



CHAPTER 3 FINITE ELEMENT MODELING OF SLAB-ON-GIRDER BRIDGE

3.1 Introduction

The use of FEA on the analysis of slab on girder bridges, which are considered as hybrid structure has been recently recognized. Over the years, various three-dimensional modeling techniques have been used by many researchers to idealize these structural components as discussed in Chapter 2. The first important step in finite element modeling is the selection of appropriate modeling techniques and elements. This Chapter aims to present an effective method used to construct the finite element model of a composite concrete slab on multi steel I-girder bridges. By using a useful computer program SAP2000 (SAP2000, 2002), the evaluation of several specific modeling techniques were carried out so as to accurately predict bridge response with reasonable computational time. Furthermore, the effect of composite action between slab and girder on the accuracy of the obtained results is also investigated. In particular, this Chapter has been focused on the accuracy of finite element methods for this hybrid structure. Several modeling approaches, namely the Eccentric Shell Model (ESM), Eccentric Shell-Beams Model (ESBM) and Simple Shell-Beam Model (SBM) were presented herein to simulate the behavior of considered bridge decks. Finally, those finite element models were verified against a reliable field load test results of a Texas Bridge deck experiment carried out by Fang et al. (1990).

3.2 FEA Modeling of Slab-on-Girder Bridge Decks

The modeling of primary structural elements, i.e. concrete bridge deck and steel I-girders are conducted in this section. In addition, the technique used to model composite action between concrete bridge deck and steel I-girders is also presented. It should be noted that this study does not consider the use of three-dimensional solid elements to model all of the components of bridge superstructures. This is because this type of modeling usually produces meshes with a very high number of degrees of freedom, which makes them impractical with current computing capabilities. Furthermore, usually the additional accuracy obtained when using this modeling technique does not justify the additional computational expense.

3.2.1 Modeling of Deck Slab

In general, bridge deck behavior is governed mainly by flexure, which is best modeled by using high order elements such as solid, shell and plate elements. However, the use of low order elements such as beam element appears to be appropriate in some bridge

components. In this research, four-node quadrilateral linear thick and thin shell elements and nonlinear layered shell element with six degrees of freedom per node as shown in Figure 3.1 were selected. These elements were tested numerically in the literature to evaluate their ability to accurately model the bridge deck.

SAP2000 shell elements provide unique element formulations which avoid the 'shear locking' problems found with classical finite elements. The ability of SAP2000 to output shell forces and moments at each shell joint which are integrated over the shell thickness, and then sum them, provides several advantages. The shell element is a four-node formulation that combines separate membrane and plate-bending behavior. The four-joint element does not have to be planar. The membrane behavior uses an iso-parametric formulation that includes translational in-plane stiffness components and a rotational stiffness component in the direction normal to the plane of the element (Taylor and Simo, 1985 and Ibrahimbegovic and Wilson, 1991).

The plate bending behavior includes two-way, out-of-plane, plate rotational stiffness components and a translational stiffness component in the direction normal to the plane of the element. A thin-plate (Kirchhoff) formulation that neglects transverse shearing deformation is used to represent thick shell elements. Likewise, a thick-plate (Mindlin/Reissner) formulation which includes the effects of transverse shearing deformation is employed for thick shell elements. Each Shell element has its own local coordinate system for defining material properties and loads, and for interpreting output. Each element may be loaded by gravity or uniform load in any direction. Stresses and internal forces and moments in the element local coordinate system are evaluated at the 2-by-2 Gauss integration points and extrapolated to the joints of the element. An approximate error in the element stresses or internal forces can be estimated from the difference in values calculated from different elements attached to a common joint. This will give an indication of the accuracy of a given finite element approximation and can then be used as the basis for the selection of a new and more accurate finite element mesh.

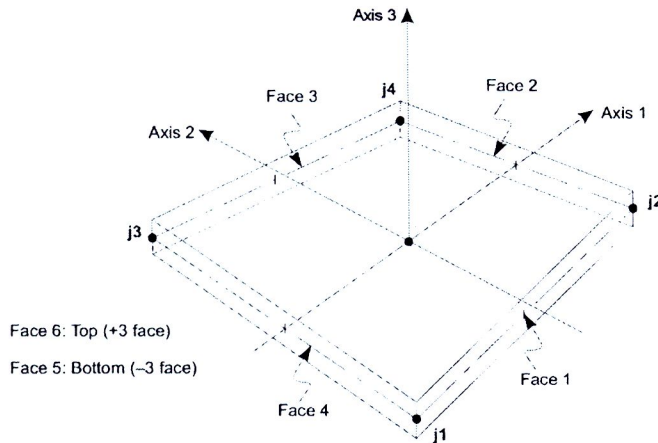


Figure 3.1 Four-node quadrilateral shell element

In general, a sufficiently fine mesh should be chosen to ensure accurate numerical results. In a study of a bridge deck by Cao (1996), it is recommended that the deck be divided into 40 elements along a longitudinal span of 15 m (49 ft in length) and that the slab be divided into 14 elements between two girders that have a 2.10-m spacing (7 ft in width). Therefore, his mesh aspect ratio is 2.45. Note that a loaded plate had a size of 0.20 m (8 inch) in width by 0.50 m (20 inch) in length. However, a finer mesh is necessary to model either the loaded plate or tire area appropriately in a finite element model. Several mesh sizes of the deck slab are performed for the mesh convergence. In this study, the point load is placed over slab constructed by the varied mesh size, which the minimum one is about the size of the loaded plate used in the strain measurement. The stress converts for a size of 0.20 m in width by 0.50 m in length at loading area that a finer mesh be used in the vicinity of rapid change in loading.

3.2.2 Modeling of Girders

In this subsection, different modeling techniques for girders are discussed. There are two different element types used in the girder model, namely shell element (discussed in previous subsection) and beam element. For beam element, a general three-dimensional beam-column formulation which includes the effects of biaxial bending, torsion, axial deformation, and bi-axial shear deformations is used (Bathe and Wilson, 1976). Two-node linear elements with six degrees of freedom per node at both of its connected joints as shown in Figure 3.2 are selected. A beam element is modeled as a straight line connecting two points. Each element has its own local coordinate system for defining section properties and loads, and for interpreting output. Each beam element may be loaded by self-weight, multiple concentrated loads, and multiple distributed loads. Element internal forces are produced at the ends of each element and at specific stations along the length of the element.

In this research, three different modeling techniques as shown in Table 3.1 are considered. The first technique called herein afterward G1 uses shell elements for modeling both flanges and web of the girder. The second called herein afterward G2 uses beam elements and dimensionless beam elements for modeling of the entire girder and lower support base, respectively. The last one called herein afterward G3 uses only beam elements to represent the entire girder at its centroid.

Table 3.1 Girder models according to adopted element types

Model of girder	Element type	
	Web	Flange
G1	shell element	shell element
G2	beam element + dimensionless beam element	
G3	beam element	

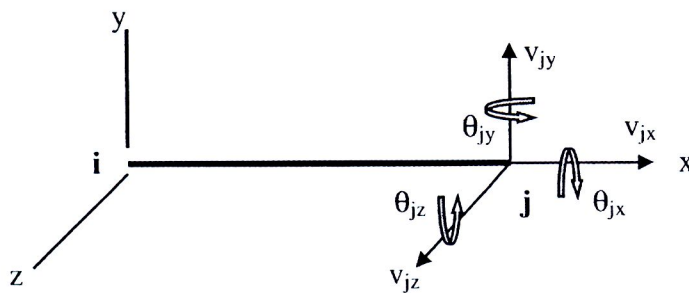


Figure 3.2 Beam element with six degrees of freedom at each node

3.2.3 Interaction Modeling between Girders and Slab

In this study, the modeling of both full composite action (**CAM**) or Eccentric Modeling of Girders and non composite action (**NCAM**) or Non-Eccentric Modeling of Girders between concrete deck and girders is considered. Both modeling techniques of interactions are represented by thick or thin shell and beam elements for deck and girders, respectively as demonstrated in Table 3.2. It should be noted that the element of shear connectors and the effect of eccentric girders are not considered for **NCAM**.

Table 3.2 Composite and Non-composite modeling of bridge superstructures

Modeling Approach		Type of Element	
		Deck Slab	Girder
CAM	Eccentric Shell Model (ESM)	Thick/Thin/Layered Shell	G1 : Thick Shell
	Eccentric Shell-Beams Model (ESBM)	Thick/Thin/Layered Shell	G2 : Beam & Dimensionless
	Simple Shell-Beam Model (SBM)	Thick/Thin Shell	G3 : Beam (Eccentric)
NCAM	Simple Shell-Beam Model (SBM)	Thick/Thin Shell	G3 : Beam (Non-Eccentric)

Due to its composite-action behavior, the concrete deck slab tends to deform as one unit with the girder through shear studs. The occurrence of in-plane axial membrane forces in the deck represented **CAM** are investigated via the next laboratory testing. To this end, three different modeling approaches of bridge girders are investigated for **CAM**. In general, accuracy of the result mainly relies on the element types used. Therefore, great careful consideration on element mesh selection is then paid attention in this study. The parametric study has been conducted to investigate the effect of different element type on bridge deck responses. The Eccentric Shell Model (**ESM**), Eccentric Shell-Beams Model (**ESBM**) and Simple Shell-Beam Model (**SBM**) discretize the concrete slab by shell elements and the girders by **G1**, **G2** and **G3**, respectively. It is note that **SBM** is recognized as a simple finite element modeling approach widely used in the literature by several researchers (Fang et al., 1990; Hays et al. 1994 and Cao 1996). Figure 3.3(a), 3.3(b) and 3.3(c) respectively show these **CAM** of **ESM**, **ESBM** and **SBM**.

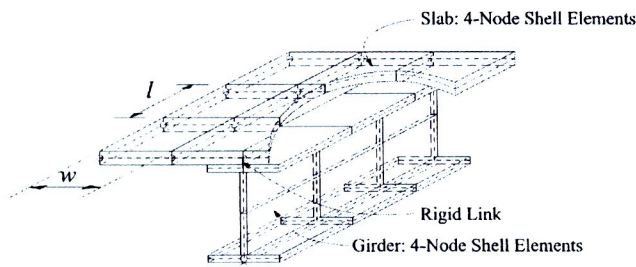
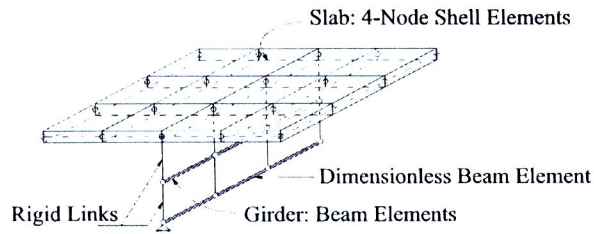
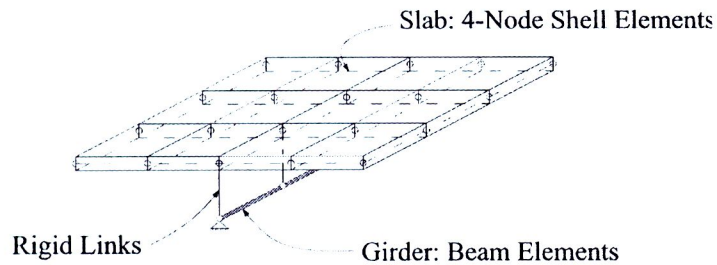
(a) Eccentric Shell Model (**ESM**)

Figure 3.3 Composite Action Model (**CAM**) using Eccentric Girder:
 (a) Eccentric Shell Model (**ESM**); (b) Eccentric Shell-Beam Model (**ESBM**); (c) Simple Shell-Beam Model (**SBM**)



(b) Eccentric Shell-Beam Model (ESBM)



(c) Simple Shell-Beam Model (SBM)

Figure 3.3 (Con't) Composite Action Model (CAM) using Eccentric Girder:

(a) Eccentric Shell Model (ESM); (b) Eccentric Shell-Beam Model (ESBM): (c) Simple Shell-Beam Model (SBM)

In general, the resistance offered by steel girders to different loads on the bridge depends upon the amount composite action between the bridge deck and steel girders. To model the partial composite action in FEA, different methods are proposed by some researchers. To capture well the bridge structural behavior, the eccentric beam model based on CAM is used to model the bridge superstructure. The bridge deck and girders are typically connected by a so-called rigid link element at nodal points to take into account for the composite action and the offset between the centroidal axis of the slab and that of a girder. These rigid links are represented by very stiff beam elements, which are properly used to resist the combined action between bending and shear in the bridge deck. Rigid links kinematically constrain all six degrees of the freedom between the two connected nodes. By using this rigid links, beam elements with very massive stiffness are selected and connected in series to model the partial composite action. However, to ensure the real behavior of composite action, the stiffness property of rigid links needs to be investigated. In literature, rigid links can be generally modeled by beam elements, with the minimum elastic modulus assumed to be about 10^3 times of that of the girders and the section properties are the same as those of the girder elements (Hoit, 1995 and Liu and Quek, 2003).

In this study, a parametric study on the stiffness of rigid links was carried out. The CAM, which follow Texas Bridge deck study (Fang et al. 1990) was constructed so that rigid links were completely attached to the deck element along bridge span to produce

composite-action mechanism. The results showed that the stress responses occurring at a slab over interior girder converged to a certain value when the elastic modulus of rigid links (E_r) is increased. As shown in Figure 3.4, E_r which is equal to 10^6 times of E_s (elastic modulus of steel) appears to be a suitable value and will be accepted in the CAM.

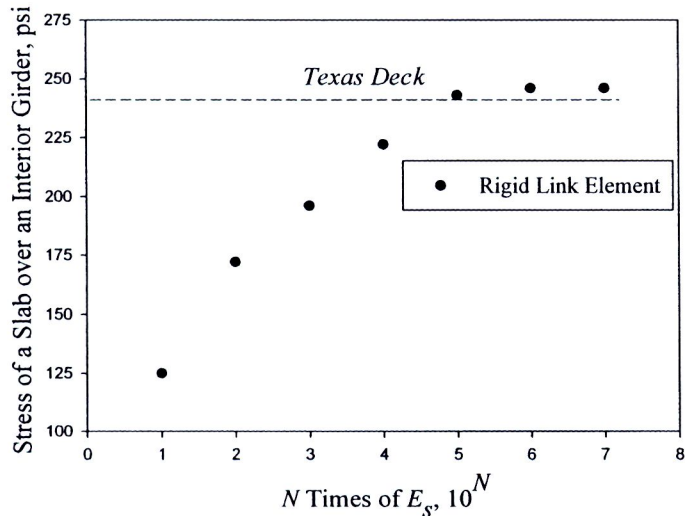


Figure 3.4 Stress response at contact point for various mesh sizes

The model of NCAM as shown in Figure 3.5 is also examined to observe its modeling reliability. This type of model was originally introduced by Hays et al. (1986). The deck slab are modeled using quadrilateral shell elements with five degrees of freedom per node and the steel girders using three dimensional beam elements with six degrees of freedom per node (Table 3.2). The bridge deck slab and steel girders share nodes where steel girder is present. This is essentially a two-dimensional finite element analysis and it is not capable of capturing the effect of the offset between the center of gravity of the steel girder and the center of gravity of the deck slab. In addition, this model cannot capture the systems actual boundary conditions since the supports in the actual system are located at the bottom of steel girder rather than at the level as the center of gravity of the deck slab.

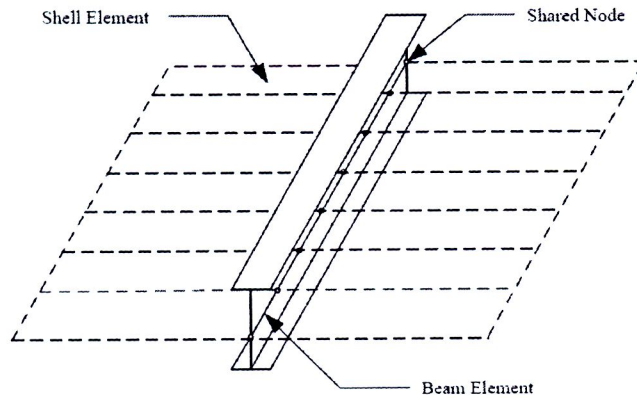


Figure 3.5 Hays et al. (1986) idealization

3.2.4 Modeling of Boundary Conditions

In this study, the bridge substructure such as piers and abutments are assumed to be perfectly rigid. Therefore, the supports are modeled by restraining the appropriate degrees of freedom at the underside of the bottom flange using zero dimensional elements. Rigid links are used to connect the girders to the zero dimensional elements. To simulate a pin support, displacements in all three directions are constrained. To model a roller support, vertical and transverse displacements as well as the rotations along longitudinal and vertical axes are constrained. To model a frozen support, displacements and rotations in all three directions are constrained. However, the actual support condition lies in between the ideal support conditions discussed. If the actual support details or field test results are available then spring elements can be used to simulate the actual level of restraining provided by the supports.

3.2.5 Modeling of Truck Patch Loads

A point load on a slab may also introduce a stress concentration problem. This point load is normally considered as a distributed pattern from top to mid plane of deck slab to avoid singularities (Figure 3.6). In this study, a finer mesh around the tire load area is necessary to ensure the accuracy of load distribution in a finite element model. The way that the loads are applied on the bridge deck (load pattern) is also carefully treated. In general practice, American Association of State Highway and Transportation Officials (AASHTO, 2004) uses tire patch loading with tire pressure uniformly distributed over the contact area for the analysis of bridges subjected to truck loading. Dimensions of the tire patch can be calculated using the following expressions given in AASHTO C3.6.1.2.5 (2004).

In this study, the truck wheel load is modeled as a patch load distributed over a finite area in FEA models. The tire contact area for an HS20-44 truck is assumed as a rectangle, with a length of 0.51 m (20 in) and a width of 0.25 m (10 in) (AASHTO,

2004). To attain a more accurate analysis, the tire print loads are enlarged by spreading outwards through the mid-plane of the slab. The contact area of a wheel load given by AASHTO (2004) is enlarged by projecting on the mid-plane of the slab with a distribution angle of 45 degrees (Bakht and Jaeger, 1987; Hambly, 1991; Pucher, 1977 and AASHTO, 2004) as illustrated in Fig. 3.6. Larger footprints of middle and rear wheels as a rectangular area (wheel width + t) by (wheel length + t) are loaded on shell deck of FEA model.

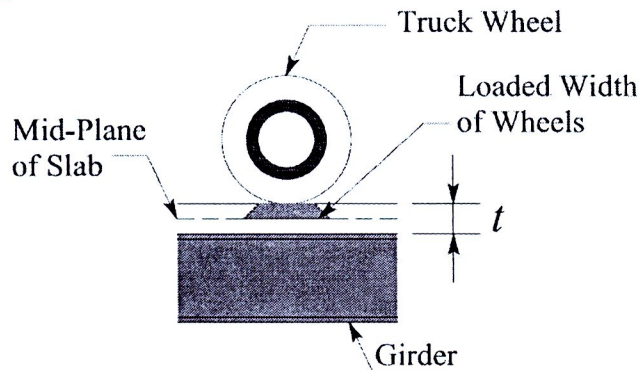


Figure 3.6 Dispersion of truck wheel loads on deck slab

3.3 Model Validation

The verification process is used to investigate the reliability of CAM and NCAM models by comparing calculated data with available field test data (Fang et al., 1990). In this study, the model validation was conducted in order to determine the most appropriate FEA model of the considered slab on girder bridges.

3.3.1 Laboratory Work Procedure by Fang et al. (1990)

This experiment is selected as a benchmark since it provides a full-scale test based on realistic boundary conditions at the girder ends. The test model consists of 0.19 m (7.50 in) slab thickness (t) and span length (L) of 14.94 m (49 ft) as shown in Fig. 3.7.

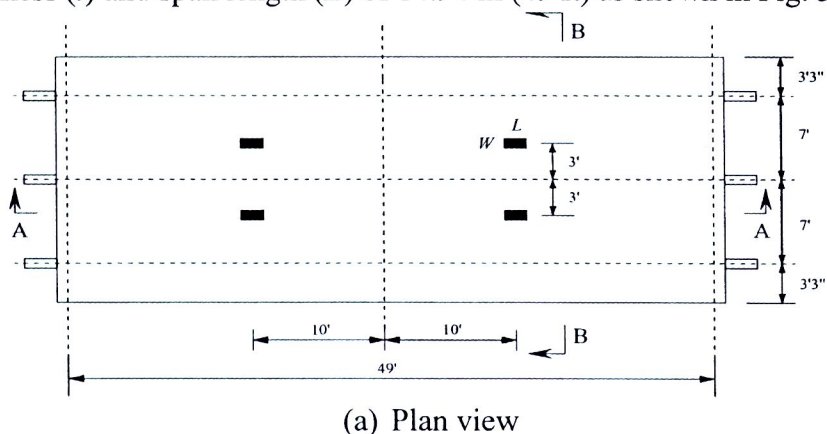


Figure 3.7 Full-scale bridge deck configuration (Fang et al., 1990): (a) Plan view; (b) Section A-A; (c) Section B-B

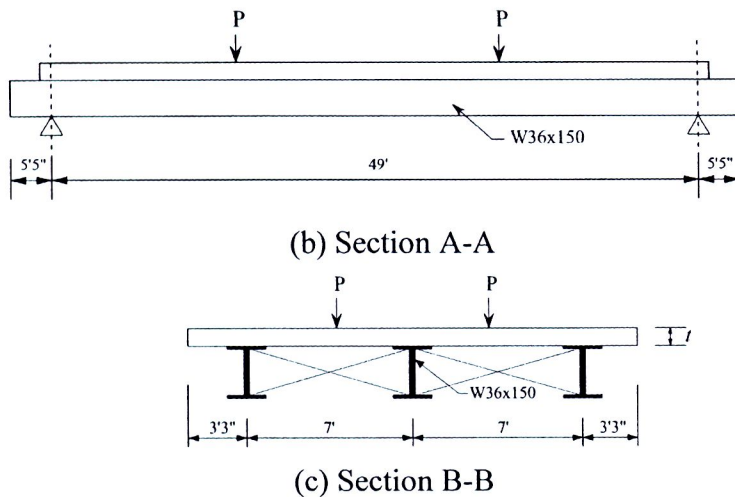


Figure 3.7 (Con't) Full-scale bridge deck configuration (Fang et al., 1990):
 (a) Plan view; (b) Section A-A; (c) Section B-B

The concrete slab was casted on 3 numbers of W36x150 girders. The deck was tested under static loads P of 89 kN (20 kips) applied on a load plate of 0.51 m (20 in) length by 0.20 m (8 in) width at a distance of 4.42 m (14.50 ft) away from center lines of the girder supports. The maximum tensile stress over the center line of top flange (σ_x) of an interior girder was estimated from the measured tensile strain at the gauge point above the interior girder (Cao 1996).

3.3.2 Comparison of Present Numerical and Test Results

The accuracy and reliability of **ESM**, **ESBM** and **SBM** as explained in previous section are evaluated by comparing the calculated deflection and stress responses with those obtained from the experimental results previously conducted by Fang et al. (1990). In general, the accuracy of any complex finite element model is limited by the mesh refinement. The best approximation is easily improved by grading the element mesh. The size of element meshes is therefore necessarily taken into consideration for all present FEA models. To this point, the deck is divided into 58 elements along a longitudinal span and 11 elements between two adjacent girders. Accordingly, the minimum of mesh discretization used for the deck slab is develop into approximately 9 elements per square meter causing the aspect ratio (length/width) of the element lower than 1.5. This results in the total number of element meshes of 1,856 used in the bridge deck model. Subsequently, the models are executed and the results are validated with the test results as summarized in Table 3.3.

Table 3.3 Comparison of the present numerical and test results

Model	Element meshes		No. of Equilibrium Equations	Mid girder Deflection, δ mm (inch)	Stress σ_x MPa (psi)	% Different from Test	
	Girder	Slab				δ	σ_x
Eccentric Shell Model (ESM)	G1	Thin shell	19,113	3 (0.123)	1.73 (250)	-8.89	2.50
		Thick shell		3 (0.123)			8.61
		4 Layer Shell	64,257	3.17 (0.125)	1.43 (207)	-7.41	-15.16
Eccentric Shell-Beams Model (ESBM)	G2	Thin shell	15,243	3.2 (0.126)	1.56 (226)	-6.67	-7.38
		Thick shell		3 (0.122)			1.69 (246)
		4 Layer Shell	49,479	3 (0.121)	1.62 (235)	-10.37	-3.69
Simple Shell-Beam Model (SBM)	G3	Thin shell	13,485	3.25 (0.128)	1.39 (202)	-5.19	-17.21
		Thick shell		3 (0.124)			1.54 (224)
NCAM (Hays et al., 1994)	G2	Thin shell	11,673	7.35 (0.290)	0.91 (132)	215	-45.90
Test (Fang et al., 1990)	-	-	-	3.43 (0.135)	1.68 (244)	-	-

Apparently, ESBM using thick-shell-element bridge deck can capture well the bridge structural behavior as it can predict the stress with acceptable accuracy compared with the test results. The minimum error less than 1% can be observed leading this type of model most appropriate on present bridge deck evaluation. In addition, such **ESBM** using thin-shell-element bridge deck trends to give compatible results with the test as it produces the error less than -8%. Likewise, good agreement with the test can be also noticed through **ESM** using both thin-shell-element and thick-shell-element bridge decks by inducing discrepancies smaller than 3% and 9%, respectively. It should be noted that although **ESBM** with a nonlinear 4-layer-shell-element bridge deck is likely to give comparative stress results (3.69% error), the more expansive in computational time than those of thin and thick shell counterparts can be recognized. Namely, a number of equilibrium equations used in the analysis is about 3 times larger than those of thin and thick shell counterparts. This nonlinear element indicates that the effect of amount of reinforcement could be included in the model by using layer shell; and due to the increase in the concrete cover, this should also provide an increase in the overall durability of the bridge deck without affecting its service behavior. However, a lack of nonlinear design for crack section has hindered using of layer shell at service behavior.

To be a more conservative for deck slab, the layer shell will not be likely to consider in this study.

More results also show that larger percentage discrepancies of -17.21 and -8.20 can be observed by means of **SBM** using thin-shell-element and thick-shell-element bridge decks, respectively. Related to the model efficiency, **ESBM** with thick-shell-element deck slab appears to yield good accuracy with reasonable computational times as it offers not much number of equilibrium equations of 15,243.

When modeled the bridge as **NCAM** (see Figure 3.5), the tensile stress over the center line of top flange, σ_x of an interior girder is equal to 0.91 MPa (132 psi). The model of **NCAM** appears to provide the lowest σ_x since the larger deflection of the interior girder at the loading section can be observed. This should correspond with the use of elastic modulus E of rigid links is assumed to be about 1 of that of the girders for **NCAM**, which agrees well with Hoit (1995) and Liu and Quek (2003) recommendation. For the reasons discussed above, **ESBM** using thick-shell elements for bridge deck will be acceptable in further bridge parametric study discussed in the next Chapter.