PERFORMANCE MONITORING OF JOINTLESS BRIDGES

***Phase I Report***

# SYNTHESIS OF TECHNICAL INFORMATION FOR JOINTLESS BRIDGE CONSTRUCTION

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## 1.0 INTRODUCTION

The primary concern in the design of integral bridges[1] is that high stresses can develop in the superstructure and substructure as a result of secondary loads because of the continuity connection between the superstructure and the substructure. These stresses are the result of restrained thermal expansion and contraction, creep, shrinkage, and settlement. The thermal-induced movement of an integral bridge has been shown to induce larger stresses in integral bridge components than live loads (Lawver et al, 2000) and should, therefore, be given careful consideration. The stresses developed as a result of thermal movement are functions of bridge geometry, soil properties, and pile design among others. Because of the complex interaction between the variables affecting bridge performance, especially the soil-structure interaction, the stress levels are difficult to predict. The effects of creep and shrinkage have largely been ignored because their effects tend to act against each other and have been considered small in relation to the effects of thermal movement. Settlement stresses are often ignored as well and are not well understood in integral bridges.

Despite the uncertainty and complexity of integral bridge behavior, many states are adopting the jointless bridge concept. A recent survey received responses from 39 states or provinces (Kunan and Alampalli, 2000). Of those, only eight had no experience with integral abutment bridges, and only Arizona reported poor performance and is no longer constructing integral bridges. Several states, such as California and Tennessee, have constructed more than 1,000 integral bridges over the last 40 years or more and have amassed a history of good performance.

This report provides a synthesis of the research conducted on integral abutment bridges in order to describe the behavior of bridges utilizing the jointless concept. The report focuses on research conducted on bridges with continuous connections of the slab and girders to the abutment wall, as opposed to semi-integral bridges, which do not have a continuous connection between the girders and the abutment. Furthermore, this report is intended to describe the key variables that affect integral bridge behavior and performance. It is not intended to be a summary of current design practices. As this study will ultimately lead to recommendations for bridge instrumentation programs, it was necessary to structure the report content in this manner. The report highlights key variables that may require consideration in a future instrumentation program.

## 2.0 THERMAL STRESS

High stresses can develop in the components of an integral bridge when the structure undergoes the thermal length changes of its bridge deck. Differences often exist in measured and theoretical temperature induced length changes and is one reason why integral bridges in some states have performed satisfactorily even though structural analysis indicated there should have been thermal stress problems. These differences can be attributed to errors in the coefficient of thermal expansion, temperature gradients across the bridge cross sections, and resistance to movement provided by the abutment system and the soil pressure, which depend on the poorly understood soil-structure interaction.

### *2.1 Coefficient of Thermal Expansion*

Some researchers have concluded that differences between the measured thermal length change of a bridge and the theoretical thermal length change can be attributed to a difference between the measured and assumed coefficient of thermal expansion. The coefficient of thermal expansion is a function of cement quality, aggregates, mix proportions, temperature, humidity, and concrete age and, therefore, can

[1] Also called integral abutment bridge (IAB), jointless bridge, rigid-frame bridge, or U-frame bridge.

vary widely from the commonly assumed value of $6.0 \times 10^{-6}$ in/in/°F. Girton et al (1991) found that the coefficient for the bridges in their study ranged from $4.5 \times 10^{-6}$ in/in/°F to $5.0 \times 10^{-6}$ in/in/°F. Construction Technology Laboratories researchers (Oesterle et al, 1998) experimentally determined an average coefficient of $4.9 \times 10^{-6}$ in/in/°F.

Because thermal movements of integral bridges are a key parameter of their behavior, it is important to make an accurate estimate of the coefficient of expansion. The American Concrete Institute (ACI) publication ACI 209R provides an empirical equation to estimate the coefficient of thermal expansion based on the environmental conditions and aggregate characteristics. Methods developed by Emanuel and Hulsey (1977) may also be used to determine an accurate estimate when the concrete mix design is known.

### *2.2 Temperature Range*

There is a lag between the ambient air temperature and the temperature of the bridge components, which affects the design temperature range of the bridge. The lag is primarily affected by the materials used in bridge construction. Hybrid structures (those with steel girders and a concrete deck) tend to follow the extremes of ambient temperature more closely than concrete structures. This can be attributed to the larger thermal mass of concrete structures and the difference in thermal conductivity and diffusivity between steel and concrete. This is reflected in the AASHTO Standard Specifications for Highway Bridges, which recommends larger design temperature ranges for hybrid structures.

It has been suggested that the temperature ranges given by AASHTO are not accurate enough for integral bridge design and that temperature ranges should be determined on a regional or local level. Flores (1994) compared the temperature ranges from several studies to the AASHTO temperature recommendations. Results indicate that the AASHTO temperature ranges are too conservative for steel stringer bridges and may underestimate the temperature range for concrete bridges. The differences in the temperature ranges between hybrid and concrete bridges in the studies were much smaller than those suggested by AASHTO.

Jorgenson (1983) monitored an integral bridge with a concrete deck and concrete box girders to determine, in part, how the lag in bridge temperature affected the design temperature range. He found that the lag was non-linear, with similar bridge and air temperatures recorded at approximately 7 a.m. and 6 p.m. Applying the standard coefficient of expansion to the change in temperature of the concrete deck over a 24-hour period over-estimated the total expansion by more than 100%. The difference was attributed to thermal gradients in the bridge setup, in part, due to lags in heating and cooling across the concrete mass. It should be noted that the report does not indicate if Jorgenson measured the actual coefficient of expansion to determine if a difference between the actual and assumed coefficient contributed to the error. Calculating the change in length of the bridge based on deck temperatures at sunrise each day over the course of a year gave a good correlation to measured length changes. Therefore, Jorgenson proposed calculating the design temperature range as the temperature difference between dawn on the hottest and coldest day of the year and adding one-third of the difference between the maximum temperature and the temperature at dawn during the hottest day of the year.

### *2.3 Temperature Gradients*

Temperatures throughout a bridge structure are not uniform because of varying rates of heat transfer of different materials, heat sources and sinks, and varying exposure to direct solar radiation. The stresses induced by thermal gradients have been compared to those caused by creep and shrinkage (Arsoy et al, 1999 and Burke, 1993) which are typically ignored. However, Hoppe and Gomez (1996) noted that the stresses in steel girders caused by daily temperature fluctuations might be more critical than the compressive forces caused by the restraining force of the backfill. The stresses developed in the bridge components as a result of thermal gradients are affected by the gradient distribution, the relative flexibility of the substructure, and the materials used in the bridge (concrete vs. steel girders). Research has indicated that the stresses can vary widely, even change sign, depending on the previously noted conditions.

There is fairly wide agreement that temperature gradient distributions are nonlinear through the depth of the superstructure (Russell and Girken, 1994). However, a bilinear distribution was measured by Girton et al (1991), who found moderate temperature gradients in pre-stressed concrete girders with a steep temperature gradient in the slab. An analytical model by Sayers (2000) predicted a nonlinear distribution of temperature strains through the depth of the superstructure. He noted that internal strains lead to compressive stresses in the deck and tensile stresses in the girders as a result of the difference between the unrestrained temperature length change and the actual final profile, which is assumed to remain plane, as shown in Figure 1a. This particular case holds true when the coefficient of expansion of the deck is similar to the coefficient of expansion of the girders.

The effect of asphalt topping on the internal temperature distribution is not clear. It seems intuitive to assume that the dark surface will absorb more solar radiation than a concrete surface and, therefore, create a larger thermal gradient in the superstructure. However, it has been reported that asphalt surfacing acts as insulation and actually reduces the maximum temperature in the deck concrete (Potgieter and Gamble, 1989). Elbadry and Ghali (1983) conducted a numerical analysis and found that an asphalt topping increased the temperature-induced stresses in the concrete. Clearly additional study is needed to clarify the effect of asphalt topping.

The superstructure behavior is greatly influenced by the relative stiffness of the abutment. As pile/abutment systems become more flexible, they provide less restraint against induced superstructure curvature, such that gradient-induced axial and bending stresses in the superstructure will typically be lower than expected. Thippeswamy et al (1994) conducted a two dimensional frame analysis of a hybrid bridge and abutment and measured the effects of temperature gradients on the maximum moments. They found that as the flexibility of the abutment increased (i.e. the ratio of superstructure to substructure stiffness increased) the bending stresses at midspan due to the temperature gradient decreased.

When superimposed on the stresses from dead and live load, thermal gradient stresses may reverse the sign of the axial stresses in the superstructure. Thippeswamy and GangaRao (1995) conducted a two dimensional frame analysis of five steel girder integral bridges and found that a temperature gradient of 30°F through the depth of the superstructure produced considerable tensile stresses at the superstructure/abutment joint. Five abutment systems were checked. Systems A, B, and C were spread footing abutments with a fixed base, pinned base with pinned intermediate pier, and pinned base with free intermediate pier, respectively. Systems D and E were pile-supported abutments with piles oriented for strong axis and weak axis bending, respectively. The thermal gradient stresses for the various systems at several locations throughout the structure are shown in Table 1. As shown in Table 1, stress at the bottom of the steel girders at the abutment joint switched from tension to compression when spread-footing abutments were used.

Because full composite action was assumed in the model, the top of the steel girder, which has a larger coefficient of expansion than concrete, could cause tensile stresses in the deck if the gradient and depth of the concrete are shallow enough to allow the free expansion of the steel girder to exceed that of the concrete deck. Compressive stresses would then be present in the top portion of the steel girder. This concept is illustrated in Figure 1b. This stress distribution contrasts with the idealized distribution for a homogeneous deck/girder under a typical temperature gradient, as shown in Figure 1a.

**TABLE 1 - Summary of Thermal Gradient Stresses from Thippeswamy and GangaRao**

| Location | | Stresses (psi) | | | | |
|---|---|---|---|---|---|---|
| | | System A | System B | System C | System D | System E |
| Superstructure/ Abutment Joint | Top | 123 | 934 | 914 | 480 | 479 |
| | Bottom | -8,290 | -8,580 | -8,580 | 7,560 | 7,580 |
| Midspan | Top | -278 | 309 | 309 | 318 | 317 |
| | Bottom | 2,590 | 7,910 | 7,910 | 12,000 | 12,010 |
| Pier | Top | 44 | 452 | 452 | 812 | 813 |
| | Bottom | 9,680 | 14,490 | 14,950 | 10,240 | 10,240 |

*Note: Tensile stress is positive.*

It is not clear why such high compressive stresses were developed at the bottom of the steel girders with negligible stresses in the concrete deck when spread footing systems were used. Compressive stresses could be achieved in the girders if the abutment was sufficiently rigid to restrain the expansion of the composite section, as illustrated by Figure 1c, but compressive stresses might also be expected in the concrete deck. To provide this restraint, the footing and soils would have to be sufficiently rigid to prevent significant rotation and translation of the abutment wall to accommodate the length change of the superstructure. As will be discussed shortly, abutment systems typically rotate and translate to accommodate expansion of the superstructure, so Systems A through C do not likely represent practical field conditions.

Temperature gradients in the superstructure do not have a significant effect on the stresses in the piles or footings, although they may cause the redistribution of load to the piles. Thippeswamy and GangaRao (1995) found that thermal gradient induced stresses were negligible in foundation systems consisting of piles or flexible spread footings. Lawver et al (2000) monitored a 216.5 ft (66 m) long integral bridge with H-piles oriented for weak-axis bending. They observed that the magnitude of the compressive axial strain in the exterior pile decreased while the magnitude of the axial strain in the interior pile increased as the deck temperature increased. They attributed this to thermal gradients across the width of the deck, which caused the redistribution of dead load to the abutment piles.

There are several situations where thermal gradients have typically been ignored, apparently with no adverse side effects. The stresses induced by thermal gradients are often ignored in moderate climates (Arsoy et al, 1999). Burke (1993) reported that the sum of secondary effects is very small in comparison to the load effects for bridges less than 300 feet long and can usually be ignored with the exception of single span bridges and the continuity connection of continuous spans.

### *2.4 Restraint*

The soil behind the abutment wall provides restraint against thermal expansion that will affect the overall length change of the bridge and induce secondary stresses in the structure. However, the restraint provided by abutment wall backfill is usually considered ineffective in reducing the free thermal expansion of the superstructure because the superstructure to abutment stiffness ratio in the direction of bridge expansion is high, and the reactive soil pressure at the top of the wall is often considered low. Observations of integral bridges have confirmed that the soil restraint does not significantly reduce the expansion of the bridge.

Several bridges monitored by Lawver, French, and Shield (2000) exhibited nearly free expansion behavior. The theoretical unrestrained length change nearly matched the measured length change for the

66 meter prestressed concrete bridge investigated when using a coefficient of thermal expansion of $10.5x10^{-6}$/°C (approximately $6.0x10^{-6}$/°F) and the temperature change measured at the girders. The theoretical length change also nearly matched the measured length change in a study conducted by Girton et al (1991). However, it was noted that the AASHTO coefficient of thermal expansion was considerably higher than the experimentally determined coefficient and the temperature range was significantly smaller than the measured range. Elgaaly et al (1992) measured strains in the steel frame of an integral bridge and found they correlated well with free thermal expansion of the bridge. Construction Technology Laboratories (Oesterle et al, 1998) found that abutment and pier restraints (soil restraint) had a negligible effect on thermal expansion of the bridge in their study. Finally, Sayers (2000) conducted a finite element analysis that varied soil stiffness behind the abutment wall and at the level of the piles. The overall change in bridge length was relatively unaffected by the original stiffness of the soils at the piles. He reported that the magnitude of the bridge expansion and hence the longitudinal displacement at the top of the abutment, will be nearly the same regardless of the amount of rotation the abutment must undergo to accommodate the length change.

Although it does not appear that restraint from soils significantly influences the overall expansion of the superstructure, varying soil properties at each end of a bridge can have a significant impact on the distribution of the length change. Jorgenson (1983) measured an abutment movement of 1.96 inches on one side of the bridge while the opposite abutment moved 0.74 inches. This was attributed to the difference in effective soil stiffness. Thomas (1999) also measured unequal abutment movement. The south end of the bridge experienced a decreased rate of expansion as the temperature rose and the north end experienced an increased rate of expansion, but the net expansion of the bridge maintained a linear relationship with the change in temperature. Thomas hypothesized that the non-uniformity was due to increasing backfill stiffness behind the south abutment as the abutment expanded. A similar change in stiffness was not mobilized behind the north abutment.

## 3.0 CREEP AND SHRINKAGE STRESS

The effects of creep and shrinkage are often assumed to have opposite effects and are difficult to quantify, particularly considering the composite construction and continuity of integral bridges. Therefore, creep and shrinkage are commonly ignored in the evaluation and design of jointed bridges. Results from numerical analysis show that the effects can be additive in certain regions of integral bridges that consist of steel girders and a concrete deck.

### *3.1 Creep*

Thippeswamy and GangaRao (1995) conducted numerical analysis of five integral bridges with steel girders designed for composite action with the concrete deck. For pile-supported abutment bridges, creep decreased the compressive dead load stress in the girder at the abutment connection by about 10%. For integral bridges on spread footings, creep decreases compressive dead load stress in the girder at the abutment by about 40%. It should be noted that the live load stresses at the abutment are very low as compared to the dead load stresses. When creep effects are combined with dead and live load, the integral abutment bridge behaves more like a continuous structure with simple supports at the abutment. This is only true for bridges founded on spread footings or piles aligned in weak axis bending. Creep stresses at midspan decreased the top compressive dead load stress nearly 40% and increased the tensile stress at the bottom of the superstructure slightly less than 10%. Over piers, top tensile dead load stress was reduced about 50% and bottom compressive stress increased a maximum of 10%. They found that creep stresses were greater in the fixed spread-footing model but were lower for hinged spread footings and piles. Figure 2 summarizes the stresses from primary loads, creep, and shrinkage at critical areas of the bridge superstructure for a hybrid integral bridge, i.e. steel superstructure with composite concrete deck. As illustrated in the figure, the overall effect of creep is to reduce the stresses in the bridge deck and increase the stresses in the girder.

Siros and Spyrakos (1995) also conducted a numerical analysis of a composite integral bridge. They concluded that the creep stresses in the concrete deck were as high as 26% to 55% of the dead load stresses and the creep stresses in the steel were between 2% and 21% of the dead load stresses. The results of Thippeswamy and GangaRao's (1995) study indicate that creep stresses reduce the magnitude of the stresses in the concrete deck, so the high percentages in the deck are not a concern. However, creep adds to the stress at the bottom of the superstructure. The creep stresses in the steel girders correlated to 1% to 9% of the allowable stress for steel. In concrete girders, the creep stresses are 3% to 42% of the allowable for concrete.

It should be noted that the reduction of top-of-slab compressive stresses at midspan can theoretically be large enough to eventually produce tensile stresses (McHenry, 1943). However, Siros and Spyrakos (1995) reported that this has not been observed in the analysis of several bridges of various span lengths and geometries.

### *3.2 Shrinkage*

Bridges composed of a cast-in-place deck and steel or precast concrete girders will have a non-uniform shrinkage through the depth of the structure. Because the steel or precast girders restrain the free shrinkage of the concrete deck, tensile stresses are developed in the concrete and compressive stresses are developed in the girders at midspan, including regions over piers as long as the superstructure is free to slide over the piers. Research by Thippeswamy and GangaRao (1995) confirms this behavior. Shrinkage stresses were small in the concrete deck, less than 300 psi tensile, and had a much larger effect on the steel girders, up to 4,000 psi compressive. Shrinkage of the deck concrete produced small tensile stresses in the girder at the abutment/ superstructure joint because the slab shrinkage would tend to pull the girder from its integral connection. The free body diagram of the steel girder in Figure 3 illustrates this.

As illustrated by Figure 2, stresses caused by shrinkage in hybrid integral bridges are opposite in sign to those produced as a result of dead and live load towards mid-span and the same in sign as the stresses produced in negative moment regions over piers. At the abutment/superstructure connection, the tensile stresses at the top of the joint are additive and the stresses towards the bottom of the joint are opposite in sign. If the additional tensile stress over piers and at the abutment joints are not accounted for in design, hinges could form as the concrete cracks and the superstructure may behave like a simply supported span at the intermediate pier. As mentioned previously, the flexibility of the pile-supported abutment system allowed the structure to behave like a simply supported span at the abutment connection.

### *3.3 Combined Effect of Creep and Shrinkage*

The assumption that creep and shrinkage forces tend to cancel each other out does not appear to be valid for hybrid integral bridges. As shown in Figure 2, the creep and shrinkage stresses are similar in sign at the top of the superstructure at mid-span and at the bottom of the structure over piers. Additionally, at locations where the stresses are opposite in sign, they are not equal. The combined effects of creep and shrinkage are shown in Figure 2d. The creep and shrinkage stresses are much larger than the primary load stresses in the girder at the abutment and change the stress there from compressive to tensile. Combined tensile stresses in the slab at mid-span reduce the magnitude of the compressive stress from primary loads. The other significant effect is that compressive stress in the girder over piers was increased by 20% by the combined effects of creep and shrinkage.

Descriptions of creep and shrinkage for concrete girder bridges by Burke (1993) indicate that creep has the opposite effect of shrinkage at all locations. As discussed above, this may not be an accurate statement. Shrinkage stresses discussed by Burke (1993) were similar in nature to those described above. Following research completed by Mattock (1961), he noted that maximum shrinkage moments occur within 30 days of form striping with negligible creep effects. Creep effects balance shrinkage effects after 7 to 8 months. After about 2 years, the continuity moments at the abutment connection and vertical reactions (not considering primary loads) are reversed.

There is limited information available on the shrinkage and creep stresses in the foundation system. Thippeswamy and GangaRao (1995) indicated that shrinkage stresses at the foundations were negligible for all footing systems except for a fixed spread footing.

### *3.4 Settlement Stress*

Settlement of the abutments of an integral bridge is typically a result of cyclic loading of the soil and the vertical loads on the abutments or approach slab. Washout and scour also cause settlements in bridges spanning bodies of water or bridges with a defective means of preventing significant amounts of water from infiltrating the backfill. The effects of settlements on the stresses in integral bridges have not been thoroughly studied, and the available information is somewhat conflicting.

A study by GangaRao (1981) determined that, for two and four span steel stringer bridges, a differential settlement of one inch or more would produce unacceptable stresses for spans of up to 50 feet. Effects of a 3 in. settlement were small for bridges with 100 to 200 foot spans and negligible for bridges with spans greater than 200 feet. He concluded that the settlement stresses in a single span bridge were insignificant and could be disregarded in analysis and design.

Integral bridges supported by spread footings appear to develop high stresses in the superstructure at the abutment joint whereas bridges supported by flexible piles develop much lower stresses under the same settlement. It is not entirely clear why the same settlement produces different stresses in the superstructure, although it is likely related to the stiffness and deflected shape of the abutment system. Thippeswamy and GangaRao (1995) conducted a numerical analysis and reported that the stress in the superstructure at the abutment due to spread footing settlement was nearly a quarter of the dead load stress and had the same effect as primary loads. The settlement stress was negligibly small at the superstructure/abutment joint for bridges on piles and at the bottom of the abutment for all systems.

Information presented by Burke (1993) conflicts with the suggestion that settlement stresses have the same effect as primary loads. Burke (1993) stated that settlement of abutments of multiple span integral bridges induces bending moments in the superstructure similar to those induced by shrinkage, and settlement of piers induces moments similar to those induced by creep. In both cases, the moments at the piers are opposite to and substantially greater than those at the abutments. Greimann et al (1986) also conducted a numerical analysis of the effects of abutment pile settlement. The load-settlement curves were unaffected by pile orientation, thermal movements of the superstructure, or the presence of backfill behind the abutment wall. After about 0.25 inches of vertical settlement the piles were unable to carry substantial additional load.

## 4.0 SKEW AND CURVATURE

Skew and curvature are problematic for bridges of all types. The mechanics of skewed bridges are fairly well understood, but much less is known about the effects of curvature. Most states that use integral bridges have adopted an upper limit skew of 20 to 30 degrees based on performance data of in-service integral bridges. In the past decade, several studies have investigated the effect of bridge skew through experimental and analytical methods to provide refined design guidelines. It has been found that higher skew angles result in lateral displacements of the abutment wall towards the acute side of the bridge with corresponding high stresses in the superstructure and substructure nearest the obtuse corners.

### *4.1 Mechanics*

The skew of a bridge, abutment wingwalls, and length-to-width ratio of the bridge affect the magnitude of transverse movement of the abutment. Figure 4 illustrates the reaction forces acting on an expanding bridge. Because the abutment walls on skewed bridges are not perpendicular to the centerline of the bridge, the resultant soil pressure force has a component normal to the abutment wall and a component parallel to the abutment wall. The normal force couple acting on the bridge will tend to cause rotation of the abutment and superstructure about a vertical axis such that the acute corners of the abutment walls move away from the soil (clockwise in Figure 4). The force-couple from frictional resistance of the soil causes rotation in the opposite direction and balances the reactive pressure force couple. The frictional resistance consists of the friction force between the backfill and the abutment wall and the internal shearing resistance of the backfill. Usually the frictional resistance between the wall and the soil is smaller than the internal shearing resistance of the soil and will, therefore, control behavior of the backfill interface (Burke, 1994). If the frictional force of the soil is exceeded, the force couple will be unbalanced and the bridge will rotate.

As the skew increases, the soil pressure reaction force due to thermal expansion of the bridge has a larger component in the transverse direction. Thus, as the skew of a bridge increases, the frictional resistance is more likely to be exceeded. The literature indicates that skew angle greater than 15 to 20 degrees cause forces sufficient to move the abutments (Burke, 1994; Sayers, 2000). Burke showed that skews greater than 15 degrees cause instability and require guide bearings for semi-integral bridges with a backfill angle of friction of 22 degrees. He makes a valid case that the force required to stabilize rotation of the abutments on a 30 degree skewed abutment approaches 50% of the passive pressure on the abutment wall and 70% for a 45 degree skewed abutment. These values far exceed the frictional resistance of the wall or the shearing resistance of the soil.

If the frictional resistance between the soil and the wall is exceeded, the piles must resist the deficit force parallel to the abutment wall and the wall will move laterally as the piles deflect. The lateral deflection of the piles is affected by the magnitude of the forces, the stiffness of the piles and soil, and the pile-soil interaction. Piles will be biaxially stressed if the web of the pile is parallel or perpendicular to the abutment wall. For piles with webs perpendicular or parallel to the centerline of the roadway, the pile stresses are similar to bridges without skew (Greimann et al, 1983).

It should be noted that longer abutment walls (wider bridges) offer greater frictional resistance to lateral movement. Wingwalls can also act to restrain the soil and may provide additional transverse support depending on the abutment to wingwall connection, the wingwall orientation, and the wingwall interaction with the soil.

The behavior of curved girder bridges is more unpredictable than straight skewed bridges. It is unclear if the overall thermal movements follow the chord between abutments, the chord between consecutive supports, or act perpendicular to the abutments. Field measurements (Roeder and Moorty, 1991) have indicated that thermal expansion can be radial depending on the slenderness of the supporting piers.

### *4.2 Distribution of Stresses*

Soil pressures from thermal loads and live loads are greatest at the obtuse sides of a skewed bridge because of the rotation of the abutment wall, which tends to push the obtuse side into the soil and pull the acute side away. It should be noted that the pressure at the acute side is still likely to be positive because of the overall elongation of the bridge.

Sandford and Elgaaly (1993) and Elgaaly et al (1992) observed stresses on the upper part of the abutment wall at the obtuse corner of a 20 degree skewed Forks Bridge in Maine that were nearly three times the value at the acute corner. They also noted that the pressures caused by thermal expansion were of the same magnitude or higher than the live load response. During periods of expansion, the pressure on the obtuse side increased four to six times the cold weather value, whereas the acute side pressure increased two to three times. These differences occurred seasonally as a result of long-term temperature changes and were relatively unaffected by short-term variations, indicating that the difference was not a reflection of non-uniform temperatures in the structure. The pressure differences on cells 6 feet lower on the abutment wall were categorized as almost negligible. The effect of skew also appeared to be diminishing with each annual cycle so there was no indication that cyclic stiffening of the soil was increasing pressure differences.

The live load response of integral bridges also appears to cause higher stresses at the obtuse corner of a bridge. Elgaaly et al (1992) observed that the soil pressure near the obtuse corner side of the abutment wall increased while the acute side soil pressure decreased during live load tests. Dagher, Elgaaly, and Kankam (1991) concluded that the shear and moments from primary loads were relatively high near the obtuse corner of the slab. The report also pointed out that slab cracking had recently been observed near the obtuse corner of an integral bridge in Maine.

Thomas (1999) measured lower pier strains at the obtuse angle side of a 30 degree skewed bridge when compared to the acute angle side. Although the soil pressure behind the abutment wall was not measured, the lower pier strains indicate that there was more restraint against longitudinal movement at the obtuse side and, therefore, higher soil pressures at the obtuse side.

## 5.0 <u>ABUTMENT WALL-SOIL INTERACTION</u>

The soil-structure interaction has the largest single influence on the behavior of integral bridges. Unfortunately, it is also the most difficult to accurately predict because the reactive soil pressures are a non-linear function of the magnitude of the displacement and deflected shape of the wall, and the deflected shape of the wall is a function of soil pressures. Variables that affect the soil-abutment interaction include abutment wall, pile, wingwall, approach slab, and pile configurations; soil characteristics (primarily soil stiffness); total movement; and superstructure stiffness among others.

### *5.1 Abutment Movement*

Abutment movement is the result of two effects: the thermal movement of the superstructure and the temperature dependant volumetric expansion of the pile cap concrete. The volumetric expansion of the pile cap concrete is small in comparison to the thermal expansion and contraction of the bridge and is usually ignored. Thermal expansion of the bridge pushes the abutment wall into the soil, which affects the earth pressure on the abutments, piles, diaphragms, and wingwalls, and causes movement of the approach slabs. Thermal expansion behavior of the superstructure was discussed earlier. The abutment walls can also translate laterally, as described in the discussion on bridge skew.

It is important to understand the deflection behavior of abutment walls to arrive at an understanding of the soil-abutment interaction. Thermal gradients in the bridge superstructure and the eccentricity of the soil pressure resultant force with respect to the height of the superstructure produce forces that tend to rotate the abutment as shown in Figure 5. Research indicates that a typical continuity connection of the superstructure and abutment, where the superstructure is cast integrally with the abutment wall, is sufficiently rigid to prevent large rotations. Lawver et al (2000) measured abutment rotations less than 0.06 degrees. This was further corroborated by evidence of double curvature in the piles which is expected when the abutment walls translate horizontally with little or no rotation. Thomas (1999) measured abutment rotations between 0.06 degrees and 0.075 degrees for both 15 degree and 30 degree skew bridges.

Differences in the abutment geometry at each end of an integral bridge will have an impact on the behavior of the bridge, although very little has been published on this topic. The Massachusetts Highway Department (Crovo) reported that a difference in abutment heights caused an unbalanced lateral load resulting in sidesway at the abutments, which should be considered in the design by balancing the earth pressure.

### *5.2 Soil Pressure*

Soil pressures on abutments should be considered carefully. Bending moments induced by passive pressures on abutments counteract the dead and live load bending moments in simple spans. Therefore, overestimating earth pressure is not necessarily a conservative approach if the bridge deck behaves as a simple span. For continuous span bridges, the negative moments are increased at abutments and are reduced at piers for two and three span bridges, and center span positive moments of three span bridges are slightly increased (Burke, 1993).

It has been reported that many studies agree that near full passive pressure occurs against the abutment wall and piles during periods of bridge expansion (Ting and Faraji, 1998). However, the literature indicates that a wide range of pressures have been measured at various depths on the abutment system. Comparing results is often difficult because of the many variables in soil properties and abutment designs. The Massachusetts Highway Department (Crovo) found that near the ground surface, full passive resistance was nearly achieved for a thermal movement of 0.5 inch at each footing-supported abutment. The backfill density was approximately 135 pcf. At greater depths, the full passive pressure was approached even for small horizontal displacements. Burke (1993) recommended that only 2/3 of the full passive pressures be used in modest length bridges and that the passive pressure can be ignored completely in short single and multiple span bridges.

Sandford and Elgaaly (1993) measured soil pressures on a 7-meter high abutment wall resting on spread footings. The pressure approached at rest pressures during periods of contraction. During periods of expansion, combining the passive pressure distribution over the top third of the wall with a transition to the active pressure case at the base of the wall gave a conservative envelope for the measured soil pressure. This recognizes the larger movement of the top of the wall into the backfill and the lack of movement at the bottom of the wall.

Because the soil pressure is dependent on the deflected shape of the wall, the pressure distribution is inherently non-linear, although a linear distribution is often assumed to simplify the design. A typical soil response is shown in Figure 6. The Massachusetts Highway Department (Crovo) found that the earth pressure distribution generated by passively displacing a shallow foundation supported prototype abutment was approximately parabolic in shape for all wingwall configurations and relative displacements. However, compaction earth pressures could be idealized as linearly increasing with depth.

Numerical analysis conducted by Wood and Nash (2000) indicates that the shape of the soil pressure distribution is controlled by the relative stiffness' of fill and abutment while the magnitude of the stress distribution is primarily controlled by the stiffness of the fill. As the ratio of the soil stiffness to abutment stiffness increased, the peak of the stress distribution moved towards the top of the wall. A nine-fold increase in the fill stiffness increased the maximum moment in the abutment wall by a factor of 1.5, and a ten-fold decrease in the abutment stiffness produced a reduction in moment by a factor of about two (2). Thus, there was a nonlinear relationship between relative stiffness and resultant abutment moment. Other parameters, such as soil friction angle, had very little effect on the earth pressures or abutment moments.

### *5.3 Soil Stiffness*

Research indicates that soil stiffness, as opposed to soil strength, is the most significant factor influencing soil pressures on the abutment system. The compaction level of a soil is a strong indicator of the soil stiffness, so soil density will be used interchangeably with soil stiffness here. Denser soils provide more resistance to thermal movement and develop higher pressures on the abutment system as a result. This introduces higher axial forces into the superstructure and also causes more rotation of the abutment, thereby influencing moments in the superstructure.

Several research studies have attempted to quantify the change in soil pressure when soils of varying densities are used. The Massachusetts Highway Department (Crovo) utilized a nonlinear three dimensional analysis to model a 45.6 meter steel stringer bridge with integral abutments supported on H piles oriented for weak axis bending over a temperature range of 44.4°C. The axial force and moment in the superstructure more than doubled when compaction was varied from the loose to dense soil compaction range. Field tests of pile supported abutments showed that the maximum moment in the piles increased by a factor of about 1.75 when dense soil was used in lieu of loose soil.

Construction Technology Laboratories (Oesterle et al, 1998) determined that backfill slope had a significant influence on soil pressures, in addition to soil compaction. A finite element analysis indicated that a decrease in soil compaction from 90% to 80% decreased the passive pressure by a factor of 2.5. A decrease in the slope of the in-situ soil behind the abutment wall prior to backfilling from 45 to 30 degrees decreased the passive soil pressure by a factor of 2.

As these results indicated, the soil stiffness can have a significant influence on the rotation of the wall. Numerical analysis conducted by Ting and Faraji (1998) resulted in the abutment translating 35% less and rotating 67% more for dense backfill than for loose backfill. Sayers (2000) attempted to match the abutment rotations from finite element models to those measured in the field, which were smaller than the model results. He tried applying a horizontal restraining force at the deck to represent approach slab restraint, reducing the temperature gradient through the superstructure, reducing the coefficient of expansion between the deck and steel girders, and using different backfill pressures other than the assumed triangular distribution. Approach slab restraint had a negligible effect on abutment rotation. Reducing the temperature gradient or the difference in the coefficient of thermal expansion slightly reduced the rotation, and changing the linear pressure distribution did not correct the problem. Nonlinear distributions were not attempted, but it was suggested that they be investigated in the future. Lowering the stiffness of the soils in a second model gave a good correlation with field measurements.

## 6.0 PILE-SOIL INTERACTION

The interaction between the piles and the soil is similar to the interaction between the abutment wall and the soil, but research shows that the pile-soil interaction has much less influence on the overall behavior of the bridge than the pile-abutment interaction. Therefore, the pile-soil interaction primarily influences only the stresses in the piles. Soil stiffness and the deflected shape have significant impacts on the stresses developed. Piles in soils with sufficient friction and bearing capacity will deflect laterally. This action results in a reduction in the vertical load carrying capacity, particularly in dense soils. Pre-drilled holes filled with loose soil have been used as an effective solution to prevent this capacity reduction.

### *6.1 Soil Stiffness*

Stiff soils have been shown to induce large bending stresses in piles. This may not seem intuitive because it would seem that stiff soils would restrain movement of the piles, and smaller deflections naturally result in smaller bending moments. However, research indicates that the soil-abutment wall interaction has much more influence than the soil-pile interaction on the overall deflection at the pile/pier cap interface. Therefore, if it can be assumed that the lateral deflection at the top of the pile is constant regardless of the type of soil next to the piles, the induced curvature in the piles is more severe in dense soil, as illustrated by Figure 7.

The Massachusetts Highway Department (Crovo) showed through numerical analysis that the level of soil compaction adjacent to the piles did not have a significant influence on the wall or superstructure moments or deflections. However, when dense soil was in place behind the abutment wall, a change from loose soil to dense soil at the level of the pile increased the pile moment by a factor of about 1.75. In all cases, the peak moment occurred at the abutment-pile interface.

A finite element study by Faraji et al (2001) achieved similar results. The magnitude of the axial force in the superstructure varied significantly depending on the compaction level of the soil behind the abutment wall, but was only slightly affected by the soil compaction at the piles. The compaction of the soil both next to the piles and behind the abutment had a significant influence on the moment in the HP piles.

Several studies show agreement that pile stresses are most affected by the soil stiffness towards the top of the pile. This is not unexpected, since there is typically less lateral movement at the bottom of the pile. Jorgenson (1983) conducted a numerical study of piles oriented for strong axis bending and found that doubling the modulus of subgrade reaction on the bottom half of a pile while holding the modulus of subgrade reaction at the top half of the pile constant had less than a 5% influence on the maximum moment. For the maximum measured abutment movement of 1.96 inches, the stress at the top of the pile exceeded the yield stress of the steel but was insufficient to cause formation of a full plastic hinge. Sanford and Elgaaly (1993) measured little to no seasonal change in pressure towards the bottom of the abutment wall. Changes were thought to be due to varying effective pressure under changing water levels. Numerical studies by Kamel et al (1995) showed that the soil type along the bottom 50 feet of a 60 foot long concrete pile with a hinged top had little influence on the pile stress when using a 10 foot deep upper layer of loose sand. Laboratory pile tests by Kamel et al (1996) confirmed that maximum lateral displacements for a given pile were dependent on the stiffness of the soil in the upper 10 feet of pile. Soil stiffness below this depth had a negligible effect.

On a final note, frost pressures should be given careful consideration, particularly if the water table is high. Elgaaly et al (1992) measured a significant effect from frost build up even though the soil was free draining and considered a frost-free material. The mechanism for frost build up appeared to be related to the change of the river water level. Thus, bridges spanning bodies of water should be given careful consideration.

### *6.2 Failure Mechanisms*

Pile failure is categorized into one of two mechanisms. The first is a slip mechanism where the ultimate frictional resistance of the soil is exceeded and the pile settles while remaining essentially undeformed. The second is a lateral displacement mechanism where the pile fails due to the interaction of geometric instability (Euler buckling) and the development of a plastic hinge. Lateral soil failure, where the pile remains essentially undeformed, is also sometimes included in this mechanism.

The failure mechanism that will control depends on the magnitude of the lateral displacement, the pile embedment length, the soil density, and the pile stiffness. Greimann and Wolde-Tinsae (1988) developed a design model and finite element analysis to study the load capacity of end-bearing piles and friction piles bent about the weak axis with the pile head fixed against rotation. The slip mechanism tended to control for friction piles with small displacements and the lateral mechanism tended to control for end-bearing piles and friction piles with large displacements. The ultimate vertical load for the lateral failure mechanism decreased with increasing lateral deflection.

A description of the models and design equations developed to predict the failure mechanism and the ultimate strength is beyond the scope of this report. A series of studies funded by the Iowa Department of Transportation have been performed at Iowa State University and have been published in various articles by Wolde-Tinsae, Greimann, and Yang. Flores (1994) provides a good summary and description of the work performed there.

### *6.3 Capacity Reduction from Lateral Displacement*

The ability of piles to accommodate lateral displacements from secondary loads is a significant factor in determining the maximum length of integral bridges. As piles deflect under abutment movement, the axial load capacity is reduced for a number of reasons. First, pile-bending stresses from movement will be superimposed on axial stresses, thereby reducing the axial load capacity. A number of research projects have demonstrated that steel piles are subjected to superimposed movement stresses plus axial stresses that exceed the yield stress of the steel (Lawver et al, 2000; Girton et al, 1991; Jorgenson, 1983; Thomas, 1999). Secondly, movement of the pile could affect the behavior of the soil, such as the ability of the soil to carry vertical load through frictional resistance. Lastly, an additional vertical force, or thermal load, will be introduced into the bridge-pile system in order to maintain static equilibrium as shown in Figure 8. A moment, M, and shear, V, are introduced because of the eccentricity between the soil pressure resultant force and the elevation of the superstructure. Greimann et al (1986) created a mathematical model for piles in dense sand that predicted an additional 20 kip force for a 60°F thermal temperature change. Wolde-Tinsae et al (1987) used the same model to predict a pile thermal load of 46 kips for a 400 foot steel bridge with very stiff clay soil and a temperature range of 150° F. The state of Maine enclosed an example design in response to a survey conducted by Franco (1999) where the thermal load contributed to 24% of the total vertical load on the pile.

The relationship between the pile head displacement and the reduction in vertical load carrying capacity is not clear because of the pile interaction with the soil and other variables involved, such as pile stiffness. A study conducted in 1973 by South Dakota State University (Lee and Sarsam, 1973) conducted full-scale model tests on integral abutments with HP 10x42 piles driven into silty clay over glacial till. Strains corresponding to stresses of up to 42 ksi were measured just below the bottom of the concrete abutment for lateral displacements of 1 inch. They concluded that horizontal movement greater than 1/2 inch would cause yielding of the piles. The literature did not indicate the orientation of the piles.

Research conducted at Iowa State University and reported by Wolde-Tinsae et al (1982) suggested that the vertical load capacity of displaced piles is reduced in very stiff soil because the soil is sufficiently stiff to force formation of a plastic hinge. The numerical research showed that the vertical load-carrying capacity of an H pile was not significantly affected by lateral displacements of 2 inches in soft and stiff clay, and in loose, medium, and dense sands. In very stiff clay (average blow count = 50), piles failed by elastic-plastic buckling and the vertical load-carrying capacity of the pile was reduced by about 50% for a 2 inch displacement and 20% for 1 inch displacement.

However, further development of design models by Greimann et al (1984), Wolde-Tinsae et al (1988), and Greimann and Wolde-Tinsae (1988) showed that the vertical load capacity of piles was not significantly affected by displacements of up to 4 inches for steel H piles and up to 2 inches for concrete and timber piles in six different soil types. Greimann and Wolde-Tinsae reported that both their design method and the selected finite element program predicted only a slight reduction in the ultimate vertical load capacity of HP bearing piles in strong and weak axis bending as pile head deflections increased to 4 inches. The capacity was reduced in all of the soils tested, but there was not a strong correlation between the magnitude of the reduction and the soil type. Their design method predicted that plastic buckling of the piles became increasingly dominant over elastic buckling as the stiffness of the soil increased.

### *6.4 Pre-drilled Holes*

Many states use predrilled oversize holes filled with loose granular soil to reduce the stresses in the piles. An additional feature of predrilled holes is reduction of downdrag forces when compressible soil is present or minimization of the effects of elastic shortening when prestressed concrete superstructures are used.

Numerical analysis and laboratory pile tests conducted by Kamel et al (1995 and 1996) revealed that placing piles in 10 foot deep holes filled with loose sand significantly increased the lateral displacement that could be achieved before the maximum allowable lateral load was reached. Yang et al (1985) demonstrated that pre-drilling holes to replace stiff soils with loose sand greatly increased the vertical load carrying capacity of piles. The predrilled length of the holes was a significant factor. For a 10x42 steel H pile, 6 to 10 feet of length was necessary to take full advantage of pre-drilling. Wolde-Tinsae et al (1987) also modeled predrilled holes in very stiff clay conditions. Using predrilled holes and H piles in weak axis bending reduced the axial load in the girder by 25%, reduced the pile pre-load by 30%, and reduced the maximum girder stresses by 30%. Abutment deflections increased about 5%.

The Massachusetts Highway Department (Crovo) sponsored research to evaluate the response of a full-scale integral abutment bridge prototype. The original soils were composed of a 2 foot layer of gravel and pulverized asphalt and a 4 to 6 foot layer of moderately to heavily overconsolidated clay over normally consolidated clay. Backfill with a target density of 135 pcf was placed from the top of the abutment wall to a point 1 to 2 feet above the bottom of the wall. They determined that the magnitude of the earth pressure curve for the pile-supported foundation with compacted gravel borrow was greater than the design curves published by the National Cooperative Highway Research Program (NCHRP) for dense sand. However, the use of a 600 millimeter wide loose sand cushion between the abutment and the compacted gravel borrow gave results of earth pressure magnitude and stiffness that were comparable to the medium dense curve published by NCHRP.

## 7.0 FLEXIBILITY OF THE SUBSTRUCTURE

Integral abutment bridges rely on the flexibility of the substructure to accommodate thermal movements. The pile orientation, superstructure to substructure stiffness, and the abutment system boundary conditions have been shown to have a significant effect on the behavior of integral bridges. A properly designed pile system will accommodate superstructure and abutment movement by flexure near its top and will be sufficiently strong to withstand passive soil pressures, any pressures generated by traffic on the approach slab, and vertical loads. The relative flexibility of the substructure, which is affected by the boundary conditions, will influence the distribution of stresses in an integral bridge.

### *7.1 Pile Orientation*

If the abutment piles are overly stiff, the full passive pressure might be developed along the entire length of the pile (the pile will move as a rigid body into the soil) and high stresses will develop in the pile and superstructure. Overly stiff abutment systems can also restrain the deck sufficiently and cause shrinkage related damage (Burke, 1990). Therefore, the overwhelmingly preferred pile arrangement is a single row of piles under the abutment wall.

The majority of state agencies and researchers have concluded that weak axis bending of the piles is preferable to strong axis bending, although states that use strong axis bending have reported that the piles have performed satisfactorily. Piles oriented for strong axis bending are less susceptible to flange buckling, but piles in weak axis bending provide less resistance to thermal deformations and develop smaller bending stresses for a given displacement. Wasserman and Walker (1996) noted that reducing the bending stresses in the piles makes it easier to achieve fixed pile head behavior. Also, the lower stresses will reduce the likelihood of fatigue damage to the piles from the displacement cycles. If fatigue cracking occurs, it will occur at the tips of the flanges, possible resulting in less reduction in cross sectional area than would be expected in strong axis bending (Nielson and Schmeckpeper).

An analysis model created by the Massachusetts Highway Department (Crovo) showed that when the piles were oriented for strong axis bending, the pile moments and shears increased by a factor of two when the soil behind the abutment wall was changed from loose to dense. Other combinations of soil conditions did not result in a significant change from the weak axis bending condition.

Orienting the piles for weak axis bending also reduces the stresses in the abutment wall and superstructure. Research by Thippeswamy and GangaRao (1995) indicates that the orientation of the piles has a significant impact on the stresses in the superstructure. Weak axis bending of piles resulted in stresses at the superstructure and abutment joint that were three times lower than stresses developed when the piles were oriented for strong axis bending. Models developed by Wolde-Tinsae et al (1987) predicted a 20% increase in pile pre-load, a 10% increase in girder axial load, and a 20% increase in superstructure stresses when piles were oriented for strong axis bending instead of weak axis bending. However, a study conducted by the Massachusetts Highway Department (Crovo) indicated that the orientation of the HP piles did not affect the abutment wall deflections, wall pressures, or superstructure moments and forces.

### *7.2 Superstructure to Substructure Stiffness Ratio*

The ratio of the superstructure stiffness to the substructure stiffness dictates the distribution of forces and moments in the bridge system. The ratio is lower for bridges with longer spans and/or stiff abutments and piles. As the superstructure to abutment stiffness ratio decreases, the effects of continuous frame action will be more pronounced. For bridges with short spans and/or flexible substructures, the superstructure tends to behave more like a simply supported structure.

Flexible substructures will tend to relieve some of the tensile stress in the deck at the abutment, but will increase tensile stresses at mid-span. Thippeswamy and GangaRao (1995) conducted a numerical analysis of five steel girder integral bridges with varying skew and pile orientations. The effects of dead load, live load, creep, shrinkage, temperature gradient, settlement, and earth pressures were examined for spread footings and pile systems with varying stiffness. For more flexible pile supported abutments, tensile stresses in the bridge deck at the abutment caused by dead and live loads were reduced significantly. Overall, the stress from combined loads at the abutment joint was 2.5 times lower for piles than for stiff spread footing systems. The compressive stresses from combined loads at midspan were reduced, but the tensile stresses in the steel girders increased significantly. In fact, the bottom tensile stress in flexible systems exceeded the allowable stress in the steel. More flexible systems also had tensile stresses from dead loads nearly twice that for stiff systems in the superstructure at the intermediate piers. Creep, settlement, and temperature gradient stresses were also affected by the substructure stiffness. Creep reduced top tensile stresses at the abutment up to 40% for a very stiff spread footing and reduced stresses 10% for the systems on piles. Stresses in stiff footing systems at the superstructure and abutment joint due to temperature gradients were nearly double the stresses for flexible systems. Settlement stresses were also considerable for stiff footings and very small for flexible systems. At the foundation level, large stresses developed in the stiff footings. Total stresses from creep, temperature gradient, shrinkage, and settlement in the piles and more flexible spread footings were considered negligible.

Thippeswamy et al (1994) used a two-dimensional frame model to study the effects of varying superstructure to substructure stiffness, among other variables, on the moments within a structure. The superstructure to substructure stiffness ratio had a significant influence on the magnitude of the moments developed in the bridges. The moments at midspan of the superstructure due to dead and live load increased as the substructure became more flexible while the moment due to temperature gradients decreased. The moment at the footing and at midspan of the superstructure due to earth pressure became increasingly negative as the flexibility of the substructure increased. Finally, the magnitude of the moment in the footing due to a one-inch settlement decreased. Settlement moments were negligible at midspan.

### *7.3 Boundary Conditions*

Piles are usually considered fully fixed at the pile cap and fixed or free to rotate at the pile tip. Although piles have typically been constructed integrally with the abutment wall, researchers have experimented with pinned head connections. There may be some advantage to this because lower bending stresses will be developed. Numerical analysis of concrete piles conducted by Kamel et al (1995) confirmed that piles with fixed heads had significantly higher bending stresses than piles with hinged heads for a constant lateral deflection regardless of the soil density.

Although bending stresses may be lower in pinned head piles, the axial load capacity of the piles could decrease. Models developed by Greimann et al (1984) resulted in a 10% reduction in the vertical load capacity of steel H piles with a fixed head at a lateral displacement of 4 inches. The axial capacity was reduced 20% for a pinned head pile at a lateral displacement of 4 inches. Contrary to these results, Mourad and Tabsh (1998) found that the axial load in the piles was unaffected by changing the connection from fixed to pinned.

The fixity condition at the base of the pile does not appear to have a significant influence on the overall behavior of the pile because the piles usually have sufficient embedment depth to develop double curvature. The fixity of the base of the pile may be a concern when the pile is shallow or the soil is loose.

For abutment walls without piles, the fixity of the base of the abutment wall footing appears to have only a small influence on the stresses in the superstructure. A frame model studied by (Thippeswamy et al, 1994) showed that fixing the base of the abutment caused a slight reduction in the superstructure moments at midspan and at the supports due to primary loads. The moment due to earth pressure at the support switched from negative to positive. Frame models by Dahger et al (1991) also showed that the degree of fixity did not affect the moments and shears in the superstructure.

## 8.0 ALTERNATIVE SUPPORT SYSTEMS

Steel H or HP piles are most frequently used to support the abutment, but cast-in-place, prestressed, pipe, and concrete-filled steel pipe piles have also been used by state agencies. There does not appear to be a published comparison of the performance of these different pile systems, however, several researchers have examined the use of alternative piles or support systems. Full scale testing indicates that concrete piles generally do not perform as well as steel H piles. Concrete filled steel pipes have been considered, but little research has been conducted to determine the feasibility of using them. Research indicates that use of spread-footing abutments causes the development of high stresses in the superstructure and should be limited to bridges with small movements. Spread footing abutments supported, but not keyed into, rock have been successfully utilized for bridges with up to ¼ inch of total movement. Finally, tall abutment walls that are hinged at their base have been used for movements up to 2 inches (Wasserman and Walker, 1996).

Kamel et al (1995) studied the application of prestressed concrete piles in integral abutment bridges through numerical analysis. Steel piles with a hinged connection at the top of the pile showed slightly more capacity to accommodate large lateral deflection than concrete piles with a hinged connection. A lateral force of 7.8 kips was required to deflect the concrete pile 0.34 inches whereas only 5.1 kips was required to deflect the steel H piles 0.40 inches. Loading was stopped when the compressive stresses reached their allowable values.

Cyclic tests conducted by Oesterle et al (1998) on steel H piles and prestressed concrete piles showed that both were able to sustain the applied vertical load throughout the test. However, the concrete piles sustained damage that was considered unacceptable. Pipe piles filled with concrete are more ductile than prestressed concrete piles and have greater resistance to local buckling than steel H-piles. However, additional research is needed to determine if pipe piles are an economical and durable solution.

Numerical analysis conducted by Thippeswamy and GangaRao (1995) indicates that significant stresses may develop at the superstructure/abutment joint and in the substructure when spread-footing abutments are used in lieu of pile supported abutments. However, the results of this analysis are questionable because a fixed translation condition was used for the base of the footings and measurements by the Massachusetts Highway Department (MHD) indicate that spread footings are subject to translational movement (Crovo). The MHD reported that because the base of the spread footing is less restrained, lower stresses are developed as a result. Despite this, the analysis results from Thippeswamy and GangaRao (1995) demonstrate that the fixity of the spread footing plays a critical role in the performance of an integral bridge. Furthermore, the results give valuable insight into the effect of varying substructure stiffness on the stresses in the superstructure as discussed in the previous section.

A critical area in the performance of integral bridges is the bridge reaction to settlements. Spread footings appear to be more susceptible to large settlement stresses than piles (Ng et al, 1998 and Thippeswamy and GangaRao, 1995). This is discussed in greater detail in the section entitled "Settlement Stress".

## 9.0 SPECIAL CONSIDERATIONS FOR CYCLIC LOADING

The cyclic movement of the substructure as a result of thermal movement of the superstructure presents a unique challenge for integral bridges. First, cyclic loading may compact the soil behind the abutment wall and piles. When the bridge contracts, a soil wedge could fill the gap between the abutment wall left as a result of soil compaction, which impacts the behavior of the bridge during the next cycle of expansion. Secondly, the compaction of the soil will cause settlements behind the abutment wall or under spread footings. Secondly, the cyclic loading may cause fatigue-related distress of the piles.

### *9.1 Soil Stiffening and Settlement*

During the annual winter contraction, a wedge-shaped portion of soil behind the abutment wall could move towards the abutment and fill the void left as a result of the abutment movement. When the bridge expands towards the summer maximum, the wedge will not return to its original position due to the inherent non-linear nature of the soil. Soil that moved towards the abutment during the period of contraction will be densified during the next cycle of expansion and the soil pressures on the abutment will be higher as a result. Thus, earth pressures build up with each annual cycle. The build up of stress will eventually reach a plateau because there is a limit to how much soil is available for wedging, and as the soil is densified, it will be less likely to fill the void during successive periods of contraction. This phenomenon of soil wedging and buildup of lateral earth pressures is often referred to as *ratcheting* or *soil stiffening*. Ratcheting may not always take place. Soil may not fill the gap and the bridge will expand more freely during the next cycle of expansion with corresponding lower stresses in the structure as a result. The use of granular, non compactible materials in predrilled oversized pile holes near the top of the pile serve to limit soil stiffening under cyclic loading.

Results from centrifuge tests of spread-footing abutments indicate that soil stiffening does take place behind the abutment wall. Ng et al (1998) observed soil settlement behind the abutment wall as a result of the reduction in volume from repeated cycles of soil densification and gap filling. The settlement profile from cyclic loading is dependent on the soil characteristics. The centrifuge tests conducted by Ng et. al. (1998) resulted in a maximum settlement of 0.7 meters adjacent to the abutment wall in loose sand under 100 cycles at ±60mm. The settlement profile was linear from the wall to a point 9 meters away from the wall, where no settlement occurred. For dense soils, the maximum settlement of 0.66 meters occurred about 0.9 meters away from the wall and the zone of settlement extended 5.4 meters away from the wall. The length of the soil settlement region behind the abutment was expected to be about 60% of the abutment wall height (Springman et al 1996). Springman et al (1996) recommended placing medium dense or dense backfill to solve the settlement problem.

Despite the results from model tests, observations of in-service bridges indicate that permanent soil deformations take place as a result of soil stiffening (Crovo). This has the effect of lowering the initial soil pressures on the substructure. Ng et. al. (1998) also noted that gap-filling behavior was not observed in field tests of spread abutments and stated that "clearly, more long-term reliable field monitoring data are needed to assist us in understanding of the behavior of integral bridge abutments". Other researchers have noted (Sanford and Elgaaly, 1993) that permanent soil deformation of the backfill appeared to be occurring.

The study by Ng et al (1998) also indicated that spread-footing abutments are more likely than piles to develop vertical settlements because as the abutment wall rotates, the footing densifies the soil below the footing. Ng found that significant settlements occurred during the first 20-50 cycles and the rate of settlement gradually decreased until a stable trend developed after about 200 cycles.

### *9.2 Fatigue Cracking of Piles*

Steel piles in integral bridges could be subjected to fatigue related distress due to the cyclical nature of the environmental loads. The following rationalization was given by Loveall (1985), who reported that fatigue research conducted under stress ranges well below yield stress indicates that fatigue cracking should occur at a low number of stress cycles. For piles in weak axis bending, fatigue cracking will commence at flange tips and propagate toward the web when the flexural component of stress sufficiently exceeds the axial component along the flange edge in tension. Because the flexural component is proportional to the effective width of the flange, cracking of the flange will reduce the effective width and the flexural component, thereby producing a hinge-like condition where the axial compression protects the piles from further crack growth. Loveall (1985) recommended additional research to describe the behavior of piles and to determine if there are sufficient stress cycles to result in the fatigue crack growth just discussed. Observations of fatigue cracking of piles have not been reported in the literature.

## 10.0 APPROACHES

It has become common practice to build an approach slab integrally with the abutment to span the backfill behind the abutment wall. Approach slabs constructed with a joint at the bridge abutment have tended to shift towards the flexible approach pavement because of the continual cyclic movement of the bridge and debris infiltration (*Steel Bridges*, 1993 and Sanford and Elgaaly, 1993). Moving the joint to the approach pavement eliminates shifting of the approach slab off of the slab seat and reduces water infiltration of the backfill, thus preventing erosion of the abutment backfill or freeze/thaw damage resulting from saturated backfill. Designing the approach slab to span the backfill also prevents traffic consolidation of the backfill soils, which helps avoid differential settlement of the approach and bridge, and reduces lateral surcharge loads on the piles or abutment wall. Spanning the backfill has the added benefit of reducing the stresses developed in the approach slab as a result of the expansion and contraction of the bridge and the resistance to movement from friction with the soil. Properly constructed approach slabs also eliminate pressure on the abutment wall from residual expansion of approach pavement, which has been known to cause severe abutment and pier damage (Arsoy et al, 1999; Xanthakos, 1995; Burke, 1987). Finally, an integral support slab can also serve as a “bridge” over soil settlements produced from the ratcheting action previously discussed.

There are few guidelines for selecting the proper length of the approach slab. It is often argued that the length of the zone of settlement extends from the abutment a distance equal to twice the height of the abutment; thus approach slabs lengths should be made two to three times the height of the abutment. This argument is based on the fact that displacing an abutment causes movement of a wedge of the backfill with a height equal to the height of the abutment and a length equal to tan $(45+\phi/2)$ times the abutment height. A finite element analysis conducted by Arsoy et al (1999) indicated that the length of the settlement zone extended three to four times the height of the abutment. Centrifuge tests indicated that the length of the soil settlement region behind the abutment was expected to be about 60% of the abutment wall height (Springman et al, 1996). This is contrary to the proposed two to three times abutment wall height guideline stated above. Additional research is needed to better quantify the extent of the settlement zone.

## 11.0 WINGWALLS

Wingwalls confine the backfill behind the abutment and can, therefore, influence the soil pressures on the abutment walls. The orientation of the wingwalls can have a significant influence, because as the wingwall orientation changes from parallel to the abutment wall to perpendicular to the abutment, the soil becomes more confined. The influence of wing walls and wing wall orientation is further complicated for skewed bridges, as was previously discussed. Research has confirmed that wingwalls oriented at 90 degrees produce the greatest confining effect, which increases the earth pressure on the abutments by as much as a factor of 2 (Crovo). Despite the increase in earth pressure, turnback (90 degree or U) wingwalls have the benefit of reducing approach fill settlements (Arsoy et al, 1999).

Wingwalls will also affect the distribution of loads in the structure. Mourad and Tabsh (1998) studied the effect of adding a 90° wingwall and changing the length of the wingwall. Only live loads were considered in the analysis. For bridges without wingwalls, the analysis showed that the applied live load was distributed more uniformly among the piles under the abutment walls. Pile axial stresses were larger when a wingwall was in place, but decreased as the wingwall length increased because of the larger moment arm length under a constant applied moment. The pile under the end of the wingwall farthest from the abutment was always in tension. The abutment wall-wingwall system did not behave as a rigid block under the applied truck loads, so the piles at the corners of the substructure were not always the most heavily loaded.

Different behavior is observed when bridge movement is considered. The wingwalls will provide additional resistance to bridge expansion and will also resist the tendency of the abutment to rotate as the bridge expands. Compressive axial forces are developed in the wingwall piles that resist the vertical abutment rotation and may result in axial strains that are much larger than the predicted axial strains in the abutment piles (Sayers, 2000).

## 12.0 PRACTICAL CONSIDERATIONS

The evolution of integral bridge design and construction has largely been through incremental changes based on observations of bridges in service. Some of the current practices regarding bridge length, construction sequence, and selection of steel or concrete girders will be discussed here briefly. Sources of additional information on design details and construction methods developed to overcome some of the unique challenges posed by integral construction are listed below.

A number of authors have reviewed construction methods, construction details, and bridge distress. Among the more informative sources are “Integral Construction” (1989), Burke (1990), Soltani and Kukreti (1992), Flores (1994), Wasserman and Walker (1996), Alampalli and Yannotti (1998), Franco (1999), Kunan and Alampalli (2000), and Wasserman (2001). Application of compressible material behind the abutment wall to reduce soil pressure and provide adequate drainage has been studied by the North Dakota Department of Transportation (Jorgenson, 1983) and Horvath (2000).

Martin P. Burke Jr. has written numerous articles on integral bridge construction from a practitioner’s perspective. In “Integral Bridges: Attributes and Limitations” (1993), Burke provides a good overview of the advantages and disadvantages of integral construction and provides some general design guidelines. “Bridge Approach Pavements, Integral Bridges, and Cycle-Control Joints” (1987) has a good discussion of approach pavements and several approach slab design details. He gives his recommendations for a general design approach in “The Design of Integral Concrete Bridges”(1993). Finally, Burke describes typical distress observed on integral bridges in Ohio in "Cracking of Concrete Decks and Other Problems with Integral-Type Bridges" (1999).

### 12.1 Length and Movement Limitations

Research results on the acceptable length of integral bridges varies, but there seems to be a consensus among state agencies that the practical movement limitation is near 2 inches. Studies examined in this report indicate that deflections of between ½ inch and 4 inches can be accommodated. Wasserman and Walker (1996) reported that properly designed integral abutments can accommodate up to 2 inches of total movement and more based on Tennessee's experience with over 1,000 integral bridges. A survey conducted by Franco (1999) gave similar results. For example, Virginia restricts movement to 1½ inches and Maine restricts movement to 2½ inches.

A survey sent to the highway departments of all 50 states conducted by Soltani and Kukreti (1992) reported that the acceptable range of length limitations is 200 to 300 feet for steel, 300 to 400 feet for concrete, and 300 to 450 feet for prestressed concrete. A few states use longer limitations for each structure type. Missouri has built a 500 foot steel bridge and a 600 foot concrete bridge. Tennessee has built a 927 foot prestressed bridge and a 416 foot steel bridge. These states have been building integral bridges longer than most states and thus have amassed a longer history of integral bridge performance knowledge from which they have refined design and construction practices.

### 12.2 Construction Sequence

Shrinkage cracking can pose a more serious problem for integral bridge decks than jointed bridge decks. Early age deck slab cracking can be the result of lapses or omissions in the application of one or more of the precautionary measures usually used to minimize deck slab cracking. Diagonal deck slab cracks located at acute corners of integral bridges, uniformly spaced straight cracks located over and perpendicular to previously placed concrete end diaphragms, and cracks perpendicular to longitudinal supporting steel stringers are occasionally reported (Burke, 1999). Proper sequencing can help to control flexural stressing of fresh composite sections during concrete placement and finishing.

Special care must be taken to avoid thermal movements on fresh concrete because the abutments and superstructures of an integral bridge are joined by cast-in-place continuity connections and differential movement of the separate elements can damage fresh concrete. This is of particular concern for hybrid integral bridges when steel girders are placed in a bridge with a concrete abutment. Several concrete placement procedures have been developed to address this problem and are presented by Burke (1993 and 1994). Wasserman and Walker (1996) also made recommendations for reducing the effects of thermal movements on fresh concrete.

Burke (1993) noted that earthwork should be placed and compacted prior to driving piles or pre-drilling holes to control lateral earth movements of soils below and within embankments. This limitation is also important for piers, which subsurface movement can adversely affect if they are placed before embankment construction has been completed. If they are placed before embankment construction, they should be designed to resist lateral earth pressure without depending on a connection to the superstructure.

### 12.3 Steel vs Concrete Girders

There does not seem to be a preference for either steel or concrete girders from a design performance perspective. A survey by Kunan and Alampalli (2000) indicated that thirteen state agencies found no difference in the performance of steel and concrete girder bridges. Colorado reported that steel bridges experience more rotation at bearings, causing some cracking and spalling in bearing areas of the abutment diaphragm, but New York has observed less deck cracking on steel girder bridges (Alampalli and Yannotti, 1998). Kansas noted that steel girders move more and that concrete girders shrink. Oregon stated that bridges with prestressed or post-tensioned concrete will experience creep shortening after construction.

## 13.0 DISCUSSION OF LITERATURE SYNTHESIS

A synthesis of technical information for integral bridges was carried out under Phase I, of a larger three-phased project for the Vermont Agency of Transportation (VTrans) entitled, "Performance Monitoring of Jointless Bridges." This report is the result of the Phase I work. Prior to initiating the synthesis, VTrans developed a list of fundamental and advanced questions regarding the performance of integral abutment bridges. A summary of these questions is provided in Appendix A.

A comprehensive literature search was then carried out to identify the body of knowledge available for integral bridges. Sources searched included the in-house WJE library, which contains more than 1,400 volumes with additional subscriptions to 75 professional periodicals. Interlibrary searches using NERAC, OCLC, university library systems, professional society libraries, government agency libraries such as Transportation Research Board and Federal Highway Administration were conducted. Internet searches utilizing sites such as northernlight.com and others were also carried out.

From these searches, a collection of over 90 references was collected. A comprehensive bibliography was prepared and is included as Appendix B. The references were organized into divided sections within five, three-ring binders. These binders have been submitted separately to VTrans. The references were then subjected to a cursory review and categorized with regard to their relevancy to the subject study. References deemed to be most advantageous were studied in depth, and formed the basis for the work here. These references are cited throughout the Phase I report and are identified at the conclusion of the report as "Cited References." At the end of each cited reference there is a notation referencing the three-ring binder where the article or its reference can be found. An example notation of (2,4) refers to binder volume 2 and the article under tab number 4. The comprehensive bibliography presented in Appendix B reflects the organization of references within the five-binder set.

The literature searches focused on integral bridge performance. It was not the intent of the search to compile a list of design guides, state departments of transportation bridge design manuals, construction memorandum, specifications, etc. While some of this material was obviously generated under the literature search effort and may have been included in the reference list, the majority of these types of references were ignored and not included.

The objective of the Phase I study was to answer as many of the questions listed in Appendix A as possible, while 1) providing appropriate documentation of the results of integral bridge performance monitoring to date, 2) outlining current performance issues, and 3) providing recommendations for future performance monitoring programs. Based on the findings of the Phase I study and upon authorization from VTrans, additional work would include the development of an instrumentation and monitoring plan for a newly constructed integral bridge (Phase II) and the execution of these plans and subsequent collection and evaluation of field data over a three year period (Phase III).

In order to maximize the potential results of the Phase II and Phase III study, it is necessary to study the findings of the literature synthesis to define those issues that require further evaluation and research. The most important four or five issues need to be identified. Based on the synthesis, the following table lists the performance variables that are considered most important to the behavior of integral bridges, regardless of bridge type and construction material. When considering the intricacies of various bridge configurations, site-specific conditions, materials, span lengths, etc, the permutations and relative significance of the issues listed in the table below can vary widely.

These issues would become the focus of subsequent instrumentation and monitoring plans to be developed under Phase II and III. VTrans had requested that these issues be restated as a research hypothesis, and that conceptual instrumentation plans be developed.

### *13.1 Phase II Study*

In order to develop an understanding of integral abutment bridge behavior under conditions specific to the state of Vermont, in-service integral bridges need to be studied from initial construction through service. During the initial life of the bridge, affects of creep and shrinkage can have a profound effect on superstructure and substructure stresses, which will influence behavior later in the bridge's life. The influences of creep and shrinkage on bridge performance lessen with time, but are replaced by more significant factors, principally soil-structure interaction. The flexibility of the substructure systems, either piles, footings or abutment walls, and their interaction with the surrounding soil have the single largest influence on integral bridge behavior. Factors that affect soil-structure interaction include superstructure and substructure response to thermal, live and dead loads; soil stiffness, including any changes associated with cyclic expansion and contraction of the bridge; local geology; structural configuration and construction procedures. The relative influence of these factors will be intrinsic to the bridge under investigation, and will vary from bridge to bridge. For example, steel and concrete superstructure, pile orientation in weak or strong axis bending, flare angle of wingwalls, use of predrilled and oversized pile holes, approach slab attachment, skew, length, etc. are just a few of the intrinsic bridge characteristics that can effect behavior.

The permutations of bridge configuration, environmental and situational factors are so numerous that detailed instrumentation plans for the monitoring of integral bridges can not be developed until the test bridge is identified. However, it is possible to identify the most significant variables to integral bridge behavior and identify generalized hypothesis' for their study in in-service integral bridges. The following hypothesis' are presented, and for each a conceptual instrumentation plan is presented. In order to better clarify the conceptual instrumentation plans, an instrumentation scheme for an idealized bridge is presented in Figure 9.

**Hypothesis No. 1 - Temperature**

*Background:* Temperature induced movements can create "ratcheting" effects in integral bridges, which over time can mobilize soil pressures approaching the theoretical passive state. This phenomenon is related to seasonal contractions (winter) and expansions (summer) of the structure and their influence on soil-structure interaction. Under contraction, the abutment is pulled toward the center of the bridge. The soil wedge behind the abutment also moves with the abutment toward the center of the bridge. Under expansion, the abutment is pushed back toward its original position. However, the nonlinear behavior of the soil wedge prevents the abutment from returning to its pre-contracted position. The resistance provided by the soil wedge results in a lateral earth pressure on the back of the abutment wall, which over time increases with each subsequent seasonal cycle, hence the term "ratcheting." This phenomenon is mentioned with frequency throughout the literature reviewed herein. Additional factors that affect thermal movements include the design

**TABLE 2 – Key Performance Parameters For Integral Bridges**

| Performance Variable | Impact to Phase II and III of Study |
|---|---|
| Coefficient of Thermal Expansion | Determination of global coefficient of thermal expansion is required for accurate determination of bridge behavior |
| Temperature Range and Temperature Gradients | Knowledge of experienced temperature input and temperature distribution throughout structure is required to determine distribution of forces to substructure |
| Creep and Shrinkage | Generally not significant factors over time. However for testing during early years of bridge service, it may be necessary to determine the stress accumulation from these sources. |
| Settlement | Generally not a significant factor for superstructure performance; however, settlement of integral approach slabs needs to be considered. |
| Skew and Curvature | Skew and/or curvature under certain substructure configurations can yield unique behaviors of the substructure and stress concentrations at the substructure to structure junction that will need to be considered. Analytical analysis of specific bridge configurations will be required in order to develop effective instrumentation strategies. |
| Abutment Wall and Soil Interaction (Abutment Wingwalls) | Determination of soil pressure distribution under cyclic environmental loading is required. Variations in the design and configurations of these elements can enhance or lessen the importance of this variable on bridge behavior. |
| Pile and Soil Interaction | Variations in the design of these elements can enhance or lessen the importance of this variable on bridge behavior. The quantification of the pile stresses, especially near the pile cap embedment is critical to understanding the pile-soil structure interaction. |
| Flexibility | The superstructure to substructure stiffness is a function of the bridge boundary conditions. Instrumentation of the structures will be highly dependent upon the relative stiffness' of the various elements. |
| Approach Slab | The participation of the approach slab is generally of lessor importance than other factors; however, having an integral approach slab eliminates adverse influences on the soil structure below the slab which are unrelated to the overall bridge behavior. |
| Construction Material and Length | The bridge configuration and relative stiffness of the constituent materials will need to be considered in developing an instrumentation plan. |

temperature range, temperature gradients within the structure, and restraint and flexibility of the substructure. The influence of these factors is discussed in Section 2.0.

*Hypothesis:* Thermal movements and the corresponding response of the soil-structure interaction are not well defined by current design specifications.

*Conceptual Instrumentation Plan:* The instrumentation plan should include provisions for the measurement of the bridge-specific coefficient of thermal expansion and ambient air temperature with time. The thermal response of the structure over time should also be monitored. Thermal couples should be placed strategically to measure thermal gradients in the principle orthogonal directions. For example, thermocouples should be placed laterally across the bridge at midspan, abutments, and the pier supports, if any, in order to measure the thermal gradient across the bridge width. This arrangement provides for a distribution of thermocouples in the longitudinal direction as well, and hence a thermal gradient along the bridge length can also be obtained. Finally, it is recommended that vertical temperature gradients be obtained at several slab sections and at a typical interior and typical exterior girder by placing thermocouples at the top and bottom of the slab/girder. The vertical temperature gradients should be obtained for typical interior and exterior girders at midspan, abutments, and piers, if any.

Temperature gradients develop within the structure due to naturally occurring directional heating and cooling and variations in thermal conductivity of the different bridge materials. The imposed temperature gradients produce internal stresses that are proportional to the differential movements of the structure. Boundary conditions, which restrict structural movement due to these thermal gradients, also induce stresses within the structure. Stresses induced by the boundary conditions are proportional to both the temperature gradients and boundary condition stiffnesses. The principal boundary conditions for integral bridges are substructure flexibility and soil-structure interaction. If the measured thermal gradients are uniformly proportional to the measured stresses, then boundary condition flexibility is uniform. If however, the thermal gradients are not uniformly proportional to the measured stresses, then variations exist in substructure flexibility and soil-structure interaction across the substructure width and/or depth. The careful placement of thermocouple sensors will allow for the monitoring of thermal gradients through the structure's width, depth and length. A similar placement of strain monitors will allow for the measurement of the structural response (output) to the thermal gradients (input). Finally, comparison of the structural response and thermal gradients allows for an assessment of the global influences of substructure flexibility and soil-structure interaction. The difficult nature of measuring soil-structure interaction directly at the soil-structure interface makes the above methodology even more advantageous to the determination of integral bridge performance.

**Hypothesis No. 2 - Creep and Shrinkage**

*Background:* Because creep and shrinkage effects are assumed to be opposite in nature and hence tend to cancel each other out, they are generally ignored in bridge design. For integral bridges, especially those constructed entirely of concrete, shrinkage results in a permanent shortening of the bridge, which will be principally resisted by the bending stiffness of the substructure. Creep, especially for bridges constructed entirely of concrete, will result in a gradual elongation of the bridge, which will be principally resisted by the stiffness of the soil and structure acting together. For integral bridges with steel stringers and concrete decks, research shows that creep and shrinkage may be additive [Thippeswamy and GangaRao (1995)]. Nonetheless, the influence of these secondary stresses has been generally ignored in integral bridge design. Furthermore, very little research is available to quantify the effects of creep and shrinkage in integral bridges. A more extensive discussion of the effects of creep and shrinkage on integral bridge performance has been presented in Section 3.0.

*Hypothesis:* Creep and shrinkage, in effect, constitute a load cycle of expansion and contraction, respectively, and hence produce stresses and strains that will influence bridge behavior.

*Conceptual Instrumentation Plan:* In general, the instrumentation plan established for Hypothesis No. 1 will also serve to quantify the strains from creep and shrinkage. Additional modifications to the instrumentation plan should include the construction of on-site creep and shrinkage specimen, which will be instrumented and monitored by the on-site data acquisition equipment. The use of instrumented ring specimen and modulus of rupture specimen are recommended. NCHRP research on transverse cracking in bridge decks showed that these specimens could accurately predict when shrinkage stresses exceed the tensile capacity of the concrete. Following cracking, the specimen can be monitored for creep. The data collected from these specimens will be beneficial in future modeling, should it be undertaken.

**Hypothesis No. 3 - Soil-Structure Interaction**

*Background:* While variations in the design and configuration of the abutment can enhance or lessen the effects of this variable, overall soil-structure interaction will have a significant impact on integral bridge behavior. Unfortunately, it is also the most difficult to accurately predict and measure because of the non-linear relationship between soil pressures and the magnitude and shape of the displaced abutment. The principal driving input behind soil-structure interaction is thermal movement of the superstructure and thermal induced volumetric changes in the substructure. The influence of temperature and thermal gradients on the structure is discussed under Hypothesis No. 1 above. The principal result of the thermal movements is a change in soil stiffness. Ultimately, the variations in soil-structure interaction over time can lead to cracking in concrete bridge abutments and yielding of piles. Understanding the degradation of structural performance precipitated by changes in soil-structure interaction is essential in developing economical integral bridge designs.

*Hypothesis:* Changes in soil-structure interaction under cyclic loading produces corresponding changes in bridge behavior, the result of which may result in cracking or yielding of substructure elements.

*Conceptual Instrumentation Plan:* The incorporation of thermocouples and strain sensors as described under Hypothesis No. 1 will contribute significantly to the understanding of structural response from soil-structure interaction. Additional sensors will be required on the substructure elements, and will include, but not limited to, strain sensors on piles, tilt meters on abutment walls, soil pressure sensors on the abutment backwall, and displacement sensors on the abutment. In order to capture the variations expected both laterally and vertically across the abutment wall, pile, footings, etc., it is recommended that substructure sensors be installed vertically on typical substructure elements and laterally on substructure elements positioned at the bridge edges, quarter points, and bridge centerline. Depending on the substructure system employed, sensors would be installed vertically near the surface and at several intervals vertically, say every 2 to 4 ft. Pressure sensors, while beneficial to the subject study, are susceptible to localized variations in soil properties. Therefore, it is recommended that abutment movements be monitored using linear potentiometers affixed to a permanent reference point. Such sensors would be positioned as described above.

**Hypothesis No. 4 - Settlement**

*Background:* The lateral and downward movement of the soil wedge behind the abutment wall that is associated with the "ratcheting" effect described above can produce settlement behind the abutment. The consequences of this settlement are dependent upon the approach slab construction and its connectivity to the structure. Under certain scenarios, a differential condition between the structure and approach slab can develop, which will result in increased impact stresses from traffic. Differential settlement of the structure will create varied structural responses dependent on the foundation system (spread footing or pile), span length, and number of spans. Affects from settlement typically worsen with time, and may not be significant during the time period over which the monitoring is planned.

*Hypothesis:* For certain integral bridge configurations and abutment types, settlement can create significant stresses within the structure.

*Conceptual Instrumentation Plan:* To assure that settlement does not occur and adversely effect the test results during the monitoring program, fixed reference points should be installed at the four bridge corners and at each end of the interior piers. Fixed reference points should also be installed in the approach slab immediately behind the abutment and at some distance away from the abutment. The number of fixed reference points may need to be increased for certain integral bridge configurations and abutment types. A permanent reference point away from the bridge should be established. Optical surveys of the fixed reference points to the permanent reference point should be conducted at periodic intervals during the monitoring program, preferably once each quarter.

## 14.0 CONCLUSION

The basis of integral bridge design guidelines has been largely empirical. Most states do not have specific design procedures, but accommodate secondary stresses through careful detailing. In the last decade, interest has grown enormously in the application of integral bridges in order to take advantage of the many economies associated with these structures. Research to predict the behavior of integral bridges based on theory, rather than empirical evidence, is ongoing. Specifically, researchers at the Tennessee Department of Transportation, Penn State University, and Purdue University are preparing to conduct multi-year monitoring programs of integral bridges. This work will begin over the next two years and should involve approximately a dozen structures.

Research is also underway at Iowa State University under the direction of R. Abendroth and L.F. Greimann and funded by the Iowa Department of Transportation to further evaluate the design procedures developed at Iowa State in 1987 through 1989. A major part of the work involves a two-year field monitoring program of two prestressed concrete girder integral abutment bridges. Several questions will be addressed regarding the magnitude of the thermal movement, the yearly temperature range of major bridge components, the rotational restraint provided by the pile cap, and the overall behavior of the abutment assembly. Work started in 1997 and is expected to be complete in April, 2002.

Clearly, additional research is required to fully understand how an integral bridge reacts with its environment. The behavior of integral bridges is defined by the complex interaction of a large number of variables. The secondary loads introduce stresses into the structure that are generally well understood but not easily quantified. The true fixity of continuity connections, soil-structure interaction, soil conditions, bridge length and geometry, climate, abutment and pile design, and design of features such as approach slabs and wingwalls are among the variables that play an important role in the overall behavior of the bridge.

Upon identification of a suitable test bridge, the authors will develop a bridge-specific instrumentation plan. The instrumentation of the test bridge will include placement of instrumentation at key bridge sections, so as to capture the behavior variations that exist through and along the principal or orthogonal directions. The instrumentation will generally be placed longitudinally at the abutments, piers and maximum positive moment locations, and transversely at the bridge edges, quarter points and the bridge centerline. Vertically, instrumentation will be placed at the top and bottom of the superstructure and points between. For the substructure, instrumentation will be placed vertically near the top of the abutment and at intervals below up to the point of assumed counterflexure texture. Bridge symmetry and budget will be considered when developing the instrumentation and monitoring plans. The instrumentation will be monitored at hourly intervals for a period to be defined by the Owner.

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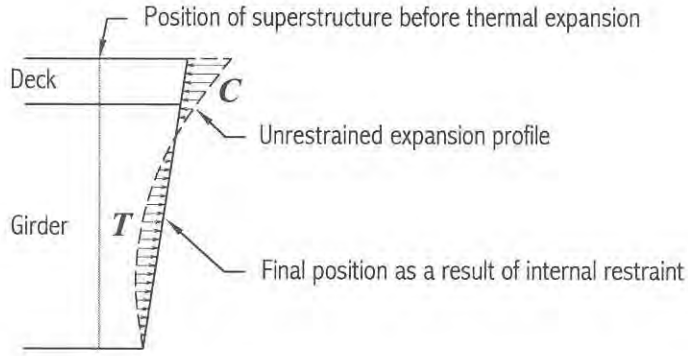
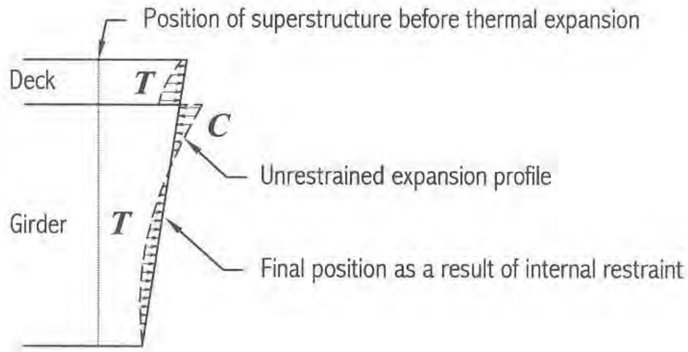
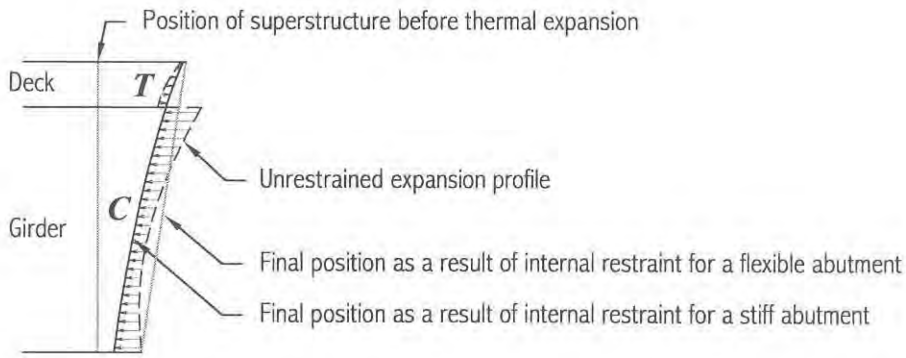
# FIGURES




*(a) Stress in a homogeneous deck and girder for a typical temperature gradient*

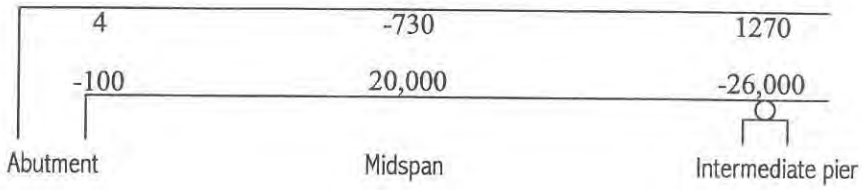



*(b) Stress in a concrete deck and steel girder when the free expansion at the top of the girder is greater than the free expansion of the deck*

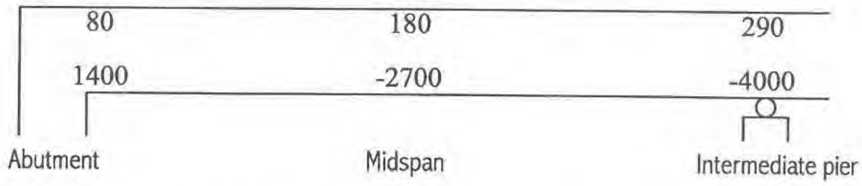



*(c) Stress in a concrete deck and steel girder in a bridge with a stiff abutment*

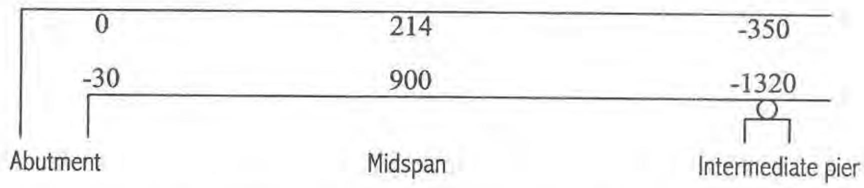
*Figure 1 - Thermal gradient stress scenarios in an integral bridge superstructure subject to internal restraint, T = tension and C = compression*



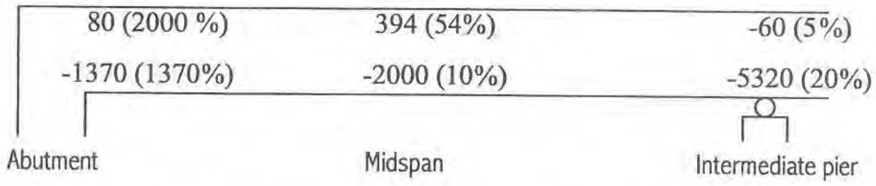

*(a) Dead load + live load stresses, in psi with tensile stress shown positive*




*(b) Shrinkage stresses, in psi with tensile stress shown positive*




*(c) Effective creep stresses, in psi with tensile stress shown positive*




*(d) Superimposed creep and shrinkage stresses also shown as a percentage of the magnitude of the dead and live load stresses*

*Figure 2 - Effects of primary loads, shrinkage, and creep on the superstructure of a hybrid integral bridge reported by Thippeswamy and GangaRao (1995)*

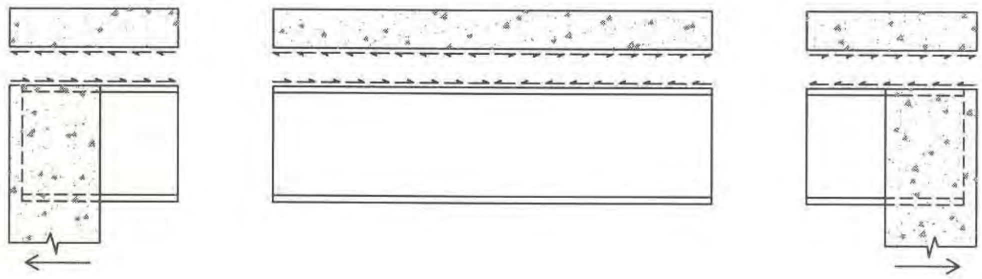


*Figure 3 - Shrinkage-induced stresses in a steel girder built integrally with the concrete deck and abutments*

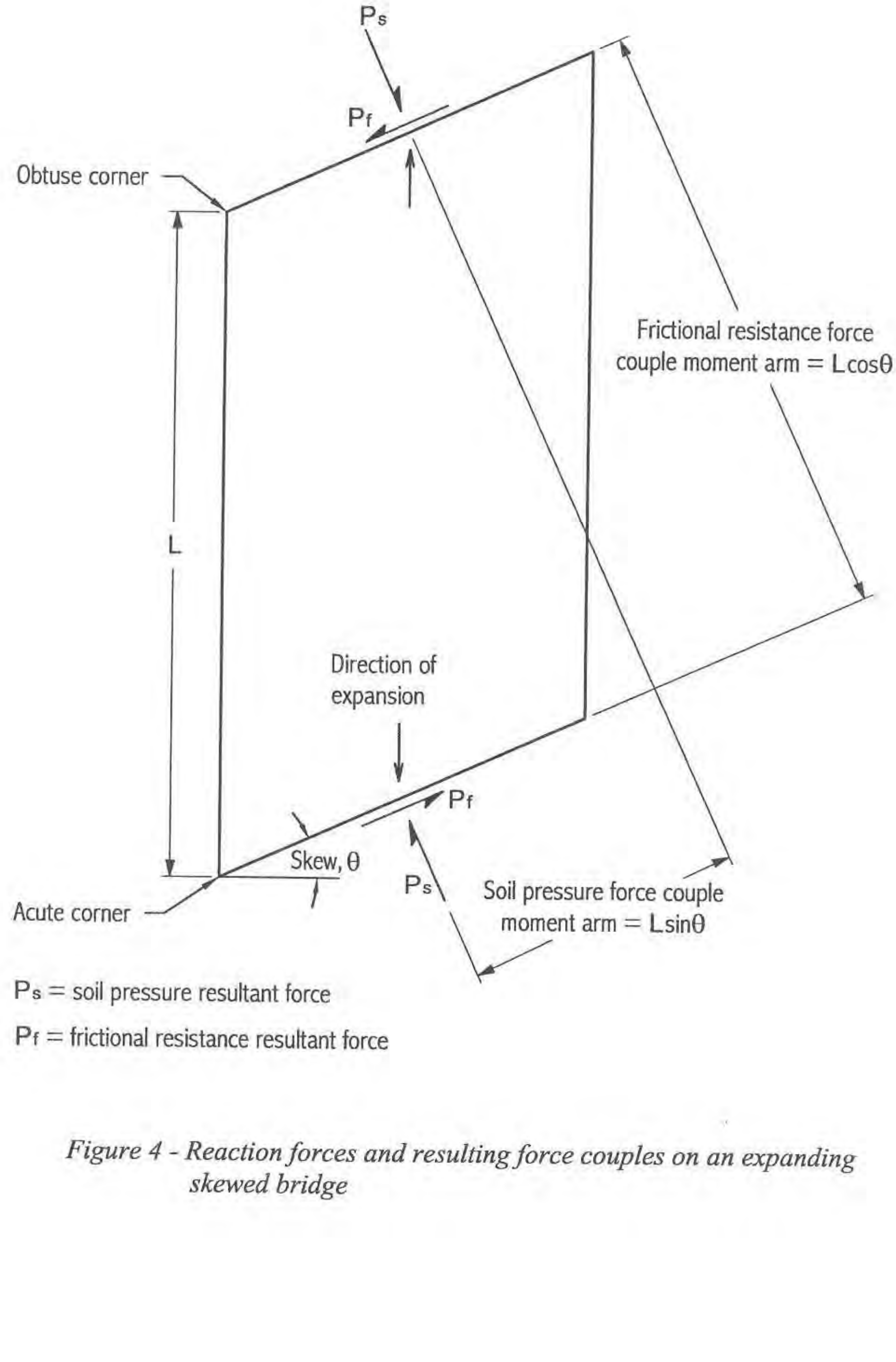



*Figure 4 - Reaction forces and resulting force couples on an expanding skewed bridge*

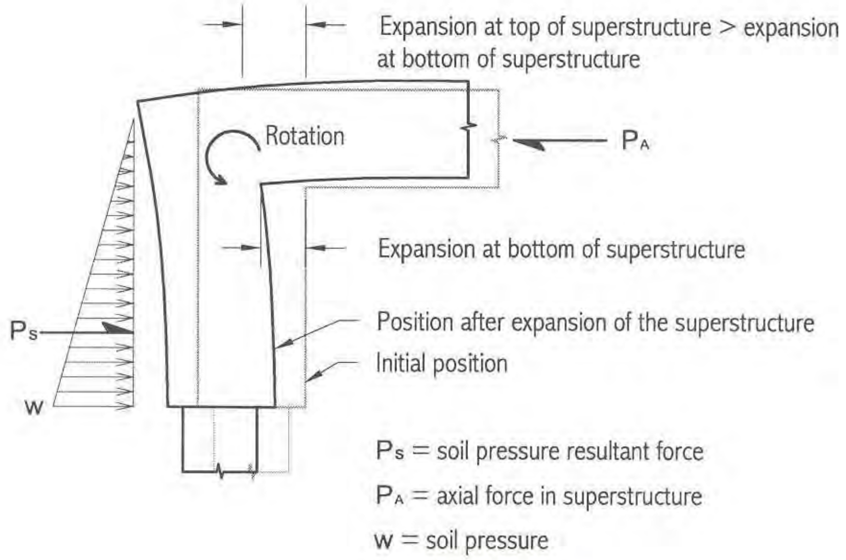



*Figure 5 - Abutment rotation as a result of thermal gradient and eccentricity between soil pressure and axial force in the superstructure*

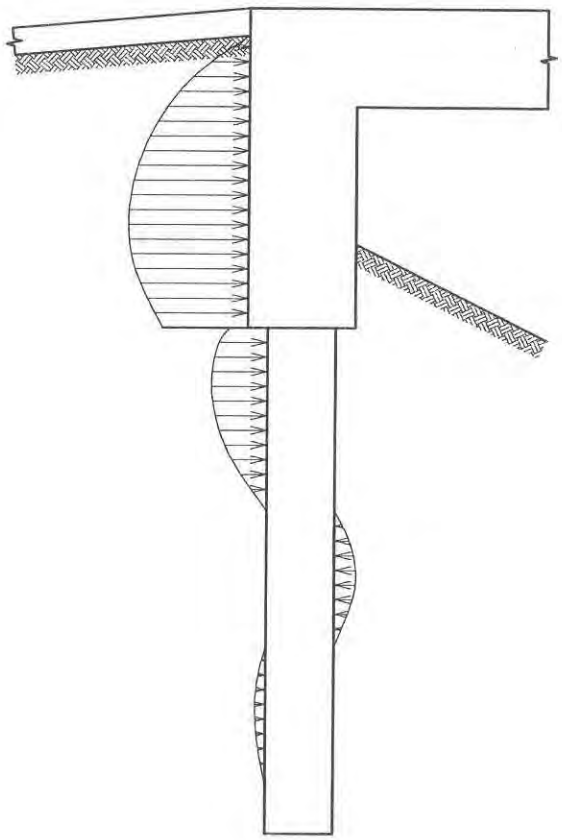
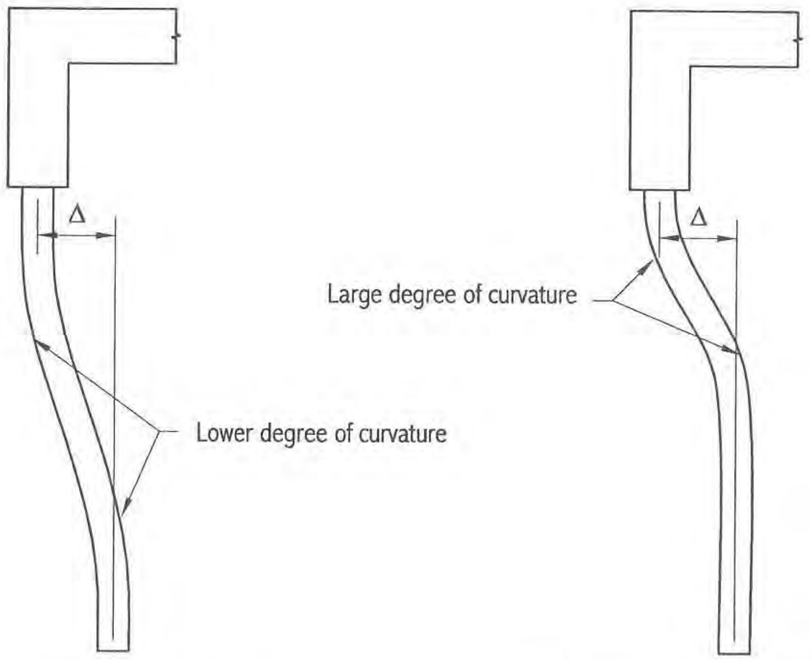


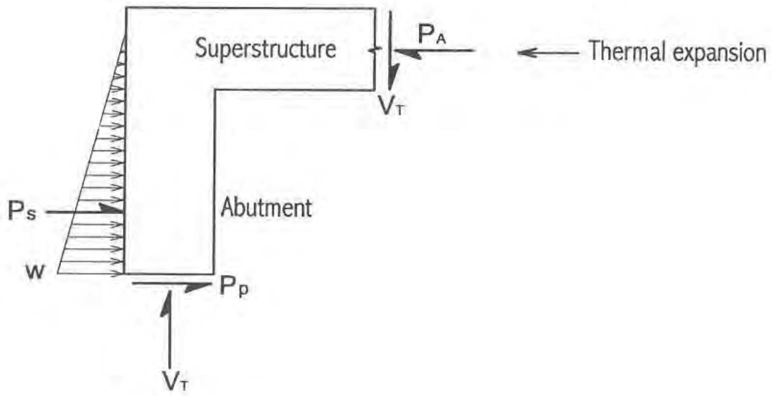
*Figure 6 - Non-linear soil pressure response adjacent to abutment and piles*




*(a) Pile curvature in loose soil*

*(b) Pile curvature in dense soil*

*Figure 7 - Quantitative comparison of pile deflection behavior for a constant abutment deflection in loose soil and dense soil*




w = soil pressure

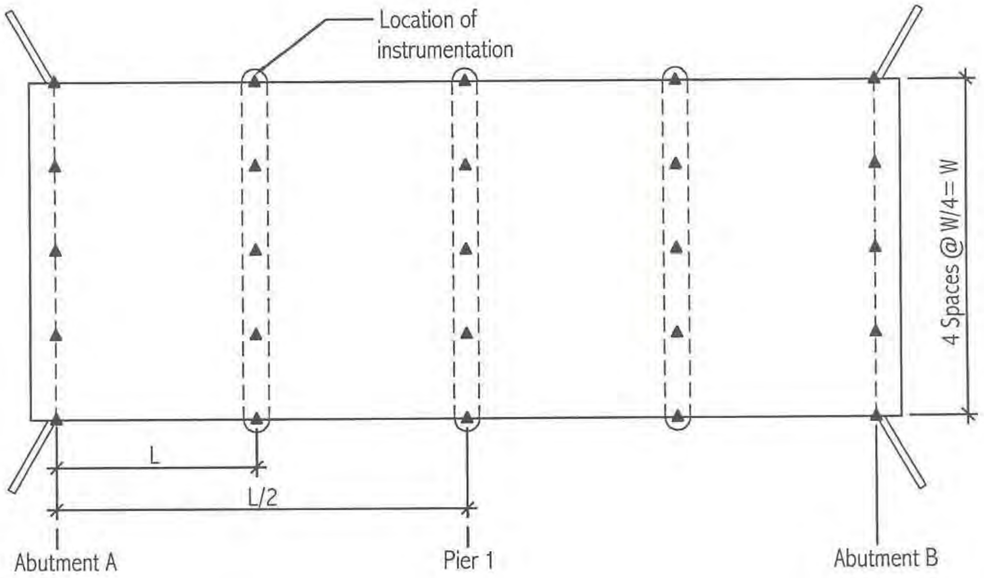
$P_S$ = soil pressure resultant force

$P_A$ = axial force in superstructure (= $P_s + P_p$)

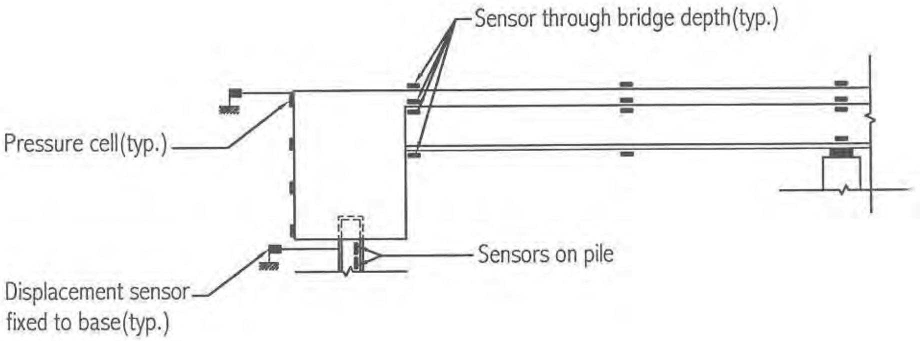
$P_p$ = shear resistance from the pile

$V_T$ = thermal load introduced into system to maintain rotational equilibrium

*Figure 8 - Free body diagram of the abutment and superstructure showing introduction of 'thermal load' to maintain equilibrium*




(a) Plan




(b) Elevation

*Figure 9 - Schematic instrumentation plan*

# APPENDIX A

In order to develop the level of understanding of integral bridges it is necessary to examine some fundamental and advanced questions regarding their performance. A literature search is necessary in order to address the following questions:

1. How does pile orientation affect rotation or translation of the integral abutments? Do integral abutments rotate or translate and what effect does compaction of backfill, pile size and orientation have on the mechanism of movement? What positive and/or negative impacts can be expected by orienting piles about either the weak or strong axis? Should piles be aligned with the direction of movement or centerline of bearing?

2. Does a plastic hinge actually form within the pile or are the stresses transferred to other abutment components? Where are the stress concentrations localized? There are two prevalent methods used to model integral abutments, the elastic and inelastic methods. Which method is most widely used and why? Have there been any more recent design methods proposed or researched? Additionally, if a plastic hinge doesn't form within the pile why orient the piles about the weak axis? Is the formation of a plastic hinge desired and if so is there a significant loss of pile capacity?

3. Is pre-excavation or pre-boring of holes for piling necessary? Only necessary for stiff soils? Does current research offer any recommendations? If you pre-excavate, should the piles be backfilled with sand, peastone, or gravel? Does the material type matter if the material is recommended to be loosely backfilled with minimum compaction?

4. How does the compactness of the backfill affect the ability of the abutments to move? Should loosely compacted backfill be required or should the backfill be compacted to standard specifications? Will the bridge simply "plow" the soil out of the way and develop a space for it to move on its own? How does backfill type and compaction level effect the formation of the "bump" at the end of the bridge?

5. There are three common wingwall designs, flare, in-line, and u-wall configurations. Hydraulic Engineers prefer the flared wingwalls while Structures Engineers prefer the u-wall configuration. What available research has addressed the performance of these various wingwall configurations, and does the u-wall configuration outperform the flared wingwalls or vice-versa? Does this issue even matter?

6. Realizing that current research limits the maximum abutment height to 4 meters, has research been conducted that identifies the amount or degree of passive earth pressures developed not only behind the integral abutments but also behind and on the bottom of the wingwalls? Has research been conducted on materials or methods to limit the amount of passive pressure distributed to the abutment by the backfill material?

7. Have integral abutments been analyzed using finite element analysis to map the stress concentrations (pile head, reinforcing steel, etc) as a result of thermal expansion/contraction? If so, are there performance monitoring results which support the documentation?

8. Research has been conducted by Iowa University that compares the actual deflection of the piles with the results from finite element difference methods. However, these piles were jacked to specified displacements rather than as a result of thermal expansion. Have piles been monitored on actual integral bridge abutments under thermal expansion or contraction? Is this an area where more data is required or is this an area of redundancy?

9. Many states recommend using only compact sections. What does current research suggest regarding the use of compact sections?

10. What are the current research findings with respect to approach slab design? What detail has proven to be most effective in accommodating cyclic bridge movements?

11. What are the current research recommendations on skew limitations and curved girder bridges?

12. Regarding the use of the AASHTO interaction equation 10.54.2, should designers consider the moment due to live load rotation? Should impact be included in the live load reaction?

13. What are the effects of lateral displacement on the frictional capacity of the pile? Should piles for jointless bridges be designed as end-bearing only?

14. Approach slab performance has been an issue often discussed at Integral Abutment Committee meetings. Have movements at the end of the bridge and approach slabs been monitored, if so have these measurements been utilized to design effective and efficient design details? What are the most recent research recommendations regarding approach slab design details?

15. Have construction materials such as closed cell foam or lightweight fill material been evaluated for assisting in unrestricted movement of jointless bridges? If so, has this resulted in lower stress concentrations within the piles, abutment, and/or approach slab? Has research been conducted regarding the use of a geotextile beneath approach slabs to facilitate the movement of the approach slab?

16. What research has been conducted to date evaluating the strains experienced in the deck concrete and steel superstructure?

17. What are the known limitations of integral abutment bridges? Are there any documented failures of integral abutment bridges? What research, if any, has been conducted on the performance of jointless superstructures on spread footings?

18. The different components of jointless bridges act together as a unit, what field measurements are necessary to model the performance of this unit as a whole?

19. Has there been a study conducted to verify and track the amount of savings experienced, especially maintenance costs? Are savings in life cycle costs actually realized?

20. Are jointless bridges best served by providing an expansion detail at the end of the bridge or at the end of the approach slabs?

21 What temperature range is recommended for design purposes for calculating the anticipated thermal deflection of both steel and concrete superstructures? Are the AASHTO recommendations overly conservative? Is the determination of effective temperature in accordance with ASHRAE temperature data, as outlined by Ralph Oesterle of CTL Laboratories in his summary report to FHWA on *Jointless and Integral Abutment Bridges*, valid for our region?

# APPENDIX B

# PERFORMANCE MONITORING OF JOINTLESS BRIDGES

## Phase I:  Synthesis of Technical Information for Jointless Bridge Construction

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