Ultimate behaviour of composite steel concrete bridges with rigid and simple diaphragm connections.

Mohamed Hamed. Soliman

University of Windsor

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Canada
ULTIMATE BEHAVIOUR OF COMPOSITE STEEL CONCRETE BRIDGES WITH RIGID AND SIMPLE DIAPHRAGM CONNECTIONS

BY

MOHAMED HAMED SOLIMAN

A Dissertation submitted to the Faculty of Graduate Studies through the Department of Civil and Environmental Engineering in Partial Fulfilment of the requirements for the Degree of Doctor of Philosophy at the University of Windsor

Windsor, Ontario, Canada
1992
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ISBN 0-315-78907-7
To my family
ABSTRACT

In the design of composite bridges, the North American practice either ignores or minimizes the contribution of transverse diaphragms to the transverse load distribution; this results, among other disadvantages, in heavier longitudinal steel girders than necessary as well as in the loss of headroom. In this thesis, parametric and experimental studies were carried out to investigate the influence of simply and rigidly connected diaphragms on the structural response of composite bridges.

In the analyses, both the yield line theory and the finite element method were used. Formulae based on yield line theory are developed to predict the ultimate load capacity of composite bridges. These formulae were based on both the parametric study as well as on laboratory test results on composite bridges. The developed formulae can be used to predict the ultimate load capacity or the required ultimate moment of resistance for the design of simple span and continuous span composite bridges. On the other hand, the finite element method
is used to conduct a parametric study of the response of composite bridges under elastic and post-elastic loading ranges.

The experimental study was conducted on four composite bridge models in two groups. Group A consists of two simply-supported single span bridge models and Group B consists of two continuous two-span bridge models. The test results were used to verify the assumption made in the theoretical analyses and to substantiate the results from such analyses.

Good correspondence between the theoretical and experimental results is found. Both the analytical and experimental results demonstrated that the response of rigidly connected diaphragm composite bridges is superior to that of the simply connected diaphragm bridges. Rigidly connected diaphragm composite bridges are stiffer and stronger than simply connected diaphragm bridges. Furthermore, they have a better ability to distribute the load over the bridge under both service and ultimate loadings. The concrete deck slab also plays an important role in the transverse load distribution over the bridge. The results also revealed that there is an optimum number of the rigidly connected diaphragms in composite bridges beyond which no improvement in the load distribution should be expected. Connection types, load locations, aspect ratio of the bridge, sectional moment
capacities, and the number of transverse diaphragms used influence the ultimate load capacity and the load distribution characteristics of composite bridges under service and ultimate loads. Comparison of the load distribution factors obtained from the finite element results and those of the Ontario Highway Bridge Design Code showed the code factors to be overly conservative.
ACKNOWLEDGEMENTS

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The author wishes to thank the laboratory technicians. Their help and enthusiasm during the preparation and testing of the bridge models are very much acknowledged.

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The author is also grateful to his parents for their support and continued encouragement.

Last but not least the author wishes to thank his wife Feroza for her great support throughout the course of this study. Without her understanding, encouragement and patience the completion of this work would not have been possible.
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NOMENCLATURE

$A_s$  Steel girder cross section area.

$A_r$  Reinforcing bar cross section area in a composite beam section.

$a$  Simulated truck axle length.

$D$  Load Distribution Factor.

$d_1, d_2$  Distance of AASHTO axle loads.

$[E]$  Element elastic constants matrix.

$f_c$  Concrete material compressive strength.

$f_y$  Steel material yield strength.


$L$  Bridge model length.

$l_c$  Composite Bridge positive yield line length.

$M$  Sectional bending moment.

$m$  Bending moment/Unit width.

$N$  Sectional normal force.

$[P]$  Load vector.

$P_c$  Calculated collapse load.

$Q$  AASHTO axle truck load.

$q$  Uniformly distributed dead load.

$S$  Composite beam spacing.

$W$ Bridge model width.

$W_e$ External work done.

$W_i$ Internal work done.

$w$ Nodal points deflection in the $z$ direction

$X$ Applied load location measured from the side of the bridge.

$\gamma$ Ratio of the remote axle truck loads to the maximum centre axle load.

$\Delta$ Incremental change notation.

$\Delta e^i$ Plastic strain increment.

$\Delta \sigma'$ Non-linear stress increment.

$\delta_c, \delta_w$ Collapse load and dead weight virtual work displacements.

$\delta_1, \delta_2$ Virtual work deflection at locations specified.

$\theta$ Virtual hinge rotation.

$\omega$ Uniformly distributed load.

**Subscript**

$c$ Concrete

$l$ Longitudinal

$n$ Negative

$p$ Positive

$r$ Reinforcement

$s$ Steel

$t$ Transverse

$u$ Ultimate
Chapter I

Introduction

1.1 General

During the last few decades, Composite Steel Concrete Structures have been used widely to form the basic superstructure for numerous bridges throughout the world. This was mainly due to the reduction in the structure's dead weight, better structural load carrying capacity, and a considerable reduction in the bridge depth. In the design of composite bridges, the North American Codes of practice (2,57) minimize the contribution of steel I-diaphragms in the transverse load distribution over the bridge "1". This results in heavier longitudinal steel girders than necessary, leading to an uneconomical structure. Some aspects of

1 Section 3-4.6 of OHBDC stated: "Diaphragms and bracing systems shall conform to the relevant requirements of sections 8, 9, 10 and 13, and shall be analyzed in accordance with the relevant requirements of Appendix A3-1."

A 3-1(f) stated: "Diaphragms and cross frames in shallow superstructure type bridges, the effect of diaphragms and cross frames between supports on the structural responses may be ignored."
this problem had been examined by several research workers (47,85,86). Results of these efforts demonstrated the influence of the diaphragms on reducing the main members' stresses by improving the load distribution over the bridge. The previously cited literature, however, assume bolted connection between the transverse diaphragms and the main longitudinal girders. Furthermore, no comparative study was carried out to examine the structural response of bridges with bolted and welded connections between the main girders and the transverse diaphragms.

In this thesis, the elastic and post-elastic structural responses of single-span and two-span continuous composite bridges with simply and rigidly connected diaphragms are examined. Furthermore, the effects of bridge connectivity type, continuity, aspect ratio, loading position and the presence of the concrete deck slab on the load-distribution characteristics of the bridge are also studied. The theoretical analyses are verified and substantiated by results from tests on simply supported and continuous composite bridge models.

In this work, the bolted-diaphragm connection which can only transfer shear between the transverse and longitudinal main girders will be referred to as Simply Connected Diaphragm or SCD. On the other hand, the welded-diaphragm
connection which can transfer both moment and shear will be referred to as Rigidly Connected Diaphragm or RCD.

1.2 Objectives and Scope

This study is an attempt to examine the behaviour of composite bridges in both elastic and post-elastic loading ranges with different types of diaphragm connections. In this respect, experimental and analytical efforts were carried out to examine both simply supported and continuous composite bridges. The following theoretical approaches were used in the analyses:

1- Finite Element Method.

2- Yield Line Theory.

The scope of this study is restricted to straight, short and medium, single and multiple span slab-on-girder composite bridges. The primary objectives of this study may be summarized as follows (see Figures 1.1 to 1.4):

1. To examine, theoretically and experimentally, the elastic behaviour of the steel grid structure (before casting the concrete slab) with RCD and SCD.

2. To study, theoretically and experimentally, the elastic behaviour of composite bridges with RCD and SCD.
3. To determine, using the finite element technique, the elastic load distribution factor D for composite bridges with RCD and SCD and compare it to the load distribution factor provided by the Ontario Highway Bridge Design Code (OHBDC).

4. To study, theoretically and experimentally, the post-elastic behaviour of composite bridges with RCD and SCD.

5. To investigate, using the finite element method, the change in the load distribution factor, D, as the load level is increased from service (elastic) to failure (post-elastic) for composite bridges with RCD and SCD.

6. To examine the influence of bridge continuity on the load distribution factor D within the elastic and ultimate loading ranges.

7. To develop, using the yield line theory, simple expressions to predict the collapse load of composite bridges with RCD and SCD under different loading conditions. In this respect, the experimental and finite element analytical results are used to provide the relevant information required to develop these expressions.

The experimental program consisted of testing four composite steel concrete bridge models in two groups: single-span and two-span continuous. The two tested bridge models in each group were identical except for the type of connection used
between the longitudinal main girders and the transverse diaphragms.

The contents of this thesis are as follows:

**Chapter II**, reviews the historical literature on composite bridges with an emphasis on their elastic and post-elastic (ultimate) behaviour and the influence of the secondary members on their response.

**Chapters III and IV**, focus on the analytical formulation using F.E.M. and the yield line theory, respectively.

**Chapter V**, deals with the experimental program with a full description of the designed models along with the loading instrumentations and measurement equipment.

**Chapter VI**, presents the discussion of the results obtained from the experimental tests and the theoretical analyses.

**Chapter VII**, contains the summary and conclusions of the study as well as recommendations for future research.
Fig. 1.1 Theoretical Study Layout [Section I: FEM]

* Load Distribution Factors at Service and Ultimate Limit State.
* Parametric Study on the elastic and Post-elastic behaviour of Composite Bridges.
* Yield Line Formulation.
Fig. 1.2 Theoretical Study Layout (Section II: Yield Line Method)
Fig. 1.4 Overall Layout of the Study
Chapter II

Literature Review

2.1 General

Composite steel concrete bridges have become one of the most widely used bridges throughout the world today. In the last forty years considerable volume of research efforts was directed to study this type of bridge superstructure. However, no research effort was carried out to examine the influence of the type of connection between the main girders and the transverse diaphragms on the load distribution and on the post-elastic response of the composite bridge.

2.2 Elastic Behaviour of Composite Bridges

The first notable experimental effort to study the elastic behaviour of composite steel-concrete bridges, was perhaps due to Newmark, Siess and Penman (55) in 1946. The work presented experimental results of tests conducted
on simple span right I-Beam bridge models. From the results, it was observed that no significant effect on either the strains in the slab or the deflection of the beams was produced by the interior diaphragms having a stiffness of 2.5 % relative to those of the main girders. This was followed by another report in 1948 by Newmark et al. (56), in which it was indicated that the transverse bridging is not practically effective except for loads at or close to the section where the transverse diaphragms are located. Siess et al. continued their research efforts to study continuous composite steel-concrete bridges (71). In this study, more attention was given to the influence of the studs on the load transfer between the concrete deck slab and the steel grid.

The positive contribution of the transverse diaphragms in composite steel concrete structures was reported by Lount in 1957 (47). Lount concluded that the addition of diaphragms to bridge structures may provide better transverse distribution, stiffened the bridge, reduced the vibrational and deflection effect, increased the safety of the structure, reduced hazards from fatigue loading and allowed crossing of very heavy individual loads in emergencies. In the same year,
White et al. (86) stated that,

"Effective lateral distribution of the test loads was obtained, with about 80% of lateral transfer of load being through the roadway slab and only 20% being carried by the diaphragm".

In 1959, Wei (85) indicated that the transverse diaphragms are most effective at mid span. However, Stevens and Gosbell (76) later argued that the intermediate transverse diaphragms do not significantly influence the load distribution over the bridge. In 1965, Carpenter and Magura (7) recognized the effect of the transverse diaphragms but only on the exterior girder design and not on the interior ones. Culham and Ghali (13) in 1977, confirmed that the transverse diaphragms do affect the transverse distribution of the live load. In 1979, Fisher, and Kostem (16) stated that,

"the effectiveness of diaphragms and cross framing in distributing live load ranges from 0 to 20% and depends upon the location of the measurements".

In 1981, Heins et al. (28) indicated that the cross bracing and diaphragms help in the live load distribution over the bridge. Furthermore, results presented by Kennedy and Soliman (37,72,73) showed the positive contribution of the diaphragms in the load distribution over the bridge. This contribution was even more pronounced when these diaphragms were welded to the main girders.

The elastic response of composite bridges was examined analytically using
different theoretical approaches. Guyon was perhaps the first to model a bridge deck into an orthotropic plate (19,20). He did not, however, consider the torsional rigidities of the members. Massonet later extended Guyon’s method to cover the torsional rigidities of the bridge elements (50,51,52). Because Poisson’s ratio was disregarded in Massonet’s formulation, the calculated transverse moment was not accurate (67). This was later considered by Rowe (66) in which he was able to achieve better results. Sanders et al (68) and Roesli et al (65) used a different technique to solve the orthotropic plate equation. They considered the applied load to be uniformly distributed over a small rectangular area.

Vitols, Clifton and Au (78), Chu and Krishnamoorthy (11,12), Jaeger and Hendry (32,29) and Heins and Loony (22,23,24) used the Fourier Theory in conjunction with the orthotropic equation to study the behaviour of composite bridges. Heins (25,26,27) also used the finite difference method to solve the orthotropic plate equation. The simplified modelling of bridge superstructures as an open grid structure was used by Lightfoot and Sawko in 1959 (45) and Livesly in 1964 (46). The bridge was analyzed as an equivalent open grillage system with equivalent properties in the two directions. Furthermore, the finite element method was developed and used to solve the orthotropic plate equation and further
expanded to solve stiffened plate problems. Today, the development of the digital computer has extensively increased the use of the finite element method (14,89) solving a wide range of structural problems.

2.3 Post-elastic Behaviour of Composite Bridges

Due to the rapid increase in the axle truck load in the last few decades, a very large number of overweight permits were issued in the United States. The overloading of composite bridges may induce longitudinal and transverse concrete deck slab cracking. This may in turn cause spalling of the reinforcement concrete covers and eventually lead to corrosion of the steel reinforcement. Furthermore, the cracked concrete deck may initiate the corrosion of the main steel girders under the slab due to the seepage of the deicing salt-laden water during the winter months. These may clearly reflect the danger of overloading existing composite bridges. It was reported that in 1966, over 700,000, overweight permits were issued in forty States. This rose to 1,250,000 permits in 1975 (21) and to over 2,000,000 in 1990. Subsequently, significant research efforts were directed towards the study of the overloading behaviour of composite bridges under unexpected heavy truck axle loads.
The yield line theory was used to predict the collapse load of composite bridges. Two major techniques were recognized when dealing with the yield line theory, these were: The virtual work approach which would lead to an upper bound solution, and the equilibrium approach which would lead to a lower bound solution (33,34). As early as 1969, Reddy and Hendry (64) used the virtual work approach to predict the collapse load of composite bridges. They did not, however, consider the influence of the transverse diaphragms. Similarly, many researchers, who studied this type of bridge superstructure, have used the virtual work approach to predict the structure’s ultimate load carrying capacity (36,69,75). In 1967, Pillai et al. modelled the actual structure as an open grid (58). Each grid element was given flexural and torsional capacities. The results obtained were in fair agreement with those obtained from experimental tests. Lowe and Flint (48), and Lash and Nagaraja (44) determined the collapse load for composite bridges using similar techniques. The minimum critical collapse load was selected from the calculation of the collapse loads of various assumed failure mechanisms. Fu (17), Kuo and Heins (43) and Bozler et al. (4) used the finite difference method to predict the collapse load deformation response of composite bridges. Their efforts were a continuation of the linear analysis of composite bridge models using the same
technique. The Finite Element method was also used to predict the ultimate load capacities of composite bridges. This was done using the first order theory; i.e. the small deflection theory and material non-linearities. Wegmuller et al. (80 - 84), Kostern et al. (38 - 42) and Gustafson et al. (18), have considered the non-linear material behaviour using the finite element method to analyze both composite steel-concrete and concrete-concrete bridges. In this approach, an incremental iterative technique was used. A layered beam plate model was introduced to describe the post-elastic response of the eccentrically stiffened plate. For simplicity the concrete slab material was modelled as an elastic-perfectly plastic material. These research efforts were concluded by developing computer programs to be used in the design of various types of bridges. These programs were developed as part of a research study sponsored by Pennsylvania Department of Transportation (38 - 42).

Recently, Razaqpur and Nofal (59 - 62) used the non-linear finite element technique in the analysis of composite bridges. Their comprehensive parametric efforts were directed to study the trend of the load distribution characteristics of concrete slab-on-steel girders composite bridges at the ultimate limit state. Several parameters, such as: bridge width, main girders spacing and number of girders,
loading truck position, relative rigidity of slab to that of girder, were examined.

Similar but less comprehensive studies were carried out by Fan (15), Heins and Kuo (30), Bakht et al. (3), and Cheung et al. (8 - 10). In their study, Razaqpur and Nofal did not deal with other important parameters such as: bridge continuity, skewness, curved bridges in plane and bridges with diaphragms.
Chapter III

Finite Element Approach

3.1 General

The finite element method provides a complete description for the structural behaviour within the elastic and post-elastic loading stage as well as a detailed theoretical simulation of both the steel and concrete structural components. In this chapter, a general description of the finite element approach adopted in the analysis is presented. The linear and material non-linear analytical concepts are explained. The theoretical modelling of the RCD and SCD composite bridge models is established.

3.2 Finite Element Analysis

During the last few decades, the use of the Finite Element Method in almost all the structural engineering problems became evident. This was due to
the rapid development and improvement of digital computers. The NASTRAN (54) and ABAQUS (1) Finite Element commercial codes were used extensively in this study. The two programs were used to study both the elastic and the post-elastic behaviour of composite bridges. Because of the existence of these programs it was not necessary to develop similar finite element computer codes for this study.

The development of the NASTRAN F.E. program (54) was initiated in 1966 under the sponsorship of the United State National Aeronautics and Space Administration. It is a general purpose finite element program capable of solving a large variety of engineering problems including linear and non-linear material and geometric ones. The program is well documented and quality assurance tested while continuously being enhanced with additional capabilities.

ABAQUS is another multi-purpose finite element code (1), developed and distributed by Electric Power Research Institute [EPRI] under the name ABAQUS-EPGEN. The code is used world-wide to estimate both linear and non-linear structural responses of power plant structural components due to accident and operating stresses. It is, however, further used in a much wider scale and covers all types of engineering problems. Similar to the NASTRAN program, the ABAQUS program is well documented and quality assurance tested and is continuously
being enhanced with additional capabilities. ABAQUS offers an extensive library of non-linear capabilities. Only a portion of these capabilities will be used in this study.

3.3 Scope of the Analysis

In this work, the goal of using the Finite Element Method of analysis is to study the elastic and the post-elastic behaviour of composite steel-concrete bridges. The problem was examined within the frame work of the following aspects:

1. Elastic analysis of the steel grid before casting the concrete deck slab.

2. Elastic analysis of the composite bridges.

3. Post-elastic analysis of the composite bridges.

The NASTRAN finite element code was used in the elastic analysis portion of this study. On the other hand, the ABAQUS finite element code was primarily used in the post-elastic portion of this study. This preference was mainly due to the better non-linear reinforced concrete modelling capabilities offered by the ABAQUS code.
Among the numerous types of elements available in the NASTRAN Finite Element Program directory, two element types were selected for use. These were the four-node plate element or QUAD4, as identified in the program element directory, and the two-node beam element or CBEAM. Description of the elements used is given in Table 3.1 (68). The program assigned a solution number for each type of analysis. In this study, Solution-3 for the static linear analysis type was used.

Similarly, in the ABAQUS program, among the numerous elements available in the program directory, two elements were selected for use (1). These were: the four-node shell element (S4R), and the three dimensional two-node beam element (B31). Descriptions of these elements are given in Table 3.2. The von Mises yield criterion was used for steel and the modified Mises criterion shown in Fig. 3.2 was used for concrete. Unlike NASTRAN, ABAQUS provides a "rebar" option to model the reinforcing steel bars. This option is used with standard metal plasticity models to describe the behaviour of the rebar material and is superimposed on a mesh of standard element types used to model the concrete. This allows the concrete behaviour to be considered independently of the rebar. Effects associated with the rebar/concrete interface, such as bond slip and dowel action were modelled
approximately by introducing tension stiffening to simulate the load transfer across cracks through the concrete around the rebar. Figure 3.3 represents the computational flow chart for a typical non-linear material static analysis case used in the code.

3.4 Basic Formulation

In the analysis, the matrix equation relating the generalized displacement and the loads may be expressed as:

\[ [K] [U] = [P] \quad 3.1 \]

where

\([K]\) = structure stiffness matrix

\([U]\) = displacement vector at the nodes

\([P]\) = applied loads vector at the nodes

The structure stiffness matrix \([K]\) is the assemblage of the element stiffness matrices \([k_i]\) as:

\[ [K] = \Sigma [k_i] \quad 3.2 \]

where

\([k_i]\) = element i stiffness matrix
and

\[ [k] = \int_v [B]^\varepsilon [E] [B] \, dv \tag{3.3} \]

in which the integration in Eq. 3.3 is performed over the volume \( v \) and;

\( [B] = \) strain displacement matrix

\( [E] = \) elasticity matrix

In the procedure to solve the incremental displacement, the governing equation, Eq. 3.1, is written in an incremental form as:

\[ [K] \Delta [U] = \Delta [P] \tag{3.4} \]

where the prefix notation \( \Delta \) refers to the rate of the parameter change with respect to the loading history. \( [K] \), represents the elastic-plastic stiffness matrix which is assembled from all the working elements as:

\[ [K] = \int_v [B]^\varepsilon [E] [B] \, dv \tag{3.5} \]

where

\( [E]_r \) = elasticity matrix expressed in the following relation between the stress and the total strain as;

\[ \Delta \sigma = E_T \Delta e^\varepsilon \tag{3.6} \]
where $\Delta \sigma$ and $\Delta \varepsilon$ represent the incremental stress and strain respectively which were generated during the load increment application.

For the plastic analysis the following general matrix relation is used:

$$[K] [\Delta u] = [\Delta P + \Delta Q]$$ \hfill (3.7)

where $\Delta Q$ is the plastic pseudo-load vector and it is the assemblage of all the element pseudo-load vectors $\Delta [q]$, where:

$$\Delta [q] = [K]^* \Delta [E]$$ \hfill (3.8)

with

$$[K]^* = \int_{\nu} [B]^T \varepsilon [E] \, d\nu$$ \hfill (3.9)

Hence,

$$\Delta [q] = \int_{\nu} [B]^T \varepsilon [E] \, \Delta \varepsilon \, d\nu$$ \hfill (3.10)

in which $\Delta \varepsilon$ is the plastic strain.

Alternatively, the pseudo-load vector can be written as:

$$\Delta [q] = \int_{\nu} [B]^T \Delta \varepsilon^T \, d\nu$$ \hfill (3.11)

where $\Delta \varepsilon^T$ represents the non-linear stress increment.
Either Eq. 3.4 or 3.7 can be used to solve for an increment of displacement associated with the applied increment of load. The displacement solution is used to obtain the total strain increments. The estimate of any loading increment is based on the results of the current loading step. The solution may be described as a sequence of linear solutions in which either the stiffness update or the pseudo-load is introduced in order to maintain equilibrium. If the equilibrium cannot be maintained with the least set loading increment, the structure is considered to be unable to carry any further load and therefore has reached its loading capacity.

3.4.1 Assumptions

In the analysis, the following assumptions were made:

1- First order deformation theory is valid for short term monotonically applied small load increments.

2- Complete interaction between the concrete slab and the longitudinal steel girders is present.

3- The steel material has an elastic perfectly plastic behaviour in biaxial
compression and tension.

4- For simplicity, the concrete material has an elastic perfectly plastic behaviour in biaxial compression, and elastic brittle behaviour in tension, as shown in Fig. 3.4.

3.5 **Elastic Analysis of Steel Grid Before Casting the Concrete Deck Slab**

This analysis was aimed to study the elastic behaviour of the steel grid with simple (bolted) and rigid (welded) connections between the main girders and the transverse diaphragms. The analysis was conducted on the tested single span and two span continuous bridge models.

3.5.1 **Rigidly Connected Diaphragm Bridge Models**

The two tested RCD steel grids were modelled, as shown in Figs 3.5(b) and 3.6(b), using the prescribed beam element. As it was explained earlier, the RCD type is capable of transferring both moment and shear between the main girders and the diaphragms. Three degrees of freedom were assigned to each grid nodal point. These were; the vertical displacement, w, and the nodal point rotations, Θx and Θy. Only the vertical displacement, w, was restrained for all nodal points located at the support lines.
3.5.2 Simply Connected Diaphragm Bridge Models

In the laboratory SCD bridge models, two bolts were used to connect the diaphragm beams to the main girders through back to back angles as shown in Figure 3.7. The nature of this type of connection cannot be theoretically modeled as a full hinge between the two members. The two bolts are expected to present a small flexural restraint in the connection. This means that the connection is capable of transferring a small magnitude of moment between the diaphragms and the main girders in addition to its shear transfer capability. The two tested bridge models were therefore, theoretically modelled as shown in Figs. 3.5(c), 3.6(c) and 3.7. Each diaphragm was modelled using three beam elements, elements one and three having relatively smaller flexural rigidities to simulate the bolted connections and element two having a full sectional rigidity. The rigidities of elements one and three were estimated to be about 5% of the full flexural rigidity of the member. The estimate was based on the results obtained from the laboratory tests. Similar to the welded bridge model, each of the beam element was given three degrees of freedom at each nodal point except over the supports where the vertical displacement, w, was restrained.
3.6 **Elastic Analysis of the Composite Bridge Models**

Based on the convergence study as well as on the required execution time, it was decided to use the following number of elements for the composite bridge models:

1. For the single-span bridge models: 64 shell elements and, 80 and 100 beam elements in the RCD and SCD bridge models, respectively.

2. For the continuous two-span bridge models: 96 shell elements and, 59 and 113 beam elements in the RCD and SCD bridge models, respectively.

Typical discretizations of the deck slab in the four models are shown in Figs. 3.5(a) and 3.6(a). The prescribed quadrilateral element was used to model the concrete deck slab. Each of the element's four nodal points were given three degrees of freedom, $\Theta_x$, $\Theta_y$ and $w$. The last degree of freedom, $w$, was suppressed for nodal points located at the support line. The reference coordinate $(XY)$ coincided with the deck slab centroid as shown in Fig. 3.8. The steel grid discretization was similar to that previously outlined in Section 3.5 with the beam's reference coordinate system $(XY)$ shifted from the deck slab coordinate system, as shown in Fig. 3.8. Only the elastic behaviour of both the steel grid and
the concrete slab was considered here. The concrete and steel material properties were obtained from the sample tests as well as from the supplier information sheet, Appendix A-8. Figure 3.9 presents the average test results of the stress-strain relationship for the steel beam material used in the tested bridge models. The figure also shows the adopted elastic perfectly plastic criterion used in the analysis.

3.7 Post-elastic Analysis of the Composite Bridges Models

In this work, the same finite element mesh described in sections 3.5 and 3.6 was used in the post-elastic analysis of the composite bridge models. In the analysis, an incremental loading approach (step-by-step technique) was employed to obtain the post-elastic behaviour of composite bridges. The material yield criterion associated with the solution for the ABAQUS general purpose finite element program was discussed in section 3.4 along with the outline of the elasto-plastic theoretical approach used in the analysis.
3.8 Parametric Study

In addition to the analytical work carried out on the four tested composite bridge models, a parametric study was also conducted on prototype composite bridges with rigid and simple diaphragm connections. Bridges with cross-sections shown in Fig. 3.10, were selected from the Standard Plans for Highway Bridges (73). The finite element models layout used for the composite bridge prototypes are shown in Figs. 3.11 and 3.12.

The goals of the parametric study were:

1. To investigate the influence of following variables on the load distribution characteristics of the bridge and on the service and ultimate loading responses: (i) the bridge aspect ratio, (ii) girders spacing, number of diaphragms, (iii) ratio of negative to positive moments of resistance, (iv) bridge continuity, and (v) types of connections between the main girders and the transverse diaphragms

2. To use the collapse load results obtained in (1) in the development of collapse load expressions for composite bridges under different loading conditions using the yield line theory.

3. To investigate the ratio of the first girder yield load to the final collapse load for different composite bridges under a variety of loading conditions.
3.9 Load Distribution Factor D

One of the main objectives of this study was to investigate the changes in the transverse load distribution factors, D, as the load on the bridge is increased up to the collapse of the bridge. In this respect, a wide range of prototype composite bridges subjected to different loading conditions was examined. The influence of the longitudinal main girders and transverse diaphragm connection type on the load distribution response was also investigated. The concept of the load distribution factor D and its use in bridge design and analysis have been explained before in the OHBDC (57).

In Fig. 3.13, let \( M_y \) = total live load moment that the composite beam section under a vehicle is required to resist; \( M_{x_{\text{max}}} \) = intensity of the maximum longitudinal moment under the vehicle per unit width; and \( S \) = girder spacing. Hence, the moment design of the girder requires that:

\[
M_y = S \cdot M_{x_{\text{max}}} \tag{3.12}
\]

A load distribution factor \( D \) can be defined as:

\[
D = \frac{M}{M_{x_{\text{max}}}} \tag{3.13}
\]

in which \( M \) is the total longitudinal moment due to one line of wheel loads at the section under consideration. \( D \) will have a unit of length. Thus, combining the
above two equations yields:

\[ M_y = (S/D) M \]  \hspace{1cm} \text{(3.14)}

Hence, if \( D \) is known, the maximum live load moment to be resisted by a steel girder can be determined. If \( D \) is unknown, then it can be determined from equation 3.13, since \( M \) can be calculated from statics, while \( M_{x \text{ max}} \) is found from the structural analysis of the bridge. In this study the change in the load distribution factor, \( D \), was examined for a group of composite bridges as the load was increased from service (elastic) to ultimate (collapse) load.
Fig. 3.1 Von Mises Yield Criterion
Fig. 3.2  Concrete Material Yield Criterion
Fig. 3.3 Finite Element Post-elastic Analysis Flow Chart
Fig. 3.4  Idealized Concrete Material Stress–Strain Relation Used in the Analysis
Fig. 3.5(a) Simply Supported Concrete Slab Finite Element Mesh.
Fig. 3.5(b) Rigid Connections Steel Grid Finite Element Mesh "Simply Supported Case"
Fig. 3.5(c) Simple Connections Steel Grid Finite Element Mesh "simply Supported Case"
Fig. 3.6(a) Continuous Span Concrete Slab Model Finite Element Mesh
Fig. 3.8(c) Simple Connections Steel Grid Finite Element Mesh "Continuous Span Case"
a) Actual Simple Connection Details

b) Theoretical Modelling of Simple connection.

Fig. 3.7 Details of Theoretical Simulation of Simple Connection
Fig. 3.8 Typical Cross Section of Composite Steel Concrete Bridge
Fig. 3.9 Stress Strain Relationship for the steel girder material

Note: 1.00 ksi = 6.895 MPa
Fig. 3.10
Geometries of Prototype Composite Bridges used in the Parametric Study

(1 ft = 0.3048 m; 1 inch = 25.4 mm)
Fig. 3.11  Finite Element Mesh layout for the 2–Lane Composite Bridge Prototypes

Note:
The Steel Grid Structures in the RCD and SCD Bridges were modelled Similar to those Shown in Figs. 3.5 (b,c) and 3.6 (b,c)
Fig. 3.12 Finite Element Mesh Layout for the 3 and 4 Lane Composite Bridge Prototypes.
Fig. 3.13  Load Distribution Factor in Composite Bridges
### TABLE 3.1 NASTRAN ELEMENTS DESCRIPTION

**QUAD4**

- Four Node Element
- Force Components; $F_x, F_y, F_z$
  - $M_x, M_y, M_{xy}$
  - $Q_x, Q_y$
- Stress Components; $S_x, S_y, S_{xy}$ at the centre
- Displacement Components; $U_x, U_y, U_z$ and $\Theta_x, \Theta_y$ and no rotation normal to the element.
- Nonlinear Capabilities; Geometric and Material Nonlinearities
  - No REBAR available

**BEAM**

- Two Node Element
- Force Components;
  - Axial Force $P$
  - Total Torque $T$
  - Warping Torque $T_w$
  - Bending moment $M_1, M_2$
  - Shearing Force $V_1, V_2$
- Displacement Components, $U_x, U_y, U_z$
  - $\Theta_x, \Theta_y, \Theta_z$
- Nonlinear Capabilities; Geometric and Material
TABLE 3.2 ABAQUS ELEMENTS DESCRIPTION

**S4R**

- Four Node Element
- Force Components; $F_x, F_y, F_z$
  $M_x, M_y, M_{xy}$
  $Q_x, Q_y$
- Stress Components; $S_x, S_y, S_{xy}$ at the Top, Bottom and Centre Plane.
- Displacement Components; $U_x, U_y, U_z$ and $\Theta_x, \Theta_y$ and no rotation normal to the element.
- Nonlinear Capabilities; Geometric and Material Nonlinearities

REBAR option available in two directions

**B31**

- Two Node Element
- Force Components; Axial Force $P$
  Total Torque $T$
  Warping Torque $Tw$
  Bending moment $M_1, M_2$
  Shearing Force $V_1, V_2$
- Displacement Components, $U_x, U_y, U_z$
  $\Theta_x, \Theta_y, \Theta_z$

Nonlinear Capabilities; Geometric and Material
Chapter IV

Yield Line Approach

4.1 General

One of the aims of any structural design is to come up with a structure that will carry the working loads safely at minimum cost. The yield line theory has been used successfully to predict the collapse load of plate-like structures. Its success, however, is mainly attributed to the designer’s experience in predicting the yield-line pattern of failure. Predicting the collapse load may be very useful in finding the true safety factor and failure pattern of the structure under the most critical loading conditions. It is well known that an analysis based on the yield-line approach theoretically would lead to an upper bound solution to the collapse load; however, such an approach neglects some enhancing factors resulting in a collapse load which, in practice, is lower than the actual collapse load. The prediction of the
most probable yield-line patterns of failure for composite bridges treated herein was based on the laboratory test results, previous experience and the results from parametric study using the finite element technique.

4.2 Assumptions

In the yield line analysis the following assumptions were made:

1. At failure, the structure is divided into few plane segments bounded by the yield lines along which the ultimate moment of resistance may be developed.

2. Yield lines are in general straight.

3. Yield lines pass through the intersection of the axes of rotation.

4. The ultimate load of a composite bridge is reached when sufficient yield lines have been formed to enable the composite bridge to deform as a mechanism.

The ultimate collapse load is derived by equating the external and internal virtual work, $W_e$ and $N_r$, respectively. The external virtual work is generated by the work done by the external load on the structure at collapse, $P_e$, as well as by the uniformly distributed dead load, $q$, through a virtual displacement, $\delta$. The internal virtual work is done by the moment of resistance, $M$, along yield lines of failure.
Thus, from the principle of virtual work:

\[ W_s = W_l \]

This may be expressed as:

\[ \sum \int \int w \delta dx dy = \sum (M \Theta) \]  \hspace{1cm} (4.1)

where

\[ w = \text{Load/unit area at the point } x,y \text{ over the bridge.} \]

\[ \Theta = \text{The normal rotation of the yield-line.} \]

In order to allow for the rotation of the failed segments, the following were assumed:

1. Composite sections are under-reinforced.
2. Shear, punching or bond failure are precluded.
3. The concrete material has an elastic perfectly plastic behaviour in biaxial compression, and an elastic brittle behaviour in tension as shown in Fig. 3.4
4. Strain hardening of steel is neglected, with steel having elastic-perfectly plastic characteristics as shown in Fig. 3.9.

### 4.3 Yield Line Theory and Composite Bridges

The composite bridge was divided into composite beam elements in both
longitudinal and transverse directions. The ultimate flexural moment (positive and negative) and torsional moment capacities were determined. These were then transferred to equivalent moments per unit width. The composite beams' sectional moment capacities calculations are presented in Appendix A-1.

In the formulation of the collapse load, the internal work-done included the following contributions:

1. Positive moment contribution;

2. Negative moment contribution;

3. Torsional moment contribution.

The above contributions were considered in the longitudinal and transverse directions, as well as over the intermediate support, as applicable.

On the other hand the external work done included the following contributions:

1. The applied external load;

2. The structure's dead weight.

In this study, typical loading cases were considered, namely, eccentric and concentric loading cases. Based on the finite element results, test results from the bridge models and previously reported results, the assumed failure patterns are shown in Figs. 4.1 to 4.9. The figures show the failure patterns for composite
bridges with single span and continuous two-span subjected to eccentric and concentric loadings. It should be noted that these failure patterns exclude load panel failure mechanisms since these are unlikely to occur in view of the codes requirements for minimum slab thickness in composite bridges.

4.4 General Formulation

From Eq. 4.1,

The sum of the external work done = The sum of the internal work done.

This may now be re-expressed as:

\[ \sum P_c \delta_c + \sum q \delta_q = \sum M_{pl} \theta_{pl} + \sum M_{pc} \theta_{pc} + \sum M_{nc} \theta_{nc} + \sum M_{nl} \theta_{nl} + \sum M_c \theta_c + \sum M_s \theta_s \]

4.2

in which

\( P_c \) = External applied collapse load;

\( q \) = Uniformly distributed dead load;

\( M_{pl}, M_{pc} \) = Positive moment capacity in the longitudinal and transverse directions, respectively;

\( M_s \) = Bridge deck torsional moment capacity.

\( M_{nl}, M_{nc} \) = Negative moment capacity in the longitudinal and transverse directions, respectively;
\( M_i \) = Negative moment capacity over the intermediate supports;

\( \delta_c \) = Virtual deflection under the applied load;

\( \delta_q \) = Virtual deflection of the structure's dead weight;

\( \theta_{pl}, \theta_{pt} \) = Virtual longitudinal and transversal yield line rotations, respectively, associated with positive moment of resistance;

\( \theta_{nl}, \theta_{nt} \) = Virtual longitudinal and transversal yield line rotations, respectively, associated with negative moment of resistance; and,

\( \theta_t \) = Virtual torsional rotation of the bridge deck, segment (abcg) in Fig. 4.2.

\( \theta_s \) = Virtual intermediate support yield line rotation, associated with negative moment of resistance.

From Eq. 4.2:

\[
\sum P_c \delta_c = \sum M_{pl} \theta_{pl} + \sum M_{pc} \theta_{pc} + \sum M_{nc} \theta_{nc} + \sum M_{nt} \theta_{nt} + \sum M_c \theta_c + \sum M_s \theta_s - \sum q \delta_s
\]

4.3

The contribution of each term that appears in the equation will now be expressed in detail under both eccentric and concentric load cases.
4.5 Collapse Load Under Eccentric Load Case

4.5.1 General

Let the maximum virtual deflection be equal to unity, i.e. (δ=1) as shown in Fig. 4.1.

1. Internal Work done by the moment along the Positive Yield Line:

From Fig. 4.2:

$$\sum M_{pl} \theta_{pl} = \sum (M_{pl} \theta_{pl})_S + \sum (M_{pl} \theta_{pl})_{l_c-S} \quad 4.4$$

in which the bracketed subscripts (S) and (l_c-S) refer to the locations where the internal work is calculated, and:

- \(l_c\) = Length of the positive moment yield line.
- \(S\) = Main girder spacing.

Thus,

$$\sum (M_{pl} \theta_{pl})_S = S \ m_{pl} (\phi \ sin \ \alpha) \quad 4.5(a)$$

which may be re-expressed as:

$$\sum (M_{pl} \theta_{pl})_S = 2 \ S^2 \ \frac{m_{pl}}{l_c L} \quad 4.5(b)$$

If we assume a longitudinal finite strip dy at a distance y from the edge of the bridge as shown in Fig. 4.2, then:
\[ \sum (M_{pl} \theta_{pl})_{lc-y} = \int_{0}^{l_{c}-y} m_{pl} \theta_{y} \, dy \]  \hspace{1cm} 4.6(a)

from which

\[ \sum (M_{pl} \theta_{pl})_{lc-y} = m_{pl} \frac{1}{L_{c}L} \int_{0}^{l_{c}-y} (l_{c} - y) \, dy \]  \hspace{1cm} 4.6(b)

this leads to

\[ \sum (M_{pl} \theta_{pl})_{lc-y} = m_{pl} \frac{1}{L_{c}L} (l_{c}^2 - S^2) \]  \hspace{1cm} 4.6(c)

for the two sides of the yield line:

\[ \sum M_{pl} \theta_{pl} = 2 m_{pl} \frac{(l_{c}^2 + S^2)}{L_{c}L} \]  \hspace{1cm} 4.7

2. Internal Work done by the moment along the Negative Yield Line:

From Fig. 4.2:

\[ \sum M_{nc} \theta_{nc} + \sum M_{nl} \theta_{nl} = 2 \left( m_{nl} S \phi \sin \alpha + \frac{L}{2} \phi \cos \alpha \right) \]  \hspace{1cm} 4.8(a)

from which

\[ \sum M_{nc} \theta_{nc} + \sum M_{nl} \theta_{nl} = 2 \left( m_{nl} S \frac{1}{l_{c} \cos \alpha} \sin \alpha + \frac{m_{nc} L}{2 l_{c}} \right) \]  \hspace{1cm} 4.8(b)
Thus, the total work done is:

\[ \sum M_{nt} \theta_{nt} + \sum M_{nl} \theta_{nl} = \frac{1}{I_c} \left( \frac{m_{nt} L^2}{L} + 4 m_{nl} S^2 \right) \]  \hfill (4.9)

3. **Internal Work done by the moment along the Negative Yield Line at the intermediate support:**

From Fig. 4.2:

\[ \sum (M_y \theta_y)_{I_c-S} = \int_0^{I_c-S} m_y \theta_y \, dy \]  \hfill (4.10)

from which

\[ \sum M_y \theta_y = \frac{m_y}{L \cdot I_c} (I_c^2 - S^2) \]  \hfill (4.11)

4. **Internal Work done by the torsional moment of segment (abcg) shown in Fig. 4.2:**

From Fig. 4.2:

\[ \sum M_{tc} \theta_t = m_{c}\left(\frac{2.0}{I_c}\right) I_c \]  \hfill (4.12)

5. **External Work Contribution:**

i. **Due to Uniformly Distributed Dead Load:**

If one assumes a transverse finite strip \( dx \) at a distance \( x \) from the support as shown in Fig. 4.3, then:
\[
\sum q \cdot \delta_q = 2\int_0^{L/2} q \left[ \frac{1}{c - S} \right] \left( \frac{1}{2} \cdot \frac{2S}{L} \right) x \, dx \\
+ 2\int_0^{L/2} q \left( \frac{x}{L} \right) S \left( \frac{1}{2} \cdot \frac{S}{L} \right) \frac{2}{L} x \, dx
\]

Hence,

\[
\sum q \cdot \delta_q = 2q \left[ \frac{1}{c - S} \right] \left( \frac{x^2}{2} \right) \\
+ 2q \left( \frac{2x^3}{3} \right) \left( \frac{S^2}{L^3} \right)
\]

from which

\[
\text{Dead Load Work Done} = \frac{qL}{12c} (3l_c^2 - S^2)
\]

ii. Due to External Applied Load:

From Fig. 4.4:

\[
\sum P_c \delta_c = P_c \left[ \frac{1}{c} \left( l_c - x_1 \right) + \frac{1}{c} \left( l_c - x_2 \right) \right]
\]

which may re-expressed as:

\[
\sum P_c \delta_c = P_c \frac{2l_c - x_1 - x_2}{l_c}
\]

where \( X_1, X_2 \) = Concentrated load locations, Fig. 4.4.

For simplicity, two equal loads were applied to the side of the model. These loads were to simulate single axle truck load. They were also assumed to remain
equal and at a specified location throughout the loading history until complete
bridge collapse is achieved.

4.5.2 Collapse Load Formulae for RCD and SCD Bridges
associated with Eccentric Loadings

The following two expressions may be derived by substituting equations 4.4
through 4.16 into equation 4.2:

1. For Simply Supported Bridge:

\[ P_c = \frac{1}{2l_c-x_1-x_2} \left[ \frac{(2m_{pl}(l_c^2+S^2)+4m_{nl}S^2+m_{ce}L^2+2m_{ce}L)}{L} - \frac{QL}{12} (3l_c^2-S^2) \right] \]

4.17

2. For Continuous Two-Span Bridge:

\[ P_c = Eq. 4.16 + \frac{1}{2l_c-x_1-x_2} \left( \frac{m_s}{L} (l_c^2-S^2) \right) \]

4.18

It is important to note that both the expressions for \( m_{nl} \) and \( m_{nl} \) for SCD type bridge
are calculated differently from those calculated for RCD type bridge. These expressions
are explained in detail in Table 6.4.

4.6 Collapse Load Under Concentric Load Case

Based on the results from the analytical and experimental studies as well
as on results reported by other researchers, the yield line patterns of failure for
the single and two span continuous bridges are assumed, see Figs. 4.6 and 4.7.
It should be noted that these failure patterns are appropriate for relatively wide composite bridges as studied herein. Furthermore, these failure patterns exclude load punching failures since these are unlikely to occur in view of the codes requirements for minimum slab thickness in composite bridges. Under this loading category, the following two types of concentric loadings were considered: the two-wheel loads and the four wheel loads as shown in Fig. 4.5. In here, the collapse load expressions are first derived in terms of central and midspan-side deflections, \( \delta_1 \) and \( \delta_2 \), respectively, shown in Fig. 4.6. Later, some assigned values for \( \delta_1 \) and \( \delta_2 \) are given based on the achieved parametric and experimental studies results carried in here.

It was observed from the tests that the failure of the RCD type bridge occurs in two steps as shown in Fig. 4.8. First, the bridge deflects uniformly at the centre and across the width resulting in a positive yield line across. This is then followed by a further displacement at the centre point, associated with the development of additional diagonal negative and longitudinal positive yield lines, shown in Fig. 4.9.
4.6.1 General

1. Internal Work Done by the moment along the positive Yield Line (aba) shown in Figs. 4.6 and 4.7:

i. Due to a uniform displacement:

\[ \sum (M_{pl} \theta_{pl})_1 = 4 \left( \frac{\delta_1}{L} \right) \bar{W} m_{pl} \]

ii. Due to an additional displacement at the bridge centre:

\[ \sum (M_{pl} \bar{\theta}_{pl})_2 = 4 \left( \frac{\delta_2 - \delta_1}{L} \right) \bar{W} m_{pl} \]

2. Internal Work done by the moment along the positive yield line (bc) shown in Figs 4.6 and 4.7:

\[ \sum M_{pc} \theta_{pc} = 4 \ m_{pc} \frac{L}{\bar{w}} (\delta_1 - \delta_2) \]

3. Internal Work Done by the moment along the Negative Yield Line at the intermediate support:

\[ \sum M_s \theta_s = \frac{(\delta_2)}{L} m_s \bar{w} \]

4. Internal Work Done by the moment along the Negative Yield Line (bc) shown in Figs 4.6 and 4.7:

From Fig. 4.9:
\[
\sum M_{n1}\theta_{n1} - \sum M_{n2}\theta_{n2} = 4 \left[ \frac{m_{n1}}{2} \left( \phi \cos \alpha \right) + m_{n2} \frac{L}{2} \left( \phi \sin \alpha \right) \right]
\]

This may be expressed as:

\[
\Sigma M_{n1}\theta_{n1} + \Sigma M_{n2}\theta_{n2} = 4 \left( \delta_1 - \delta_2 \right) \left( \frac{W}{L} m_{n1} + \frac{L}{W} m_{n2} \right)
\]

5. External Work Done:

i. Due to Bridge Dead Load

External Work Done by the Dead Load = \( W_{e1} + W_{e2} \)

From Fig. 4.9, this may be expressed as:

\[
\text{Dead Load Work Done} = \frac{\delta_2}{2} qLW + 4 \int_0^{\frac{W}{2}} q(\delta_1 - \delta_2) \frac{x^2}{W} \frac{L}{W^2} \, dx
\]

4.25(a)

Thus:

\[
\text{Dead Load Work Done} = \frac{\delta_2}{2} qLW + \frac{(\delta_1 - \delta_2)}{6} qLW
\]

4.25(b)

ii. Due to external applied load

- For Single Truck Load Case shown in Fig. 4.6:

\[
\sum P_c \delta_c = 2P_c \left( \delta_1 + \frac{a}{W} (\delta_2 - \delta_1) \right)
\]

4.26
4.27 \sum P_c \delta_c = 4 P_c \left( \frac{\delta_1 + \delta_2}{2} \right)

4.6.2 Collapse Load Formulae for RCD Bridge associated with Concentric Loadings

Based on the achieved parametric and experimental results and from Eqs. 4.19 to 4.27 the final collapse load expressions for RCD bridges associated with concentric loadings are:

1. For Two Wheel Load with $\delta_i=1.0$ (Unity) and $\delta_c=0.75$

   - Single Span Bridge:
     \[
P_c = \frac{4 \frac{m_{pl}}{L} \frac{W}{L} + m_{nl} \frac{W}{L} + m_{nc} \frac{L}{W} + m_{pt} \frac{L}{W} - \frac{5}{12} Q \frac{W}{L}}{2(1 - 0.25 \frac{A}{W})}
     \]

   - Continuous Two Span Bridge:
     \[
P_c = \text{Eq.} 4.38 + \frac{1.5 \frac{m_s}{L} \frac{W}{L}}{2(1 - 0.25 \frac{A}{W})}
     \]

2. For Four Wheel Load with $\delta_i=1.0$ (unity) and $\delta_c=0.875$

   - Single Span Bridge:
     \[
P_c = \frac{4 \frac{m_{pl}}{L} \frac{W}{L} + 0.5 \frac{m_{nl}}{L} \frac{W}{L} + 0.5 \frac{m_{nc}}{W} \frac{L}{W} + m_{pt} \frac{L}{2W} - 0.4583 Q \frac{W}{L}}{3.75}
     \]
- Continuous Two Span Bridge:

\[ P_c = Eq. 4.40 + \frac{1.75 \, m_s \, \frac{W}{L}}{3.75} \quad 4.31 \]

4.6.3 Collapse Load Formulae for SCD Bridge associated with Concentric Loadings

Based on the achieved parametric and experimental results and from Eqs. 4.19 to 4.27, the final collapse load expressions for SCD bridge type associated with concentric loadings are:

1. For Two Wheel Load with \( \delta_i = 1.0 \) (Unity) and \( \delta_s = 0.25 \)

- Single Span Bridge:

\[ P_c = \frac{4 \, m_{pl} \, \frac{W}{L} + 3 \, m_{nl} \, \frac{W}{L} + 3 \, m_{nc} \, \frac{L}{W} + 3 \, m_{pc} \, \frac{L}{W} - 0.25 \, q \, W \, L}{2 \left(1 - 0.75 \, \frac{a}{W}\right)} \quad 4.32 \]

- Continuous Two Span Bridge:

\[ P_c = Eq. 4.38 + \frac{0.5 \, m_s \, \frac{W}{L}}{2 \left(1 - 0.75 \, \frac{a}{W}\right)} \quad 4.33 \]

2. For Four Wheel Load with \( \delta_i = 1.0 \) (unity) and \( \delta_s = 0.75 \)

- Single Span Bridge:

\[ P_c = \frac{4 \, m_{pl} \, \frac{W}{L} + m_{nl} \, \frac{W}{L} + m_{nc} \, \frac{L}{W} + m_{pc} \, \frac{L}{W} - 0.4167 \, q \, W \, L}{3.50} \quad 4.34 \]
- **Continuous Two Span Bridge:**

\[
P_c = Eq. 4.40 \times \frac{1.50 m_s \frac{W}{L}}{3.50}
\]

4.35

### 4.7 Simple Beam Collapse Mechanism

In here, the simple beam failure mechanism for both single span and continuous two span bridges was also considered. Figure 4.10 presents the collapse modes in the two-cases along with the derived failure load expressions. For relatively long span composite bridges with \(L/W \geq 2\), it is noted that for concentrically loaded composite bridges, the collapse load calculated for an equivalent beam system is expected to be lower than the actual RCD bridge collapse load and much lower than that of SCD bridge. On the other hand, for an eccentrically loaded bridge, the collapse load calculated for an equivalent composite beam design is expected to be higher than the actual RCD bridge collapse load and much higher than that of SCD bridge.

### 4.8 Truck Loading Treatment

In this study, the derived yield line formulations for the minimum collapse load of a composite bridge are for one or two pairs of wheel loads. However, in practice, the design is based on truck loadings given by relevant codes. To deal
with actual bridge loadings composed of a group of wheel loads, such as, the OHBDC loading (Ontario Highway Bridge Design Code) (57), and AASHTO HS20 loading (American Association of State Highway and Transportation Officials 1983) (2), two expressions are derived in here for a single concentrated load equivalent to a group of wheel loads. Figure 4.11 shows longitudinal sections of a 2-span continuous composite bridge subjected to the following loadings:

a. OHBDC truck loading causing a yield line to form with the heaviest axle acting along the transverse centre line passing through O.

b. AASHTO HS20 truck loading causing a yield line to form with the heaviest axle acting along the transverse centre line passing through O.

c. Concentrated wheel load, \( P \), at the centre of one span through which a yield line passes.

4.8.1 OHBDC Truck Load

Since the internal work generated by the single concentrated load and the equivalent OHBDC truck loading are equal, their external work are also equal. Thus:

\[
P = \left[ 1 + \gamma_1 \left( 1 - \frac{2d_1}{L} \right) + \gamma_2 \left( 1 - \frac{2d_2}{L} \right) + \gamma_3 \left( 1 - \frac{2d_3}{L} \right) + \ldots \right] Q
\]

4.36
in which $d_1$, $d_2$, $d_3$ and $d_4$ are the distance of the axle loads $\gamma_1 Q$, $\gamma_2 Q$, $\gamma_3 Q$ and $\gamma_4 Q$, respectively, from the axle load $Q$. For an OHBDC truck, $\gamma_1 = \gamma_2 = 0.7$, $\gamma_3 = 0.3$ and $\gamma_4 = 0.8$. Therefore Eq. 4.35 reduces to:

$$P = \left[ 3.5 - \frac{1.4 (d_4 + d_3) + 0.6 d_3 + 1.6 d_4}{L} \right] Q \quad 4.37$$

Applying the live load factor ($\beta_z$) to $C$ (57), the factored equivalent wheel load $P_c$ becomes, from Eq. 4.36:

$$P_c = (\beta_z) P \quad 4.38$$

Thus, in design, given the load on the bridge, the factored collapse load $P_c$ can be derived from Eqs. 4.36 and 4.37. For multilane OHBDC truck loading, a load modification factor, $m = 1$, 0.90, 0.80 and 0.70 corresponding to the given number of loaded lanes, $n = 1, 2, 3, 4$ respectively, is applied to the load (57). The equations derived herein are for one-lane and two-lane loadings. Therefore, in case of three and four-lane loadings, the total factored load for design is reduced by 0.80 and 0.70, respectively.

**4.8.2 AASHTO Truck Load**

Since the internal work done by the single concentrated load and the equivalent AASHTO HS20 truck loading are equal, their external work are also
equal. Thus:

\[ P = \left[ 1 + \gamma_1 \left(1 - \frac{2d_1}{L}\right) + \gamma_2 \left(1 - \frac{2d_2}{L}\right) \right] \frac{Q}{2} \quad 4.39 \]

In which; \( d_1 \) and \( d_2 \) = distance of the axle loads \( \gamma_1 Q \) and \( \gamma_2 Q \), respectively, from the axle load \( Q \). For an AASHTO HS 20 truck(2), \( \gamma_1 = 0.25 \) and \( \gamma_2 = 1 \). Therefore Eq. 4.38 reduces to:

\[ P = \left[1.125 - \frac{(d_1 + 4d_2)}{4L}\right] Q \quad 4.40 \]

Applying the live load factor \( \gamma_{L} \) to \( Q \) (2), the factored equivalent wheel load \( P_c \) becomes, from Eq. 4.39:

\[ P_c = (\gamma_{L} \gamma_{L}) P \quad 4.41 \]

Thus, in design, given the load on the bridge, the factored collapse load \( P_c \) can be derived from Eqs. 4.39 and 4.40. For multilane AASHTO HS 20 truck loading, a modification factor, \( m = 1, 1, 0.90, \) and 0.75 corresponding to the given number of loaded lanes, \( n = 1, 2, 3, 4 \) respectively, is applied to the load. The equations derived herein are for one-lane and two-lane loadings. Therefore, in case of three and four-lane loadings, the total factored load for design is reduced by 0.90 and 0.75, respectively.
Fig. 4.1  Yield Line Failure Pattern for Single Span and Two-Span Continuous Composite Bridges under Eccentric Loading.
\[
\phi = \frac{\delta}{gg'} = \frac{\delta}{l_c \cos \alpha} \\
\delta_y = \frac{\delta (l_c - y)}{i_c}
\]

Fig. 4.2 Virtual Work Calculation in the Longitudinal Direction.
Fig. 4.3  Virtual Work Calculation in the Transverse Direction.
Fig. 4.4  External Work Calculation.
Fig. 4.5 Locations of Concentric Loads Considered in the Analyses.
Fig. 4.6 Yield Line Failure Pattern for Single and Two-Span Continuous Composite Bridges Under Simulated Two-Wheel Truck Load
Negative Yield Line in the Continuous Case Only

Fig. 4.7 Yield Line Failure Pattern for Single and Two-Span Continuous Composite Bridges Under Simulated Two-Wheel Truck Loads
Concentric Loading

Step (1) Bridge deflects uniformly.

Step (2) Additional deflection at the centre.

Fig. 4.8 Bridge Failure Steps Under Concentric Loadings.
Fig. 4.9 Derivation of the Concentric Collapse Load Expression.
\[ P_c = 4 \frac{M_p}{L} \]

\( M_p \): Ultimate Positive Moment Capacity of the Composite Section.

\[ P_c = \left(\frac{2}{L}\right) (2M_p + M_n) \]

\( M_n \): Ultimate Negative Moment Capacity of the Composite Section.

Fig. 4.10 Simple Beam Collapse Mechanism in Single and Two-Span Composite Bridges
Fig. 4.11 Longitudinal Section of a Continuous Two-Span Composite Bridge Under OHBDC and AASHTO Truck Loads.
Chapter V

Experimental Program

5.1 General

The experimental work involved the fabrication, instrumentation and testing of four composite bridge models in two groups. Group A, Fig. 5.1, consists of two simply supported single-span bridge models I and II, while group B, Fig. 5.2, consisted of two continuous two-span bridge models III and IV. The principal difference between the two models in each group was the type of connection between the main beams and transverse diaphragms, with one being bolted and the other being welded. These tests were intended to examine the influence of the bolted and welded type of connections on the structural response of composite bridges subjected to service and ultimate loads. The testing program steps may be summarized as follows:
1. Elastic testing of the steel grid models before casting of the concrete deck slab.

2. Elastic (service load) testing of the composite steel-concrete models.

3. Ultimate (collapse load) testing of the composite bridge models.

Throughout the testing program the necessary measurements were taken to examine the overall structural response under several loading combinations. These measurements were used to verify the theoretical results and to provide a reliable theoretical model to be used in the analysis of other bridges.

5.2 Models Description

5.2.1 Group A: Simply Supported Bridge Models (I and II)

Bridge models I and II were identical in all aspects except for the type of connection between the main beams and the transverse diaphragms. In bridge model I, these connections were bolted, while in bridge model II they were welded. Fig. 5.3. The two models were approximately 1/5 scale models of a two lane composite highway bridge. This was chosen in accordance with the available space in the structural laboratory. Each model was ten feet (3.05 m) in span and seven and one-half feet (2.28 m) in width; the concrete slab was two inches thick.
(50 mm) with $f_{c'} = 5300$ psi (36.5 MPa) in strength. This strength was achieved using a High Early Strength Portland Cement with a maximum aggregate size of 0.375 inch (10 mm). The design of the concrete mix was carried in accordance with the CPICA (Canadian Portland Cement Association) guide lines (5). The concrete slab was reinforced in two orthogonal directions with 0.2% minimum reinforcement of 1/8 inch (3 mm) bars and spaced 3 inch (75 mm) in the two directions. The steel grid consisted of five main beams and five rows of diaphragms; all beams were of W 6x15 steel sections grade 40.21-44W in accordance with CSA (6).

In bridge model I, the diaphragms were bolted to the main girders with back to back angles which were welded to the web of the main girder as shown in Figure 5.3(a,b). In the bridge model II, the diaphragms were welded to the web and flange of the main girder using 1/4 inch (6.35 mm) fillet and groove weld, Fig. 5.3(c). To insure the full interaction between the concrete deck slab and the steel frame-work, Nilson shear studs with 1/2 inch (13 mm) diameter and 1.5 inch (38 mm) height were used. The spacing between the studs was 4 inches (100 mm) as shown in Fig. 5.4.
5.2.2 Group B: Continuous Bridge Models (III and IV)

Similar to group A, the two models III and IV differed only in the type of connection between the main beams and the diaphragms. In model III, the connections were bolted and in model IV the connections were welded, Figs. 5.5 and 5.6. The two models were 1/10 scale models of two-lane composite bridge. This was chosen to conform with laboratory space limitations and loading frame capacity. The layout of the two bridge models is shown in Fig. 5.2. Each of the two spans was 6.25 feet (1.9 m) in length and 3.125 feet (0.95 m) in width. The concrete slab was 2 inch (50 mm) thick with fc' = 5300 psi (36.5 MPa). Similar to group A the reinforcement was 1/8 inch (3 mm) bars with 3 inch (75 mm) spacing. The steel grid consisted of four S 5x10 main beams and five S 4x7.7 transverse diaphragms in each span. The connection set-up was similar to that in group A. To insure a full interaction between the main beams and the concrete deck slab, shear studs were also used.

5.3 Testing Instruments

5.3.1 Strain Gauges

The Maxwell's reciprocal theorem and the bridge model symmetry were used to determine the location of the strain gauges. This was done to optimize the
number of strain gauges needed to provide sufficient information about the bridge behaviour under load. Figure 5.7 shows the locations of the gauges in the four tested bridges. Due to differences of the concrete slab and steel beams material different sizes of strain gauges were employed. Detailed description of the gauges used is given in Appendix A-3. The strain readings were taken using an Optilog 200 Data Acquisition System (DAS) attached to an IBM PC, Fig 5.8.

5.3.2 Dial Gauges

The deflections were measured using mechanical dial gauges located under the bridge, Fig. 5.9. The dial gauges used were with a travel sensitivity of 1/1000 inch (0.025mm). The gauges were placed beneath the same locations chosen for the strain gauges.

5.3.3 Loading Apparatus

The four bridge models were supported on stationary roller supports Fig. 5.10. The models were clamped to the supports on all sides to prevent the bridge from uplift during loading; Fig. 5.11. The load was applied using a 50 and 200 kips (222 and 890 kN) hydraulic jacks bearing against a steel frame which in turn was anchored to the laboratory floor. The 50 kips (222 kN) hydraulic jack was used for the service load testing while the 200 kips (890 kN) hydraulic jack was used for the
failure testing of the bridge models. This loading set-up is shown in Fig. 5.12. In order to monitor the values of the applied load, two flat load cells with full bridge circuit were used with 50 kips and 200 kips capacity for both the lower and higher load application range respectively. For each tested model, the maximum applied load may varied depending on the service load limit and the applied load location. All models were loaded to failure following the desired testing combinations under service load.

5.4 Experimental Setup and Testing Procedures

The testing sequence for the four bridge models was divided in two phases: the elastic (service load) phase and the failure (ultimate load) phase. The first phase consisted of service load testing of the steel grid before and after casting of the concrete deck slab. In the second phase, each of the four bridge models was loaded to failure. In this phase the load was applied eccentrically to the side of the model until complete collapse was achieved.

5.4.1 Phase One

i. Testing of the Steel Grid:
Each of the steel grid models was loaded with a concentrated load applied at several locations over each of the main girders. Figures 5.13 and 5.14 show a typical loading position. This may also be described as an influence line loading application.

II. Testing of The Composite Bridge Model:

After casting and curing the concrete slab, each of the models was tested with concentrated influence line load applied in a similar way to that on the steel grid model, Figs. 5.15 and 5.16. Furthermore, to simulate truck axle loading, the load was applied in one and two pairs to simulate single and two truck loadings. The single pair loading was to simulate one truck moving along the span in one direction while the two pairs loading was to simulate two trucks moving along the span in two opposite directions. In all the studied cases, the load was applied at the mid-span but in different locations transversely across the bridge models.

5.4.2 Phase Two

i. Failure Load:

In all models a single pair of the simulated truck wheels was placed on one side of the bridge model, Fig. 5.17. The load was then increased until failure, Fig.
5.18. Failure was reached when the bridge model could not carry any further increase in load. This load location was chosen based on the data obtained from testing. This location was selected in order to demonstrate clearly the influence of the diaphragms and the diaphragm connections on the overall behaviour of the composite structures particularly at the failure stage. Figures 5.19 and 5.20 show the observed experimental failure yield line pattern.

ii. **Testing of the Two-Span Continuous Composite Beam:**

A composite two-span continuous beam was tested to failure. The geometric details, shown in Fig. 5.21, were identical to those of a single beam strip in the two-span continuous composite bridge models. The aim of this test was to examine the collapse load capacity of the composite beam and to compare it to the collapse load of a composite two-span continuous bridge.
Fig. 5.2 Layout of the Laboratory Two-Span Continuous Composite Bridge Model.

$L \times W = 6.5 \text{ ft} \times 3.125 \text{ ft}$
Fig. 5.3 (a) Bolted-Diaphragm Connection in Simply Supported Model.
Fig. 5.3 (b) Bolted-Diaphragm Connection in Simply Supported Model.
Fig. 5.3 (c) Welded-Diaphragm Connection in Simply Supported Model.
Fig. 5.4 Studs Provide Full Interaction Between the Main-Girder and The Concrete Deck Slab.
Fig. 5.5  Bolted-Diaphragm Connection in Continuous Model.
Fig. 5.6  Welded-Diaphragm Connection in Continuous Model.
Fig. 5.7   Locations of the Strain Gauges in the Tested Bridge Models
Fig. 5.8   (a) Strain Measuring Equipment
Fig. 5.8 (b) Strain Measuring Equipment
Fig. 5.9   Dial Gauges Used to Measure Deflection of Models.
Fig. 5.8  (a) Strain Measuring Equipment
Fig. 5.11 "C" Clamps used to Prevent the Model From Uplift during Loading.
Fig. 5.12  Testing Setup of Composite Bridge Model
Fig. 5.13  Steel Grid Test in the Simply Supported Model.
Fig. 5.14    Test of Two-Span Continuous Steel Grid.
Fig. 5.15 Test of Simply Supported Composite Bridge Model.
Fig. 5.16 Test of Continuous two-Span Composite Bridge Model.
Fig. 5.17  Ultimate Loading Position.
Fig. 5.18 Failure Yield Line Pattern of Simply Supported Model with Welded-Diaphragms.
Fig. 5.19 Failure Yield Line Pattern of Simply Supported Model with Bolted-Diaphragms.
(b) Welded Diaphragm Model

Fig. 5.20 Failure Yield Line Patterns of the Continuous Two-Span Tested Bridge Models.
Fig. 5.21 Geometric Details of the tested Composite Beam.
Fig. 5.22 Composite Beam before Casting the Concrete.
Fig. 5.23 Composite Beam Section.
Fig. 5.24  Testing of the Composite Beam.
Chapter VI

Results and Discussions

6.1 General

The experimental study on the elastic and post-elastic behaviour of composite steel-concrete bridges was carried out by testing four laboratory bridge models. Both the finite element method and the yield line theory were used in the analyses. The analytical effort was extended to cover some actual size bridges selected from the Standard Plans for Highway Bridges, Volume II, recommended by the Federal Highway Administration of the United States Department of Transportation (74). In this chapter, the experimental and theoretical results achieved are presented and compared. The influence of the type of connection between the longitudinal main beams and the transverse beam diaphragms on the response of composite bridges subjected to elastic and post-elastic loadings is also discussed.
A. FINITE ELEMENT APPROACH

6.2 Elastic Response of The Steel Grid

The steel grid structures of all the laboratory bridge models were tested elastically before casting the concrete deck slab. This was done in order to study the influence of the simple and rigid diaphragm connection types SCD and RCD on the steel grid response under load, and to determine the contribution of the concrete deck slab to the load distribution in the composite bridge. Figures 6.1 and 6.2 show the application of a concentrated load on both the single span and the two-span continuous steel grid models. The concentrated load was applied at different locations over the grid as shown in Fig. 6.3. The experimental and theoretical deflections at mid-span of the grid modes (I to IV) are compared and presented in Figs. 6.4 to 6.8. The results show good correspondence between the experimental and the theoretical results. The maximum deflection of the SCD grid structure was in some cases almost three times that of the RCD grid. Furthermore, the deflection in the SCD grid structure was much more localized than that of the RCD grid structure.
6.3 **Elastic Response of Composite Bridge Structures**

The elastic response of composite bridge structures was examined for the tested bridge models by applying single concentrated and simulated truck wheel loads over the bridge decks. The loads were applied at different locations across the mid-span in the manner shown in Figs. 6.9 to 6.12. The experimental and theoretical results are presented in Figs. 6.13 to 6.24 and were found to be in good correspondence with one another. From the results of the single span and the continuous two-span composite bridges, it was observed that the concrete deck slab substantially reduces the bridge deflection, particularly in the SCD bridge type, when compared to the deflection under the same load before casting of the concrete deck slab. This is attributed to the increased stiffness and better load distribution characteristics of the composite bridge after casting the concrete deck slab. The deck slab was observed to play a more significant role in the eccentric loading case. It is well known that an eccentric load on the bridge would give rise to twisting moments that are much greater in magnitude than those caused by the same load applied concentrically. Thus, the uncracked concrete deck slab, with its significant torsional resistance, is able to distribute the eccentric load transversally quite effectively in both the SCD and RCD bridge models. Figures 6.13 to 6.24
also present comparisons between the deflections in the SCD and RCD bridge models. It is observed that the deflections in the SCD bridge is always higher than that in the RCD, particularly in the loading zone. In the SCD type, the deflections remain more localized even after casting the concrete deck slab. From Fig. 6.25 it is interesting to note that the deflection response of a composite bridge model with only two rigidly connected end diaphragms was found to be quite similar to that of a bridge with five simply connected diaphragms. Furthermore, the deflection response of composite bridge models with three and five rigidly connected diaphragms was also similar. This indicates that there is an optimum number of welded diaphragms that can be used, beyond which no further improvement in the load distribution over the bridge may be expected.

6.4 **Post-elastic Response of Composite Bridges**

Each of the four bridge models was loaded beyond its elastic load to failure after completing the elastic loading (service load) tests. The observed yielding patterns at failure are shown in Fig. 6.26 (a to d). Comparison of the yielding patterns reveals that: (i) The failure of the SCD bridge models is generally localized; (ii) While the torsional resistance of the concrete slab is evident it was
less so in the SCD bridge models where only the area of the deck slab in the vicinity of the load suffered severe cracking. (iii) Cracks of significant length developed in the concrete deck slab as the loaded girder began to yield. The cracks ran longitudinally at mid-span and curved until they were diagonal at the supports; (iv) Due to the flexibility of the bolted connections in the SCD bridge models, the transverse negative and positive moment capacities were those of the concrete deck slab in the transverse direction. These were much less than those in the RCD bridge models where both the concrete deck slab as well as the transverse beams contribute to the transverse negative and positive moment capacities of the bridge; and (v) Both the SCD and RCD bridge types experienced similar concrete slab compression failure with different lengths along the positive moment yield line.

The wide flange section, W 6×15, used in the single span bridge models has a (1/2 the flange width/flange thickness) ratio or (b/t) >170/f_y. Subsequently, at failure the top flange of the outside main girder located under the applied load had locally buckled. The concrete slab rebars were also buckled along the positive moment yield line as shown in Fig. 6.27.

Based on the experimental and the FEM analytical results, Figs. 6.28 to
6.31, it was observed that the failure of the SCD bridge models was precipitated by large deflection under the load after considerable cracking of the slab had taken place as well as after yielding of the longitudinal steel beam under the load. This was not the case in the RCD models where deflections remained linear after severe cracking of the concrete slab and the initial yielding of the longitudinal beams had taken place. Figures 6.28 to 6.31 present the experimental and analytical load-deflection results obtained from each of the tested models. It is observed that there is an excellent agreement between theory and experiment in the elastic range and reasonable agreement in the post-elastic range. This is mainly due to the assumptions adopted in the analysis such as ignoring strain hardening of the steel material as well as the tension strength of concrete; these assumptions led to conservative results. Figures 6.28 to 6.31 also demonstrate the difference in the structural response between the SCD and RCD bridge models. The sequence of yielding of the longitudinal steel beams in all tested bridge models subjected to eccentric loading is shown in Figs. 6.32 to 6.35. In the SCD single span bridge model, the sequence of yielding shown in Fig. 6.32 is: beams 2, 1 and 3 with beams 4 and 5 not yielding at all. On the other hand in the single span RCD bridge model, Fig. 6.33, the sequence of yielding at failure is: beams
1, 2, 3 and 4 with beam 5 not yielding.

In the continuous two-span SCD bridge model, Fig. 6.34, beams A, B yielded while beams C and D did not yield. In the continuous two-span RCD model, Fig. 6.35, the sequence of beams that yielded is: beams A, B and C with beam D not yielding. The difference observed in the sequence of yielded girders in the SCD and RCD bridge models could be attributed to the greater degree of flexibility of the SCD bridge models when compared to the RCD type.

Table 6.1 presents the results of the parametric study carried out to compare the ultimate load capacities of SCD and RCD composite bridges. The study covered both single span and continuous two-span bridges, with various aspect ratios (L/W) and truck loading cases. From the results it can be observed that the structural stiffness as well as the ultimate load carrying capacities of the RCD bridges are significantly higher than those of SCD bridges.

6.5 Composite Beam Failure

One typical continuous composite beam was tested, the size of which was identical to that of a single beam strip in the continuous two-span bridge models (models III & IV), Fig. 5.21. Figure 6.36 shows the elastic load deflection results
obtained from the tested beam compared with those obtained from the exterior loaded girders under eccentric load case in the SCD and RCD two-span continuous composite bridge models. From the results, it was observed that under the same applied load the deflection of the single beam is higher than that of the loaded girder in the SCD bridge model and almost twice that of the RCD bridge model. This demonstrates clearly the influence of the welded connections in distributing the load over the bridge in an orthotropic manner with a two way slab action. The tested composite beam collapsed under the application of a concentrated load of about 28 kips applied at the centre of each span. If one considers the continuous bridge model to be made of three composite beam sections, the bridge model failure load would have been equal to 168 kips, i.e., \( P_c = 28 \text{ kips} \times 2 \text{ (No. of Spans)} \times 3 \text{ (No. of girders)} \). However, the SCD eccentrically loaded bridge model collapse load was 128 kips, 30% lower than that of the composite beam, whereas the collapse load of the eccentrically loaded RCD bridge model was 164 kips, 3% lower than that of the composite beam. This reflects clearly the better load distribution characteristics over the RCD composite bridge model.
6.6 Load at First Yield

Tables 6.2 and 6.3 present results of the parametric study carried out to obtain the ratio between the load at first yield in the loaded girder and the bridge collapse load for single span and continuous two-span composite bridges under eccentric and concentric loads, respectively. The tables cover a wide range of bridge models with different aspect ratios, diaphragm connection types and load locations. In general, it is observed that the load at first yield falls within 60% and 76% of the bridge collapse load with an average value of 70%. Under eccentric and concentric loading the ratio was between 60% and 68% for the SCD bridges and between 69% to 76% for the RCD bridges. This could be attributed to the higher structural flexibility of SCD bridges over RCD bridges. Consequently, the expected reserve capacity after the detection of the first yield in the loaded girder, with respect to the final collapse load, will be higher in SCD bridges than that of RCD bridges. Nevertheless, RCD bridges offer better load distribution in the elastic and post-elastic loading ranges and higher load carrying capacities over those offered by SCD bridges.
6.7 Load Distribution Factor D

The change in the load distribution factor D (defined in Chapter III) was examined for a wide range of prototype composite bridges subjected to different loading conditions. Figures 6.37 to 6.48 present the variations in the D factor with load applied to failure. Figures 6.37 & 6.38 and 6.39 & 6.40 present comparisons for the D factor between the SCD and RCD bridge models I and II tested under eccentric loading. The results reveal that the longitudinal steel beams in the RCD bridge model tend to share carrying the applied load at a much earlier stage of the load history than those in the SCD bridge model. Furthermore, in contrast to the beams in the RCD bridge model, only about half of the longitudinal beams in the SCD bridge model reach their ultimate moment capacity. The reason for this is that the concrete deck slab fails in compression before these remote beams can reach their ultimate capacity. Once the concrete deck slab fails the simple diaphragm connections have very little to offer in the load transfer between the beams. This is obviously not the case in the RCD bridge models. Thus, this shows that the rigidly connected diaphragms in composite bridges not only have the ability to distribute the load effectively but also to redistribute any overload that the bridge may be subjected to during its life.

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Figure 6.41 presents a comparison between three RCD bridge prototypes having two, three and five transverse diaphragms, respectively. From the results, it can be observed that the behaviour of the bridge prototype with only two rigidly connected diaphragms, one at each end, is similar to an SCD bridge prototype with five diaphragms and under the same load. This confirms the assumptions made in the calculations of the negative moment contribution in the SCD bridge models, shown in Table 6.4. It is also observed that with five rigidly connected diaphragms, the load distribution over the bridge is considerably improved when compared to the other two cases. No significant improvement is expected with additional diaphragms.

The D factor for the tested two-span continuous bridge models III and IV under eccentric loading are shown in Figs. 6.42 and 6.43. Due to the relatively larger stiffness ratio of main beams to transverse diaphragms and the larger (L/W) ratio, the behaviour of bridge models III and IV was somewhat different from that of bridge models I and II. It was observed that, in addition to the higher load carrying capacity of the continuous RCD bridge model, the participation of the remote beams at failure was better than that in the SCD bridge model. The D factors for bridge models III and IV under concentric load are shown in Fig. 6.44.
From the results it can be observed that the rigid diaphragm connection types provide a better load distribution over the bridge at failure than that the simple diaphragm connection types.

Table 6.5 shows the elastic load distribution factors, D, for single span composite bridge models with five main girders subjected to eccentric loading. Results for the rigidly connected and simply connected diaphragm bridges for different aspect ratios (L/W) are presented.

The details of these bridges are as follows, Fig. 3.10 (a):

Width of bridge = 28 ft. (8.5 m)
Spacing of main girders = 7 ft. (2.075 m)
Size of girders for each span are specified by the U.S. Dept. of Transportation (74).

Number of diaphragms (size W18x50) = 5
yield strength of Steel $f_y$ = 44 Ksi (300 MPa)
Concrete compressive strength $f_c$ = 4.5 Ksi (30 MPa)
Slab thickness = 8.0 inch (203 mm)
Percentage of steel rebar in the slab = 0.2%

The results reveal that girder A in both RCD and SCD bridges has the critical D factor. However, as the span increases relative to the bridge width (i.e. when L/W increases) the critical D factor increases. Furthermore, the differences
between the girder D factors in the SCD bridges are greater than those in the RCD bridges. This confirms that rigidly connected diaphragms provide a better load distribution than the simply connected type. The D factors at the ultimate limit state, \((D)_{ult}\), for eccentric loading are shown bracketed in Table 6.5. These factors are invariably higher in the RCD bridge types. The differences between the girder's \((D)_{ult}\) factors for the RCD bridges are smaller than those in the SCD bridges. Thus, a better transverse load distribution at the ultimate limit state also can be effected by the use of rigidly connected diaphragms. Unlike the elastic (service) loading case, the ultimate loading case shows that the critical \((D)_{ult}\) factors decrease with increase in the aspect ratio \((L/W)\) for SCD bridges. However, for RCD bridges the relationship between the critical \((D)_{ult}\) factor and the aspect ratio appears to be bilinear. Furthermore, the critical \((D)_{ult}\) factor shifts from girders A and B to the middle girder C as the aspect ratio \((L/W)\) increases.

In Table 6.6, results for the elastic and ultimate distribution factors, D and \((D)_{ult}\), for continuous two-span composite bridges under full four lanes truck load are presented. The details of these bridges are as follows, Fig. 3.10(b):

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Six main girders with spacing $S = 10$ ft. (3.05 m) Cross sections specified by the U.S. Dept. of Transportation (74).

Number of diaphragms (size W18x50) = 9

yield strength of Steel $f_y = 44$ ksi(300 MPa)

Concrete compressive strength $f_c = 4.5$ ksi(30 MPa)

Slab thickness = 9.0 in.(229 mm)

Percentage of steel rebar in the slab = 0.2%

The results indicate that for a bridge with an aspect ratio of $(L/W) = 3.0$, the critical elastic $D$ factor is for the outside girders A and F while the least $D$ factor is that of the central girders C and D. As the aspect ratio increases, the load distribution is reversed. This trend applies to both RCD and SCD bridges. At the ultimate limit state the critical value $(D)_{ul}$ is for girder B rather than for girder A; again as the aspect ratio $(L/W)$ increases the critical $(D)_{ul}$ becomes that of the two central girders C and D. The results also reveal that the type of diaphragm connection does not significantly influence the elastic or ultimate distribution factors for the case of concentric loading. From Table 6.6 it is also observed that, as the $(L/W)$ ratio increases to $(L/W=5.0)$, no difference is observed in the load distribution response between SCD and RCD bridges where the structure behaves in one way action.
Theoretical results for the elastic and ultimate load distribution factors were also obtained for continuous two-span bridge models with \((L/W) = 2.0\). The geometry of these bridge models was the same as that of the tested models III and IV described earlier. Table 6.7 shows the results for three cases of loading. It is observed that the differences in the elastic as well as the ultimate load distribution factors for the four longitudinal girders are somewhat smaller in the RCD bridges than in the SCD bridges. As expected, the largest differences occurred for loading case I. Under an eccentric load (cases I & II) the critical \((D)_{ult}\) factors are slightly larger in the RCD bridges than those in the SCD bridges.

The load distribution factor, \(D_d\), introduced by the OHBDC (57) was calculated for some of the bridges used in the parametric study. This was compared with the elastic and ultimate load distribution factors, \(D\) and \((D)_{ult}\), calculated using the finite element method. Table 6.8 presents the comparison. From the table it is observed that the \(D_d\) factors in the OHBDC are much lower than those calculated using the finite element approach. These discrepancies appeared to be dependent on the geometric characteristics and loading condition of the bridges. Despite these low values for the load distribution factors the OHBD code method was found to yield safe and conservative results.
B. YIELD LINE APPROACH

Table 6.9 presents a summary of the developed yield line formulae. Whether it is a design or an analysis problem, proper load and resistance factors, whenever appropriate, should be incorporated with the derived formulae. In order to determine accurately the ultimate load carrying capacity of the bridge, it is essential to know the degree of moment of resistance at the instance of collapse. Based on the experimental and theoretical results of this study it was revealed that the effective moments of resistance along the yield lines depend on the loading position and the connection type between the main girders and the transverse diaphragms. Table 6.4 shows how to determine the various resisting moments, \( m_p \), \( m_v \), \( m_n \), and \( m_i \) at the instance of the composite bridge collapse.

6.8 Collapse Load Using Yield Line Formulae and Finite Element Method

Table 6.10 presents comparison of the theoretical and experimental collapse loads of the four tested bridge models. The theoretical analyses were carried out using the developed yield line formulae and the finite element technique. It can be
observed that the theoretical collapse loads in Table 6.10 are in good agreement with the experimental collapse loads, the former being conservative and lower than the latter, notwithstanding that the yield line theory leads to an upper bound for the collapse load.

The testing of bridge model IV was terminated prematurely due to the failure of the loading beam. Consequently, the experimental collapse load for bridge model IV was less than what was expected theoretically. The experimental collapse load was estimated to be 164 Kips based on the test results from the other three tested bridge models.

A parametric study was carried out on few SCD and RCD composite bridges in order to compare their ultimate loading capacities and behaviour. Table 6.11 presents comparison between the calculated theoretical collapse load using both the finite element method and the developed yield line formulae. The table demonstrates good correspondence between the two methods. It should be noted that aspect ratio, connection type, applied load location and relative stiffness in the longitudinal and transverse directions play a significant role in determining the magnitude of the collapse load by the yield line method. For example the larger the aspect ratio (L/W) the smaller the contribution of the negative moment to the
collapse load. With relatively large aspect ratio, the structure will perform in one way action with a simple collapse mechanism.

The developed yield line expressions for the composite bridge collapse load were also checked against some reported experimental and theoretical results, as shown in Table 6.12. The table presents good correspondence between the reported results and those calculated using these expressions. It should be noted that cases 3 and 4 listed in Table 6.12 were for reinforced concrete grid and slab bridges. This clearly demonstrates that the developed collapse load yield line expressions can also be used in the design and analysis of other bridge types.

6.9 Effect of Different Parameters on the Formed Positive Yield Line Length

The influence of the bridge aspect ratio, positive and negative sectional moment capacities on the length of the positive yield line \( l_c \) in the concrete deck slab under eccentric loading was also examined; see Tables 6.13 to 6.16. The results revealed that the positive yield line length \( l_c \) increases when the bridge aspect ratio \( (L/W) \) increases; as the main girders' positive moment capacity increases the yield line length decreases; \( l_c \) also increases with the longitudinal and/or transverse negative sectional moment capacities increase.
6.10 **Critical Positive Yield Line Length**

From the derived yield line equations, it is observed that the predicted collapse load, \( P_c \), is a function of the yield line length \( l_c \) when the composite bridge is loaded eccentrically. The minimum collapse load can then be calculated from the stationary condition \( \delta P / \delta l_c = 0 \) applied to Eqs. 4.16 and 4.17. This procedure will yield a second order equation in the variable \( l_c \), which when solved will yield \( l_c \) and hence the minimum collapse load \( P_c \) from Eqs. 4.16 and 4.17. This procedure is shown in Appendix A-2. However, from the extensive results obtained from the parametric study, an iterative procedure may also be adopted to determine the lowest collapse load for assumed values of \( l_c \). Thus, from the parametric study, for SCD bridges with aspect ratio, \( L/W \) between 1 and 3, the ratio \( l_c/W \) would generally fall between 0.55 to 0.75. A good starting value in the iterative procedure would be \( l_c = 0.55 \ W \). On the other hand, for RCD bridges with \( L/W \) between 1 and 3, it is expected that the ratio \( l_c/W \) would be in the range of 0.60 to 1.0; a reasonable starting value in this case would be \( l_c = 0.6 \ W \). Experience has shown that the lowest collapse load can be found in two or three cycles.
Fig. 6.1  Steel Grid Concentrated Load Test "Simply Supported Model".
Fig. 6.2  Steel Grid Concentrated Load Test "Continuous Two-Span Model".
Fig. 6.3 Apied Load Locations For The Tested Steel Grid Models.

a) Simply Supported Steel Grid Models [Group A]  b) Two Span Continuous Steel Grid Models [Group B]
Deflection of the Steel Grid Model across the Width at Midspan Under P=12 Kips
Fig. 6.5  Deflection of the Steel Grid Model across the Width at Midspan Under $P=12$ Kips
Deflection of the Steel Grid Model across the Width at Midspan Under P=12 Kips
Fig. 6.7  Deflection of the Steel Grid Model across the Width at Midspan Under P=6 Kips
Fig. 6.8  Deflection of the Steel Grid Model across the Width at Midspan Under P=6 Kips
Fig. 6.9 (b)  Applied Load Locations For The Tested Composite Bridge Models.

a) Simply Supported Bridge Models [Group A]

b) Two Span Continuous Bridge Models [Group B]
Fig. 6.10 Composite Model under Concentrated Load Test.
Fig. 6.11 Simulated Single Truck Load.
Fig. 6.12   Simulated Pair Truck Load.
Fig. 6.13  Deflection of the Composite Bridge Model across the Width at Midspan Under P=12 Kips
Fig. 6.14  Deflection of the Composite Bridge Model across the Width at Midspan Under $P=12$ Kips
Fig. 6.15  Deflection of the Composite Bridge Model across the Width at Midspan Under P=12 Kips
Fig. 6.16  Deflection of the Composite Bridge Model across the Width at Midspan Under P=12 Kips
Fig. 6.17  Deflection of the Composite Bridge Model across the Width at Midspan Under P=12 Kips
Fig. 6.18  Deflection of the Composite Bridge Model across the Width at Midspan Under P=8 Kips
Fig. 6.19  Deflection of the Steel Grid Model across the Width at Midspan Under $P=6$ Kips
Fig. 6.20  Deflection of the Steel Grid Model across the Width at Midspan Under $P=6$ Kips
Fig. 6.21  Deflection of the Steel Grid Model across the Width at Midspan Under P=5 Kips
Fig. 6.22  Deflection of the Steel Grid Model across the Width at Midspan Under P=5 Kips
Fig. 6.23  Deflection of the Steel Grid Model across the Width at Midspan Under P=5 Kips
Deflection in inch

○  5 Rigidly Connected Diaphragms
△  3 Rigidly Connected Diaphragms
□  2 Rigidly Connected Diaphragms
●  5 Simply Connected Diaphragms

(1.00 inch = 25.4 mm)

Fig. 6.25   Deflection of Composite Bridge Model with different Numbers of Diaphragms Under P=12 Kips
Yield line patterns of the tested bridge models.

Fig. 6.26

Support Lines

Two Span Continuous Bridge Models

Support Line

Single Span Bridge Models
Fig. 6.27(a) Steel Girder Local Buckling of the top Flange.
Fig. 6.27(b) Steel Girder Local Buckling of the top Flange.
Fig. 6.28  Load Deflection Response of Bridge Model I

1.00 inch = 25.4 mm
1.00 kip = 4.448 kN
Fig. 6.29  Load Deflection Response of Bridge Model II
Fig. 6.30  Load Deflection Response of Bridge Model III

1.00 inch = 25.4 mm
1.00 kip = 4.448 kN
Fig. 6.31  Load Deflection Response of Bridge Model IV
Fig. 6.32  Load Strain Response of Bridge Model I

1.00 inch = 25.4 mm
1.00 kip = 4.448 kN
Fig. 6.33  Load Strain Response of Bridge Model II
Load in Kips

Strain με

1.00 inch = 25.4 mm
1.00 kip = 4.448 kN

Fig. 6.34 Load Strain Response of Bridge Model III
Fig. 6.36  Load-Deflection Relation of Loaded Girder in Bolted and Welded bridges and of Composite Beam.
Fig. 6.37  Load Distribution Factor VS Collapse Load Ratio.
Fig. 6.38  Load Distribution Factor VS Collapse Load Ratio.
Fig. 6.40
Load Distribution Factor vs Collapse Load Ratio.
Fig. 6.41  Load Distribution Factor VS Collapse Load Ratio.
Continuous Two Span (RCD) Bridge Model

Fig. 6.43 Load Distribution Factor vs Collapse Load Ratio.
Fig. 6.45 Load Distribution Factor VS Collapse Load Ratio.
Fig. 6.46 Load Distribution Factor VS Collapse Load Ratio.
Simply Supported (SCD) Bridge Prototype

L/W = 1.5, L = 42 ft, W = 28 ft

Fig. 6.47 Load Distribution Factor VS Collapse Load Ratio.
Fig. 6.48 Load Distribution Factor VS Collapse Load Ratio.

Simply Supported (RCD) Bridge Prototype
L/W = 1.5, L = 42 ft, W = 28 ft
<table>
<thead>
<tr>
<th>Bridge Type and No. Lanes</th>
<th>Loading</th>
<th>L x W (ft x ft)</th>
<th>L/W</th>
<th>(P_d)<em>{RCD} / (P_d)</em>{SCD}</th>
</tr>
</thead>
<tbody>
<tr>
<td>S.S., 2–lanes</td>
<td>Two Trucks</td>
<td>42 x 28</td>
<td>1.5</td>
<td>1.35</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Two Trucks</td>
<td>56 x 28</td>
<td>2.0</td>
<td>1.46</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Two Trucks</td>
<td>84 x 28</td>
<td>3.0</td>
<td>1.26</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Truck</td>
<td>42 x 28</td>
<td>1.5</td>
<td>1.53</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Truck</td>
<td>56 x 28</td>
<td>2.0</td>
<td>1.44</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Truck</td>
<td>84 x 28</td>
<td>3.0</td>
<td>1.45</td>
</tr>
<tr>
<td>S.S., 4–lanes</td>
<td>Single Truck</td>
<td>72 x 36</td>
<td>2.0</td>
<td>1.67</td>
</tr>
<tr>
<td>S.S., 4–lanes</td>
<td>Single Truck</td>
<td>250 x 50</td>
<td>5.0</td>
<td>1.15</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Simulated Truck</td>
<td>15 x 7.5</td>
<td>2.0</td>
<td>1.38</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Simulated Truck</td>
<td>13.13 x 7.5</td>
<td>1.75</td>
<td>1.29</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Simulated Truck</td>
<td>11.25 x 7.5</td>
<td>1.5</td>
<td>1.28</td>
</tr>
<tr>
<td>S.S., 2–lanes*</td>
<td>Single Simulated Truck</td>
<td>10.0 x 7.5</td>
<td>1.33</td>
<td>1.27</td>
</tr>
<tr>
<td>S.S., 2–lanes</td>
<td>Single Simulated Truck</td>
<td>9.375 x 7.5</td>
<td>1.25</td>
<td>1.28</td>
</tr>
<tr>
<td>C., 4–lanes</td>
<td>Four Trucks</td>
<td>150 x 50</td>
<td>3.0</td>
<td>1.30</td>
</tr>
<tr>
<td>C., 2–lanes*</td>
<td>Single Simulated Truck</td>
<td>6.5 x3.125</td>
<td>2.0</td>
<td>1.31</td>
</tr>
</tbody>
</table>

**NOTE:**
1 ft. = 0.305 m; S.S.: Simply Supported; C: Continuous 2–Span; L: Length; W: Width; *: Laboratory Model; RCD: Rigidly Connected Diaphragm; SCD: Simply Connected Diaphragm.
Table 6.2
Load at First Girder Yield VS Collapse Load of Bridge (Eccentric Load Application).

<table>
<thead>
<tr>
<th>Description of Bridge Model</th>
<th>(1) kips Load at First Yield</th>
<th>(2) kips Load at Failure</th>
<th>(3) (2)/(1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L/W = 1.33, 5 SCD, S.S., two wheels, one truck</td>
<td>56.0</td>
<td>84.0</td>
<td>1.49</td>
</tr>
<tr>
<td>L/W = 1.33, 5 RCD, S.S., two wheels, one truck</td>
<td>80.0</td>
<td>114.0</td>
<td>1.41</td>
</tr>
<tr>
<td>L/W = 1.33, 3 RCD, S.S., two wheels, one truck</td>
<td>80.0</td>
<td>110.0</td>
<td>1.37</td>
</tr>
<tr>
<td>L/W = 1.33, 2 end RCD, S.S., two wheels, one truck</td>
<td>80.0</td>
<td>100.0</td>
<td>1.25</td>
</tr>
<tr>
<td>L/W = 1.0, 5 RCD, S.S., two wheels, one truck</td>
<td>100.0</td>
<td>145.0</td>
<td>1.45</td>
</tr>
<tr>
<td>L/W = 1.25, 5 RCD, S.S., two wheels, one truck</td>
<td>82.0</td>
<td>120.0</td>
<td>1.47</td>
</tr>
<tr>
<td>L/W = 1.53, 5 RCD, S.S., two wheels, one truck</td>
<td>72.0</td>
<td>104.0</td>
<td>1.45</td>
</tr>
<tr>
<td>L/W = 1.75, 5 RCD, S.S., two wheels, one truck</td>
<td>64.0</td>
<td>88.0</td>
<td>1.39</td>
</tr>
<tr>
<td>L/W = 2.00, 5 RCD, S.S., two wheels, one truck</td>
<td>56.0</td>
<td>80.0</td>
<td>1.43</td>
</tr>
<tr>
<td>L/W = 2.00, 9 RCD, C., two wheels, one truck</td>
<td>140.0</td>
<td>200.0</td>
<td>1.43</td>
</tr>
<tr>
<td>L/W = 2.00, 9 SCD, C., two wheels, one truck</td>
<td>120.0</td>
<td>180.0</td>
<td>1.49</td>
</tr>
<tr>
<td>L/W = 2.00, 9 RCD, C., two wheels, one truck</td>
<td>105.0</td>
<td>150.0</td>
<td>1.43</td>
</tr>
<tr>
<td>L/W = 2.00, 9 SCD, C., two wheels, one truck</td>
<td>80.0</td>
<td>125.0</td>
<td>1.56</td>
</tr>
<tr>
<td>L/W = 1.76, 9 RCD, C., two wheels, one truck</td>
<td>110.0</td>
<td>160.0</td>
<td>1.45</td>
</tr>
<tr>
<td>L/W = 1.44, 9 RCD, C., two wheels, one truck</td>
<td>140.0</td>
<td>200.0</td>
<td>1.43</td>
</tr>
<tr>
<td>L/W = 1.28, 9 RCD, C., two wheels, one truck</td>
<td>160.0</td>
<td>240.0</td>
<td>1.49</td>
</tr>
<tr>
<td>L/W = 1.00, 9 RCD, C., two wheels, one truck</td>
<td>180.0</td>
<td>280.0</td>
<td>1.53</td>
</tr>
</tbody>
</table>

Note: S.S.: Single span bridge; C.: Continuous two span bridge; 1.00 Kip = 4.448 kN
Table 6.3

Load at First Girder Yield VS Collapse Load of bridge (Concentric Load Application).

<table>
<thead>
<tr>
<th>Description of Bridge Model</th>
<th>(1) kips Load at First Yield</th>
<th>(2) kips Load at Failure</th>
<th>(3) (2)/(1)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L/W = 1.33, 5 RCD, S.S., two wheels, one truck</td>
<td>100.0</td>
<td>136.0</td>
<td>1.35</td>
</tr>
<tr>
<td>L/W = 1.33, 5 SCD, S.S., two wheels, one truck</td>
<td>64.0</td>
<td>104.0</td>
<td>1.61</td>
</tr>
<tr>
<td>L/W = 1.33, 5 RCD, S.S., two wheels, one truck</td>
<td>112.0</td>
<td>160.0</td>
<td>1.43</td>
</tr>
<tr>
<td>L/W = 1.33, 5 RCD, S.S., two wheels, one truck</td>
<td>96.0</td>
<td>130.0</td>
<td>1.32</td>
</tr>
<tr>
<td>L/W = 1.33, 5 SCD, S.S., two wheels, one truck</td>
<td>96.0</td>
<td>152.0</td>
<td>1.59</td>
</tr>
<tr>
<td>L/W = 2.00, 9 RCD, c., two wheels, one truck</td>
<td>160.0</td>
<td>200.0</td>
<td>1.25</td>
</tr>
<tr>
<td>L/W = 2.00, 9 SCD, c., two wheels, one truck</td>
<td>140.0</td>
<td>190.0</td>
<td>1.35</td>
</tr>
</tbody>
</table>

Note: S.S.: Single span bridge; C.: Continuous two span bridge; 1.00 kips = 4.448 kN.
Table 6.4
Effective Moments of Resistance of Sections at Ultimate Load of a Composite Bridge

<table>
<thead>
<tr>
<th>Moments of Resistance (1)</th>
<th>Eccentric Loading</th>
<th>Concentric Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simply Connected Diaphragms (2)</td>
<td>Rigidly Connected Diaphragms (3)</td>
</tr>
<tr>
<td></td>
<td>Same as in Col. (2)</td>
<td>Same as in Col. (2)</td>
</tr>
<tr>
<td>( m_{pl} )</td>
<td>( m_{s} )</td>
<td>( m_{nl} )</td>
</tr>
<tr>
<td>= ultimate positive moment of resistance of composite section/unit width (a)</td>
<td>= ultimate negative moment of resistance of composite section/unit width (b)</td>
<td>= ultimate negative moment of resistance of concrete deck slab/unit length (c)</td>
</tr>
</tbody>
</table>

Note:
(a) Ultimate compressive stress in concrete = 0.85 \( f_c \); ultimate uniform stress in steel section = \( f_y \).
(b) Ultimate stress in top rebars in slab = \( f_{yT} \); ultimate uniform stress in steel section = \( f_y \).
(c) Ultimate stress in top rebars in slab = \( f_{yT} \); ultimate compressive stress in concrete = 0.85 \( f_c \).
(d) Tensile stress at top of slab = concrete modulus of rupture.
Table 6.4 Cont.

Effective Moments of Resistance of Sections at Ultimate Load of a Composite Bridge

<table>
<thead>
<tr>
<th>Moments of Resistance (1)</th>
<th>Eccentric Loading</th>
<th>Concentric Loading</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Simply Connected Diaphragms (2)</td>
<td>Rigidly Connected Diaphragms (3)</td>
</tr>
<tr>
<td>$m_{pl}$</td>
<td>$m_t$</td>
<td>$m_t$</td>
</tr>
<tr>
<td></td>
<td>$=$ ultimate positive moment of resistance of composite Section/ unit width (c)</td>
<td>Same as in Col. (2)</td>
</tr>
<tr>
<td></td>
<td>$=$ ultimate negative moment of resistance of composite Section/ unit width (f)</td>
<td>Same as in Col. (2)</td>
</tr>
</tbody>
</table>

Note: (c) Ultimate compressive stress in concrete $= 0.85 f_c$; ultimate uniform stress in steel section $= f_y$. (f) Ultimate stress in top rebars in slab $= f_{yr}$; ultimate uniform stress in steel section $= f_y$. 
Table 6.5
Elastic and Ultimate Distribution Factors, D and \(D_{ult}\), for Eccentric Loading case in Simply Supported Bridges

<table>
<thead>
<tr>
<th>L/W</th>
<th>Diaphragm Connection to Girder</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td></td>
<td>2.85</td>
<td>3.45</td>
<td>4.50</td>
<td>5.55</td>
<td>6.60</td>
</tr>
<tr>
<td></td>
<td>(4.00)</td>
<td></td>
<td>(4.00)</td>
<td></td>
<td>(4.00)</td>
<td>(5.00)</td>
</tr>
<tr>
<td>2.0</td>
<td>Rigidly Connected Diaphragm</td>
<td>2.95</td>
<td>3.50</td>
<td>4.40</td>
<td>5.45</td>
<td>6.75</td>
</tr>
<tr>
<td></td>
<td>(3.95)</td>
<td></td>
<td>(3.95)</td>
<td></td>
<td>(4.45)</td>
<td>(5.90)</td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td>3.13</td>
<td>3.54</td>
<td>4.29</td>
<td>5.25</td>
<td>6.52</td>
</tr>
<tr>
<td></td>
<td>(4.13)</td>
<td></td>
<td>(4.11)</td>
<td></td>
<td>(4.64)</td>
<td>(6.31)</td>
</tr>
<tr>
<td>1.5</td>
<td></td>
<td>2.80</td>
<td>3.46</td>
<td>4.57</td>
<td>5.56</td>
<td>6.60</td>
</tr>
<tr>
<td></td>
<td>(3.88)</td>
<td></td>
<td>(3.88)</td>
<td></td>
<td>(4.60)</td>
<td>(6.20)</td>
</tr>
<tr>
<td>2.0</td>
<td>Simply Connected Diaphragm</td>
<td>2.90</td>
<td>3.50</td>
<td>4.40</td>
<td>5.50</td>
<td>7.00</td>
</tr>
<tr>
<td></td>
<td>(3.15)</td>
<td></td>
<td>(3.35)</td>
<td></td>
<td>(5.50)</td>
<td>(6.90)</td>
</tr>
<tr>
<td>3.0</td>
<td></td>
<td>2.93</td>
<td>3.40</td>
<td>4.25</td>
<td>5.67</td>
<td>7.73</td>
</tr>
<tr>
<td></td>
<td>(2.57)</td>
<td></td>
<td>(3.19)</td>
<td></td>
<td>(6.34)</td>
<td>(10.10)</td>
</tr>
</tbody>
</table>

Note: \(D_{ult}\) in brackets.
Table 6.6
Elastic and Ultimate Distribution Factors, $D$ and $D_{ult}$, for Concentric Loading on Continuous Two-Span Bridges

<table>
<thead>
<tr>
<th>L/W</th>
<th>Diaphragm Connection to Girder</th>
<th>Girders</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>D</td>
<td>E</td>
</tr>
<tr>
<td>3.0</td>
<td>Rigidly Connected Diaphragm</td>
<td>7.42 (7.64)</td>
<td>7.50 (7.46)</td>
<td>7.69 (7.62)</td>
<td>7.69 (7.62)</td>
<td>7.50 (7.46)</td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td>7.78 (7.85)</td>
<td>7.55 (7.55)</td>
<td>7.46 (7.43)</td>
<td>7.46 (7.43)</td>
<td>7.55 (7.55)</td>
</tr>
<tr>
<td>3.0</td>
<td>Simply Connected Diaphragm</td>
<td>7.44 (7.70)</td>
<td>7.48 (7.46)</td>
<td>7.71 (7.59)</td>
<td>7.71 (7.59)</td>
<td>7.48 (7.46)</td>
</tr>
<tr>
<td>5.0</td>
<td></td>
<td>7.78 (7.82)</td>
<td>7.55 (7.56)</td>
<td>7.46 (7.44)</td>
<td>7.46 (7.44)</td>
<td>7.55 (7.56)</td>
</tr>
</tbody>
</table>

Note: $D_{ult}$ in brackets.
Table 6.7

Elastic and Ultimate Distribution Factors, $D$ and $D_{ult}$, for Continuous 2-Span Bridge Models with Aspect Ratio $(L/W) = 2.0$

<table>
<thead>
<tr>
<th>Diaphragm Connection to Girder</th>
<th>Loading Condition</th>
<th>Girders</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>A</td>
<td>B</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td>Case I</td>
<td>0.76</td>
<td>1.14</td>
<td>2.80</td>
<td>35.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.31)</td>
<td>(1.31)</td>
<td>(1.40)</td>
<td>(7.90)</td>
</tr>
<tr>
<td></td>
<td>Case II</td>
<td>1.11</td>
<td>1.08</td>
<td>1.63</td>
<td>-4.88</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.18)</td>
<td>(1.18)</td>
<td>(1.18)</td>
<td>(-2.44)</td>
</tr>
<tr>
<td></td>
<td>Case III</td>
<td>2.32</td>
<td>1.38</td>
<td>1.38</td>
<td>2.32</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.57)</td>
<td>(1.57)</td>
<td>(1.57)</td>
<td>(1.57)</td>
</tr>
<tr>
<td>RCD</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Case I</td>
<td>0.76</td>
<td>1.08</td>
<td>3.10</td>
<td>50.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.30)</td>
<td>(1.30)</td>
<td>(1.41)</td>
<td>(10.0)</td>
</tr>
<tr>
<td></td>
<td>Case II</td>
<td>1.32</td>
<td>1.02</td>
<td>1.16</td>
<td>-7.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.15)</td>
<td>(1.15)</td>
<td>(1.15)</td>
<td>(-1.83)</td>
</tr>
<tr>
<td></td>
<td>Case III</td>
<td>3.03</td>
<td>1.27</td>
<td>1.27</td>
<td>3.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td>(1.57)</td>
<td>(1.57)</td>
<td>(1.57)</td>
<td>(1.57)</td>
</tr>
<tr>
<td>SCD</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Note: $D_{ult}$ in brackets ( ).

Loadings
- Case I: Wheel loads on Beams A and B.
- Case II: Wheel loads at centre of AB and BC.
- Case III: Wheel loads on Beams B and C.
- RCD: Rigidly Connected Diaphragm Bridge.
- SCD: Simply Connected Diaphragm Bridge.
<table>
<thead>
<tr>
<th>Type</th>
<th>LXW (ft.)</th>
<th>Girder</th>
<th>F.E.M.</th>
<th>OHBDC Da</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>RCD</td>
<td>SCD</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Elastic</td>
<td>Ultimate</td>
</tr>
<tr>
<td>S.S.</td>
<td>40.0 x 28.</td>
<td>Exterior</td>
<td>4.70</td>
<td>4.16</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>4.05</td>
<td>4.16</td>
</tr>
<tr>
<td>C.</td>
<td>150.0 x 50.</td>
<td>Exterior</td>
<td>7.42</td>
<td>7.64</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>7.69</td>
<td>7.62</td>
</tr>
<tr>
<td>S.S.</td>
<td>54.0 x 28.</td>
<td>Exterior</td>
<td>4.25</td>
<td>4.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>4.25</td>
<td>4.30</td>
</tr>
<tr>
<td>S.S.</td>
<td>10.0 x 7.5</td>
<td>Exterior</td>
<td>4.32</td>
<td>3.50</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Interior</td>
<td>3.14</td>
<td>3.50</td>
</tr>
</tbody>
</table>

Note:  
S.S.: Simply Supported Single Span Bridge.  
C.: Continuous Two Span Bridge.  
RCD: Rigidly Connected Diaphragm Bridge.  
SCD: Simply Connected Diaphragm Bridge.
<table>
<thead>
<tr>
<th>No.</th>
<th>Model Description</th>
<th>Type</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td><img src="image1" alt="Image" /></td>
<td>SCD &amp; RCD</td>
<td>$P_c = \frac{1}{2L - x_1 - x_2} \left[ \frac{(2m_{p1}(I_c^2 + S^2) + 4m_{n1}S^2 + m_{nt}L^2 + 2m_{c1c}L)}{L} - \frac{QL}{12} (3I_c^2 - 2S^2) \right] \quad 4.17$</td>
</tr>
<tr>
<td>2</td>
<td><img src="image2" alt="Image" /></td>
<td>SCD &amp; RCD</td>
<td>$P_c = Eq. \ 4.16 + \frac{1}{2L - x_1 - x_2} \left( \frac{m_g}{L} (I_c^2 - S^2) \right) \quad 4.18$</td>
</tr>
<tr>
<td>3</td>
<td><img src="image3" alt="Image" /></td>
<td>SCD</td>
<td>$P_c = \frac{4m_{p1} \frac{W}{L} + m_{n1} \frac{W}{L} + m_{nt} \frac{L}{W} + m_{pt} \frac{L}{W} - \frac{5}{12} q \frac{W}{L}}{2 \left( 1 - 0.25 \frac{a}{W} \right)} \quad 4.28$</td>
</tr>
<tr>
<td>4</td>
<td><img src="image4" alt="Image" /></td>
<td>SCD</td>
<td>$P_c = Eq. \ 4.38 + \frac{1.5 \frac{m_g}{W}}{2 \left( 1 - 0.25 \frac{a}{W} \right)} \quad 4.29$</td>
</tr>
<tr>
<td>NO.</td>
<td>Model Description</td>
<td>Type</td>
<td>Formula</td>
</tr>
<tr>
<td>-----</td>
<td>------------------</td>
<td>------</td>
<td>---------</td>
</tr>
<tr>
<td>5</td>
<td><img src="image1" alt="Diagram" /></td>
<td>RCD</td>
<td>[ P_c = \frac{4 , m_{pl} \frac{W}{L} + 0.5 , m_{nl} \frac{W}{L} + 0.5 , m_{nt} \frac{L}{W} + m_{pt} \frac{L}{2W} - 0.4583 , q , W , L}{3.75} ]</td>
</tr>
<tr>
<td>6</td>
<td><img src="image2" alt="Diagram" /></td>
<td>RCD</td>
<td>[ P_c = \text{Eq. 4.40} + \frac{1.75 , m_s , \frac{W}{L}}{3.75} ]</td>
</tr>
<tr>
<td>7</td>
<td><img src="image3" alt="Diagram" /></td>
<td>SCD</td>
<td>[ P_c = \frac{4 , m_{pl} \frac{W}{L} + 3 , m_{nl} \frac{W}{L} + 3 , m_{nt} \frac{L}{W} + 3 , m_{pt} \frac{L}{W} - 0.25 , q , W , L}{2(1 - 0.75 \frac{a}{W})} ]</td>
</tr>
<tr>
<td>8</td>
<td><img src="image4" alt="Diagram" /></td>
<td>SCD</td>
<td>[ P_c = \text{Eq. 4.38} + \frac{0.5 , m_s , \frac{W}{L}}{2(1-0.75 \frac{a}{W})} ]</td>
</tr>
<tr>
<td>Model Description</td>
<td>Formula</td>
<td></td>
<td></td>
</tr>
<tr>
<td>-------------------</td>
<td>---------</td>
<td>---</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_c = \frac{4 \cdot m_p \cdot l + m_n \cdot l}{3.50} - \frac{m_p \cdot l}{W_L} - 0.4167 \cdot W_L$</td>
<td>4.34</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_c = \frac{1.50 \cdot m_p \cdot W}{L}$</td>
<td>4.35</td>
<td></td>
</tr>
<tr>
<td></td>
<td>$P_c = E_{4.40} + \frac{1.50 \cdot m_p \cdot W}{L}$</td>
<td>3.50</td>
<td></td>
</tr>
</tbody>
</table>

**Type**

<table>
<thead>
<tr>
<th>Type</th>
<th>RCD</th>
<th>RCD</th>
</tr>
</thead>
</table>

**Model Description**

<table>
<thead>
<tr>
<th>NO.</th>
<th>9</th>
<th>10</th>
</tr>
</thead>
</table>

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### TABLE 6.10

Comparision of Experimental and Theoretical Collapse Loads

<table>
<thead>
<tr>
<th>Bridge Model</th>
<th>Experimental (kips)</th>
<th>Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Finite Element (kips)</td>
</tr>
<tr>
<td>I S.S., SCD</td>
<td>84.0</td>
<td>88.0</td>
</tr>
<tr>
<td>II S.S., RCD</td>
<td>120.0</td>
<td>112.0</td>
</tr>
<tr>
<td>III C., SCD</td>
<td>128.0</td>
<td>128.0</td>
</tr>
<tr>
<td>IV C., RCD</td>
<td>140.0</td>
<td>168.0</td>
</tr>
</tbody>
</table>

**NOTE:** S.S.: Simply Supported; C: Continuous 2-Span; SCD: Simply (Bolted) Connected Diaphragm Model; RCD: Rigidly (Welded) Connected Diaphragm.
Table 6.11
Comparison Between Theoretical Collapse Loads using F.E.M. and the developed Yield Line Formulae.

<table>
<thead>
<tr>
<th>Bridge Model Description</th>
<th>Collapse Load</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>F.E.M. kips</td>
<td>Yield Line Theory kips</td>
</tr>
<tr>
<td>L/W =1.0, Single Span, RCD Type, Side Load, Two Wheels (One Truck). (L = 7.5 ft, W = 7.5 ft)</td>
<td>150.0</td>
<td>142.0</td>
</tr>
<tr>
<td>L/W =1.25, Single Span, RCD Type, Side Load, Two Wheels (One Truck). (L = 9.38 ft, W = 7.5 ft)</td>
<td>120.0</td>
<td>121.5</td>
</tr>
<tr>
<td>L/W =1.50, Single Span, RCD Type, Side Load, Two Wheels (One Truck). (L = 11.25 ft, W = 7.5 ft)</td>
<td>104.0</td>
<td>106.0</td>
</tr>
<tr>
<td>L/W =1.33, Single Span, SCD Type, Side Load, Two Wheels (One Truck). (L = 10.0 ft, W = 7.5 ft)</td>
<td>56.0</td>
<td>55.0</td>
</tr>
<tr>
<td>L/W =1.33, Single Span, SCD Type, Centre Load, Four Wheels (Two Trucks). (L = 10.0 ft, W = 7.5 ft)</td>
<td>136.0</td>
<td>134.0</td>
</tr>
<tr>
<td>L/W =1.33, Single Span, RCD Type, Centre Load, Two Wheels (One Truck). (L = 10.0 ft, W = 7.5 ft)</td>
<td>170.0</td>
<td>178.0</td>
</tr>
<tr>
<td>L/W =1.33, Single Span, RCD Type, Centre Load, Four Wheels (Two Trucks) (L = 10.0 ft, W = 7.5 ft)</td>
<td>150.0</td>
<td>150.0</td>
</tr>
<tr>
<td>L/W =1.75, Two Span Cont., RCD Type, Side Load, Two Wheels (One Truck) (L = 5.5 ft, W = 3.13 ft)</td>
<td>180.0</td>
<td>174.0</td>
</tr>
<tr>
<td>L/W =1.50, Two Span Cont., RCD Type, Side Load, Two Wheels (One Truck) (L = 4.5 ft, W = 3.13 ft)</td>
<td>210.0</td>
<td>203.0</td>
</tr>
<tr>
<td>L/W =1.25, Two Span Cont., RCD Type, Side Load, Two Wheels (One Truck) (L = 4.0 ft, W = 3.13 ft)</td>
<td>240.0</td>
<td>224.0</td>
</tr>
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</table>
Table 6.12

Comparison Between the calculated Collapse Load using the Developed Yield Line Formulae and Results in Referenced Literature.

<table>
<thead>
<tr>
<th>Case No.</th>
<th>Ref. No.</th>
<th>Span Length ft.</th>
<th>Bridge Width ft.</th>
<th>mpl K.in/ln</th>
<th>ntm K.in/ln</th>
<th>nml K.in/ln</th>
<th>ms K.in/ln</th>
<th>Concrete Deck Cracked Length (inch)</th>
<th>Type of Support</th>
<th>Experimental Failure Load (kips)</th>
<th>FEM Failure Load (kips)</th>
<th>Yield Line Calculated Failure Load (kips)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>87</td>
<td>60</td>
<td>33.75</td>
<td>470</td>
<td>2.0</td>
<td>88.75</td>
<td>220</td>
<td>15</td>
<td>Single Span Simply Supported</td>
<td>750</td>
<td>760</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>49</td>
<td>45</td>
<td>30.00</td>
<td>335</td>
<td>2.0</td>
<td>65.20</td>
<td>268</td>
<td>20</td>
<td>Continuous Two Span</td>
<td>1485</td>
<td>1495</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>58</td>
<td>15</td>
<td>7.50</td>
<td>13.4</td>
<td>6.24</td>
<td>9.37</td>
<td>90</td>
<td>0.3</td>
<td>Single Span Simply Supported</td>
<td>33</td>
<td>34.2</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>58</td>
<td>15</td>
<td>7.50</td>
<td>13.4</td>
<td>6.24</td>
<td>9.37</td>
<td>90</td>
<td>0.3</td>
<td>Single Span Simply Supported</td>
<td>27.6</td>
<td>28.5</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>53</td>
<td>6</td>
<td>4.00</td>
<td>7.85</td>
<td>1.0</td>
<td>1.60</td>
<td>42</td>
<td>0.2</td>
<td>Single Span Simply Supported</td>
<td>20.9</td>
<td>20.7</td>
<td></td>
</tr>
</tbody>
</table>

Note: 1 kip = 4.45 kN; 1 ft = 0.3048 m.
Table 6.13

Influence of Bridge Aspect Ratio on the Length of the Positive Yield Line Developed at Failure

<table>
<thead>
<tr>
<th>Variable Inputs</th>
<th>Bridge Aspect Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$m_p=380$ k.in/in, $m_{nl}=2.0$ K.in/in, $m_{nl}=90$ K.in/in, Single Span, $m=15.0$ K.in/in, Constant Bridge Width (W) =27 ft., Side Load.</td>
</tr>
<tr>
<td>L/W</td>
<td>1.00 1.50 2.00 2.50 3.00</td>
</tr>
<tr>
<td>L/W</td>
<td>0.65 0.66 0.66 0.67 0.58</td>
</tr>
</tbody>
</table>

Table 6.14

Influence of Bridge Ultimate Positive Moment Capacity on the Length of the Positive Yield Line Developed at Failure.

<table>
<thead>
<tr>
<th>Variable Inputs</th>
<th>Bridge Ultimate Positive Moment Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$m_n=2.0$ K.in/in, $m_{nl}=90$ K.in/in, Single Span, $m=15.0$ K.in/in, Bridge Width (W) =27 ft., Bridge Length (L) = 54 ft., Side Load.</td>
</tr>
<tr>
<td>$m_p$ K.in/in</td>
<td>190.0 285.0 380.0 475.0 570.0</td>
</tr>
<tr>
<td>Lc/W</td>
<td>0.69 0.68 0.66 0.65 0.63</td>
</tr>
</tbody>
</table>
Table 6.15
Influence of Bridge Longitudinal Negative Moment Capacity on the Length of the Positive Yield Line Developed at Failure.

<table>
<thead>
<tr>
<th>Variable Inputs</th>
<th>Bridge Longitudinal Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$m_p=380 \text{ k.in/in, } m_{nt}=2.0 \text{ K.in/in, Single Span, Bridge,}$</td>
</tr>
<tr>
<td></td>
<td>$m_t=15.0 \text{ K.in/in, Bridge Width (W) =27 ft,}$</td>
</tr>
<tr>
<td></td>
<td>$\text{Bridge Length (L) = 54 ft, Side Load.}$</td>
</tr>
<tr>
<td>$m_{nt} \text{ K.in/in}$</td>
<td>45.0</td>
</tr>
<tr>
<td>$L/W$</td>
<td>0.65</td>
</tr>
</tbody>
</table>

Table 6.16
Influence of Bridge Transverse Negative Moment Capacity on the Length of the Positive Yield Line Developed at Failure.

<table>
<thead>
<tr>
<th>Variable Inputs</th>
<th>Bridge Transverse Negative Moment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$m_p=380 \text{ k.in/in, } m_u=90.0 \text{ K.in/in, } m_a=90 \text{ K.in/in, } m_t=15.0 \text{ K.in/in,}$</td>
</tr>
<tr>
<td></td>
<td>$\text{Bridge Width (W) 27 ft, Bridge Length (L) = 54 ft, Single Span,}$</td>
</tr>
<tr>
<td></td>
<td>$\text{Side Load.}$</td>
</tr>
<tr>
<td>$m_{nt} \text{ K.in/in}$</td>
<td>2.0</td>
</tr>
<tr>
<td>$L/W$</td>
<td>0.65</td>
</tr>
</tbody>
</table>
Chapter VII

Summary, Conclusions and Future Research

7.1 Summary

Extensive experimental and theoretical studies were carried out to investigate the behaviour of composite steel-concrete bridges with rigidly and simply connected diaphragms. A detailed literature review was conducted in order to establish the foundation for this study. It was observed that the previously cited literature has always assumed bolted connections between the main girders and the transverse diaphragms. From the theoretical and experimental study, it was found that the rigid diaphragm connection type provides both better service and ultimate load distributions over the bridge; it also has a higher ultimate load carrying capacity when compared with simple diaphragm connection type.

The analytical studies were based on the yield line theory and on the finite
element method. Simplified yield line expressions were developed to predict the collapse load of composite bridges with different diaphragm connection types. Good comparisons between the analytical and the experimental results were noted.

A parametric study was conducted to examine the influence of a number of variables on the load distribution characteristics of composite bridges. The results revealed that the type of connections between the main girders and the transverse members, load locations, aspect ratio of the bridge, sectional moment capacities in the longitudinal and transverse directions, and number of transverse diaphragms used, significantly influence the load distribution in the bridge when subjected to service and ultimate loads.

7.2 Conclusions

Based on the theoretical and the experimental results the following conclusions are drawn:

1- In composite bridges, l-diaphragms rigidly connected to the longitudinal girders are significantly more effective in the transverse load distribution than when the diaphragms are simply connected to the girders. This was demonstrated under
both service and ultimate load conditions.

2- The concrete deck slab plays an important role in the transverse load distribution in a simply connected diaphragm composite bridge than in a rigidly connected diaphragm composite bridge; it is essential, therefore, to have a healthy deck slab in simply connected diaphragm composite bridges in order to secure the expected ultimate load capacity of the bridge.

3- Rigidly connected diaphragm composite bridges are stiffer and stronger than simply connected diaphragm bridges.

4- Rigidly connected diaphragms can substantially increase the transverse rigidity of the bridge and therefore can lead to a more economical design of the longitudinal girders.

5- The failure pattern of eccentrically loaded simply connected diaphragm composite bridges is more localized than that of rigidly connected diaphragm bridges. As a result, the former has a smaller capacity to mobilize its negative moment of resistance when compared to the latter.

6- Considerably larger deflections can be expected in eccentrically and concentrically loaded simply connected diaphragm bridges when compared to rigidly connected diaphragm bridges. Thus, the stiffness and the ultimate load
capacity of the latter are much higher than those of the former.

7- The proposed yield line formulae can effectively predict the collapse load provided the contribution of the negative moment of resistance of the composite bridge is assessed correctly.

8- The bridge aspect ratio has a significant influence on the load distribution factors at the ultimate load.

9- The behaviour of the load distribution factors of continuous two-span composite bridges subjected to service and ultimate loads is not significantly different from those of simple-span bridges.

10- The degree of eccentricity of the applied load measured from the longitudinal centre line of the bridge influences the differences in the girder load distribution factors; such influence is greater for simply connected diaphragm bridges.

11- There is an optimum number of rigidly connected diaphragms for improving the load distribution characteristics of a bridge beyond which no improvement should be expected.

12- The estimated load distribution D factors in the OHBDC appear to be overly conservative in some cases.

13- L-diaphragms rigidly connected to the longitudinal girders can be used
effectively for bridge rehabilitation as well as for increasing the posted load rating of existing bridges.

7.3 **Recommendations for Future Research**

It is recommended that further research efforts be directed towards the following:

1- The study of the ultimate load behaviour of composite steel-concrete skew and curved bridges with simple and rigid diaphragm connections.

2- The development of empirical expressions for load distribution at the ultimate limit state for composite steel-concrete bridges, as a function of: aspect ratio, ratio between the main and secondary member stiffness, spacings of girders, and connection types between the main beams and the transverse diaphragms.

3- Testing to failure of other bridge modes with concentric and eccentric loadings in order to expand the available experimental data base.
References
References


70- Schaeffer, H. G., MSC/NASTRAN Primer, Static and Normal Modal Analysis, Schaeffer Analysis, Inc., Kendall Hill Road, Mont Veron, New Hampshire, 03057, 1982.


Appendix A-1

Moment-Carrying Capacity of Composite Bridges

The calculations of the ultimate flexural moment capacities of the composite bridges are based on the following assumptions:

1. Initially plane sections remain plane after bending.
2. At the ultimate capacity all steels reach their yield stress.
3. The concrete stress-strain relationship is known.
4. Concrete is ineffective in tension.
5. Slip between the main longitudinal steel girders and the concrete slab is negligible.

The ultimate flexural moment capacity for a composite section is calculated based on the material stress-strain relationship and on the requirement that normal forces acting on the section must always remain in equilibrium.
I. Ultimate Positive Moment

a) Neutral axis in the slab (see Fig. A1.1):

\[ N_c = 0.85 f_c b h \quad N_r = A_r f_r \]

\[ N_s = A_s f_y \quad A_s f_y = 0.85 f_c b h + A_r f_r \]

\[ h = (f_s A_s f_y)/(0.85 f_c b) \] (for neutral axis to be in the slab i.e. \( h < d \))

where,

- \( N_c \) = Compressive normal force acting on the concrete.
- \( N_s \) = Steel beam ultimate normal force.
- \( N_r \) = Steel reinforcement normal force.
- \( f_c \) = Concrete material compressive strength.
- \( f_y \) = Steel material yield strength.
- \( f_r \) = Reinforcing steel material yield strength.
- \( A_s \) = Steel beam cross section area.
- \( A_r \) = Steel reinforcement area.
- \( b \) = Width of composite beam flange.
- \( h \) = Neutral axis depth measured from the top surface of the concrete.
- \( d \) = Concrete slab thickness.

Hence, the ultimate positive moment capacity, \( M_{up} \), of the composite section with
the neutral axis in the slab may be expressed as:

\[ M_{up} = A_s f_y (h_s - 0.5h) + A_{tr} f_{tr} (0.5h - h_r) \]

b) Neutral axis in the beam (see Fig. A1.2):

\[ N_c = 0.85 f_c bd \quad N_{tr} = A_{tr} f_{tr} \]
\[ N_{sc} = A_s^* f_y \quad N_{st} = 2N_{sc} + N_{tr} + N_c \]
\[ A_s f_y = 2A_s^* f_y + A_{tr} f_{tr} + 0.85 f_c bd \]
\[ A_s^* = (A_s f_y - A_{tr} f_{tr} - 0.85 f_c bd)/(2f_y) \]

where

\( A_{sc} \) = Area of the compression part in the steel beam.

\( A_s^* \) = Area of the tension part in the steel beam.

\( N_{sc} \) = Steel beam ultimate compressive normal force.

\( N_{st} \) = Steel beam ultimate tensile normal force.

\( h_r \) = Reinforcement bars location measured from the top surface of the concrete.

\( h_s^* \) = Steel section compressive force location measured from the top surface of the concrete.

Hence, the ultimate positive moment capacity, \( M_{up} \), of the composite section with the neutral axis in the steel beam may be expressed as:
\[ M_{up} = A_t f_y (h_t - 0.5d) + A_r f_y (0.5d - h_t) - 2A_{s'} f_y (h_{s'} - 0.5d) \]

**II. Ultimate Negative Moment** (see Fig. A1.3)

\[ N_{st} = A_{s'} f_y \]
\[ N_{sc} = A_{s} f_y \]
\[ N_{ct} = 2N_{st} + N_r \]
\[ N_r = A_r f_y \]
\[ A_{s'} = \frac{(A_{s} f_y - A_r f_y)}{2f_y} \]

Hence, the ultimate negative moment capacity, \( M_{un} \), of the composite section may be expressed as:

\[ M_{un} = A_t f_y (h_t - h_r) - 2A_{s'} f_y (h_{s'} - h_r) \]

From which, the section flexural moment capacity per unit width may now be expressed as:

\[
\text{moment / unit width} = \frac{\text{Number of Composite beam sections} \times \text{section Capacity}}{\text{Bridge Width}}
\]
Fig. A1.1 Positive Ultimate Moment Capacity of Composite Beam Section with Neutral Axis in the Slab.
Fig. A1.2 Ultimate Positive Moment Capacity of Composite Beam Section with Neutral Axis in the Steel Beam
Fig. A1.3 Ultimate Negative Moment Capacity of Composite Beam Section
Appendix A-2

Yield Line Formulae Optimization

It is observed from Chapter IV that the collapse load, $P_c$, in the eccentric load case is a function of the positive yield line length $l_c$. The minimum collapse load can then be determined mathematically from the stationary condition applied to the derived equations as:

$$\frac{\delta P_c}{\delta l_c} = 0$$

Since the $l_c$ parameter appeared in the denominator and numerator of Eqs. 4.17 and 4.18, the first derivative may be expressed in the following fashion;

let, $$P_c = \frac{u}{v}$$

Hence,

$$\frac{dP_c}{dl_c} = \frac{v \frac{du}{dl_c} - u \frac{dv}{dl_c}}{v^2}$$  \(\text{A2.1}\)

From Eq. 4.18 let,

$$v = (2l_c - (x_2 + x_2))$$

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If \( X_o = x_1 + x_o \), the above equation may be re-expressed as;

\[
\nu = (2I_c - X_o)
\]

The differentiation of Eq. 4.16 with respect to \( I_c \) would lead to the following expression;

\[
\frac{\delta P_c}{\delta I_c} = 0 \quad \text{(A2.2)}
\]

This may be expressed as;

\[
A I_c^2 + B I_c + C = 0 \quad \text{(A2.3)}
\]

where \( A, B \) and \( C \) were found to be;

\[
A = 4m_{pr} + 2m_x - L^2q
\]

\[
B = 0.5 (L^2qX_o) - 2m_xX_o - 4m_{pr}X_o
\]

\[
C = 2m_xS^2 - 4m_{pr}S^2 - 2m_xL^2 - L^2SqX_o - 8m_{pr}S^2 - 2m_xLX_o
\]

Knowing \( A, B \) and \( C \), the general solution of equation A2.1 for the minimum collapse load, \( P_c \), may be expressed as;

\[
I_c = \frac{-B \mp \sqrt{B^2 - 4AC}}{2A} \quad \text{(A2.4)}
\]
Appendix A-3

Measuring Equipment

I. Load Cells

The load application was monitored by the use of full bridge, flat load cells.

The elastic-load testing was monitored using a 50 kip (222 kN) capacity load cell; a 200 kip (888 kN) capacity load cell was used in the post-elastic load testing. The cells were calibrated several times during the testing process, Figs. A3.1 and A3.2.

II. Strain Gauges

As indicated in chapter IV, two types of strain gauges for the steel girders and the concrete slab, were used to measure the strains in the bridge. The placement of these strain gauges relied on Maxwell’s reciprocal theorem and the symmetry of the model, Fig. 4.6. In order to ensure uniformity of electric resistance wiring of all gauges was made using the same length regardless of their location with respect to the Data Acquisition System.
a) Concrete Gauges:-

Strain gauges were placed in pairs at the locations shown in Fig. 4.6. Each pair consisted of one gauge running in the longitudinal and the other in the transverse direction. The concrete surface at these locations was prefinished to mount the gauges in accordance with the requirements for concrete gauge application. The gauges were of the electronic foil gauge type N11-FA-30-120-11 with an average resistance of 119.8 Ohms and a gauge factor of 2.16.

b) Steel Girders Gauges:-

Figure 4.6 shows the location of the gauges on the steel. Foil gauges with 10 mm length were used here of the type N11-FA-10-120-11 with an average resistance of 119.8 Ohms and 2.12 gauge factor.

III. Data Acquisition System (D.A.S.)

Figure A3.3 presents the layout of a half bridge circuit used to measure the strain readings of the gauges through the Optilog 200 D.A.S. The D.A.S was able to read up to 200 strain gauges readings in 1/20 of a second and store them on a PC disk. Such an advantage enables the user to collect the exact readings of all strain gauges at the moment of load application.
IV. Dial Gauges

Deflections were measured using conventional type dial gauges of 1/1000 accuracy factor. The gauges were placed under the model at points where strain gauges were placed. The reading of these gauges were manually taken.
Fig. A3.1 200 kips Load Cell Calibration Chart
Fig. A3.2. 50 Kips Load Cell Calibration Chart
Fig. A3.3  OPTILOG 200 System Circuit
Appendix A-4

Design Example

It is required to estimate the ultimate moments of resistance for a continuous two-span composite bridge. The relevant data are:

Length of each span = \( L \) = 80 ft (24.4 m)

Width = three 12 ft (3.75 m) traffic lanes

Total bridge width (W) = 42 ft (12.8 m)

Uniformly Distributed Dead Load (q)

= 0.2 kip/ft\(^2\) (9.6 kPa)

AASHTO HS20 truck loading with \( d_1 = d_2 = 14 \) ft (4.27 m)

\[
\begin{align*}
\frac{m_a}{m_{pl}} &= 0.80 \\
\frac{m_{pl}}{m_{pl}} &= 0.10 \\
\frac{m_{w}}{m_{pl}} &= 0.10 \\
\frac{m_{s}}{m_{pl}} &= 1.00 \\
\frac{m_{l}}{m_{pl}} &= 0.01
\end{align*}
\]
Use transverse steel beams rigidly connected to the longitudinal steel beams.

Based on AASHTO Bridge Specifications (2):

Live Load Factor

\[ = 1.3 \times 1.67 \times (1 + \text{impact factor}) \]

\[ = 2.17 \times (1 + 50/(80+125)) \]

\[ = 2.70 \]

If eccentric loading is a critical case for design, then it is reasonable to expect that only one HS20 truck can be accommodated in that region.

From Eqs. 4.39 and 4.41 with;

\[ Q = 32 \text{ kip (142.2 kN)} \]

\[ P_l = (1.125 - 70/320) \times (2.70) \times (32) = 78.3 \text{ kip (348.4 kN)} \]

The factored dead load \( q_{dl} \)

\[ = (1.3)(0.2) = 0.26 \text{ kip/ft}^2 \times (12.4 \text{ kPa}) \]

Using Eq. 4.17, with;

\[ P_l = P_c = 78.3 \text{ kip (348.4 kN)} \]

\[ q_{dl} = 0.26 \text{ kip/ft}^2 \times (12.4 \text{ kPa}) \]

\[ x_1 = 3 \text{ ft (0.91 m)} \]

\[ x_2 = 3 + a = 3 + 6 = 9 \text{ ft} \]
Several starting values for $l_c$ and the assumed moment ratios, show that such eccentric loading is not the critical one for this design when compared to a 3-lane truck loading.

Thus, using Eq. 4.31 with $m = 0.90$ yields:

$m_{pl} = 233 \text{ kip.ft/ft (1037 kN.m/m)}$ which becomes;

$m_{pl} = 259 \text{ kip.ft/ft (1153 kN.m/m)}$ after applying the capacity reduction factor

$\phi = 0.90$

Thus, $m_{pl} = 259 \text{ kip.ft/ft (1153 kN.m/m)}$ is the design moment. Hence, the other moments can now be estimated from the $m_{pl}$ and the assumed moment ratios.

Preliminary calculations show that the following bridge geometry is adequate to resist the above moments:

- In the longitudinal direction, use a composite section of W30 x 132 A36 steel with 9 inch (229 mm) thick concrete slab with $f_c' = 4 \text{ ksi (28 MPa)}$; six W30 x132, spaced 8.0 ft (2.45 m) apart will be required.

Resisting moment $m_{pl} = 306 \text{ kip.ft/ft (1362 kN.m/m)}$ versus 259 kip.ft/ft (1153 kN.m/m) required.

- At the intermediate pier support use the same steel section with 1 inch diameter rebars at 6 inch (152 mm) spacing as top slab reinforcing with $f_p=50 \text{ ksi (345}$
MPa) steel.

- In addition a 10.5 inch x 1 inch (267 x 25 mm) thick steel plate (A36 grade) welded to the bottom flange of each beam is required.

Resisting \( m_s = 303 \text{ kip.ft/ft (1348 kN.m)} \) versus 259 kip.ft/ft (1153 kN.m/m) required.

- Also, in the longitudinal direction the moment \( m_{pl} \) provided by the steel beams in composite action with the slab is 207 k.ft/ft (921 kN.m/m) versus 0.8 \( m_{pl} = 207 \) k.ft/ft (921 kN.m/m).

- In the transverse direction, use 13 of W24 x 94 A36 steel diaphragms rigidly connected to the longitudinal steel beams; one diaphragm is placed at each support plus five diaphragms in each span spaced approximately 13 ft (3.97 m) apart. Use 3/4 in. (20 mm) diameter rebars at 6 inch (152 mm) spacing top and bottom of the concrete deck slab.

- Both moments \( m_{nt} \) and \( m_{pt} \) provided by the diaphragm steel beams and reinforced concrete deck slab are 25.6 k.ft/ft (114 kN.m/m) versus 

\[ 0.1 \, m_{pl} = 25.9 \text{ k.ft/ft (115 kN.m/m) required.} \]
Appendix A-5

Concrete Mix Design and Cylinders
Test Results

The concrete mix design for the laboratory composite bridge models was based on the design guidelines given by the Canadian Portland Cement Association (73). The following is a typical calculation for one of the bridge models.

Bridge model description:

Width : 7.50 ft (2286 mm)
Length : 10 ft (3048 mm)
Slab thickness : 2 inch (51 mm)

Volume of concrete required:

Bridge Slab Deck = 2286 mm X 51 mm = 0.373 m³
4 test cylinders = 0.0655 X 4 = 0.022 m³
Total = 0.395 m³
Batch Volume = 0.400 m³
High Early Strength Portland Cement (Type 30) was used. The mix was designed for 4 inch (100 mm) slump and 4500 psi (30 MPa) 7 days ultimate compressive strength ($f'_{cu}$). The design parameters are as follows:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>Type 30</td>
</tr>
<tr>
<td>Slump</td>
<td>4 inch (100 mm)</td>
</tr>
<tr>
<td>Maximum Aggregate</td>
<td>3/8 inch (10 mm)</td>
</tr>
<tr>
<td>Water/Cement Ratio</td>
<td>0.5</td>
</tr>
<tr>
<td>Water requirement</td>
<td>225 kg/m³</td>
</tr>
<tr>
<td>Cement required</td>
<td>450 kg/m³</td>
</tr>
<tr>
<td>Modulus of Fineness</td>
<td>2.40</td>
</tr>
<tr>
<td>Coarse Aggregate required</td>
<td>800 kg/m³</td>
</tr>
<tr>
<td>Fine Aggregate required</td>
<td>810 kg/m³</td>
</tr>
</tbody>
</table>

Hence for 0.4 cubic meters:

- Water = 90 kg
- Cement = 180 kg
- Coarse Aggregate = 320 kg
- Fine Aggregate = 324 kg

This leads to the following mix proportion:

\[(W : C : AC : AF) = (1 : 2 : 3.55 : 3.60)\]

The testing of the concrete cylinders resulted in an average 28 days compressive strength of $f'_{cu}$ = 5300 psi (35 MPa)
Fig. A5.1  Concrete Mix Testing
Appendix A-6

Bolted Connection Design

The bolted connections were designed based on CAN3-S16.1-M78 (6) guidelines. The following are typical design steps for the bolted connection of the single span bridge model I.

1. **Web area** \( A_w = 5.0 \text{ in} \times 0.23 \text{ in} = 1.15 \text{ in}^2 \) (742 mm²)

2. **Shear Stress** \( F_s = 0.66 \times f_y = 0.66 \times 43.5 \text{ ksi} \) (300 MPa)

   \[ = 28.7 \text{ ksi} \] (200 MPa)

3. **Web Shear Resistance** \( V_r = \phi \times A_w \times F_s \)

   \[ = 0.9 \times 1.15 \times 28.7 \]

   \[ = 29.7 \text{ Kips} \] (132.1 kN)

Select 2, 1/2 in (12.7 mm) bolts with 1.6 in (40 mm) distance from the centre of the bolt to the edge of the web and angle used. For illustration see Figures A6.1 and A6.2.
(4) Bearing resistance of the web $B_r$

\[ B_r = \phi X t X n X e X f_u \]
\[ = 0.67 X 0.23 X 2 X 1.6 X 65 \]
\[ = 32 \text{ Kips (140 kN)} \]

or

\[ B_r = 3 \times \phi X t X n X d X f_u \]
\[ = 3 \times 0.67 X 0.23 X 2 X 0.5 X 65 \]
\[ = 30 \text{ Kips (133 kN)} \]

Choose 2, 3 X 2 X 3/16 (75 X 50 X 5) angles, back to back

Bearing area $A_b = 2 \times 3/16 \times 0.5$

\[ = 0.1875 \text{ in}^2 (127 \text{ mm}^2) \]

(5) Bolts shear resistance

\[ V_r = 0.6 \times \phi_b X n X m X A_b X f_u \]
\[ = 0.6 \times 0.67 X 2 X 2 X 0.1875 X 120 \]
\[ = 36.20 \text{ Kips (161 kN)} \]
Fig. A6.1 Simply Supported Bridge Model, Bolted Connection Details
Fig. A6.2 Simply Supported Bridge Model, Bolted Diaphragm Details.
Appendix A-7

Design of Shear Connectors

A 1/2 inch (12.7 mm) diameter and 1 1/4 in (31.8 mm) stud was used in all the tested bridge models I to IV. The shear resistance for each stud $Q_s$ may be calculated according to CAN3-S16.1-M78 (6), clause 17.3.6(a) as the lesser of the following:

$$Q_s = 0.5 \phi_s A f_c E_c$$ \hspace{1cm} \text{(A7.1)}

or

$$Q_s = 450 \phi_s A$$ \hspace{1cm} \text{(A7.2)}

where, $\phi_s = 0.8$, $A =$ Area of the stud, $E_c =$ Concrete material modulus of elasticity, $f_c =$ Concrete material compressive strength at the time of testing.

From Eqs. A7.1 and A7.2, for the tested single span bridge models I and II,
Q_r = 10.25 kips/stud or 45.60 kN/stud

The shearing force between the concrete deck slab and the steel girders V may be calculated as:

\[ V = \phi A_s f_y \]  \hspace{1cm} A7.3

where, \( A_s \) = Area of the main girder, \( f_y \) = Steel girders material yield strength, and \( \phi = 0.9 \).

From Eq. A7.3, for the tested single span bridge models I and II,

\[ V = 173 \text{ kips (769.5 kN)} \]

Hence; the required number of studs = \( 2V/Q_r = 34 \) Studs
Appendix A-8

Supplier Information Sheet
<table>
<thead>
<tr>
<th>CUST ITEM</th>
<th>OUR ITEM</th>
<th>IDENTIFICATION OF FIELD</th>
<th>MATERIAL</th>
<th>NUMBER OF PIECES</th>
<th>THICKNESS OF TEST</th>
<th>CONDITION OF TEST</th>
<th>TENSILE PROPERTIES</th>
<th>IMPACT PROPERTIES</th>
</tr>
</thead>
<tbody>
<tr>
<td>7081B</td>
<td>5140</td>
<td>2959</td>
<td>2.30</td>
<td>45910</td>
<td>71150</td>
<td>21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7081B</td>
<td>C</td>
<td>10.09.5</td>
<td>0.0239</td>
<td>0.016.51</td>
<td>0.25</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Vita Auctoris

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