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3D FE modeling considering shear connectors representation and number in CBGB

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Abstract. The use of composite structures is increasingly present in civil building works. Composite Box Girder Bridges (CBGB), particularly, are study of effect of shear connector's numbers and distribution on the behavior of CBGBs is submitted. A Predicti structures consisting of two materials, both connected by metal devices known as shear connectors. The main functions of these connectors are to allow for the joint behavior of the girder-deck, to restrict longitudinal slipping and uplifting at the element's interface and to take shear forces. This paper presents 3D numerical models of CBGBs to simulate their actual structural behavior, with emphasis on the girder-deck interface. Additionally, a Prediction of several FE models is assessed against the results acquired from a field test. A number of factors are considered, and confirmed through experiments, especially full shear connections, which are obviously essential in composite box girder. A good representation for shear connectors by suitable element type is considered. Numerical predictions of vertical displacements at critical sections fit fairly well with those evaluated experimentally. The agreement between the FE models and the experimental models show that the FE model can aid engineers in design practices of box girder bridges. Preliminary results indicate that number of shear studs can be significantly reduced to facilitate adoption of a new arrangement in modeling CBGBs with full composition. However, a further feasibility study to investigate the practical and economic aspects of such a remedy is recommended, and it may represent partial composition in such modeling.

Keywords: composite box girder bridge; full connections; FE technique; shear connectors

1. Introduction

Box girders are utilized extensively in the assembly of urban highway, horizontally arced, and long-span bridges. Box girders have higher flexural capacity and torsion rigidity, and the closed form reduces the exposed external, making them less susceptible to corrosion.

Box girders also provide smooth, aesthetically pleasing structures. A composite box section usually consists of two webs, a bottom flange, two prime flanges and shear connectors welded to the top flange at the interface between concrete deck and the steel section. Although three-dimensional Finite Element (FE) modeling is probably the most involved and time

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consuming, it is still the most general and comprehensive technique for static and dynamic analyses, capturing all aspects affecting the structural response.

The other methods proved to be adequate but limited in scope and applicability. Due to recent development in computer technology, the method has become an important part of engineering analysis and design. For the time being, FE computer programs are used practically in all branches of engineering.

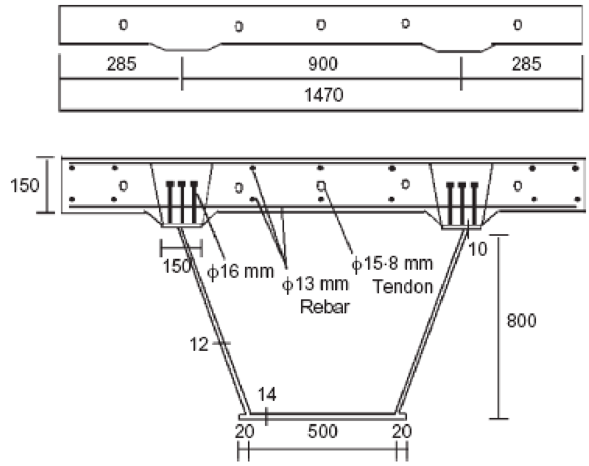
In which the concrete slab and the steel girder were modeled with four-node shell elements. addition, FE method has been used to simulate successfully the behavior of bridges. Chang and Robertson (2003) using ANSYS to study thermal loadings created a three-dimensional solid FE model. Considering longitudinal strains, modal analysis, and deformations, this model simulated a three span, 220-meter concrete bridge built to replace an existing six span concrete bridge spanning the Kealakaha Stream. In the same year (Ryu *et al*. 2003) submitted a three-dimensional finite-element model

The stress analysis of a long-span cable-stayed bridge using FE analysis compared very well with a full-scale static experimental loading performed by (Lertsima *et al*. 2004). Magdy (2004) at the same year employed three-dimensional FE analysis to investigate the static and dynamic responses of continuous curved composite box girder bridges. (Yamaguchi *et al*. 2005) conducted three-dimensional nonlinear FE analysis of a two plate girder bridge to obtain dry shrinkage and pre-stressing. The dynamic interaction between a heavy truck and highway is presented by the FE analysis by (Kwasniewski *et al*. 2006). In addition, the studies conducted by El-Lobody and Lam (2003) and Chung and Sotelino (2006) used FE modeling to predict the stress and deflection of steel-concrete composite girders.

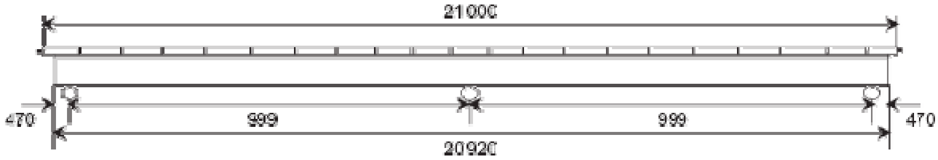
Zheng (2008) developed several 3-D FE models using ANSYS to propose new distribution factor equations of live load moment and shear for steel open-box girder bridges. The structural behavior of bridge deck slabs under static patch loads in steel-concrete composite bridges was studied by using non-linear 3D-FE analysis models with ABAQUS software by (Zheng *et al*. 2009). Multi- response objective function was introduced by (Schlune *et al*. 2009), which allow the combination of static and dynamic measurements to obtain a solid basis for parameter estimation.

(Song *et al*. 2010) performed a three-dimensional FE simulation of the composite continuous box-girder bridge with corrugated steel webs. (Sanguanmanasak *et al*. 2010) Presented a three-dimensional FE analysis model of combined steel-concrete bridges to simulate the actual bridge behavior, Thai trucks are loaded at possible locations of the bridge to obtain the maximum stresses on the bridge. Although shear connectors are the main cause of restraint in a composite bridge deck; however, very few works on the effect of configuration and properties of shear stud is available in the literature. (French *et al*. 1999) recommended the use of fewer shear connectors with smaller rows and lengths.

In present study, main attention is focused on developing representative numerical models for a CBGB. To achieve this aim several FE models of a laboratory specimen are developed using different approaches available within ANSYS software. A good representation for shear connectors is used. Rigid link elements extended the models with full interaction composition. (Ryu *et al*. 2003) published the performance of the test model. Modeling details and results of different models are presented. The acquired results from numerical models are assessed against test results and performances of models are detailed. In addition, a study of effect of shear connector's numbers and distribution on the behavior of CBGBs is submitted. This effect caused by reduction shear connectors in longitudinal and transverse directions with several different



(a) cross-section



(b) elevation (dimensions in mm)

Fig. 1 Continuous bridge model (Ryu *et al*. 2004)

percentages.

2. Experimental composite box girder bridge model

(Ryu *et al*. 2004) presented test results of a two-span continuous composite box girder bridge in 2003. Fig. 1 depicts geometrical configuration of the bridge model in conjunction with boundary conditions. The numerical evaluation in present study are undertaken to simulate behavior of presented model.

The height of the steel section was 800 mm, and the thickness of the precast concrete slab was 150 mm. The slab width was 1470 mm, as shown in Fig. 1(a). Twenty-one precast panels -each 980 mm in length- were installed on the top flange of the steel girder. Each precast panel has six block-outs for stud shear connectors that are installed on the top flanges of the steel girder to achieve full shear connection. The thickness of the upper flange, web and lower flange were 10 mm, 12 mm and 14 mm respectively.

3. Material properties

Yield stress and tensile strength of steel material used to build box section are 240 MPa and

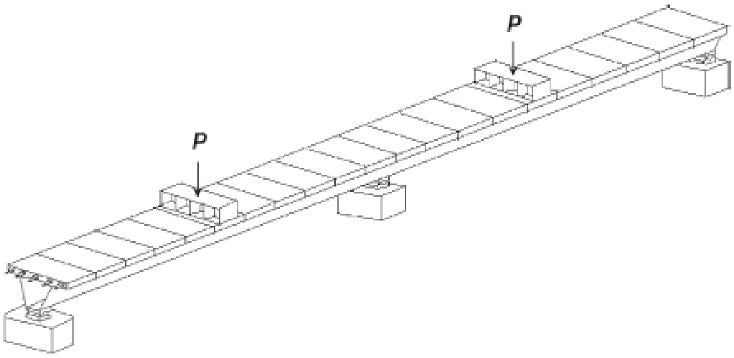


Fig. 2 Bridge loading (Ryu *et al*. 2004)

520 MPa, respectively. Elastic modulus of steel is 190 Gpa. 28-day compression strength of concrete used to build deck portion is 35.5 MPa, the average value of all the precast concrete panels for 28 days compressive strength is 35.3 MPa. Elastic modulus of concrete is 30 Gpa. (Ryu *et al*. 2004).

4. Experimental model loading

Two concentrated loads were applied at the mid-spans of the composite bridge by an MTS closed-loop electrohydraulic testing system of 2000 KN capacity, as shown in Fig. 2. Static tests for observation of the elastic behavior of the model were performed with 250 KN value for each span. Displacements of the continuous beam were measured at each mid-span with linear variable differential transformers (LVDTs) (Ryu *et al*. 2004).

5. FE Models

CBGB models were simulated using a commercial FE program ANSYS. Since the materials were stressed in elastic limits in the study of H.K. Ryu *et al.* (2004), linear analyses of bridge models are undertaken in a present study. The slab and the box girder were connected by rigid links to simulate full interaction between concrete slab and steel girder. Fig. 3 presents FE model 1 that is developed using shell elements both in concrete deck and in steel box girder portions Point load is applied in this model as shown in Fig. 3. Model 2 differs from model 1 in having different loading at mid-spans in order to represent line loading. The vertical translation Degrees Of Freedom (DOF) of the nodes across the width of the deck is coupled as shown in Fig. 4.

Coupling is a way to force a set of nodes to have the same DOF value. Similar to a constraint, except that the DOF value is usually calculated by the solver rather than user-specified. A coupled set is a group of nodes coupled in one direction.

Thickness of concrete deck portion is considerable compared to steel and other geometrical details. Another convenient way to represent this thickness is to adopt 3D brick elements. Since the cross section is prismatic, employing 3D brick elements would not cause complicated modeling approach. In order to evaluate performance of this modeling technique against shell

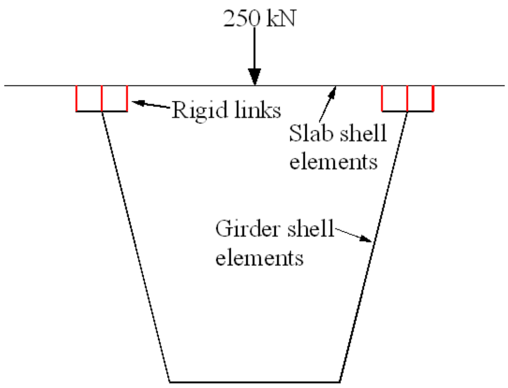


Fig. 3 FE Model 1

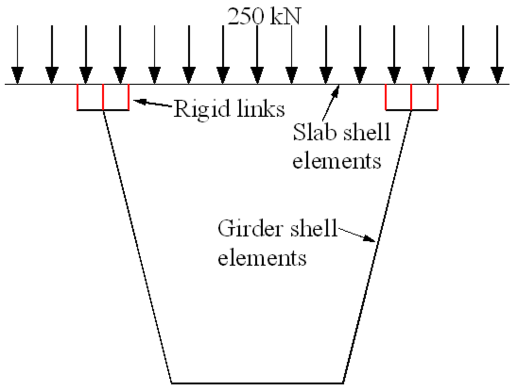


Fig. 4 FE Model 2

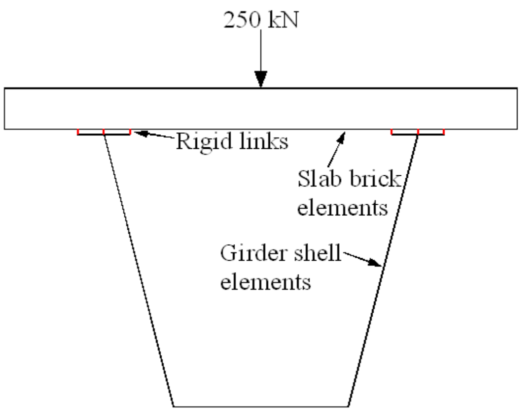


Fig. 5 FE Model 3

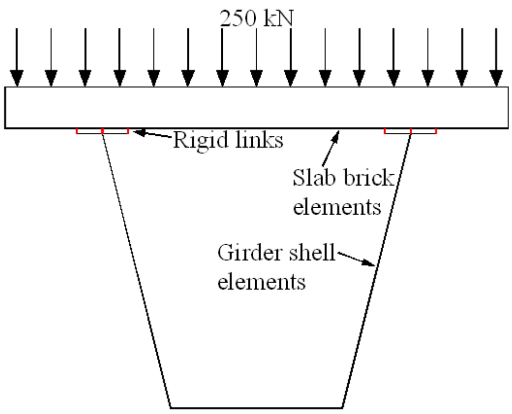


Fig. 6 FE Model 4

models and test results Models 3 and 4 are developed using eight nodes brick elements. Brick elements are just used to model concrete deck portion of the bridge where shell elements still represent the steel box portion as with Models 1 and 2.

The loading condition, which makes Models 1 and 2 different, also creates the difference between Models 3 and 4. Figs. 5 and 6 present the details of Models 3 and 4.

In this study, the uppermost concrete flanges were divided into thirty-four quadrilateral elements across width for an appropriate aspect ratio of the elements and four divisions for each top steel flange. The bottom flanges were divided into ten elements, and webs were divided into twenty quadrilateral elements. The longitudinal two spans were divided into 160 elements.

Two models are built using Shell 181 Elements-which are four-node elements with six DOF at each node for steel webs, concrete top flange, and the steel bottom flange. The plate thickness and the material properties are required input for Shell181. In addition, another two models are build-using Shell181, for steel bottom flanges and webs. While solid185 elements are used for 3-D modeling of concrete top flanges. Eight nodes having three DOF at each node define them. The MPC184 inflexible elements are used to model a stiff constraint between two bodies, steel and concrete in this case, which are used to transmit forces and moments in all above models.

Boundary conditions are handled in such a way to represent simply supported conditions of test specimens. It was assumed that the shear connectors were uniformly distributed along the length of a composite member.

6. FE Models after reduction of shear connectors

It is now consented that the two portions of any composite structure are joined together by an infinitely rigid shear connection. The two members then behave as one. Slip and slip strain are everywhere zero, and it can be assumed that plane sections remain plane.

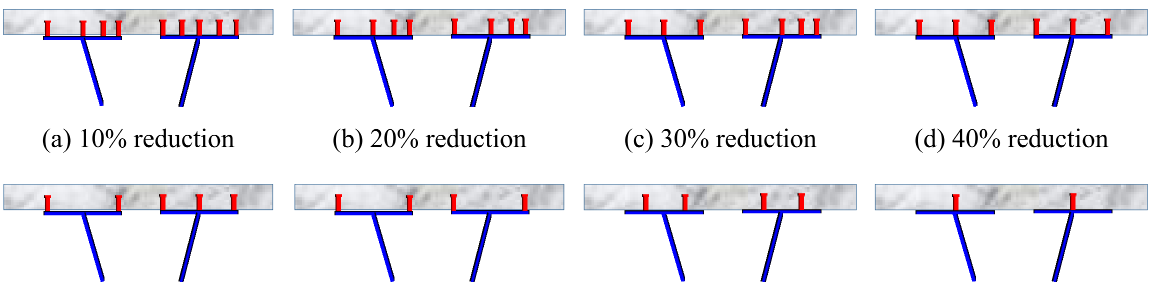
This situation is known as full composite interaction. All design of composite structures in practice is established on the assumption that full interaction is achieved.

In General according to BS 5400-5, the spacing of the connectors should be not greater than 600 mm or three times the thickness of the slab or four times the height of the connector, including any hoop, which is an integral part of the connector, whichever is the least. Except that, connectors may be placed in groups, with the group spacing greater than that specified for individual connectors, provided consideration is given in design to the non-uniform flow of longitudinal shear and of the greater possibility of slip and vertical separation between the slab and the steel member.

In order to study the effect of stud spacing and distribution, the previously presented models of CBGBs (Figs. 2 and 4) with different shear stud distributions is considered, as shown in Fig. 7. Several extreme cases, 10% to 80% percentage of reduction, considered as well. First reduction applies to the shear connectors in transverse direction to Models 2 and 4 with 10%, 20%, 30%, 40%, 50%, 60%, and 80% percent, as shown below.

For every single case above two models created one is duplicate as Model 2 and one more is alike as Model 4, sixteen models are resulted with clear deference caused by reduction for each case.

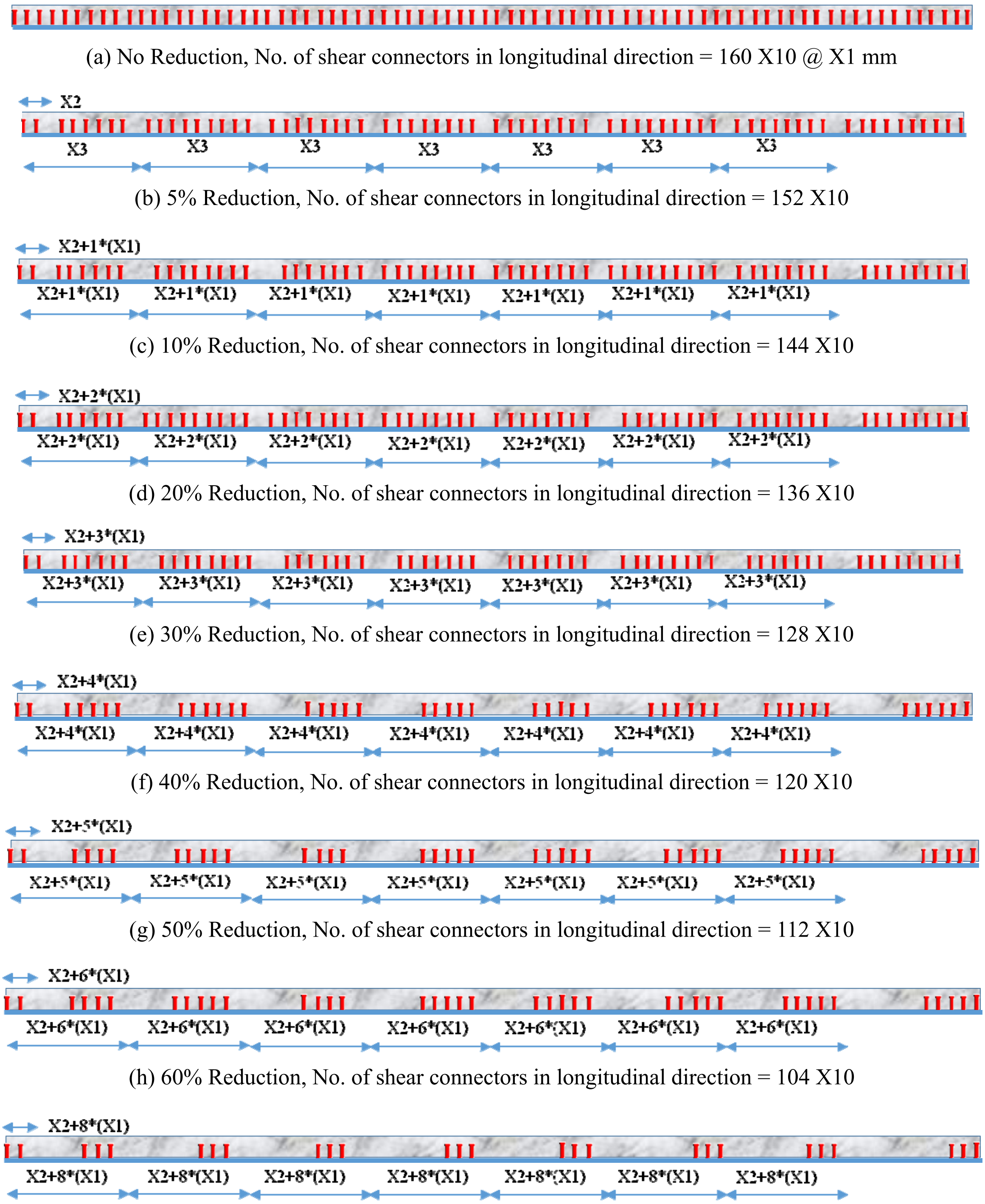
Second reduction applies to same models two and four because these two models acquired the best results comparing with the other two models. Additionally, sixteen models modeled but with longitudinal reduction with 5%, 10%, 20%, 30%, 40%, 50%, 60% and 80% percentage of connectors reducing.



(a) 10% reduction (b) 20% reduction (c) 30% reduction (d) 40% reduction

(e) 50% reduction (f) 60% reduction-1 (g) 60% reduction-2 (h) 80% reduction

Fig. 7 Transverse reduction of shear connectors



(a) No Reduction, No. of shear connectors in longitudinal direction = 160 X10 @ X1 mm

(b) 5% Reduction, No. of shear connectors in longitudinal direction = 152 X10

(c) 10% Reduction, No. of shear connectors in longitudinal direction = 144 X10

(d) 20% Reduction, No. of shear connectors in longitudinal direction = 136 X10

(e) 30% Reduction, No. of shear connectors in longitudinal direction = 128 X10

(f) 40% Reduction, No. of shear connectors in longitudinal direction = 120 X10

(g) 50% Reduction, No. of shear connectors in longitudinal direction = 112 X10

(h) 60% Reduction, No. of shear connectors in longitudinal direction = 104 X10

(i) 80% Reduction, No. of shear connectors in longitudinal direction = 96 X10

Fig. 8 longitudinal reduction of shear connectors at top flange

[X1 = 131.25 mm, X2 = 525 mm, X3 = 2625 mm]

These reductions are completed according to a plan of removing shear connectors at specified distances on the upper flange of the bridge. The procedure of this plan arranged for 5% by removing first line of 10 connectors at initial distance 525 mm from the beginning of the highest flange and seven lines of 70 connectors (10 for each line) every 2625 mm from the beginning of the top flange as shown in Fig. 8(b). Other reductions are increased cumulative with spacing 131.25 mm adding to the spacing of 5%; therefore, the numbers of connectors are reduced eight lines (80 connectors) for each percentage of reduction as shown in the Fig. 8(a-i) below.

7. Numerical results and discussions

The results by the ANSYS finite element analysis (FEA) using the Model 1-Model 4 are shown in Table 1 and Fig. 9 together with the loading-test results and the design values. It is observed that the design analysis tends to overestimate the stress, and the vertical displacement measured in the loading test. The differences at the mid-span of Model 4 are as much as 0.2 mm or 8% for the vertical displacement.

Obtained results from FE analysis can be utilized to understand behavior of CBGB. In addition, it can also be used to compare the stress profiles. During the static test done by (Ryu *et al*. 2004) in the elastic range of loading, the flexural stiffness of the composite bridge showed linear elastic behavior. Mid-span deflections from the analysis were compared with the test results. In the experimental test, the mid-span deflection was 2.52 mm and in the analysis performed by same researchers, it was 2.76 mm at a load of 250 KN. Deflections results obtained from ANSYS FE model's Model 1 to Model 4 can be observed in Fig. 9 below, and they are summarized in Table 1.

It is interesting to note that in case of Model 4 (which has steel box girder modeled with shell 181 elements and concrete deck with solid 185 elements, and distributed load, the best result comparing with experimental test was obtained, so the focusing will be on Model 4 to make a comparison with experimental data submitted by (Ryu *et al*. 2004). There is very good agreement between the two set of results whereas in the case of other models, some deviations exist.

Mid-span deflection from the FE analysis was compared with the test results as shown in Fig. 8. In the test results, the deflection was 2 • 52 mm and in the analysis, it was 2 • 56 mm at a load of 250 KN. As noted from results above, Model 4 midspan deflection is closer to test results than the FE model submitted by (Ryu *et al*. 2004). These realities and agreement of the results gives us a good indicator about the new procedure in modeling such as structures. In addition, the representation of shear connectors by meaning of MPC elements succeeds to build a model better

Table 1 Deflection comparisons between present models and from (Ryu *et al*. 2004)

Model	Midspan deflection (mm) present study	Results of (Ryu *et al*. 2004)	
		Midspan deflection (mm) for test results	Midspan deflection (mm) for FE model
Model 1	2.960	2.52	2.76
Model 2	2.567		
Model 3	3.041		
Model 4	2.558		

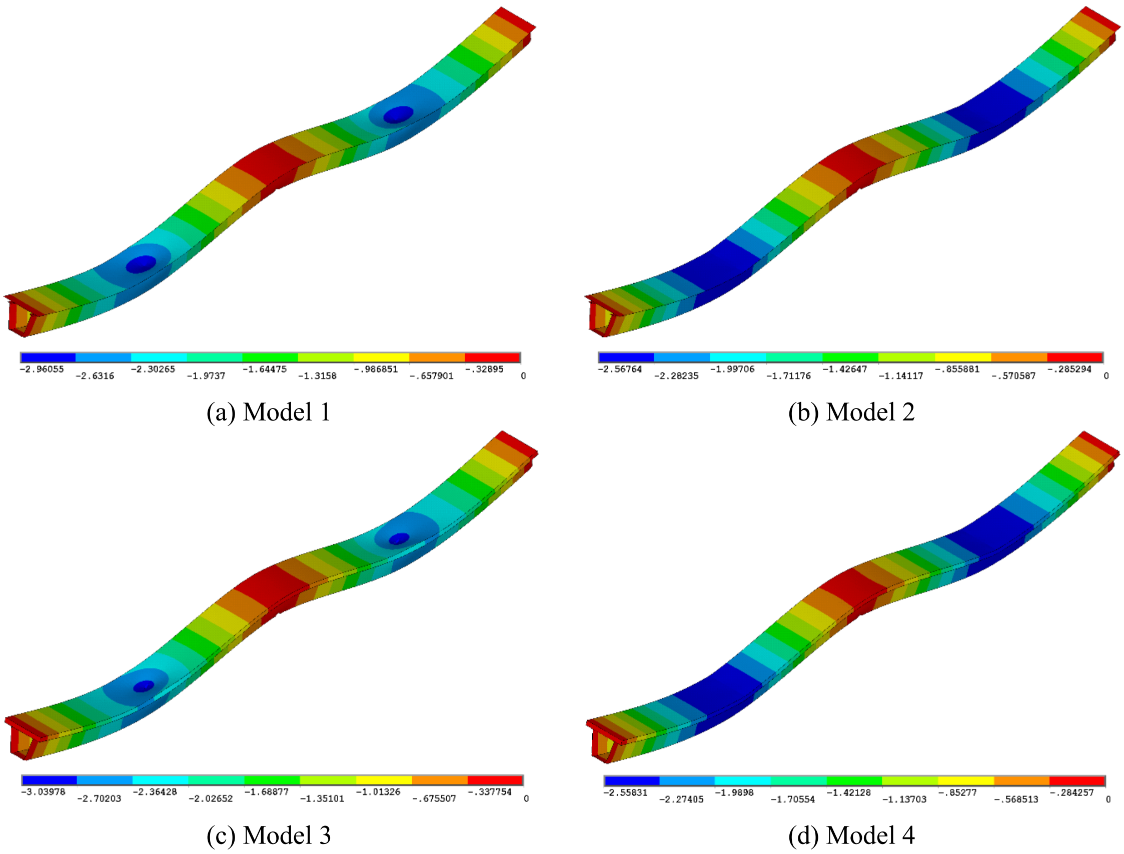


Fig. 9 Deflections of numerical models (Magnification factor = 400)

than that modeled by (Ryu *et al*. 2004).

7.1 Results for shear connectors reduction

In construction practice, the stud spacing or "pitch" is usually around 250 mm to 300 mm, with three to five studs in each row. In the mid-span portion, the spacing may be increased to 600 mm and all these values within BS standards. In order to study the effect of stud spacing and distribution, the previously presented model with different shear stud distributions is considered, as shown in Figs. 7 and 8.

Since Models 4 and 2 exhibited the best performance, in this section we consider them to study reduction of shear connectors. The transverse and longitudinal reduction for shear connectors are performed. The results of midspan deflection are listed in Table 2 and Figs. 10 and 11. It is possible to infer from Table 2 that the decrease of the number of connectors implies an increase of the vertical displacement, noting that all characteristics of shear connectors are fixed and reducing the number of shear studs is a major factor in determining the economic and practicality of the proposed modification.

A simple study was performed to determine the needed shear stud to develop hybrid action in

ANSYS. It was determined that full composite action can be achieved with fewer shear stud connectors as compared to existing state of the practice. The results of this study are presented here. From the results, in transverse reduction the deference between the cases (f) and (g) in Fig. 7 is diagnosed clearly with percentage difference 2.56% in Model 2 and 3.3% in Model 4, as noted these two values represent the same percentage of reduction but with difference spacing between the connectors.

Furthermore, very interesting difference accrued between the cases in Figs. 7(f) and (h), when the case (h) deformed less than (f) even with 28% more reduction and resulting percentage difference 2.5% in Model 2 and 3.3% in Model 4. The other cases of reduction from (a) to (d) show increase in deflection while shear connectors decrease with percentage of difference 0.045% in Models 2 and 4, this percentage increase from case (e) to (f) in Fig. 7 to become 1.54% in Model 2 and 2.7% in Model 4. Fig. 10 shows these differences graphically.

The longitudinal reduction in shear connectors numbers applied with uniform steps, according to the FE modeling the bridge divided along its span to 160 divisions, there is a line of (10) MPC184 elements at each division, which represent the shear connectors of the composite bridge.

The shear connectors are reduced according to simple equations to get a uniform procedure of longitudinal reduction of shear connectors, these equations explained with the cases of reduction in Fig. 8, and the equation for the distance before reduction is

$$S_1 = x_1 = L/n \tag{1}$$

where

x_1 = Spacing between shear connectors without reduction

n = Number of divisions in FE model

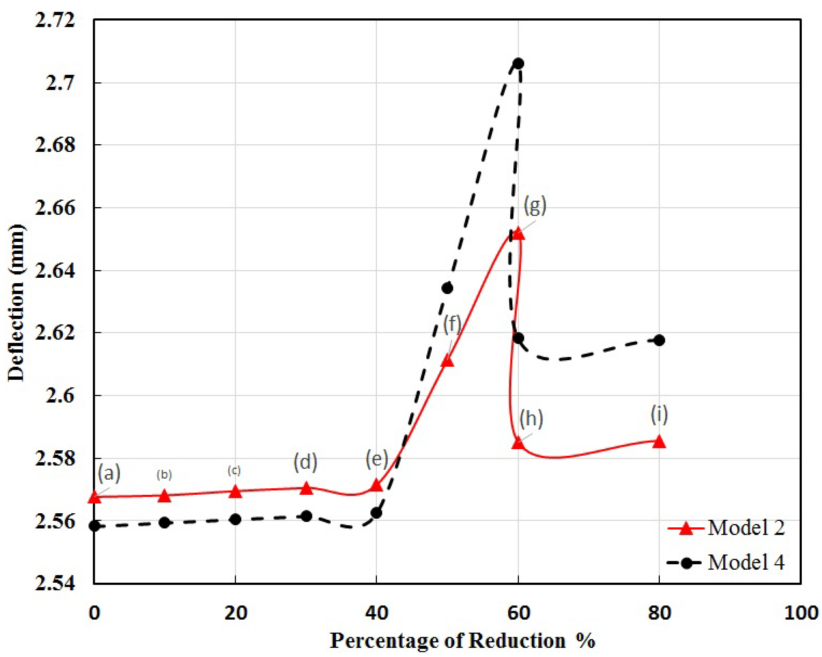


Fig. 10 Transverse reduction percent vs. vertical displacement

L = Length of model

Moreover, the second equation is used to keep the initial distance from the edge of top flange without reduction of shear connectors, the equation is

$$S_2 = x_2 + \left[\left(\frac{p}{10}\right) \times x_1\right] \tag{2}$$

where:

p = Percentage of reduction %

x_2 = $L \times (0.025)$

The last equation represent the cumulative uniform spacing that is used to reduce the number of shear connectors longitudinally, and the equation is

$$S_3 = x_3 + \left[\left(\frac{p}{10}\right) \times x_1\right] \tag{3}$$

where:

p = Percentage of reduction %

When there are longitudinal reductions from 5% to 60% percent, there is about 0.4 to 0.7% increasing in percentage difference occurred in both Models 2 and 4 in deflection results, this increasing occurred when the reduction changed from 5% to 10% and from 10% to 20% and so on. Further, there is a 5.0% percentage difference when 80 % reduction is present.

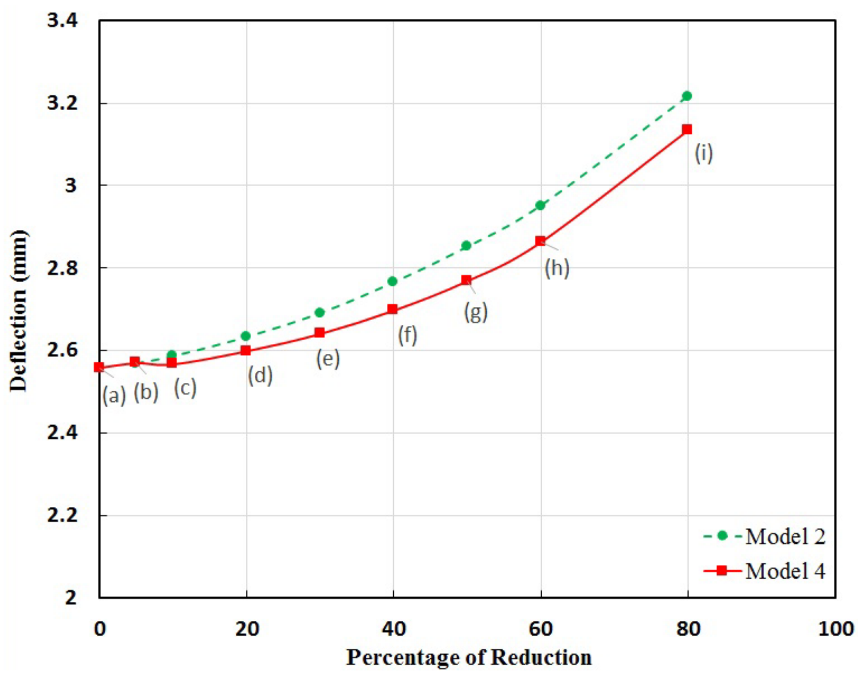
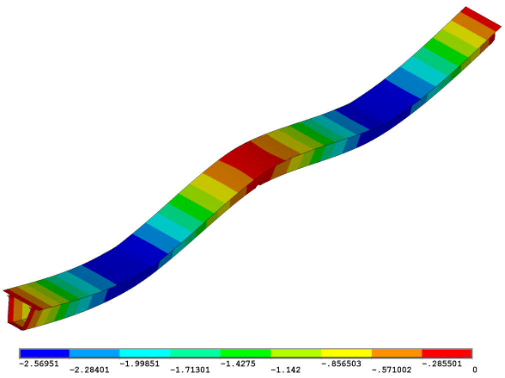


Fig. 11 Longitudinal reduction percent vs. vertical displacement

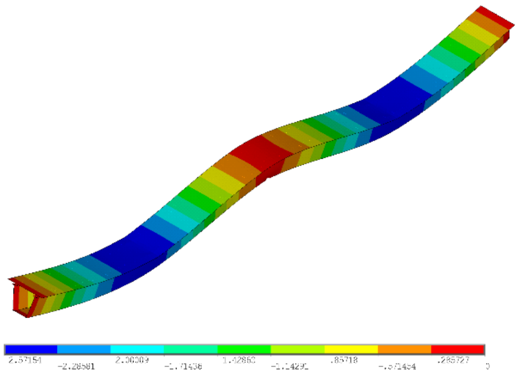
Table 2 Deflection results for ANSYS Models 2 and 4 after reduction in connectors

Percentage of reduction %	Midspan deflection (mm) after transverse reduction		Midspan deflection (mm) after longitudinal reduction	
	Model 2	Model 4	Model 2	Model 4
0	2.568	2.559	2.568	2.558
5	-	-	2.568	2.569
10	2.568	2.559	2.586	2.567
20	2.569	2.561	2.633	2.598
30	2.571	2.562	2.690	2.640
40	2.572	2.563	2.765	2.697
50	2.611	2.634	2.851	2.769
60	2.652	2.706	2.951	2.863
60*	2.585	2.618	-	-
80	2.586	2.618	3.215	3.134

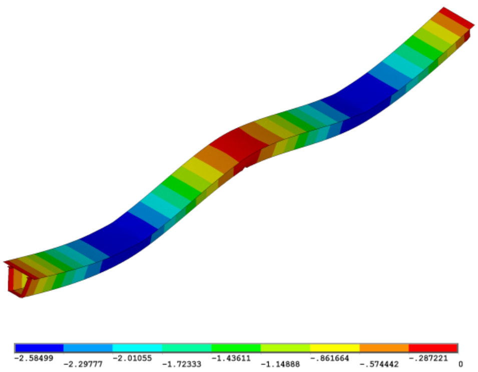
- Not tested
*Centered distribution of connectors



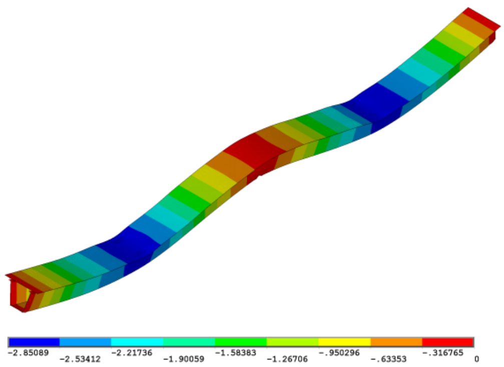
(a) 20% transverse reduction



(b) 40% transverse reduction

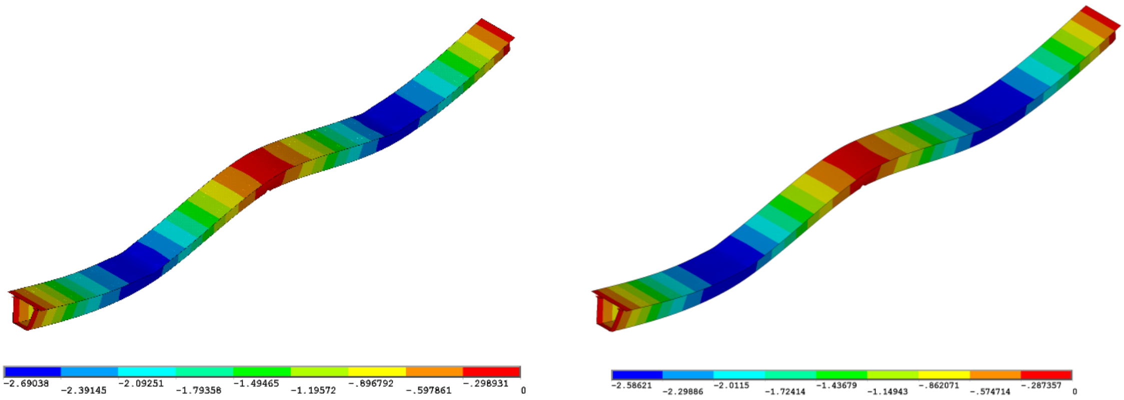


(c) 60% transverse reduction



(d) 10% longitudinal reduction

Fig. 12 Deflections of numerical Model 2 (Magnification factor = 400)

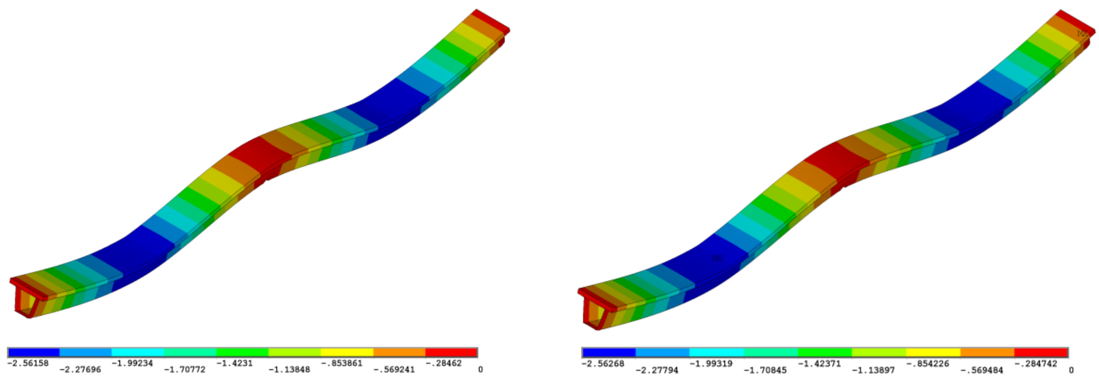


(e) 30% longitudinal reduction (f) 50% longitudinal reduction

Fig. 12 Continued

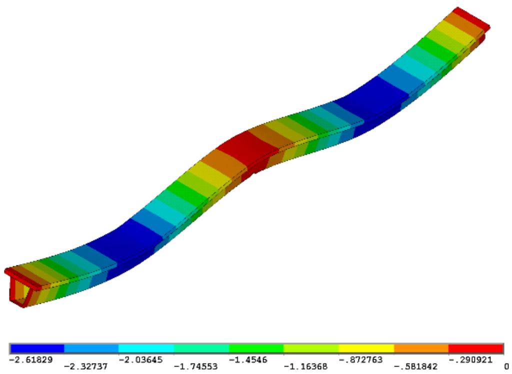
The set of equations formulated above are suitable for any kind of CBGB when reduction of shear connectors applied, the behavior of composite components is still as they before reduction that mean full interaction between portions is kept. The complication of modeling CBGBs with partial interaction between steel and concrete leads us to use such equations to model partially interacted shear connectors, but two important notes must considered: first one - the slip calculation is ignored; and the second - knowing which percentage of reduction is more accurate to represent the incomplete interaction situation. These equations in some stages of reduction will be useful to model CBGBs with partial interaction between components and reasonable results without complicated procedures.

Consequently, these equations when they used to modeled CBGB with incomplete interaction between steel and concrete only general behavior depends on deflection calculations is obtained, also a new study is required to performance the exact percentage of reduction can be used to

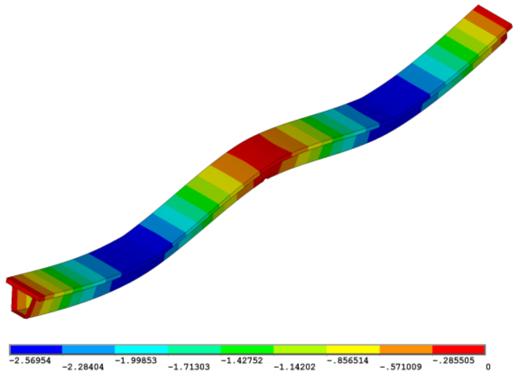


(a) 30% transverse reduction (b) 40% transverse reduction

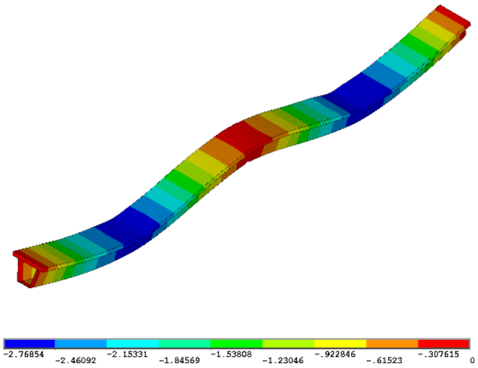
Fig. 13 Deflections of numerical Model 4 (Magnification factor = 400)



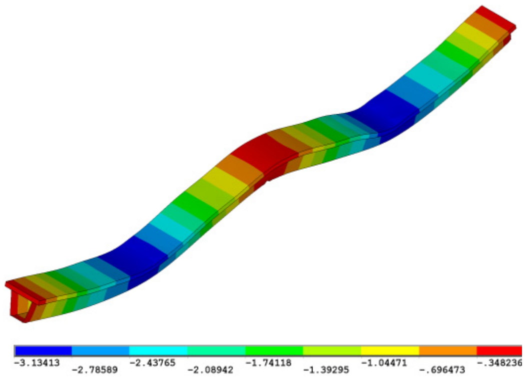
(c) 60% -2 transverse reduction



(d) 5% longitudinal reduction



(e) 50% longitudinal reduction



(f) 80% longitudinal reduction

Fig. 13 Continued

represent the partial composition. Calculations of midspan deflection in Models 2 and 4 are graphically presented in Fig. 11.

From the results listed above the effect of longitudinal reduction appears more efficacious than the transverse reduction, also a set of equations are obtained from longitudinal reduction of shear connectors. These equations are very easy to use and to model CBGB with the non-complete number of shear stud and full interaction between steel and concrete. Table 2 below showed the whole results for models with reduction of shear connectors.

Deflection results for Model 2 after reduction for some percentage of reduction are given below in Fig. 12.

Deflection results for Model 4 after reduction for some percentage of reduction are given below in Fig.13.

8. Conclusions

The theoretical three-dimensional FE models developed herein can predict quite well the elastic behavior as well as the mode shapes of continuous composite single box girder bridges when assessed against experimental behavior.

The interaction between the two parts of the bridge in the analysis modeled using rigid links to give full interaction between components. The thickness of precast concrete 15 cm is big of simulate using shell elements, so noteworthy difference can be observed (about 2 %) by using 3-D solid elements to model such thickness.

A new technique to model shear connectors has been adopted; this technique relies on the use of a rigid links MPC element which is used first time in this paper to represent shear connectors. This kind of element is perfect to model shear connectors also its able to simulate complicated CBGB and can be used as a practical modeling technique for long span CBGB.

The value of the DOF is coincident for all the points to be coupled, was important thing effect on a result of simulation of constrained point load. Big difference appeared (15%) when the loading simulated by Coupling to force a set of nodes to have the same DOF value.

The FE analysis can simulate the structural behavior of a steel-concrete CBGB very well; the results would be in good agreement with those of experimental test. For further study, more complicated three-dimensional FE modeling should be investigated, for example, modeling of bearing pad included, bracing and diaphragm and more details of pier foundation. Furthermore, the application of a proposed model to various types of CBGBs should be explored, such as curved bridges, high-strength concrete, prestressed concrete bridges.

Preliminary results indicate that number of shear studs can be significantly reduced to facilitate adoption of a new arrangement in modeling CBGBs with full composition. However, a further feasibility study to investigate the practical and economic aspects of such a remedy is recommended, and it may represent partial composition in such modeling.

Although an attempt has not made in this study, the stress analysis for these models and that need for stress analysis in experimental studies also. This would be a significant task in future studies along with the development of structural details for practical application.

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