REINFORCEMENT-FREE DECKS USING A MODIFIED STRUT-AND-TIE MODEL: PART 2 – DESIGN

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ABSTRACT

This paper presents a direct design procedure using a modified strut-and-tie model (STM) that considers material and geometrical nonlinearity for reinforcement-free bridge decks on wide flange (or bulb tee) concrete girders. A modified STM was previously developed as a design tool for

restrained decks where the compressive membrane action is responsible for the enhancement of the capacity in the decks. This modified STM requires a nonlinear FEM analysis to find certain design variables, which defeats practicality. Therefore, these design variables are defined here for a practical range of configurations and dimensions of reinforcement-free bridge decks to be used in a direct design procedure. A required design resistance capacity for wheel loads was also defined considering failure caused by moving vehicle loads. A design example using the direct design procedure and a simplified design procedure using an equation alternative to the modified STM are provided for practical use.

Keywords: design procedure; strut-and-tie model; reinforcement-free deck; bridge decks; compressive membrane action; deck design; and wide flange precast concrete girder.

INTRODUCTION

Compressive membrane action is a well known load resisting mechanism occurring in laterally (in plane) restrained concrete slabs such as highway bridge decks. According to previous research findings[1-5] on compressive membrane action in bridge decks, the strength enhancement occurs as the deck reaches its failure capacity. Before the load on the deck approaches the failure load, flexural cracks develop at midspan and over the girders, in the positive and negative moment regions. After the flexural cracking, a lateral compressive membrane force develops as the deck starts hinging at the flexural cracks as shown in **Fig. 1.** The lateral restraint from adjoining portions of the deck slab itself, the stiffness of any diaphragms between girders, and the lateral stiffness of the girders themselves work to resist the motion and develop compressive membrane force. This behavior restricts growth of the flexural cracks and enhances flexural capacity of the deck. With added load, the deck will finally fail in flexure or punching shear depending on the degree of lateral resistance.

One situation where compressive membrane action can effectively be used in design is with reinforcement-free bridge decks[6] on wide flange concrete girders as shown in **Fig. 2**. In the design of the reinforcement-free bridge deck, deck reinforcing is eliminated and flexural strength enhancement from the compressive membrane action is relied upon with additional lateral restraint provided by adding steel ties between girders. Fiber reinforced polymer (FRP) stay-in-place (SIP) formwork[7] may be provided to prevent a single large crack from forming in the positive moment region and fiber reinforced concrete (FRC) is used in the deck to control early age shrinkage cracks. A pilot bridge with a reinforcement-free bridge deck was built in Wisconsin, U.S.A. to show the feasibility of the new deck system[6] as shown in **Fig. 3**.

An effective analysis and design tool which captures flexural and punching failure of these reinforcement-free bridge decks on concrete wide flange girders is required. A modified STM[8] analysis was developed to serve that purpose. The new model provides a 2D axisymmetric representation of the restrained 3-D deck and considers material and geometrical nonlinearity which differ from previously developed STMs[9-12].

The STM model was originally developed for reinforcement-free decks on Wisconsin 54W precast concrete bulb tee girders (**Fig. 4**) and showed that the model can effectively replace FEM analysis as described in the accompanying paper (Part 1). Further examination proves that the model has wider applicability on other wide flange bulb tee girders such as the Wisconsin 72W girders and Washington State WS53 girders[8] with similar torsional stiffness. The torsional constant (*J*) of the Wisconsin 54W girder is 15,333 in.^{4} (683,200 cm^4). Unfortunately nonlinear FEM analyses is required to define values for some of the design variables used in the modified STM approach. The results of those analyses are provided here for a range of precast bulb tee girder sections.

In this paper, a rational and efficient direct design procedure using a modified STM is presented for bridge decks on concrete wide flange girders that are tied together laterally by steel ties.

A required design capacity for the deck system is also suggested based on the current AASHTO[18] bridge design specification, nonlinear FEM analyses and a previous study[14] on the capacity of concrete decks under moving vehicle loads.

A simplified design procedure using an equation alternative to the modified STM is provided in addition to the direct design procedure. A step-by-step example for the design of a reinforcement free concrete bridge deck on precast girders with steel ties between girders is illustrated using the suggested design procedures.

RESEARCH SIGNIFICANCE

A new direct design procedure for reinforcement-free concrete bridge decks on wide flanged girders, based on a modified 2D axisymmetric STM, is presented. The design procedure considers punching and flexural failure of the restrained bridge deck while previously developed design methods[15-17] only considered one of the failure modes. The authors believe that the design procedure is based on a realistic mode of failure, compared to current AASHTO deck design, and it provides a practical replacement for a more accurate but time consuming design using a FEM analysis. A simplified design procedure is developed in addition to the direct design procedure.

LIMITATION OF THE DESIGN PROCEDURE

The following limitations are applied for the design of a reinforcement-free deck using the proposed design procedures.

1) The proposed design procedure should only be used for cast-in-place concrete decks made composite with 50 to 72 in. (1270 to 1830 mm) deep wide flange (bulb tee) concrete girders that are connected by lateral steel ties,
2) The decks are presumed to be built without reinforcing added to provide strength. Special reinforcing, however, should be placed in the deck over the end diaphragms, parallel to the diaphragms, to control deck shrinkage cracking caused by restraint from the diaphragms,
3) Non-metallic fibers should be added to the concrete to control early age shrinkage cracks,
4) A fiber reinforced polymer form panel bonded to the concrete, or equivalent system, should be provided for flexural crack control at the bottom of the deck in the region between girders to

meet a service limit state "good appearance" requirement,

5) The depth of the concrete deck shall not be less than 7 in. (178 mm),

6) The girder center-to-center spacing is limited to 10 ft. (3048 mm),

7) The spacing of the lateral steel ties shall not be wider than 10 ft. (3048 mm),

8) The clear span of the deck shall not be greater than 6 ft. (1829 mm),

9) The 28day compressive strength of the concrete for the deck shall be larger than 4000 psi (27.6 MPa),

10) The material of the lateral steel tie must be structural steel with a strength of at least 36 ksi (248 MPa),

11) The design of the overhanging portion of the deck is excluded from this design procedure. An overhang needs to be designed using a conventional design method since compressive membrane action does not occur at the overhangs. The best design would eliminate a deck overhang beyond the edge of the girder flange.

SELECTION OF DESIGN LOAD

The concrete deck must be designed to be able to withstand specific limit states. The limit states are normally established in three major categories – service, fatigue and strength. Different design loads are used with each limit state.

Service limit states

The service limit state in concrete is primarily for limiting deflections. Nonlinear FEM analyses[18] were used to compare deck displacements with the limits suggested in the AASHTO LRFD (2007) bridge design specification's section 9.5.2[13]. The comparison proved that deflections were not a controlling issue for decks deeper than 7 in. (178 mm) with 3 to 6 ft. (914 to 1829 mm) clear span as shown in **Table 1**.

Fatigue limit state

A structure may fail prematurely due to stationary pulsating loads or moving loads which induce fatigue, even if the load is below the static strength capacity. Research on fatigue of concrete decks using moving loads has found that failure can occur after 100 million cycles of load at values as low as 14% of the static ultimate capacity of the deck.[14]. In other words, the deck can resist 100 million cycles of a moving load if the ultimate capacity of the deck is greater than 700% of the load.

For satisfactory fatigue performance, the required design strength capacity of the deck is selected as 7 times the service load (with dynamic load allowance) for fatigue. The service wheel live load including dynamic load allowance for fatigue (15 %) is 18.4 kips (81.8 kN) per AASHTO LRFD bridge design specifications[13]. The fatigue design load would then be equal to 129 kips (574 kN). This will provide a sufficient safety margin against fatigue failure when the reinforcement-free deck is designed for strength using a static wheel load.

Strength limit state

The largest strength load factor for vehicle live load and vehicle dynamic load allowance is 1.75 in the AASHTO LRFD bridge design specifications Table 3.4.1-1[13]. Therefore the design strength load, including multiple presence factor (1.2) from AASHTO LRFD Table 3.6.1.1.2-1[13], for the strength limit state is 44.7 kips (198.8 kN).

Design load for reinforcement-free decks

Comparing the strength limit state demand of 44.7 kips (198.8 kN) with the strength needed to avoid long term fatigue, 129 kips (574 kN), the fatigue strength requirement controls. The design load for the reinforcement-free deck is selected as a 129 kip (574 kN) vehicle single wheel load.

PROPOSED DESIGN PROCEDURES

The primary aim of the design procedure is to have the deck withstand the design strength load of

129 kips (574 kN) with a cost effective section. The strength procedure should also reflect actual mechanisms observed in bridge deck failures. While the AASHTO[13] procedure assumes flexural failure, actual bridge deck load tests and moving load tests generally have shown that final failure occurs through a punching shear mechanism. The proposed procedure includes both flexural and punching failure modes. Detailed derivation of the equations used in the design procedures can be found elsewhere[8] and Part 1 of this paper.

Factors for direct design procedure using the modified STM

Three critical factors are needed to apply the modified STM: 1) the shape of the compressive strut that develops in the deck, 2) the length of a tension crack that might develop perpendicular to the strut, and 3) the restraint provided in the deck from the torsional rigidity of an external girder.

As detailed in Part 1, these special factors are most easily obtained by using nonlinear FEM analysis of the deck and the girders. The shape of the compressive strut is described in terms of the spreading angle at the ends of the strut (θ_2) and the ratio R_2 defined as a weighted average width of the compressive strut over the average of the widths at the ends. The spreading angle (θ_2) at the strut ends can be found from the compressive principal stress trajectories near the ends. The weighted average width of a compressive strut is defined as the average of widths at two ends weighted by 0.5 and width at mid-length weighted by 0.5. If the widths at the ends are known, R_2 can be found when the width of the strut at mid-length is known. The width of the strut at mid-length can be found from the width of the compressive region of the principal stress.

The length of a tension crack is described in terms of the ratio R_1 defined as the cracked length of the diagonal strut over the length of the strut. The cracked length can be found from the length of the compressive strut where the strain in a perpendicular direction to the strut is plastic at failure.

The restraint provided in the deck from the torsional rigidity of an external girder is described in terms of the lateral translational displacement under lateral unit distributed patch load at the top of the girder due to torsional deformation of the exterior girder. The procedure to find the displacement is provided in Part 1 of this paper.

Based on results of numerical studies **Tables 2-5** can be used to select these parameters. A linear interpolation scheme may be used to find values not given in **Tables 2-5**. The values in **Tables 2-5** were derived for the Wisconsin 54W girder [54 in. (1372 mm) deep girder], but have been found[8] to be applicable to similar girders with depths between 54 in. and 72 in. (1372mm to 1830mm). The capacities of the decks on Wisconsin 54W and 72W [72 in. (1830 mm) deep girder] were found to be close enough to indicate that the values in **Table 2-5** are less dependant on the type of the girder compared to depth and clear span of the deck.

Design steps for direct design procedure using the modified STM

The design is initiated by the following steps:

Step 1: select clear span of the deck which is determined from the girder spacing and girder flange width,

Step 2: select a deck thickness,

Step 3: select the deck restraining factor (R) calculated using Equation (1), with an initial recommended design value of 900 lb/in^2 (6.20 N/mm^2), from which the girder tie bars are selected.

$$R = \frac{(axial\ stiffness\ of\ a\ lateral\ steel\ tie) \times (thickness\ of\ deck)}{(center\ to\ center\ spacing\ of\ girders) \times (spacing\ of\ lateral\ steel\ tie)} \quad (1)$$

Then the adequacy of the deck is checked using a strut and tie (truss) analysis to determine the internal forces and compared with the strength of the struts and tie as follows:

Step 4: calculate the stiffness of the truss elements for the strut and tie model of the deck,

Step 5: estimate the strength capacity of the struts and tie in the truss model,

Step 6: analyze the forces developed in the truss under the vehicle load on the deck, and

Step 7: compare the truss forces from the loading with the strut and tie strength capacities to ensure safety.

Selection of deck depth

Typically an initial selection of deck thickness is based on experience. The ultimate capacities of deck systems with a restraining factor of 870 lb/in^2 (6.00 N/mm^2) from FEM analysis[18] is listed in **Table 6**. In lieu of selecting a deck thickness based on experience, **Table 7** can be used to select an initial thickness based on the span length. The values in **Table 7** are based on the nonlinear FEM analysis results listed in **Table 6**.

Selection of tie bars using the deck restraining factor

An initial spacing selection for the lateral steel tie of 10 ft. (3048 mm) is suggested. The 10 ft. (3048 mm) tie spacing is the largest spacing in the scope of the proposed system. A wide spacing would reduce the need for handling of material during construction and the number of holes in the girders for the ties.

Next select the cross-sectional area of the lateral steel tie. The initial cross-sectional area of the lateral steel tie can be calculated using Equation (1) with the selected deck thickness, girder spacing, and tie spacing while using the recommended deck restraining factor (R) of 900 lb/in^2 (6.20 N/mm^2). Since any improvement in the ultimate strength of the deck is minimal with an increase of the deck restraining factor (R) over 900 lb/in^2 (6.20 N/mm^2), as shown in a parameter study using FEM and STM analysis[8] and Part 1 of this paper, this value should be used as a target design value.

Calculation of the stiffnesses of the truss elements

The STM analysis to determine forces in the deck will use a truss model as illustrated in **Fig. 5**. The diagonal strut stiffness in the truss depends on the effective cross sectional area of deck concrete that participates in the strut.

The modulus of the strut, E_d, is taken as the concrete modulus. The strut actually represents a 3-D cone of compressive resistance that develops around the wheel patch load. The 2-D representation, from finite element stress analysis, is bottle shaped with narrow ends and a wider middle. The effective cross sectional area of the resisting strut can

be taken as Equation (2). Equations (3)-(4) can be used to calculate Equation (2).

$$A_{ds} = \frac{R_2 \pi}{4}(L+8)w_{ends} \qquad \left(A_{ds} = \frac{R_2 \pi}{4}(L+200)w_{ends} \right) \tag{2}$$

$$w_{ends} = \frac{t}{3}\cos(\theta_1) + 4\sin(\theta_1) \qquad \left(w_{ends} = \frac{t}{3}\cos(\theta_1) + 100\sin(\theta_1) \right) \tag{3}$$

$$\theta_1 = \tan^{-1}\left(\frac{4t}{3(L-8)} \right) \qquad \left(\theta_1 = \tan^{-1}\left(\frac{4t}{3(L-200)} \right) \right) \tag{4}$$

The bottom tension tie represents the lateral restraint against outward spread at the bottom of the truss. The actual restraint is provided by adjacent deck slab, lateral and torsional girder stiffness, and the steel tie stiffness. The restraint contributed by each of these sources is calculated in Equations (5)-(7) and the total restraint on the truss is given in Equation (8).

$$K_{lt} = \frac{\pi A_t E_s}{2S_t (S_g + t_w)} S_w \tag{5}$$

$$K_{\lg b} = \frac{24\pi E_g I_{yg}}{S_t^{\,3} - \frac{L^2}{2} S_t + \frac{L^3}{8}} \tag{6}$$

$$K_{\lg t} = \frac{1000\pi L}{2\delta_{\lg t}} \qquad \left(K_{\lg t} = \frac{175\pi L}{2\delta_{\lg t}} \right) \tag{7}$$

$$\frac{1}{K_{lm}} = \frac{1}{K_{lt}} + \frac{1}{K_{\lg b}} + \frac{1}{K_{\lg t}} \tag{8}$$

Estimating the strength capacity limits of the struts and tie

The diagonal compression strut capacity, when transverse tension cracking and shear punching occurs, can be calculated using Equation (9).

$$P_{uds} = 5\sqrt{f_c'}\left[\frac{R_1 \pi (L+8)}{4} \sqrt{\frac{(L-8)^2}{4} + .444t^2} \right] \Big/ \tan\left(\theta_2 / 2 \right)$$

$$\left(P_{uds} = 0.415\sqrt{f_c'}\left[\frac{R_1\pi(L+200)}{4}\sqrt{\frac{(L-200)^2}{4} + .444t^2} \right] \Big/ \tan\left(\theta_2/2\right) \right) \quad (9)$$

The capacity of the bottom tie is used to represent the flexural capacity of the deck. Flexural resistance in the deck is assumed to develop over the effective deck strip width given in AASHTO LRFD Table 4.6.2.1.3-1[13]. The compression force at the top of the deck is equal to the tension in the bottom tie. When the compression stress block in the concrete at the top of the deck reaches the crushing point, the capacity of the bottom tie may be calculated by first finding the depth of the concrete compression block using Equations (10)-(11) and then solving for P_{utls} using Equation (12) which is also the capacity of the equivalent bottom tie:

$$A_{vr} = K_{lm}\frac{2E_w}{\pi E_s} \quad (10)$$

$$a^2 + \frac{A_{vr}E_s}{.85f'_c E_w}\left(0.003a - 0.0025t\beta_1\right) = 0 \quad (11)$$

$$P_{utls} = 0.85f'_c E_w a \quad (12)$$

Analyzing the forces developed in the truss under the vehicle load

The forces in the truss should now be determined using an elastic model, but it is necessary to include geometric effects with a P-Δ analysis. Nearly all structural analysis software packages allow the user to select a P-Δ analysis when examining a structure. In this case the P-Δ analysis is essential in determining the forces in the truss. The truss should be loaded with the factored design load of 129 kips (574 kN) noted earlier.

Comparing the truss forces and tie capacities to ensure safety

The safety of the truss may be determined by comparing the calculated internal forces with the capacities already calculated for the diagonal compression struts and the bottom tension tie. If the strut capacity is exceeded, the deck will fail in punching shear before reaching the design load. If the bottom tie capacity is exceeded, the deck is expected to fail with compression crushing of the top concrete in flexure under the wheel load. If neither capacity is exceeded the deck design is satisfactory. If the deck is found to fail in punching shear, the deck thickness should be increased. If failing in flexure, either the restraint R or the thickness should be increased.

Design steps with the simplified design procedure

An alternate simpler method to predict the ultimate capacity of reinforcement-free decks on 54 to 72 in. (1372 to 1830 mm) deep wide flanged precast girders is provided in Eq. (13). This equation can replace the STM analysis steps (Steps 4-7) in the direct design procedure. The equation is based on results from 99 FEM analyses. The analyses looked at decks with various amounts of lateral "membrane" restraint represented by restraint factors "R" as calculated using Equation (1). Restraint factors, R, of 200 lb/in^2 to 1200 lb/in^2 (1.38 N/mm^2 to 8.27 N/mm^2) were used with clear spans of 3 ft to 6 ft. (914 mm to 1830 mm), deck thicknesses of 7.0 in. to 9.0 in. (178 mm to 229mm) deck thickness, and lateral steel tie spacing of 6 ft. to 10 ft. (1830 mm to 3048 mm).

Using the results from the parametric study, the deck's wheel load capacity, for a wheel having an AASHTO[13] patch size, is given by P_d.

$$P_d = 13t^{1.894}L^{-0.541}\left(\frac{K_t}{S_t}\right)^{0.225} \qquad \text{(SI units: } P_d = 2.227t^{1.894}L^{-0.541}\left(\frac{K_t}{S_t}\right)^{0.225}\text{)} \qquad (13)$$

The equation on average predicted 94.8% of the deck capacity defined by more accurate FEM analysis results. The standard deviation was 5.8%. The relationship between the predicted capacities of the deck using the proposed equation and those of the deck using nonlinear FEM analysis is shown in **Fig. 6**. The bold line in **Fig. 6** indicates the result if the two analyses matched perfectly. Most of the points are at the upper side of the bold line indicating that the proposed equation is conservative.

DECK DESIGN EXAMPLE

A step by step design using the suggested direct design procedure for a reinforcement-free deck on 54 in. (1372mm) deep wide flanged girders is illustrated here. A comparison was made with the proposed simplified design procedure.

1) Given information for the sample deck on Wisconsin 54W girders when the clear span of the deck between girders is known:

Clear span of the deck: L = 4ft. (1219 mm)

Spacing of the girders: S_g = 8ft. (2438 mm)

Thickness of the web of the girder: t_w = 6.5 in. (165 mm)

Design compressive strength of the deck: $f_c^{'}$ = 4000 psi (27.6 MPa), β_1 =0.85

Design compressive strength of the girder: $f_c^{'}$ = 8000 psi (55.1 MPa)

Modulus of elasticity of the deck: E_d = 3605 ksi (24.8 GPa)

Modulus of elasticity of the girder: E_g = 5098 ksi (35.1 GPa)

Moment of inertia of the girder in weak axis: I_{yg} = 125056 in.^{4} (52,052$\times 10^6$ mm^4)

Modulus of elasticity of the lateral steel tie: E_s = 29,000 ksi (199.8 GPa)

Spacing of the vehicle axles in longitudinal direction: S_w = 14 ft. (4267 mm)

2) Selection of the deck depth

Select the deck depth using **Table 7**.

Selected trial depth of the deck: t = 7.5 in. (191 mm)

3) Select the spacing of the lateral steel tie as 10 ft. which is the largest spacing within the scope of the proposed system.

Spacing of the lateral steel ties: S_t = 10 ft. (3048 mm)

4) Find the needed axial stiffness of a single lateral tie (K_t) from Equation (1) using the recommended deck restraining factor (R) = 900 lb/in^2. (6.20 N/mm^2)

$$K_t = (900 \text{ lb/in}^2)\ S_g\ S_t\ /\ t = 1382.4 \text{ kip/in } (242.1 \text{ kN/mm})$$

5) Calculate the cross-sectional area of a single lateral tie from the axial stiffness found above.

$$K_t = \frac{A_t E_s}{(S_g + t_w)}$$

$$A_t = \frac{K_t (S_g + t_w)}{E_s} = 4.886 \text{ in.}^2 \ (3152 \text{ mm}^2)$$

Assume tension tie bars with 2.7 in. (69 mm) diameter.

$$A_t = \frac{\pi (2.7 in)^2}{4} = 5.726 \text{ in.}^2 \ (3694 \text{ mm}^2)$$

6) Find factors from **Tables 2-5** for the direct design method.

$$\theta_2 = 53.117°, \text{R}_2 = 2.00, \ R_1 = 0.565, \ \delta_{\lg t} = 0.0257 \text{ in. } (0.653 \text{ mm})$$

7) Predict the ultimate capacity of the deck system using the modified STM analysis.

Calculate cross-sectional area of the diagonal strut using Equations (2)-(4).

$$\theta_1 = 14.03°, \ w_{ends} = 3.395 \text{ in. } (86.23 \text{ mm}), \ A_{ds} = 298.7 \text{ in.}^2 \ (1{,}926{,}699 \text{ mm}^2)$$

Calculate combined restraint K_{lm} using Equations (5)-(8).

$$K_{lt} = 3{,}562{,}652 \text{ lb/in. } (623{,}885 \text{ N/mm})$$

$$K_{\lg b} = 29{,}977{,}388 \text{ lb/in. } (5{,}249{,}583 \text{ N/mm})$$

$$K_{\lg t} = 2{,}933{,}783 \text{ lb/in } (513{,}759 \text{ N/mm})$$

$$K_{lm} = 1{,}526{,}939 \text{ lb/in. } (267{,}395 \text{ N/mm})$$

Calculate capacity of the truss members using Equations (9)-(12)

$$P_{uds} = \; = 324{,}087 \text{ lb } (1{,}441{,}541 \text{ N}), \quad E_w = 26 + 6.6S \; = 78.8 \text{ in. } (2001 \text{ mm})$$

$$A_{vr} \; = 2.641 \text{ in.}^2 \; (1704 \text{ mm}^2), \quad a \; = 1.748 \text{ in. } (44.40 \text{ mm})$$

$$P_{utls} \; = 468{,}437 \text{ lb } (2{,}083{,}606 \text{ N})$$

Construct the modified STM and perform 2nd order (P-delta) analysis to check if the model has sufficient capacity to withstand the design load [129kips (574 kN)].

The member forces of the model under the design load were 289.9 kips (1290 kN) for the diagonal member and 284.1 kips (1264 kN) for the lateral member. This result indicates that the trial section of the deck system is safe under the design load since the member forces were less than their capacities.

The ultimate capacity found from the second order truss analysis with gradually increased load over the design load 129 kips (574 kN) was 142.4 kips (633 kN). The member forces at the failure of the model were 324.1 kips (1442 kN) for the diagonal member and 318.0 kips (1414 kN) for the lateral member. The diagonal member reached its capacity first - indicating that the failure mode was a punching failure. The design of the deck is completed since the predicted capacity is higher than the design load.

8) Comparison with the simplified design procedure using the proposed equation alternative to the modified STM analysis.

Use Eq.(13) to calculate the predicted capacity of the deck.

$$t = 7.5 \text{ in. } (191 \text{ mm}), \; L = 4\text{ft.} = 48 \text{ in. } (1219 \text{ mm}),$$

$$K_t \; = \; \frac{A_t E_s}{(S_g + t_w)} = 1620 \text{ kip/in } (283.7 \text{ kN/mm}),$$

$$S_t = 10 \text{ ft.} = 120 \text{ in. } (3048 \text{ mm}),$$

$$P_d = 13 t^{1.894} L^{-0.541} \left(\frac{K_t}{S_t} \right)^{0.225} \; = 130.6 \text{ kips } (581 \text{ kN})$$

The predicted capacity using the alternative equation is 130.6 kips (581 kN) versus the predicted capacity using the modified STM [142.4 kips (633 kN)].

RECOMMENDED FUTURE RESEARCH

Based on the present examination of restrained decks, the following future work is recommended to create a more generally applicable design method:

1 The developed equation alternative to the modified STM analysis to predict the ultimate capacity of the reinforcement-free deck system on wide flange (or bulb tee) concrete girders can be modified for general application to reinforcement-free decks with other types of girders by adding the stiffness of the girders to the equation; and

2 The required design capacity for the deck is based on fatigue testing of reinforced concrete decks. Testing to prove that similar fatigue capacities are needed in reinforcement-free decks is desirable. Current designs using AASHTO or similar procedures, however, are not based on deck fatigue capacity but just the deck flexural capacity. The overall importance of moving loads and deck fatigue capacity for design needs to be defined more clearly.

SUMMARY

The following summary is based on the development of the design procedure for reinforcement free-decks on wide flange concrete girders.

1 The design load for reinforcement-free bridge decks on concrete wide flange girders was defined based on the current AASHTO specification and previous studies on the capacity of concrete decks under moving vehicle loads. The controlling limit state for the design load was selected as the fatigue limit state and the design load was selected as a 129 kip (574 kN) vehicle single wheel load. This is a much more severe design load than used in present design methods and specifications.

2 A safety margin was considered in the selection of the design load by using a design load that should allow the deck to withstand over 100 million cycles of repeated loading at the service design level.

3 A rational and efficient direct design procedure using the modified STM for bridge decks on concrete wide flange girders that are tied together laterally by steel ties to provide deck membrane restraint was presented.

4 The new direct design procedure using the modified STM and the simplified design procedure using an equation alternative to the modified STM can be used to design bridge decks within the dimensions and support limits defined for this research.

5 The proposed equation alternative to the modified STM to design a reinforcement-free deck system on 54 to 72 in. (1372 to 1830 mm) deep wide flange (or bulb tee) concrete girders can predict the ultimate capacity of the bridge decks with sufficient accuracy.

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NOTATION

a = depth of the compression block at the top of the deck, in. (mm)

A_{ds} = area of the diagonal strut, in^2 (mm^2)

A_t = cross-sectional area of the single tension tie, in.^{2} (mm^2)

A_{vr} = cross-sectional area of virtual tie, in^2 (mm^2)

E_g = elastic modulus of the girder, psi (MPa)

E_s = modulus of elasticity of steel, psi (MPa)

E_w = 26 + 6.6S (660.4 + 0.55S), in. (mm)

f_c' = concrete compressive strength, psi (MPa)

I_{yg} = moment of inertia of the girder about weak axis, in^4 (mm^4)

K_{lgb} = lateral restraint due to weak axis bending of the girder, lb/in (N/mm)

$K_{\lg t}$ = lateral restraint due to torsion of the girder, lb/in (N/mm)

K_{lm} =the combined restraint at the bottom of the 2D model, lb/in (N/mm)

K_{lt} = lateral restraint due to the tension ties resisting a single vehicle wheel, lb/in (N/mm)

K_t = axial stiffness of single lateral tie [=$\dfrac{A_t E_s}{(S_g + t_w)}$, kips/in (kN/mm)]

L = deck clear span between girder flanges, in. (mm)

P_d = wheel load capacity of the deck, kips (kN)

P_{uds} = the capacity of the diagonal compression strut, lbs (N)

P_{utls} = the flexural capacity of the deck, and the capacity limit for the bottom tension tie, lbs (N)

R_1 = ratio of crack length over strut length

S = center to center spacing of girders, ft. (mm)

S_g = center to center spacing of girders, in (mm)

S_t = spacing of the tension ties, in. (mm)

S_w = the spacing of the vehicle axle and wheels in longitudinal direction (in direction parallel to the girder), in. (mm)

t = deck thickness, in. (mm)

t_w = thickness of the web of the girder, in (mm)

w_{ends} = average of the strut width at the two ends, in. (mm)

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth

$\delta_{\lg t}$ = the lateral torsional displacement at the top of the girder, in. (mm)

θ_1 = angle of the compression strut

θ_2 = the angle of spread at the ends of the compression strut

REFERENCES

1. Westgaard, H.M., Slater, W.A., "Moments and Stress in Slabs," *Journal of American Concrete Institute Proceedings,* V. 17, 1921, pp. 415-538.

2. Ockleston, A.J., "Load Tests on a Three Story Reinforced Concrete Building in Johannesburg," *The Structural Engineer*, V. 33, 1955, pp. 304-322.

3. Ockleston, A.J., "Arching Action in Reinforced Concrete Slabs," *The Structural Engineer*, V. 36, 1958, pp. 197-201.

4. Park, R., "Ultimate Strength of Rectangular Concrete Slab under Short-term Uniform Loading with Edges Restrained against Lateral Movement," *Proceedings of the Institute of Civil Engineers*, V. 28, 1964, pp.125-150.

5. Park, R., "The Lateral Stiffness and Strength Required to Ensure Membrane Action at the Ultimate load of a Reinforced Concrete Slab-and-Beam Floor," *Magazine of Concrete Research*, V. 17, No. 50, 1965, pp. 29-38.

6. Oliva, M.G..; Bae, H.; Bank, L.C.; Russell, J.S.; Carter, J.W.; and Becker, S., "Design and Construction of a Reinforcement Free Concrete Bridge Deck on Precast Bulb Tee Girders," *Proceedings 2007 PCI/FHWA National Bridge Conference*, Phoenix, AZ, October 21-24, 2007, CD-ROM.

7. Bank, L.C., Oliva, M.G., Bae, H., Barker, J.W., and Yoo, S., "Pultruded FRP Plank as Formwork and Reinforcement for Concrete Members," *Advances in Structural Engineering*, V. 10, No. 5, 2007, pp. 525-535.

8. Bae, H., "Design of Reinforcement-free Bridge Decks with Wide Flange Prestressed Precast Girders," PhD Thesis, University of Wisconsin, Madison, WI, 2008.

9. Schlaich, J., Schafer, K., and Jennewein, M., "Towards a Consistent Design of Structural Concrete," *Journal of the Prestressed Concrete Institute*, V. 32, 1978, pp. 74-150.

10. Collins, M.P., and Mitchell, D., "A Rational Approach to Shear Design," *ACI Structural Journal*, V. 83, No. 6, 1986, pp. 925-933.

11. Cook, D.W., and Mitchell, D., "Studies of Disturbed Region near Discontinuities in Reinforced Concrete Members," *ACI Structural Journal*, V. 85, No. 2, 1988, pp. 206-216.

12. Park, J., and Kuchma, D., "Strut-and-tie Model Analysis for Strength Prediction of Deep Beams," *ACI Structural Journal,* V. 104, No. 6, 2007, pp. 657-666.

13. AASHTO., *AASHTO LRFD Bridge Design Specifications*, 4th Ed., American Association of State Highway and Transportation Officials, Washington, D.C., 2007.

14. Petrou, M. F.; Perdikaris, P. C.; and Wang, A., "Fatigue Behavior of Noncomposite Reinforced Concrete Bridge Deck Models," *Transportation Research Board 1460*, TRB, National Research Council, Washington, D.C., 1994, pp. 73-80.

15. Hewitt, B. E,. and Batchelor, B. deV., "Punching Shear Strength of Restrained Slabs," *Journal of Structural Division*, ASCE, V. 101, No. ST9, 1975, pp. 1837-1853.

16. Kuang, J. S., and Moley, C. T., "A Plasticity Model for Punching Shear of Laterally Restrained Slabs with Compressive Membrane Action," *International Journal of Mechanical Sciences*, V. 32, No. 5, 1993, pp. 371-385.

17. Mufti, A. A., and Newhook, J. P., "Punching Shear Strength of Restrained Concrete Bridge Deck Slabs," *ACI structural journal*, V. 95, No. 4, 1998, pp. 375-381.

18. Hibbitt, Karlsson & Sorensen Inc., "ABAQUS User's manual," Hibbitt, Karlsson & Sorensen Inc., Pawtucket, RI, USA, 2004.

TABLES AND FIGURES

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Table 1–Deflection of the deck compared with the AASHTO[18] prescribed limit

Depth of the deck, in. (mm)	Clear span of the deck, ft. (mm)	Spacing of the girders, ft. (mm)	Deflection caused by live load plus dynamic load allowance, in. (mm)	Max. limit AASHTO, in. (mm)	Result
7.0 (178)	3 (914)	7 (2134)	0.012 (0.3048)	0.070 (1.778)	O.K.
7.0 (178)	4 (1219)	8 (2438)	0.017 (0.4318)	0.080 (2.032)	O.K.
7.0 (178)	5 (1524)	9 (2743)	0.022 (0.5588)	0.090 (2.286)	O.K.
7.0 (178)	6 (1829)	10 (3048)	0.028 (0.7112)	0.100 (2.540)	O.K.

Table 2–Spreading angle (θ_2, degree) of the compressive force in diagonal direction found from nonlinear FEM analyses

		Clear span of the deck, ft. (mm)			
		3 (914)	4 (1219)	5 (1524)	6 (1829)
Depth of the deck, in. (mm)	7.0 (178)	55.889	54.491	48.932	25.455
	7.5 (191)	55.889	53.117	47.508	35.216
	8.0 (203)	47.547	43.663	40.211	30.623
	8.5 (216)	46.181	41.740	38.310	34.353
	9.0 (229)	43.663	43.470	41.740	44.510

Table 3–Ratio (R_2)found from nonlinear FEM analysis

		Clear span of the deck, ft. (mm)			
		3 (914)	3 (914)	3 (914)	3 (914)
Depth of the deck, in. (mm)	7.0 (178)	2.0	2.0	2.0	2.5
	7.5 (191)	2.0	2.0	2.0	2.5
	8.0 (203)	2.0	2.0	2.0	2.0
	8.5 (216)	2.0	2.0	2.0	2.0
	9.0 (229)	1.5	2.0	2.0	2.0

Table 4–Ratio (R_1) of the cracked length over the length of the diagonal strut found from nonlinear FEM analyses

		Clear span of the deck, ft. (mm)			
		3 (914)	4 (1219)	5 (1524)	6 (1829)
Depth of the deck, in. (mm)	7.0 (178)	0.730	0.546	0.441	0.265
	7.5 (191)	0.733	0.565	0.438	0.270
	8.0 (203)	0.787	0.683	0.449	0.272
	8.5 (216)	0.802	0.719	0.444	0.337
	9.0 (229)	0.891	0.719	0.444	0.335

Table 5–Lateral torsional displacement [δ_{lgt} , in. (mm)] at the top of the girder found from FEM analyses under unit lateral load of 1 kip/in (0.177 kN/mm)

		Clear span of the deck, ft. (mm)			
		3 (914)	4 (1219)	5 (1524)	6 (1829)
Depth of the deck, in. (mm)	7.0 (178)	0.0155 (0.394)	0.0273 (0.693)	0.0339 (0.861)	0.0404 (1.026)
	7.5 (191)	0.0146 (0.371)	0.0257 (0.653)	0.0319 (0.810)	0.0380 (0.965)
	8.0 (203)	0.0137 (0.348)	0.0242 (0.615)	0.0304 (0.772)	0.0358 (0.909)
	8.5 (216)	0.0149 (0.378)	0.0228 (0.579)	0.0286 (0.726)	0.0337 (0.856)
	9.0 (229)	0.0152 (0.386)	0.0203 (0.516)	0.0256 (0.650)	0.0294 (0.747)

Table 6–Ultimate capacity of the deck system with an 870 lb/in^2 (6.00 N/mm^2) restraining factor, from FEM analysis, kips (kN)

		Clear span of the deck, ft. (mm)			
		3 (914)	4 (1219)	5 (1524)	6 (1829)
Depth of the deck, in. (mm)	7.0 (178)	128.86 (573.17)	126.31 (561.83)	114.60 (509.74)	93.45 (415.67)
	7.5 (191)	143.69 (639.13)	136.83 (608.62)	126.85 (564.23)	105.30 (468.37)
	8.0 (203)	153.35 (682.10)	144.80 (644.07)	142.01 (631.66)	117.32 (521.84)
	8.5 (216)	161.32 (717.55)	155.57 (691.98)	152.15 (676.76)	128.41 (571.17)
	9.0 (229)	193.89 (862.42)	166.06 (738.63)	164.58 (732.05)	143.26 (637.22)

Table 7–Initial selection of the depth of the deck for the given clear span of the deck

		Clear span of the deck, ft. (mm)			
		3 (914)	4 (1219)	5 (1524)	6 (1829)
Depth of the deck, in. (mm)	7.0 (178)	X	X	X	X
	7.5 (191)	O	O	X	X
	8.0 (203)	Δ	Δ	O	X
	8.5 (216)	Δ	Δ	Δ	X
	9.0 (229)	Δ	Δ	Δ	O

O: Highly recommended, Δ: Recommended and X Not recommended

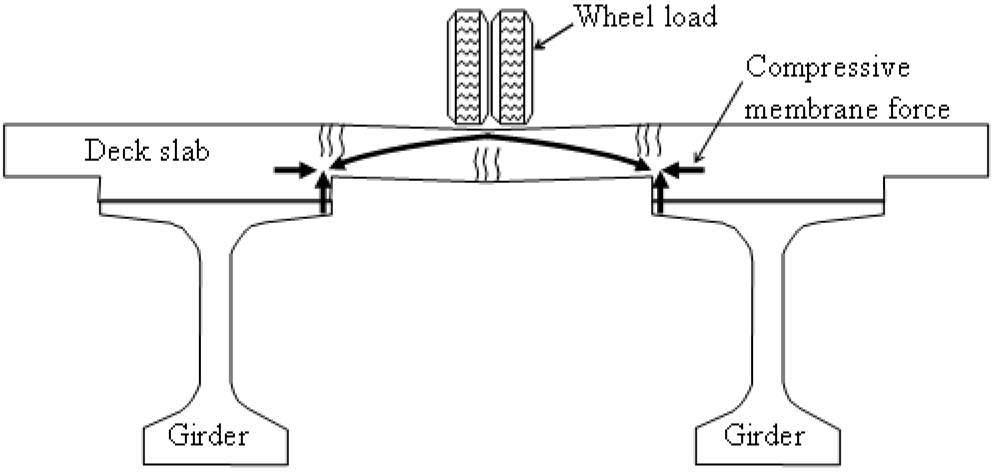

Fig. 1–Compressive membrane action in the cross section of the bridge deck.

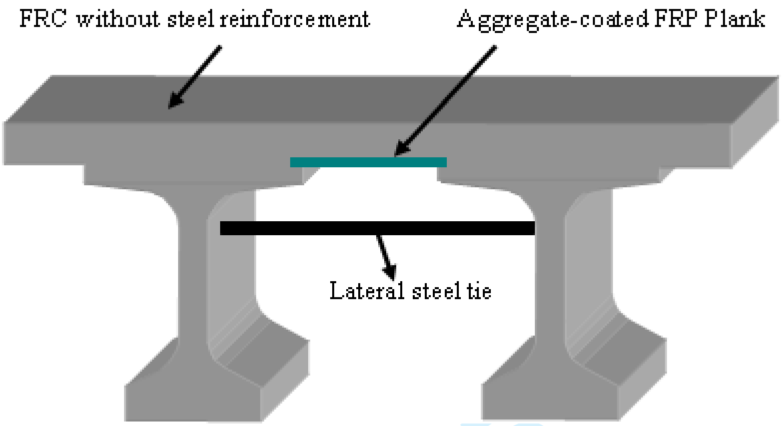

Fig. 2–Concept of the reinforcement-free deck.

Fig. 3–Construction of a pilot bridge with a reinforcement-free bridge deck.

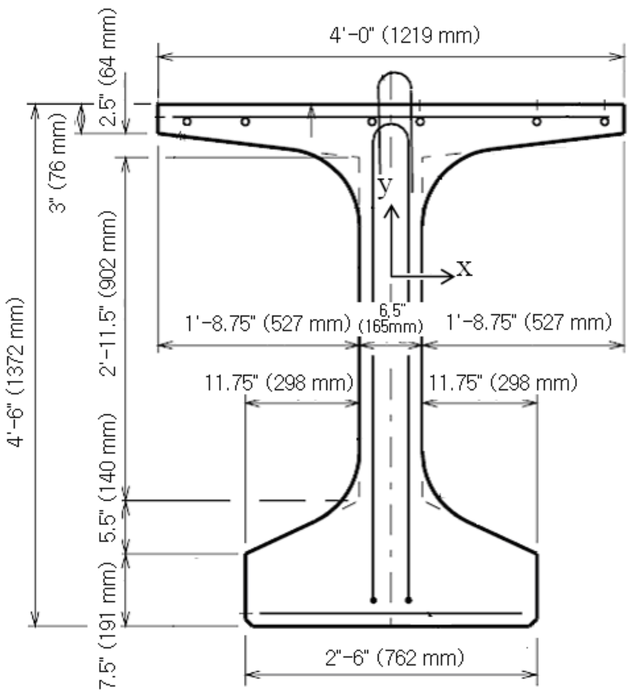

Fig. 4–Wisconsin 54W girder.

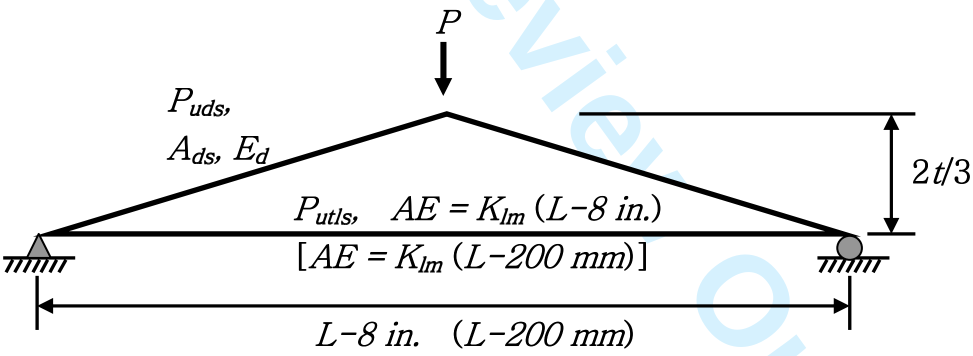

Fig. 5–Truss model for deck analysis.

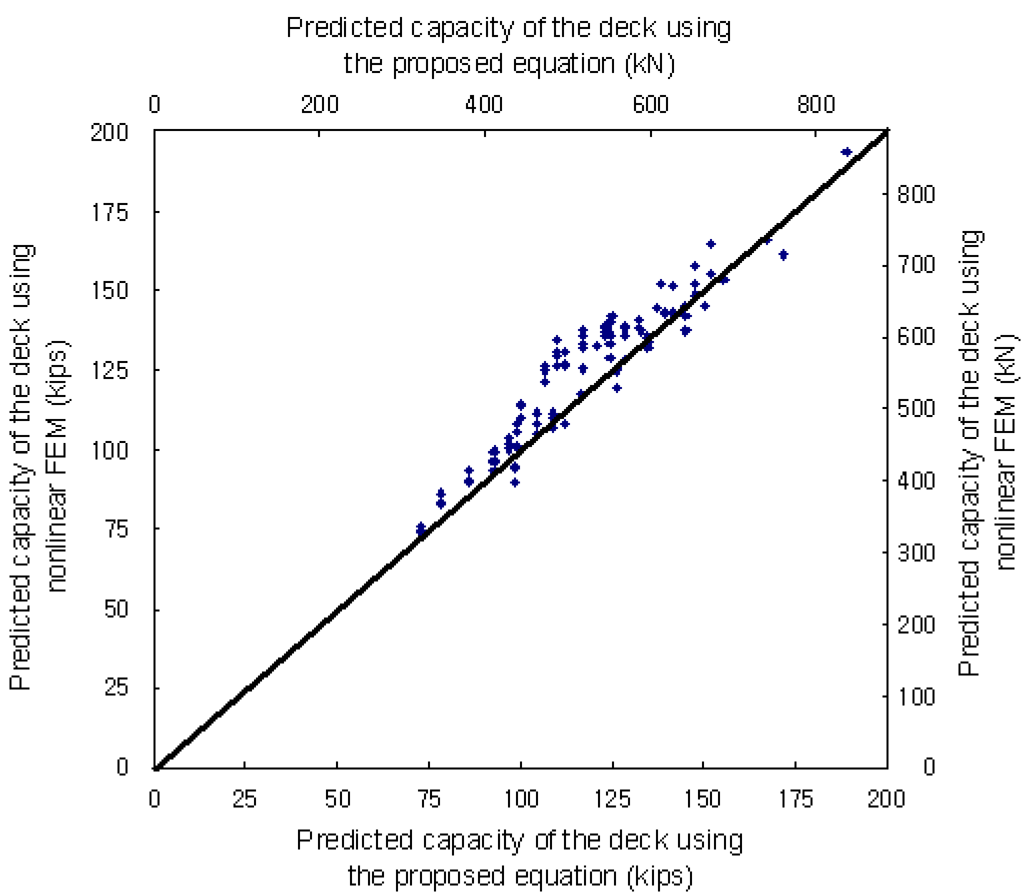

Fig. 6-Relationship between the predicted capacities of the deck using the alternative simplified design method and those of the deck using 99 nonlinear FEM analyses.