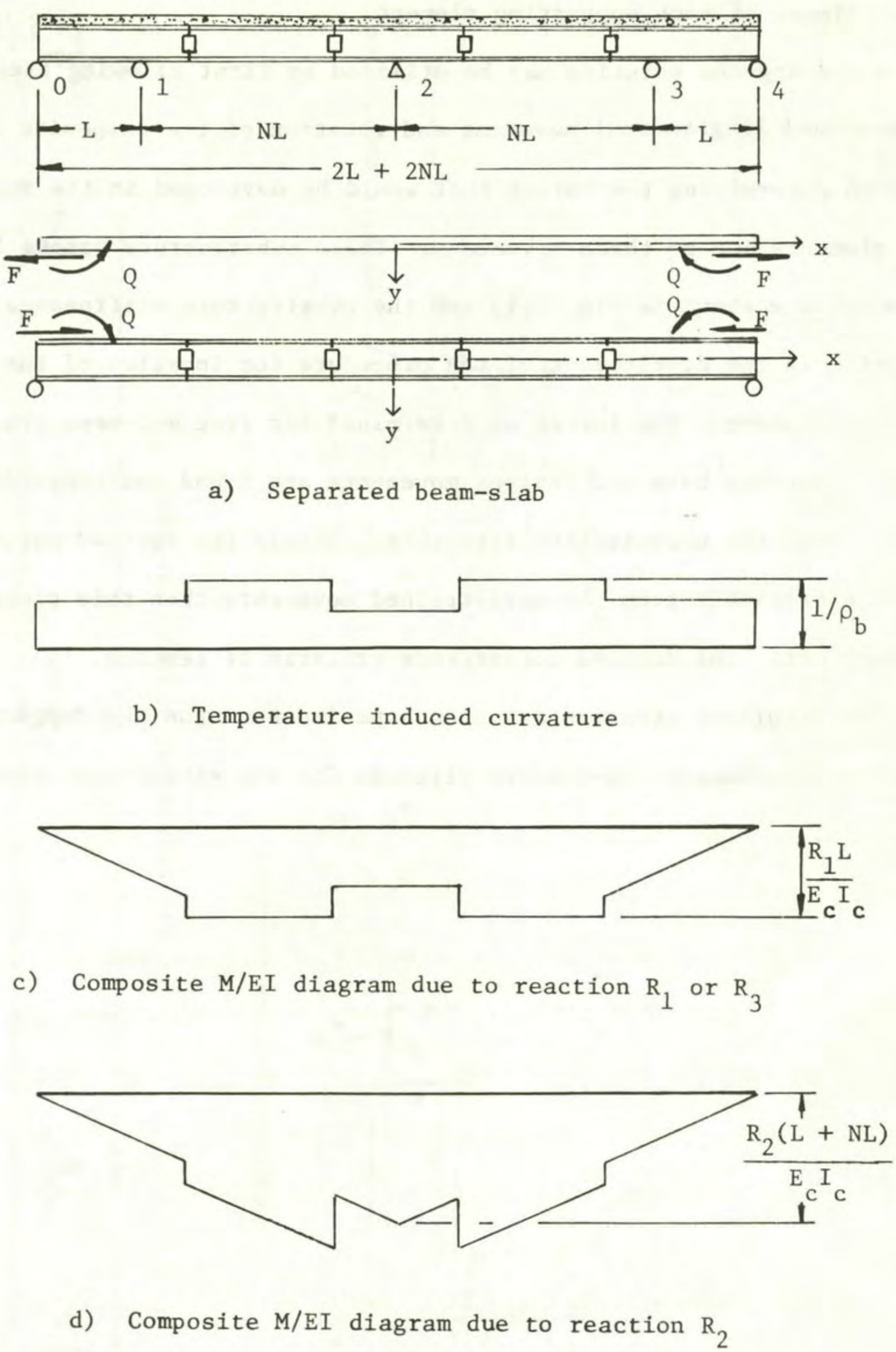
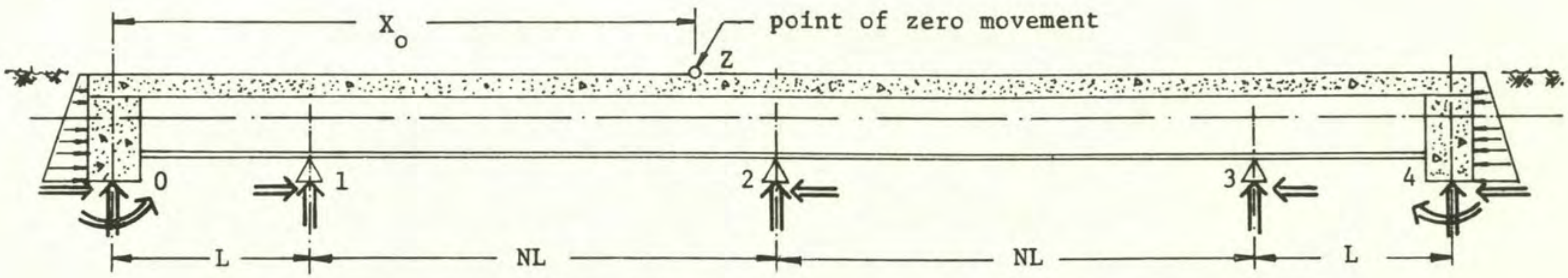


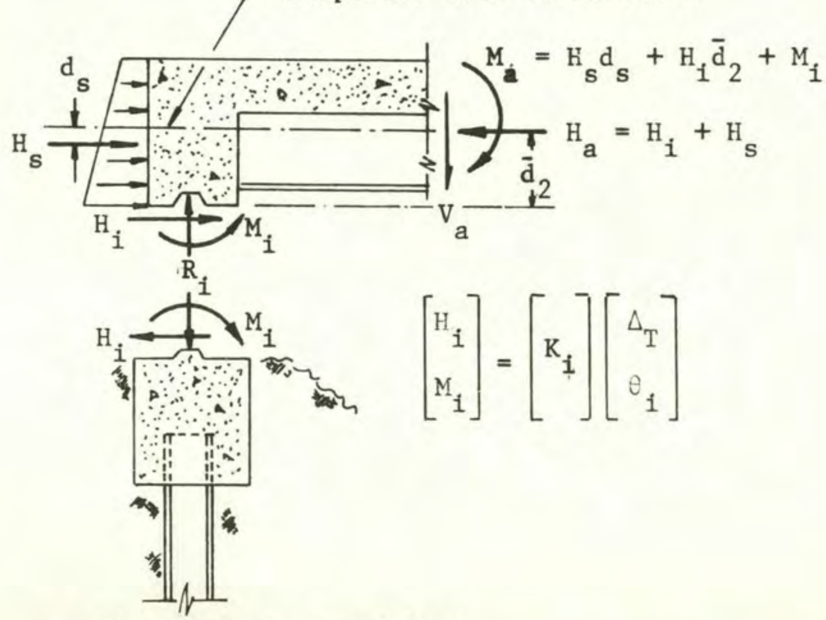
Fig. C.12. Continuous composite bridge beam subjected to a temperature distribution supports will have forces introduced into the substructure as a result of the stiffness of each supporting element.

A convergence solution may be utilized by first allowing free unrestrained longitudinal movement and rotation of the composite beam, and then determining the forces that would be developed in the substructure elements due to these movements. These substructure forces for abutments are shown in Fig. C.13 and the substructure stiffnesses are discussed in the development of the procedure for location of the point of zero movement. The forces as determined for free movement are applied to the composite beam and revised movements are found and compared with movements of the unrestrained structure. Should the revised movements differ appreciably from the unrestrained movements then this procedure is repeated until the desired convergence criteria is reached.

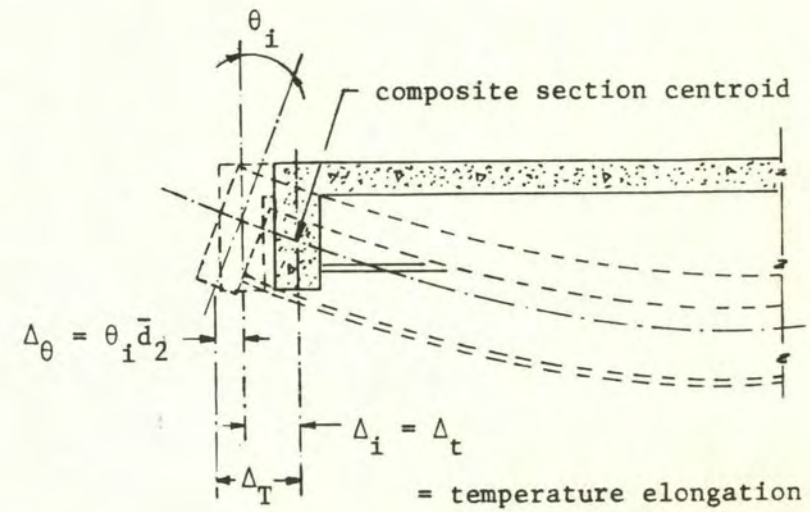
The resultant stresses are then superimposed upon the longitudinally-unrestrained temperature-induced stresses for the simply supported structure.



a) Restrained abutment bridge



b) Restrained end bent



c) Bridge end movement

Fig. C.13. Movements and induced forces in restrained bridges