Journal of Engineering and Applied Sciences 12 (1): 133-139, 2017
ISSN: 1816-949X

Static Response of an Orthotropic Steel Deck-Open Ribs Type By Considering the Influence of Floor Beams Flexibility

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Abstract: The orthotropic deck is a bridge deck system which has been used since the mid 19th century and utilized by many modern bridges. It employs a stiffened steel deck instead of the conventional reinforced concrete slab as the structural base for the distribution of traffic loads for the bridge system. The Pelikan-Esslinger method (P-E method) is developed to calculate the response of such bridge deck configuration. This method considers the bridge to be divided into three systems whose individual actions are superimposed to yield the final bridge responses. This study is conducted to compare the stresses on open ribs of an orthotropic deck by considering floor beam flexibility using the P-E method and the Finite Element Method (FEM) using ABAQUS. Generally, the result shows that the longitudinal stresses on the ribs by considering the influence of the floor beam flexibility are higher than the analitycal method. For design purpose, the longitudinal stress at the top of the ribs of the support section obtained from the P-E method could be reduced. The FEM with Spring elements to represent the floor beam flexibility more accurately models the bridge deck system than the FEM neglecting the flexibility of floor beam.

Key words: Orthotropic deck, open type ribs, P-E method, FEM, floor beam flexibility

INTRODUCTION

The term orthotropic is a conjunction of two words, orthogonal-anisotropic which means material properties having differences at right angles or in the Orthotropic Steel Deck (OSD)'s case the steel plate has different stiffness in the transverse and the longitudinal direction. This stiffness is achieved by stiffening the plate element with open or closed ribs in the longitudinal direction and the floor beams in the transverse direction with the plate acting as the flange of both elementsm as shown in Fig. 1.

Using a rather unconventional configuration, the Pelikan-Esslinger method (P-E method) was developed to determine the response on an orthotropic deck system. The method considers the bridge to be divided into three systems whose individual actions are then added to yield the final bridge response. System 1 covers the local response of the deck plate spanning the distance between the supporting ribs whereas System 2 covers the response of the deck plate, ribs and transverse floor beams which form the bridge deck. This system is considered as a continuous orthotropic plate on flexible supports. Finally, System 3 covers the response of the main longitudinal girder, acting with the deck plate and longitudinal ribs. This system is assumed to be a large beam spanning the distance between the main supports. The major contribution of the P-E method is in the behaviour of System 2. This system is designed in two steps: design of continuous orthotropic deck plate on rigid supports and correction to this system by considering the floor beams elastic stiffness (Heins and Firmage, 1978).

The Finite Element Method (FEM) has been developing rapidly in the last few decades through computerized developments. Engineers have employed finite element based softwares in carrying out complex design analysis due to it being more practical and time-efficient compared to the analytical approach. For example, to determine the stress distribution of an orthotropic deck system, 3-D finite element models were developed. Special attention was given to study the stress concentration at the welded diaphragm plate to closed rib connections. Using ABAQUS Software the study concluded that the substructre model provides an effective tool to estimate stress concentration at the termination of the diapharagm plate cut-out at the diaphragm plate to closed rib connection (Feng, 1996). Rasmus *et al*. (2005) investigated the stiffness enhancement of the traditional orthotropic bridge deck by using a cement-based overlay. This investigation was conducted on real size Faro Bridges, Denmark. The finite element analysis using DIANA software was then used to model the bridge deck. This study shows that debonding

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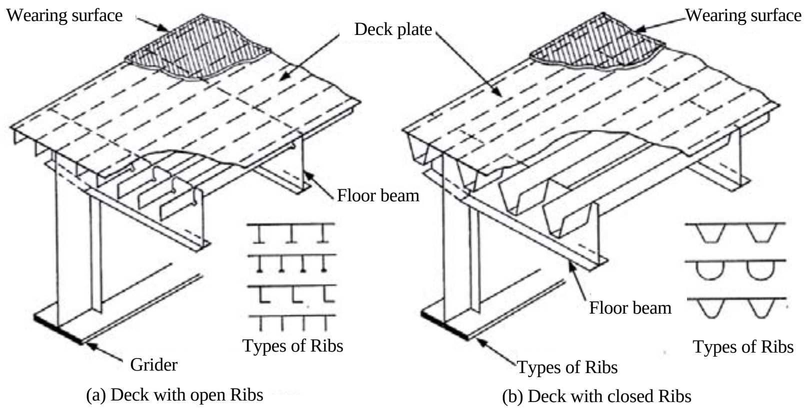

Fig. 1: Typical orthotropic steel deck bridge configurration AISC

s initiated for a certain crack width in the overlay. The load level where cracking and debonding is initiated depends on the stress-crack opening relationship of the material. Other investigation conducted to observe the behavior of the cracks in several welded connections between the vertical web stiffeners and the upper flange of the main girders of Varby bridge. The orthotropic Vårby bridge was modelled with shell elements, including its concrete deck using ABAQUS Software. By comparing the results from the models with the measurements there were very good agreement when looking at the behavior of the cracks (Bengtsson and Widen, 2010).

A number of investigations related to the use of the P-E method and the FEM were conducted by some researchers. Wolchuk (2014,1963) suggested the use of the pragmatic design for orthotropic decks due to ignoring the effects of stresses induced in the process of fabrication with respect to Aashto (1998). Numerical techniques using ABAQUS software for approximate solution of sandwich cylindrical panels was developed by Garooschi and Barati (2016). It was concluded that the circumferential dimension has significant effect on the axial mode of vibration than the circumferential mode of vibration. Finally, Aldavat *et al.* (2016) conducted investigation of the seismic response of steel arch bridge using SAP2000. The study concluded that under the effect of vertical acceleration of earthquake, the maximum displacement increase up 7%. All of these research show that the use of FEM to model bridge system and its element has been widely accepted.

A previous study was conducted to get the response of an orthotropic deck based on the P-E method by modeling the System 2 using finite-element based software ABAQUS. The model followed the first step of System 2 which considers the plate to be continuous on rigid supports. The second step, however was not considered due to difficulties in taking into account the flexibility of the floor beams to achieve the moment modification (Victoriawan, 2015). The results of the previous study will be assessed together with the results of the study conducted in this study.

The main focus of this study is to compare the response of the orthotropic deck system using the analytical procedure and the FEM by considering the influence of floor beams flexibility and to evaluate whether the previous study is based on a more conservative design compared to this analysis. The finite element modeling is carried out using ABAQUS. Shell element with three or four nodals for each element is used to model the the whole bridge deck system (Khennane, 2013). The parameter which will be observed and compared are the longitudinal stresses induced on the orthotropic deck system based on System 2 of the P-E method.

MATERIALS AND METHODS

According to Timoshenko and Woinowsky-Krieger (1959), Heins and Looney (1968) and Heins and Firmage (1978), the general equation represents the load-displacement response of a continous orthotropic plate is:

$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_Y \frac{\partial^4 w}{\partial y^4} = q \quad (1)$$

Where:

- Dx = Bending stiffness per unit width in x (longitudinal) direction
- Dy = Bending stiffness per unit width in y (transverse) direction
- H = Torsional stffness per unit width
- w = Displacement in z (perpendicular to plate surface) direction
- q = Distributed load in z (perpendicular to plate surface) direction

For deck plates with open type ribs the plate have minimal stiffness in the transverse and torsional directions relative to the longitudinal stiffness (Dy = H = 0). Therefore, the general plate equation becomes:

$$D_x \frac{\partial^4 w}{\partial x^4} = q \quad (2)$$

The first step of System 2 as mentioned before considers the deck continuous on rigid supports or in this case the steel plate acting as the flange of the ribs are supported rigidly by the transverse floor beams. The behaviour of such configuration is predicted by the use of influence lines. By solving Eq. 2 due to the point wheel load and using the influence lines from Fig. 2 and 3, the moments at midspan, Mc and support, Ms are given by the equation as follow:

$$\left[\frac{M_c}{Pl}\right] = \left[\begin{array}{l} -0.1830\frac{y}{l}+0.3- \\ 70\left(\frac{y}{l}\right)^2 - 0.1340\left(\frac{y}{l}\right)^3 \end{array}\right]\left(-0.268\right)^m \quad (3)$$

$$\left[\frac{M_s}{Pl}\right] = \left[\begin{array}{l} -0.5\frac{y}{l}+0.866\left(\frac{y}{l}\right)^2 - \\ 0.366\left(\frac{y}{l}\right)^3 \end{array}\right]\left(-0.268\right)^m$$

Where:
P = Any concentrated wheel load
l = Spacing between floor beams
m = The smaller of the two support numbers enclosing the span under consideration
y = Location of the level with respect to left support of loaded span

The second step of System 2 is applying correction to the system by considering the flexibility of the floor beam. To account for the floor beam flexibility, the interactions of the deck and the floor beam is considered. Such interaction is first related by the stiffness according to:

$$\gamma = \frac{I_r b^4}{I_f l^3 a \pi^4} \quad (4)$$

Where:
I_r = Inertia of rib
I_f = Inertia of floor beam
b = Spacing of open ribs
l = Spacing of floor beams
a = Flange width of rib

Whereas the bending moment modification at the center of the rib is given by:

$$\Delta M_c = Q_0 la \frac{Q_{ix}}{Q_0} \sum \frac{F_m}{P} \frac{\eta_m}{l} \quad (5)$$

Where:
Q_o = P $(2g)^{-1}$
P = Wheel load intensity
g = Contact width of tires
F_m = Reaction due to load P at support m of continuous beam on rigid supports
η_m = Influence line ordinates at flexible support m for the bending moment at midspan

The modification of the support moment Ms due to the flexibility of the floor beams generally reduces these moments and therefore, the influence of floor beam flexibility is generally neglected for design purpose.

Bridge description: The orthotropic bridge considered in this study is a six-lane, four-span simply supported bridge with span length and width of 105 and 22 m, respectively. The floor beams are spaced at 3.6 m and the type of ribs used for this bridge deck system is the open rib type with a spacing of 0.45 m as seen in Fig. 4 and 5, adopted from the previous study (Victoriawan, 2015). The live load to be

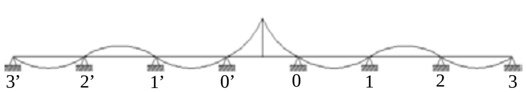

Fig. 2: Influence lines for midspan moent (m_c) of deck

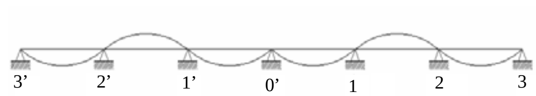

Fig. 3: Influence lines for support moent (m_c) of deck

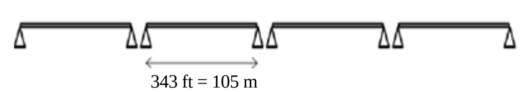

Fig. 4: Bridge layout (Victoriawan, 2015)

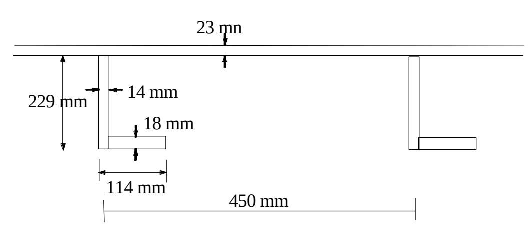

Fig. 5: R-rib section (Victoriawan, 2015)

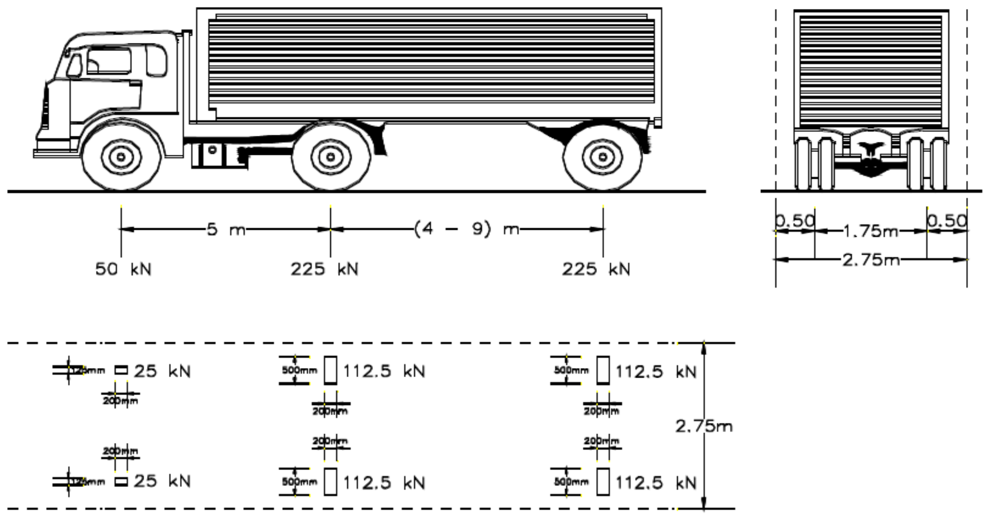

Fig. 6: Truck loads based on SNI T-02-2005(SNI, 2005)

used for the deck design are truck wheel loads which are more representative of the actual loading (Heins and Firmage, 1978). The truck wheel loads are in accordance with the SNI T-02-2005 Specification and is shown in Fig. 6 (SNI, 2005).

Section properties and material: The cross-section of the open type T-ribs shown in Fig. 5 can be found using Eqs. 3-5 as well as based on truck loadings configuration shown in (Fig. 6). The material used for the deck, longitudinal ribs, and floor beams is a steel grade equivalent to ASTM A572 grade 50 which has the yield strength of 360 MPa. These section properties and material were used in the previous study (Victoriawan, 2015).

Finite element model: As mentioned before the objective of the study is to model the orthotropic deck system based on the P-E method particularly for the System 2. The System 2 considers the deck to be continuous over flexible supports. As stated earlier a previous study which was conducted did not achieve the desired results due to the second step being neglected. In this study, the modelling will take different approaches in order to obtain a more accurate result. The orthotropic deck is modelled by employing the shell element to create the whole system. Following the definitions of the System 2 of the P-E method Springs were created as supports on the orthotropic deck to create the flexible behaviour. In the first approach, the Spring stiffness was obtained by having two spans of the section of the rib simply supported at both ends and displaced 1 mm at the mid of the span. This approach yields a force per unit length of 9346 N mm^{-1} as a Spring stiffness, K_1 as shown in Fig. 7a.

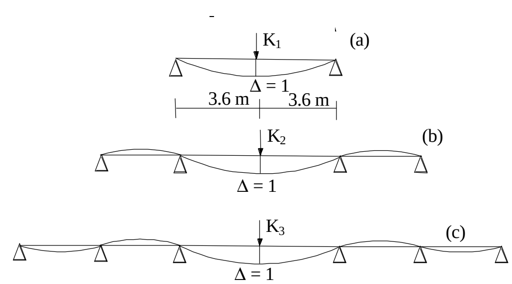

Fig. 7: Schematic of rib for obtaining Spring stiffness

To examine the influence of floor beam's existence on both sides, the rib was analyzed further as continous beams. The Spring stiffness factors obtained using this approach are K_2 = 9357 N mm^{-1} for the continous beams on four supports and K_3 = 9357 N mm^{-1} for the continous beams on six supports, respectively as shown in Fig. 7b and c. These results show that the existence of more floor beams has no effect on the value of Spring stiffness factor. To consider more accurately the influence of the stiffness of floor beam on the Spring stiffness factor, the first aproach was improved by considering a part of floor beam under the point load as shown in (Fig. 8). In this second approach, the boundary conditions imposed on the floor beams are freed in the vertical direction but restrained in transverse and longitudinal directions of the bridge. The study part of a floor beam considered in this approach are: the depth of web = 1270 mm; the thickness of web = 14.3 mm; the width of flange = 305 mm and the thickness of flange = 17.4 mm. This second approach yields K_1 = 10280 N mm^{-1}, K_2 = 10290 N mm^{-1} and

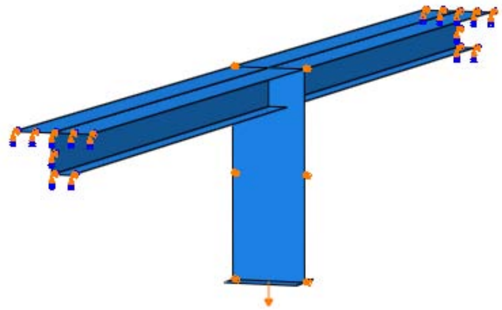

Fig. 8: Modeling of ribs including a part of floor beam for obtaining Spring stifness

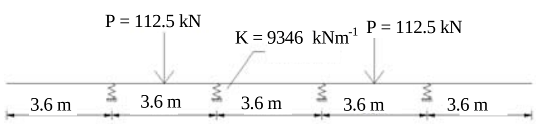

Fig. 9: Truck loading for evvaluating midpan moments

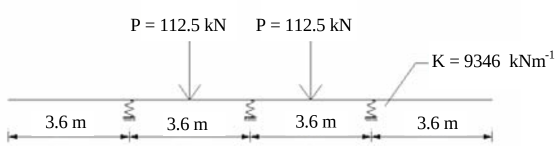

Fig. 10: Truck loading for evvaluating supports moments

Tabel 1: Spring stiffness for variations of supports number (N mm^{-1})

Spring stifness	FE+Spring	FE+Spring+part of floor beam
K_1	9346	10280
K_2	9357	10290
K_3	9357	10290

K_3= 10290 N mm^{-1}, respectively. These Spring stiffness factors were higher than the first approach due to the contribution of the stiffness of part of floor beam. The results of both approaches were summarized in Table 1.

Having acquired the Spring stiffnesses for both approaches, the orthotropic deck can now be modeled. The Spring stiffnesses used for the analysis are 9357 N mm^{-1} and 10290 Nmm^{-1} for the first and second approaches, respectively. The Springs are placed below the ribs where the floor beams acts as a flexible supports. The wheel load positionings to achieve the maximum moments at midspan and support are shown in Fig. 9 and. 10, respectively. The positionings of the Springs in 3-D ABAQUS model is shown in Fig. 11.

The orthotropic deck model considered in ABAQUS is 1.8 m wide with 3 ribs spaced at 0.45 m and is 18 m in length for the midspan moments while the model for the support moments is 14.4 m in length, both with the ribs spanned at 3.6 m as shown in Fig. 12 and 13.

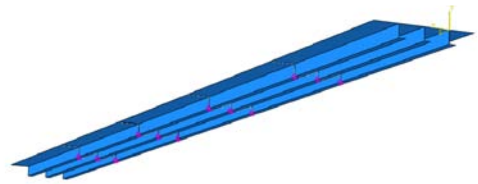

Fig. 11: Springs modeled on ABQUS

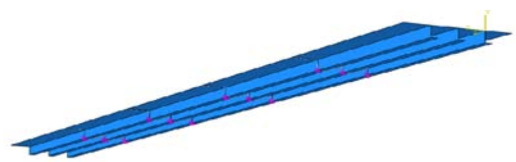

Fig. 12: Positioning of wheel loads for evaluating midpan moments on rib model

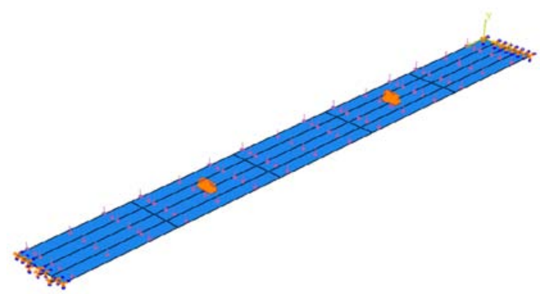

Fig. 13: Positioning of wheel loads for evaluatingsupport moments on rib model

RESULTS AND DISCUSSION

The contours of the longitudinal stresses for the midspan moments and the support moments are shown in Fig. 14 and 15, respectively. The present analysis generally yields slightly higher values when compared to the results of the analytical procedure as shown in colums 3 and 4 of Table 2. Further, the present analysis also gives closer results to the results of the analytical procedure than the previous study (Victoriawan, 2015) as shown in Column 2 of Table 1. This indicates that there is a difference in concept of analyzing the sections of the rib to achieve the critical value of stresses in both methods. The first difference can be explained as follow: the increasing of the longitudinal stresses in the FEM method could be caused by warping stresses. It should be noted that in the theory of thin-walled structures and especially for open cross-section, the contribution of warping stress to the longitudinal stresses due to bimoments (or warping moments) has a significant value

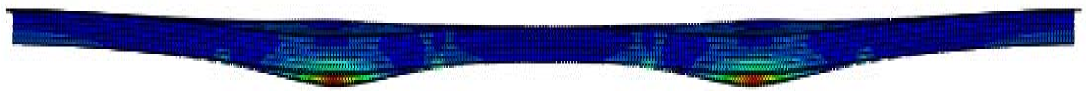

Fig. 14: Longitudinal stress results for midpan moments on rib

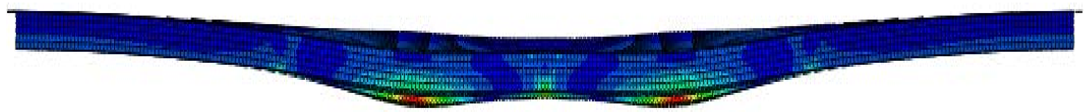

Fig. 15: Longitudinal stress results for support moments on rib

Table 2: Comparison of longitudinal stresses on ribs (Mpa)

Location	Analytical (1)	FE (2)	FE+Spring K = 9357 N mm^{-1} (3)	FE+Spring+part of floor beam K = 10290 N mm^{-1} (4)
Midspan				
Top	-68	-33	-86	-86
Bottom	80	133	100	98
Support				
Top	29	25	26	17
Botto	-73	-100	-79	-71

(Vlasov, 1961; Sugihardjo, 1990). In this phenomenon the floor beams act as rigid diaphragms restrained against warping. This warping phenomenon is not considered in the pure bending moment of the P-E method. The second difference is the way the wheel loads are considered in both methods. The P-E method considers the wheel loads as concentrated loads whereas the loads used in the ABAQUS Models are area loads (this assumption is more representative of actual loading). The final difference was the assumption used in P-E method in which neglecting the floor beam flexibility for calculating support moments for design purpose.

It can be seen from Column 3 of Table 2 that the longitudinal stresses on the ribs by considering the floor beam flexibility are generally higher than the analitycal method with the approximate differences of (8~26%), except and especially for the longitudinal stress at the top of ribs of the support section. If the floor beam flexibility was not taken into account by using FEM, the deviation to the analitycal method results is in the range of (18~100%) as shown in Column 2 of Table 2 (Victoriawan, 2015). Further observations, the differences between the longitudinal stresses on the ribs obtained from the P-E method and the FEM by considering the stiffness of the floor beam range from (3~41%)as listed in Column 4 of Table 2. The most significant difference is the longitudinal stress at the top of the ribs of the support section as shown in Column 4, i.e., 41%. As mentioned above it could be caused by assumption used in the P-E method which neglecting the floor beam flexibility for design purpose. Therefore the analytical longitudinal stress at the top of the ribs of the support section is higher than the actual stress as shown in Column 1. Column 3 and 1 of Table 2 further show that the longitudinal stress at the top of the ribs of the support section using the P-E method for design purpose could be reduced up to 10%. Finally, by comparing the stresses in Column 3 and 4 of Table 2, it was found that the longitudinal stresses of the ribs have a slight difference for both approaches. This shows that the existence of floor beam stiffness has some effect on FEM analysis results. However, for design purpose using FEM analysis, the first and simpler approach could be used for calculating Spring stiffness.

CONCLUSION

Based on the results of the investigations carried out in this study, the following conclusion can be drawn: The longitudinal stresses on the ribs obtained from the FEM by considering the floor beam flexibility have differences to the P-E method results in the range of (3~41%). Meanwhile, the differences between the longitudinal stresses on the ribs obtained from the FEM neglecting the floor beam flexibility and the P-E method results range from (180~100). Therfore, it could be concluded that tha

FEM analysis by considering tha flexibility of floor beams more accurately model the bridge deck system then the flexibility of floor beams

For design purpose, the longitudinal stress at the top of the ribs of the support section obtained from the P-E method which neglecting the floor beam flexibility, could be reduced up to 10% with respect to the FEM results by considering the floor beam flexiblity. For design purpose using FEM analysis, the simplified method can be used for calculation Spring stifiness as representative of a floor beam neglecting the exitence of floor beams

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