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Assessing the needs for intermediate diaphragms in prestressed concrete girder bridges

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ASSESSING THE NEEDS FOR INTERMEDIATE DIAPHRAGMS IN PRESTRESSED CONCRETE GIRDER BRIDGES

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ABSTRACT

Reinforced concrete Intermediate Diaphragms (IDs) are still being used in prestressed concrete (PC) girder bridges in Louisiana. Some of the advantages of providing IDs are disputed and also use of IDs increases the cost and time of construction. There is no consistency in the practice of providing IDs among various states and codes of practice and overall there is a lack of clarity on the effectiveness of IDs and their needs in prestressed concrete bridges.

The objectives of this research are to assess the need of reinforced concrete (RC) IDs in PC girder bridges and to determine their effectiveness and also to search for an alternative steel diaphragm configuration which would be as effective as RC IDs and could replace them if necessary. Systematic parametric studies for various bridge configurations, which are representative of an entire range of bridge geometries with different parameters, are analyzed through simplified and solid finite element models, which were already calibrated under live loads. This study was carried out on right and skewed bridges which are simply supported and continuous. A reduction factor which could be multiplied by a load distribution factor to account for the influence of the diaphragm in load distribution was developed. To assess the effectiveness of various diaphragms in protecting the girders against the lateral impact and to determine the design forces in the steel bracing members during construction of deck, a finite element analysis was carried out using 3-D solid models.

The results from the parametric studies indicated that several parameters such as skew, span length, spacing, stiffness of diaphragm and girder have different levels of influence on the effectiveness of diaphragms in live load distribution for bridges.

Correction factors were developed which could quantify the ID influence on load distribution. Results from various studies indicated that a steel diaphragm section can possibly replace the RC steel diaphragms.

1. INTRODUCTION

1.1. Background

AASHTO Standard Specifications (2002) defines diaphragms as a transverse stiffener which is provided between girders in order to maintain section geometry. There has been a long standing debate on the need for intermediate diaphragms (IDs) in bridges and still the controversy exists. Some of the reasons in favor of using a diaphragm are that it prevents the girders from twisting during the process of construction and helps in distributing vertical live loads between girders, when under service. The major disadvantage of having a diaphragm is that it increases the cost and time of construction. Also there is controversy on the effectiveness of IDs during an impact caused by over height trucks, with some reports stating it would distribute the lateral impact loads, thereby protecting the girder, while some state that diaphragms would rather spread the damage.

The AASHTO Standard Specifications recommends that IDs be used at points of maximum positive moment for spans exceeding 12m (40ft) and also states that diaphragms contribute to the load distribution among the girders. AASHTO LRFD (2004) also recommends the use of diaphragms in similar lines. But while developing load distribution formulas the effect of diaphragms was not considered in both these codes because of existing discrepancy on the effectiveness of diaphragms. Another reason for this was accounting the influence of diaphragms which is a function of several bridge parameters would have made the currently existing load distribution formulae more complex.

There is inconsistency in practice for design of IDs and the guidelines specified in different states. Texas has eliminated the practice of using IDs. In Florida diaphragms are not required to be provided for non-skewed bridges. Iowa the practice is to use reinforced concrete (RC) IDs in the case where traffic flows under the bridge and to permit the usage of steel diaphragms in prestressed concrete (PC) bridges for the case where there is no traffic flowing under the bridge (Andrawes 2001). Similarly, inconsistency exists in the guidelines that are put forward by the local state departments of transportation in different states in terms of the number of IDs provided, type, depth and their connection to the girder,. In Louisiana, the current practice is to provide RC IDs for PC bridges.

1.2. Need for Study

Providing RC IDs increases the cost and also time of construction of PC bridges. Current literature indicates that still controversy does exist on the need for providing intermediate diaphragms, hence there is a greater need to reassess the need for intermediate diaphragms. Even if the diaphragms are needed, current codes which mandate provision of diaphragms do not include the influence of diaphragms in load distribution for bridge design. Considering the contribution of diaphragms in load distribution could alter the design load for girder which could in turn potentially influence the cost of the structure.

Different studies have indicated various degrees of ID effectiveness. Some of the possible sources for this discrepancy could be that these studies considered different stiffness contributions of the diaphragms in load distribution, differences in modeling IDs. In the conventional methods of analysis, like the grillage method, modeling the diaphragms was difficult and inaccurate as the modeling was limited to two dimensions. With the advent of powerful finite element packages and high speed computers, the bridge and its components can be appropriately modeled in 3-dimensions, which would generate more rational results. Using these packages, sensitivity studies could be conducted with more confidence. The need for using these models for reassessing the influence of IDs was advocated by Garcia (1999).

The current design practice is moving towards a more rational approach. A more rational approach would mean trying to include all the possible factors affecting the design by quantifying the effects of these parameters on design. The conventional AASHTO Standard Specifications considers the live load distribution factor (LDF) to be a function of spacing. It has given the LDF as an S over D formula where S is the beam spacing and D is a constant. AASHTO LRFD includes the effect of span length, spacing, relative stiffness of the slab to that of the girder and modular ratio between slab and girder in the calculation of the moment distribution factor. However, AASHTO LRFD does not include all the factors effecting the load distribution such as the effects of a haunch, parapet stiffening, bearing stiffening and diaphragm stiffening in the LDF being used. Among these factors the contribution of IDs in load distribution could be significant and it has the potential to reduce the LDF and maximum strain up to 30% for interior girders if the diaphragm-girder connection is absolutely rigid (Cai et al. 2002).

As discussed earlier, providing RC IDs increases the cost of construction. To reduce costs there is a need to look for an alternative steel configuration which can potentially perform as good as RC diaphragms and could replace them if the provision of IDs is found to be necessary. These reasons necessitate a further study to assess the need for intermediate diaphragms and their influence on prestressed concrete girder bridges.

1.3 Objectives

The objectives of this study could be summarized as follows

- To quantify the influence of intermediate diaphragm on load distribution factor

- Propose an alternate steel ID which could replace steel ID and to check its adequacy
- Compare the stability provided by steel ID to RC ID at the time of construction
- Assessing the ID influence on bridge performance during impact caused by over-height trucks

1.4. Scope of Study

This study is limited to simply supported and continuous, straight slab on girder bridges, both with and without skew. Only the common type of AASHTO girders and Bulb-T sections with the dimensions specified in the Louisiana Bridge Design Manual are taken in the current study. Box girder and curved girder bridges were excluded from this study as they have special requirements regarding IDs.

One of the important components of current study is to determine the effect of diaphragms on the vertical live load distribution of the bridges. The influence of IDs on load distribution was not included in the AASHTO LRFD as the effectiveness of IDs has been observed to be controversial. The lack of a uniform practice and policy regarding intermediate diaphragms among different states, and its dependence on various bridge parameters are less understood. Therefore in order to understand the influence of various parameters on load distribution, a parametric study was conducted. In this bridges of various configurations were analyzed and the results of these have been used in deducing formulas for IDs influence on load distribution.

The diaphragms connection to girders is a cold joint with the connection through rebars. Because of the possible cracking at the ends of diaphragm at higher loads, the entire section does not contribute to the stiffness thereby reducing the effectiveness of diaphragms. In the past no significant work has been done in quantifying the stiffness contribution of diaphragms in load distribution. In this work, a relation between the effective stiffness of diaphragms influencing the load distribution to the LDF was developed.

In this work an alternative configuration of steel diaphragms was determined which could potentially replace the reinforced concrete (RC) IDs and could provide a stiffness greater than the target stiffness value, which was taken as 40 percent of the absolute stiffness of RC diaphragm. The configurations of IDs considered where channel section placed horizontally connecting the girder webs and X type bracing with a bottom strut based on girder geometry. The stiffness contribution of these steel diaphragm configurations were calculated and their influence in load distribution was also determined.

Along with this work, a study was made to assess how different diaphragms affect the bridge performance under impact of over height trucks at the bottom of girders. Also, design forces developed in the steel bracing members during construction of deck are determined by carrying out a finite element analysis using a 3-D solid model to check whether the bracing members could carry the loads coming into it during construction.

2. LITERATURE REVIEW

One of the important components of this research was to do a thorough review of existing literature on various aspects of intermediate diaphragms for prestressed concrete girder bridges. This was done both at the time of proposing the project for LADOTD (Louisiana Department of Transportation and Development) and also while working on the project, to get a better understanding of issues related to intermediate diaphragms in PC bridges. Literature review was also done on load distribution factor, diaphragm effects in steel bridges, and finite element modeling.

Debate on the need for IDs in slab over prestressed concrete girder bridges started in late 1960's with some studies stating that IDs in some cases are counterproductive. The first of the reports which raised the question on the need for IDs in PC bridges was the report by Lin and Van Horn (1968). From the results obtained from the field tests conducted on a bridge in Philadelphia, they reached the following conclusions. Diaphragms were found to transmit the loads laterally, but when various lanes were loaded at the same time, the experimentally determined distribution factors were not appreciably affected and the deflections in the girder reduced slightly with the provision of IDs in bridges, thereby putting the advantage of IDs in load distribution into question.

Sithichaikasem and Gamble (1972) carried out parametric study for various bridge geometries for simply supported right bridges, to understand the influence of IDs in PC bridges. The parameters considered in this study are

- Aspect ratio which is the ratio between girder spacing to span length
- Relative flexural stiffness parameter which is the ratio between composite girder stiffness to deck stiffness

- Relative torsional stiffness which is the ratio between the torsional stiffness to flexural stiffness of the girder
- Wraping stiffness which is the ratio of the warping rigidity of the girder to the product of the square of the span of the bridge and the torsional rigidity of the girder and
- Relative flexural stiffness of diaphragm which is the ratio of the flexural stiffness of diaphragm to that of the girder.

Charts were plotted between the moment coefficients versus girder spacing which includes the variation in the number and location of diaphragms and also for different positions of live load. Some of the observations they made are

- When the loading is close to exterior girders, the diaphragms increases the controlling moment hence prove to be detrimental while for other cases it may either be helpful or harmful.
- The influence of number of diaphragms is insignificant and
- The diaphragm must be of correct flexural stiffness to be effective, otherwise any increase in diaphragm stiffness beyond a particular limit would increase the girder moments.

Similar kinds of results were obtained by Kostem and DeCastro (1977) who performed a finite element analysis on two existing simple span non-skewed PC girder bridges. Other interesting observation they made was that only 20-30 % of the stiffness of RC diaphragms contribute to load distribution.

Wong and Gamble (1973) also considered the same parameters as Sithichaikasem and Gamble (1972), in determining the effect of diaphragms and continuity on load distribution in straight continuous bridges. The conclusions of this research were

- Changes in maximum moments are not sensitive to diaphragm stiffness and the effects of diaphragms are more pronounced in simply supported bridge than in continuous bridges.
- The effects of continuity tend to cause a greater reduction in the maximum positive moment in the edge girder than in interior girder.
- Bridges with spacing to span ratio less than 0.05, presence of diaphragm may do more harm than good
- Except for temporary erection purposes diaphragms are not required in straight bridges.

Sengupta and Breen (1973) have conducted experimental tests on four test bridges, which were scaled down to 1/5.5 ratio with the variables being length, skew and location of intermediate diaphragms for simply supported bridges under both static and dynamic loading in vertical and lateral direction. The testing was done with and without diaphragms under cyclic and impact loads. They have observed similar pattern of results as their earlier researchers. Coming to the dynamic behavior Sengupta and Breen stated that, when bridges are subjected to cyclic load, IDs did not influence the natural frequency of the bridges and no effect was observed on damping coefficient of bridge vibration. And for bridges under lateral impact revealed that diaphragms reduce the energy absorption capacity of the girders, which would make the girders more vulnerable to damage under lateral impact.

In later part of 1980's, Florida Department of Transportation (FDOT) reconsidered its long-standing practice of using IDs in PC bridges. It was concluded that the cost and time saved by not having to build diaphragms far outweighed the benefits when IDs are used (Garcia 1999). The FHWA reviewed and approved the FDOT petition for eliminating the IDs on tangent bridges, but with a caveat that, for bridges where diaphragms are not provided the design live load of girders needs to be increased by 5% which was adopted by FDOT.

Cheung et al. (1986) found that previous researchers disagreed not only on the effectiveness of IDs in the lateral distribution of vertical live loads, but also on the optimal position of the IDs. They have proposed the need for carrying out a through theoretical study of structural behavior of diaphragms in slab over girder bridges.

Abendroth et al. (1995) summarized the survey results of various bridge design agencies and analytical and experimental investigations of a full scale PC girder slab bridge with different diaphragm configurations. The objective of the research was to determine the effectiveness of IDs in distributing lateral loads and to determine whether a steel diaphragm can replace a regular RC diaphragm. A bridge model was constructed and was tested for different diaphragm configurations at midspan and mid third positions and also without diaphragms. A finite element model was built for the experimental bridge with different diaphragm configurations. It was concluded that the vertical load distribution is independent of the type and location of the IDs; the horizontal load distribution is a function of the type and location of the IDs and the steel diaphragms can essentially replace RC IDs. It was also observed that a diaphragm with X bracing and a bottom strut was structurally equivalent to RC IDs.

Griffin (1997) researched the influence of IDs on the load distribution in PC girder bridges. The experimental study was done on two similar bridges with 4 spans and 50 degrees skew angle which carry excessive loads due to the passage of coal haul trucks. For one of the bridges, RC IDs were provided while for other, steel diaphragms were provided and the bolts connecting the diaphragms to girder were loosened after setting of slab concrete, which leaves the diaphragm ineffective. Bridges of similar design along coal haul routes have experienced unusual concrete spalling at the interface of the diaphragms and girder bottom flange which furthered the rate of deterioration. Both static and dynamic field testing was conducted on these two bridges. The finite element models were calibrated using the experimental results. Later an analysis was conducted with actual coal haul truck traffic to investigate the load distribution and the cause of the spalling at the diaphragm-girder interface. Based on the results obtained from this research, it was concluded that presences of IDs does not cause much significant advantage in structural response. Although large differences were noted percentage wise between responses of the two bridges, the analysis suggested that the bridge without IDs would experience displacements and stresses well within AASHTO and ACI design requirements. One of the major reasons for the spalling was attributed to the tendency of the girders to separate as the bridge was loaded, which generates high stress concentrations in the interface region. According to Girfin (1997), IDs increase the load distribution (so reduce the load distribution factor) and reduce the deflection in girder, but this is coming at a cost of increased construction and maintenance costs. Therefore elimination of RC IDs seems to be a suitable solution. But keeping in view the need for providing lateral stability to girders during the period of construction , Girffin suggested that providing temporary steel diaphragms as a better alternative to RC diaphragms.

Barr et al. (2001) studied the evaluation of flexural live load distribution factors in a three-span prestressed concrete girder bridge, where a three span bridge with span lengths of 80, 137, and 80 ft and skew angle of 40 degrees was tested. A finite element model was developed to assess the live load distribution procedures recommended by the AASHTO code. For both interior and exterior girders the addition of IDs had the least effect on the live load distribution factor among the variables investigated in this study. For the exterior girders, IDs slightly increased the live load distribution factor for low skew angles. For skew angles larger than 30 degrees, the addition of IDs was slightly beneficial. According to this study, for design consideration from a structural standpoint, the largest changes would be credited to the addition of end diaphragms, while almost no changes would occur due to the addition of IDs as these showed almost no change in the distribution factors.

Eamon and Nowark (2002) studied the combined effects of secondary elements such as diaphragms, sidewalks, and barriers on load distribution in the elastic and inelastic domains, as well as their effects on the bridge ultimate capacity in steel girder bridges. According to this study diaphragms tend to be more effective at wider girder spacing and longer spans in terms of maximum girder moment reduction, while increasing the number of them does not significantly affect the results. Diaphragms showed to reduce the maximum girder moment up to 13% with an average reduction of about 4%. The ratio of girder stiffness to diaphragm stiffness was observed to be the most important factor effecting load distribution. It was observed that the improvement of the ultimate capacity due to IDs in the inelastic region was not very significant. They found that the girder spacing has very little effect on the moment capacity increase factor, and that the effect of diaphragms on the ultimate load carrying capacity in the inelastic region is insignificant. Eamon and Nowark (2004) furthered this work by assessing the effect on reliability of the bridge due to the presence of secondary elements if these elements were designed to resist the structural loads. According to them the results suggested that a variation of reliability will exist on bridge structural systems if secondary elements are included, and this was found to be function of span length and spacing. They also suggested that a structural system based calibration of LRFD code may be useful to provide a uniform level of reliability to bridge structures and their components.

Cai et al. (2002) investigated six prestressed concrete bridges in Florida and the results were compared with field test results of these bridges. It was found that the finite element prediction without considering IDs or considering only partial ID stiffness contribution has better agreement with field test results than considering the full ID stiffness, implying that the effectiveness of IDs of these bridges are insignificant in distributing the live loads for these bridges. Further examination into the details of these bridges found that the diaphragm connections are weak. Numerically, the diaphragms would have more significant effects on the vertical live load distribution, if a full moment connection is ensured between the diaphragms and girders where the ID stiffness is about 10 % of the girder stiffness. In case of bridges without diaphragms increase of skew angle will decrease the load distribution factor as recognized in the LRFD codes (AASHTO 1998). However, when IDs with absolute stiffness are in position, the increase of skew angle tends to increase the load distribution factor further for the bridges considered.

Green et al. (2002) analyzed bridge performance, considering temperature change effects on bridges of different skew angles with and without IDs. When full ID stiffness was considered in the analysis, where diaphragms were modeled using solid finite elements, the diaphragms were found to have contributed up to 15% reduction in load distribution. Both intermediate diaphragm and the positive temperature gradient decrease the maximum girder moment and the stresses at the midspan. Green et al. (2004) have extended this research by making a study on the influence of skew and bearing stiffness on maximum deflection of girder at midspan. The results show that the influence of intermediate diaphragm decreases with increase in skew angle. Decrease in deflection due to presence of ID for 0º, 15-30º and 60º skew is about 18%, 11% and 6% respectively. By increasing the stiffness of bearing from 0 .0 to 0.655 GPa, the maximum deflection reductions are of 11.5 and 5.9% for 30º and 60º respectively. Increasing the bearing stiffness further to 6.895 MPa, the girder deflections decreased by 14.3 and 10.2% for 30 and 60º skewed bridges.

Khaloo and Mirzabozorg (2003) had taken skew angle, girder spacing and span length for bridges as the parameters for carrying out parametric study for skew bridges. They considered four kinds of configurations of bridges in their study, and these being, (1) without ID, (2) being ID parallel to the supporting lines, for the third and fourth configurations, the diaphragms were perpendicular to the girders. The 3rd type, IDs were provided as per AASHTO requirement while for the 4th type the diaphragms were provided at the quarter and midspan. The following conclusions were drawn from this study

- The configuration of ID in the bridges has significant effect on the load distribution pattern and their effect varied for different skew angles.
- Bridges with ID perpendicular to the longitudinal girders are the best arrangement for load distribution.
- The effect of girder spacing on influence of ID in load distribution was found to be insignificant.

Abendroth et al. (2003) developed a finite element model for skew and non skewed PC girder bridges. They analyzed the bridge model for a lateral impact load, both at and away from the location of the diaphragm configurations. Dynamic loading with 0.1 sec impact duration was used and a single impact load was applied on either exterior girder. It was observed that when the impact load was applied at the diaphragm location, diaphragms reduced the strains effectively and the performance of ID is dependent on its type and configuration. But when the impact is away from the diaphragms, the diaphragm does not help in distributing impact load effectively and also there is no significant difference in the performance of different diaphragms. When the impact load was applied at diaphragms for both skewed and non-skewed bridges, reinforced concrete diaphragms provided largest degree of impact protection.

Cai and Shahawy (2004) studied the effects of field factors, such as bearing restraints and non-structural members such as barriers and diaphragms, on prestressed concrete bridge performance. They have stated that the collective contribution of these field factors result in much less girder moment than that calculated according to AASHTO code specifications.

The other reviews included the work done by Zokaie et al. (1991) and Zokaie (2001) which gave the background behind AASHTO LRFD. Chen (1995a, b and c) and Chen and Aswad (1996) discussed in detail about finite element modeling and load distribution factor calculation for bridges, which were as well reviewed.

3. FINITE ELEMENT MODELS

3.1. Introduction

In this work, two finite element models were used. A simplified 3-D model developed in GT-STRUDL was used for carrying out parametric study in determining the effectiveness of IDs in load distribution for various bridge configurations. The usage of this model was limited to cases where the loading is vertical. In cases where a more refined analysis and where analysis for lateral loading was required, a 3-D solid model built in ANSYS was used. Each model is explained in detail and the methodology adopted for calculating the load distribution factors is discussed in this chapter.

3.2. Simplified 3-Dimensional Finite Element Model

In the current study of the influence of diaphragms on the distribution of vertical loads, parametric studies were carried out using a simplified 3-D finite element model of the bridge. This type of modeling was found to be computationally more efficient and saves great amount of time which are very essential for carrying out parametric study. The simplified 3- dimensional solid model was built in GT STRUDL.

3.2.1. Model Description

The components of the bridge are modeled using plate and line elements along the three dimensions of the bridge. The slab was modeled using four noded quadrilateral plate bending elements along the plane of centroidal axis of deck. Beams and diaphragms were modeled as line elements along their centroidal axis. These elements were formed by connecting nodes offset from the nodes used in modeling deck, along their respective centroidal axis. Rigid links were used for connecting beam and diaphragm elements to the slab. Elevation and top view of this model is shown in Fig. 3.1 and Fig. 3.2.

The X axis is taken along the longitudinal direction of bridge, Y axis in the transverse direction and the Z axis along the depth. At all the simply supported ends the moments are released at the end nodes at the location of supports. At one end of the simply supported bridge, all the girder supports are modeled as roller supports by releasing force in longitudinal direction (F_x). In case of continuous bridges, the continuity is assumed only in the slab and the girders are modeled as simply supported. Though in some bridges the continuity diaphragm makes the girders continuous, this contribution was not considered. All the section properties for girders were available in literature but for the torsional moment of inertia and this was obtained through ANSYS by preparing the geometry of the section in graphical user interface and then obtaining the section properties of the section drawn.

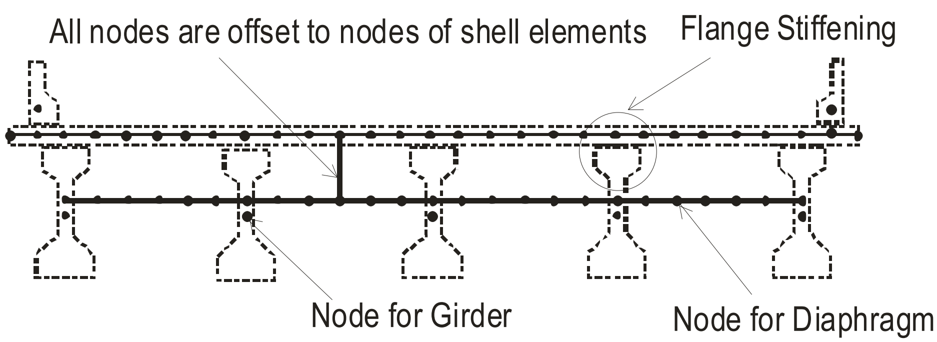

Fig 3.1. Elevation of bridge model

A spread sheet was developed using macros in GT STRUDL. In this spreadsheet all the input characteristics such as geometry, material properties, boundary conditions etc, needed for creating an analytical model and the results needed from the analysis were defined. This information for each model was transferred to text file which was finally imported into GT STRUDL, wherein the model is generated and analyzed. The advantage

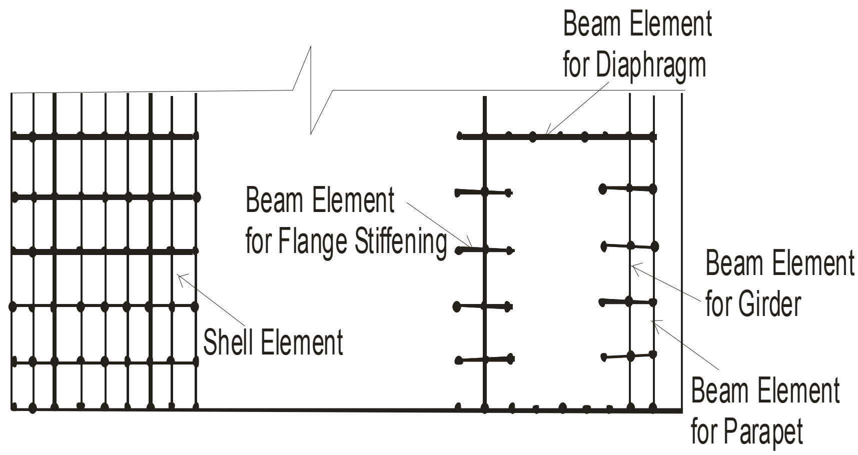

Fig 3.2. Partial plan view of model

of using this technique is that by altering few parameters in the spreadsheet an entire bridge model could be created easily. The spread sheets have been designed keeping in view the large number of bridges that are to be analyzed in the current study, as building a model each time in graphical user interface (GUI) is very cumbersome and time consuming process. The simplified model so built is computationally more efficient thereby saving a lot of time in the analysis. The spreadsheet was used for modeling skew and continuous bridges as well. A portion of this spreadsheet is attached in Appendix 1.

Basic assumptions based on which the model is based upon:

1) The material is assumed to be homogenous and isotropic
2) Cracking of the bridge had been ignored
3) All the connections are assumed to be rigid (including the girder-diaphragm connection).
4) A small deflection theory was used
5) Loading was considered to be static
6) The beams and slab were assumed to act compositely

7) Effect of stiffening due to beams was taken into consideration while that of stiffening due to secondary elements were ignored.

8) Effect of cracking, creep, fatigue and prestressing force is not considered.

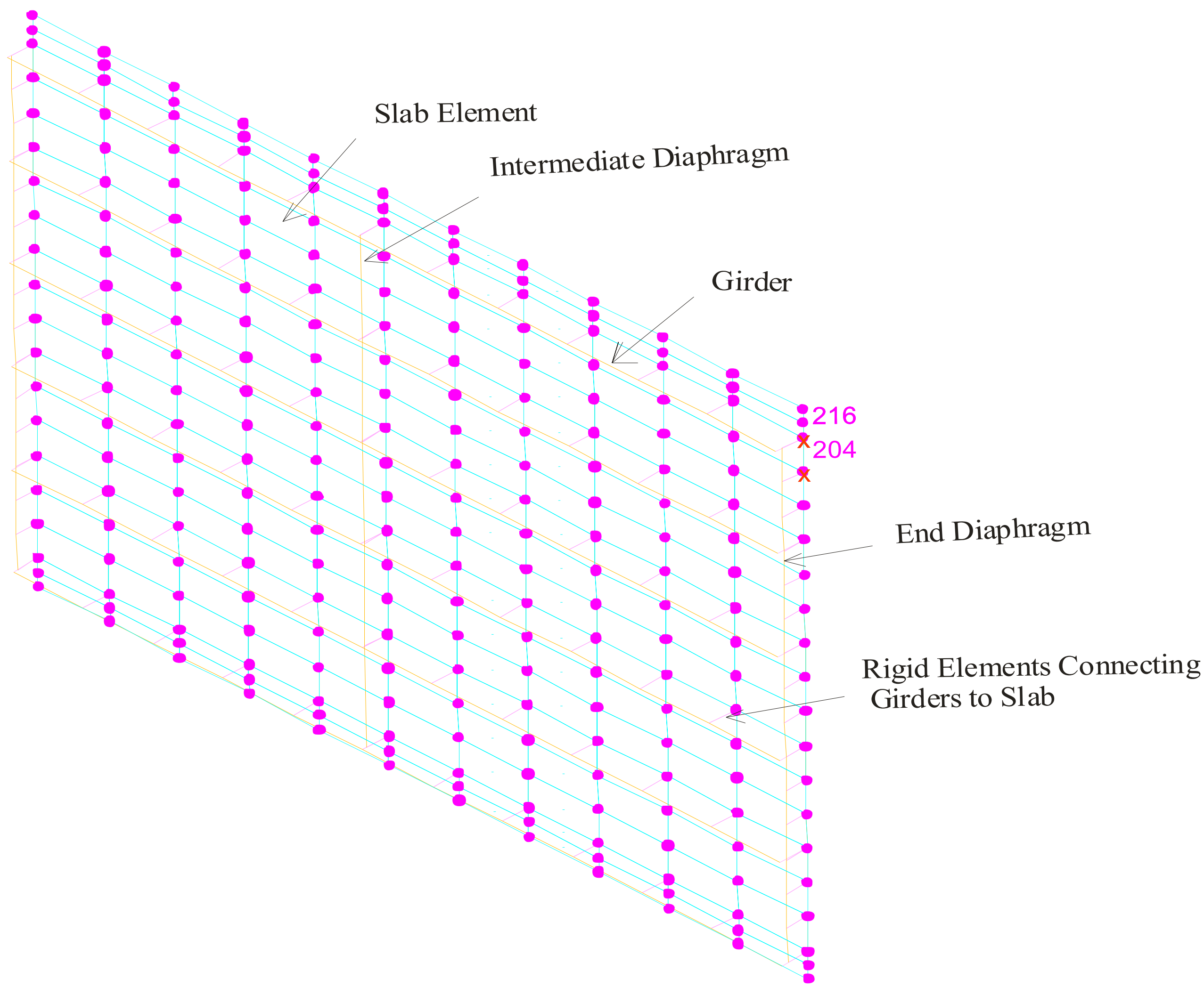

Fig 3.3. Simplified 3-D bridge model in GT STRUDL

3.2.2. Loading and Discretization

The study was limited to static analysis and all the live loads were taken as point loads. If the point of application of loads does not coincide with the nodal location an equivalent static load was taken on the four nodes enclosing the load location. The accuracy of results obtained by analysis depends on the extent of discretization of the model. The smaller the size of the element the greater will be the accuracy. The bridge system was discretized by keeping the spacing between the nodes in both transverse and longitudinal direction in order of 1ft. The type of loads and the location where it is applied is described in Section 3.4.6.

3.3. Solid Finite Element Model

Many of the features for the solid finite element are the same as that of the simplified model. The basic difference between the models lies in the elements used in preparing the models. In this model, 8 noded solid brick elements are used in modeling all the elements of the bridge and only exception being modeling the steel diaphragm elements which were modeled using line or shell elements. ANSYS package has been used for this analysis and the solid element used was SOLID 45 element, with 6 degrees of freedom at each node. LINK8 and SHELL 28 elements were used in modeling steel diaphragm, which are a 3-D truss element and a two dimensional shell element in ANSYS. Keypoints were used to define the geometry of the bridge and later these were joined to generate volumes. The element type SOLID45 is chosen and then the model is auto-meshed which would make the bridge components into solid elements.

A spread sheet using macros in ANSYS for building 3-D solid models was developed for preparing finite element models similar to that developed for the simplified 3-D model. This model is used specifically where the simplified model cannot be used, such as analyzing bridges under transverse loading, for determining forces in bracing during construction and was also used for gauging the accuracy of the simplified model.

The Z axis of the bridge is oriented in the longitudinal direction, with Y axis in the vertical direction and X axis along the transverse direction with the origin located at mid point of the bottom flange at one end of the first girder.

Some of the differences between the solid and simplified model lie in how loading is done, boundary conditions and how results are obtained. In solid model the loading is done by plate loading and is applied as uniform pressure in the contact area of wheel, unlike loads applied as point loads in the case of simplified model for analysis of bridges for live load. While defining the boundary conditions for bridges, caution was taken to restrict the number of constraints provided. Only a minimum number of constraints required for providing stability to the bridge and that needed to simulate the actual bridge performance were provided. This was done because additional constraints would generate secondary stresses, which would alter the stresses in the components thereby affecting the load distribution factor, hence needed to be avoided. Boundary conditions are applied for two nodes at the location of each support on the girder bottom. For the first girder, at one end the displacements along all degrees of freedom are restricted. At the other end of this girder, displacements were restricted along X and Y axis. For all other girders at supports the displacements were restricted along Y axis. For the last girder, translation at one end of the girder is additionally restrained along the Z axis. This model has been shown in Fig.3.4 where girder, both end and intermediate diaphragms and boundary conditions are shown.

Difference exists in the type of results obtained between the two models. In solid model stresses and strains in elements are obtained directly while in the case of simplified model, forces and moments are the final results and the stresses and strains are calculated using formulas which are discussed in Section 4.3.

3.4. Notation Used for Representing Bridge Configuration

As the current work involves parametric study, at several places different bridge configurations needs to be referred. It would be very tedious to describe each individual parameter and its value each time, when a bridge configuration is to be expressed. To make things simpler a notation has been introduced and hereafter this notion would be used for referring to different bridge configurations. This system of notation would include three strings S, L and D which stands for spacing, length and number of intermediate diaphragms respectively. Both the spacing and length are in feet. Each of the string would be followed by the value of the parameter and after putting the strings and corresponding values for parameters, the notation is extended by words "int" or "ext" in braces to indicate the loading configuration. The word "int" refers to the loading configuration which would generate maximum straining action for interior girders and similarly "ext" refers to loading configuration generating maximum straining actions in exterior girders. These loading configurations have been discussed in detail in Section 4.2.3. By default the skew of the bridge is 0^{o}, compressive strength of concrete in girder (f'_{cg}) is 6,000 psi and the bridge is a single span bridge. If the values of the parameters mentioned in the above statement is any different from those indicated, then these parameters and its values has been referred explicitly.

Example : A single span bridge with a spacing of 9ft , a span length of 110 ft, 2 intermediate diaphragms and loading to generate maximum straining action in interior girder, skew = 0 , f'_{cg} = 6,000 Psi would be denoted as S9L110D2 (int).

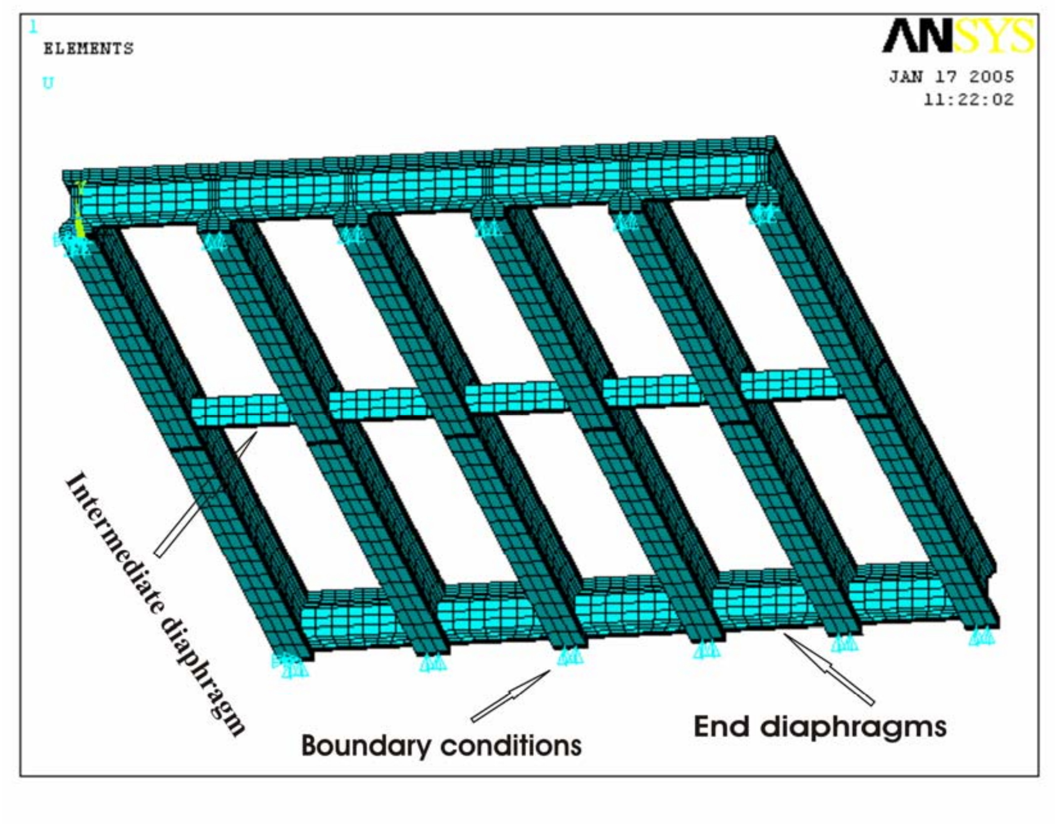

Fig. 3.4. 3-D solid model showing girders, diaphragms and boundary conditions

4. METHODOLOGY

4.1. Introduction

The objective of this chapter is to describe the methodology adopted in carrying out this study. This chapter includes a discussion on parameters and cases considered for parametric studies, various geometric configurations and the loading configurations adopted for bridges, computation of load distribution factor for girders, and comparison between the results obtained from the two finite element models. The results of interest in this study are strain at the girder bottom, girder deflection under live loads and the load distribution factor (LDF) at midspan.

4.2. Parametric Studies

The parameters adopted in this study were, the type of girder, girder spacing, span length, ID type, skew angle, number of spans, and compressive strength of concrete in the girder. All these parameters were varied to observe the influence of each parameter on load distribution and on the effectiveness of diaphragms. For a successful study numerous cases of bridges and loading configurations are required. The parameters in this study are suitably chosen from the possible range of these variables so as to quantitatively represent the bridges of all the configurations in the defined range.

4.2.1. Geometric Configuration of Bridges

A typical two-lane highway bridge with two shoulders is considered in the entire study. The width of the bridge is taken as 50/ft with two lanes, shoulders and cantilevers with each being 12/, 10/, and 3/ ft, respectively. For placing the loading system close to the edge, 18 inch thick barrier was assumed along the edges, but these barriers were not considered in actual design of bridge. The slab thickness was taken as 8 inches and the compressive strength of concrete for slab and diaphragm was taken as 3,500 psi.

Parameters involved in the study are

1. Four types of girders, AASHTO Type II, III, IV and Bulb T were chosen as these are the predominantly used prestressed concrete girders in Louisiana.
2. Normal concrete compressive strength in girder is taken as 6,000 psi and for high strength concrete this is taken as 10,000 psi. For all the configurations of bridges, analysis was done using normal concrete compressive strength while the study on the influence of using girders of high strength concrete was limited to few cases.
3. The girder spacings of 5/ft and 9/ft were chosen which are the minimum and the maximum spacing, specified by LaDOTD Manual (2002).
4. The minimum and the maximum values of the span length for each type of girder were chosen as specified in LaDOTD Manual with slight modification.
5. All bridge configurations were analyzed without diaphragms and then with diaphragms. The number of diaphragms was chosen based on the LaDOTD specifications.
6. In addition to analyzing right bridges, skew bridges with skew angles of 30^o and 50^o were also analyzed.
7. Continuous bridges were also considered in the analysis
8. For limited number of cases, analysis was done for bridges with different steel diaphragm configurations.

4.2.2. Diaphragm Configuration

At the locations of support for all the bridges considered in the parametric study, end diaphragms are provided parallel to the direction of support. The end diaphragms extend from the bottom of slab to the bottom flange of girders.

As mentioned earlier, there is a controversy on the number of diaphragms to be provided, size and its type. IDs are provided based on specific guidelines put forward by state department of transportations (DOT), and these guidelines differ from one state to other. As this project was being done for the state of Louisiana, the diaphragm type, number, spacing and location were provided as per the LADOTD specifications. All the RC diaphragms were considered to be of 8/in thick.

LaDOTD Bridge Design Manual (2002) has put forward the following criterion for providing ID:

- For $L \leq 15$ m, no diaphragm is required.
- For 15 m $< L \leq 30$ m, one diaphragm is required.
- For $L > 30$ m, two diaphragms are required.

The number of diaphragms to be provided is chosen based on the above specifications. For bridges with a single diaphragm, the ID is provided at the midspan while for bridges with two diaphragms these are located at one-third span. The current practice in Louisiana is to connect girders through IDs in the region of girder web and the same was adopted in modeling the bridges.

In the case of skew bridges, diaphragm construction is a difficult task and there are various possible geometric configurations of IDs in skew bridges. The diaphragms can be parallel to the support, perpendicular to the girder line or perpendicular to the girder line but discontinuous with the IDs starting at equal distances from the support as shown in Fig. 4.2 . The third type of configuration described above is predominantly used in Louisiana; hence this configuration of IDs has been used for modeling diaphragms in skew bridges. For small skew angles the orientation of IDs does not influence the results as the distance between the positions of IDs for different configurations would be small.

One of the objectives of this current project is to search for alternative steel configurations which could replace RC diaphragms if found to be effective. Therefore, a parametric study is made by analyzing bridge configurations as shown in Table 4.2, where appropriate steel diaphragms were chosen for the corresponding bridges.

4.2.3. Loading System

HS-20 standard truck loading was used for loading the bridge which is a common truck used for design loading. The lane loading was not considered in this study as the difference between the load distribution of lane and truck loading is very insignificant as observed by Chen et al. (1995). Hence, only the effect of truck loading on the bridges is studied. This is also consistent with the methodology used in developing the AASHTO LRFD (2004) Code Specifications where only truck loads are considered in determining the load distribution factor (LDF).

The truck consists of 3 axles with both the wheels of each axle carrying the same load. The weight of the first axle is 8 kips and the other two axles weigh 32 kips each. The spacing between the first axle and the second axle is 14’ and the spacing between the second axle and the third axle is variable and this could be altered from 14’ to 30’ to generate maximum load effects in the span and the width of each axle is 6ft. The minimum spacing of 2’ was provided between the curb and the wheel line of the truck and the closest wheel lines of the two trucks were placed no closer than 4' as per AASHTO specifications.

Since all the bridges in the current study are two lane bridges, two-lane loading was applied throughout. The truck is moved parallel to the direction of the bridge. The spacing between the second axle and third axle is taken as 14' for all the cases because this configuration of truck generates the maximum load effect for all bridge configurations. The loading was intended to generate maximum straining action at the mid-span section of the bridge and this was achieved by placing the middle axle of the truck at the mid-span for right bridges. Two kinds of loading positions were adopted, one for obtaining the maximum straining action on the exterior girder and another for the interior girder.

For obtaining maximum straining action for the exterior girder the trucks were placed as close as possible to the exterior girder. Unless specified, the distance between exterior girder and edge is taken as 30 inches. The maximum straining action for an exterior girder may be for the case where the wheel line of the first truck is on the exterior girder. But as the minimum spacing between the curb and the wheel line must be 2ft, the first wheel line was placed at 42/ in from the bridge edge (18/in (barrier width) + 24/in (minimum distance of barrier to the wheel line)) by default. The adjacent wheel lines of the two trucks were placed 4/ft apart from each other, which is the minimum spacing that is to be provided between two trucks as per AASHTO specifications (2004). For obtaining the maximum straining action for the interior girder, the second wheel line of the first truck is placed above the innermost girder (third girder in the case of 9/ft spacing and fifth girder for 5/ft spacing) and the first wheel line of second truck is placed 4/ft away from the first truck (Fig. 4.1).

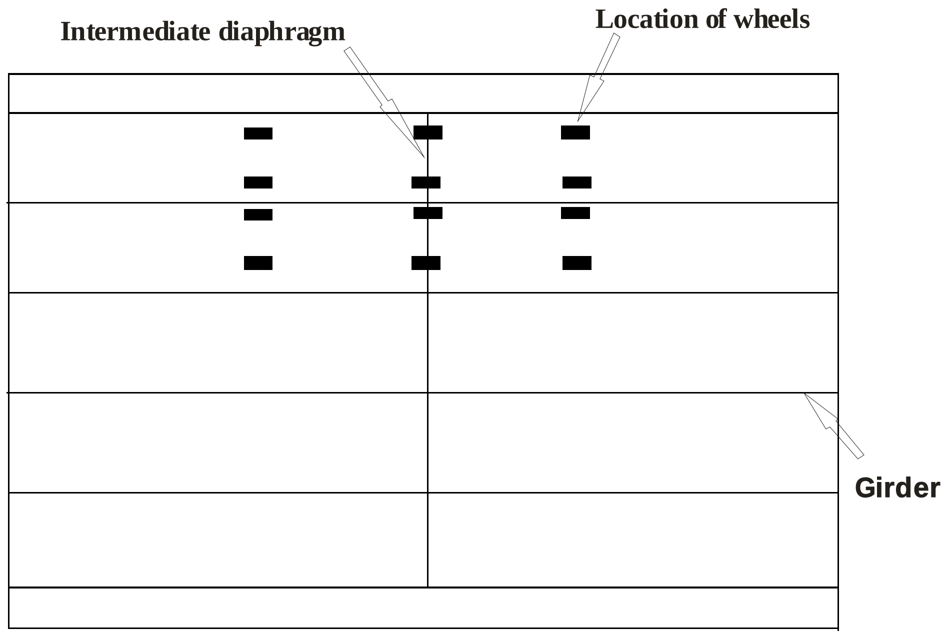

Fig 4.1. Loading configuration for obtaining maximum straining action for exterior girder in right bridge

For skew bridges the loading configuration is the same as that for right bridges except for one difference. Both wheels of an axle cannot be above the mid-span section at the same time as the section under consideration is not perpendicular to the direction of the girder. Hence only the first wheel of the second axle of both trucks is placed at mid-span as observed in Fig. 4.2.

4.2.4. Initial Proposed Cases for Parametric Studies

An initial proposal was made, for the bridge configurations to be analyzed for parametric study which are listed in Tables 4.1 and 4.2. This was made by altering parameters thought to influence the diaphragm effectiveness, which are spacing, span length, girder type, skew angle, continuity and strength of concrete. The parametric study is carried out for both the loading configurations which generate maximum straining

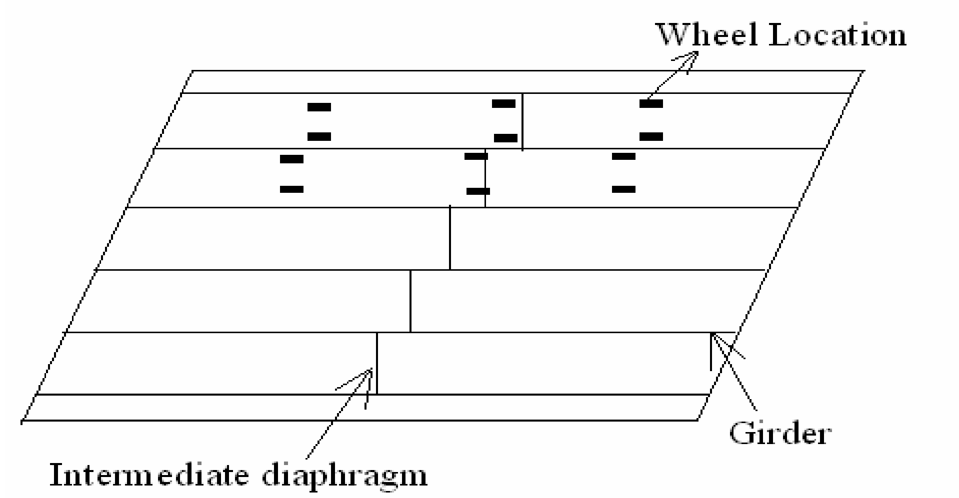

Fig 4.2. Skew bridge loading configuration for generating maximum straining action for exterior girder.

action for both interior and exterior girders. If initial results indicate that the influence of a particular parameter on diaphragm effectiveness is not significant, that parameter will not be considered further in parametric studies. In cases where there is lack of clarity on the influence of a particular parameter from the initial results obtained and/or a need for more results for developing formulas for the diaphragm effect on load distribution, then the analysis is done for more cases.

4.3. Computation of Girder Moment and Load Distribution Factor

The finite element output for simplified model was in terms of axial force (P) and bending moment (M) for the beam elements of the bridge. The maximum stress at any section of the beam (σ) was obtained by summing up the axial stress and bending stress.

Table 4.1. Proposed cases for Parametric study for bridges with RC diaphragms

Concrete Diaphragm								
Girder Type	Girder Concrete Strength (psi)	Girder Spacing (ft)	Span Length (ft)	Number of IDs	ID Type	Skew Angle (degree)	Number of Spans	Total Case Number
II	6000	5, 9	50, 65	0, 1	1	0, 30, 50	1, 3	48
III	6000	5, 9	70, 90	0, 1	1	0, 30, 50	1, 3	48
IV	6000 (10,000)	5, 9 (9)	95, 110 (110)	0, 2	1	0, 30, 50	1, 3	60
Bulb T	6000 (10,000)	5, 9 (9)	105, 130 (130)	0, 2	1	0, 30, 50	1, 3	60
							Sub Total =	216

Table 4.2. Proposed cases for parametric study for bridges with steel diaphragms

Steel Diaphragm								
Girder Type	Girder Concrete Strength (psi)	Girder Spacing (ft)	Span Length (ft)	Number of IDs	ID Type	Skew Angle (degree)	Number of Spans	Total Case Number
II	6000	9	65	0, 1	2	0, 30, 50	1, 3	24
IV	6000 (10000)	9	110	0, 2	2	0, 30, 50	1, 3	48
Bulb T	6000 (10000)	9	130	0, 2	2	0, 30, 50	1, 3	48
							Sub Total =	120

$\sigma = P/A + M/S$ (4.1)

where A = area of the beam cross-section

S = section modulus at the bottom of the beam cross-section.

Maximum strain for this corresponding stress was obtained by dividing the maximum stress by Young's modulus of elasticity

$\varepsilon = \sigma / E$ (4.2)

where E = Young's modulus of elasticity

ε = Strain.

The load distribution factor (LDF) for a girder at a section was obtained by dividing the strain of the girder under consideration at that section, to the summation of strains of all the girders of the bridge at the same section. This factor is then multiplied by the number of wheel lines or the number of trucks, depending on whether the LDF is defined in terms of wheel lines or lanes (axles or trucks). For all the cases LDF was determined at the midspan.

This can be represented in form of equation as follows:

$LDF = \varepsilon_i / \Sigma \varepsilon_i * N$ (4.3)

$\varepsilon_i =$ Strain in girder number i at the section considered.

$\Sigma \varepsilon_i =$ Sum of the strains in all the girders along the section considered, "i" is girder number.

N = Number of wheel lines.

4.4. Comparison between Simplified and Solid Models

The two finite element models described earlier were calibrated with test results obtained from a field bridge. The simplified model had been used by Cai et .al (2004) to the extent of carrying out sensitivity studies to find the influence of various field factors such as flange stiffening, parapet stiffening and bearing stiffening on the load distribution, thereby justifying the applicability of this model in the current study. To make sure that a simplified model could be used for the current studies, the results obtained from the simplified model were compared to the results from the solid model. The comparison was limited to strains and load distribution factors.

Comparison is done between the two models for the same bridge and loading configurations. This study was done for bridges with a span length of 110 ft and 9ft spacing, with and without diaphragms for loading configurations generating the maximum straining action for both interior and exterior girders. The results of this comparison are presented in Tables 4.3 to 4.6 and Fig 4.3 to 4.7. In these tables, though the variation in the results percentage wise is larger for girders away from the loading system, but the actual difference between the results is very small. As the results are of interest for girders, for which loading system is placed to generate maximum loading effect, these results are taken as the representative results for the corresponding cases. Therefore the percentage difference between the results for these girders, have been highlighted in Tables 4.3 to 4.6. For interior girders the difference between the results obtained from two models was in order of 2% where the loading generates maximum straining action for interior girders. The maximum difference in results for exterior girder for a load configuration which generates maximum straining effects at the exterior girder midspan was in order of 4%. Figs. 4.3 and 4.4 show how diaphragms affect the load distribution factor for interior and exterior girders, respectively. Diaphragm effect on girder strains for interior and exterior girders has been presented in Fig 4.5 and 4.6. It was observed that effect of diaphragm on load distribution obtained from the two models were same and has given the confidence that the simplified model can be used in determining ID influence on bridge performance.

Table 4.3. Comparison of results between simplified model and solid 3-D model for bridge S9L110D0 (int)

Girder #	1	2	3	4	5	6
3-D Solid Model						
Maximum micro strain	60	128	160	127	64	24
LDF (Wheels/girder)	0.43	0.91	1.14	0.9	0.45	0.17
Results from simplified model						
Maximum micro strain	63	127.5	162.8	127.3	62.5	17.9
LDF (Wheels/girder)	0.4	0.9	1.2	0.9	0.4	0.1
Difference between simplified model and 3-D model						
% difference in strain	5	0	2	0	-2	-34
% change in LDF	5	0	2.1	1	-2	-34

Table 4.4. Comparison of results between simplified model and solid 3-D model for bridge S9L110D1(int)

Girder #	1	2	3	4	5	6
3-D Solid Model						
Maximum micro strain	83	120	136	114	69	43
LDF (Wheels/girder)	0.588	0.85	0.963	0.807	0.488	0.304
Results from simplified model						
Maximum micro strain	73.6	108.5	138.2	121.1	74.2	44.5
LDF (Wheels/girder)	0.53	0.77	0.99	0.87	0.53	0.32
Difference between simplified model and 3-D model						
% difference in strain	-13	-11	2	6	7	3
% change in LDF	-12	-10	2.4	7	8	4

Table 4.5. Comparison of results between simplified model and solid 3-D model for bridge S9L110D0 (ext)

Girder #	1	2	3	4	5	6
3-D Solid Model						
Maximum micro strain	156	168	127	63	32	20
LDF (Wheels/girder)	1.102	1.187	0.898	0.445	0.226	0.141
Results from simplified model						
Maximum micro strain	150	171	134	66	25	2
LDF (Wheels/girder)	1.095	1.25	0.976	0.485	0.226	0.141
Difference between simplified model and 3-D model						
% difference in strain	-4	2	5	5	-29	-1222
% change in LDF	-1	5	8	8	-24	-1179

Table 4.6. Comparison of results between simplified model and solid 3-D model for bridge S9L110D2 (ext)

Girder #	1	2	3	4	5	6
3-D Solid Model						
Maximum micro strain	162	157	120	68	40	20
LDF (Wheels/girder)	1.143	1.108	0.847	0.480	0.282	0.141
Results from simplified model						
Maximum micro strain	165.9	167	126	67.8	31	9.5
LDF (Wheels/girder)	1.169	1.177	0.88	0.477	0.219	0.067
Difference between simplified model and 3-D model						
% difference in strain	2	6	5	0	-29	-111
% change in LDF	2	6	5	0	-29	-111

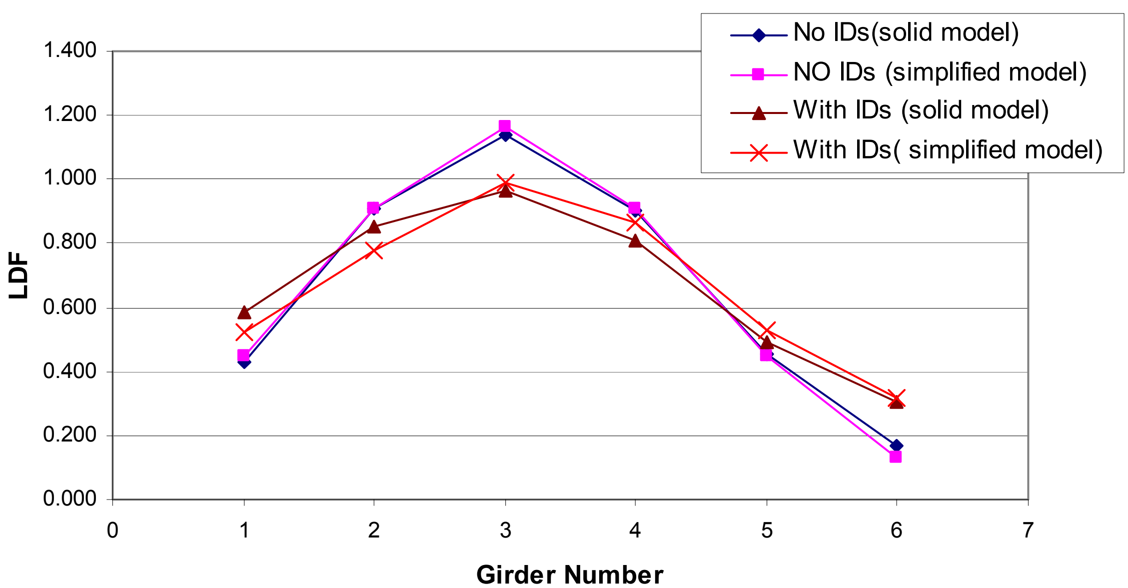

Fig 4.3. Comparison between LDF values for bridge girders in S9L110 with and without diaphragm for the two models for loading generating maximum straining action for interior girder.

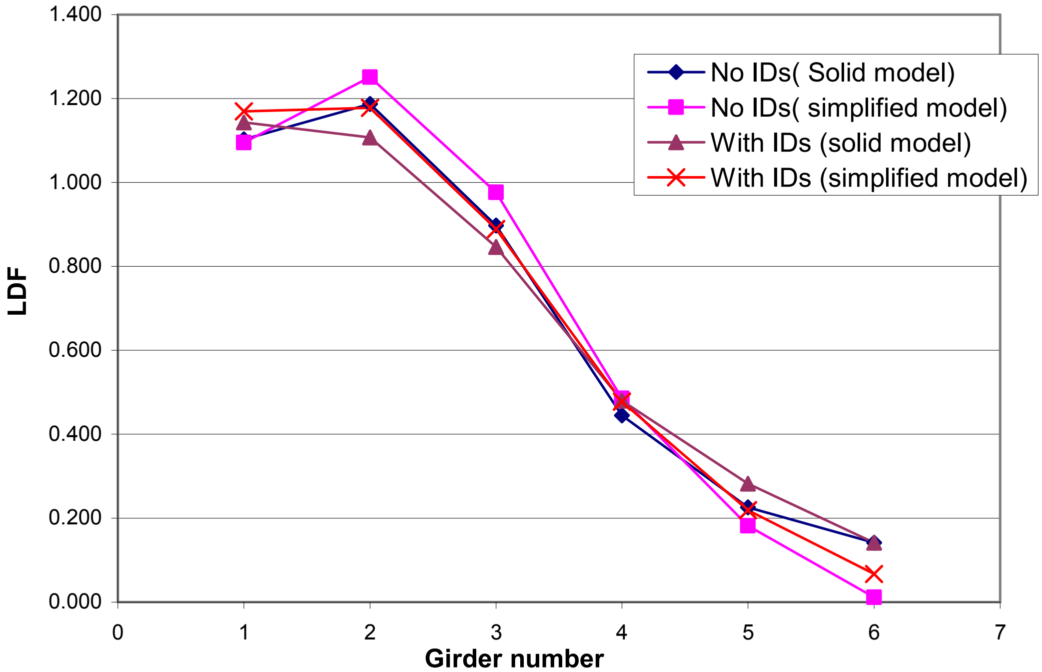

Fig 4.4. Comparison between LDF values for bridge girders in S9L110 with and without diaphragm for the two models for loading generating maximum straining action for exterior girder.

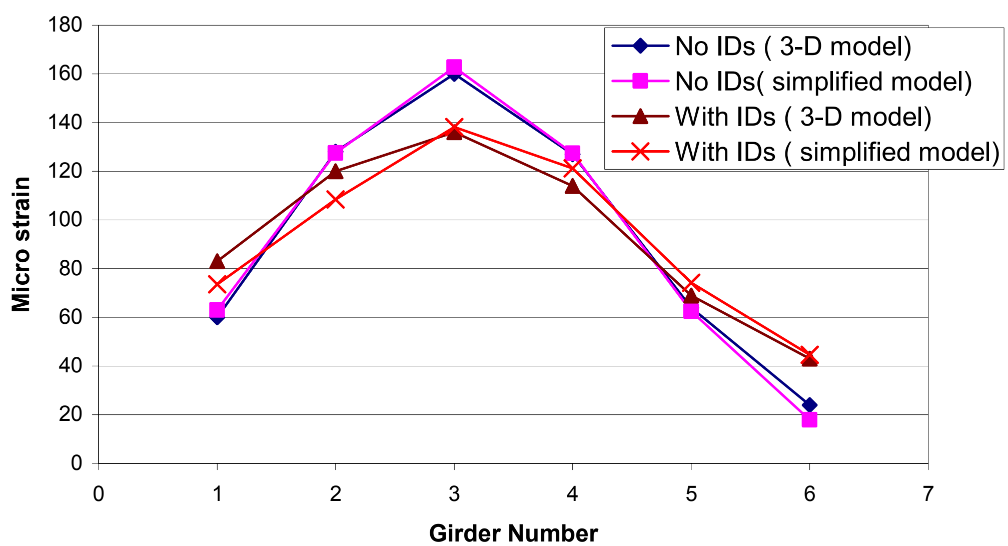

Fig 4.5. Comparison between micro strain values for bridge girders in S9L110 with and without diaphragm for the two models for loading generating maximum straining action for interior girder.

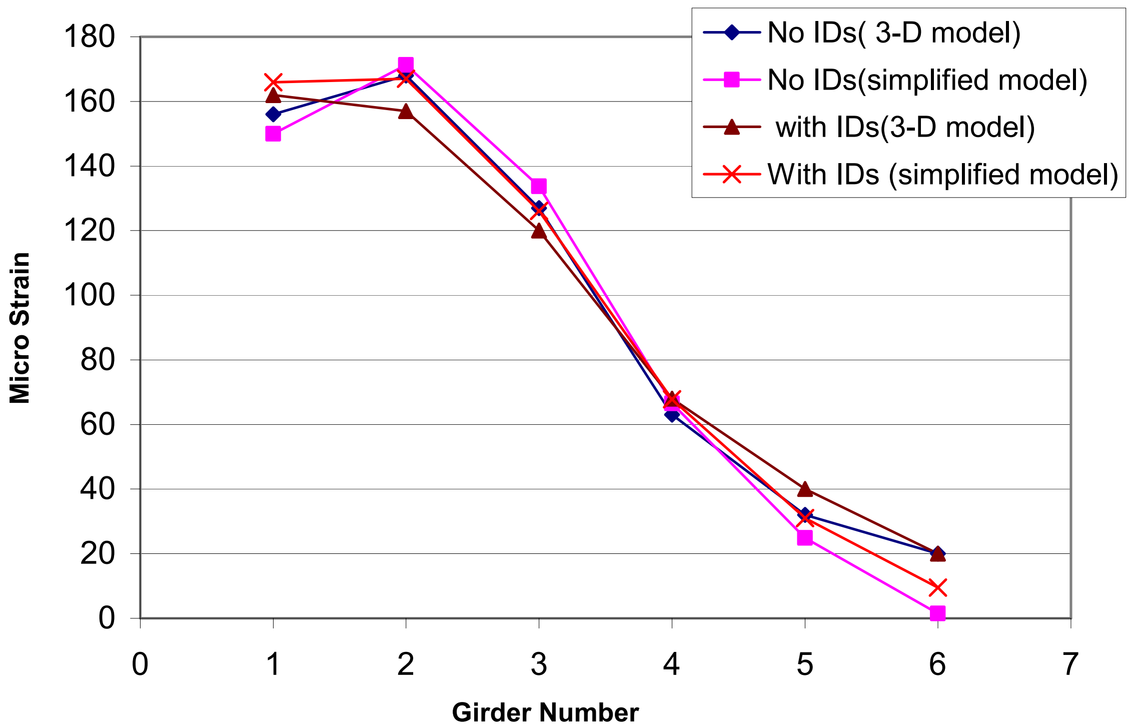

Fig 4.6. Comparison between micro strain values for bridge girders in S9L110 with and without diaphragm for the two models for loading generating maximum straining action for exterior girder.

5. DIAPHRAGM MODELING AND PARAMETRIC STUDIES

5.1. Introduction

In this section problems related to modeling the reinforced concrete diaphragm and how modeling the diaphragm differently yields different results are discussed. Preliminary parametric study which was carried out to understand the effect of parameters thought to influence the ID effect on bridge behavior is presented in this chapter. This was furthered by a detailed parametric study, which is presented as well. In the end each parameters effect on ID influence on bridge behavior is discussed in length.

5.2. Modeling Diaphragm

5.2.1. Problems Concerning Modeling Diaphragms

Traditional analysis methods like grillage analysis, modeling diaphragm was a difficult task. Now with the availability of many advanced finite element packages, diaphragms could be modeled appropriately without much difficulty. The problem lies in quantifying the stiffness contribution of the diaphragm. It is observed that the connection between the girder and the diaphragm is essentially a cold joint and is structurally “weak” with usually one or two reinforcement bars connecting these elements. The stiffness at the connection is variable and is based on the load levels (Cai et al. 2004). At low loads the connection is close to full moment connection. As loads increase up to the ultimate stage, the cold joint may crack and open, leaving only the steel reinforcement effective in tension region of ID girder interface. In different studies, researchers modeled diaphragms differently and have considered different levels of stiffness contribution of diaphragms. This could be a possible reason for reaching contradictory conclusions and different measures of diaphragm effectiveness in these studies. Hence the need exists to model the diaphragm rationally to simulate the actual behavior which otherwise may not lead to an appropriate estimate of the diaphragm effectiveness.

Two bridges were considered in which the diaphragms were modeled differently to understand clearly, how difference in modeling the ID affects the bridge behavior. The bridge configurations chosen for this study were S9L110 and S9L130 with diaphragms being modeled differently, and the results for these are compared to the results for the case where the diaphragms are absent. For all the cases strain, deflection and load distribution factor from the finite element model, AASHTO STD, and LFRD and the strains in diaphragms are calculated. These results have been presented in Table 5.1. Also the difference a particular diaphragm modeling creates in the values of strain, LDF and deflection when compared with the respective values for the same bridge without diaphragms are also presented in Table 5.1. This difference between the results is expressed in terms of percentage change in respective values of results for the case without diaphragm to that with diaphragm, namely, (Result value $_{\text{without ID}}$ - Result value $_{\text{with ID}}$)/ Result value $_{\text{without dia}}$ * 100. Load distribution factor by LRFD is the number of design lanes per girder while other load distribution factors are in terms of the number of wheel lines per girder. Hence while comparing the various load factors the LRFD load distribution factor is multiplied by a factor of two. And the same procedure is adopted throughout this work wherever a comparison between different load distribution factors is being made.

The different ways the diaphragms were modeled for these bridges were

1) Case1- Rigid diaphragm connection with absolute stiffness (100%)

The ID is modeled as rigid element offset from the slab to the location of geometric centroidal axis of ID, between the girders. Here absolute stiffness of diaphragm has been considered and this model is equivalent to elastic solid modeling (Green et al., 2002). But in reality under heavy loads there is a possibility of development of crack at diaphragm girder connection and only a part of diaphragm section effectively contributes to load distribution. Hence diaphragm effectiveness obtained through this diaphragm model is an upper bound for the results as a maximum stiffness contribution of diaphragm is taken into account (Cai et al. 2004).

2) Case 2- Diaphragm stiffness equal to 30% of absolute stiffness

In case of cracking at diaphragm-girder interface, as mentioned earlier the entire section of diaphragm does not affect load distribution; hence the diaphragm effective stiffness contribution in load distribution is low. To observe the impact of stiffness of ID on straining actions, the diaphragm stiffness was taken as 30% of the absolute stiffness and this was accomplished by taking the Young's Modulus as 30% of its original value for concrete in diaphragms.

3) Case 3- Rigid without offset

In some of the research in the past, the diaphragms were modeled in the plane of the deck that is without providing any offset for diaphragm (Hays et al.1994). This case was considered to see how offsetting diaphragm changes effectiveness of diaphragm.

4) Case 4- As truss element

The connection between the diaphragm and girder might not be absolutely rigid and may not be in a position to carry moments. In this case diaphragm is modeled as rigid truss element to know the impact of releasing moment carrying capacity of ID on ID effectiveness.

5) Case5- Only rebar connection between diaphragm and girder

In this case the stiffness contribution of concrete in the diaphragm is ignored completely and the girders are assumed to be connected only through rebars in the diaphragm. The rebars were modeled as truss elements and the results obtained from this case would be the lower bound values.

5.2.2. Discussion of Results

The results shown in Table 5.1 clearly indicate that modeling the diaphragm differently yields different results. The Case 1 as assumed indicated maximum contribution of diaphragm and Case 5 predicts the least contribution by diaphragms and results for other cases lie in-between these extreme values. By reducing the stiffness of IDs to 30% from the absolute (100%) diaphragm stiffness, it decreases the effectiveness of diaphragm in reducing load distribution by about 6% for the interior girder. The effect of modeling diaphragms differently has more impact on the results obtained for the interior girder as diaphragms effect on interior girder is larger. All these results indicate the need for quantifying the effective stiffness of diaphragm, its influence on ID effectiveness and usage of appropriate diaphragm model. Another observation is that, the tensile stresses due to live loads in the diaphragms for Case 1 may exceed or reach close to the rupture modulus ($7.5\sqrt{f_c^{'}} = 7.5\sqrt{3500} = 444\,psi$), indicating possible cracking of concrete. The stresses listed in the table are only due to action of live loads and if the influence of dead loads, temperature stresses are also included, the stresses could reach much beyond the rupture stress. From the stress contour diagram in Fig 5.1 to Fig 5.3, which were obtained from solid model analysis in ANSYS, it could be observed that there is significant stress concentration at the diaphragm-girder interface.

Table 5.1. Diaphragm stiffness effect on bridge performance

Model of diaphragm	Girder Case	Strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%change in LDF	AASHTO STD LDF	LRFD LDF	Stress in diaph. (psi)
	(a) Type 4 AASHTO girder with span length 110 ft, fc' = 6000 psi, girder spacing = 9 ft									
No Diaphragm	Interior Exterior	160.0 177.3		0.568 0.707		1.14 1.26		1.64 1.64	1.42 1.25	
Case 1 Rigid Moment	Interior Exterior	138.2 182.2	13.7 -2.8	0.482 0.706	15.2 0.0	0.99 1.30	13.7 -2.8	1.64 1.64	1.42 1.25	690.5(T) 376.5(T)
Case 2 30% stiffness of Case 1	Interior Exterior	147.8 180.8	7.6 -2.0	0.520 0.722	8.5 -2.1	1.06 1.29	7.6 -2.0	1.64 1.64	1.42 1.25	348.7(T) 195(T)
Case 3 Rigid without offset	Interior Exterior	156.9 178.2	2.0 -0.5	0.555 0.710	2.3 -0.5	1.12 1.27	2.0 -0.5	1.64 1.64	1.42 1.25	564.3(T) 383.0(T)
Case 4 As truss element	Interior Exterior	140.0 182.7	12.5 -3.1	0.492 0.729	13.5 -3.1	1.00 1.30	12.5 -3.1	1.64 1.64	1.42 1.25	372.6(T) 144.8(T)
Case 5 Only steel connnection	Interior Exterior	157.8 179.0	1.4 -1.0	0.559 0.715	1.6 -1.1	1.13 1.27	1.4 -1.0	1.64 1.64	1.42 1.25	N.A. N.A.
	(b) BT-72 girder with span length 130 ft, fc' = 6000 psi, girder spacing = 9 ft									
No Diaphragm	Interior Exterior	167.4 182.9		0.597 0.739		1.20 1.30		1.64 1.64	1.41 1.24	
Case 1 Rigid Moment	Interior Exterior	129.5 190.3	22.6 -4.0	0.444 0.773	25.6 -4.6	0.93 1.36	22.7 -4.0	1.64 1.64	1.41 1.24	438(T) 168.2(T)
Case 2 30% stiffness of Case 1	Interior Exterior	138.5 189.6	17.3 -3.6	0.481 0.769	19.5 -4.1	1.0 1.4	17.3 -3.6	1.64 1.64	1.41 1.24	318.5(T) 128.2(T)
Case 3 Rigid without offset	Interior Exterior	143.0 188.3	14.6 -2.9	0.50 0.76	16.6 -3.4	1.02 1.34	14.7 -2.9	1.64 1.64	1.41 1.24	407(T) 197.4(T)
Case 4 As truss element	Interior Exterior	138.4 189.9	17.3 -3.8	0.484 0.769	19.0 -4.1	0.99 1.4	17.4 -3.8	1.64 1.64	1.41 1.24	273.4(T) 82.5(T)
Case 5 Only steel connnection	Interior Exterior	162.9 185.3	2.7 -1.3	0.579 0.750	3.0 -1.5	1.16 1.32	2.7 -1.3	1.64 1.64	1.41 1.24	N.A. N.A.

5.3. Preliminary Studies

There is a possibility that the parameters adopted for carrying out parametric study might not have an appreciable effect on ID effectiveness. Analyzing the bridges for all values of parameters proposed, which have no influence on ID effectiveness, would be unnecessary. In order to avoid this, a preliminary study is done by analyzing a limited

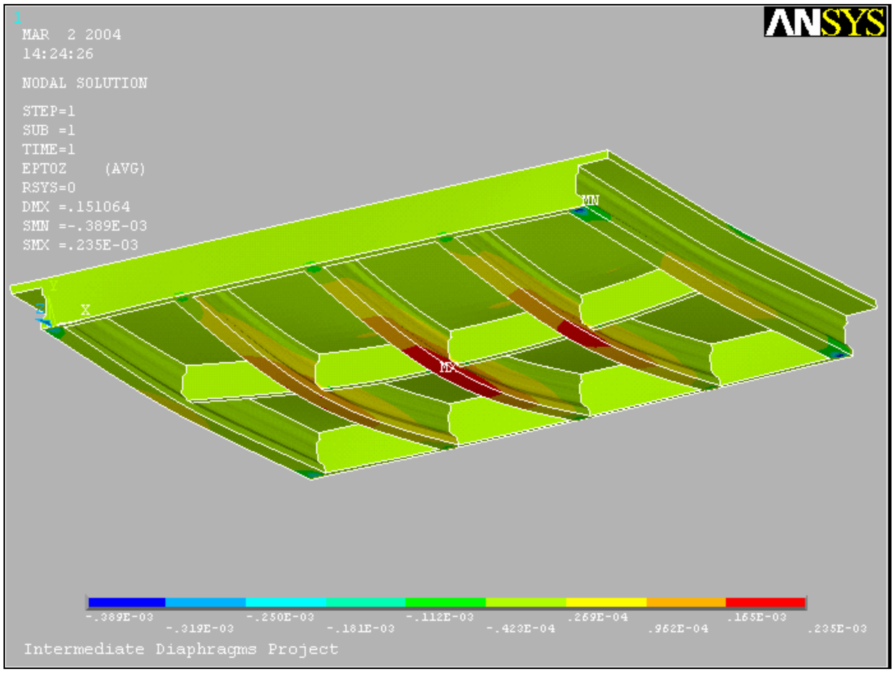

Fig. 5.1. Stress contour for solid model under live load (full model)

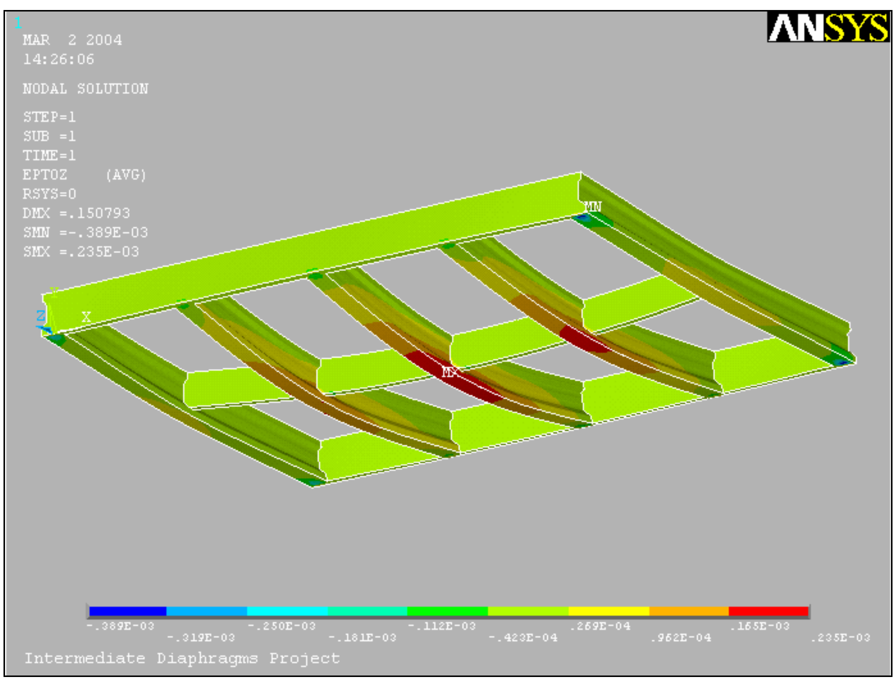

Fig. 5.2. Stress contours in girders and diaphragms

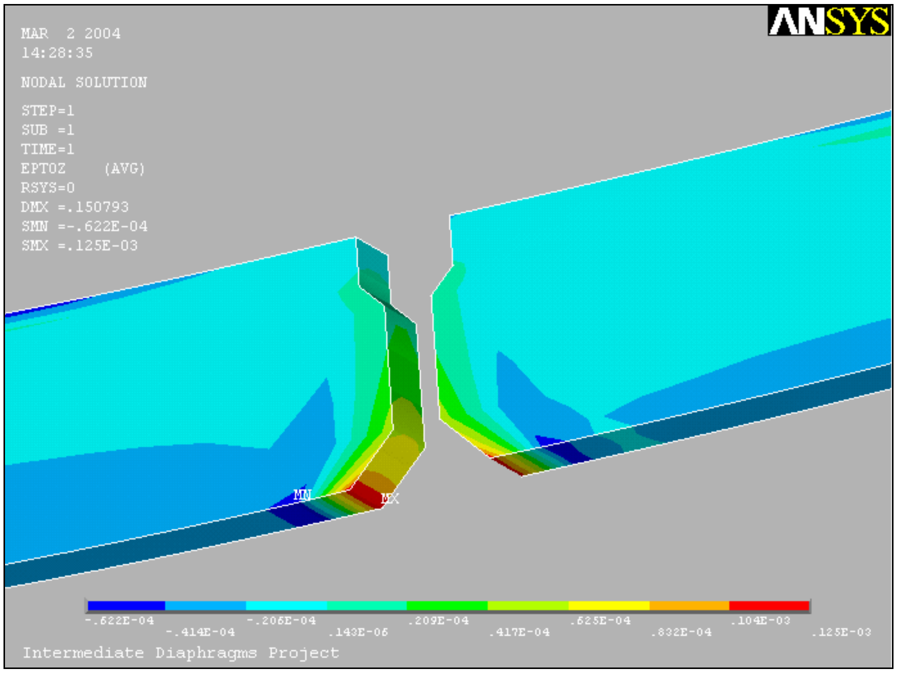

Fig.5.3. Stress contour at the location of diaphragm where it connects to girder

number of cases with values of each parameter far away so as to cover its entire range of values to determine a parameter's influence on bridge performance. A conclusion on whether the parameter has significant influence on bridge performance was reached based on the results obtained through these studies. If the influence of a parameter is found to be appreciable, then further analysis is done for remaining cases involving this parameter otherwise the parameter was not be subjected to further study. The results constitute the values of strain, deflection, load distribution factor at the midspan from the model, and the difference between these values between the cases with diaphragm to that without diaphragm. The results also include LDF values obtained from AASHTO Standard and LRFD specifications. By comparing these values, the effectiveness of diaphragm for each case and influence of each parameter on ID effectiveness is determined.

5.3.1. Effect of Skew

It has been observed in previous studies that skew increases the distribution of loads in bridges, i.e., it reduces the maximum load coming onto a girder. The effect of skew has not been considered in AASHTO STD while AASHTO LRFD has accounted for the effect of skew by including a skew reduction factor, which is multiplied with the calculated load distribution factor for right bridge.

The available literature indicates that the effect of skew is negligible for small angles of skew (up to 20 degrees) and their influence on load distribution increases with increasing skew angle. Not much research has been done yet, to determine the influence of skew on diaphragm effect on load distribution. Early research by Ebeido and Kennedy (1996) and Khaloo and Mirzabozorg.(2003) has indicated that IDs in skew bridges decreases the load distribution factor further. The diaphragm configuration has an effect on load distribution, and the configuration with diaphragms running perpendicular to the girders, distributes the loads more effectively than other possible configurations (Khaloo and Mirzabozorg, 2003). They also concluded that the skew angle has a significant effect on load distribution.

Results of the preliminary study for skew bridges are listed in Table 5.2. Analysis was done for bridge with a span length of 110 ft and a girder spacing of 5/ft for both with and without IDs for skew angles of 0, 30 and 50^{o}. The influence of IDs is significantly different with the maximum difference of about 5% for the bridge with the same geometry but with a different skew. This clearly indicates the influence of skew on ID effectiveness, hence skew was considered as a parameter for further study.

Table 5.2. Preliminary study to understand skew effect on load distribution

case	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
Skew=0									
S5L110D0	In	98.7		0.385		0.65		0.91	0.94
	Ex	130.0		0.545		0.86		0.91	0.83
S5L110D2	In	85.9	12.9	0.331	13.9	0.57	12.9	0.91	0.94
	Ex	131.0	-0.8	0.552	-1.3	0.87	-0.7	0.91	0.83
Skew =30									
S5L110D0	In	94.4		0.376		0.65		0.91	0.92
	Ex	120.5		0.487		0.83		0.91	0.80
S5L110D2	In	86.3	8.6	0.340	9.7	0.60	8.5	0.91	0.92
	Ex	122.3	-1.4	0.496	-1.9	0.85	-1.5	0.91	0.80
Skew =50									
S5L110D0	In	84.4		0.342		0.66		0.91	0.86
	Ex	113.8		0.437		0.86		0.91	0.76
S5L110D2	In	78.2	7.3	0.309	9.5	0.61	7.2	0.91	0.86
	Ex	115.5	-1.6	0.445	-1.9	0.88	-1.4	0.91	0.76

5.3.2. Effect of Continuity

Continuity decreases the positive moments in the span but its influence on load distribution might be quite different. AASHTO STD and LRFD do not take into account the effect of continuity in load distribution. Zokaie et al. (1991) has given modification factors for accounting the effect of continuity. The effect of continuity in this study was given based on the results obtained from analysis of a number of two span continuous bridges with each span being equal to the average span length. They related the distribution factor obtained from the continuous bridge to that obtained for a simply supported bridge by applying a modification factor. The values of modification factor for positive bending moment and negative bending moment were given as 1.05 and 1.10 respectively. But these recommendations are not accepted by the AASHTO LRFD code yet.

In order to understand effect of continuity on ID influence on load distribution, a preliminary study was carried out for continuous bridges. In the preliminary study a few three span bridges with both the exterior spans of equal size and with different ratios of exterior span to interior span were considered. Some two span bridges of equal span were also considered for analysis. In Table 5.3 results have been presented for middle span by default and it is the geometry of this span which has been used to define the continuous bridge geometry in results. The only exception for this is bridge S9L90, in which for one of the cases the geometry has been specified for the end span and the results for the same have been presented. This is distinguished from other cases by placing a star at the end of the number of spans in the bridge.

The results are grouped for comparing bridges with different number of spans with other configuration remaining the same in Table 5.3. The results indicated that continuity has insignificant effect on ID influence on load distribution factor, with the maximum difference in ID contribution for bridges with different spans being about 1%. Along with the cases listed in the Table 5.3, some other cases were also explored to know the effectiveness of IDs before reaching this conclusion. And all the cases yielded similar results, hence continuity was not considered as a parameter in further studies.

5.3.3. Effect of Span Length

Length is one of the major parameters affecting load distribution. Here an attempt is made to understand the influence of span length on diaphragm effectiveness. The analysis was done for bridges of two different span lengths, for different girder types

Table 5.3. Preliminary study to understand effect of continuity

No. of spans	case	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
	Skew=0									
3	S9L130D0	In	160.4		0.5639		1.23		1.64	1.41
	S9L130D2	In	124.1	22.7	0.4076	27.7	0.95	22.9	1.64	1.41
1	S9L130D0	In	167.4		0.5975		1.20		1.64	1.41
	S9L130D2	In	129.5	22.6	0.4444	25.6	0.93	22.7	1.64	1.41
2	S9L130D0	In	159.1		0.5783		1.20		1.64	1.41
	S9L130D2	In	122.7	22.9	0.4291	25.8	0.92	23.0	1.64	1.41
	Skew=0									
3	S5L50D0	In	108.8		0.1199		0.86		0.91	1.02
	S5L50D1	In	95.4	12.3	0.1086	9.5	0.75	12.2	0.91	1.02
2	S5L50D0	in	115.2		0.1287		0.84		0.91	1.02
	S5L50D1	in	101.4	12.0	0.1170	9.1	0.74	11.5	0.91	1.02
1	S5L50D0	In	119.4		0.1364		0.82		0.91	1.02
	S5L50D1	In	104.9	12.2	0.1237	9.3	0.72	12.0	0.91	1.02
	Skew=0									
3	S9L90D0	In	180.1		0.496		1.21		1.64	1.41
	S9L90D1	In	146.3	18.8	0.4310	13.0	0.98	18.9	1.64	1.41
3*	S9L90D0	in	196.8		0.5317		1.2		1.6	1.4
	S9L90D1	in	161.0	18.2	0.4647	12.6	1.0	18.2	1.6	1.4
1	S9L90D0	In	203.3		0.562		1.18		1.64	1.41
	S9L90D1	In	166.7	18.0	0.4914	12.5	0.97	18.0	1.64	1.41
	Skew=0									
2	S9L110D0	In	156.0		0.5478		1.2		1.6	1.4
	S9L110D2	In	134.1	14.0	0.4605	15.9	1.0	14.5	1.6	1.4
1	S9L110D0	In	160.0		0.5682		1.1		1.6	1.4
	S9L110D2	In	138.2	13.7	0.4820	15.2	1.0	13.7	1.6	1.4
	Skew=0									
2	S9L95D0	In	141.5		0.3772		1.2		1.6	1.5
	S9L95D2	In	118.7	16.1	0.3126	17.1	1.0	16.1	1.6	1.5
1	S9L95D0	In	144.4		0.3895		1.2		1.6	1.5
	S9L95D2	In	122.3	15.3	0.3253	16.5	1.0	15.2	1.6	1.5
	Skew = 50									
2	S5L105D0	In	62.9		0.1576		0.79		0.91	0.89
	S5L105D2	In	57.5	8.6	0.1408	10.6	0.72	8.7	0.91	0.89
3	S5L105D0	In	50.4		0.1145		0.84		0.91	0.89
	S5L105D2	In	46.7	7.4	0.1030	10.0	0.77	7.8	0.91	0.89
1	S5L105D0	In	76.3		0.2058		0.75		0.91	0.89
	S5L105D2	In	69.3	9.2	0.1833	10.9	0.68	9.0	0.91	0.89
	Skew = 50									
2	S9L130D0	In	123.2		0.4141		1.22		1.64	1.25
	S9L130D2	In	113.1	8.2	0.3711	10.4	1.11	9.2	1.64	1.25
1	S9L130D0	In	145.7		0.5370		1.15		1.64	1.25
	S9L130D2	In	132.8	8.9	0.4795	10.7	1.04	9.6	1.64	1.25

while keeping all other parameters in the group constant and these results are presented in separate groups. The results of these preliminary studies are listed in Table 5.4 and it could be observed that span length has a significant effect on ID effectiveness, with difference caused due to changing span length up to 5%. Therefore in further parametric studies span length was considered as parameter.

5.3.4. Effect of Spacing

The results of the preliminary studies for understanding the influence of spacing on effectiveness of ID is listed in Table 5.5. The comparison has been done between three groups of bridges, with spacing being the only difference between the bridges in the group and the groups differ in their span length and girder type. The difference between the results in each group is in order of 1% for all cases considered. Though spacing does not seem to have much impact on diaphragm's influence on load distribution from the results obtained, it was still considered as parameter in further study keeping in view the influence of spacing in load distribution.

5.3.5. Effect of High Strength Concrete Girders on Diaphragm Effectiveness

High strength concrete is being used for prestressed concrete girders. AASHTO standard does not take into account the influence of compressive strength of concrete in girder while AASHTO LRFD takes this factor into account through the inclusion of modular ratio in load distribution formulas.

To understand the impact of using high strength concrete girder on diaphragm effectiveness, comparison is done between the results obtained from analysis of bridges which only differ in their girder concrete compressive strength. The bridges with girders

Table 5.4. Preliminary study to understand the effect of span length on load distribution

case	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
S9L70D0	In	167.6		0.289		1.33		1.64	1.51
	Ex	158.2		0.305		1.25		1.64	1.33
S9L70D1	In	140.0	16.5	0.253	12.4	1.11	16.4	1.64	1.51
	Ex	171.1	-8.2	0.319	-4.7	1.35	-8.1	1.64	1.33
S9L90D0	In	203.3		0.562		1.18		1.64	1.41
	Ex	221.6		0.679		1.28		1.64	1.24
S9L90D1	In	166.7	18.0	0.491	12.5	0.97	18.0	1.64	1.41
	Ex	235.5	-6.3	0.700	-3.0	1.36	-6.2	1.64	1.24
S9L95D0	In	144.4		0.389		1.23		1.64	1.48
	Ex	140.5		0.431		1.19		1.64	1.30
S9L95D2	In	122.3	15.3	0.325	16.5	1.04	15.2	1.64	1.48
	Ex	148.3	-5.6	0.453	-5.1	1.26	-5.6	1.64	1.30
S9L110D0	In	160.0		0.568		1.14		1.64	1.42
	Ex	177.3		0.707		1.26		1.64	1.25
S9L110D2	In	138.2	13.7	0.482	15.2	0.99	13.7	1.64	1.42
	Ex	182.2	-2.8	0.706	0.0	1.30	-2.8	1.64	1.25
S9L105D0	In	144.8		0.335		1.34		1.64	1.49
	Ex	138.9		0.378		1.27		1.64	1.31
S9L105D2	In	105.2	27.4	0.243	27.4	0.96	28.4	1.64	1.49
	Ex	150.3	-8.2	0.408	-8.1	1.37	-8.2	1.64	1.31
S9L130D0	In	167.4		0.597		1.20		1.64	1.41
	Ex	182.9		0.739		1.30		1.64	1.24
S9L130D2	In	129.5	22.6	0.444	25.6	0.93	22.7	1.64	1.41
	Ex	190.3	-4.0	0.773	-4.6	1.36	-4.0	1.64	1.24

Table 5.5. Preliminary study to understand the effect of spacing

case	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
S5L65D0	In	153.3		0.288		0.733		0.91	0.95
	Ex	177.8		0.357		0.849		0.91	0.84
S5L65D1	In	131.0	14.6	0.260	9.8	0.628	14.4	0.91	0.95
	Ex	190.9	-7.3	0.370	-3.5	0.912	-7.3	0.91	0.84
S9L65D0	In	247.4		0.431		1.263		1.64	1.44
	Ex	252.1		0.480		1.277		1.64	1.26
S9L65D1	In	214.1	13.5	0.387	10.2	1.092	13.5	1.64	1.44
	Ex	267.6	-6.1	0.496	-3.4	1.355	-6.1	1.64	1.26
S5L90D0	In	125.6		0.380		0.68		0.91	0.94
	Ex	155.3		0.505		0.84		0.91	0.82
S5L90D1	In	102.2	18.621	0.335	11.7	0.55	18.4	0.91	0.94
	Ex	166.4	-7.126	0.519	-2.7	0.92	-9.2	0.91	0.82
S9L90D0	In	203.3		0.562		1.18		1.64	1.41
	Ex	221.6		0.679		1.28		1.64	1.24
S9L90D1	In	166.7	18.0	0.491	12.5	0.97	18.0	1.64	1.41
	Ex	235.5	-6.3	0.700	-3.0	1.36	-6.2	1.64	1.24
S5L110D0	In	98.7		0.385		0.65		0.91	0.94
	Ex	130.0		0.545		0.86		0.91	0.83
S5L110D2	In	85.9	12.928	0.331	13.9	0.57	12.9	0.91	0.94
	Ex	131.0	-0.763	0.552	-1.3	0.87	-0.7	0.91	0.83
S9L110D0	In	160.0		0.568		1.14		1.64	1.42
	Ex	177.3		0.707		1.26		1.64	1.25
S9L110D2	In	138.2	13.652	0.482	15.2	0.99	13.7	1.64	1.42
	Ex	182.2	-2.795	0.719	-1.8	1.30	-2.8	1.64	1.25

of high compressive strength of concrete have been differentiated from bridges with girders of normal compressive strength by placing a suffix H in the parenthesis at the end of bride geometry definition. The difference in load distribution factor due to usage of high strength concrete in bridge girders is in order of 1% (comparing the group with high strength concrete and that with regular concrete). Difference in LDF values obtained from AASHTO LRFD for girders with normal and high compressive strength of concrete

Table 5.6. High strength concrete girders on diaphragm effectiveness

case	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF	% LRFD
S5L95D0(H)	In	72.1		0.2215		0.71		0.91	1.00	
	Ex	82.8		0.2807		0.82		0.91	0.88	
S5L95D2(H)	In	62.2	13.7	0.1902	14.1	0.61	13.6	0.91	1.00	
	Ex	86.5	-4.4	0.2906	-3.5	0.85	-3.5	0.91	0.88	
S5L95D0	In	88.5		0.2613		0.70		0.91	0.98	2.14
	Ex	104.1		0.3357		0.82		0.91	0.86	
S5L95D2	In	75.8	14.4	0.2219	15.1	0.60	14.3	0.91	0.98	
	Ex	108.0	-3.8	0.3477	-3.6	0.85	-3.8	0.91	0.86	
S9L95D0(H)	In	117.7		0.3306		1.25		1.64	1.51	
	Ex	111.4		0.3595		1.18		1.64	1.33	
S9L95D2(H)	In	100.3	14.8	0.2789	15.7	1.06	14.7	1.64	1.51	
	Ex	118.0	-5.9	0.3769	-4.8	1.25	-6.0	1.64	1.33	
S9L95D0	In	144.4		0.3895		1.23		1.64	1.48	2.27
	Ex	140.5		0.4307		1.19		1.64	1.30	
S9L95D2	In	122.3	15.3	0.3253	16.5	1.04	15.2	1.64	1.48	
	Ex	148.3	-5.6	0.4526	-5.1	1.26	-5.6	1.64	1.30	
S9L110D0(H)	In	130.8		0.4837		1.16		1.64	1.45	
	Ex	140.8		0.5898		1.25		1.64	1.27	
S9L110D2(H)	In	113.1	13.5	0.4132	14.6	1.01	13.5	1.64	1.45	
	Ex	145.4	-3.2	0.6076	-3.0	1.29	-3.2	1.64	1.27	
S9L110D0	In	160.0		0.5682		1.14		1.64	1.42	2.26
	Ex	177.3		0.7066		1.26		1.64	1.25	
S9L110D2	In	138.2	13.7	0.4820	15.2	0.99	13.7	1.64	1.42	
	Ex	182.2	-2.8	0.7064	0.0	1.30	-2.8	1.64	1.25	

is also listed in the Table 5.6 and this difference is about 2%. The change in effect of girder concrete compressive strength on influence of ID in load distribution is still lower than the actual change in LDF values due to bridges with different girder concrete compressive strength. As the effect of using girders of high strength concrete does not cause significant difference in ID influence in LDF, girder concrete compressive strength is ignored as a parameter in further studies.

5.4. Results for Parametric Study

From the results of the preliminary study discussed in section 5.3, span length, spacing and skew were considered as the parameters for detailed parametric study. Parametric study is done for the reaming cases listed in Table 4.1 keeping results from preliminary parametric study in view and the results have been tabulated in Tables 5.7 to 5.12.

5.5. Observations Made from Parametric Study

As the results for several bridge configurations are available after the parametric study, some conclusions regarding how each parameter influences the ID effect on load distribution could be drawn. In this section the various observations that could be made from the results from parametric study are discussed. For the cases where the existing results were found to be insufficient to draw conclusions, more cases were analyzed.

5.5.1. Interior Girders

5.5.1.1. Influence of Girder Type on Effectiveness of Diaphragm in Load Distribution

In Fig.5.4, a plot is drawn between the percentage reduction in LDF due to diaphragm and the span length for bridges having different girder sections with girder spacing of 9ft.

Fig 5.4 indicates that significant difference exists between the percentage reduction in load distribution due to diaphragm, for different girder types. It could be basically because of the existing difference in the stiffness of the girder and the ID due to the geometry of the sections. In Bridges with Type III girders the reduction in LDF due

Table 5.7. Results for bridges with type II and III girders for skew 0^o skew

S.No	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
1	S5L50D0	ll	In	119.4		0.136		0.82		0.91	1.02
			Ex	119.7		0.147		0.82		0.91	0.90
2	S5L50D1	ll	In	104.9	12.2	0.124	9.3	0.72	12.0	0.91	1.02
			Ex	130.4	-9.2	0.155	-5.4	0.90	-9.0	0.91	0.90
3	S5L65D0	ll	In	153.3		0.288		0.73		0.91	0.95
			Ex	177.8		0.357		0.85		0.91	0.84
4	S5L65D1	ll	In	131.0	14.6	0.260	9.8	0.63	14.4	0.91	0.95
			Ex	190.9	-7.3	0.370	-3.5	0.91	-7.3	0.91	0.84
5	S9L50D0	ll	In	194.5		0.207		1.43		1.64	1.54
			Ex	167.9		0.196		1.23		1.64	1.36
6	S9L50D1	ll	In	172.4	11.4	0.188	9.4	1.27	11.3	1.64	1.54
			Ex	180.3	-7.4	0.205	-5.0	1.31	-7.3	1.64	1.36
7	S9L65D0	ll	In	247.4		0.431		1.26		1.64	1.44
			Ex	252.1		0.480		1.28		1.64	1.26
8	S9L65D1	ll	In	214.1	13.5	0.387	10.2	1.09	13.5	1.64	1.44
			Ex	267.6	-6.1	0.496	-3.4	1.35	-6.1	1.64	1.26
9	S5L70D0	lll	In	102.8		0.193		0.76		0.91	1.00
			Ex	112.2		0.229		0.83		0.91	0.88
10	S5L70D1	lll	In	85.3	17.0	0.170	11.9	0.63	16.8	0.91	1.00
			Ex	122.8	-9.5	0.240	-4.8	0.91	-10.2	0.91	0.88
11	S5L90D0	lll	In	125.6		0.380		0.68		0.91	0.94
			Ex	155.3		0.505		0.84		0.91	0.82
12	S5L90D1	lll	In	102.2	18.6	0.335	11.7	0.55	18.4	0.91	0.94
			Ex	166.4	-7.1	0.519	-2.7	0.92	-9.2	0.91	0.82
13	S9L70D0	lll	In	167.6		0.289		1.33		1.64	1.51
			Ex	158.2		0.305		1.25		1.64	1.33
14	S9L70D1	lll	In	140.0	16.5	0.253	12.4	1.11	16.4	1.64	1.51
			Ex	171.1	-8.2	0.319	-4.7	1.35	-8.1	1.64	1.33
15	S9L90D0	lll	In	203.3		0.562		1.18		1.64	1.41
			Ex	221.6		0.679		1.28		1.64	1.24
16	S9L90D1	lll	In	166.7	18.0	0.491	12.5	0.97	18.0	1.64	1.41
			Ex	235.5	-6.3	0.700	-3.0	1.36	-6.2	1.64	1.24

Table 5.8. Results for bridges with type IV and BT girders for 0^{o} skew

S.No	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
17	S5L95D0	IV	In	88.5		0.261		0.70		0.91	0.98
			Ex	104.1		0.336		0.82		0.91	0.86
18	S5L95D2	IV	In	75.8	14.4	0.222	15.1	0.60	14.3	0.91	0.98
			Ex	108.0	-3.8	0.348	-3.6	0.85	-3.8	0.91	0.86
19	S5L110D0	IV	In	98.7		0.385		0.65		0.91	0.94
			Ex	130.0		0.545		0.86		0.91	0.83
20	S5L110D2	IV	In	85.9	12.9	0.331	13.9	0.57	12.9	0.91	0.94
			Ex	131.0	-0.8	0.552	-1.3	0.87	-0.7	0.91	0.83
21	S9L95D0	IV	In	144.4		0.389		1.23		1.64	1.48
			Ex	140.5		0.431		1.19		1.64	1.30
22	S9L95D2	IV	In	122.3	15.3	0.325	16.5	1.04	15.2	1.64	1.48
			Ex	148.3	-5.6	0.453	-5.1	1.26	-5.6	1.64	1.30
23	S9L110D0	IV	In	160.0		0.568		1.14		1.64	1.42
			Ex	177.3		0.707		1.26		1.64	1.25
24	S9L110D2	IV	In	138.2	13.7	0.482	15.2	0.99	13.7	1.64	1.42
			Ex	182.2	-2.8	0.706	0.0	1.30	-2.8	1.64	1.25
25	S5L105D0	BT	In	88.4		0.227		0.76		0.91	0.99
			Ex	97.7		0.279		0.85		0.91	0.87
26	S5L105D2	BT	In	64.6	26.9	0.164	28.1	0.56	26.8	0.91	0.99
			Ex	103.5	-5.9	0.294	-5.7	0.89	-5.0	0.91	0.87
27	S5L130D0	BT	In	103.1		0.397		0.70		0.91	0.94
			Ex	127.8		0.540		0.86		0.91	0.82
28	S5L130D2	BT	In	79.1	23.3	0.298	24.8	0.54	23.3	0.91	0.94
			Ex	130.1	-1.8	0.553	-2.5	0.88	-1.8	0.91	0.82
29	S9L105D0	BT	In	144.8		0.335		1.34		1.64	1.49
			Ex	138.9		0.378		1.27		1.64	1.31
30	S9L105D2	BT	In	105.2	27.4	0.243	27.4	0.96	28.4	1.64	1.49
			Ex	150.3	-8.2	0.408	-8.1	1.37	-8.2	1.64	1.31
31	S9L130D0	BT	In	167.4		0.597		1.20		1.64	1.41
			Ex	182.9		0.739		1.30		1.64	1.24
32	S9L130D2	BT	In	129.5	22.6	0.444	25.6	0.93	22.7	1.64	1.41
			Ex	190.3	-4.0	0.773	-4.6	1.36	-4.0	1.64	1.24

Table 5.9. Results for bridges with type II and type III girder bridges for 30^o skew

S.No	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
1	S5L50D0	II	In	106.2		0.125		0.82		0.91	0.98
			Ex	112.5		0.134		0.85		0.91	0.86
2	S5L50D1	II	In	94.6	10.9	0.117	6.0	0.74	7.7	0.91	0.98
			Ex	117.3	-4.3	0.139	-3.5	0.89	-4.9	0.91	0.86
3	S5L65D0	II	In	139.9		0.270		0.73		0.91	0.93
			Ex	169.5		0.328		0.87		0.91	0.81
4	S5L65D1	II	In	122.3	12.6	0.254	6.0	0.68	7.6	0.91	0.93
			Ex	176.2	-4.0	0.337	-2.6	0.91	-4.4	0.91	0.81
5	S9L50D0	II	In	172.3		0.188		1.40		1.64	1.47
			Ex	158.0		0.180		1.28		1.64	1.29
6	S9L50D1	II	In	155.2	9.9	0.174	7.3	1.30	7.2	1.64	1.47
			Ex	165.7	-4.9	0.188	-4.2	1.35	-5.8	1.64	1.29
7	S9L65D0	II	In	224.2		0.400		1.24		1.64	1.38
			Ex	240.5		0.446		1.31		1.64	1.21
8	S9L65D1	II	In	205.7	8.3	0.372	6.9	1.14	7.9	1.64	1.38
			Ex	249.7	-3.8	0.459	-2.9	1.38	-4.7	1.64	1.21
9	S5L70D0	III	In	95.4		0.183		0.76		0.91	0.96
			Ex	107.2		0.209		0.85		0.91	0.84
10	S5L70D1	III	In	82.2	13.9	0.170	7.1	0.69	9.2	0.91	0.96
			Ex	113.5	-5.9	0.217	-3.7	0.90	-6.1	0.91	0.84
11	S5L90D0	III	In	118.4		0.365		0.69		0.91	0.91
			Ex	150.3		0.467		0.85		0.91	0.80
12	S5L90D1	III	In	100.3	15.3	0.339	7.1	0.62	10.1	0.91	0.91
			Ex	157.2	-4.6	0.477	-2.3	0.89	-4.7	0.91	0.80
13	S9L70D0	III	In	155.2		0.273		1.31		1.64	1.44
			Ex	151.5		0.282		1.28		1.64	1.27
14	S9L70D1	III	In	135.6	12.6	0.250	8.7	1.18	9.8	1.64	1.44
			Ex	159.9	-5.6	0.293	-4.0	1.36	-6.4	1.64	1.27
15	S9L90D0	III	In	190.8		0.535		1.17		1.64	1.36
			Ex	214.4		0.634		1.30		1.64	1.19
16	S9L90D1	III	In	164.8	13.6	0.491	8.3	1.04	10.7	1.64	1.36
			Ex	223.6	-4.3	0.650	-2.6	1.37	-5.0	1.64	1.19

Table 5.10. Results for bridges with Type IV and BT girders with 30^{o} skew

S.No	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
17	S5L95D0	IV	In	84.2		0.254		0.71		0.91	0.95
			Ex	100.8		0.310		0.84		0.91	0.83
18	S5L95D2	IV	In	76.1	9.6	0.227	10.4	0.66	9.7	0.91	0.95
			Ex	104.0	-3.2	0.319	-3.1	0.86	-2.7	0.91	0.83
19	S5L110D0	IV	In	94.4		0.376		0.65		0.91	0.92
			Ex	120.5		0.487		0.83		0.91	0.80
20	S5L110D2	IV	In	86.3	8.6	0.340	9.7	0.60	8.5	0.91	0.92
			Ex	122.3	-1.4	0.496	-1.9	0.85	-1.5	0.91	0.80
21	S9L95D0	IV	In	137.0		0.376		1.22		1.64	1.41
			Ex	143.6		0.420		1.28		1.64	1.24
22	S9L95D2	IV	In	121.6	11.2	0.331	12.1	1.08	11.3	1.64	1.41
			Ex	148.4	-3.3	0.433	-3.1	1.32	-3.3	1.64	1.24
23	S9L110D0	IV	In	152.6		0.550		1.14		1.64	1.37
			Ex	172.7		0.662		1.28		1.64	1.20
24	S9L110D2	IV	In	137.4	10.0	0.489	11.2	1.02	10.2	1.64	1.37
			Ex	176.0	-1.9	0.676	-2.0	1.31	-1.8	1.64	1.20
25	S5L105D0	BT	In	84.2		0.222		0.75		0.91	0.96
			Ex	95.1		0.260		0.85		0.91	0.84
26	S5L105D2	BT	In	72.3	14.1	0.188	15.3	0.65	13.8	0.91	0.96
			Ex	99.6	-4.7	0.272	-4.7	0.89	-4.6	0.91	0.84
27	S5L130D0	BT	In	99.0		0.387		0.69		0.91	0.91
			Ex	124.9		0.507		0.87		0.91	0.80
28	S5L130D2	BT	In	86.1	13.0	0.332	14.2	0.60	13.2	0.91	0.91
			Ex	126.4	-1.2	0.517	-2.0	0.88	-1.2	0.91	0.80
29	S9L105D0	BT	In	137.8		0.336		1.31		1.64	1.43
			Ex	135.3		0.356		1.28		1.64	1.25
30	S9L105D2	BT	In	119.0	13.6	0.286	15.0	1.13	13.9	1.64	1.43
			Ex	142.0	-4.9	0.373	-4.8	1.34	-4.8	1.64	1.25
31	S9L130D0	BT	In	160.4		0.580		1.18		1.64	1.36
			Ex	179.0		0.699		1.31		1.64	1.19
32	S9L130D2	BT	In	141.9	11.5	0.502	13.5	1.04	11.8	1.64	1.36
			Ex	182.1	-1.7	0.715	-2.3	1.34	-1.6	1.64	1.19

Table 5.11. Results for bridges with type II and III girders for 50^o skew

S.No.	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
1	S5L50D0	II	In	82.7		0.100		0.83		0.91	0.91
			Ex	102.3		0.118		0.94		0.91	0.80
2	S5L50D1	II	In	74.9	9.4	0.093	6.7	0.80	4.0	0.91	0.91
			Ex	107.3	-4.8	0.122	-3.5	1.00	-6.5	0.91	0.80
3	S5L65D0	II	In	113.9		0.226		0.70		0.91	0.87
			Ex	155.7		0.291		0.93		0.91	0.76
4	S5L65D1	II	In	95.7	16.0	0.211	6.6	0.65	6.8	0.91	0.87
			Ex	159.5	-2.4	0.297	-2.4	0.98	-6.1	0.91	0.76
5	S9L50D0	II	In	130.7		0.148		1.34		1.64	1.32
			Ex	141.8		0.158		1.40		1.64	1.16
6	S9L50D1	II	In	114.9	12.1	0.135	8.6	1.27	5.2	1.64	1.32
			Ex	146.1	-3.1	0.164	-3.9	1.49	-6.1	1.64	1.16
7	S9L65D0	II	In	179.4		0.328		1.19		1.64	1.14
			Ex	218.9		0.395		1.40		1.64	1.00
8	S9L65D1	II	In	156.3	12.9	0.301	8.4	1.12	5.8	1.64	1.14
			Ex	224.2	-2.4	0.407	-2.9	1.48	-5.1	1.64	1.00
9	S5L70D0	III	In	80.7		0.157		0.72		0.91	0.90
			Ex	99.9		0.186		0.90		0.91	0.79
10	S5L70D1	III	In	68.1	15.6	0.147	6.3	0.66	8.3	0.91	0.90
			Ex	103.8	-4.0	0.193	-3.3	0.96	-7.0	0.91	0.79
11	S5L90D0	III	In	102.7		0.323		0.67		0.91	0.86
			Ex	140.6		0.417		0.89		0.91	0.76
12	S5L90D1	III	In	85.2	17.0	0.301	6.9	0.62	8.2	0.91	0.86
			Ex	145.1	-3.2	0.426	-2.3	0.94	-5.9	0.91	0.76
13	S9L70D0	III	In	129.4		0.234		1.27		1.64	1.31
			Ex	140.7		0.253		1.36		1.64	1.15
14	S9L70D1	III	In	112.0	13.4	0.212	9.4	1.19	7.0	1.64	1.31
			Ex	146.3	-3.9	0.263	-3.8	1.45	-6.5	1.64	1.15
15	S9L90D0	III	In	163.3		0.467		1.14		1.64	1.25
			Ex	200.1		0.571		1.36		1.64	1.10
16	S9L90D1	III	In	141.1	13.6	0.428	8.4	1.05	7.5	1.64	1.25
			Ex	206.3	-3.1	0.586	-2.6	1.43	-5.2	1.64	1.10

Table 5.12. Results for bridges with type IV and BT girders for 50^{o} skew

S.No.	case	Girder Type	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflection	FEM LDF	%fem change	AASHTO STD LDF	LRFD LDF
17	S5L95D0	IV	In	74.6		0.229		0.70		0.91	0.89
			Ex	94.9		0.278		0.87		0.91	0.78
18	S5L95D2	IV	In	68.3	8.4	0.206	10.3	0.65	8.7	0.91	0.89
			Ex	97.7	-2.9	0.285	-2.9	0.90	-2.8	0.91	0.78
19	S5L110D0	IV	In	84.4		0.342		0.66		0.91	0.86
			Ex	113.8		0.437		0.86		0.91	0.76
20	S5L110D2	IV	In	78.2	7.3	0.309	9.5	0.61	7.2	0.91	0.86
			Ex	115.5	-1.6	0.445	-1.9	0.88	-1.4	0.91	0.76
21	S9L95D0	IV	In	120.0		0.335		1.20		1.64	1.29
			Ex	135.1		0.379		1.33		1.64	1.13
22	S9L95D2	IV	In	105.3	12.3	0.292	12.9	1.06	11.3	1.64	1.29
			Ex	139.5	-3.3	0.390	-2.9	1.38	-4.0	1.64	1.13
23	S9L110D0	IV	In	135.3		0.496		1.12		1.64	1.26
			Ex	162.8		0.600		1.33		1.64	1.10
24	S9L110D2	IV	In	121.4	10.2	0.439	11.4	1.01	10.1	1.64	1.26
			Ex	165.1	-1.4	0.609	-1.5	1.35	-1.7	1.64	1.10
25	S5L105D0	BT	In	76.3		0.206		0.75		0.91	0.89
			Ex	91.7		0.240		0.88		0.91	0.79
26	S5L105D2	BT	In	69.3	9.2	0.183	10.9	0.68	9.0	0.91	0.89
			Ex	95.2	-3.8	0.249	-3.8	0.91	-3.5	0.91	0.79
27	S5L130D0	BT	In	90.6		0.364		0.68		0.91	0.86
			Ex	120.4		0.469		0.88		0.91	0.75
28	S5L130D2	BT	In	83.1	8.3	0.327	10.2	0.62	8.5	0.91	0.86
			Ex	122.0	-1.3	0.478	-1.8	0.90	-1.3	0.91	0.75
29	S9L105D0	BT	In	123.5		0.308		1.27		1.64	1.49
			Ex	130.4		0.331		1.32		1.64	1.31
30	S9L105D2	BT	In	109.6	11.2	0.271	12.0	1.13	11.2	1.64	1.49
			Ex	135.1	-3.6	0.342	-3.5	1.37	-3.9	1.64	1.31
31	S9L130D0	BT	In	145.7		0.537		1.15		1.64	1.25
			Ex	172.1		0.650		1.34		1.64	1.10
32	S9L130D2	BT	In	132.8	8.9	0.479	10.7	1.04	9.6	1.64	1.25
			Ex	173.6	-0.9	0.658	-1.3	1.35	-0.7	1.64	1.10

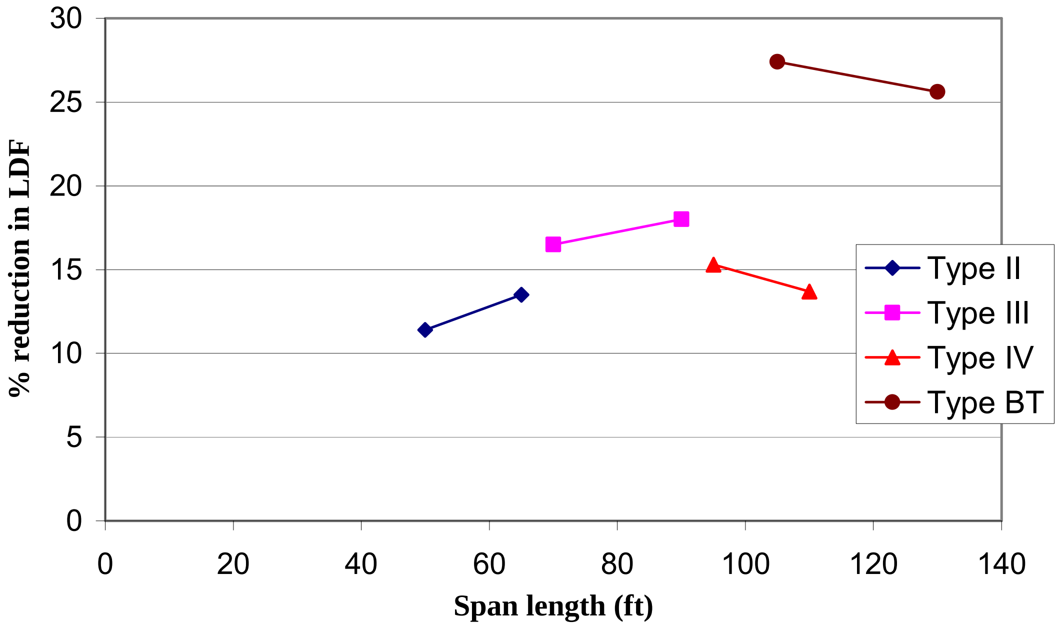

Fig 5.4. % reduction in LDF for bridges with different girder types and span lengths

to ID is greater than the bridges with Type II girder because of greater ID stiffness in bridges Type III girder. Though diaphragms in bridges with type IV girders have greater stiffness compared to the diaphragms for bridges with type III girders, there is a significant drop in percentage reduction in load distribution, which is possibly due to location of diaphragms. For bridges with Type IV girders the diaphragms are located at mid third locations, while for Type III girders the diaphragm is located at midspan. Since the LDF is calculated at the midspan, the influence of diaphragms significantly apart from the section considered would be less, which could be the reason behind lesser reduction in load distribution due to diaphragms for bridges with type IV girder. This was confirmed by considering a bridge configuration S9L90, with one and two diaphragms, where diaphragms are located at midspan(one ID Case) and mid third span(two ID case) respectively. It was observed that the percentage reduction in load distribution values for bridges due to single and two IDs are 18% and 13.2% respectively. This behavior also leads to the conclusion that the results obtained for the cases with a single diaphragm would be quite different than those obtained for bridges with two diaphragms and are to be considered separately. Though the diaphragms are away from the midspan for bridges with BT girders, the percentage reduction in LDF due to diaphragm is large when compared to bridges with other girders because of its large diaphragm section.

5.5.1.2. Influence of Girder Spacing on Effectiveness of Diaphragms

On expected lines, the influence of girder spacing on diaphragm's effectiveness in load distribution is minimal, and this could be observed clearly from the results in Table 5.13. The difference between the reduction in LDF and strain between 5/ft and 9/ft spacing is about 1% for all the cases considered. Therefore which ever spacing yields more conservative results is adopted in developing the formulae for effectiveness of ID in load distribution, which would be discussed in Chapter 6.

Table 5.13. Percentage decrease in LDF for different bridge configurations

Span length (ft)	Skew=0		Skew=30		Skew=50	
	Spacing (ft)					
	5	9	5	9	5	9
50	12	11.3	9.9	7.2	4	5.2
65	14.4	13.5	6	7.9	6.8	5.8
70	16.8	16.4	9.2	9.8	8.3	7
90	18.4	18	10.1	10.7	8.2	7.5
95	14.3	15.2	7.8	11.3	8	11.3
105	26.8	28.4	13.8	13.9	9	11.2
110	12.9	13.7	8.5	10.2	7.2	10.1
130	23.3	22.7	13.2	11.8	8.5	9.6

5.5.1.3 Influence of Span Length on Effectiveness of Diaphragms in Load Distribution

From the results in Tables 5.7 to 5.12 and also from Fig. 5.4, it was observed that span length significantly affects the diaphragm's effectiveness in load distribution. Though the range of span length used for a particular girder type is small, the difference between the reduction in LDF is as high as 6%.

Table5.14. Percentage decrease in strain for different bridge configurations

Span length (ft)	Skew=0		Skew=30		Skew=50	
	Spacing (ft)					
	5	9	5	9	5	9
50	12.2	11.4	10.9	9.9	9.4	12.1
65	14.6	13.5	12.6	8.3	16	12.9
70	17	16.5	13.9	12.6	15.6	13.4
90	18.6	18	15.3	13.6	17	13.6
95	14.4	15.3	9.6	11.2	8.4	12.3
105	26.9	27.4	14.1	13.6	9.2	11.2
110	12.9	13.7	8.6	10	7.3	10.2
130	23.3	22.6	13	11.5	8.3	8.9

5.5.1.4. Influence of Skew on Effectiveness of Diaphragms on Bridge Performance

As the skew angle increases, from the results in Tables 5.7 to 5.12, it is observed that for the bridges without diaphragm, the strain, deflection, and LDF decreases as skew increases. The decrease in strain and LDF is limited for 30^o skew and reduces significantly as skew reaches 50^o. The presence of diaphragm decreases the strains in the interior girder still further. ID effectiveness in terms of reduction of strains due to the presence of diaphragms for 0^o skew is the highest. It could be observed from the Tables 5.13 and 5.14 and also from the two plots in Figs 5.5 and 5.6, that the reduction in LDF and strain due to diaphragm show a different trend. For skewed bridges reduction in strain is greater than reduction in LDF due to diaphragm. It was noted that the trends in the figures are different for different length region since they corresponds to different girder types.

The summation of strains at the midspan section of all girders for both with and without diaphragm is the same when the diaphragm is continuous. But when the diaphragm is discontinuous (as shown in Fig.4.2), the summation of strain at the midspan is less than the case where ID is absent. This reduction in summation of strains of girders at midspan, effects the change in LDF the diaphragms cause. Therefore the change in LDF is less than the change in strain for interior girder in skew bridges considered in the current study where the diaphragms are discontinuous.

Table 5.15 shows the effect of diaphragm configuration for bridges S9L70 with different skew angles and diaphragm configurations. In this table:

$\sum\varepsilon$ = Summation of strains at girder midspan

$\Delta\varepsilon$ = % reduction in strain due to diaphragm

Δ LDF= % reduction in LDF due to diaphragm

The results clearly indicate that for discontinuous diaphragm the effect of diaphragm on reducing strain is more than that on effect of diaphragm in reducing load distribution factor.

For skew bridges the presence of diaphragm does not make as much difference in load distribution factor as it makes in case of bridges without skew. For bridges with diaphragms, LDF increases with increasing skew and similar results were obtained by

Cai et al. (2002). Another important observation made was regarding the deflection. The presence of ID reduces the deflection marginally and the deflection is within permissible limits irrespective of presence of ID.

Table 5.15. Effect of diaphragm configuration on skew bridges with spacing of 9/ft and 70/ ft span length

Skew angle	Diaphragm configuration	Maximum strain	Maximum LDF	$\sum\varepsilon$	$\Delta\varepsilon$	Δ LDF
30	No ID	155.2	1.31	472.7	---	---
30	ID continuous	137.5	1.16	474.3	11.4	11.7
30	ID discontinuous	135.6	1.18	457.8	12.6	9.8
50	No ID	129.4	1.27	406.0	---	---
50	ID continuous	123.6	1.22	406.1	4.5	4.5
50	ID discontinuous	112.0	1.19	378.1	13.4	7.0

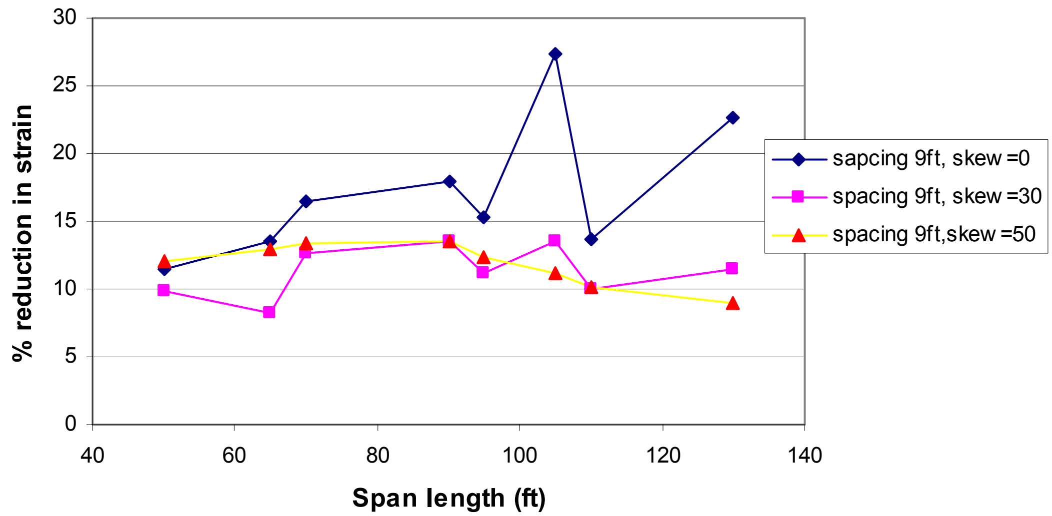

Fig 5.5. Relation between percentage reduction in strain and span length for different skew angles due to diaphragm

5.5.2. Exterior Girder

In this section the percentage change in strain and LDF caused due to ID on exterior girder has been discussed and the results in this section refer to these values. In Tables 5.16 and 5.17, these values have been summarized. As observed from the various studies like the one by Sithichaikasem and Gamble (1972), diaphragm increased the strain and LDF values for the exterior girder. The influence of diaphragm on the LDF and strain were smaller numerically, when compared to those for interior girders in Tables 5.13 and 5.14. The results in Tables 5.16 and 5.17 being close, the influence of each individual parameter is understood less clearly than the case for interior girder.

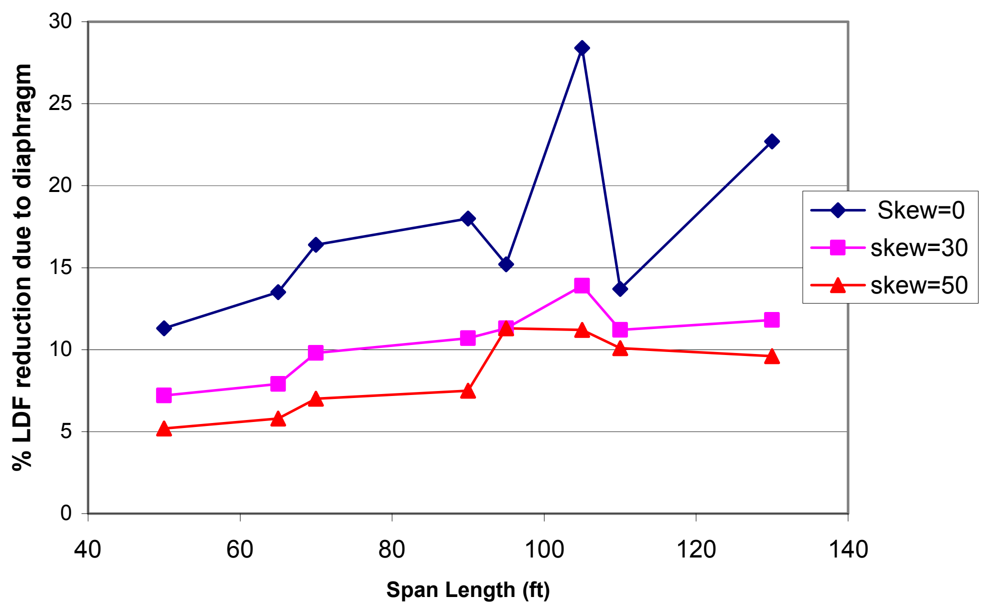

Fig 5.6. Relation between percentage reduction in LDF and span length for different skew angles

For Type II and Type III girders there is no significant difference in the results. But in case of bridges with Type IV girders, the influence of diaphragm was very small (less than 3.0%), when compared to the results for bridges with Type II and III girders.

This was because the diaphragm was located away from midspan where the strains and LDF values are compared. The same behavior was reflected by BT girders as well, but as the stiffness of diaphragm in bridges with Type BT girders being larger, its influence on load distribution and strain was higher than that of bridges with Type IV girder.

The effect of spacing seemed to be very random and also its effect was observed to be small. Therefore spacing was decided not to be considered explicitly as a parameter effecting LDF and strain values in quantifying the ID's influence on load distribution. In order to achieve conservative results, the maximum increment in LDF and strain values due to ID among the different spacing was considered.

The influence of skew on both LDF and strain followed the same trend as that for interior girders. Span length seemed to be an important parameter that could be quantified and also its influence was observed to be significant, with the maximum difference of about 4% for two different span lengths with other parameters remaining the same.

The IDs increases the deflection of the exterior girder marginally, but still the deflections are in permissible limits given by ACI code.

Table 5.16. Percentage change in strain due to diaphragm for exterior girder in different bridges

Span length (ft)	Skew=0^o		Skew =30^o		Skew = 50^o	
	Spacing (ft)					
	5	9	5	9	5	9
50	-9.2	-7.4	-4.3	-4.9	-4.8	-3.1
65	-7.3	-6.1	-4	-3.8	-2.4	-2.4
70	-9.5	-8.2	-5.9	-5.6	-4	-3.9
90	-7.1	-6.3	-4.6	-4.3	-3.2	-3.1
95	-3.8	-5.6	-3.2	-3.3	-2.9	-3.3
110	-0.8	-2.8	-1.4	-1.9	-1.6	-1.4
105	-5.9	-8.2	-4.7	-4.9	-3.8	-3.6
130	-1.8	-4	-1.2	-1.7	-1.3	-0.9

Table 5.17. Percentage change in LDF due to diaphragm for exterior girder in different bridges

Span Length (ft)	Skew=0^o		Skew =30^o		Skew = 50^o	
	Spacing (ft)					
	5	9	5	9	5	9
50	-9	-7.3	-4.9	-5.8	-6.5	-6.1
65	-7.3	-6.1	-4.4	-4.7	-6.1	-5.1
70	-10.2	-8.1	-6.1	-6.4	-7	-6.5
90	-9.2	-6.2	-4.7	-5	-5.9	-5.2
95	-3.8	-5.6	-2.7	-3.3	-2.8	-4
110	-0.7	-2.8	-1.5	-1.8	-1.4	-1.7
105	-5	-8.2	-4.6	-4.8	-3.5	-3.9
130	-1.8	-4	-1.2	-1.6	-1.3	-0.7

6. FORMULAE DEVELOPMENT FOR DETERMINING THE EFFECTIVENESS OF DIAPHRAGM

6.1. Introduction

One of the important objectives of this research is to develop correction factors for load distribution factors to account for the influence of diaphragms in load distribution. In this unit the deduction of the formulas to calculate the diaphragm effect on load distribution based on the results obtained from the parametric study and the accuracy of these formulas developed have been discussed. When these correction factors are multiplied with the LDF, which were obtained without considering diaphragm, it gives LDF values which accounts for diaphragm effects.

6.2. Procedure

The effect of IDs as mentioned in Section 5.2.1 is gauged in terms of percentage change in LDF value for the case where diaphragm influence is considered to the case where diaphragm influence is not considered. This is given by the expression ((LDF value $_{\text{without ID}}$ - LDF value $_{\text{with ID}}$)/ LDF value $_{\text{without ID}}$)* 100 and this value here after is referred as R_d. The R_d values for bridges for interior girders and exterior girders have been presented in Table 6.1 and Table 6.2 respectively.

The ID effectiveness is based on several parameters and in our study only span length, ID stiffness, stiffness of girder, girder spacing and skew angle were considered as the possible parameters influencing ID effectiveness. The initial attempts were made to develop a single general formula to calculate the reduction factors involving all the parameters considered. This was done by combining parameters in several possible ways, while keeping in view the actual behavior of bridge in relation to these parameters. The above mentioned task could not be accomplished, as the results obtained by combining the influence of each individual parameter did not converge with the actual results. This could be because all the parameters might not have been considered and importantly the parameters are related to each other in a complicated manner.

Table 6.1. R_d values for interior girders

Span length (ft)	Skew=0°		Skew =30°		Skew = 50°	
	Spacing (ft)					
	5	9	5	9	5	9
50	12	11.3	9.9	7.2	4	5.2
65	14.4	13.5	6	7.9	6.8	5.8
70	16.8	16.4	9.2	9.8	8.3	7
90	18.4	18	10.1	10.7	8.2	7.5
95	14.3	15.2	7.8	11.3	8	11.3
105	26.8	28.4	13.8	13.9	9	11.2
110	12.9	13.7	8.5	10.2	7.2	10.1
130	23.3	22.7	13.2	11.8	8.5	9.6

Table 6.2. R_d values for exterior girders

Span Length (ft)	Skew=0°		Skew =30°		Skew = 50°	
	Spacing (ft)					
	5	9	5	9	5	9
50	-9	-7.3	-4.9	-5.8	-6.5	-6.1
65	-7.3	-6.1	-4.4	-4.7	-6.1	-5.1
70	-10.2	-8.1	-6.1	-6.4	-7	-6.5
90	-9.2	-6.2	-4.7	-5	-5.9	-5.2
95	-3.8	-5.6	-2.7	-3.3	-2.8	-4
110	-0.7	-2.8	-1.5	-1.8	-1.4	-1.7
105	-5	-8.2	-4.6	-4.8	-3.5	-3.9
130	-1.8	-4	-1.2	-1.6	-1.3	-0.7

As single formulae for determining diaphragm effectiveness for all girders and parameters was not successful, different sets of formulae were developed systematically in parts. From the values of R_d in Table 6.1 and Table 6.2 it could be observed that diaphragm effect on LDF for exterior girder and interior girder is very different. Hence the formulae development for diaphragm effect for interior and exterior girder was carried out differently. As the effectiveness of diaphragm depends on the location of diaphragm (discussed in Section 5.5.1.1), different formulae have been developed for bridges with single ID and two IDs. At the same time accounting the influence of skew seemed to be a difficult task, therefore the ID effect on LDF for right and skew bridges has also been carried out separately.

From results in Table 6.1 and Table 6.2 it could be observed that spacing does not have much influence on R_d hence not been considered as a parameter in the formulae being developed and the R_d value which is more conservative is used in developing formulae.

In this study, wherever the number of cases considered in the previous parametric study carried out in Chapter 5 is found to be insufficient in describing the influence of a particular parameter, analysis was done for more bridge configurations. The bridge configurations were chosen by limiting parameters within permissible limits and distinct from the cases considered earlier.

According to LADOTD regulations bridges with span lengths less than 95/ft and greater than 50/ft are to be provided with a single diaphragm. In this range of span lengths only three standard AASHTO prestressed concrete girder types, Type II, Type III and Type IV are possibly used. Bridges with Type IV girder were also analyzed for bridges with span lengths less than 95/ft and having a single diaphragm keeping in view the possibility of using this girder in bridges with span lengths less than 95/ft. While for bridges with span lengths greater than 95/ft, two diaphragms are provided and for bridges above this span lengths only Type IV and Type BT, AASHTO standard prestressed concrete girders are used. Hence only bridges with these girder types over the span length of 95/ft are considered to have two diaphragms.

6.3. Formulae Development for R_d for Interior Girders in Bridges With a Single ID

6.3.1. R_d for Right Bridges

The formula development is started with building a relationship between span length and diaphragm effectiveness. A plot was drawn between length and R_d values for various bridge configurations with different girder types (Fig 6.1). The results appear to be very much scattered, but the results for each girder type lie fairly on a straight line. When a linear fit was used, R^2 value for all the datasets for each girder type was found to be greater than 0.95. Hence a linear relationship between length and R_d has been assumed, and the linear trend lines were observed to be nearly parallel to each other. The trend line equation for Type II girder is

$$R_d = 0.1319*L + 4.85 \qquad \textbf{(6.1)}$$

Where

L = span length in ft.

R_d = % change in load distribution due to diaphragm

The Eq. (6.1) is taken as a basic equation for R_d and a constant term C is added to it, to generate a set of new equations which are close to the results for Type III and Type IV girders. The values of C are taken as 2 and 3.5 for Type III and Type IV girders respectively which were obtained based on the mean difference between the results for diaphragm effectiveness through Eq. (6.1) and the actual results for bridges with Type III and Type IV girders.

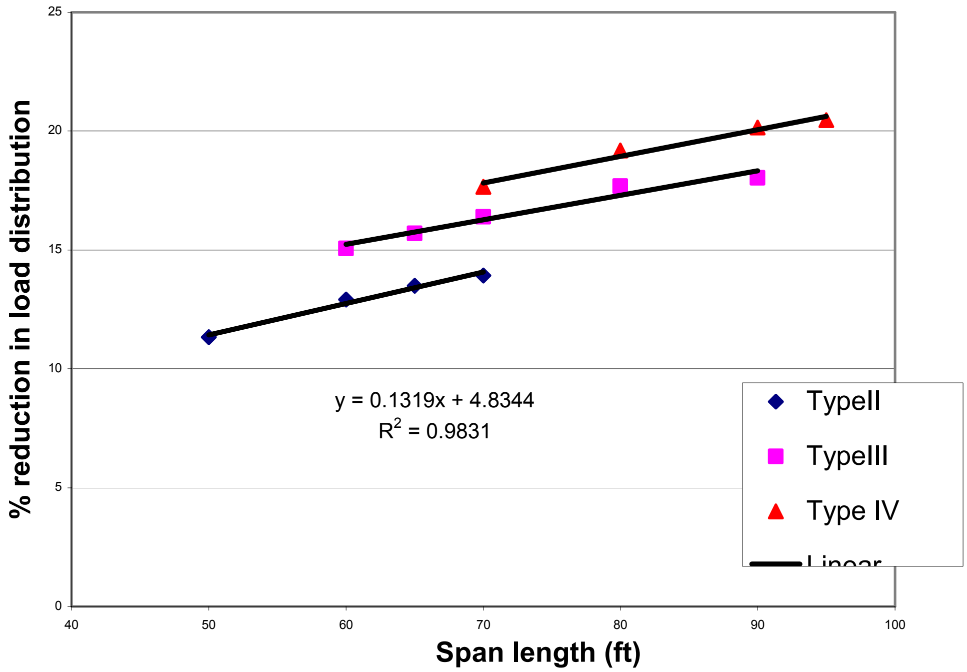

Fig 6.1 Length VS % reduction in load distribution (R_d) due to single diaphragm

Hence a general equation for a single diaphragm for bridges with different girder types could be written as

$$R_d = [(0.1319*L + 4.85) + C] \qquad \textbf{(6.2)}$$

Where

L= Span length in ft

C = 0, 2 and 3.5 for girder Types II, III and IV respectively

R_d = % reduction in load distribution due to diaphragm

The Eq. (6.2) is based upon absolute stiffness (100%) contribution of diaphragm in load distribution. As discussed earlier in reality only a portion of diaphragm's total stiffness might be effecting load distribution due to cracking at the diaphragm girder interface. R_d is clearly a function of the effective stiffness contribution of diaphragm and there is a need to develop a relationship between these two factors to take into account the influence of stiffness which otherwise might lead to non conservative results. Determining the exact stiffness contribution of diaphragm in load distribution is beyond the scope of this work, as this requires a nonlinear finite element analysis or extensive experimental work to know the extent of cracking at the diaphragm-girder interface. The analysis in the present study is based on a known effective stiffness of diaphragms.

Analysis is done for more bridges, with diaphragms of different stiffness for each girder type. The stiffness of diaphragm is varied by altering the Young's modulus of elasticity in the finite element model. For example if 30% of diaphragm stiffness contributes to load distribution, this case was modeled by taking the E value of compressive strength of concrete in diaphragm to be 30 % of the actual E value. A portion of absolute value of the stiffness of diaphragm is taken for few cases and the R_d is obtained for the new diaphragm stiffness value. Ratio between R_d values for possible diaphragm stiffness contributing to load distribution and R_d value for absolute value (100%) of diaphragm stiffness is calculated and this factor is represented as S_t.

Values of S_t for different lengths for the same girder type have been presented in Table 6.3, where two different span lengths of 50 and 65/ ft were considered. The results indicated that values of S_t for different lengths for the same stiffness ratio is nearly the same, i.e. the S_t values for particular girder type and stiffness ratio are independent of span length. Therefore only one span length for each girder type was chosen for which S_t values were obtained as shown in Table 6.3. The S_t values increase from Type II girder to Type IV girder for a particular diaphragm stiffness ratio but these values are close to each other for all girders with the maximum difference among them, less than 10% that will represent a much smaller effect on the load distribution (perhaps less than 1%). Therefore instead of using 3 different equations for S_t values, a single equation for S_t is deduced, which is obtained by exponential fitting of S_t values for Type II girder (Fig. 6.2) as choosing this would yield conservative results for most of the cases.

$$S_t = 0.0264*X^{0.8062} \qquad \textbf{(6.3)}$$

where X = (possible diaphragm stiffness effecting load distribution/ absolute diaphragm stiffness)*100

S_t = Stiffness reduction factor.

Table 6.3. S_t Values for two different span lengths

Diaphragm stiffness ratio (% of abs. stiff)	Span Length (ft)	
	50	65
100	1	1
60	0.762	0.73
30	0.443	0.447

Table 6.4. S_t Values for various girder types with different diaphragm stiffness

Diaphragm stiffness ratio(% of abs. stiff)	Girder type		
	II	III	IV
100	1	1	1
60	0.726	0.762	0.768
30	0.443	0.493	0.512

When the right hand side of Eq. (6.2) is multiplied with the S_t value, the new equation so formed takes the influence of diaphragm stiffness into account and the equation would be

$$R_d = [(0.1319*L + 4.85) + C] *S_t \qquad \textbf{(6.4)}$$

where L= Span length in ft ,

C = 0, 2 and 3.5 for Type II, III and IV girders respectively

S_t = $0.0264*X^{0.8062}$ = Stiffness reduction factor = (R_d for possible diaphragm stiffness / R_d for absolute diaphragm stiffness)

X = (Possible diaphragm stiffness contributing to load distribution / absolute diaphragm stiffness)*100

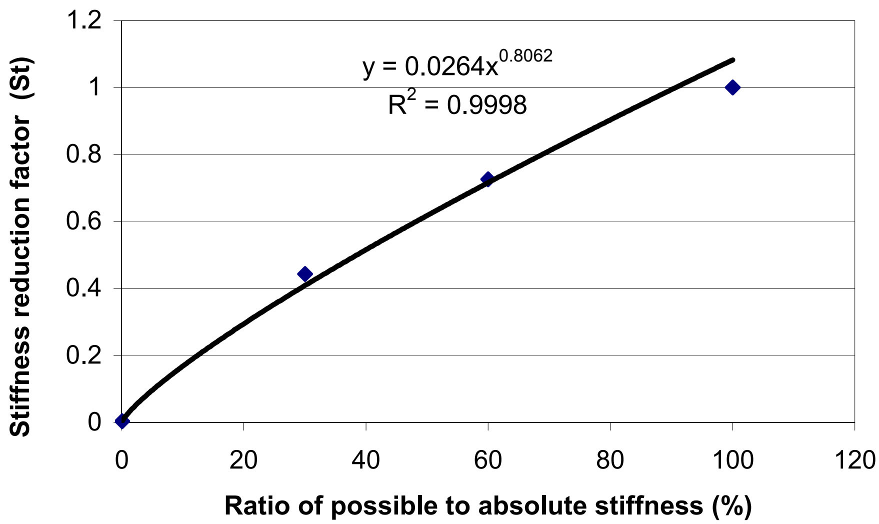

Fig 6.2 Ratio of possible to absolute diaphragm stiffness in percentage VS stiffness reduction factor (S_t)

Comparison between R_d values obtained from FEM and Eq. (6.4) for bridges with single diaphragm is presented in Table 6.5. The maximum difference in the corresponding values of FEM and Eq. (6.4) is less than 1.5%, thereby indicating that the formula is accurate.

6.3.2. Influence of Skew

Accounting the influence of skew on diaphragm performance in load distribution is very difficult as the effect of skew is influenced by several parameters in a complicated manner. The results from the parametric study indicate that percentage reduction in LDF is lesser than percentage reduction in strain. This could be observed by comparing results in Table 6.6 and Table 6.7 where percentage reduction in strain and LDF are presented respectively for different bridge configurations. The reason for this behavior has been explained in Section 5.5.1.3. As taking the percentage reduction in LDF as the representative value leads to a conservative estimate of diaphragm effect, it has been chosen as the parameter defining the diaphragm effect in skew bridges.

Table 6.5. Comparison between R_d value by FEM and Eq. (6.4) for bridges with single diaphragm

Bridge Configuration	Diaphragm Stiffness Ratio X (% of abs stiffness)	% reduction in LDF due to diaphragm	
		FEM	Factored
S9L70D1	40	9.7	8.5
S9L90D1	60	12.1	13.4
S9L60D1	30	5.76	5.68
S9L80D1 (Type IV girder)	50	13.17	13.23
S9L65D1	20	4.3	3.9

Table 6.6. Percentage reduction in strain values for interior girder at midspan

Skew	0^o		30^o		50^o	
Spacing (ft)	5	9	5	9	5	9
Span (ft)						
50	12.2	11.4	10.9	9.9	9.4	12.1
65	14.6	13.5	12.6	8.3	16	12.9
70	17	16.5	13.9	12.6	15.6	13.4
90	18.6	18	15.3	13.6	17	13.6

Table 6.7. Percentage reduction in LDF values for interior girder at midspan

Skew	0^o		30^o		50^o	
Spacing (ft)	5	9	5	9	5	9
Span (ft)						
50	12	11.3	7.7	7.2	5	5.2
65	14.4	13.5	7.6	7.9	6.8	5.8
70	16.8	16.4	9.2	9.8	8.3	7
90	18.4	18	10.1	10.7	8.2	7.5

The ratios between R_d for skew bridges to R_d for right bridge of the same span length with 9ft spacing are calculated and these values are presented in Table 6.8. From the results in this table, skew reduction factors of 0.55 and 0.4 were safely chosen for skew angles of 30^o and 50^o respectively. Skew reduction factor (S_k) at skew = 0 degrees would naturally be equal to one. The skew reduction factors for other intermediate skew angles were assumed to be linearly related between any two successive limits. As shown in Fig 6.3, the skew reduction factor could be written in the form of

$$S_k = 1-0.015*\theta \qquad (\text{When } \theta \leq 30^o)$$

$$S_k = 0.775 - 0.0075 * \theta \qquad (\text{When } \theta > 30^o) \qquad \textbf{(6.5)}$$

where S_k = Skew reduction factor

θ = Skew angle in degrees

Table 6.8. Ratio of R_d for skewed bridge to the R_d of equivalent right bridge, with same span length and 9/ft spacing.

Span Length (ft)	Skew = 30^o		Skew=50^o	
	Spacing (ft)			
	5	9	5	9
50	0.681	0.637	0.442	0.460
65	0.563	0.585	0.504	0.430
70	0.561	0.598	0.506	0.427
90	0.561	0.594	0.456	0.417

The accuracy of the skew reduction factor determined is checked by conducting a study on two bridges for different skew angles and the results of which are presented in Tables 6.9 and 6.10. The maximum difference between the R_d obtained by applying the skew factor developed and that obtained through FEM analysis is less than 1%, for all the skew angles considered thereby indicating that the factors chosen were appropriate for all skew angles. Hence the skew reduction factor chosen is appropriate.

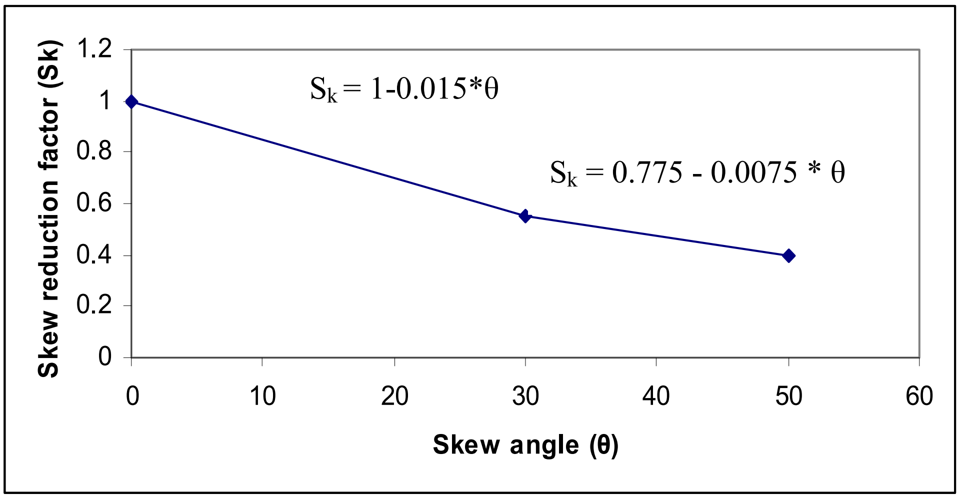

Fig 6.3. Skew reduction factor VS skew angle

Table 6.9. Accuracy of reduction factor for skew in S9L90 bridge configuration for various skew angles

Skew angle (degrees)	Skew reduction factor	R_d values for skewed bridge		
		FEM Analysis(1)	Factored(2)	(1) - (2)
15	0.775	13.6	13.8	-0.2
30	0.55	10.7	9.8	0.9
40	0.475	8.6	8.5	0.2
50	0.4	7.9	7.1	0.8
60	0.325	6.2	5.8	0.4

Table 6.10. Accuracy of reduction factor for skew in S9L70 bridge configuration for various skew angles

Skew	Skew reduction factor	R_d values for skewed bridge		(1) - (2)
		FEM Analysis (1)	Factored (2)	
8	0.88	14.9	14.6	0.4
15	0.775	12.8	13.8	-1.0
30	0.55	9.9	9.1	0.8
40	0.475	7.6	7.9	-0.2
50	0.4	6.3	6.6	-0.3

The results obtained for skew bridges considered so far, were obtained by considering that the absolute stiffness of diaphragm contributes to load distribution. But in reality only a portion of diaphragm stiffness (as mentioned for right bridges) effects the load distribution. In order to take into account the actual stiffness contribution of diaphragm in load distribution, a stiffness modification factor is to be applied to results obtained for diaphragm effectiveness by considering absolute diaphragm stiffness. Before determining an expression for new stiffness reduction factor (S_t) for skew bridges, the stiffness reduction factor determined for right bridges is checked to see whether this holds good for skew bridges as well. Some skew bridges were analyzed by taking partial stiffness of diaphragm. R_d values obtained by analyzing bridges for partial diaphragm stiffness is compared to the value obtained by multiplying stiffness correction factor(S_K) to the R_d value obtained for absolute diaphragm stiffness for the respective bridge. This comparison is listed in Table 6.11. Results indicate that the R_d values obtained from FEM analysis for bridges with diaphragms contributing partial stiffness, is practically close to values obtained by applying stiffness reduction factor of right bridges to R_d values obtained by considering absolute diaphragm stiffness. Therefore stiffness reduction factor for right bridges can be safely applied for skew bridges.

By including the skew reduction factor in determining the R_d value, all the parameters considered to affect diaphragm effectiveness in reducing the load distribution factor have been covered.

Table 6.11. Checking the suitability of stiffness reduction factor of right bridges for skew bridges.

Bridge configuration	Skew	% Stiffness ratio	R_d absolute	S_t	R_d Factored	R_d FEM analysis	(1)-(2)
S9L90	30	30	10.1	0.41	4.15	6.4	-2.25
S9L70	50	30	8.3	0.41	3.4	3.9	-0.50
S9L50	50	30	5.2	0.41	2.13	2.4	-0.27
S9L65	50	30	5.8	0.41	2.38	2.52	-0.14
S5L70	30	20	8.3	0.30	2.45	4	-1.55

Finally R_d value can be summarized by the following expression:

$$R_d = [(0.1319*L + 4.85) + C] * S_t * S_k \quad \textbf{(6.6)}$$

Where

L= span length in ft,

C = 0, 2 and 3.5 for Type II, III and IV girders respectively

$S_t = 0.0264*X^{0.8062}$ = Stiffness reduction factor. (= R_d for possible diaphragm stiffness / R_d for absolute diaphragm stiffness)

X = (Possible diaphragm stiffness contributing to load distribution / absolute diaphragm stiffness)*100

$S_k = 1-0.015*\theta$ (When $\theta \leq 30^o$)

$= 0.775 - 0.0075 * \theta$ (When $\theta > 30^o$)

S_k = Skew reduction factor

θ = Skew angle in degrees

The load distribution factor considering the diaphragm effects can be therefore written in the following format:

$$(LDF)_{WD} = (1 - R_d / 100)*(LDF)_{ND} \qquad \textbf{(6.7)}$$

Where

$(LDF)_{WD}$ = Load distribution factor for bridges by including diaphragm effectiveness in load distribution

$(LDF)_{ND}$ = Load distribution factor for bridges without considering diaphragm effectiveness in load distribution

6.4. Formulae Development for R_d for Interior Girders in Bridges with Two Diaphragms

For deducing formulae for determining diaphragm effectiveness for bridges with two diaphragms**,** similar approach as that used in developing R_d for bridges with a single diaphragm was adopted.

6.4.1. R_d for Right Bridges

The process of deducing formulae for determining R_d for bridges with two diaphragms was initiated for right bridges. Fig 6.4 shows the effect of R_d for both Type IV and Type BT girders. It was observed that for bridges with both these girder types, R_d decreases with increasing span length. A linear trend line fits the R_d values for each girder type well, with the R^2 value greater than 0.97. The two trend lines unlike for the trend line obtained for R_d values for bridges with a single diaphragm are not parallel to each other. But the ratio between R_d values for bridges with Type BT to that with Type IV girders, for the same lengths of span (R_d values for bridges with span lengths other than those analyzed are obtained from the trend line equations) is close to 1.98 for all the cases. By these observations the equation for R_d is expressed as

$$R_d = (-0.112*L + 25.814) * C \qquad \textbf{(6.8)}$$

Where R_d = % reduction in load distribution due to diaphragm

L = span length of girder in ft

C = 1 and 1.98 for Type IV girder and BT girder respectively

The Eq. (6.8) does not account for the effect of stiffness of the diaphragm contributing effectively in load distribution. For bridges with two diaphragms the S_t (stiffness correction factor) values are determined by taking partial diaphragm stiffness like for bridges with a single diaphragm. A plot is drawn between the ratio of stiffness of diaphragm contributing in load distribution to the absolute diaphragm stiffness (S_r) and stiffness reduction factor (S_t) values (Fig. 6.5). Including the stiffness factor to the right hand side of Eq. (6.8) would give a new formula for R_d where the actual stiffness of diaphragm is taken into consideration as:

$$R_d = (-0.112*L + 25.814) * C*S_t \qquad \textbf{(6.9)}$$

where C = 1 for Type IV girder

C = 1.98 for BT girder

R_d = % reduction in load distribution factor by considering diaphragm effectiveness

$S_t = 0.0873 * X^{0.5358}$ for Type IV girder

$=0.3024 * X^{0.2641}$ for type BT girder

= (R_d for possible diaphragm stiffness / R_d for absolute diaphragm stiffness).

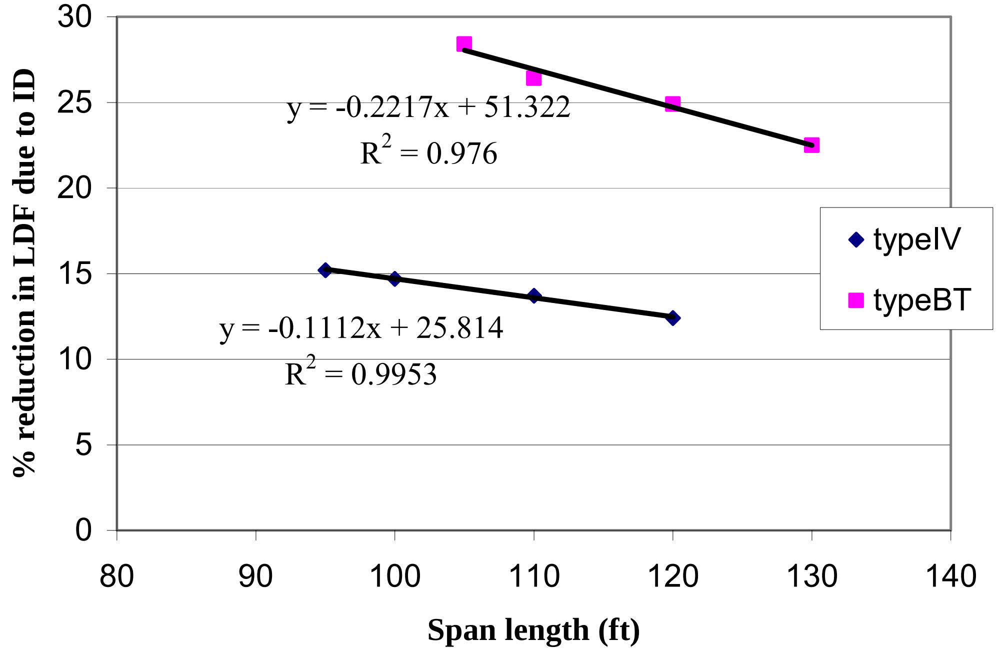

Fig6.4. Span length VS reduction in LDF due to diaphragm in bridges with two diaphragms

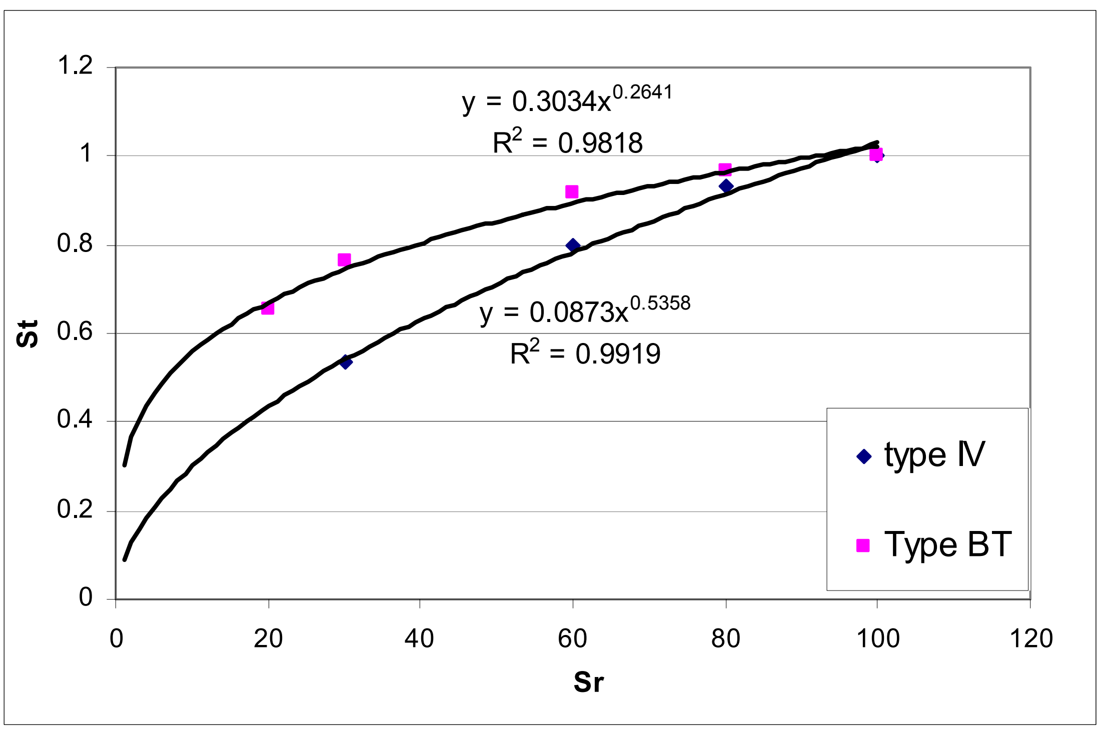

Fig6.5. Diaphragm stiffness ratio VS stiffness reduction factor

Table 6.12 shows the comparison between the values of R_d obtained from finite element analysis and the value obtained from Eq. (6.9). The difference in results from finite element analysis and Eq. (6.9) is less than 1% for all the cases considered which proves the accuracy of the formula.

6.4.2. Influence of Skew

Attempt was made to determine the influence of skew on diaphragm effectiveness on load distribution, in similar lines as that for bridges with a single diaphragm. Table 6.13 shows R_d values for bridges having two diaphragms. The ratios between R_d values for skew bridges to the R_d values for right bridge of the same span length with 9ft spacing are calculated and these values are presented in Table 6.14. From these results, skew reduction factors of 0.5 and 0.35 were safely chosen for skew angles of 30 and 50 degrees

Table 6.12. Comparison of R_d values obtained by analysis to the values obtained by applying stiffness reduction factor to R_d obtained for absolute diaphragm stiffness

Bridge configuration	Diaphragm Stiffness (% of abs stiffness)	% reduction in LDF due to diaphragm (R_d)	
		FEM	Factored
S9L105	25	19.76	19.75
S9L120	20	15.7	16.3
S9L130	40	18.8	18.16
S9L125	25	17.01	16.6
S9L110	40	9.0	8.63

respectively. Skew reduction factor (S_k) at skew = 0 degrees would naturally be equal to one. As shown in Fig 6.6, the skew reduction factors for other intermediate skew angles were assumed to be linearly related between any two successive limits obtained earlier. The skew reduction factor could be written in the form of

$S_k = 1 - 0.0167*\theta$ (When $\theta \leq 30^o$) **(6.10)**

$S_k = 0.725 - 0.0075 * \theta$ (When $\theta > 30^o$)

Where

S_k = Skew reduction factor

θ = Skew angle in degrees

Table 6.13. R_d values for interior girder at midspan in bridges with two diaphragms

Girder type	Skew	0^o		30^o		50^o	
	Spacing (ft)	5	9	5	9	5	9
	Span (ft)						
IV	95	14.3	15.2	9.66	11.3	8.7	11.3
IV	100	14.49	15	7.26	10.8	7.3	11.1
IV	110	12.9	13.7	8.5	10.2	7.2	10.1
IV	120	12.3	13.9	8.2	9.25	7.14	8.4
BT	105	26.8	28.4	13.8	13.9	9	11.2
BT	115	26.44	25.36	14.1	12.47	8.74	10.57
BT	130	23.3	22.7	13.2	11.8	8.5	9.6

When the factor S_k is multiplied to the right side of Eq. (6.8), the new R_d value developed takes into account the influence of skew. The equation for R_d could be expressed as

$R_d = (-0.112*L + 25.814) * C*S_k$ **(6.11)**

where L = span length in ft

C = 1 for Type IV girder

C = 1.98 for BT girder

$S_k = 1 - 0.0167*\theta$ (When $\theta \leq 30^o$)

$S_k = 0.725 - 0.0075 * \theta$ (When $\theta > 30^o$)

R_d = % reduction in load distribution factor by considering diaphragm effectiveness

S_k = Skew reduction factor

θ = Skew angle in degrees

Table 6.14. Ratio of R_d value for skewed bridge to R_d value of right bridge of same span length and with 9ft spacing.

Span Length (ft)	Skew = 30^o		Skew=50^o	
	Spacing (ft)			
	5	9	5	9
95	0.636	0.743	0.572	0.743
100	0.484	0.720	0.487	0.740
110	0.635	0.745	0.526	0.737
120	0.590	0.665	0.514	0.604
105	0.486	0.489	0.317	0.394
115	0.559	0.492	0.345	0.417
130	0.581	0.520	0.374	0.423

The expression for R_d in Eq. (6.11) is obtained by considering total diaphragm stiffness. In order to obtain diaphragm effectiveness due to actual diaphragm stiffness contributing to load distribution, a stiffness reduction factor is to be applied to the already determined diaphragm effectiveness factor in Eq. (6.11). It was checked whether the stiffness reduction factor obtained for right bridges is suitable for skew bridges with two diaphragms and the results of which are presented in Table 6.15. From the results it could be observed that for all the cases the stiffness reduction factor (S_t) for straight bridges yields safe results. Though there exists a difference of about 1.5 to 2% between actual results to that obtained from using stiffness reduction factor, for the purpose of maintaining uniformity the same expression of stiffness reduction for right bridges is used for skew bridges as well.

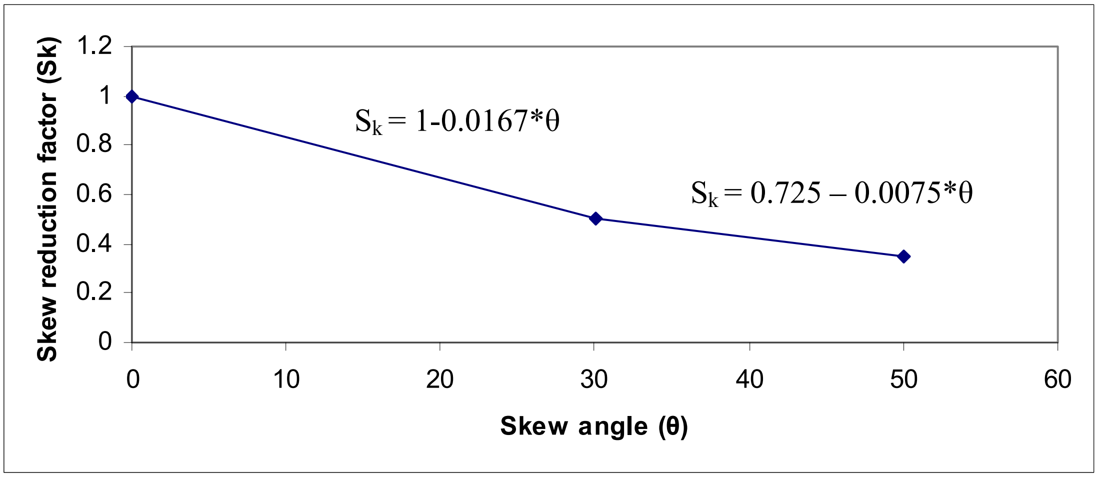

Fig 6.6 Skew angle VS skew correction factor, for LDF in interior girders of bridges with two diaphragms

Table 6.15. Checking the suitability of stiffness reduction factor of right bridges for skew bridges with two diaphragms.

Bridge configuration	Skew	% Stiffness ratio	R_d absolute	S_t	R_d Factored	R_d FEM analysis	(1)-(2)
S9L95	30	40	11.3	0.63	7.12	8.6	-1.48
S9L110	50	50	10.1	0.71	7.17	8.7	-1.53
S9L105	30	30	13.6	0.75	10.13	11.7	-1.57
S9L130	30	30	11.5	0.75	8.6	10	-1.4
S9L105	50	30	11.2	0.75	8.4	10.1	-1.7

6.5. Diaphragm Effect on Exterior Girder

The diaphragm influence so far discussed is for its influence on load distribution for interior girders and for all the cases the diaphragms decreased the strain and load distribution factors. But diaphragm at the same time increases the loading effect on exterior girders. The impact of diaphragm is measured in terms of values of strain, deflection and load distribution. From the results in Table 6.16 and Table 6.2, it could be observed that the increase in strain and load distribution values due to diaphragm, could reach as high as 9% of the original values where diaphragm were absent.

Table 6.16. % change in strain due to diaphragm for exterior girder in different bridges

Span Length (ft)	Skew=0°		Skew =30°		Skew = 50°	
	Spacing (ft)					
	5	9	5	9	5	9
50	-9	-7.3	-4.9	-5.8	-6.5	-6.1
65	-7.3	-6.1	-4.4	-4.7	-6.1	-5.1
70	-10.2	-8.1	-6.1	-6.4	-7	-6.5
90	-9.2	-6.2	-4.7	-5	-5.9	-5.2
95	-3.8	-5.6	-2.7	-3.3	-2.8	-4
110	-0.7	-2.8	-1.5	-1.8	-1.4	-1.7
105	-5	-8.2	-4.6	-4.8	-3.5	-3.9
130	-1.8	-4	-1.2	-1.6	-1.3	-0.7

The formulae development for exterior girder is not much different from that of interior girder. Along with the other factors which impact the diaphragm effectiveness for interior girder, another important factor considered for exterior girder is the lateral position of loading. The lateral position of loads in relation to the exterior girder varies based on the width of the overhang and barrier. In the parametric study, the wheel line of truck closest to the bridge edge was 1/ft away from the center of exterior girder, due to specific length of the slab chosen. The lateral position of loads has a large effect on the straining action and load distribution for exterior girder and is possible that it has an influence on the effect of diaphragm on load distribution for exterior girders.

Another important difference is in the study for exterior girder, diaphragm stiffness is not considered as a parameter. This is because it was already understood that the diaphragm causes a detrimental effect on exterior girder by increasing the LDF values and straining actions. The absolute stiffness contribution of diaphragm gives the upper bound value of the amount of increase in LDF value due to diaphragm, which is the result of interest. Hence diaphragm stiffness was not considered as parameter, i.e., the absolute diaphragm stiffness is used for exterior girders.

6.6. Formulae Development for R_d for Exterior Girder in Bridges with Single ID

6.6.1. R_d for Right Bridges

For right bridges with a single diaphragm, with increasing span length the effect of diaphragm on load distribution decreases for exterior girders as could be observed in Fig. 6.7. For these bridges, the effect of diaphragms on exterior girders based on span length for bridges of different girder types could be given by the following expression:

$$R_d = (0.1319 * L - 15.812 - C) \qquad (6.12)$$

Where L= span length in ft

C = 0, 3 and 5 for Type II, III and IV girders respectively

R_d = % reduction in load distribution due to diaphragm

To understand the influence of lateral position of loads on ID effect on bridge performence, analysis was carried out for bridges S9L90 and S5L90 for both with and without diaphragm for different lateral position of loads. The lateral positioning of the loading system was defined by the distance between the closest wheel line to the center of exterior girder. For this study this distance adopted for lateral loading system was 0, 1, 2 and 3/ft respectively, without any change in longitudinal position of loading. The results are presented in Tables 6.17 and 6.18. From the results it could be observed that that the effect of diaphragm on load distribution is clearly a function of the position of loading and is necessary to take the influence of this factor into account. It was also observed that the results obtained here are function of spacing as well with the bridges with 5ft spacing having larger R_d values. But the effect of spacing is not significant when the loading system is closer to the exterior girder. And it is when the loading system is closer to the exterior girder, the design moment for exterior girder governs the design. Keeping this in view the effect of spacing has not been considered. Fig 6.8 shows how position of loading affects the influence of diaphragm on load distribution in exterior girders.

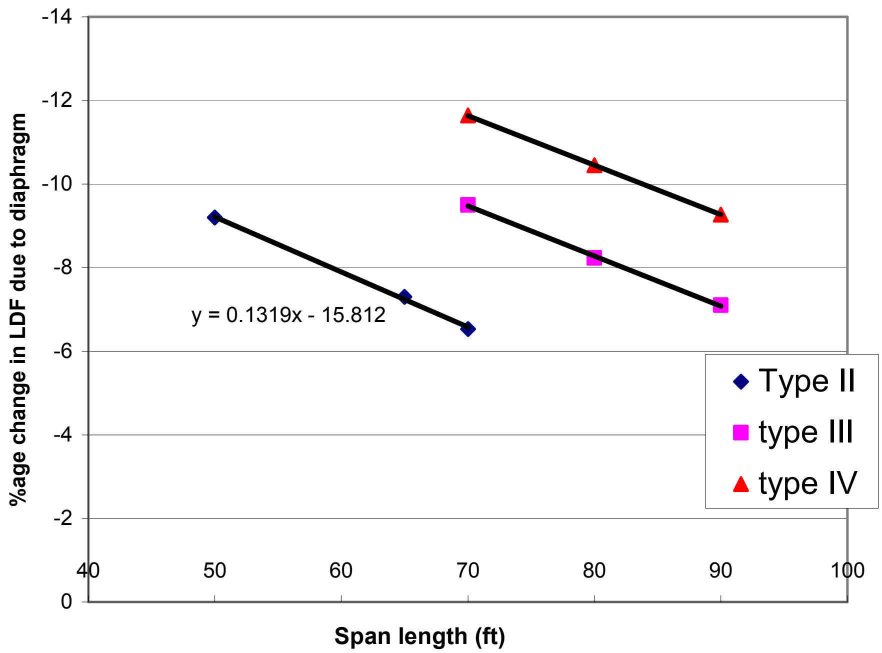

Fig 6.7. Effect of span length on R_d

From the results obtained, the following correction factor was obtained which when applied to right hand side of Eq. (6.12), takes into account the position of loading in calculating diaphragm effect on load distribution for exterior girders.

$P_L = 0.45+0.55* d$ $(0 \leq d \leq 3ft)$ **(6.13)**

Where

d = Distance between the position of wheel line closest to the exterior girder from the middle of the exterior girder in ft. (Loading must be done laterally as close as possible to bridge edge so as to generate maximum loading action on exterior girder)

P_L = Modification factor to take into account the effect of position of loading system in lateral direction for exterior girder.

Table 6.17. Effect of lateral position of loading on exterior girder for bridge with 9ft spacing and 90 ft length

First wheel line distance from center of ext. girder (d in ft)	D0 strain	D1 strain	% change in strain	D0 LDF	D1 LDF	% change in LDF
0	242.6	251.2	-3.5	1.40	1.45	-3.571
1	221.6	235.5	-6.3	1.28	1.36	-6.25
2	200.9	219.6	-9.3	1.16	1.27	-9.483
3	180.9	203.7	-12.6	1.05	1.18	-12.51

Table 6.18. Effect of lateral position of loading on exterior girder for bridge with 5ft spacing and 90 ft length

1st wheel line distance from center of Ext girder (d in ft)	D0 strain	D1 strain	% change in strain	D0 LDF	D1 LDF	% change in LDF
0	172.8	178.6	-3.4	0.93	0.96	-3.3
1	155.3	166.4	-7.1	0.84	0.92	-7.4
2	138.0	153.5	-11.2	0.74	0.83	-11.3
3	121.8	140.7	-15.5	0.66	0.76	-15.6

The correction factor to account for the lateral position of loading (P_L), in Eq. (6.13) was deduced for bridges with 90 ft span length and with Type III girders. Therefore it was necessary to check whether, this factor determined holds good for other bridge geometries. Few other bridge configurations were analyzed for different loading positions to check the suitability of these reduction factors and the results of which are listed in Table 6.19. The difference between the R_d values obtained from FEM analysis for particular loading configuration and those obtained by applying correction factor of Eq. (6.13) to the R_d values obtained for the same bridges with the closest wheel to exterior girder and 1ft away from exterior girder, is less than 2%. As the difference between these values being very small, the factors in equation could be applied confidently for all right bridges with single diaphragms.

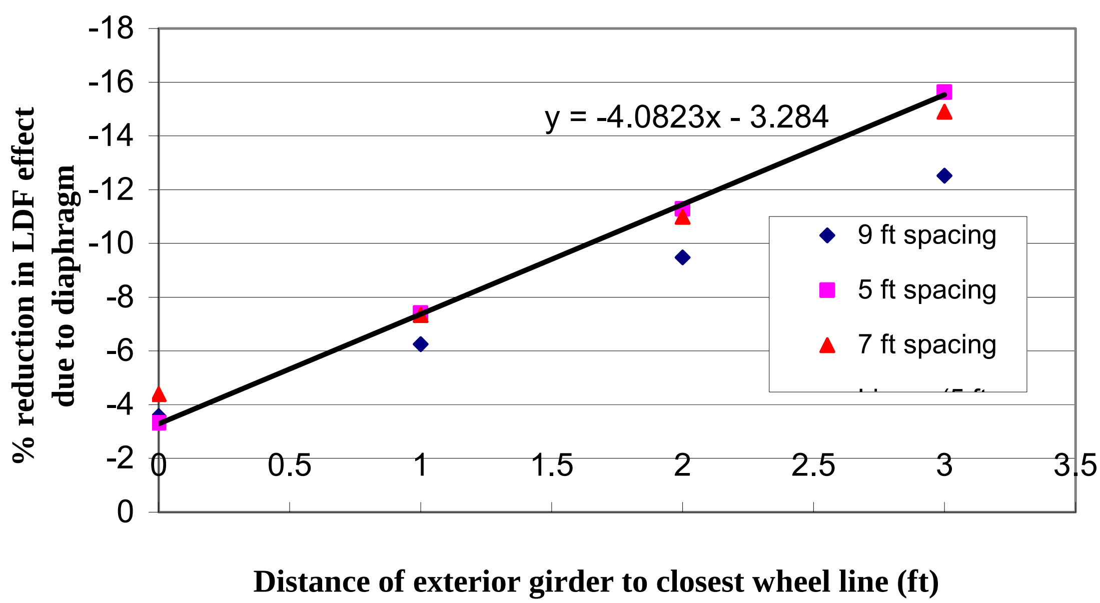

Fig 6.8. Relation between lateral position of loading system to R_d for exterior girder.

By including the correction factor of Eq.6.13 in determining R_d, the new value of R_d would take into account the position of loads with respect to the exterior girder and this factor could be applied for any spacing:

$$R_d = (0.1319 * L - 15.812 - C) * P_L \quad (6.14)$$

Where

L= Span length in ft

C = 0, 3 and 5 for Types II, III and IV girders respectively

R_d = % reduction in load distribution due to diaphragm

d = Distance between center of exterior girder to wheel line closest to edge (ft)

$P_L = (0.45+0.55^* d)$ = Correction factor for taking into account position of loading system laterally $(0 \leq d \leq 3\text{ft})$.

Table 6.19. Comparison between results obtained by analysis to the values obtained by applying correction factor P_L to the analysis results for the same bridge but with wheel line closer to edge is at 1ft away from exterior girder

1st wheel line distance from edge (d in ft)	Bridge configuration	ΔLDF (%) due to diaphragm	
		FEM	Applying reduction factors of Eq (6.13)
0	S9L65	-3.8	-2.8
3	S9L65	-11.6	-13.2
2	S9L50	-10.7	-12.3
3	S7L50	-16.9	-18.0

6.6.2. R_d for Skew Bridges

The methodology basically applied for deducing formulae for determining effect of diaphragm on exterior girders was similar to that used for interior girders. An additional factor which was considered was the influence of lateral position of loading on the effect of ID on exterior girder. It was checked whether the correction factor P_L in Eq. (6.13) determined for right bridges holds good for skew bridges as well. This was achieved by analyzing bridges for different lateral loading positions and determining the effect of diaphragms on exterior girders for skew bridges. Results of this study are listed in Table 6.20. The results indicate that until the spacing between the center of exterior girder to the first wheel line of the girder is less than 2/ft the correction factor gives quite accurate results. But when the same value reaches 3/ft there is significant difference between the results obtained from finite element analysis and that obtained by applying correction factor. In such case, the straining action and load distribution value at exterior girder would have reduced significantly and would not be the one which would be governing the design. Keeping this in view, decision was reached that the correction factor to take into account the effect of influence of ID based upon lateral location of load for right bridges is suitable for skew bridges as well.

Table 6.20. Comparison between R_d for skew bridges obtained from FEM analysis to the values obtained by applying correction factor P_L.

1st wheel line distance from edge (d in ft)	Skew angle (degrees)	Bridge configuration	ΔLDF (%) due to diaphragm	
			Analysis	Applying reduction factors of Eq. (6.13)
0	30	S9L70	-4.3	-3.6
2	30	S9L90	-6.9	-7.8
3	30	S9L50	-9.1	-12.2
0	50	S9L90	-3.6	-2.7
2	50	S9L65	-7.5	-7.3
3	50	S9L70	-10.1	-13.7

Skew correction factors to account for the influence of skew for exterior girders were determined in the same way as that for interior girders. The ratio between the R_d value for an exterior girder of skew bridges to the R_d value for an exterior girder of equivalent right bridge for a girder spacing of 5/ft were obtained and are presented in Table 6.21. From these results the skew correction factors for skew angles of 30^o and 50^o could be safely chosen as 0.7 and this value is adopted for intermediate skew values between 30° and 50°. At 0° angle the skew correction factor is equal to 1. A linear relation was adopted for values of skew reduction factor for intermediate skew angles between 0° and 30° (Fig.6.9).

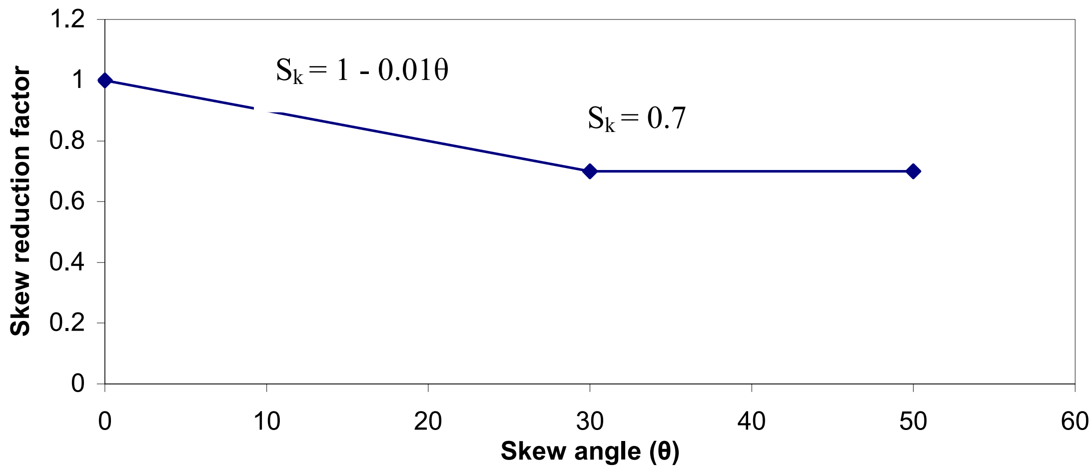

Fig 6.9. Skew angle VS skew correction factor, for LDF in exterior girders of bridges with single diaphragms

Table 6.21. Ratio of R_d for skewed bridges to the R_d for right bridge with same span length and 5/ft spacing

Span Length (ft)	Skew = 30		Skew=50	
	Spacing (ft)			
	5	9	5	9
50	0.54	0.64	0.72	0.68
65	0.60	0.64	0.84	0.70
70	0.60	0.63	0.69	0.64
90	0.51	0.54	0.64	0.57

By applying the correction factors for skew bridges

$R_d = (0.1319 * L - 15.812 - C) * P_L * S_K$ (6.15)

Where

L= Span length in ft

$C = 0, 3$ and 5 for Type II, III and Type IV girders respectively

R_d = % reduction in load distribution due to diaphragm

P_L = Correction factor for taking into account, position of lateral loading system

d = Distance between center of exterior girder to wheel line closest to edge (ft)

S_K = Skew reduction factor $= 1 - 0.01*\theta$ $\quad (\theta \leq 30^o)$

$= 0.7$ $\quad (\theta > 30^o)$

6.7. Formulae Development for R_d for Exterior Girders in Bridges with Two IDs

In deducing formulae for determining diaphragm effectiveness in bridges with two diaphragms, a similar approach as that used in developing R_d for bridges with single diaphragm was adopted.

6.7.1 R_d for Right Bridges

The results for the effect of diaphragm on LDF for exterior girders of bridges with two diaphragms are presented in Table 6.22. From the results it could be observed that the R_d values for bridges with two diaphragms are significantly smaller than the R_d values for bridges with a single diaphragm. The reason for this behavior is that the diaphragms are away from midspan where the LDF is calculated, in case of bridges with two IDs. Relation was drawn between span length and R_d for exterior girders in bridges with Type IV and Type BT girders having two diaphragms (Fig. 6.10). It could be observed that the two lines are nearly parallel and the difference between these lines is about 4.

From these observations the expression R_d for bridges with Type IV and Type BT girders with two diaphragms is given as

$$R_d = (-19.045 + 0.147 * L - C) \qquad \textbf{(6.16)}$$

Where

L = Length in ft.

C = 0 and 4 for girder Type IV and girder Type BT.

R_d = % reduction in load distribution due to diaphragm

Table 6.22 R_d for exterior girder in bridges with two diaphragms

Girder Type	Span Length (ft)	Spacing(ft)	
		5	9
IV	95	-3.80	-5.60
IV	110	-0.80	-2.80
IV	120	-1.08	-2.01
BT	105	-5.90	-8.20
BT	120	-3.11	-5.40
BT	130	-1.80	-4.00

It was later checked, whether the correction factor to account for the effect of lateral position of loading system on exterior girder, P_L (found in Eq. (6.13)) is suitable for right bridges with two diaphragms. The results (Table 6.23) indicate that the correction factor P_L for bridges with single diaphragm is suitable for bridges with two diaphragms as well.

In order to take into account the influence of lateral position of loading on how diaphragm affects the LDF, already determined R_d value in Eq. (6.16) was multiplied with correction factor $P_{L,}$ which would lead to a new expression for R_d:

$$R_d = (-19.045 + 0.147 * L - C)*P_L \qquad \textbf{(6.17)}$$

Where

L = Span length in ft.

C = 0 and 4 for girder Type IV and girder Type BT.

R_d = % reduction in load distribution due to diaphragm

P_L = Correction factor for taking into account position of lateral loading system

Table 6.23. Comparison between R_d values obtained from analysis to that obtained by applying correction factor of equation 6.13 to the R_d values obtained, when the wheel lane close to edge is 1ft away from exterior girder

1st wheel line distance from edge (d in ft)	Girder type	Bridge configuration	ΔLDF (%) due to diaphragm	
			Analysis	Applying reduction factors of Eq (6.17)
0	IV	S9L95	-2.0	-2.5
2	IV	S9L110	-4.7	-4.3
3	BT	S9L105	-17.2	-17.2
2	BT	S9L130	-7.3	-6.2

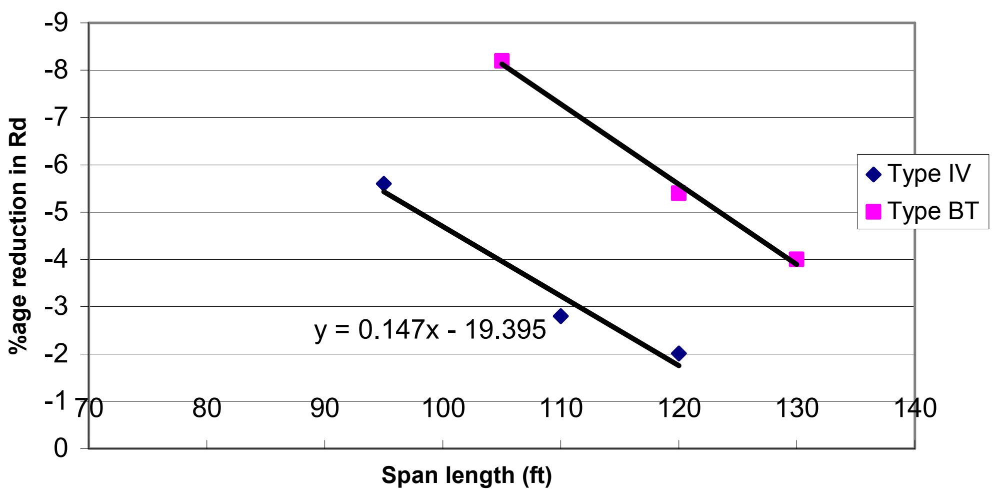

Fig 6.10 Influence of span length on ID effect in reducing LDF for bridges with two diaphragms.

6.7.2 R_d for Skew Bridges

For this case the same methodology was adopted as for skew bridges First it was checked whether the correction factor P_L of Eq. (6.13) is suitable for this case to account for the effect of lateral position of loading on exterior girder. The results (Table 6.24) indicate that the correction factor P_L (Eq. (6.13)) for taking into account the influence of lateral position of loading on exterior girder is suitable for skew bridges with two diaphragms as well.

Table 6.24. Comparison between R_d values obtained from analysis to that obtained by applying correction factor of equation 6.13 to the actual R_d values, where the wheel lane close to edge is 1ft away from exterior girder

1st wheel line distance from edge (d in ft)	Skew angle (°)	Girder type	Bridge configuration	ΔLDF (%) due to diaphragm	
				Analysis	Applying reduction factors of Eq (7.10)
0	30	IV	S9L95	-1.6	-1.5
2	50	IV	S9L110	-2.7	-3.0
3	50	BT	S9L105	-7.4	-10.3
2	30	BT	S9L130	-3.7	-2.6

Skew correction factors to account for the influence of skew for exterior girder in skew bridges with two diaphragms were determined in the same way as that for interior girders. The ratio between the R_d value for exterior girder of skew bridges to the R_d value for exterior girder of equivalent right bridge but for a girder spacing of 9ft were obtained and are presented in Table 6.25. From these results, the skew correction factors for skew angles of 30^o and 50^o was chosen as 0.6 and this value is adopted for intermediate skew values between 30^o and 50^o. In case of bridges with a span length of 130 ft the ratio is very much smaller than 0.6 but as the effect of diaphragm being significantly small for these bridges taking a skew correction factor of 0.6 does not cause much difference. At 0^o angle the skew correction factor is equal to 1. A linear relation between skew angle and skew reduction factor was adopted between 0^o and 30^o skew angle (Fig.6.11).

$$R_d = (-19.045 + 0.147 * L - C)*P_L * S_K \quad (6.18)$$

Where

L= Length of the girder in ft

C = 0 and 4 for girder Type IV and girder Type BT

R_d = % reduction in load distribution due to diaphragm

P_L = Correction factor for taking into account position of lateral loading system on load distribution

d = Distance between center of exterior girder to wheel line closest to edge in ft

$(0 \leq d \leq 3\text{ft})$

S_K = Skew reduction factor = $1-0.013*\theta$ $\quad (\theta \leq 30)$

$=0.6 \quad (\theta > 30)$

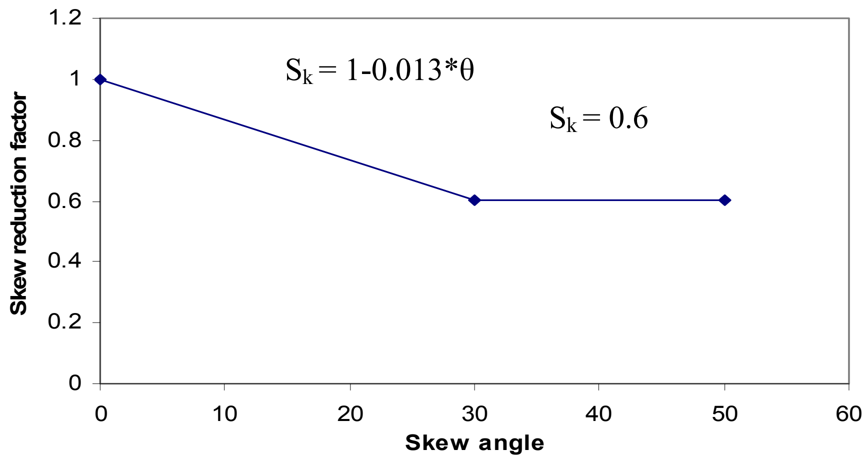

Fig 6.11. Skew angle VS skew correction factor, for LDF in exterior girders of bridges with two IDs

Table 6.25. Ratio of R_d value for skewed bridge to the R_d value for right bridge of the same span length and 5ft spacing.

Span Length (ft)	Skew = 30°		Skew=50°	
	Spacing (ft)			
	5	9	5	9
95	0.57	0.59	0.52	0.59
110	0.50	0.68	0.57	0.50
105	0.57	0.60	0.46	0.44
130	0.30	0.43	0.33	0.23

6.8. Summary of Formula Development

The final set of formulas can be put together in four equations, and these being single and two diaphragms for both interior and exterior girder. These expressions for R_d and the variables associated with R_d are listed in Table 6.26 to Table 6.28.

Table 6.26. Expressions for R_d value for different cases

No. of diaphragms	Interior(In) or exterior (Ex)	Equation for R_d
1	In	$[(0.132*L + 4.85) + C] *S_t *S_k$
2	In	$(-0.112*L +25.81) * C*S_k* S_t$
1	Ex	$(0.132* L -15.81 –C)* P_L* S_K$
2	Ex	$(-19.05 +0.147 * L - C)*P_L * S_K$

Table 6.27. Values of S_K, S_t and P_L for different bridge configurations

No. of dia.(D)	Interior girder		Exterior girder	
	S_K	S_t	S_K	P_L
1	$1-0.015*\theta$ $(\theta \leq 30^o)$ $0.775 - 0.0075 * \theta$ $(\theta > 30^o)$	$0.0264*X^{0.8062}$	$1-0.01* \theta$ $(\theta \leq 30^o)$ 0.7 $(\theta > 30^o)$	$0.45+0.55* d$ $(0 \leq d \leq 3ft)$
2	$1-0.0167*\theta$ $(\theta \leq 30^o)$ $0.725 – 0.0075 * \theta$ $(\theta > 30^o)$	$0.0873*X^{0.5358}$ (Type IV) $0.3024*X^{0.2641}$ (Type BT)	$1-0.013* \theta$ $(\theta \leq 30^o)$ 0.6 $(\theta > 30^o)$	$0.45+0.55* d$ $(0 \leq d \leq 3ft)$

Table 6.28. Values of C in expression for R_d

Girder Type	Interior		Exterior	
	No. of diaphragms		No. of diaphragms	
	1	2	1	2
II	0	-----	0	----
III	2	----- -----	3	----
IV	3.5	1	5	0
BT	-----	1.98	-----	4

In the Tables 6.26 to 6.28

L= Length of the girder in ft

C = constant

R_d = % reduction in load distribution due to diaphragm

P_L = Correction factor for taking into account position of lateral loading system

d = Distance between center of exterior girder to wheel line closest to edge in ft

$(0 \leq d \leq 3\text{ft})$

S_K = Skew reduction factor

S_t = Stiffness reduction factor

Θ = angle of skew

X = (Possible diaphragm stiffness contributing to load distribution / absolute diaphragm stiffness)*100

Finally LDF for the bridge, which takes into account the influence of diaphragm in load distribution, could be given by the following expression:

$$(LDF)_{WD} = (1 - R_d / 100)*(LDF)_{ND} \qquad \textbf{(6.7)}$$

Where

$(LDF)_{WD}$ = Load distribution factor for bridge by including diaphragm effectiveness in load distribution

$(LDF)_{ND}$ = Load distribution factor for bridge without considering diaphragm effectiveness in load distribution

6.9. Accuracy of the Formulas Developed

The accuracy of the formulae developed is determined by comparing the R_d values obtained by using formulae deduced earlier to the R_d values obtained from analysis for few bridge configurations. In Table 6.29 comparison is done between the R_d value obtained from formulae to the R_d value obtained from FEM analysis, for interior girder. The results indicate that the R_d value obtained from formulae is close to that obtained by FEM. For right bridges the difference between these results is very small but in case of large skew angle significant difference exists and this is possibly due to adopting smaller skew reduction factor values. But for all the cases except for one case considered in Table 6.29, values of R_d from formulae were smaller than R_d from FEM, thereby proving that the expression for R_d for interior girder would yield safe results. Similarly for exterior girder the comparison is done between the values of R_d obtained from the formulae and that obtained through FEM and this is listed in Table 6.30. The results indicate that the formulae developed are accurate as the difference in these values was less than 1% for most of the cases.

Table 6.29. Comparison between results obtained from formulae deduced for R_d to the R_d value obtained from FEM for interior girder

Bridge configuration	Girder type	Skew (Degrees)	% stiffness	R_d from formulae	R_d from FEM
S9L130	BT	30	30	8.26	10
S9L105	BT	50	30	7.15	10.1
S9L95	IV	30	40	4.77	8.6
S9L100	IV	40	60	4.86	3.57
S9L115	BT	20	50	14.3	15.1
S5L80	III	0	45	9.88	11.2
S5L70	III	50	50	4.15	6.8
S9L65	II	50	30	2.2	2.52
S9L80	IV	15	65	11.2	10.6

Table 6.30. Comparison between results obtained from formulae deduced for R_d to the R_d value obtained from FEM for exterior girder

Bridge configuration	Girder type	Skew (Degrees)	d (ft)	R_d from formulae	R_d from FEM
S5L70	III	30	1	-6.7	-6.1
S9L95	IV	0	1	-5.08	-5.6
S9L130	BT	50	1	-2.43	-0.7
S9L65	II	30	1	-5.06	-4.7
S5L75	III	0	0	-4.01	-4.69
S9L110	IV	30	0	-0.77	-0.48
S9L120	BT	50	0	-1.46	-0.5
S9L65	II	20	2	-8.97	-6.49
S9L110	IV	0	2	-4.45	-4.95
S5L105	BT	30	2	-7.07	-7.8
S9L90	III	50	2	-7.5	-6.9

6.10. Examples Illustrating Determination of R_d for Some Bridges

In this section examples are illustrated for determining the LDF by accounting the influence of diaphragm on LDF through the expressions for R_d listed in Tables 6.26 to 6.28. Four bridge configurations were chosen for this illustration and these are listed in Table 6.31

Table 6.31. Bridge configurations for which calculation of R_d value is illustrated

Case	Bridge configuration	Girder Type	Interior(In) or Exterior (Ex)	Skew (Degrees)	% stiffness	d (ft)
1	S9L130	BT	In	30	30	----
2	S5L80	III	In	0	45	----
3	S9L65	II	Ex	20	-----	2
4	S9L110	IV	Ex	50	-----	0

Case 1

Bridge being of 130ft length, would be having two diaphragms. As this case is for interior girder the expression for $R_d = (-0.112*L +25.81) * C*S_k* S_t$

(From Table 6.28)

Girder type being BT, C = 1.98 (From Table 6.30)

For interior girder with two diaphragms

$S_t = 0.3024*X^{0.2641}=0.3024*(30)^{0.2641}=0.742$ (From Table 6.29)

$S_k = 1-0.0167*\theta$ as $(\theta \leq 30^o)$ = 1- 0.0167*30 = 0.5

Therefore,

$R_d = (-0.112*L +25.81) * C*S_k* S_t = (-0.112*130 +25.81) * 1.98*0.5*0.742 = 8.26$

Case 2

Bridge being of 80ft length, would be having a single diaphragm. As this case deals with interior girder the expression for $R_d = [(0.132*L + 4.85) + C] *S_t *S_k$

(From Table 6.28)

Girder being of Type III, C = 2 (From Table 6.30)

For interior girder with two diaphragms

$S_t=0.0264*X^{0.8062} = 0.0264*(45)^{0.8062} = 0.568$ (From Table 6.29)

$S_k = 1-0.015*\theta = 1- 0.015*0 = 1$

Therefore,

$R_d = [(0.132*L + 4.85) + C] *S_t *S_k =[(0.132*80 +4.85) +2]*0.568 = 9.88$

Case 3

Bridge being of 65ft length, would be having single diaphragms. As this case deals with exterior girder the expression for $R_D = (0.1319* L -15.81 – C)* P_L* S_K$

(From Table 6.28)

Girder being Type II, C = 0 (From Table 6.30)

For interior girder with single diaphragms

$P_L=0.45+0.55* X = 0.45 +0.55*2 =1.55$ (From Table 6.29)

$S_k = 1-0.01*20 =0.8$ as ($\theta \leq 30^o$)

Therefore,

$R_d = (0.1319* L -15.81 – C)* P_L* S_K = (0.1319*65 -15.81 – 0)*1.55*0.8 = -8.97$

Case 4

Bridge being of 110ft length, would be having single diaphragms. As this case deals with exterior girder the expression for R_D=(-19.05 +0.147 * L - C)*P_L * S_K

(From Table 6.28)

Girder being Type IV, C = 0 (From Table 6.30)

For interior girder with single diaphragms

P_L=0.45+0.55* X = 0.45 +0.55*0 =0.45 (From Table 6.29)

S_k = 0.6 as ($\theta > 30^o$)

Therefore,

R_d = (-19.05 +0.147 * L - C)*P_L * S_K = (-19.05 +0.147*110 – 0)*0.45*0.6 = - 0.77

7. STEEL INTERMEDIATE DIAPHRAGMS AND LATERAL LOADING

7.1. Introduction

As mentioned earlier, one of the important tasks of the project is to identify steel diaphragm configurations, which could perform equivalently as RC IDs does in PC girder bridges, as it would be more economical to provide steel ID to an RC ID. This section discusses about: (1)The possible configurations of steel ID which could replace RC ID. (2) Assesses the stability these diaphragm provide when compared to RC IDs during the construction of deck. (3) How much these steel sections contribute to load distribution by doing a parametric study for bridge configurations listed in Table 4.2 and. (4) The performance of various IDs under impact is compared.

7.2. Selection of Appropriate Steel Diaphragm Section

Diaphragm configurations were chosen based on the geometry of the girder section. For girder Types II, III and IV, as the depth of the web region of the girder is small a channel section seemed to be most appropriate as it could be connected to the girder easily and also connect most of the portion of the girder web region. While for BT girder possibility of providing channel and X type bracing with a bottom strut was explored.

Three different bridge configurations of each of three different girders, and those being Type II, IV and BT girder, were analyzed by modeling diaphragm as beam element with end moments fixed and another as axial elements. The percentage reduction in LDF for interior girders due to IDs by these two different forms of diaphragm modeling is presented in Table 7.1. The results indicate that ID influence in reducing LDF for interior girder is predominantly due to the transfer of axial forces through diaphragms. Therefore different sections having nearly same axial stiffness would generate nearly equal ID effectiveness, and this criterion was used for choosing steel ID sections. As there is no large difference in the depth of the web for bridges with Type II, III and IV girders, a channel section was chosen which could be used for all bridges with these three girders types. Girder Type II has minimum web depth of all the three girder types therefore it was decided that channel section with a depth no greater than 15" would be used. Providing a steel channel which absolutely has the same axial stiffness as the RC diaphragm is not possible as the section would be very heavy and no single channel section can provide the desired stiffness. It has been discussed earlier, that only a portion of RC diaphragm section is effective in load distribution because of the possible cracking at diaphragm girder interface. Keeping this in view, a minimum target ID stiffness of 40% of RC ID was set up, based on which an appropriate steel channel section was chosen. It was thought that rather than providing three different diaphragm sections for the three girder types it would be better to choose a common section for the purpose of uniformity. For the three girders being discussed here, ID for bridge with type IV girder has the maximum stiffness among all. Therefore a channel section was set to be chosen as diaphragm for these three girder types, which could provide equivalent stiffness greater than 40% of axial stiffness contributed by RC ID for type IV girder. This was done to make sure that for all the girder types the diaphragm stiffness is greater than the target stiffness.

Finally a channel section C 15 X 33.9 was chosen as the diaphragm as it satisfies all the limiting conditions defined earlier. The depth of the channel is 15in and the axial stiffness of this section is 46.7% of the absolute stiffness of RC diaphragm for Type IV

Table7.1. Comparison between R_d obtained by modeling diaphragm as an axial truss element and beam element

Bridge configuration	R_d	
	Axial element	Beam element
S9L65	11.7	13.5
S9L90	15.2	18
S9L110	12.5	13.7

girder. In calculating the equivalent axial stiffness for the steel section, the area of steel section is multiplied by a modular ratio (m = E_{steel} / $E_{concrete}$ = 8.6). For ID with girder Types II and III the ratio of axial stiffness of C15 X 33.9 section to RC diaphragm axial stiffness is about 71% and 56.5% respectively. For the purpose of maintaining uniformity in the diaphragm sections provided, the same section was adopted for Type II and Type III girders, though a smaller section would have been sufficient to provide the target stiffness of 40% of ID stiffness for the respective girders.

For BT section the depth of the web is 54 in, making the concrete section area of 432 in^2. This would mean that for providing a stiffness equivalent to about 40% of axial stiffness of RC diaphragm a steel section of 20 in^2 would be required, which no single steel section can provide. Also as the depth of the section that could be provided being small (maximum of 18/in) when compared to the depth of the 54/in. web, the lateral stability provided by this section might not be adequate. Because of these reasons it was not possible to choose a steel channel section for ID in bridges with B-T girder, which necessitated a search for an alternative steel diaphragm configuration. Providing X type bracing with a bottom strut for ID in BT bridges, seemed to be a possible alternative.

Initial study was done by choosing MC8X20 channel section for all its bracing members. In Table 7.2, comparison was done between % reduction in LDF due to RC diaphragms considering absolute stiffness of diaphragm contributing to load distribution and X plus bottom strut for two bridge configurations S9L105 and S9L130 with bulb T girders. From the results it could be observed that reduction in LDF due to X plus bottom strut is about 0.8 times that provided by RC diaphragms, which is a very significant contribution. From the relation obtained between the stiffness of RC ID and the R_d due to ID for interior girder in section 6.4.1, it was found that for about 40% of absolute stiffness contribution of RC diaphragm yielded an R_d value of about 80% of that of R_d value obtained for absolute ID stiffness. This implies that the assumed steel ID configuration is providing an axial stiffness of about 40% of actual diaphragm stiffness of RC diaphragm, which was our target stiffness. Hence X plus bottom strut with all its members of MC 8X20 section was found to be appropriate, in terms of contribution of diaphragm in load distribution.

7.3. Stability Provided by Steel Diaphragm During Construction of Deck

One of the reasons for providing a diaphragm is to provide stability to girders during the process of construction of deck. During this process, the concrete in the deck

Table 7.2. Comparison of R_d values for X plus bottom strut and RC diaphragm

Bridge configuration	% reduction in LDF for RC dia (1)	% reduction in LDF for X + bottom strut (2)	Ratio of (2)/(1)
S9L105	27.4	22.1	0.8
S9L130	22.6	19.3	0.85

as being wet cannot transmit lateral load that are induced during the construction process and other sources of lateral loading. The diaphragms are provided to transfer these loads at one girder to another and providing lateral restraint. As the lateral stiffness of prestressed concrete girder being small, the stiffness of girder falls sharply after initiation of cracks, which makes this problem of stability of girder a nonlinear time dependent problem. This being beyond the scope of the current work, the study is limited to comparing the stability provided by steel diaphragm relative to that provided by RC diaphragm rather than determining the absolute stability provided by each of these diaphragms. This is achieved by comparing the principal tensile stresses developed in the girder web region for the bridges with different ID configurations. For carrying out this study analysis was done by analyzing a 3-D solid FEM model built in ANSYS.

7.3.1. Calculation of Construction Loads

It was assumed that steel sheets provided as formwork during the construction of deck to be of negligible stiffness. Initial studies indicated that the load carried by the bracing is maximum, when the formwork was loaded up to the center of the innermost girder along the length of the span. Hence the loading was done in this manner and was applied as pressure on the surface of the surface of the formwork (Fig 7.1). Three components of load were applied on the formwork, these being dead load of the wet concrete, dead load of formwork and construction loads due to equipments. The load values for construction loads and formwork were adopted from the values used in design of formwork for deck for some bridges designed earlier.

Dead load due to wet concrete = ρ* thickness = 150 *9/12 = 112.5 psf (assuming the thickness of deck to be 9in)

Assuming dead load of formwork = 4 psf

Construction loads = 50 psf

Sum of all the components of construction load = 166.5 psf ≈ 170 psf

Therefore the loading was done as shown in Fig. 7.1 on formwork with a load of 170 psf to determine the stability provided by the diaphragm and also to determine the forces generated in the bracing.

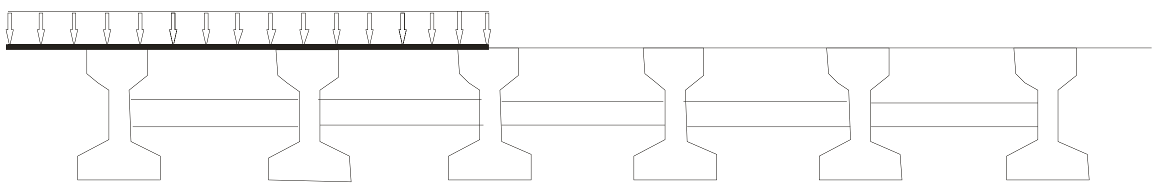

Fig. 7.1. Construction loading, generating maximum forces in bracing

Comparison of stability provided by RC ID to that provided by steel ID was done for S9L130 bridge with BT girder. As mentioned in Section 7.2, for BT girders providing X plus bottom strut with all its members of MC 8X20 section seemed to be appropriate, hence this section was adopted as the steel ID for this bridge in this study. The study of relative stability provided was done by comparing the largest principal stress at the inner face of the web at the location of diaphragm and midspan. The results show that for both RC and steel diaphragm the principal stresses obtained from FEM analysis are nearly same where there is no diaphragm, thereby indicating that both diaphragms provide nearly same order of stability to the girder.

Similar study was carried out for bridge configuration S9L90 with Type III girder. For bridges of this type using a channel section of C15X33.9 was found to be appropriate in Section 7.2. Therefore comparison between the stability provided by this diaphragm to

RC ID was done. In this case the principal stresses in the bottom portion of web on the inner face, are compared at the location of diaphragm (which is at midspan) and quarter span. The diaphragm is modeled using shell elements (SHELL 41 in ANSYS). The results of this study are presented in Table 7.4. And similar observations as in the case of BT girders were observed in this case.

From the results obtained in this study it was concluded that the lateral stability provided by the steel ID is equivalent to that provided by RC ID for this bridge as well.

Table 7.3. Comparison of principal stresses(ksi) due to construction load in S9L130 bridge with different diaphragm configuration

Girder no.	Steel ID		RC ID		No diaphragm	
	ID section	Midspan	ID section	midspan	ID section	midspan
2	0.933	1.7	0.948	1.67	1.31	2.2
3	0.668	1.4	0.7	1.37	0.98	1.7

Table 7.4. Principal stresses(ksi) due to construction load in S9L90 bridge with different diaphragm configuration

Girder no.	Steel ID		RC ID		no diaphragm	
	ID section	Midspan	ID section	midspan	ID section	midspan
2	0.4015	0.2973	0.407	0.3045	0.6126	0.49085
3	0.2975	0.2452	0.2871	0.2339	0.2831	0.2375

7.4. Loads Carried by Bracing

The load carried by bracing under various loading conditions is determined and checked whether the steel diaphragm members are capable of carrying these loads. The maximum loads in members were determined for each of the three different loading conditions and these being under uniform construction loading, concentrated load on girder during the process of construction and due to live load. The study was carried out for two bridge configurations, S9L90 with a channel diaphragm of section C15X33.9 and bridge S9L130 (with BT girders) with an X plus bottom strut diaphragm.

As mentioned in Section 7.3 bracing carries the lateral load during the construction of deck. Therefore it was thought to be essential to check whether the load carried by the diaphragm is within its load carrying capacity. The forces in the bracing is determined for a uniformly distributed construction load of 170 psf, applied from edge of the bridge to center of innermost girder as mentioned in section 7.3.1.

The extent of maximum concentrated load coming on the formwork during this period being unknown, a load of 50kips is applied as concentrated load on the edge of the interior girder. For this loading the stresses and forces in the diaphragm members are determined by analyzing 3-D solid models.

The forces in the bracing generated under live load were also determined. These were obtained by analyzing models in GT STRUDL for the same live load configuration used in determining LDF.

The values of forces and the stresses in the ID members for bridges S9L90 and S9L130 for the above mentioned loading are listed in Tables 7.5 and 7.6.

The member capacity of MC8X20 is in order of 220 kips in tension and 110 kips in compression, which is significantly larger than all the maximum forces obtained under the three loads added together in Table 7.5. Similarly C15X33.8 has tensile and compressive load carrying capacity of about 380 kips and 200 kips, which is again larger than all the forces in the braces added together under the three loads in Table 7.6. The slenderness ratio of these bracing members provided is about 120 which is less than the maximum slenderness ratio of 140, which is the limiting value used in design of bracings.

From these results it concluded that the bracing members considered can carry the forces induced in the bracing.

Table 7.5. Maximum forces and stresses in bracing members for S9L130 bridge under different loading conditions

Loading condition	Force (Kips)	Stress (ksi)
Uniform const. load	33.7 (T) 34.1 (C)	5.73 (T) 5.79 (C)
Concentrated const. load (at midspan)	23.93 (T) 16.26 (C)	4.07 (T) 2.77 (C)
Concentrated const. load (at diaphragm section)	38.6 (T) 24.5 (C)	6.57 (T) 4.16 (C)
Live load (at midspan)	29.4 (T) 3.6 (C)	5.03 (T) 0.61 (C)
Live load (at diaphragm section)	30.9 (T) 2.4 (C)	5.26 (T) 0.41 (C)

Table 7.6. Maximum forces and stresses in bracing members for S9L90 bridge under different loading conditions

Loading condition	Force (kips)	Stress (ksi)
Uniform const. load	24 (T) 5.1 (C)	2.43 (T) 0.52 (C)
Concentrated const. load	15.81 (T) 6.83 (C)	1.6 (T) 0.7 (C)
Live load	63.2 (T) 9.3 (C)	6.4 (T) 0.95 (C)

7.5. Assessing the Influence of ID in Limiting Damage Due to Impact of Over Height Trucks

There are several instances, where prestressed concrete girder bridges have underwent collision with over height trucks passing under them. There exists a controversy on effectiveness of diaphragms in limiting damage during collision and there are conflicting reports on this issue. To get a better understanding on this issue and to know how different diaphragms effect the performance of bridge during collision, an analytical study was carried out with the 3-D solid model built in ANSYS. Simulating the actual collision is a difficult task and beyond the scope of this study. This study was limited to comparison of relative performance of bridges with different diaphragm configuration under lateral impact loading, which was applied as concentrated static load.

In order to check whether the proposed finite element model could be used for studying bridge performance under lateral loading, the results obtained by analyzing the model were compared to the experimental results obtained under the lateral loading of bridge. As no experimental tests were carried out in this study, the comparison was done for results obtained from the experimental work carried out by Abendroth et al. (1995) on bridges under lateral loading. The details of the experiment and its results were obtained from the thesis work of Andrews (2003). A comparison was also made between the results of the current model under study and the finite element model by Andrews.

7.5.1. Details of Experimental Bridge (Abendroth et al. 1995)

This experimental bridge is a slab over girder bridge with 3 Iowa “A38” prestressed concrete girders spaced 6/ft apart and a 3/ft overhang measured from the center of exterior girder. Span of the bridge is of 40ft-4in and is an un-skewed bridge.

The thickness of the deck was 4in and was limited to this amount intentionally so as to make the structure flexible. At each end of the bridge, a 42/in deep by 18/in wide reinforced concrete abutment supported the PC girders and each girder was placed on elastomeric bearing pads. The end diaphragms were of 8-in thickness and different ID configurations were used. Strain gauges and direct current displacement transducers were placed near the mid-span location of girders.

As the focus of the current study is on intermediate diaphragms their configuration is explained in greater length. The tests were done for three ID configurations, and these being RC diaphragm, X type bracing configuration with a bottom strut of steel diaphragm and a steel channel section. They also studied the case where diaphragms were absent. RC diaphragm was of 6/in thickness. All the steel members were of MC8 X 20 section and were held between the girders by fixing these steel members to the gusset plates which in turn are attached to the girder through anchor bolts.

7.5.1.1. Comparison of Experimental Results to FEM for Different Diaphragm Configuration

A comparison is made between the strains and displacements obtained from experiments, current finite element model, and the FEM by Andrews for different diaphragm configurations. These results are tabulated in the columns under the title E, C and A respectively (Tables 7.7 to 7.9). This comparison was done for a lateral load value of 75 kips. The location where results were compared were referred by the girder number followed by suffix ‘R’ or ‘L’ indicating right or left side of girder. The last two columns indicate the levels of difference between results obtained from current model and Andrews’ model to that of the experimental results.

7.5.1.2. Discussion of Results

From the results, it was observed that the performance of the current finite element model was similar to that of experimental bridge and also Andrews Model. In case of deflections the variation is in the order of -25% to 5%, which is within the permissible limits. While for strain, the variation was large percentage wise as the numbers being small the actual results were close. These results have given confidence to carry out studies on lateral loading using the current 3-D solid finite element model.

Table 7.7. Strains (μ) for bridge with RC diaphragm configuration with loading on girder 1

Location	E	C	A	(E-C)/E*100	(E-A)/E*100
1R	-8.9	26.5	-22	397.8	-147.2
1L	110.7	121	91	-9.3	17.8
2R	-59	-56	-49	5.1	16.9
2L	12.1	33.1	18	-173.6	-48.8
3R	-38.9	-91	-48	-133.9	-23.4
3L	7.8	-3.3	-9	142.3	215.4

Table 7.8. Strains (μ) for bridge with X plus Strut diaphragm configuration with loading on girder 1

Location	E	C	A	(E-C)/E*100	(E-A)/E*100
1R	-67.5	-30	-35	55.6	48.1
1L	148.7	175	127	-17.7	14.6
2R	-57.6	-67.5	-58	-17.2	-0.7
2L	32.5	57.5	41	-76.9	-26.2
3R	-43.7	-90	-69	-105.9	-57.9
3L	17.4	12	4	31.0	77.0

7.6. Diaphragm Influence on Bridge Performance under Impact Loading

Study of impact on bridge behavior with different ID configurations was done for two bridges which were adopted in earlier parametric studies. The two bridges chosen were S9L90 and S9L130, for which study was done with steel, RC ID and the case where there is no ID. Steel ID configurations used for S9L90 and S9L130 were channel section and X plus bottom strut respectively, in accordance with the current proposed steel ID configurations. For X plus bottom strut diaphragm members, the elements were modeled as 3-D LINK-8 elements (line element), while channel section diaphragm was modeled as SHELL 28 elements (two dimensional shell elements) in ANSYS (Figs. 7.2 and 7.3).

Table 7.9. Deflection (in) for X plus Strut diaphragm configuration with loading on Girder 1

Diaphragm type	Load position	EXP(E)	C	A	(E-C)/E*100	(E-C)/E*100
RC Diaphragm	1	0.075	0.089	0.08	-18.7	-6.7
	2	0.075	0.079	0.065	-5.3	13.3
No diaphragm	1	0.025	0.2128	0.21	5.4	6.7
	2	0.2	0.238	0.16	-19	20
X plus Strut	1	0.095	0.107	0.08	-12.6	15.8
	2	0.075	0.095	0.065	-26.7	13.3

The magnitude of impact is a function of several parameters such as mass, speed, geometrical configuration and hardness (Abendroth et al., 2003), and there is no available literature which gives information on issues related to impact. A numerical value of impact load was assumed, which was applied as concentrated static load. This value was taken as 120 kips, the same value which was used by Abendroth et al (2003). This study was done for impact at the bottom flange of the girder.

7.6.1. Different Cases Considered in Study of Lateral Impact

For both bridges, impact load was applied at two locations, one at the location of ID and another midway between two diaphragms. For S9L130, where there are two diaphragms, impact load was applied at midway (which is the midspan) between the two IDs and at one of the ID. While for S9L90 the loading was applied midway between the ID and end diaphragm and at the location of ID.

The comparison was done between the maximum value of first principal stresses in the girder undergoing impact and two girders next to it for bridges, in the regions of interest. For each impact case the results were extracted at the location of impact and also at the other location were the impact load is intended to be applied. That is for S9L90 the results were extracted at midspan and quarter span, while for S9L130 it was at midspan and at location of ID. The results from the small regions of stress concentrations along the bridge were eliminated as taking these results into account might lead to erroneous conclusions. Therefore appropriate caution was taken to filter out these results with the aid of contour plots for the first principal stresses in ANSYS. These regions existed at location of loading, at connection between different elements and at location of supports. This could be observed in Fig 7.4., where a small region of stress concentration exists which is in red color, at the location of impact on girder undergoing collision for S9L130 bridge.

7.6.2. Results for Impact Studies

The values of principal stresses for S9L90 and S9L130 at regions of interest for different impact loading conditions are presented in Table 7.10 and 7.11. For bridge configuration S9L90, when the impact takes place at diaphragm location, RC ID gives the highest protection to girder. The principal stresses developed at ID location is about one-third of the principal stress developed in the girder for the case where diaphragm were absent in the region of impact. While in case of steel ID the principal stresses reduces by 40% with respect to the principal stress generated in case where the ID is absent. When the impact occurs at quarter span which is 22.5/ft away from both the end and intermediate diaphragm there was no difference in principal stress developed for bridges with different ID types and the bridge without ID.

Similar results were observed in bridge configuration S9L130. In this case the midspan is midway between the two IDs at a distance of 21.6 ft from each. When the impact occurs at midspan, there was no significant difference in principal stresses for the bridges with both the ID types and the one without ID. When the impact occurs at ID the principal stresses developed were 0.5, 0.9 and 3.5 for bridges with RC, steel and where ID is absent.

From the results obtained in these two bridges, it could be observed that when the impact occurs at the location where IDs are provided, different IDs reduce the stresses generated in the girder undergoing impact to different extent with respect to the case where ID is absent. As the magnitude of impact load is unknown, it could not be concluded whether the diaphragm would be in a position to transfer the impact load successfully to other girders as the structural performance is nonlinear under impact and needs a more detailed studies to reach a conclusion on how diaphragm effects the performance of bridge, when the impact takes place at the location of diaphragm. But when the impact takes place at a location significantly away from ID , the ID and its type has no effect on the behavior of the bridge under impact, thereby questioning its ability to protect the girders due to impact. If the IDs are provided for the purpose of protecting the girders under impact, they must be provided at each underneath lane of the road under the bridge. Therefore the current locations where ID is located is based upon the purpose of providing stability and this is not sufficient for protecting the girder under impact.

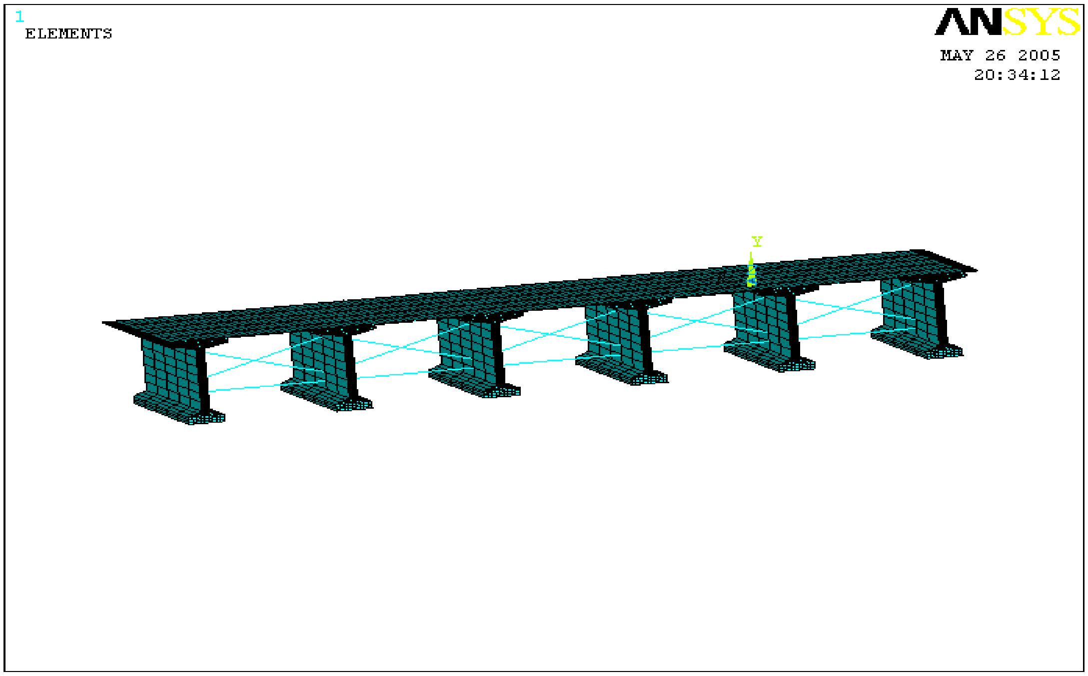

Fig 7.2. Section showing X plus bottom strut in ANSYS

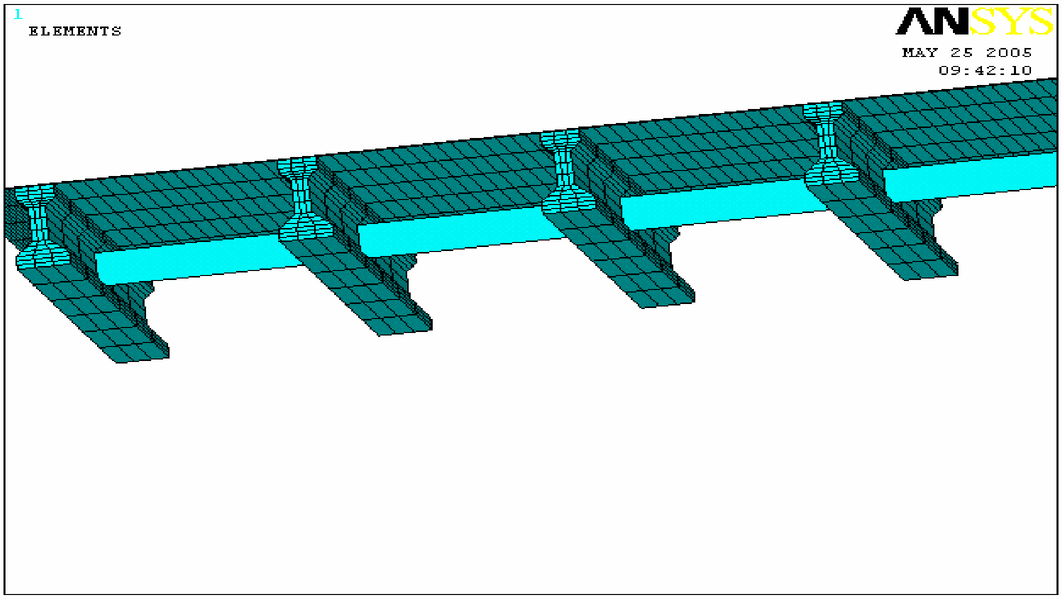

Fig. 7.3. Section showing channel ID in ANSYS

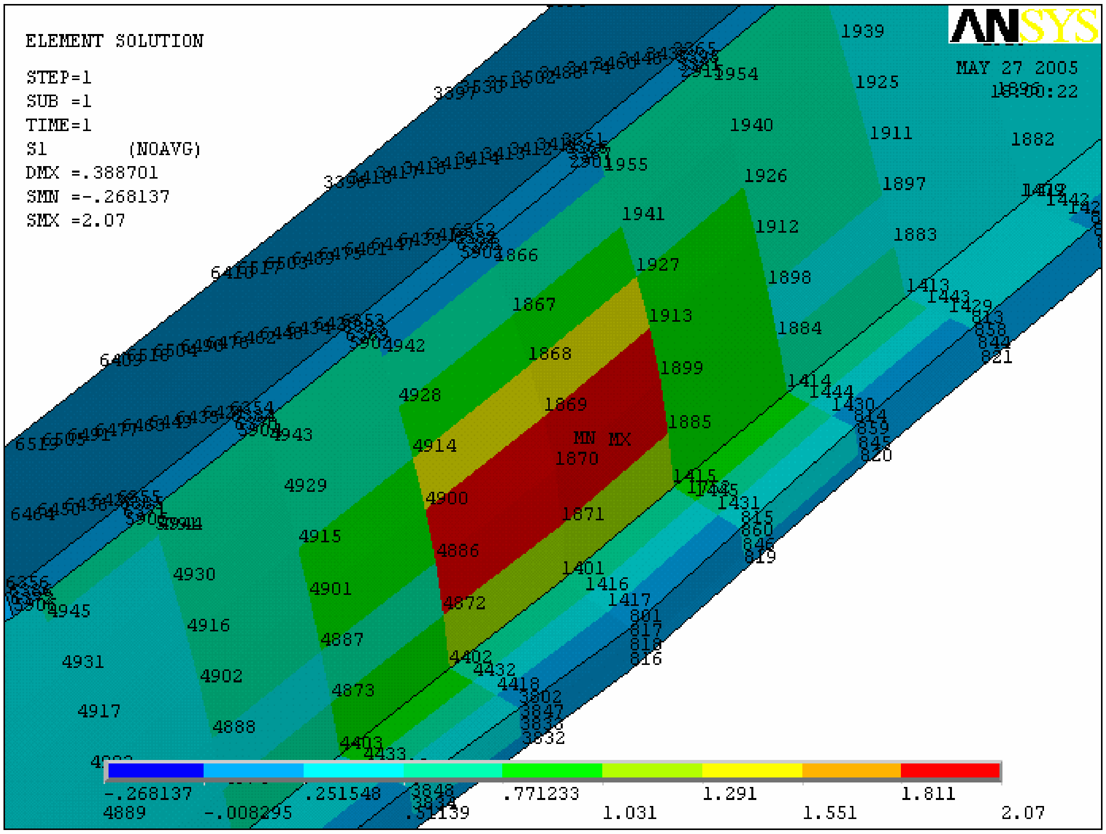

Fig.7.4. Contour for principal stress on outer face of exterior girder which underwent impact for S9L130 bridge

Table 7.10. Principal stresses (in Ksi) in region of interest for S9L90 bridge with different diaphragm configuration

Location of impact load	Results region	Girder no.	Diaphragm section		
			RC	channel	no ID
Diaphragm location	Diaphragm location	1 2 3	0.45 0.11 0.07	0.8 0.45 0.2	1.4 0.05 0.005
	quartar span	1 2 3	0.15 0.035 0.015	0.3 0.07 0.01	0.33 0.06 0.01
quartar span	Diaphragm location	1 2 3	0.3 0.09 0.04	0.23 0.06 0.03	0.25 0.03 0.008
	quartar span	1 2 3	1.2 0.07 0.03	1.3 0.08 0.025	1.3 0.07 0.025

Table 7.11. Principal stresses (in ksi) in region of interest for S9L130 bridge with different diaphragm configuration

Locction op impact	Results region	Girder no.	Diaphragm section		
			RC	X + strut	no ID
midspan	midspn	1	3.3	3.33	3.5
		2	0.125	0.128	0.07
		3	0.017	0.059	0.015
	Diaphragm location	1	1	1.11	0.95
		2	0.26	0.18	0.065
		3	0.1	0.11	0.013
Diaphragm location	midspn	1	0.12	0.2161	0.78
		2	0.058	0.057	0.05
		3	0.01	0.023	0.01
	Diaphragm location	1	0.5	0.9	3.5
		2	0.05	0.215	0.08
		3	0.05	0.19	0.013

7.7. Parametric Study for Bridges

After selecting the diaphragms and being sure that those diaphragms chosen were proven to be adequate, a parametric study was carried out for bridges listed in Table 4.2 with corresponding steel diaphragms for those bridges. Only those parameters have been considered which could affect the LDF. The results of this study have been listed in Table 7.12. In this table the values of strain, deflection and LDF values have been presented for interior and exterior girders of bridges, both with and without IDs. Percentage reduction in these values caused due to ID has been presented and the value of R_d calculated from the formulas developed for RC diaphragms is also included. Axial stiffness of steel ID to the axial stiffness of RC ID was used in determining the stiffness reduction factor, which was used in calculating R_d values. Axial stiffness ratio was considered as it was already known from Section 7.2, that the reduction in LDF due to ID is significantly dependent on the axial stiffness of ID. But for X plus strut steel ID, the axial stiffness being unknown, the stiffness ratio was taken as 40% (discussed in Section 7.2), as this value of stiffness ratio gave nearly the same value of R_d obtained from the formulae and the results obtained from FEM analysis. The stiffness ratio for the steel diaphragms used in calculating R_d values, corresponding to each girder type has been presented in Table 7.13. Though the expression for $R_{d,}$ for exterior girders does not include the influence of ID stiffness, the expression would yield an R_d value which is on the upper side as the stiffness ratio is always less than 1.

By comparing the R_d values obtained from FEM to those obtained from formulae, it was concluded that the R_d formula developed for RC diaphragms could be used for steel diaphragms also by taking axial stiffness ratio of steel to RC ID in determining stiffness reduction factor.

Table 7.12. Parametric study for bridges with steel diaphragms

S.No	case	Girder Type	Skew	Interior(In) or Exterior(Ex)	strain	%change in strain	Deflection (in)	%change in deflectio	FEM LDF	%fem change	Rd
1	S9L65D0	II	0	In	247.45		0.4311		1.26		
				Ex	252.08		0.4796		1.28		
	S9L65D1	II	0	In	218.22	11.81	0.3944	8.50	1.11	11.74	11
				Ex	265.32	-5.25	0.4948	-3.18	1.34	-5.28	-7.2
2	S9L65D0	II	30	In	224.24		0.3996		1.24		
				Ex	240.53		0.4457		1.31		
	S9L65D1	II	30	In	201.75	10.03	0.3710	7.16	1.15	7.39	6.1
				Ex	245.40	-2.03	0.4581	-2.78	1.35	-2.70	-5.06
3	S9L65D0	II	50	In	179.37		0.3281		1.19		
				Ex	218.91		0.3953		1.43		
	S9L65D1	II	50	In	158.20	11.80	0.3022	7.89	1.13	5.39	4.4
				Ex	223.95	-2.30	0.4048	-2.41	1.47	-2.93	-5.06
4	S9L110D0	lV		In	160.00		0.5682		1.14		
				Ex	177.26		0.7066		1.26		
	S9L110D2	lV		In	143.90	10.06	0.5071	10.75	1.03	10.07	9.2
				Ex	181.47	-2.37	0.6857	2.96	1.29	-2.36	-2.9
5	S9L110D0	IV	30	In	152.61		0.5504		1.14		
				Ex	172.71		0.6624		1.28		
	S9L110D2	IV	30	In	140.54	7.91	0.5015	8.88	1.05	8.07	4.6
				Ex	175.44	-1.58	0.6730	-1.59	1.30	-1.52	-1.7
6	S9L110D0	IV	50	In	135.27		0.4960		1.12		
				Ex	162.83		0.5995		1.33		
	S9L110D2	IV	50	In	123.45	8.73	0.4480	9.68	1.02	8.59	3.2
				Ex	165.02	-1.34	0.6072	-1.27	1.35	-1.55	-1.7
7	S9L130D0	BT	0	In	167.40		0.5970		1.20		
				Ex	182.90		0.7390		1.30		
	S9L130D2	BT	0	in	136.91	18.22	0.4670	21.78	0.98	18.52	17.8
				ex	189.51	-3.61	0.7686	-4.01	1.35	-3.84	-3.9
8	S9L130D0	BT	30	In	160.36		0.5804		1.18		
				Ex	179.00		0.6988		1.31		
	S9L130D2	BT	30	In	143.68	10.40	0.5027	13.38	1.05	10.66	8.9
				Ex	182.03	-1.69	0.7136	-2.12	1.34	-1.64	-2.4
9	S9L130D0	BT	50	In	145.71		0.5370		1.15		
				Ex	172.09		0.6496		1.34		
	S9L130D2	BT	50	In	133.25	8.55	0.4831	10.03	1.04	9.11	6.2
				Ex	173.75	-0.96	0.6566	-1.08	1.35	-0.88	-2.4

Table 7.13. Stiffness ratio of steel ID to RC ID for particular girder type

Girder Type	Stiffness ratio (%)
II	71
III	56.5
IV	46.5
BT	40

8. SUMMARY, CONCLUSIONS AND FUTURE WORK

8.1. Summary

Through this research, the effect of intermediate diaphragms on load distribution has been quantified. The current AASHTO design codes do not include and also not much literature was available, on quantifying the ID performance in load distribution. A systematic parametric study was carried out, using a wide range of values for possible parameters which were representative of the current prestressed concrete girder bridges existing in Louisiana. From the results obtained through this parametric study, formulas were developed for diaphragm effect on load distribution for both interior and exterior girders. Alternative steel diaphragm configurations for different bridge configurations have been proposed, which could perform equivalent to RC diaphragms. A study was made on relative performance of RC IDs and the steel IDs during the process of construction of deck. Later it was checked whether the proposed steel diaphragms could carry the potential loads coming on it. This was followed up by study of influence of reinforced concrete IDs and steel IDs under lateral impact loading, keeping in view the possible collision caused in the prestressed concrete bridges due to over height trucks passing under them. Overall through these studies, various issues relating to ID effectiveness were covered to assess the need for reinforced concrete intermediate diaphragm and alternate ID configurations have been proposed.

8.2. Conclusions

The parametric study was conducted successfully, which was done to understand how each parameter influences the diaphragm performance on load distribution factor. From the initial parametric study it was concluded that the ID influence on bridge performance is mainly a function of span length, skew, diaphragm stiffness and location of diaphragms, and was found to be relatively independent of continuity, girder spacing and number of spans.

Correction factors for ID effectiveness for both interior and exterior girders have been determined from the results obtained from parametric study. The entire results of this study relating to load distribution have been summarized in APPENDIX B. Table b.13 and b.14, list the difference between the results obtained from FEM and those by using the formulae developed for reduction in load distribution (R_d) for interior and exterior girders due to ID and the values of load distribution respectively. The absolute difference between these results obtained from FEM and those obtained from formulae being small, with very few exceptions and the results being conservative in these cases, proves the adequacy of the formulae developed in this study.

From the results obtained in parametric study and the formulae developed, it could be concluded that the ID decreases the load distribution factor for interior girders and increases the load distribution factor for exterior girders.

Using the correction factors so developed for accounting the influence of ID, a more rational load distribution factor could be obtained. The formulae developed for increase in load distribution due to ID for exterior girder gains importance as no rational formula is available for determining this increment in LDF due to IDs. The values of load distribution factor for exterior girder obtained from the formulae in section *4.6.2.2.2d* of AASHTO LRFD (2004) obtained by assuming cross section deflects and rotates as a rigid cross section were found to be 0.988 and 1.523 for 5ft and 9ft spacing respectively. The values of LDF obtained using this approach is larger than the LDF obtained by including the ID effect on exterior girder (using formulae for R_d) on AASHTO LRFD values without ID. This proves that when AASHTO LRFD is used in calculating the LDF for exterior girder including R_d to LDF value without ID would give a smaller LDF value when compared to LDF values obtained from LRFD code. The correction factors developed for calculating LDF for interior girder could be used in different design codes. AASHTO STD gives more conservative values of LDF, and including the correction factor for exterior girder would give more conservative results, therefore the ID correction factor is not recommended to be included for exterior girder.

The IDs increased the deflection marginally for exterior girder and decreased the deflection for interior girder. For both with and without IDs the deflections were observed to be within permissible limits, thereby indicating that deflection is not an important criterion influencing the decision of elimination of ID or replacement of alternate steel IDs.

The alternate steel diaphragms were proposed based on the minimum target stiffness as a proportion of the absolute diaphragm stiffness contributed by the existing RC ID. These steel IDs were found to provide stability nearly to that produced by RC IDs during the construction of deck. Therefore if the reinforced concrete diaphragms were provided only for the purpose of providing girder stability during construction, then this could as well be served by providing steel diaphragm as both these IDs were observed to provide nearly same stability

Results obtained from the impact tests carried out on the bridge with different ID configurations indicated that RC ID seemed to provide greatest protection to exterior girders undergoing impact, when the impact takes place at ID location. While when the impact takes place at a location away from ID, it was observed that the ID configuration does not significantly influence the bridge performance. From these results it was concluded that the IDs could not be counted on for its ability to protect girders if the IDs are not right above the traffic lanes. In case where there is no traffic passing under the bridge a steel ID could as well be used if IDs are provided only for the purpose of providing stability.

From all the results and from the conclusions drawn earlier it seems that a steel diaphragm could potentially replace the current RC diaphragms as it is more economical to provide a steel ID than an RC ID. For reaching the conclusion on whether ID could be eliminated totally, further study is needed to determine the absolute stability of prestressed concrete girders during the process of construction of bridge deck.

The conclusions can be summarized in brief as follows:

- Parametric study conducted successfully
- Correction factors for accounting ID influence in load distribution determined and found to be accurate
- Alternative steel ID has been proposed for all the girder type
- Steel ID was found to provide same amount of stability as RC ID. And if IDs are provided only for the purpose of providing stability, steel ID can replace RC ID
- Steel ID was found to be sufficient in carrying different loads coming onto girder
- Impact studies indicated that RC IDs provided greatest protection to girder if the impact takes place near the ID and if impact is away from ID location no ID provides protection. If IDs are to be provided to protect against impact, they must be placed at locations of possible impact.

8.3. Future Work

Based on the results obtained, the conclusions made earlier and the limitations of the current study the following work has been proposed for future work-

- The amount of stiffness contributed by diaphragm is a function of ID-girder connection and there is a need to know this value. This is due to the possible cracking at the ID-girder interface only a portion of diaphragm stiffness contributes towards load distribution. For determining this value a nonlinear finite element analysis is to be carried out to know the condition of ID-girder connection under heavy loads.
- A detailed cost analysis is needed to be done to understand the economics behind how providing different diaphragm configurations would affect the cost of construction of bridge.
- More detailed study is needed to understand the stability of girders during the process of construction of deck and how ID and ID type affect the stability, which would help in deciding over the absolute elimination of ID. As this study was limited to comparing the stability provided by RC and steel diaphragm relative to each other, it could not be concluded whether there is a need to provide ID for the purpose of providing stability. This is because the targeted construction/ accidental loads are not well defined and the nonlinearities associated with impact
- Alternative systems are to be designed and developed for protecting the girders under lateral impact

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APPENDIX A. SPREAD SHEET FOR SIMPLIFIED 3-D MODEL

*TITLE 'BEARING EFFECT'			$		INPUT
STRUDL 'BRIDGE' 'BEARING EFFECT'					
$					
$	**TOTAL NO. OF GIRDER(Max. =14)**			**10**	
$	**NODE IN X DIRECTION =**			**111**	
$	**ELEMENT IN X DIRECTION**			**110**	
$	**NODES BETWEEN GIRDER**			**5**	
$	**TOTAL NODE LINES**			**59**	
$	**SPAN LENGTH first deck edge (origin) =**			**1140**	**in**
$	**SPAN LENGTH of second deck edge =**			**1140**	**in**
$	**SPAN LENGTH INCREMENTS =**			**0**	**IN/IN**
$	**NODE INCREMENT =**			**10.363636**	**in**
$	**SUPERELEVATION =**			**0**	**IN/IN**
$	**VERTICAL CURVE**			**0**	**IN/IN**
$					
$	**HAUNCH**			**0**	**IN**
$	**SLAB THICKNISS**			**8**	**IN**
$					
$	**END DIAPHRAM CONNECTED TO SLAB**			**YES**	
$	**SLAB STIFFENING**			**YES**	
$	**PARAPET STIFFENER**			**no**	
$	**ID CONNECTION TO SLAB**			**NO**	
UNITS KIPS INCHES DEGREE					
$					
$ GENERATE MESH NODES					
$					
GENERATE	111	JOINT	CART	ID	1
GENERATE	111	JOINT	CART	ID	112
GENERATE	111	JOINT	CART	ID	223
GENERATE	111	JOINT	CART	ID	334
GENERATE	111	JOINT	CART	ID	445
GENERATE	111	JOINT	CART	ID	556
GENERATE	111	JOINT	CART	ID	667
GENERATE	111	JOINT	CART	ID	778
GENERATE	111	JOINT	CART	ID	889
GENERATE	111	JOINT	CART	ID	1000
GENERATE	111	JOINT	CART	ID	1111
GENERATE	111	JOINT	CART	ID	1222
GENERATE	111	JOINT	CART	ID	1333
GENERATE	111	JOINT	CART	ID	1444
GENERATE	111	JOINT	CART	ID	1555
GENERATE	111	JOINT	CART	ID	1666
GENERATE	111	JOINT	CART	ID	1777
GENERATE	111	JOINT	CART	ID	1888
GENERATE	111	JOINT	CART	ID	1999
GENERATE	111	JOINT	CART	ID	2110
GENERATE	111	JOINT	CART	ID	2221
GENERATE	111	JOINT	CART	ID	2332
GENERATE	111	JOINT	CART	ID	2443
GENERATE	111	JOINT	CART	ID	2554

Fig a1. Sample Sheet of a finite element model developed using spreadsheet

APPENDIX B. SUMMARY OF RESULTS

Table b.1. Reduction in LDF for interior girder of bridges with 0^o skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae		
			% reduction in strain	% reduction in Deflection	% reduction in LDF	abs. ID stiffness	40% of abs. ID stiffness	Steel ID
1	S5L50	II	12.2	9.3	12.0	11.4	5.91	9.39
2	S9L50	II	11.4	9.4	11.3	11.4	5.91	9.39
3	S5L65	II	14.6	9.8	14.4	13.4	6.94	11.01
4	S9L65	II	13.5	10.2	13.5	13.4	6.94	11.01
5	S5L70	III	17.0	11.9	16.8	16.1	8.31	10.98
6	S9L70	III	16.5	12.4	16.4	16.1	8.31	10.98
7	S5L90	III	18.6	11.7	18.4	18.7	9.67	12.78
8	S9L90	III	18.0	12.5	18.0	18.7	9.67	12.78
9	S5L95	IV	14.4	15.1	14.3	15.2	9.56	10.36
10	S9L95	IV	15.3	16.5	15.2	15.2	9.56	10.36
11	S5L110	IV	12.9	13.9	12.9	13.5	8.50	9.22
12	S9L110	IV	13.7	15.2	13.7	13.5	8.50	9.22
13	S5L105	BT	26.9	28.1	26.8	27.8	22.28	22.28
14	S9L105	BT	27.4	27.4	28.4	27.8	22.28	22.28
15	S5L130	BT	23.3	24.8	23.3	22.3	17.84	17.84
16	S9L130	BT	22.6	25.6	22.7	22.3	17.84	17.84

Table b.2. Reduction in LDF for interior girder of bridges with 30^o skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae		
			% reduction in strain	% reduction in Deflection	% reduction in LDF	abs. ID stiffness	40% of abs. ID stiffness	Steel ID
1	S5L50	II	10.9	6.0	7.7	6.3	3.3	5.2
2	S9L50	II	9.9	7.3	7.2	6.3	3.3	5.2
3	S5L65	II	12.6	6.0	7.6	7.4	3.8	6.1
4	S9L65	II	8.3	6.9	7.9	7.4	3.8	6.1
5	S5L70	III	13.9	7.1	9.2	8.8	4.6	6.0
6	S9L70	III	12.6	8.7	9.8	8.8	4.6	6.0
7	S5L90	III	15.3	7.1	10.1	10.3	5.3	7.0
8	S9L90	III	13.6	8.3	10.7	10.3	5.3	7.0
9	S5L95	IV	9.6	10.4	9.7	7.6	4.8	5.2
10	S9L95	IV	11.2	12.1	11.3	7.6	4.8	5.2
11	S5L110	IV	8.6	9.7	8.5	6.7	4.2	4.6
12	S9L110	IV	10.0	11.2	10.2	6.7	4.2	4.6
13	S5L105	BT	14.1	15.3	13.8	13.9	11.1	11.1
14	S9L105	BT	13.6	15.0	13.9	13.9	11.1	11.1
15	S5L130	BT	13.0	14.2	13.2	11.1	8.9	8.9
16	S9L130	BT	11.5	13.5	11.8	11.1	8.9	8.9

Table b.3. Reduction in LDF for interior girder of bridges with 50° skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae		
			% reduction in strain	% reduction in Deflection	% reduction in LDF	abs. ID stiffness	40% of abs. ID stiffness	Steel ID
1	S5L50	II	9.4	6.7	4.0	4.6	2.4	3.8
2	S9L50	II	12.1	8.6	5.2	4.6	2.4	3.8
3	S5L65	II	16.0	6.6	6.8	5.4	2.8	4.4
4	S9L65	II	12.9	8.4	5.8	5.4	2.8	4.4
5	S5L70	III	15.6	6.3	8.3	6.4	3.3	4.4
6	S9L70	III	13.4	9.4	7.0	6.4	3.3	4.4
7	S5L90	III	17.0	6.9	8.2	7.5	3.9	5.1
8	S9L90	III	13.6	8.4	7.5	7.5	3.9	5.1
9	S5L95	IV	8.4	10.3	8.7	5.3	3.3	3.6
10	S9L95	IV	12.3	12.9	11.3	5.3	3.3	3.6
11	S5L110	IV	7.3	9.5	7.2	4.7	3.0	3.2
12	S9L110	IV	10.2	11.4	10.1	4.7	3.0	3.2
13	S5L105	BT	9.2	10.9	9.0	9.7	7.8	7.8
14	S9L105	BT	11.2	12.0	11.2	9.7	7.8	7.8
15	S5L130	BT	8.3	10.2	8.5	7.8	6.2	6.2
16	S9L130	BT	8.9	10.7	9.6	7.8	6.2	6.2

Table b.4. Reduction in LDF for exterior girder of bridges with 0° skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae
			% reduction in strain	% reduction in Deflection	% reduction in LDF	
1	S5L50	II	-9.2	-5.4	-9.0	-9.21
2	S9L50	II	-7.4	-5.0	-7.3	-9.21
3	S5L65	II	-7.3	-3.5	-7.3	-7.23
4	S9L65	II	-6.1	-3.4	-6.1	-7.23
5	S5L70	III	-9.5	-4.8	-10.2	-9.57
6	S9L70	III	-8.2	-4.7	-8.1	-9.57
7	S5L90	III	-7.1	-2.7	-9.2	-6.93
8	S9L90	III	-6.3	-3.0	-6.2	-6.93
9	S5L95	IV	-3.8	-3.6	-3.8	-5.085
10	S9L95	IV	-5.6	-5.1	-5.6	-5.085
11	S5L110	IV	-0.8	-1.3	-0.7	-2.88
12	S9L110	IV	-2.8	0.0	-2.8	-2.88
13	S5L105	BT	-5.9	-5.7	-5.0	-7.615
14	S9L105	BT	-8.2	-8.1	-8.2	-7.615
15	S5L130	BT	-1.8	-2.5	-1.8	-3.94
16	S9L130	BT	-4.0	-4.6	-4.0	-3.94

Table b.5. Reduction in LDF for exterior girder of bridges with 30° skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae
			% reduction in strain	% reduction in Deflection	% reduction in LDF	
1	S5L50	II	-4.3	-3.5	-4.9	-6.447
2	S9L50	II	-4.9	-4.2	-5.8	-6.447
3	S5L65	II	-4.0	-2.6	-4.4	-5.061
4	S9L65	II	-3.8	-2.9	-4.7	-5.061
5	S5L70	III	-5.9	-3.7	-6.1	-6.699
6	S9L70	III	-5.6	-4.0	-6.4	-6.699
7	S5L90	III	-4.6	-2.3	-4.7	-4.851
8	S9L90	III	-4.3	-2.6	-5.0	-4.851
9	S5L95	IV	-3.2	-3.1	-2.7	-3.051
10	S9L95	IV	-3.3	-3.1	-3.3	-3.051
11	S5L110	IV	-1.4	-1.9	-1.5	-1.728
12	S9L110	IV	-1.9	-2.0	-1.8	-1.728
13	S5L105	BT	-4.7	-4.7	-4.6	-4.569
14	S9L105	BT	-4.9	-4.8	-4.8	-4.569
15	S5L130	BT	-1.2	-2.0	-1.2	-2.364
16	S9L130	BT	-1.7	-2.3	-1.6	-2.364

Table b.6. Reduction in LDF for exterior girder of bridges with 50° skew

S.No.	Case	Girder type	FEM results for .100% ID stiffness			Rd from formulae
			% reduction in strain	% reduction in Deflection	% reduction in LDF	
1	S5L50	II	-4.8	-3.5	-6.5	-6.447
2	S9L50	II	-3.1	-3.9	-6.1	-6.447
3	S5L65	II	-2.4	-2.4	-6.1	-5.061
4	S9L65	II	-2.4	-2.9	-5.1	-5.061
5	S5L70	III	-4.0	-3.3	-7.0	-6.699
6	S9L70	III	-3.9	-3.8	-6.5	-6.699
7	S5L90	III	-3.2	-2.3	-5.9	-4.851
8	S9L90	III	-3.1	-2.6	-5.2	-4.851
9	S5L95	IV	-2.9	-2.9	-2.8	-3.051
10	S9L95	IV	-3.3	-2.9	-4.0	-3.051
11	S5L110	IV	-1.6	-1.9	-1.4	-1.728
12	S9L110	IV	-1.4	-1.5	-1.7	-1.728
13	S5L105	BT	-3.8	-3.8	-3.5	-4.569
14	S9L105	BT	-3.6	-3.5	-3.9	-4.569
15	S5L130	BT	-1.3	-1.8	-1.3	-2.364
16	S9L130	BT	-0.9	-1.3	-0.7	-2.364

Table b.7. LDF for interior girder for bridges with 0^o skew

S.No.	Case	Girder type	FEM results .		applying Rd to (1)		AASHTO STD.	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)	LDF with ID (abs. stiffness)	LDF with ID (40% stiffness)		
1	S5L50	II	0.82	0.72	0.73	0.77	0.91	1.02
2	S9L50	II	1.43	1.27	1.27	1.35	1.64	1.54
3	S5L65	II	0.73	0.63	0.63	0.68	0.91	0.95
4	S9L65	II	1.26	1.09	1.09	1.18	1.64	1.44
5	S5L70	III	0.76	0.63	0.64	0.70	0.91	1.00
6	S9L70	III	1.33	1.11	1.12	1.22	1.64	1.51
7	S5L90	III	0.68	0.55	0.55	0.61	0.91	0.94
8	S9L90	III	1.18	0.97	0.96	1.06	1.64	1.41
9	S5L95	IV	0.70	0.60	0.59	0.63	0.91	0.98
10	S9L95	IV	1.23	1.04	1.04	1.11	1.64	1.48
11	S5L110	IV	0.65	0.57	0.57	0.60	0.91	0.94
12	S9L110	IV	1.14	0.99	0.99	1.05	1.64	1.42
13	S5L105	BT	0.76	0.56	0.55	0.59	0.91	0.99
14	S9L105	BT	1.34	0.96	0.97	1.04	1.64	1.49
15	S5L130	BT	0.70	0.54	0.54	0.57	0.91	0.94
16	S9L130	BT	1.20	0.93	0.93	0.98	1.64	1.41

Table b.8. LDF for interior girder for bridges with 30^o skew

S.No.	Case	Girder type	FEM results .		applying Rd to (1)		AASHTO STD	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)	LDF with ID (abs. stiffness)	LDF with ID (40% stiffness)		
1	S5L50	II	0.82	0.75	0.77	0.79	0.91	0.98
2	S9L50	II	1.40	1.30	1.31	1.35	1.64	1.47
3	S5L65	II	0.73	0.68	0.67	0.70	0.91	0.93
4	S9L65	II	1.24	1.14	1.15	1.19	1.64	1.38
5	S5L70	III	0.76	0.69	0.70	0.73	0.91	0.96
6	S9L70	III	1.31	1.18	1.20	1.25	1.64	1.44
7	S5L90	III	0.69	0.62	0.62	0.66	0.91	0.91
8	S9L90	III	1.17	1.04	1.05	1.10	1.64	1.36
9	S5L95	IV	0.71	0.66	0.66	0.68	0.91	0.95
10	S9L95	IV	1.22	1.08	1.13	1.16	1.64	1.41
11	S5L110	IV	0.65	0.60	0.61	0.62	0.91	0.92
12	S9L110	IV	1.14	1.02	1.06	1.09	1.64	1.37
13	S5L105	BT	0.75	0.65	0.65	0.67	0.91	0.96
14	S9L105	BT	1.31	1.13	1.12	1.16	1.64	1.43
15	S5L130	BT	0.69	0.60	0.61	0.63	0.91	0.91
16	S9L130	BT	1.18	1.04	1.05	1.08	1.64	1.36

Table b.9. LDF for interior girder for bridges with 50^{o} skew

S.No.	Case	Girder type	FEM results .		applying Rd to (1)		AASHTO STD	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)	LDF with ID (abs. stiffness)	LDF with ID (40% stiffness)		
1	S5L50	II	0.83	0.80	0.79	0.81	0.91	0.91
2	S9L50	II	1.34	1.27	1.28	1.31	1.64	1.32
3	S5L65	II	0.70	0.65	0.66	0.68	0.91	0.87
4	S9L65	II	1.19	1.12	1.13	1.16	1.64	1.14
5	S5L70	III	0.72	0.66	0.67	0.70	0.91	0.90
6	S9L70	III	1.27	1.19	1.19	1.23	1.64	1.31
7	S5L90	III	0.67	0.62	0.62	0.64	0.91	0.86
8	S9L90	III	1.14	1.05	1.05	1.09	1.64	1.25
9	S5L95	IV	0.70	0.65	0.67	0.68	0.91	0.89
10	S9L95	IV	1.20	1.06	1.13	1.16	1.64	1.29
11	S5L110	IV	0.66	0.61	0.63	0.64	0.91	0.86
12	S9L110	IV	1.12	1.01	1.07	1.09	1.64	1.26
13	S5L105	BT	0.75	0.68	0.68	0.69	0.91	0.89
14	S9L105	BT	1.27	1.13	1.14	1.17	1.64	1.49
15	S5L130	BT	0.68	0.62	0.63	0.64	0.91	0.86
16	S9L130	BT	1.15	1.04	1.06	1.08	1.64	1.25

Table b.10. LDF for exterior girder for bridges with 0^{o} skew

S.No.	Case	Girder type	FEM results .		LDF with ID (aplying Rd to1)	AASHTO STD	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)			
1	S5L50	II	0.82	0.90	0.90	0.91	0.90
2	S9L50	II	1.23	1.31	1.34	1.64	1.36
3	S5L65	II	0.85	0.91	0.91	0.91	0.84
4	S9L65	II	1.28	1.35	1.37	1.64	1.26
5	S5L70	III	0.83	0.91	0.91	0.91	0.88
6	S9L70	III	1.25	1.35	1.37	1.64	1.33
7	S5L90	III	0.84	0.92	0.90	0.91	0.82
8	S9L90	III	1.28	1.36	1.37	1.64	1.24
9	S5L95	IV	0.82	0.85	0.86	0.91	0.86
10	S9L95	IV	1.19	1.26	1.25	1.64	1.30
11	S5L110	IV	0.86	0.87	0.89	0.91	0.83
12	S9L110	IV	1.26	1.30	1.30	1.64	1.25
13	S5L105	BT	0.85	0.89	0.91	0.91	0.87
14	S9L105	BT	1.27	1.37	1.36	1.64	1.31
15	S5L130	BT	0.86	0.88	0.90	0.91	0.82
16	S9L130	BT	1.30	1.36	1.35	1.64	1.24

Table b.11. LDF for exterior girder for bridges with 30^o skew

S.No.	Case	Girder type	FEM results .		LDF with ID (aplying Rd to1)	AASHTO STD	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)			
1	S5L50	II	0.85	0.89	0.91	0.91	0.86
2	S9L50	II	1.28	1.35	1.36	1.64	1.29
3	S5L65	II	0.87	0.91	0.91	0.91	0.81
4	S9L65	II	1.31	1.38	1.38	1.64	1.21
5	S5L70	III	0.85	0.90	0.90	0.91	0.84
6	S9L70	III	1.28	1.36	1.36	1.64	1.27
7	S5L90	III	0.85	0.89	0.89	0.91	0.80
8	S9L90	III	1.30	1.37	1.37	1.64	1.19
9	S5L95	IV	0.84	0.86	0.86	0.91	0.83
10	S9L95	IV	1.28	1.32	1.32	1.64	1.24
11	S5L110	IV	0.83	0.85	0.85	0.91	0.80
12	S9L110	IV	1.28	1.31	1.31	1.64	1.20
13	S5L105	BT	0.85	0.89	0.89	0.91	0.84
14	S9L105	BT	1.28	1.34	1.34	1.64	1.25
15	S5L130	BT	0.87	0.88	0.89	0.91	0.80
16	S9L130	BT	1.31	1.34	1.35	1.64	1.19

Table b.12. LDF for exterior girder for bridges with 50^o skew

S.No.	Case	Girder type	FEM results .		LDF with ID (aplying Rd to1)	AASHTO STD	AASHTO LRFD
			LDF without ID (1)	LDF with ID (abs. stiffness) (2)			
1	S5L50	II	0.94	1.00	1.00	0.91	0.80
2	S9L50	II	1.40	1.49	1.49	1.64	1.16
3	S5L65	II	0.93	0.98	0.97	0.91	0.76
4	S9L65	II	1.40	1.48	1.47	1.64	1.00
5	S5L70	III	0.90	0.96	0.96	0.91	0.79
6	S9L70	III	1.36	1.45	1.45	1.64	1.15
7	S5L90	III	0.89	0.94	0.94	0.91	0.76
8	S9L90	III	1.36	1.43	1.42	1.64	1.10
9	S5L95	IV	0.87	0.90	0.90	0.91	0.78
10	S9L95	IV	1.33	1.38	1.37	1.64	1.13
11	S5L110	IV	0.86	0.88	0.88	0.91	0.76
12	S9L110	IV	1.33	1.35	1.35	1.64	1.10
13	S5L105	BT	0.88	0.91	0.92	0.91	0.79
14	S9L105	BT	1.32	1.37	1.38	1.64	1.31
15	S5L130	BT	0.88	0.90	0.91	0.91	0.75
16	S9L130	BT	1.34	1.35	1.37	1.64	1.10

- **Note**: The results for the interior girder were obtained by placing loading configuration to maximize straining effect of the innermost girder. In order to determine the actual load distribution factor for interior girder loading configuration must be moved laterally to determine the actual load distribution factor considering the worst case that could be the innermost girder or other girders. Therefore a significant difference between the LDF obtained from the present FEM analysis and AASHTO LDF exists for some cases of the interior girder. As the focus of this study is to see how ID affects the relative load distribution, the difference in loading configuration does not affect the results.

Table b.13. Difference in R_d value obtained from FEM to the value obtained from formulae for absolute (100%) ID stiffness

Interior or Exterior girder	Skew Angle (degrees)	Difference(%)		
		Minimum	Maximum	Absolute average
Interior	0	-1	1	0.5
	30	-0.2	3.7	1.2
	50	-0.7	6	1.8
Exterior	0	-2.3	2.6	1.1
	30	-0.2	1.5	0.5
	50	-1.1	1.7	0.6

Table b.14. Difference in LDF obtained from FEM to that obtained through formulae for absolute (100%) ID stiffness

Interior or Exterior girder	Skew Angle (degrees)	Difference		
		Minimum	Maximum	Absolute average
Interior	0	-0.008	0.01	0.005
	30	-0.045	0.008	0.012
	50	-0.072	0.006	0.017
Exterior	0	-0.024	0.019	0.011
	30	-0.013	0.003	0.005
	50	-0.022	-0.012	0.006

APPENDIX C: DIMENSONS AND SECTION PROPERTIES FOR ID SECTIONS

Table C1: Dimensions and section properties for ID sections used in bridges with different girder types

Girder type	Depth (in)	Width (in)	Area (in^2)	I_y (in^4)
II	30	8	240	2250
III	38	8	304	4572.7
IV	46	8	368	8111.3
BT	66	8	528	104976

VITA

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