CHAPTER 2 – LITERATURE REVIEW

2.1. Introduction

The objective of the literature review is threefold:

- Synthesize the information available on the general behavior of integral abutment bridges
- Establish the current state of knowledge with regard to cyclic loading damage to piles of integral bridges, and
- Review available solution methods for the laterally loaded pile problem.

The general behavior of integral abutment bridges was investigated in the first year of the research period and the findings of the investigation were presented in a report to the project sponsor by Arsoy, Barker and Duncan (1999). This report can be found in Appendix A.

The literature was reviewed to establish the state of knowledge with regard to cyclic loading damage to piles supporting integral bridges. Only five published papers were found in the subject area. Additional articles pertaining to the cyclic loading performance of piles were collected from the literature to supplement the limited number of studies concerning the performance of piles supporting integral bridges.

A summary of the formulation and the available solution methods for the laterally loaded pile problem was also prepared and is presented in this chapter. The summary is primarily a synthesis of the information available in the published literature.

2.2. Cyclic Loading Damage to Piles of Integral Abutment Bridges

A literature review was conducted to establish the state of knowledge with regard to cyclic loading damage to piles supporting integral bridges. The literature review yielded a limited number of published papers in the subject area. Only five studies were found that describes the performance of piles of integral bridges. These studies were concerned with the behavior of the following bridges:

- The Cass County Bridge,
- The Boone River Bridge,
- The Maple River Bridge,
- A bridge in Rochester, Minnesota, and
- Route 257 overpass on I-81.

The Cass County Bridge is located 2 miles north of Fargo, North Dakota. The bridge is 450 ft. long and 32 ft wide with no skew (Jorgenson, 1983). The bridge consists of six spans, 75 ft. each, and has a concrete deck and prestressed concrete girders. The bridge is supported by HP10x42 piles oriented in weak-axis bending under abutments and strong axis bending under integral piers. The foundation piles were installed in 20 ft predrilled boreholes.

The Boone River Bridge is located in central Iowa, which includes some of the most complete and valuable data related to the performance of integral bridges. The bridge is 324.5 ft long and 40 ft wide with a skew of 45 degrees (Girton et al., 1991). The bridge has four continuous spans and consists of a concrete deck and prestressed concrete girders. Two of the piers of the bridge are located about 50 ft from each abutment. The third pier is located at the center of the bridge. Foundation piles are HP10x42 oriented in weak axis bending and battered 4:1 in the movement direction of the bridge. Piles were installed in 9 ft predrilled boreholes.

The Maple River Bridge located in northwest Iowa, which includes valuable data related to the performance of integral bridges like the Boone River Bridge. The bridge is 320 ft long and 32 ft wide with a skew of 30 degrees (Girton et al., 1991). The bridge has three spans and consists of a composite concrete deck and steel girders. Two piers of the bridge are located about 100 ft from each abutment. Foundation piles are HP10x42 oriented in weak axis bending and battered 3:1 in the movement direction of the bridge. Piles were installed in 12 ft predrilled boreholes.

Route 257 overpass on I-81 is the longest semi-integral bridge built in Virginia (Hoppe and Gomez 1996). The bridge has two equal spans and consists of steel girders and a concrete deck. The bridge is 98 m (320 ft) long and 25 m (82 ft) wide with a skew of 5 degrees. The bridge does not have an approach slab. Two rows of HP10x42 piles were used under the semi-integral abutments. One of the rows of the piles has no batter while the other row is battered 4:1 in the movement direction of the bridge. Hoppe and Gomez (1996) monitored the performance of the bridge from summer of 1993 to January of 1996. No instrumentation was used for monitoring pile behavior. Hoppe and Gomez (1996) report no major problem neither with the piles nor with the bridge. Settlement of the approach fill was reported and maintenance of the approach fill to cop with the settlement was discussed in their study.

The bridge located in Rochester, Minnesota was built 1996 and monitored from September 1996 to September 1998. The bridge has three equal spans of 22 m each for a total length of 66 m and a width of 12 m. Mn/DOT Type 45M prestressed concrete bridge girders with a spacing of 3.4 m were used. The integral abutments were 0.9 m wide and 1.5 m high and supported by HP12x53 piles oriented in weak axis bending.

Each bridge was monitored for approximately two years. During the monitoring period, bridges were subject to real-life loading, including the temperature-induced cyclic loading. Table 2.1 summarizes the behavior of piles for each bridge described above during their monitoring period. All bridges were supported by steel H-piles. The piles were able to tolerate the loads, including those induced by temperature variations. No sign of damage to piles was reported. It appears that steel H-piles supporting integral bridges can withstand cyclic loading as long as the maximum stresses remain equal to or less than the nominal yield stress of the pile material (36 ksi).

Table 2.1. Summary of behavior of piles supporting integral abutment bridges

Bridge	Reference	Maximum pile stress (% of nominal yield)	Remarks
The Cass County Bridge	Jorgenson (1983)	100	Strain gages failed. Author estimated stresses based on analytical methods and concluded that maximum pile stresses were around the yield stress, and that plastic hinge formation in piles was not possible. Piles were able to tolerate 2 inches of bridge contraction and about 3 inches of total displacement without damage.
The Boone River Bridge	Girton et al. (1991)	60+	Piles were able tolerate 1.2 inches of bridge contraction and about 2 inches of total displacement without damage.
The Maple River Bridge		75+	Piles were able tolerate 1.6 inches of bridge contraction and about 2.5 inches of total displacement without damage.
Route 257 overpass on I-81	Hoppe and Gomez (1996)	Not reported	Pile stresses were not monitored. However, no pile damage was reported.
Rochester, Minnesota Bridge	Lawver et al. (2000)	100	Piles were able tolerate 0.65 inches of bridge contraction and 1.06 inches of total displacement without damage.

2.3. Performance of Piles under Cyclic Lateral Loading

Cyclic loading performance of piles as a stand-alone structural component provides important information about the damage potential of piles supporting integral bridges. Additional articles pertaining to the cyclic loading performance of piles were collected from the literature to supplement the limited number of studies concerning the performance of piles supporting integral bridges. Table 2.2 lists some of the studies concerning cyclic load performance of piles. Pertinent conclusions of these studies are also presented in the table. The emphasis is given to the number of cycles and the maximum stress induced in the piles during cyclic loading.

Table 2.2. Performance of piles under cyclic lateral loading

Reference	Type of study	Relevant conclusions
Alizadeh and Davisson (1970)	Full scale field experiments. A HP14x73 steel pile and a 20-inch prestressed concrete pile were subjected to 100 load controlled cycles. The soils are mostly medium dense sands and medium dense silty sands. No vertical load was applied to the piles.	Maximum stress in the H-pile was about 80% of the nominal yield stress. Stresses in the concrete pile were not reported. In the 100^{th} cycle, deflections of the concrete pile and the H-pile were 70% and 90% larger, respectively, than their deflections in the 1^{st} cycle for a given load. No damage to piles were reported
Matlock et al. (1980)	Full scale field experiments. A single, a five- and ten-pile circular pile groups were subjected to 100 load controlled cycles. All piles were 6-inch pipe piles. The soils were very soft slightly over-consolidated clays.	Maximum stresses in the piles were about 80% of the nominal yield stress. No damage to piles was reported.
Meimon et al. (1986)	Full scale field experiments. A single and a 2x3 pile group were subjected to 10,000 displacement controlled cyclic loading. The piles were steel H-piles. The soil was 1 m of highly plastic clay underlain by 4 m of low plastic clay. No vertical load was applied.	Maximum stresses were about 20% of the nominal yield stress of the piles. A reduction of 20% in bending stresses at the $10{,}000^{th}$ cycle was reported. No damage to piles was reported.
Brown et al. (1987)	Full scale field experiments. A single and a 3x3 pile group were subjected to 100 load cycles. The piles were 10-inch steel pipe piles. The soil was stiff to very stiff clay. No vertical load was applied.	Maximum stresses were about 100% of the nominal yield stress of the piles. No damage to piles was reported.
Brown et al. (1988)	Full scale field experiments. A single and a 3x3 pile group were subjected to 100 load cycles. The piles 10-inch steel pipe piles. The soil was submerged dense sand. No vertical load was applied.	Maximum stresses were about 100% of the nominal yield stress of the piles. No damage to piles was reported.
Oesterle et al. (1998)	Large scale laboratory experiments. A 14-inch square prestressed concrete and a HP10x42 steel pile were subjected to 50 displacement controlled cyclic loading. Piles were tested as cantilever columns with no soil around them. A vertical load of 90 kips was applied to both piles.	Pile stresses were well in excess of nominal yield stress even in the first load cycle for both piles. Local buckling in the H-pile was reported. The H-pile was able to carry the applied vertical load without any sign of plastic hinge development. Significant damage was reported for the concrete pile.
Jaradat et al. (1998)	Large scale laboratory experiments. Tests were performed on 10-inch circular reinforced concrete columns. Two different reinforcing ratios were used. Eight specimens were subjected to displacement controlled incremental cycles. The maximum displacement in the first three cycles was limited to the estimated yield displacements. Subsequently, the displacement levels were increased to 2, 3, 4, 5, and 6 times the estimated yield displacement unless the columns failed before. Vertical loads varied between 17 and 19 kip.	Up to the estimated yield deflection, no damage was reported. Above the yield displacement, damage started to take place. Complete failure of the columns took place for displacements between 2 to 5 times of the yield displacements.

2.4. Governing Differential Equation for the Laterally Loaded Pile Problem

The formulation of the laterally loaded pile problem is based on either the subgrade reaction approach or the elastic continuum approach (Horvath, 1991). The subgrade reaction approach is based on the Winkler hypothesis (1867), and is most widely used method in the subject area. With this approach, a laterally loaded pile is treated as a beam resting on an elastic subgrade. A series of closely spaced, independent elastic springs replaces the subgrade. The governing differential equation is given as (Prakash and Sharma, 1990):

$$EI\,\frac{d^4 y}{dx^4} + P_x\,\frac{d^2 y}{dx^2} + k_h\,y = 0 \qquad \textbf{(2.1)}$$

Where, EI = flexural stiffness of pile

y = lateral deflection

x = length along pile

P_x = axial load

k_h = spring constant to represent the soil.

The solution of the above differential equation is obtained with appropriate representation of the soil by a spring constant and with the inclusion of the proper boundary conditions. A solution can be obtained by either in closed form (exact), or using approximate solution methods. Although the closed form solutions are desired, they can be time consuming to achieve and limited in use. Approximate solutions are most likely to be used in practice since they provide a satisfactory answer for most of the time. Approximate solutions include:

- series expansion,
- finite difference method,
- finite element method, and
- other methods based on some or all of the above.

2.5. Closed Form Solution

It is usually assumed that the vertical loads and the lateral loads are not coupled. Thus, the laterally loaded pile problem is solved by neglecting the effect of the vertical load. Under this assumption, the governing differential equation becomes:

$$EI\,\frac{d^4 y}{dx^4} + k_h\, y = 0 \qquad \textbf{(2.2)}$$

If one assumes that k_h is constant, which is a good approximation for clays but not for granular soils and divide both sides of Equation (2.2) by EI, the following is obtained

$$\frac{d^4 y}{dx^4} + 4\alpha^4 y = 0 \qquad \textbf{(2.3)}$$

in which $\alpha^4 = k_h/4EI$. The solution to Equation (2.3) is given by Equation (2.4) as obtained first by Hetenyi (1946).

$$y = e^{\alpha x}\left(C_1 \cos\alpha x + C_2 \sin\alpha x\right) + e^{-\alpha x}\left(C_3 \cos\alpha x + C_4 \sin\alpha x\right) \qquad \textbf{(2.4)}$$

Where, x = depth from pile top, and

C_1, C_2, C_3, and C_4 = constants to be determined from the boundary conditions.

These constants (C_1, C_2, C_3, and C_4) for long and short piles are given in Desai and Christian (1977). The reader is referred to this source for further detail. A brief summary is presented below.

2.5.1. Long Piles

The deflection at the tip of the long piles is negligible. Mathematically, this corresponds to zero deflection when x has very large values. For this condition, $C_1 = C_2 = 0$. At the top of the pile (x=0), $y`` = M_t/EI$ from which C_4 is obtained as:

$$C_4 = -\frac{M_t}{2EI\,\alpha^2} \tag{2.5}$$

Where, M_t = applied moment at the top of the pile. C_3 is obtained from y``` = V_t /EI as :

$$C_3 = \frac{V_t}{2EI\,\alpha^3} + \frac{M_t}{2EI\,\alpha^2} \tag{2.6}$$

in which, V_t = applied shear force (lateral load) at the top of the pile.

2.5.2. Short Piles

The boundary conditions y``=M_t /EI and y```=V_t /EI at x=0 yield Equations (2.7) and (2.8), respectively.

$$C_2 - C_4 = \frac{M_t}{2EI\,\alpha^2} \tag{2.7}$$

$$-C_1 + C_2 + C_3 + C_4 = \frac{V_t}{2EI\,\alpha^3} \tag{2.8}$$

At x = L (at the tip of the pile), EIy``=0 and EIy```=0. Applying these boundary conditions, the following two equations are obtained (Desai and Christian, 1977).

$$2\,\alpha^2 e^{\alpha L}\left(C_2 \cos\alpha L - C_1 \sin\alpha L\right) + 2\,\alpha^2 e^{-\alpha L}\left(C_3 \sin\alpha L - C_4 \cos\alpha L\right) = 0 \tag{2.9}$$

$$\begin{aligned} &2\,\alpha^3 e^{\alpha L}\left(C_2 \cos\alpha L - C_1 \sin\alpha L - C_2 \sin\alpha L - C_1 \cos\alpha L\right) \\ &+ 2\,\alpha^3\, e^{-\alpha L}\left(-C_3 \sin\alpha L + C_4 \cos\alpha L + C_3 \cos\alpha L + C_4 \sin\alpha L\right) = 0 \end{aligned} \tag{2.10}$$

Equations (2.7) to (2.10) form a linear system with respect to C_1, C_2, C_3, and C_4. The solution of this linear system provides the relations for C_1, C_2, C_3, and C_4.

2.6. Approximate Solution Methods

2.6.1. Series Expansion

A series solution to Equation (2.2) can be obtained by using the following series:

$$y = \sum_{i=0}^{\infty} C_i \, x^i \qquad \textbf{(2.11)}$$

Where, x = depth,

y = lateral displacement, and

C_i = coefficients yet to be determined.

An approximate solution can be obtained by truncating the series. The accuracy of the solution will increase with increasing number of coefficients, but determining the coefficients will get equally difficult. Therefore, the series solution is likely to handle simple geometric and boundary conditions. For complicated geometric and boundary conditions numerical techniques such as the Finite Difference and the Finite Element methods become more advantageous (Desai and Christian, 1977).

2.6.2. Finite Difference Method

The finite difference method can be applied to laterally loaded piles if the piles are initially straight and carry no bending moment originated from pile driving. A detailed formulation of the method can be found in Reese and Desai (1977). A brief formulation will be presented below, which was borrowed from Reese and Desai (1977).

The pile is first divided into a number of small elements. The governing differential equation for each element is the same as Equation (2.1). Firstly, the fourth derivative is written as:

$$\left[\frac{d^2}{d\,x^2}\left(EI\,\frac{d^2\,y}{d\,x^2}\right)\right]=\left(\frac{d^2\,M}{d\,x^2}\right) \tag{2.12}$$

Secondly, the terms of the governing differential equation will have the following finite difference forms for the i^th element, as given by Reese and Desai (1977). Based on the equilibrium of forces on a pile element,

$$\left(\frac{d^2\;M}{d\;x^2}\right)_i=\frac{\left(\frac{d\;\;M}{d\;\;x}\right)_{i-1/2}-\left(\frac{d\;\;M}{d\;\;x}\right)_{i+1/2}}{h} \tag{2.13}$$

in which h is the length of the i^th pile element. The formulation in Equation (2.13) means that the second derivative of M with respect to x for the i^th element is equal to the slope of the first derivative of M, which is numerically calculated with respect to the values of the first derivatives at the midpoints of the (i-1)^th and (i+1)^th elements. After substituting the finite difference forms for dM/dx and d^2y/dx^2 into Equations (2.12) and (2.13), the following is obtained.

$$\left(\frac{d^2\,M}{d\,x^2}\right)_i\approx\frac{1}{h^4}\left[\begin{array}{l}EI_{i-1}\,y_{i-2}-2(\,EI_{i-1}+EI_i\,)\,y_{i-1}+(\,EI_{i-1}\;+\;4\,EI_i\;+\;EI_{i+1}\,)\,y_i\\ -2(\,EI_i-EI_{i+1}\,)\,y_{i+1}+EI_{i+1}\,y_{i+2}\end{array}\right] \tag{2.14}$$

The finite difference for the second term of Equation (2.1), assuming P_x is constant over the entire length of the pile, is of the form:

$$P_x\,\frac{d^2\,y}{dx^2}=\frac{P_x\;(\,y_{i-1}-2\,y_m+y_{i+1}\,)}{h^2} \tag{2.15}$$

Finally, Reese and Desai (1977) give the final form of the finite difference form of Equation 2.1 for the i^th pile element as:

$$EI_{i-1}\, y_{i-2} + (-2EI_{i-1} - 2EI_i + P_x h^2)\, y_{i-1} + (EI_{i-1} + 4EI_i + EI_{i+1} - 2P_x h^2 + k_h h^4)\, y_i$$
$$+(-2EI_i - 2EI_{i+1} + P_x h^2)\, y_{i+1} + EI_{i+1}\, y_{i+2} = 0 \quad \textbf{(2.16)}$$

Equation (2.16) needs to be written for all of the pile elements. This will lead to a set of linear equations. After applying the boundary conditions, the linear set of the equations is solved and therefore, the displacements at the nodes are determined. Other quantities (slope, moment, and shear) are numerically derived from the displacements. The moment at node *i* is calculated as:

$$M_i \approx \frac{EI_i\,(y_{i-1} - 2y_i + y_{i+1})}{h^2} \quad \textbf{(2.17)}$$

The boundary conditions involving the derivatives of y are expressed in the finite difference forms as well. In order to achieve this, one needs to introduce some imaginary nodes at the boundaries (top and bottom of the pile), according to Reese and Desai, (1977). These nodes are called fictitious or phantom points. Details can be found in their textbook.

2.6.3. Finite Element Method

The true behavior of a laterally loaded pile is a three-dimensional problem. The vertical displacement is not due to the lateral loading; it takes place due to the vertical loading. Therefore, the lateral loading causes lateral displacement on planes perpendicular to the vertical axis of the pile along the pile length. It is usually of greatest interest to know the largest displacement on these planes, particularly at the ground level. The bending moment and the applied torsion are also of interest. Equation (2.1) provides the two dimensional representation of the laterally loaded pile problem. In most cases, this will lead to a satisfactory solution. However, it should be born in mind that three dimensional representation of the pile is the most correct approach. Equation (2.1) can be modified as follows:

$$EI \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} + p_{(x)} = 0 \qquad \textbf{(2.18)}$$

Where $p_{(x)}$ = soil pressure acting on the pile ($p_{(x)} = k_h y$),

P_x = axial force, and

k_h = coefficient of horizontal subgrade reaction, which is expressed as the ratio of applied soil pressure p to pile deflection y.

The essence of the finite element method lies in obtaining a functional, function of function(s), and minimizing this functional, which is equivalent to solving the governing differential equation (Kapania class notes, 1997). Over the years, mathematical foundations of the method have been perfected and various approaches have been established in obtaining a functional for a given problem. One of the available techniques is to obtain the "weak form" of the differential equation given using variational methods. If both sides of Equation (2.18) are multiplied by a slowly varying test function v(x) and integrated over the domain (0< x <L), an integral is obtained, which leads to the weak form.

The test function can be any function with one limitation: v(x) is zero whenever the essential boundary terms are specified (Kapania, 1997). This means that the solution is exact at boundaries and approximate elsewhere. The integration is done by integration by parts, which reduces the order of derivatives, thereby gives the name "weak form" because the derivatives are weakened. After partial integration, the function v(x) is assumed to be the same as the solution y(x) and the weak form is obtained. The mathematics involved is too lengthy to fully present for the scope of this chapter. The weak form is given as follows:

$$\pi = \frac{1}{2} \int_{x=0}^{L} \left[EI \, (y'')^2 + P_x \, (y')^2 + p_{(x)} \, y \right] dx \qquad \textbf{(2.19)}$$

An approximate solution to Equation (2.19) can be found by dividing the pile into a finite number of elements and assuming a solution over each element:

$$y_{(x)} = \sum_{j=1}^{m} N_j q_j \qquad \textbf{(2.20)}$$

Where, m = No. of unknowns per node,

N_j = interpolating functions called shape functions, and

q_j = nodal unknowns yet to be calculated.

The shape functions N(x) must be twice differentiable as can be understood by examining Equations (2.19) and (2.20). The next step is to substitute Equation (2.20) and the first and the second derivatives from Equation (2.20) into Equation (2.19). The stationary values of this new equation after the substitution is obtained by equating the partial derivatives of the new π with respect to q_i to zero, by which the desired condition of the equilibrium is provided. This leads to the following equation for a given element:

$$\sum_{j=1}^{m} \left[\int \left(EI\, N''_i N''_j + P_x N'_i N'_j \right) dx \right] q_j = Q_i \qquad \textbf{(2.21)}$$

In which Q_i = nodal forces obtained by assuming a variation for $p_{(x)}$.

The symbolic representation of Equation (2.21) can be given as: $[K]\{q_j\}=\{Q_i\}$, in which K_{ij} is given in Equation (2.22) and Q_i is given in Equation (2.23).

$$K_{ij} = \int \left(EI\, N''_i N''_j + P_x N'_i N'_j \right) dx \qquad \textbf{(2.22)}$$

$$Q_i = \int N_i\, p_{(x)}\, dx \qquad \textbf{(2.23)}$$

Using Equations (2.22) and (2.23), the stiffness matrix and the load vector, respectively, is determined for each element. These quantities are then assembled together to obtain the global stiffness matrix, and the global load vector. The solution is obtained by solving $[K]\{q_j\}=\{Q_i\}$for $\{q_j\}$.

The most crucial point of the solution is the proper representation of the soil modulus through $p_{(x)}$. If $p_{(x)}$ is assumed to be linear, then a system of linear equations is obtained. The solution becomes trivial with matrix solvers. It is a well known fact that $p_{(x)}$ is a function of the lateral deflection, which leads to a set of non-linear equations. For nonlinear $p_{(x)}$, the solution is obtained by iterative procedures by assuming deflections for each node and thereby calculating $p_{(x)}$ and solving for q_j (nodal unknowns) until the assumed and the calculated nodal unknowns are the same within a tolerance range. The Newton and the Modified Newton methods are mostly used for iterations.

2.7. Empirical Methods

2.7.1. The Method of p-y Curves

The p-y method is the most widely used empirical method in the subject area. The method considers the fact that the relationship between the soil pressure (p) and the pile deflection (y) is non-linear. The greatest contributors to the development of the p-y method are Matlock (1970), Reese et al. (1974), Reese and Welch (1975), and Bhushan et al. (1979).

The essential of the method is to introduce a series of p-y curves to represent the true behavior of soils by considering the non-linearity of the soil modulus. The main purpose of the method is to obtain a representative value of k_h for the desired depth and deflection values. This is accomplished through an iterative process by assuming a deflection and calculating the value of k_h. The iterations are continued until the assumed and calculated deflections are the same within a tolerance limit. When representative p-y curves are used, the method is capable of reflecting the real deflection behavior of the pile and the moment distribution along the pile. The challenge is to obtain a representative set of p-y curves for each site.

Several procedures are available to estimate the p-y curves (Reese and Wang, 1997; Stevens and Audibert, 1979; Bhushan et al., 1979; Briaud et al., 1982; O'Neill and Gazioglu, 1984; O'Neil and Dunnavant, 1984; Dunnavant and O'Neill, 1985; Reese and Cox, 1968; Reese et al, 1975; Kooijman, 1989; and Brown et al., 1989).

2.7.2. Evans and Duncan (1982) Method

This method is based on the results of series of computer analyses/simulations with COM624 using the p-y curves method. The results are intelligently normalized and compiled in chart forms such that the lateral load at the ground level and the maximum moment can be estimated directly for a given deflection at the pile head. The charts can be used for either fixed head or free head piles in either cohesive or cohesionless soils. The normalization process included the definition of a characteristic shear load and a characteristic moment. These definitions are based on empirical equations and can be found in Evans and Duncan (1982). The deflection is normalized by the width or the diameter of the pile.

To find the lateral load and the corresponding moment, one simply needs to assume a tolerable deflection at the pile head and read off the normalized lateral load and the corresponding normalized maximum moment. These normalized load and normalized moment are then multiplied by the characteristic load and the characteristic moment, respectively, to find the lateral load at the top of the pile and the maximum moment in the pile, respectively.

2.7.3. SALLOP: Simple Approach for Lateral Loads on Piles (1997)

SALLOP is a simplification of a method developed at Texas A&M University based on the p-y curves concept. The method is geared towards making use of the pressuremeter limit pressure and the pressuremeter modulus. Briaud (1997) indicates that SALLOP is a semi-empirical and semi-theoretical method. It was developed based on the theory and then the theoretical equations were modified based on 20 full-scale pile tests.

The method assumes uniform Winkler soil and uses the closed-form solution as first developed by Hetenyi (1946). The essential of the method is based on the fact that the shear forces in a laterally loaded pile are negligible at some depth, called zero-shear depth, which is mostly responsible for the behavior of the pile. The horizontal equilibrium at this depth constitutes the basis of SALLOP. However, the challenge is to find the extent of this depth. Briaud (1997) provides a recommendation about this zero-shear depth along with the details of how one would make use of the pressuremeter test results to estimate the maximum lateral load and the maximum moment acting on the pile.

2.8. Equivalent Cantilever Method

The equivalent cantilever method is proposed for designing piles of integral bridges by Greimann and Wolde-Tinsea (1988) and Abendroth, Greimann and Ebner (1989). This method appears to be widely accepted by the bridge engineers. The method is based on analytical and finite element studies. An equivalent cantilever column is used to replace the actual pile. In other words, the soil-pile system is reduced down to an equivalent cantilever column. Two alternatives are provided, one involving elastic behavior, and the other involving inelastic behavior of the piles. Finite element simulations indicated that both alternatives were conservative. Both alternatives are concerned with the vertical load carrying capacity of piles under lateral displacements induced by temperature changes. A worked-out example on the design of an integral abutment using the equivalent cantilever method is given by Barker et al. (1990). Girton et al. (1991) who evaluated this method experimentally, concluded that the equivalent cantilever column model is sufficiently accurate for design purposes. The method does not consider the effects of the abutment/approach fill interactions and the effects of the induced stresses in the superstructure.

2.9. Conclusions

It appears that steel H-piles are the choice for support of integral bridges as all five of the integral bridges in Table 2.1 had steel H-piles. These piles were able tolerate cyclic stresses up to their nominal yield stress capacity of 36 ksi. It can be concluded that the steel H-piles of the integral bridges will not be subject to damage due to temperature variations as long as the maximum combined pile stress does not exceed the nominal yield stress of the steel.

Concrete piles may keep their integrity under cyclic lateral loading until their yield stress. Concrete piles may fail suddenly for stresses slightly higher than the yield stress. In contrast, steel piles do not fail suddenly. Due to tension cracks, the vertical load carrying capacity of concrete piles decreases with increasing cycles.

The methods for solving laterally loaded pile problems are mostly empirical since the soil modulus is not a unique soil property. Numerical methods such as finite difference and finite element methods provide very accurate results if the soil pressure is appropriately represented.

The equivalent cantilever method does not consider the effects of the abutment/approach fill interactions.