STRESS DISTRIBUTION AT THE CORNERS OF CONTINUOUS SKEW BRIDGES

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DEDICATION

TO

MY SOUL MATE " Amal" and My Kids Teeba, Jana, Yousef and Ali.

The Memory of my Father

My Mother, Brothers, Sisters, and all my Friends

Hmoud

Acknowledgement

I would like to express my grateful thanks to my supervisor Prof. Samih Qaqish, for his endless and generous assistance and support in carrying out this study. Without his guidance, effort, and helpful suggestions, this study would never have been achieved.

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NOTATIONS and ABBREVIATIONS

AASHO	American Association of State Highway and Officials now (AASHTO)
AASHTO	American Association of State Highway and Transportation Officials
L	Span length (m)
M	Bending moment (kN.m)
M^{-ve}	Max. Negative Moment (kN.m)
M^{+ve}	Max. Positive Moment. (kN.m)
Max.	Maximum
MOPWH	Ministry of Public Works and Housing
M_{11}	Bending moment in the longitudinal direction (kN.m/m)
M_{12}	Torsional (twisting) moment (kN.m/m)
M_{22}	Bending moment in the transverse direction (kN.m/m)
P	Axial force (kN)
P_{20}	16,000 pounds H20 loading (71.17 kN)
P-82	Pennsylvania permit load
Penn DOT	The Pennsylvania Department of Transportation
S	Effective span length (m)
T	Torsion (kN.m)
V	Shear force (kN)
α	Skew Angle (degree)

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ABSTRACT

Skewed bridges are becoming more commonplace in all developed countries. It is more efficient to design bridges with skewed geometries in urban areas due to the lack of space required for more traditional straight girder bridges. While there have been a multitude of studies on the response of straight, “right-angle” bridges, there has not been a great deal of research on skewed bridges. This research is a theoretical study of the effect of skew angle on analyzing the bridge deck.

The main aims of this research are; Establishing the size of the problem of skew angle in the most common type of highway bridges, The variation of flexural, and shear forces, with respect to the various skew angle of the deck for different span, and the effect of skew on reactions transmitted from the deck to the bearing supports, that is, abutments and/or piers.

A continuous span bridge was taken as a case study for studying the effect of the skew angle of the bridge deck on the analysis results. The span length was taken as 10, 12, 14, and 16 m length. For each span the skew angle was taken to be 0, 10, 20, 30, 40, 50, and 60-degrees. A comparison was made between the bridges of spans 10m, and 12m with and without intermediate diaphragms. The bridge was an equally two span bridge with a girder-slab deck type of superstructure. Analysis of these cases was done by using the computer software SAP2000 and according to AASHTO requirements for loading.

The results of the analysis showed that, increasing the skew angle would increase the Max. Positive Moment on the bridge, and move the position of the Max. Moment in the direction of the middle support. On the other hand, the effect of increasing the skew angle was significantly small on the Max. Shear force at the girders near the middle support, and the same for the Max, Negative Moment. Increasing the skew angle of the bridge deck will increase the vertical reaction at the obtuse corner, but it will decrease at the acute corner of the deck for angles up to 50^o for the span of 10m and 30^o for the other spans, then it will increase. Finally existing of intermediate diaphragms decrease the values of girders’ moments and shear forces. On the other hand, increasing the skew angle in bridges without diaphragms will not effect on the stresses of the girders.

INTRODUCTION

1. General

The design of skewed bridges is becoming more commonplace in the World, especially in the developed countries. It is more efficient to design bridges with skewed geometries in urban areas due to the lack of space required for more traditional straight girder bridges. In addition, skewed bridges are common at highway interchanges, river crossings, and other extreme grade changes where skewed geometries are necessary due to limitations in space.

The majority of skewed bridges constructed in the United States are designed as modified "right-angle" structures (Elizabeth K. Norton, 2001). The girders in a right-angle structure are placed perpendicular to the abutment. The modifications made to convert the skewed bridge to the "right-angle" bridge do not efficiently portray the additional torsional effects caused by the angle of skew.

2. Highway Bridges

The major advantage in the use of concrete for bridges is the wide variation that can be achieved in form. This flexibility, however, does not limit its exclusive use for all major structures. Because of factors like ratio of dead to live load, depth constraints, availability of material, and labor costs, steel structures may be cost effective and must be considered as a possible alternate. Many major bridge projects today include alternate designs (steel and concrete) with appropriate working drawings for use in bidding.

General reinforced concrete bridges can consist of decks, T-beams, or cells. Combinations of these types and precasting these elements can enhance their versatility (Conrad, P. and Richard, A. 1984).

To most people, describing the bridge type is a matter of ambiguous preference and perception for they might be an aware of the true nature of construction of the bridge, or its engineering features. Bridge can be characterized or classified in several ways depending on the objective of classification. The necessity of classifying bridges in various ways has grown as bridges have evolved from simple beam bridges to modern suspension bridges, and cable-stayed bridges. Bridges can be classified according to the following characteristics:

1) Material of construction.
2) Span length.
3) Structure form.
4) Span type.
5) Load path characteristics.
6) Usage.
7) Position (for movable bridges)
8) Deck type (for combination and double-deck bridges) (Taly, N. 1998)

3. Related Research

Researchers studied the effects of skewness on the analysis results and design of highway bridges.

Alasa'd. (1997) studied the effects of the skew angle of simply supported bridges on the stresses in the bridge elements. It has been found that the traditional analysis and design of non-skew bridges might be used to analyze and design skew decks, unless the designer wants to utilize computer methods for more exact results.

Ebeido, and Kennedy. (1996) studied on girder moments and shear distribution in bridges with skew angles of 45°. For this purpose a series of bridges were examined experimentally and analytically. After a series of elastic tests conducted with a simulated truck load, the bridge model was tested to failure. In addition to the deflections and strains recorded during the test, the cracking load of the deck slab, collapse load of the model and crack pattern of the deck slab were monitored during the process. A finite-element model was developed using the software ABAQUS. A sensitivity study was conducted to investigate the parameters, which affect the shear and moment distribution. From this parametric study, empirical formulas for both moment distribution factors and shear distribution factors were developed. It was found that the skew angle is the most critical parameter for the shear distribution and the controlling factor for design in the exterior girder.

Several skewed bridges were slid and fell down from their supports by the 1995 Hyogo-Ken Nanbu Earthquake. Otsuka, H et. al,(1995) studied the rotational behavior of the skewed bridges after failure of the side blocks of the bearings by the horizontal ground motion. Firstly, the geometric configuration of the skewed bridges in which the rotation is inevitable was investigated. Then, the rotational displacement of the skewed bridges were obtained by conducting non-linear time-history analyses in which a friction type hysteric model was assumed to simulate the sliding of the bridge at the supports. It was found that skewed bridges with small width-span ratios and small skew angles may have considerable sliding rotational displacements and fall down from their supports if adequate seat width is not provided.

The seismic response of a skew reinforced concrete box girder bridge-the Foothill Boulevard Under crossing, was analyzed and studied using finite element models by Meng, J. and Lui, E. (1999). The effects of superstructure flexibility, substructure boundary conditions, structural skewness and stiffness eccentricity were assessed using spectral analyses. The results showed that the internal forces and displacements of the supporting columns as well as the displacement of the deck would be underestimated if one neglected the flexibility of the bridge deck. The study also demonstrated that the seismic response of the bridge was affected quite noticeably by the boundary conditions of the bridge columns and the overall skewness of the bridge. Based on this study, a theory explaining the failure of the bridge was presented.

In 2002 Shervin Maleki. study, the importance of modeling assumption, in conjunction with AASHTO's seismic analysis methods, is highlighted. Furthermore. To investigate the effect of this assumption on analysis, a parametric study was performed on bridges with skews ranging from 0 to 60 degrees and with spans up to 30 m's. It was assumed that the bridges are elastically supported with elastomeric or pinned bearings in the longitudinal direction, and cross-frames in the skew direction at each end. Linear finite element response spectrum dynamic analysis was performed on bridges with decks being modeled as rigid and flexible shell elements. The effects of deck stiffness on the translational and rotational periods of vibration were noted. Stresses for flexible decks were evaluated and shown to be negligible. Seismic demands on supporting elements with rigid and non-rigid decks were compared. It was shown that the rigid deck assumption simplifies the analysis and was valid for practical ranges of slab-girder bridges.

Ebeido. and Kennedy. (1996) The influence of skew, as well as other design parameters on the shear and reaction distribution factors of continuous two-span composite steel-concrete bridges were investigated. Results from tests on three continuous composite steel-concrete bridge models verified the finite-element analysis for such bridges. From a parametric study, expressions for both shear and reaction distribution factors for American Association of State Highway and Transportation Officials (AASHTO) truck loading as well as for dead load were deduced.

4. Modeling Importance

Recently, much experience has been acquired in modeling highway bridges. This has been given to predictive approaches grouped loosely under the term modeling.

Three issues hamper modelling efforts:

1) Proper mathematical description of the physics and mathematics governing designing structures with complication of skewness.
2) Scarcity of data (material, geometry and different site conditions)
3) Computational power required for applying sophisticated models to realistic situations.

The goal of developing comprehensive and physically realistic models inevitably leads to increasing computational requirements and numerical complexity. Thus, there is a continued need for robust and efficient numerical algorithms. Advancement in solution techniques will also aid in development and application of management models for optimal design and operation of skew bridge applications.

Accurate knowledge of the initial boundary, and loading conditions are significant issues that needed in skew bridge design. Importance issues include:

a) Model validity and reliability, i.e. "how well does the model mimics the physical field conditions and the required design case?"

b) Knowledge of initial material specifications, bridge loading and stress distribution, including site geology and geometry, "Do we have full and accurate knowledge of the site, material properties and initial conditions affecting bridge behavior under different loading conditions?"

c) "Are the support type, material type, stresses, shear and reaction interaction well known, and how significant are they in controlling the bridge optimal design and behavior?"

5. Study Objectives

In this research, the effect of the skew angle on the analysis and design of a continuous two-spans bridge will be investigated. Also the effects of changing the span length on the internal forces of the bridge will be studied.

The span length will vary from 10m's to 16 m's by a 2-meter increment. The skew angle will also vary from zero degree to 60 degrees, with 10-degrees increment for each considered span. Concrete properties and bridge width will be kept constant for all the cases. Applied live load will be the loads specified by AASHTO for lane loading (HS20-44). Dead load will include the wearing surface, sidewalk, and parapet. AASHTO load factors will be used for computing the ultimate load applied on the bridge.

The bridge model will be analyzed utilizing the software SAP2000. Values to be monitored are moments of the deck slab, max. moment (positive and negative), max. shear force , max. torsion, and max. reactions at corners of bridge's girders . These results will be summarized in tables, graphically drawn, and studied to find the effect of changing the span length and the skew angle on the analysis results.

THEORETICAL BACKGROUND

1. General

Slab-girder bridges are the most common type of bridge construction used for short to medium range spans. (Shervin Maleki, 2002). They consist of a concrete deck spanning over concrete or steel longitudinal girders, as illustrated in Figure 1.

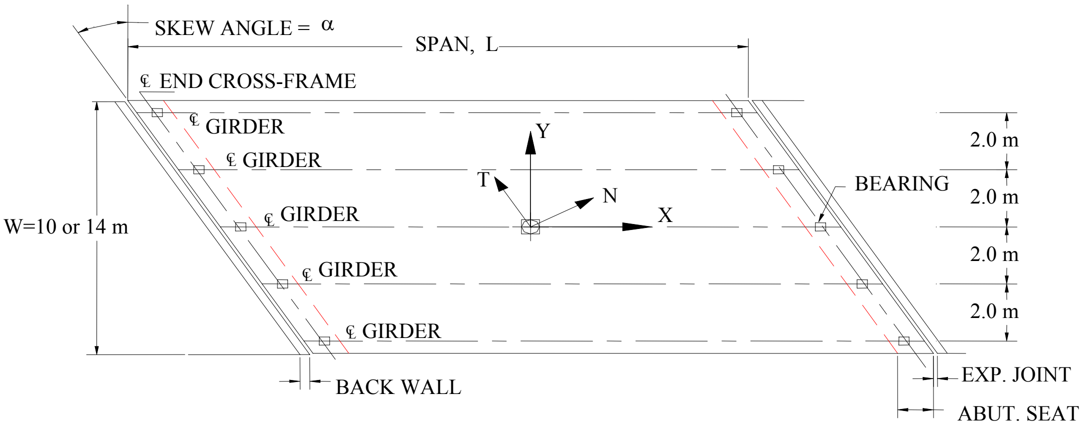


Figure 1. Typical Skewed Bridge Plan View (Shervin Maleki, 2002).

Currently AASHTO does not differentiate between straight (non-skewed) bridges and skewed bridges. It proposes a single mode (or uniform load) method for regular bridges and a multimode elastic method for irregular bridges. Although skewed bridges do not exactly match the definition of irregularity, one should resort to multimode method for analysis of these bridges, (Shervin Maleki, 2002).

2. Bridge Geometry

When talking about bridges, the term longitudinal is used to denote a direction parallel to traffic, while "transverse" denotes to the direction perpendicular to it, as shown in Figure 2.

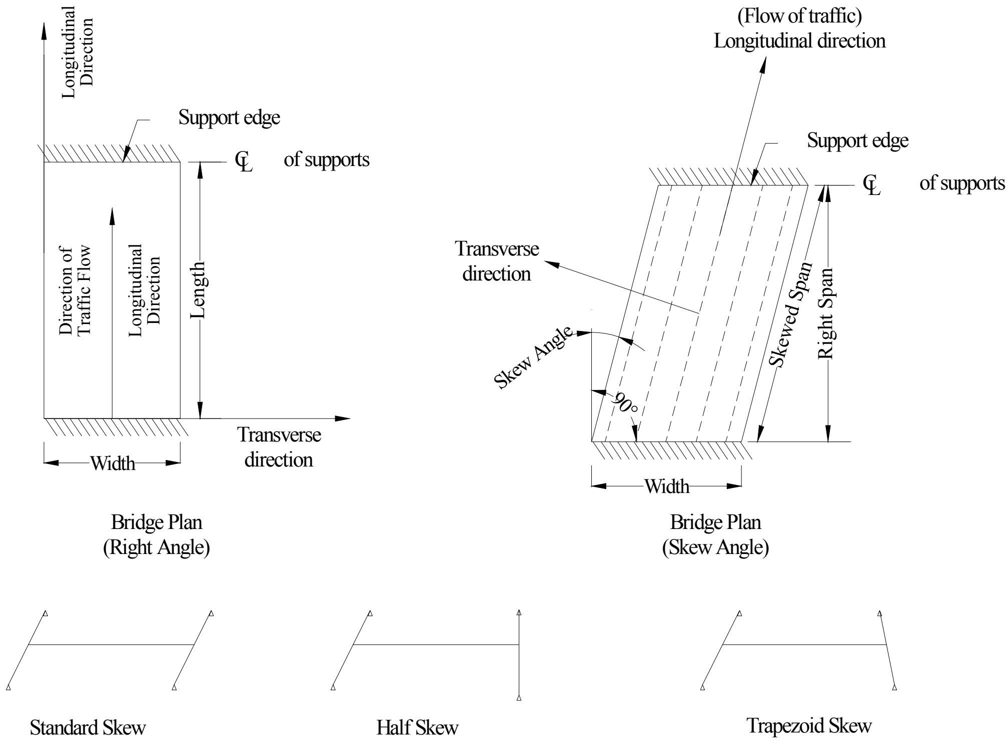


Figure 2. Definitions of right and skew bridges (Taly, N 1998).

From geometric consideration, bridges are often described as normal (right) skew or curved. Normal or right bridges are those in which longitudinal axes of the bridge, which is parallel to the longitudinal axes of the slab and the supporting beams (when present) are normal to the centerlines of the supports (abutments and/or piers) Figure 2. Often, such as a plan configuration may not be feasible because of human-created obstacles, complex intersection, space limitations, mountainous terrain, etc. resulting in a skew bridge.

Skew bridges, simple or continuous, are bridge where the longitudinal axes, forms an acute angle instead of a right angle with the centerlines of the supports, as shown in Figure 2. The skew angle is defined as the angle between the centerline of the support and the normal to the axes of the bridge. The skew angles at the two end supports may not necessarily be the same, as shown in the Figure 2. Abridge geometry with skewed but parallel lines of supports at the two opposite ends are known as a standard-skew bridge. Bridges with the centerline of support at one end normal to the bridge axes but with the other support skew is known as a half-skew bridge. For those with different skew angle at the two supports are known as a trapezoidal skew bridge.

The skew angle is an important parameter affecting the analysis of the bridge structure. Whether simple or continuous-span bridge with torsional stiff girders, the skew angle can have a considerable effect on the shear and bending moment in the girder. It has been suggested that for skew angles not exceeding 20 degrees (30 degrees for slab on beams), bridges can be safely designed as right bridges by simplified methods (Hambly, 1991).

3. AASHTO Specifications

3.1 General

Specifications for highways and bridges adopted by Ministry of Public Works and Housing of Jordan (MOPWH) stipulates that highway bridges should be designed for live load as per AASHTO specifications. Additional factor to encounter the increase in live load of 50% is additional requirement for MOPWH.

The standard of live load as per by AASHTO specifications is divided into: - truck loading and lane loading. Four classes of loading are defined within each type. Both type of loading are assumed to be applicable for a given structure. The loading, which produces maximum stress, governs the design.

3.2 Design Loads

Design live loads for highways have been and continued to be a subject of considerable research, where several design load models have been suggested in the United States. For design purposes, the design vehicular live loads are divided into three categories:

1) Design truck loading
2) Design lane loading
3) Alternate military (or design tandem) loading

3.2.1 Design truck loading

Also referred to as standard truck loading. This type of loading has originated in the 1920's and although it has been revised periodically, its basic formal has remained the same. Two systems of loadings are provided: The H loading and the heavier HS loading; the letter (S) refers to semi-trailer. In each case there are two standard classes of loadings, which are designated as follows:

- H 15-44 and H 20-44, as shown in Figure 3
- HS15-44 and HS20-44, as shown in Figure 4

In these designations, the number 44 refers to the fact that these loadings were standardized and first published in the 1944 American Association of State Highway and Officials (AASHO) specifications. The H loading consists of a two-axle truck, and the numbers 15 and 20 in the loading classification refer to the gross truck weight in US tons (1 ton =2000 lb =8.9 kN). The HS loading consists of a tractor truck with semi-trailer (designated by the letter "S" in HS). The numbers following the letters HS indicate the total load in tons carried by the axles of the tractor; the load on the semi-trailer is additional.

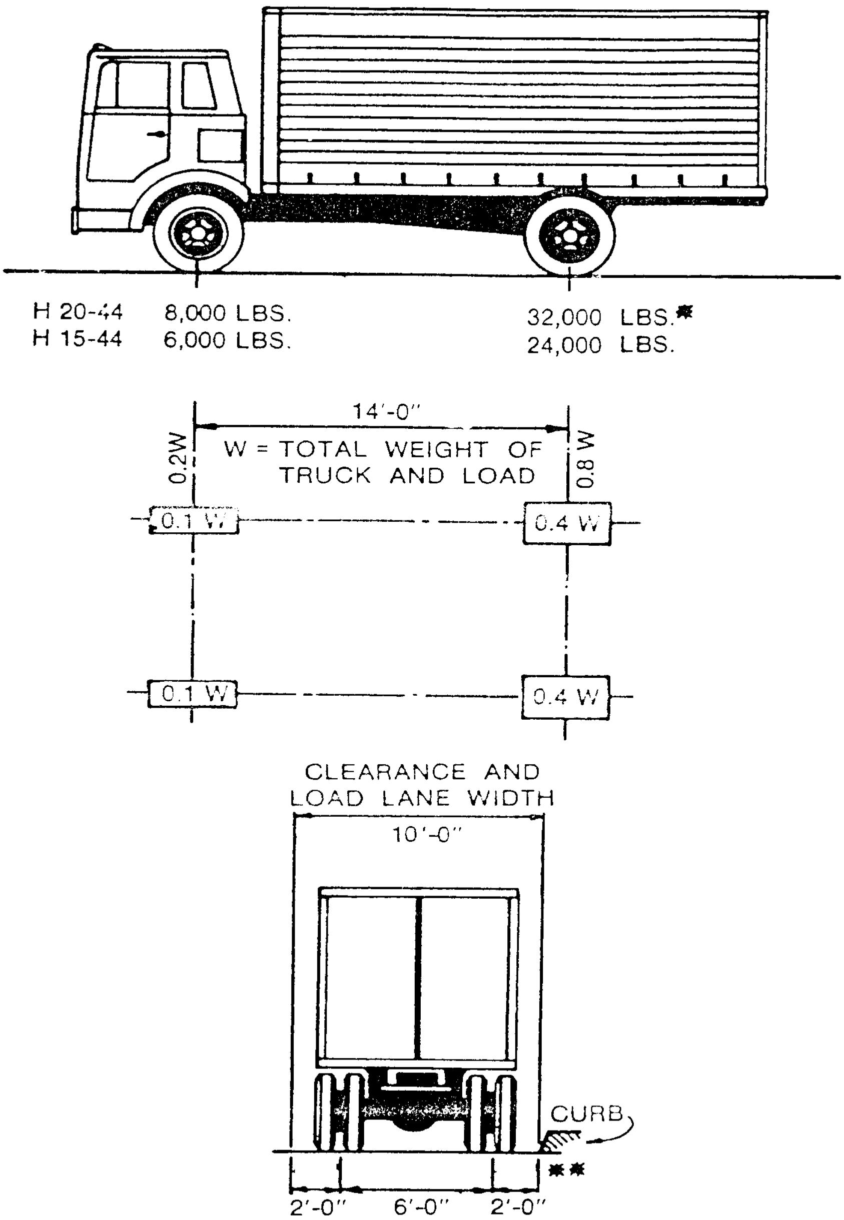
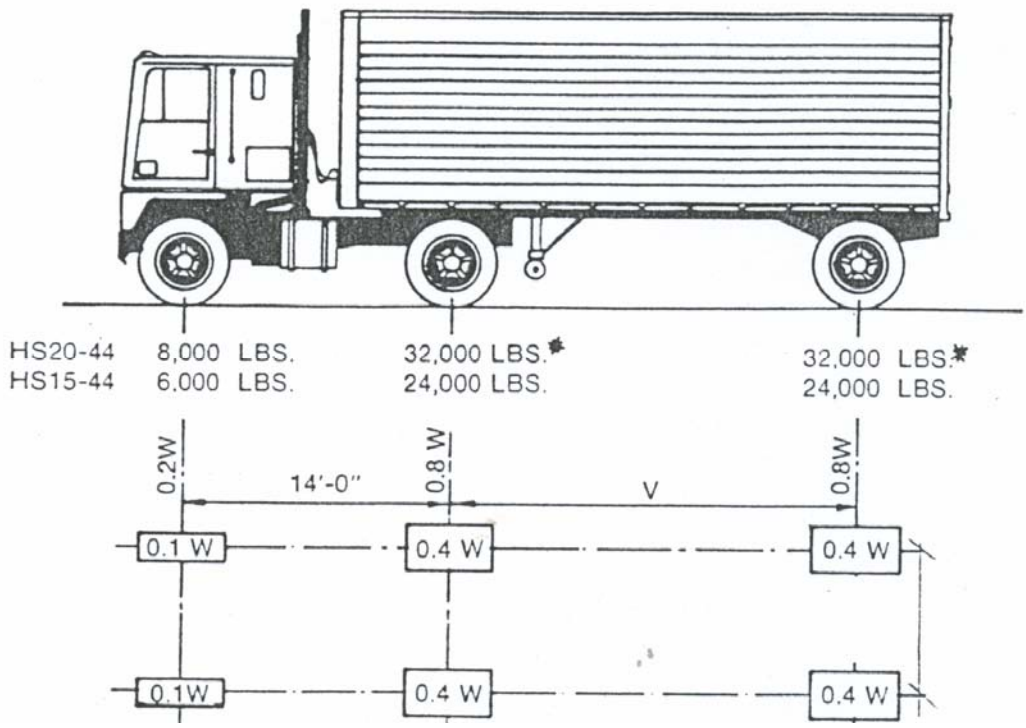


Figure 3. Standard AASHTO H15-44 and H 20-44 trucks



W = COMBINED WEIGHT ON THE FIRST TWO AXLES WHICH IS THE SAME AS FOR THE CORRESPONDING H TRUCK.
V =VARIABLE SPACING — 14 FEET TO 30 FEET INCLUSIVE SPACING TO BE USED IS THAT WHICH PRODUCES MAXIMUM STRESSES.

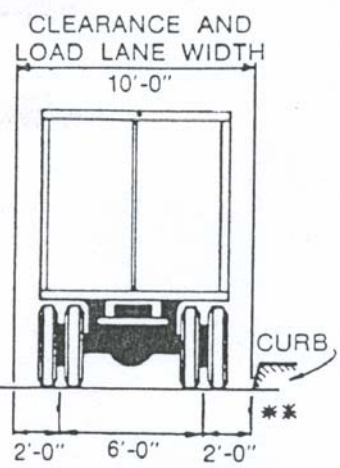


Figure 4. Standard AASHTO HS15-44 and HS20-44 trucks

The variable axle loading has been introduced for two specific reasons:

1. It approximates more closely the spacing of axles for the tractor-trailer currently in use.
2. It provides a more satisfactory design loading for Continuous spans. The variable spacing of axles permits positioning of heavy axles on the adjoining spans to produce maximum negative moments.

Note that the H15 and HS15 trucks are three-fourth as a heavy as H20 and HS20 trucks, respectively, as shown in Figure 3 and 4. The older versions of AASHTO specifications (then AASHO specifications) specified a smaller loading also, H10 and HS10 loading, with loads that are one-half as much as the H20 and HS20 1oading. However, in recent years this loading has been removed out from the specifications. It is pointed out here that some agencies design their bridges for HS25 trucks, which are simply assumed to have axle load 25 percent heavier than those of HS20 trucks. For example, the Pennsylvania Department of Transportation mandates (Penn DOT. 1993), "All new bridges regardless of roadway class or funding source shall be designed for HS25 loading (125percent of HS20-44) 125 percent of alternate loading two axles 4 feet apart with each axle carrying 30,000 pound (133.45 kN), or the Pennsylvania permit load (P-82) used for permit purposes whichever produces the greatest effect for the loading combination under consideration. When using (P-82) the design must be in accordance with loading Combination Group IB in AASHTO specifications". (Taly, N 1998).

3.2.2. Design lane loading

Lane loading was developed to better model loading on long spans, where a string of light vehicles might be critical. It approximates a 20-ton truck preceded and followed by a 15-ton truck. Essentially, the assumption of uniformly distributed lane loading obviates the necessity of having more than one design truck in a lane regardless of the span length and the number of spans, resulting in a simple design procedure for long-span bridges.

Lane loading has two classes of loadings, and in each class two different loadings are provided. These loadings are designated in the same manner as the truck loadings (H15-44, HS15-44, H20-44, HS20-44). Basically, the lane load consists of a uniform load accompanied by a concentrated load. The value of the concentrated load is different for shear than for moment. Furthermore, as with truck loadings, the loads for HS lane loading, including the concentrated loads are only three-fourths as heavy as those for the HS20 lane loading. Both the concentrated and the uniform load specified for lane loading are assumed to he distributed over a 10-ft width normal to the centerline of the lane, as shown in Figure. 5. Different concentrated loads are to be used for calculating forces in the supporting members. The lighter concentrated loads are to be used for calculating bending moment; the heavier concentrated load should be used for calculating shear.

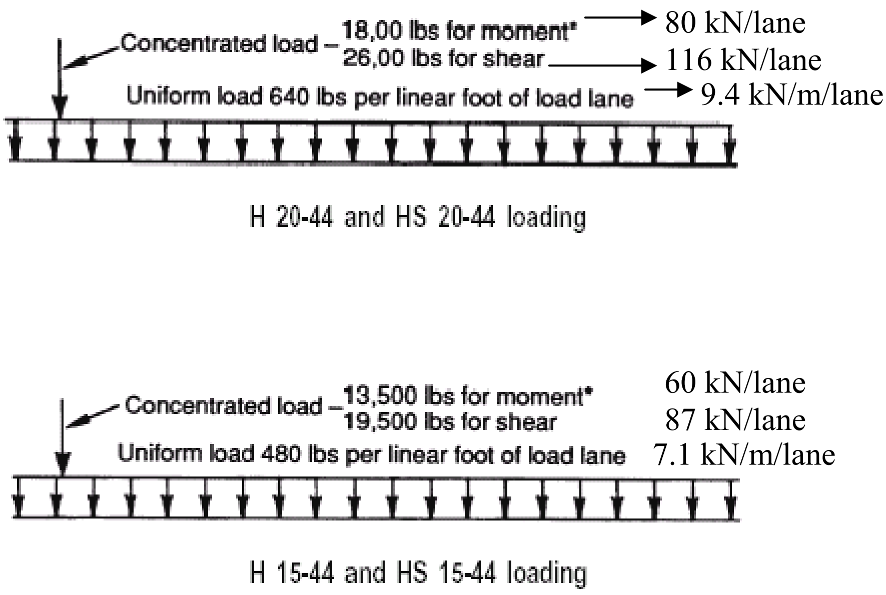


Figure 5. Standard AASHTO lane Loading

For the determination of maximum negative moment in the design of continuous spans, the lane load described above shall be modified by the addition of a second, equal weight concentrated load placed in one other span in the series in such position to produce the maximum effect. For maximum positive moment, only one concentrated load shall be used per lane, combined with as many spans loaded uniformly as are required to produce maximum moment. (Taly, N 1998).

3.2.3. Alternate military (or design tandem) loading.

This loading originated in 1956 as a Federal Highway Administration requirement for bridges on the interstate Highway System, to provide load-carrying-capacity for certain heavy military vehicles. It is applicable to certain bridges in the state highway systems. The alternate bridge loading consists of two axles spaced 4 ft (1.22m) apart with each axle carrying 24 kips (106.76 kN). This load produces slightly higher live-load moments in spans less than 40 ft (12.2m) (Taly, N 1998).

3.3 Application of live load

Standard truck or lane loading are assumed to occupy a loaded width of 10 ft (3.048m), these loads shall be placed in 12 ft (3.65m) wide-design traffic lanes spaced across the entire bridge roadway width in number and position required to produce the maximum stress in the member under consideration. The uniform and concentrated load of a lane loading shall be considered to be uniformly distributed over a 10 ft (3.048m) width on a line normal to the centerline of the lane. In computing stresses, each 10 ft lane loading or single standard truck shall be considered as a unit that can occupy position within its individual traffic lane, so as to produce maximum stress.

For continuous spans, only one standard H or HS truck per lane shall be considered and placed so as to produce maximum positive or negative moments.

The type of loading that will be used shall, be the loading which produces the maximum stress. Where maximum stresses are produced in any member by loading a member of traffic lanes simultaneously, the following percentages of live load stresses shall be in view of improbability of coincident maximum loading:

- One or two lanes 100%
- Three lanes 90%
- Four lanes or more 75%

The reduction in intensity of loads on transverse members such as floor beams shall be determined as in the case of main trusses or girders, using the number of traffic lanes across the width of roadway that must be loaded to produce maximum stresses in the floor beam.

4. Method of Analysis

The most common methods of analyzing isotropic and orthotropic skew plates are grillage and finite elements methods. In this context, isotropic material is referred to material whose properties are not dependent on the direction along which they are measured. In contrast, orthotropic material is defined as a material having elastic properties with considerable variations of strength in two or more directions perpendicular to one another.

4.1. Grillage Analysis

Grillage models became popular in the early 1960's with the advancement of digital computers. As the methodologies for the stiffness analysis (or displacement method) of frames were well known, researchers looked for convenient ways to model continua with frame elements. The grillage model is such a technique. Ideally the element stiffnesses in the grillage model would be such that when the continuum deck is subjected to a series of loads, the displacement of the continuum and the grillage are identical. (Barker, R and Puckett, J, 1998).

Some advocates of the finite element and strip methods are quick to discount the grillage method because it is nonrigorous. But such methods are used to obtain reasonable distribution of internal actions while accounting for equilibrium. Both advocates and critics have valid points and a few of these are listed below:

1) Grillages can be used with any program that has plane grid or space frame capabilities.
2) The results are easily interpreted and free body diagrams of the elements and system as a whole easily check equilibrium.

3) Mostly, all engineers are familiar with the frame analysis .

The disadvantages are several:

1) The method is nonrigorous and does not exactly converge to the exact solution of the mathematical model.
2) Obtaining good solutions requires some experience with the grillage method. The mesh design and refinement can he somewhat of an art form.
3) The assignment of the cross-sectional properties requires some discretion.

4.2. Finite Element Method

The finite element method is one of the most general and powerful contemporary numerical methods. It has the capability to model many different mathematical models and to combine these models as necessary. Like the grillage method, the most common finite element models are based on stiffness (or displacement approach), that is, a system of equilibrium equations is established and solved for the displacements at the degrees of freedom. (Barker, R and Puckett, J, 1998).

The finite element formulation is commonly used in two ways: 2-D and 3-D models. The 2-D model is the simplest and involves fewer degrees of freedom; the girders are modeled with grillage or plane grid elements with three degrees of freedom per node. Examples of these elements are illustrated in Figure 6a and 6b.

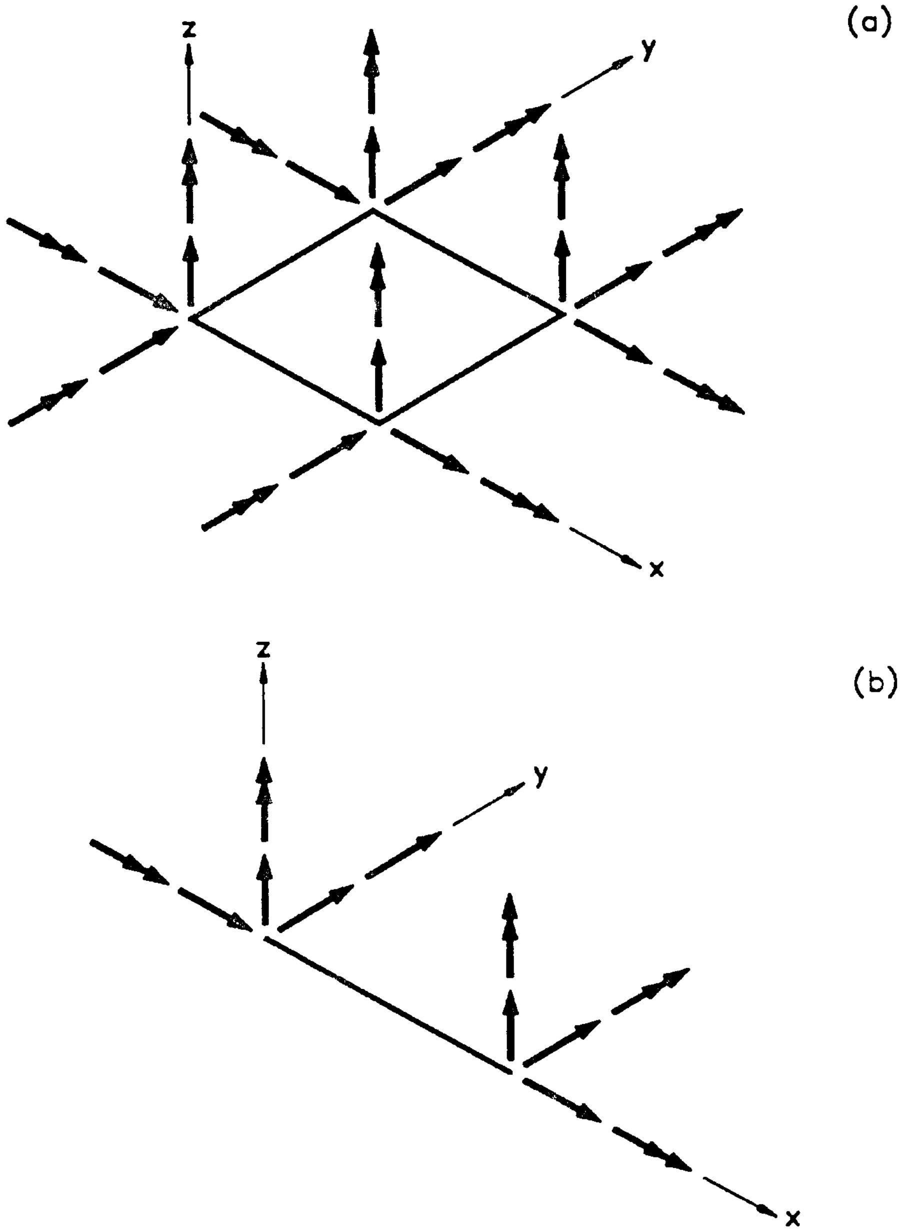


Figure 6. (a) Example of shell element and (b) Example of space frame element

Because there are many different elements with differing number of degrees of freedom and response characteristics, it is difficult to provide general guidance mesh characteristics other than those usually addressed in standard references. It is important to suggest that at least two meshes be studied to obtain some knowledge of the convergence characteristics. If the response changes significantly with refinement, a third (or fourth) mesh should likely be studied.

Because of the importance of maintaining equilibrium, the analytical results should be checked for global equilibrium. It is very easy to mistakenly apply the loads in the wrong direction or in the wrong location. It is strongly suggested that global equilibrium be checked by hand. If the program being used does not have a way to obtain reactions, then perhaps the stiff boundary spring elements can be used at the supports and the element forces are the reactions. If the program does not produce reactions, or they cannot be deduced from the element forces, then the use of another program that does is recommended.

CASE STUDY

1. General

Recent advances in computer methods and numerical analysis techniques had lead to the development of a number of computer programs in the field of structural analysis. Two types of programs in general use, general-purpose program such as SAP, STAAD, STRUDL, and FINITE, and special purpose programs for analysis of specific bridge type, such as GENDEK, CEL-4, LANEL, etc. The software that will be used in this research is SAP2000.

2. SAP2000

2.1. Overview

Bridge Analysis can be used to determine the response of bridge structures due to the weight of vehicle live loads. Considerable power and flexibility is provided for determining the maximum and minimum displacements and forces due to multiple-lane loads on complex structures, such as highway interchanges. The effects of vehicle live loads can be combined with static and dynamic loads, and envelopes of the response can be computed. The bridge to be analyzed is modeled with frame elements representing the superstructure, substructure and other components of interest. Displacements, reactions, spring forces, and frame-element internal forces can be determined due to the influence of vehicle live loads. Other element types (Shell, Plane, Asolid, Solid, and Nllink) may be used; they contribute to the stiffness of the structure, but they are not analyzed for the effect of vehicle load.

Lanes are defined on the superstructure that represent where the live loads can act. These Lanes need not be parallel nor of the same length, so that complex traffic patterns may be considered. The program computes conventional influence lines for all response quantities due to the loading of each lane. These influence lines may be displayed using the SAP2000 graphical interface.

For each maximum or minimum extreme response quantity, the corresponding values for the other components of response can also be computed. In summary, the procedure to perform a Bridge Analysis using the SAP2000 is to:

1) Model the structural behavior of the bridge with frame elements
2) Define traffic lanes describing where the vehicle live loads act
3) Define the different vehicle live loads that may act on the bridge
4) Define vehicle classes (groups) containing one or more vehicles that must be considered interchangeably
5) Define moving load cases that assign vehicle classes to act on the traffic Lanes in various combinations
6) Specify for which joints and frame elements the moving load response is to be calculated

The most extreme (maximum and minimum) displacements, reactions, spring forces, and frame element internal forces are automatically computed for each moving load case defined.

2.2. Roadways and Lanes

The vehicle live loads are considered to act in traffic lanes transversely spaced across the bridge roadway. These lanes are supported by frame elements representing the bridge deck. The number of lanes and their transverse spacing can be chosen to satisfy the appropriate design-code requirements. For simple bridges with a single roadway, the lanes will usually be parallel and evenly spaced, and will run the full length of the bridge structure.

For complex structures, such as interchanges, multiple roadways may be considered. Thus, lanes need not be parallel nor be of the same length. The number of lanes across the roadway may vary along the length to accommodate merges. (SAP2000 manual).

2.3. Roadways

Typically each roadway is modeled with a single string (or chain) of frame elements running along the length of the roadway. These elements should possess section properties representing the full width and depth of the bridge deck. They are modeled as a normal part of the overall structure and are not explicitly identified as being roadway elements.

2.4. Lanes

A traffic lane on a roadway has its length represented by a consecutive set of some or all of the roadway elements. The transverse position of the lane centerline is specified by its eccentricity relative to the roadway elements. Each lane across the roadway width will usually refer to the same set of roadway elements, but will typically have a different eccentricity. The eccentricity for a given lane may also vary along the length.

A lane is thus defined by listing, in sequence, the labels of a chain of frame elements that already exist as part of the structure. Each lane is said to "run" in a particular direction, namely from the first element in the listed sequence to the second element, and so on, to the last element. This direction may be the same or different for different lanes using the same roadway elements, depending on the order in which each lane is defined. It is independent of the direction that traffic travels.

2.5. Influence Lines

SAP2000 automatically computes influence lines for the following response quantities:

1) Frame element internal forces at the output points
2) Joint displacements
3) Reactions
4) Spring forces

For each response quantity in the structure, there is one influence line for each traffic Lane. An influence line can be viewed as a curve of influence values plotted at the load points along a traffic Lane. For a given response quantity at a given location in the structure, the influence value at a load point is the value of that response quantity due to a unit concentrated downward force acting at that load point. The influence line thus shows the influence upon the given response quantity of a unit force moving along the traffic lane. Figure 7 shows some simple examples of influence lines.

Influence lines may exhibit discontinuities (jumps) at the output point when it is located at a load point on the traffic lane. Discontinuities may also occur where the structure itself is not continuous (e.g., expansion joints).

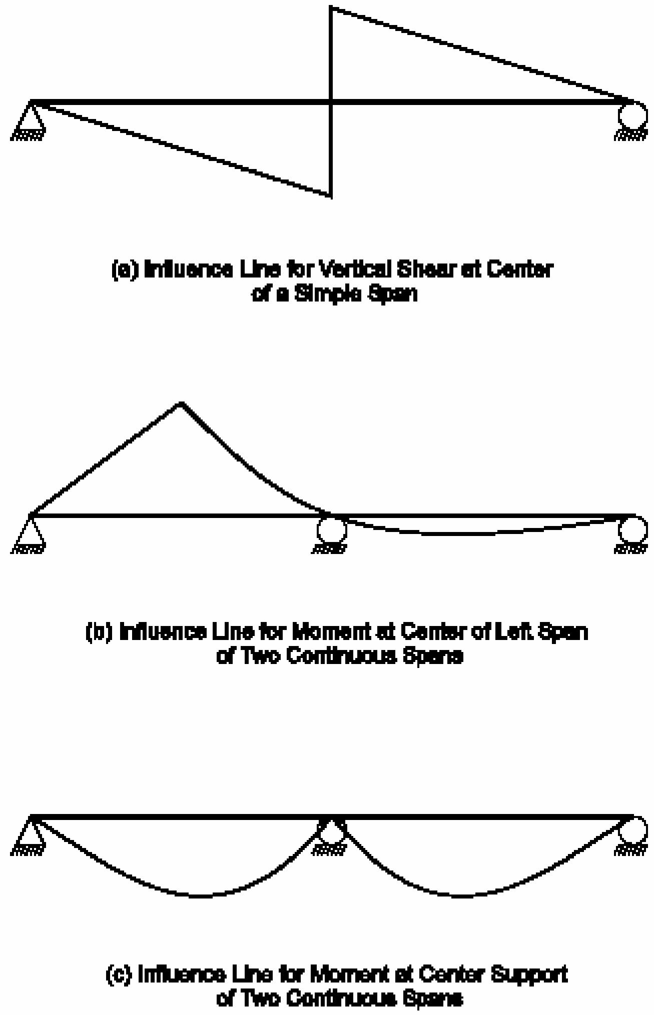
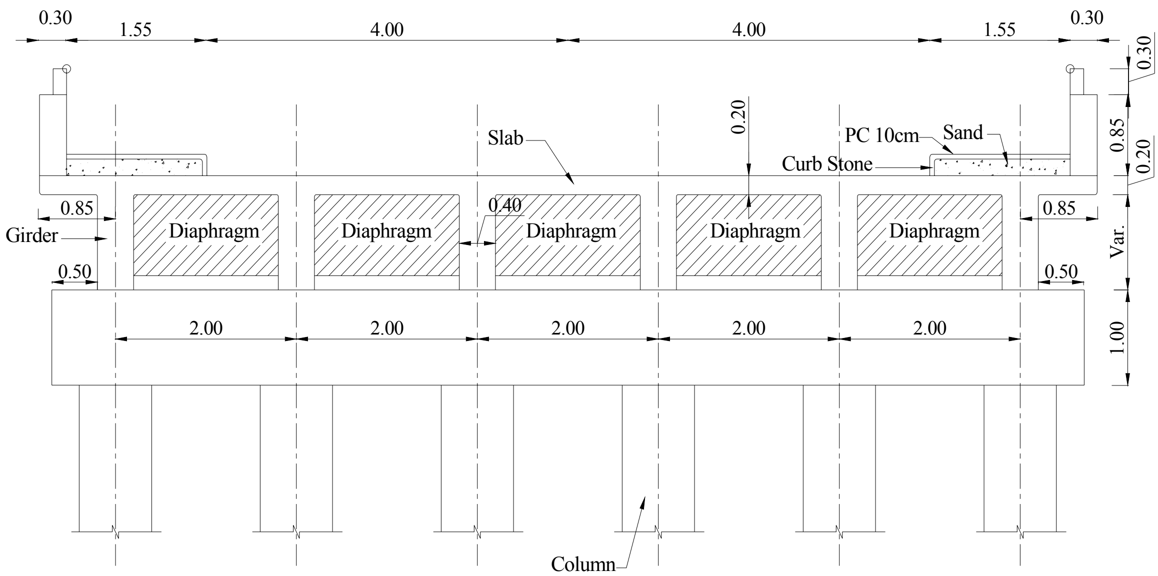


Figure 7. Some simple examples of influence lines.

SAP2000 uses influence lines to compute the response to vehicle live loads. Influence lines are also of interest in their own right for understanding the sensitivity of various response quantities to traffic loads. Influence lines can be displayed using the SAP2000 graphical user interface. They are plotted along the lane elements with the influence values plotted in the vertical direction. A positive influence value due to gravity load is plotted upward. Influence values are linearly interpolated between the known values at the load points. Influence values may also be written to a text file from the graphical interface. Influence lines are available after any analysis for which traffic lanes were defined. It is not necessary to define Vehicles, Classes, Moving Load cases, or response control in order to get influence lines.

3. Structural Model



. Figure 8. Across-section of the bridge at the intermediate bent

Figure 8 shows the geometry of the superstructure of the considered bridge in this study. The bridge was assumed to carry two traffic lanes, with 1.55-m width sidewalks and 0.3-m width parapets at each side. The bridge was taken to be two spans bridge with the same length for both spans.

The deck of this bridge consists of a 0.2-m thick slab, girders, and diaphragms. The girders have a width of 0.4 -m and a variable depth depending upon the span length. The depth of the girder is varies with the span length as given in table 1.

Table 1. Girder Depth

Span Length (m)	Girder Depth (m)
10.0	1.00
12.0	1.00
14.0	1.20
16.0	1.40

The superstructure of the bridge is carried by a 1.0mX1.0m cap beam, which is supported on 4 columns (0.8mX0.8m) at the mid length of the bridge. The bridge deck is supported on abutments at both ends.

The bridge was modeled using SAP2000 with changing the span length as follows:

Span Length: 10, 12, 14, and 16 m.

For each span length the following skew angles were considered:

Skew Angle: 0, 10, 20, 30, 40, 50, and 60 degrees.

For modeling with SAP2000, the girders were denoted as frames, and each girder was divided into two frames, near the pier, and near the abutment. According to this division, each frame has a number; Figure 9 shows the designations of the frames in the SAP2000 model.

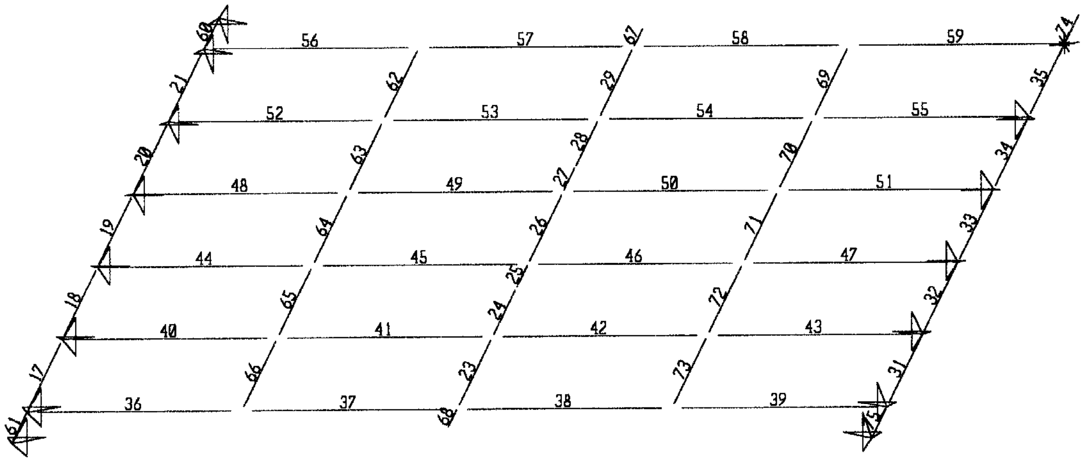


Figure 9. Frame elements designation in the SAP2000 model.

The girders that will be analyzed will be the external girders at the two sides of the bridge. So from the above figure the considered frames will be:

36, 37, 38, 39, 56, 57, 58, and 59.

For studying the effect of span length and skew angle on the vertical reaction at the support, the corners at the obtuse and acute corners will be monitored in the study.

Figure 10 shows the designation of the nodes of the model. From the figure, the considered support reactions are at the nodes: 7, 9, 22, and 24.

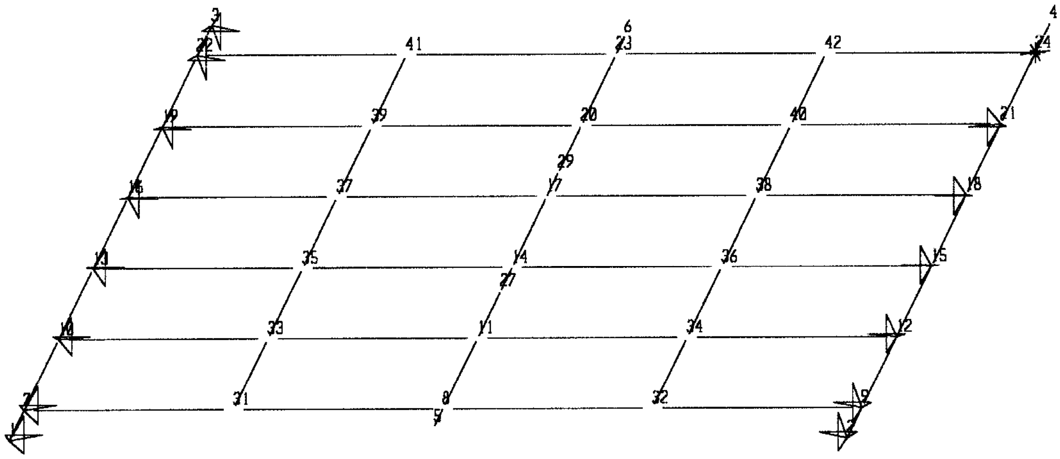


Figure 10. Nodes Designations in the SAP2000 model.

AASHTO lane loading was taken to be the live load for the analysis, and the load factors were applied as recommended by AASHTO. The load of the wearing surface and side walk were also considered and were calculated as recommended by AASHTO specifications.

Asphalt Plank 1 in thick 9 lb/sq.ft = 0.4309 kN/m^2

Sidewalk loading 60 lb/sq.ft = 2.8728 kN/m^2

According to AASHTO specifications for spans length more than 40 ft (12.2m) an intermediate diaphragm should be provided. In this research, the effect of the diaphragm will be studied for bridges with 10-m and 12-m length span. A comparison between the results for changing the skew angle for both span lengths with and without diaphragm will be held.

RESULTS AND DISCUSSION

1.General

The results of the analysis discussed in the previous chapter were summarized in Table 2. Table 2 contains the Max. Positive Moment, Max. Negative Moment, Max. Shear for all spans and skew angles.

Table 2. Max. Internal Forces for Each span length.

	Skew (degree)	Span (m)			
		10	12	14	16
M^{+ve} Max. (kN.m)	0	418	451	657	932
	10	425	458	664	982
	20	430	467	695	988
	30	434	485	725	1025
	40	436	504	748	1055
	50	438	518	766	1078
	60	438	528	779	1094
M^{-ve} Max. (kN.m)	0	497	614	933	1363
	10	498	620	938	1391
	20	500	629	945	1381
	30	500	639	955	1394
	40	499	648	967	1407
	50	498	656	977	1419
	60	497	663	986	1427
Max. Shear (kN)	0	387	408	502	605
	10	387	411	503	606
	20	387	411	503	606
	30	387	412	504	607
	40	387	413	505	609
	50	386	414	505	611
	60	386	414	506	613
Max. Torsion (kN.m)	0	103	93	99	104
	10	102	92	98	103
	20	102	92	98	102
	30	102	93	98	102
	40	102	93	98	102
	50	101	93	98	102
	60	102	93	98	102

Tables (3,4,5and 6) contain the Max. Positive Moment, Max. Negative Moment, Max. Shear force and Max. Torsion for all the internal and external side of edge girder,

Table 3. Results for External and Internal parts of the edge girder (Span = 10m)

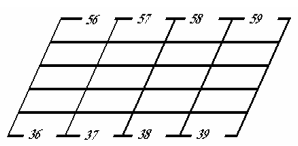
	Skew (degree)	Span =10m			
		Element No.			
		36+59	**37+58**	**38+57**	**39+56**
M^{+ve} Max. (kN.m)	0	418	418	418	418
	10	416	416	425	425
	20	414	414	430	430
	30	411	411	434	434
	40	407	407	436	436
	50	403	403	438	438
	60	400	400	438	438
M^{-ve} Max. (kN.m)	0	149	497	497	149
	10	169	498	495	149
	20	191	500	493	151
	30	213	500	493	155
	40	233	499	492	161
	50	252	498	492	168
	60	268	497	491	174
Max. Shear (kN)	0	320	387	387	320
	10	322	386	387	318
	20	325	386	387	317
	30	328	385	387	317
	40	330	384	387	317
	50	332	384	386	317
	60	334	383	386	317
Max. Torsion (kN.m)	0	95	101	95	103
	10	93	102	95	101
	20	90	102	94	99
	30	87	102	94	98
	40	84	102	93	97
	50	83	101	92	96
	60	81	102	90	96

Table 4. Results for External and Internal parts of the edge girder (Span = 12m)

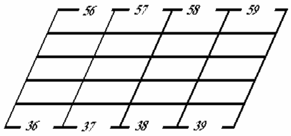
	Skew (degree)	Span = 12m Element No. 36+59	37+58	38+57	39+56
M^{+ve} Max. (kN.m)	0	401	402	402	401
	10	433	390	418	370
	20	461	381	434	341
	30	485	375	448	315
	40	504	372	461	293
	50	518	372	471	277
	60	528	373	480	266
M^{-ve} Max. (kN.m)	0	36	582	582	36
	10	59	579	587	32
	20	87	579	592	33
	30	115	581	596	37
	40	144	583	600	44
	50	170	584	602	52
	60	192	585	603	61
Max. Shear (kN)	0	319	408	408	319
	10	327	407	409	313
	20	334	407	411	308
	30	341	408	412	303
	40	347	408	413	301
	50	352	409	414	299
	60	356	410	414	298
Max. Torsion (kN.m)	0	84	92	85	93
	10	82	92	85	91
	20	80	92	85	90
	30	77	93	84	88
	40	75	93	83	88
	50	73	93	82	87
	60	72	93	81	87

Table 5. Results for External and Internal parts of the edge girder (Span = 14m)

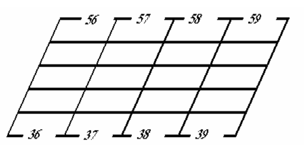
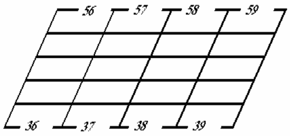
	Skew (degree)	Span =14m			
		Element No.			
		36+59	**37+58**	**38+57**	**39+56**
M^{+ve} Max. (kN.m)	0	619	621	621	619
	10	659	607	638	579
	20	695	595	656	541
	30	725	587	672	507
	40	748	583	686	477
	50	766	582	697	453
	60	779	583	706	436
M^{-ve} Max. (kN.m)	0	47	912	912	47
	10	73	909	918	40
	20	102	910	923	37
	30	132	913	927	39
	40	166	916	930	44
	50	196	919	933	51
	60	221	922	935	61
Max. Shear (kN)	0	381	502	502	381
	10	389	501	503	375
	20	396	502	503	369
	30	403	503	504	364
	40	409	504	505	361
	50	414	505	505	358
	60	418	506	505	357
Max. Torsion (kN.m)	0	93	98	95	99
	10	91	98	95	97
	20	88	98	94	95
	30	85	98	93	93
	40	83	98	93	92
	50	80	98	92	91
	60	79	98	91	90

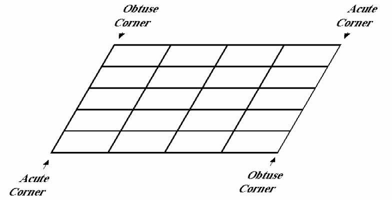
Table 6. Results for External and Internal parts of the edge girder (Span = 16m)



	Skew (degree)	Span =16m			
		Element No.			
		36+59	**37+58**	**38+57**	**39+56**
M^{+ve} Max. (kN.m)	0	897	900	900	897
	10	945	886	917	849
	20	988	875	936	804
	30	1025	867	954	762
	40	1055	863	969	725
	50	1078	862	981	695
	60	1094	862	991	672
M^{-ve} Max. (kN.m)	0	62	1350	1350	62
	10	90	1348	1355	51
	20	121	1349	1361	47
	30	155	1353	1366	46
	40	190	1358	1369	49
	50	224	1363	1372	55
	60	252	1367	1374	64
Max. Shear (kN)	0	448	605	605	448
	10	455	605	605	441
	20	463	606	606	435
	30	470	607	606	430
	40	476	609	606	426
	50	481	611	607	423
	60	485	613	607	422
Max. Torsion (kN.m)	0	100	102	102	104
	10	98	102	103	101
	20	94	102	101	99
	30	91	102	101	97
	40	88	102	100	95
	50	86	102	99	94
	60	84	102	99	93

Table 7 shows the Max. Reactions at the obtuse and acute corners of the model for all spans and for all skew angles.

Table 7. Max. Reaction at Obtuse and Acute corners



	Skew (degree)	Span (m)			
		10	12	14	16
Max. Reaction (kN) [Obtuse Corner]	0	653	654	742	831
	10	664	672	753	845
	20	676	678	767	856
	30	688	691	781	873
	40	697	703	795	889
	50	705	712	807	895
	60	710	719	815	912
Max. Reaction (kN) [Acute Corner]	0	653	654	742	831
	10	644	652	736	829
	20	639	645	736	828
	30	637	646	739	834
	40	636	649	746	843
	50	638	654	753	863
	60	639	659	761	864

The results above were graphically drown on charts to find the relation between each type of forces and the length of the spans of the bridge and the skew angles for the same span.

2.Effect of skew angles and span length on Max. Positive Moment.

Figures 11-14 show the relation between Max. Positive Moment and the skew angles for each span (10,12,14, and 16m).

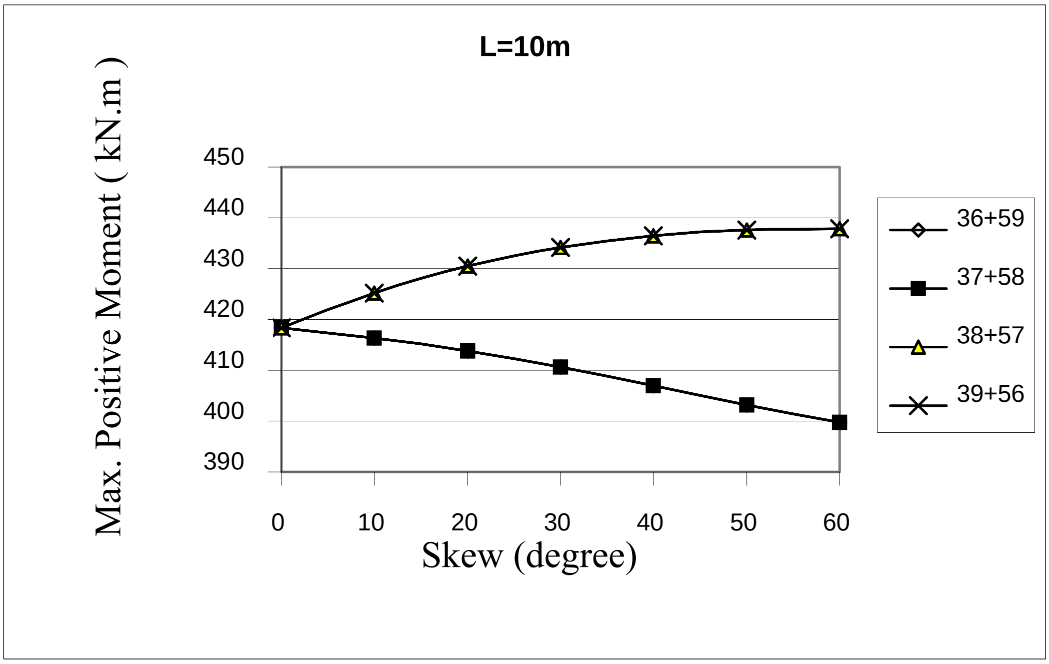


Figure 11. The relation between Max. Positive Moment of the internal and external edge girders and the skew angles for (span =10).

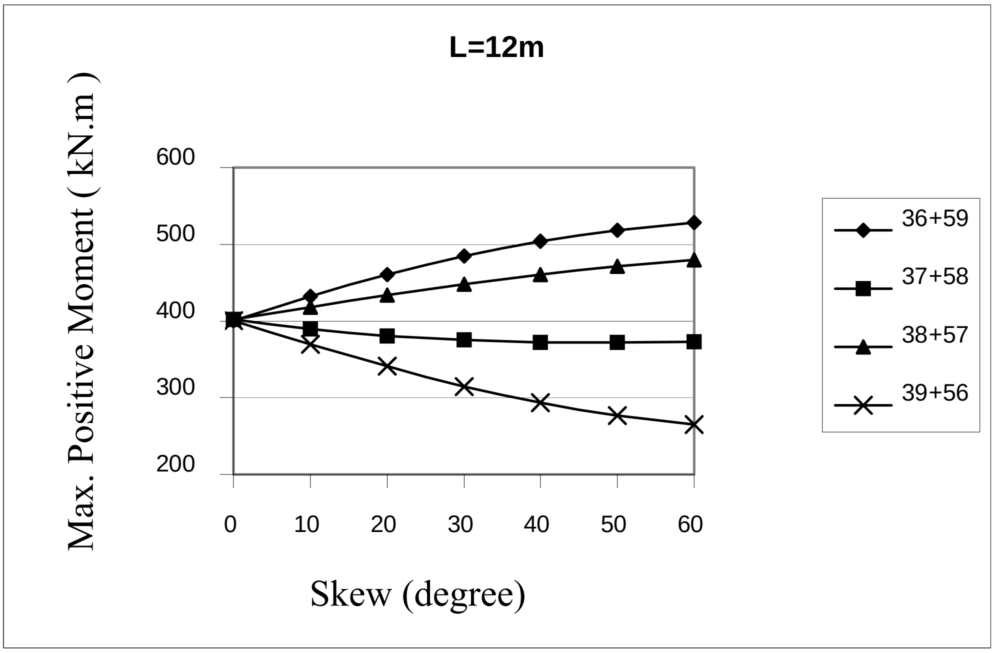


Figure 12. The relation between Max. Positive Moment of the internal and external edge girders and the skew angles for (span =12).

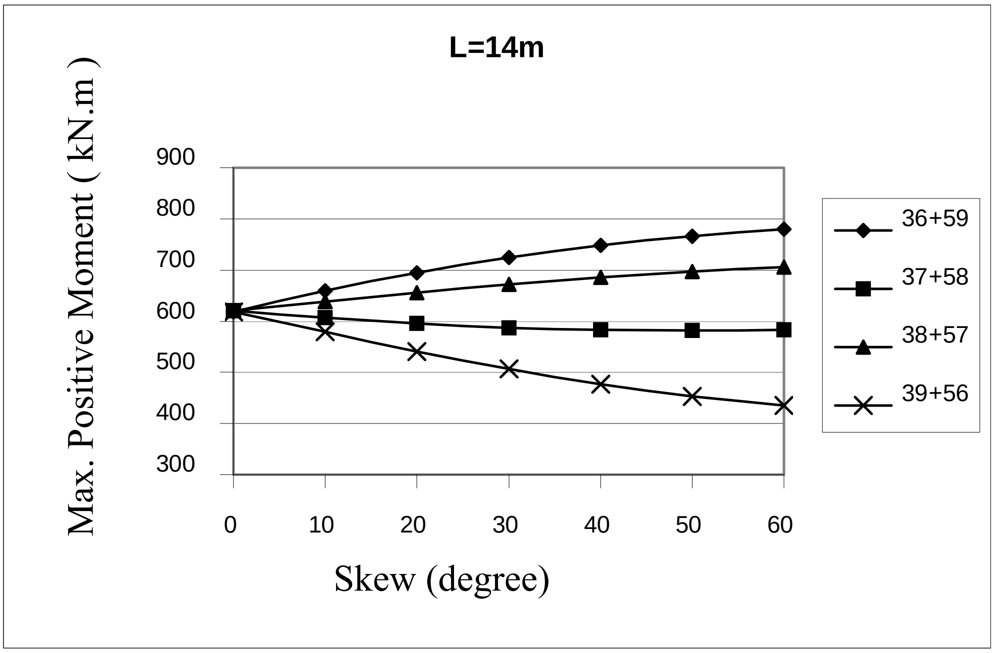


Figure 13. The relation between Max. Positive Moment of the internal and external edge girders and the skew angles for (span =14).

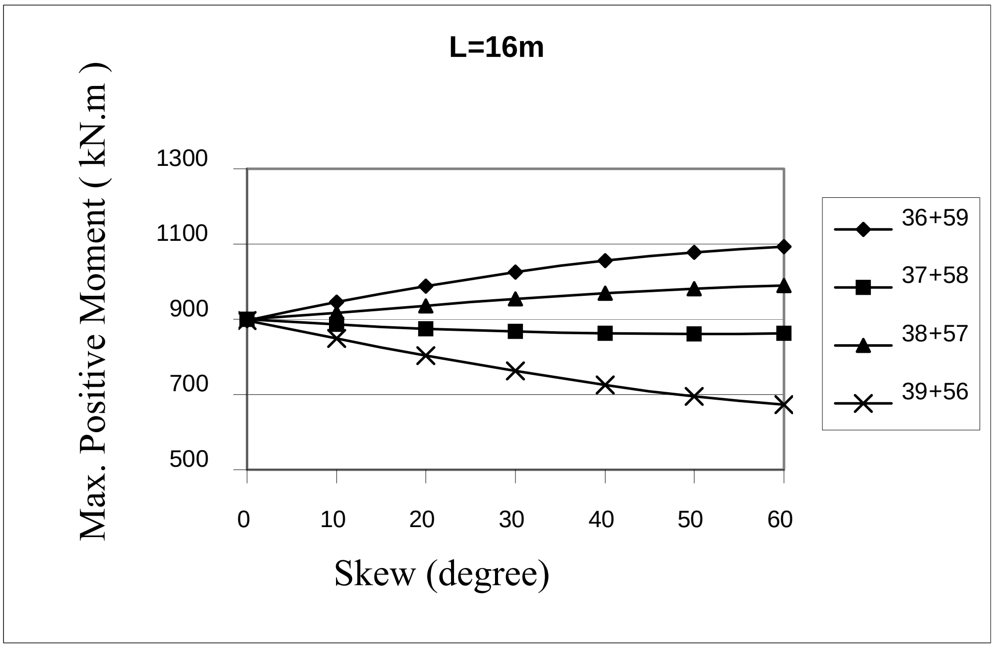


Figure 14. The relation between Max. Positive Moment of the internal and external edge girders and the skew angles for (span =16).

Figure 11 shows the effect of skew angles on the Max. Positive Moment for the bridge when the span length equals to 10m. From this figure, it can be noticed that the curves of 36 and 37 (the internal) parts of the girder are the same and the curves of 38 and 39 parts are also the same. This is because the Max. Positive Moment for the two parts of girders is the same and this means that the Max. Moment is exactly in the middle of the girder. All four curves starts from the same moment value (418 kN.m) that's because there is no skewness in the bridge. It is also clear that increasing the skew angle will increase the positive moment of the parts 38 and 39 and decrease it for 36 and 37. This mean that increasing the skew angle will be of more effect on the part of the girder that is near the obtuse angle and lead to increasing the reinforcement for resisting the moment.

Figure 12 shows the same relation but for a bridge with a span length of 12m. For skew angle of zero degree, the Max. Moment in the four curves was the same due to the symmetry in the bridge but when increasing the skewness, the positive moment for parts 36 and 38 increased, and decreased, for the parts 37 and 39. this behavior can be explained by noticing that the Max. Moment drawn in the figures was for the Max. Positive Moment in each part of girder. the decreasing of moment at 37 and decreasing at 36 means that the Max. Moment of the girder moves away from the middle of the girder.. The same behavior appears in the parts 38 and 39.The same behavior appears in the Figure 13 for span of 14m and Figure 14 for span of 16m

Figure 15 shows the relation between the Max. Max. Positive Moment in the edge girder and the skew angles for all spans.

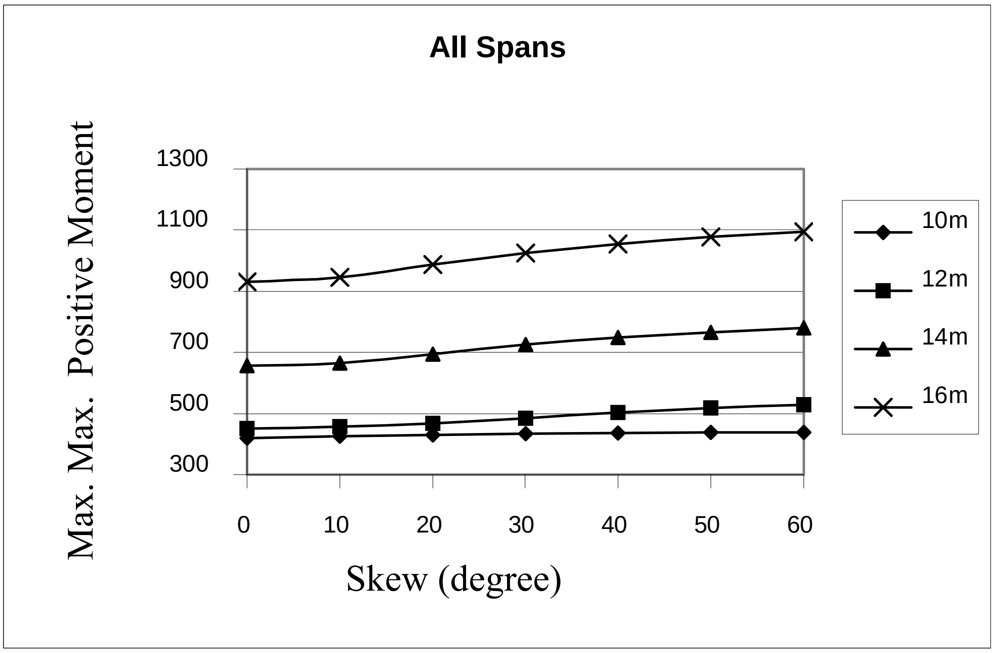


Figure 15. The relation between Max. Max. Positive Moment at all girders in the bridge and the skew angles for all spans. (10,12,14, and 16m).

From the above figure, it can be noticed that increasing the skew angle will increase the Max Positive Moment in the edge girder. This increase will differ according to the span length. For span lengths of 10 m, the maximum increase in the positive moment is about (4%)from the moment at 0^o skew to 60^o skew. for spans of 12m, this increase is about (14%), for 14m it is (15.7%), and for 16m is (15%). Thus it can be concluded that the effect of skewness is higher in long spans than in short spans and produce higher Max Positive Moment for a higher skew angle.

3. Effect of skew angles and span length on Max. Negative Moment

Figures 16-19 show the relation between Max. Negative Moment of the internal and external edge girders and the skew angles for each span length

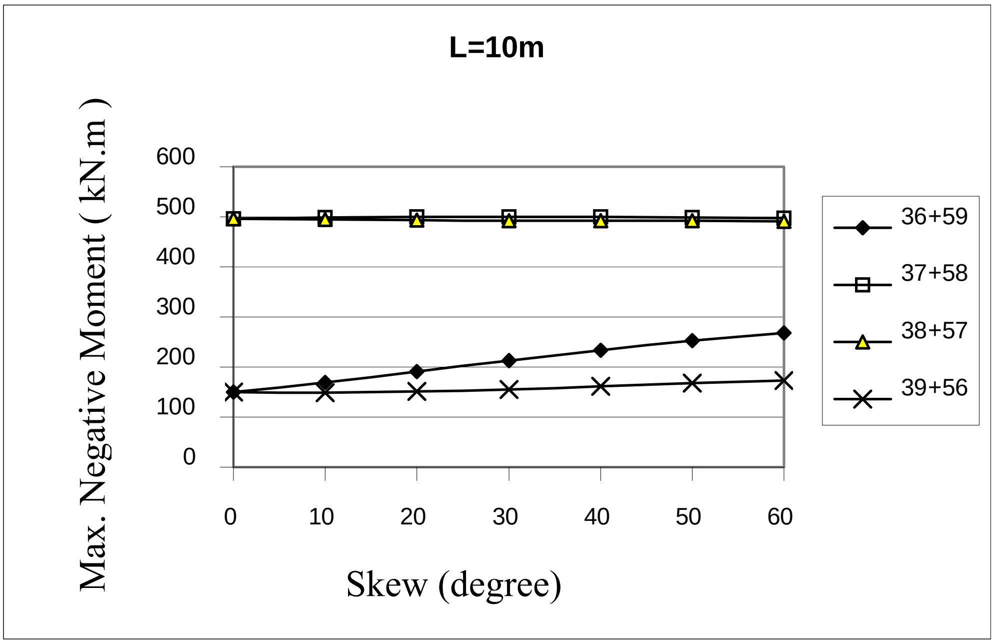


Figure 16. The relation between Max. Negative Moment of the internal and external edge girders and the skew angles for (span =10m)

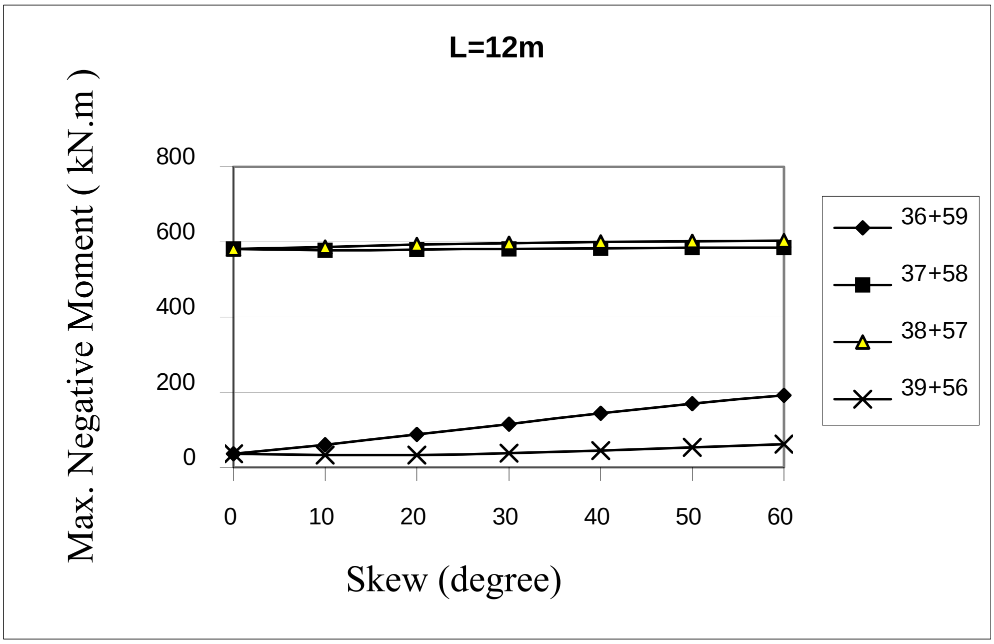


Figure 17. The relation between Max. Negative Moment of the internal and external edge girders and the skew angles for (span =12m)

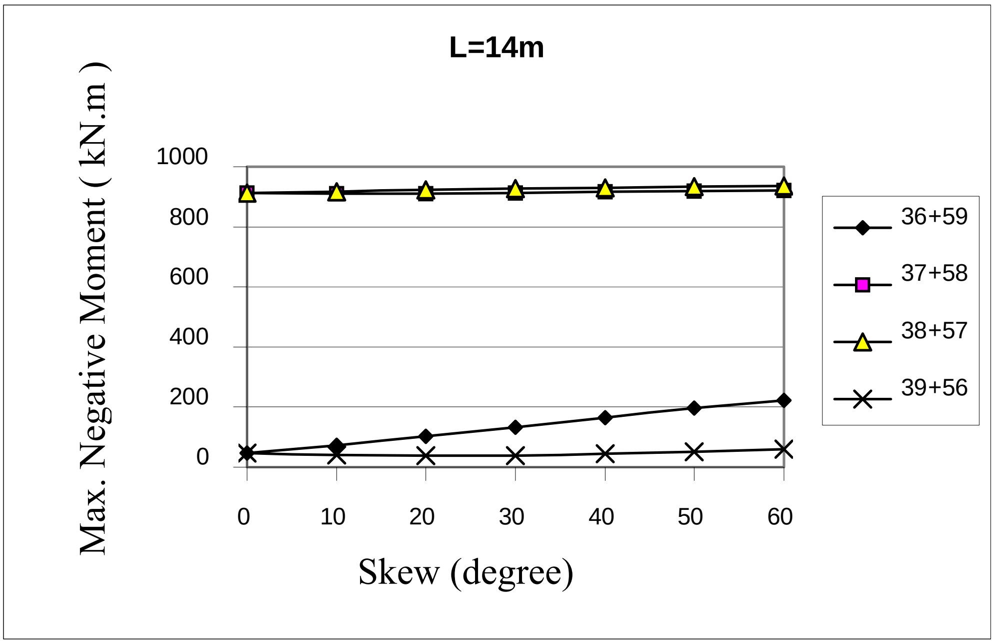


Figure 18. The relation between Max. Negative Moment of the internal and external edge girders and the skew angles for (span =14m)

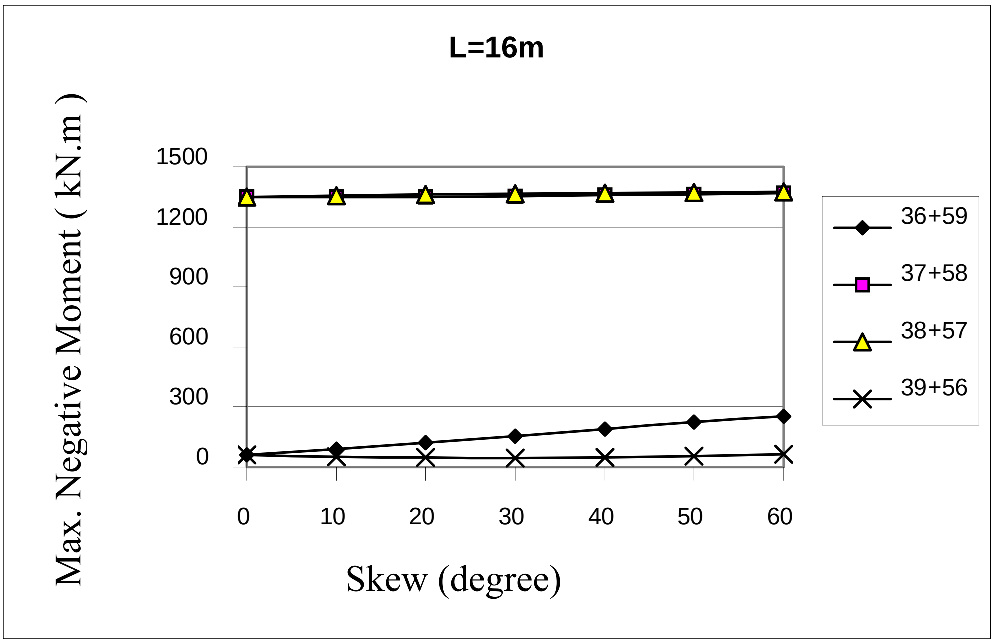


Figure 19. The relation between Max. Negative Moment of the internal and external edge girders and the skew angles for (span =16m)

In all figures, it can be noted that the effect of skewness is very small on the value of the Max. Negative Moment for all the span lengths. It can be also noted that for skew angle equal to zero the value of negative moment the same for the parts 37 and 38, which are near the cap beam, increasing the skew angle will increase the value of the

Max. Negative Moment but in a very percentage. The max increase is about 1.2% of the moment at zero skew angle at the obtuse corner. This means that increasing the skew angle increase the fixity behavior at these parts and produces more negative moment at the support. At the acute corner there is no significant effect of increasing the angle on the value of Max. Negative Moment.

Figure 20 shows the relation between Max. Negative Moment on the edge girders in the bridge with the skew angles for each span length

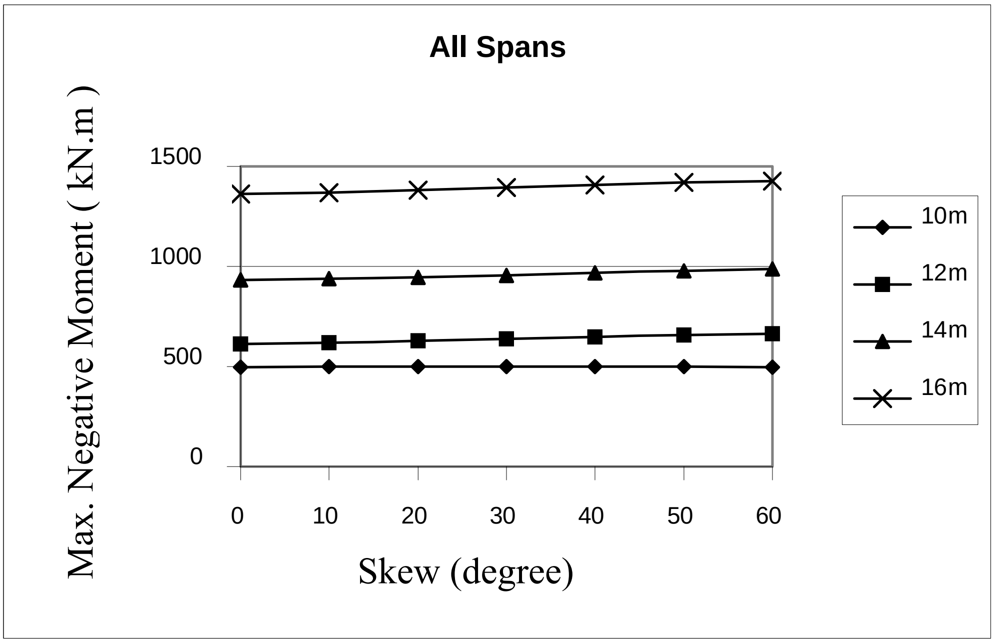


Figure 20. The relation between Max. Max. Negative Moment on all girders in the bridge with the skew angles for each span length (10,12,14, and 16m)

From Figure 20 the effect of skew angles on the Negative moment was very small, the max effect obtained was for span equal to 12 m (7%), for span 14 and 16 m the different was about (4.5 %) and for span of 10m the effect was negligible on the negative moment.

4. Effect of skew angles and span length on Max. Shear

Figures 21-24 show the relation between the Max. Shear of the internal and external edge girders and the skew angles for each span length.

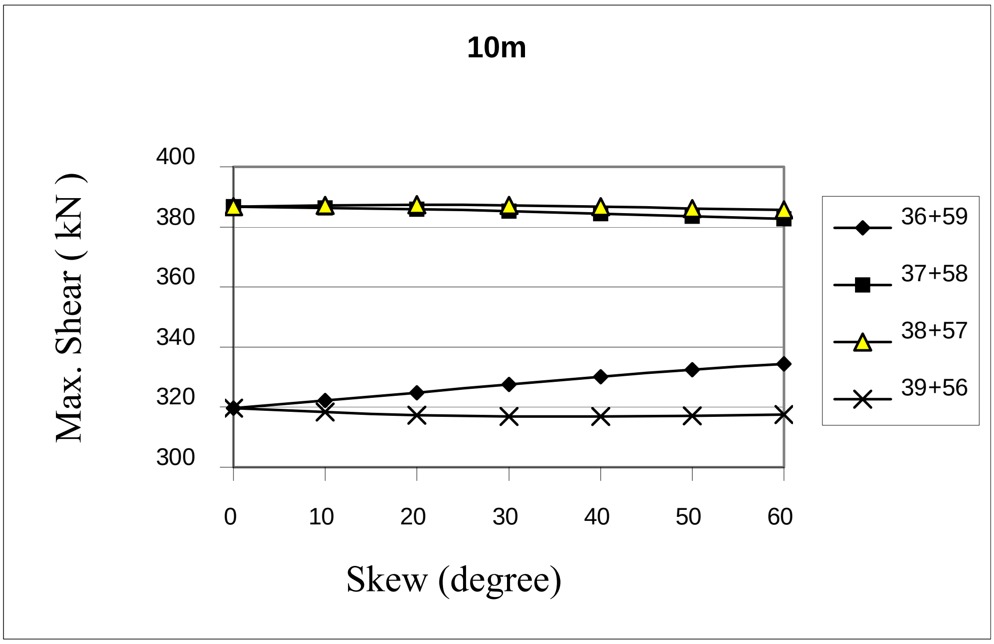


Figure 21. The relation between the Max. Shear of the internal and external edge girders and the skew angles for (span =10m).

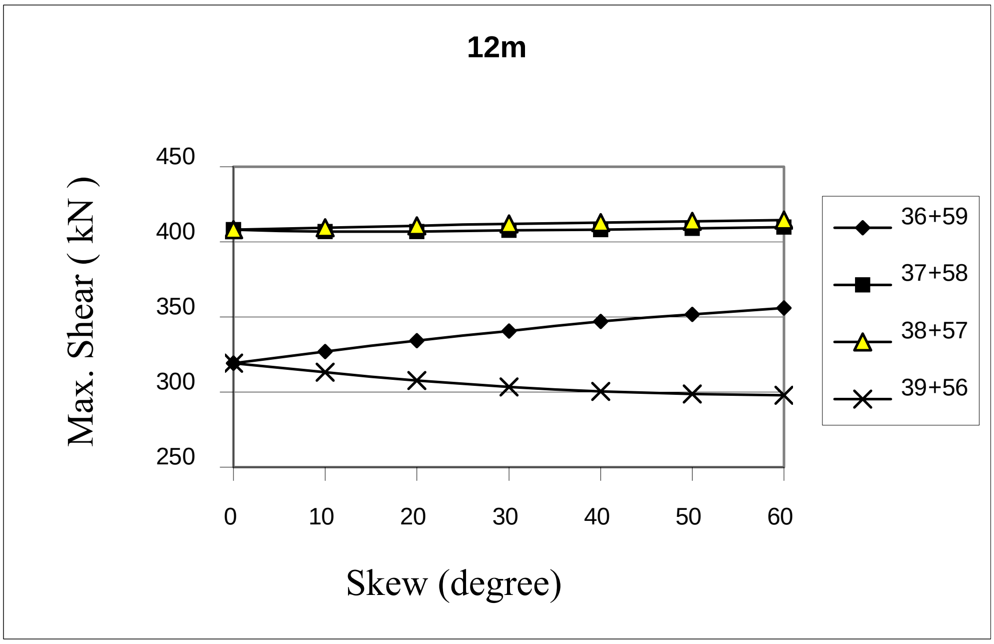


Figure 22. The relation between the Max. Shear of the internal and external edge girders and the skew angles for (span =12m).

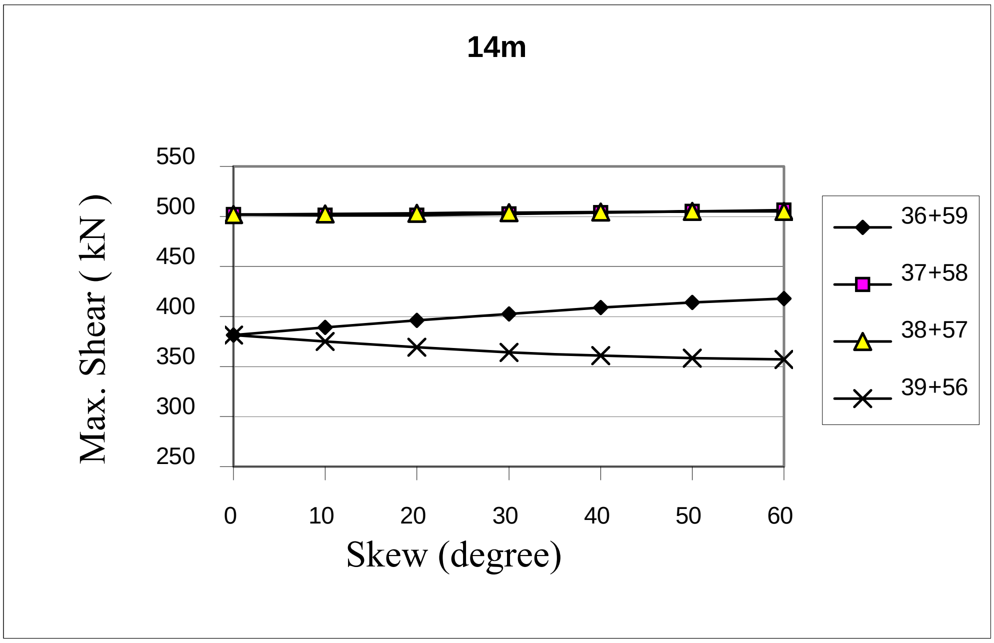


Figure 23. The relation between the Max. Shear of the internal and external edge girders and the skew angles for (span =14m).

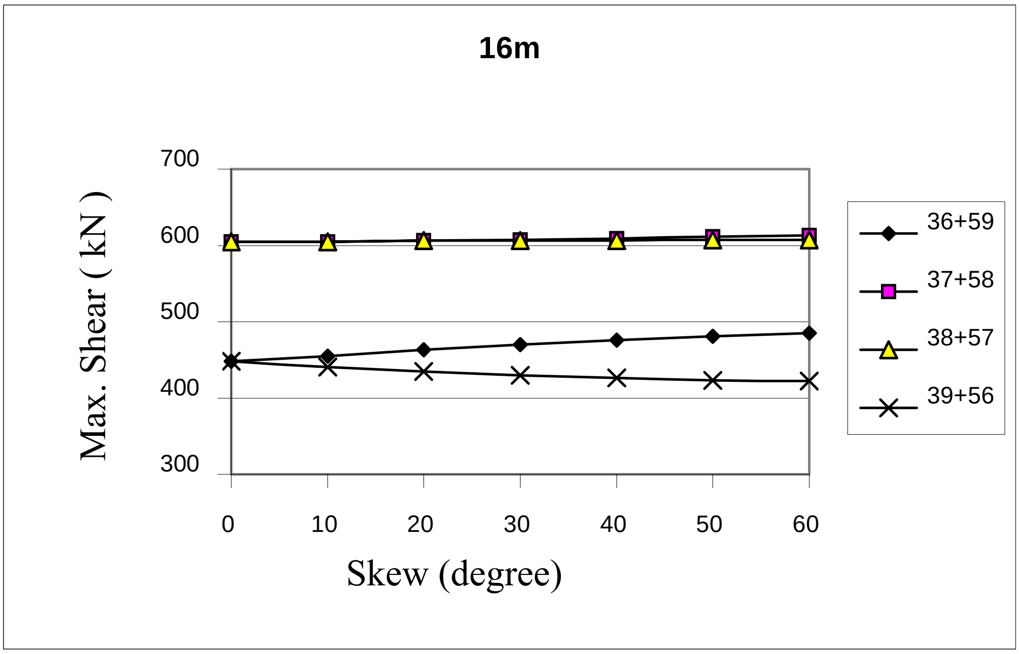


Figure 24. The relation between the Max. Shear of the internal and external edge girders and the skew angles for (span =16m).

Figures 21 through 24 show that the elements number 37 and 38 have approximately the same shear and the effect of the skew angles is almost negligible. The Max. increase in the shear force is about 1.15 % in the 16-m span length. The external elements number 36 and 39 have some different behavior. The two members have the same shear force at the zero degree angle, then the shear force near the acute angle, element number 36,increases with increasing the skew angle, and decreases for the parts near the obtuse corner angle, element number 39. This can be explained by the same reason of increasing the negative moment at the acute corner, because of increasing the fixity due to the increase of the skew angle, the negative moment increase and this produce more shear at the support.

The relation between the Max. Shear on the edge bridge girder and the skew angle for each span is shown in Figure 25.

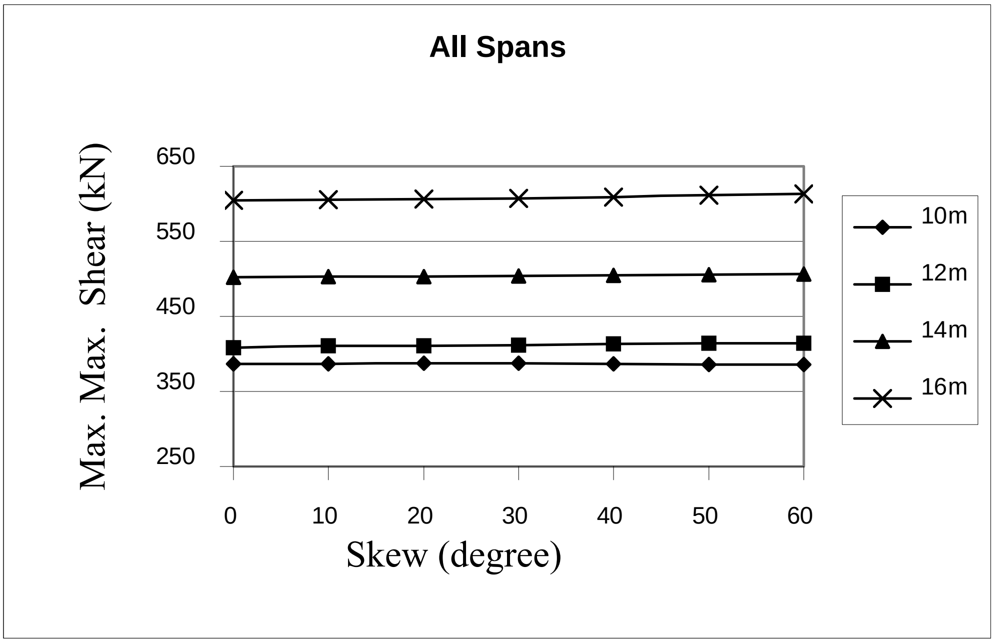


Figure 25. The relation between the Max. Max. Shear on all girders in the bridge with the skew angles for each span length (10,12,14, and 16m)

From the above figure, it can be noted that the effect of the skew angle on the Max. Shear is nearly negligible. The maximum increase in the shear force is about 1.5%.

5. Effect of skew angles and span length on Torsion

Figures 26 through 29 show the relation between Torsion and the skew angles for each span length (10,12,14,and 16m).

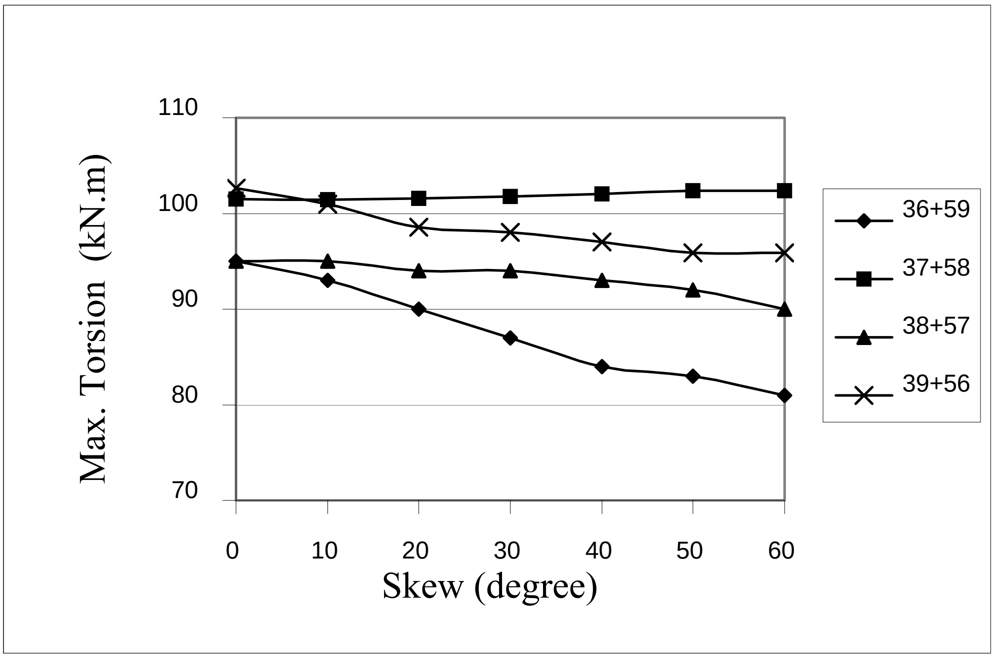


Figure 26. The relation between the Max. Torsion of the internal and external edge girders and skew angles for (Span =10m).

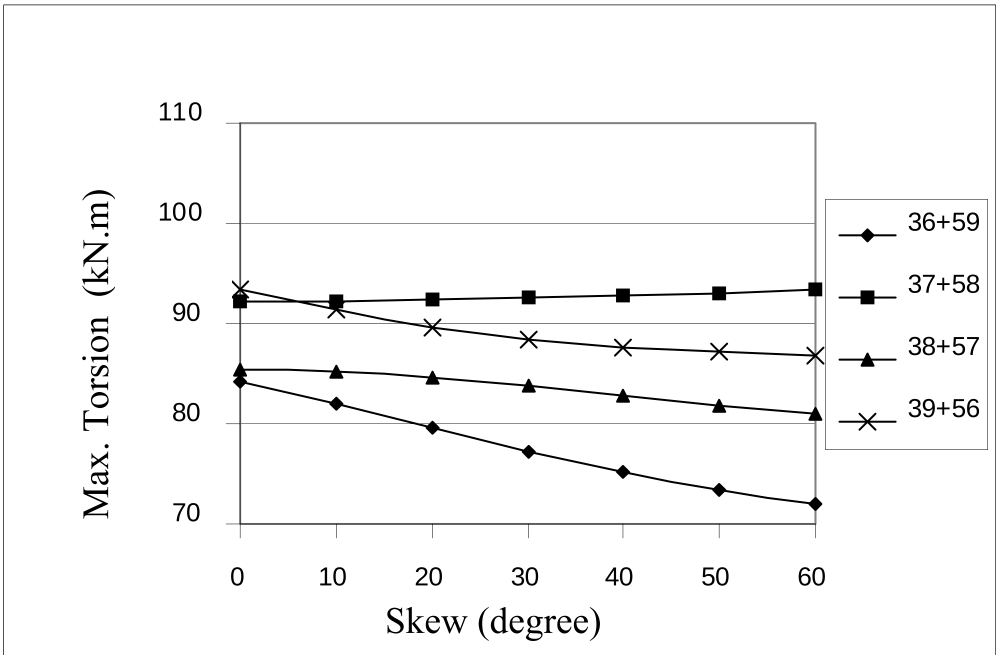


Figure 27. The relation between the Max. Torsion of the internal and external edge girders and skew angles for (Span =12m).

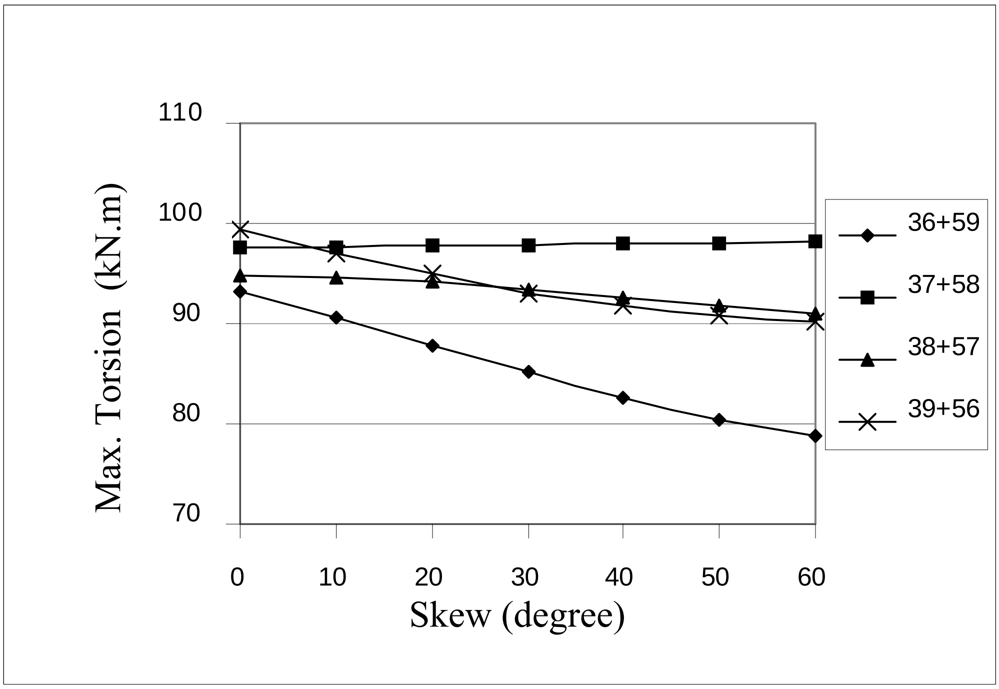


Figure 28. The relation between the Max. Torsion of the internal and external edge girders and skew angles for (Span =14m).

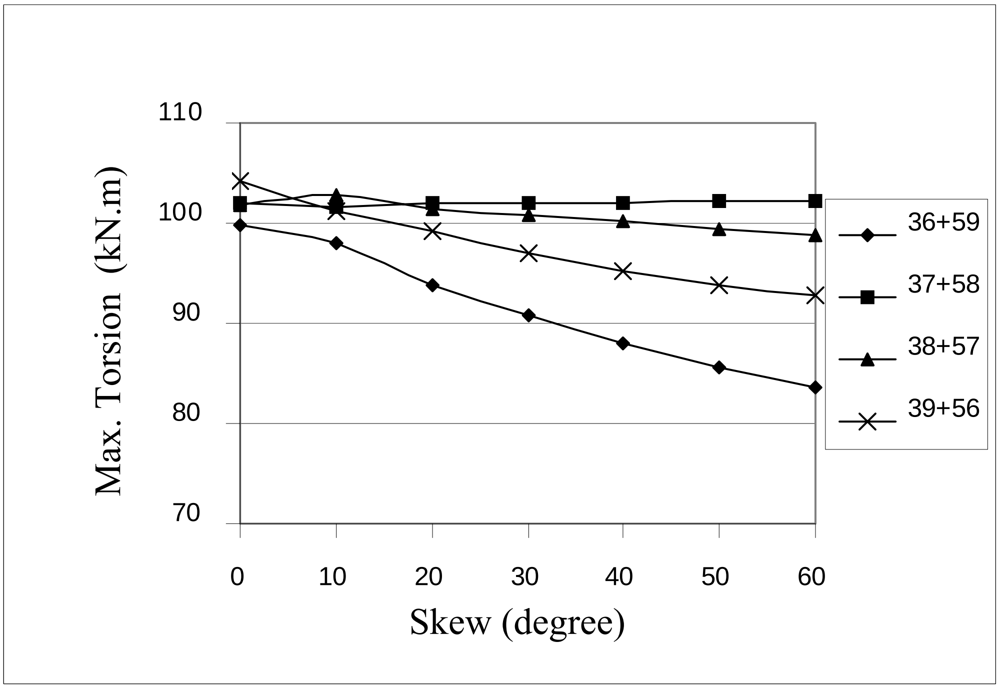


Figure 29. The relation between the Max. Torsion of the internal and external edge girders and skew angles for (Span =16m).

From Figure 26 1t can be shown that the effect of skew angle on Torsion is negligible on the edge girder parts (37 and 58), but for the other parts increasing the skewed will decrease the Torsion force resulting in the girder. From the figure, Torsion on girder part near the pier at the obtuse corner is higher than other parts. Also the effect of increasing skew angle is more significant on girder parts near the acute angle at the abutment. And decrease the value of Torsion for about (14%) for increasing skew angle from 0^o to 60^o.

Figures 27,28,and 29 show the same behavior for spans 12,14,and 16m.

Figure 30 shows the relation between Max. Torsion on girders and the skew angles for all spans.

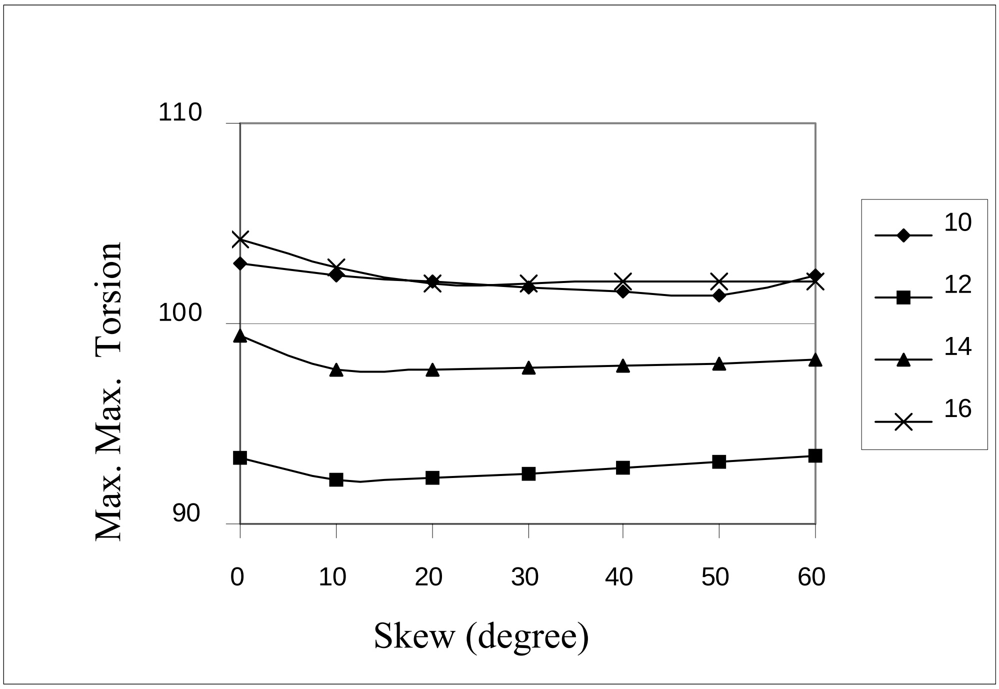


Figure 30. The relation between the Max. Max. Torsion on all girders in the bridge and skew angles for all spans (10,12,14,and16m).

From this figure it can be noted that the effect of increasing skew angle on the Max. Torsion on girders is insignificant.

The max percent of the effect was about (4%) when increasing skew angle from 0^o to 60^o.

It can be noted also that span 10 m gave higher values of Torsion than 12m and

14m and the values were very close to the values of Torsion for span 16m, this can be explained by absence of diaphragms in 10m span which gave a higher stiffness for the girder.

6. Effect of skew angles and span length on the reaction at the acute and obtuse corners

The relation between the vertical reactions on the abutment and the skew angle for each span was drawn according to the variation in the skew angles. Figure 31 shows the relation between the support reaction and the skew angle for the obtuse corner and Figure 32 shows this relation at the acute corner.

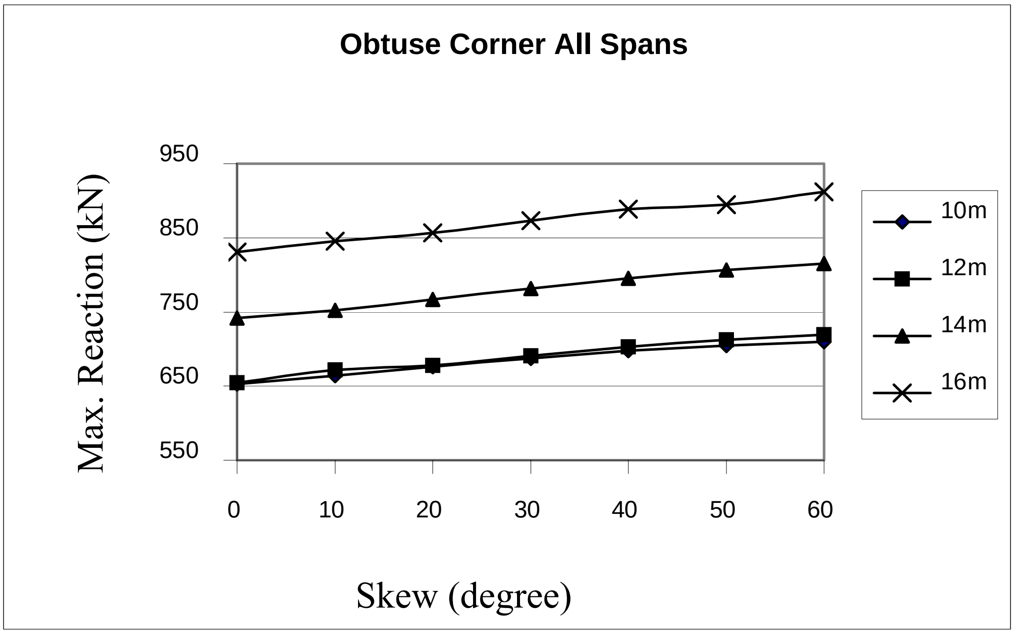


Figure 31. The relation between the Max. Reaction at the obtuse corner and the skew angles for each span (10,12,14, and 16m)

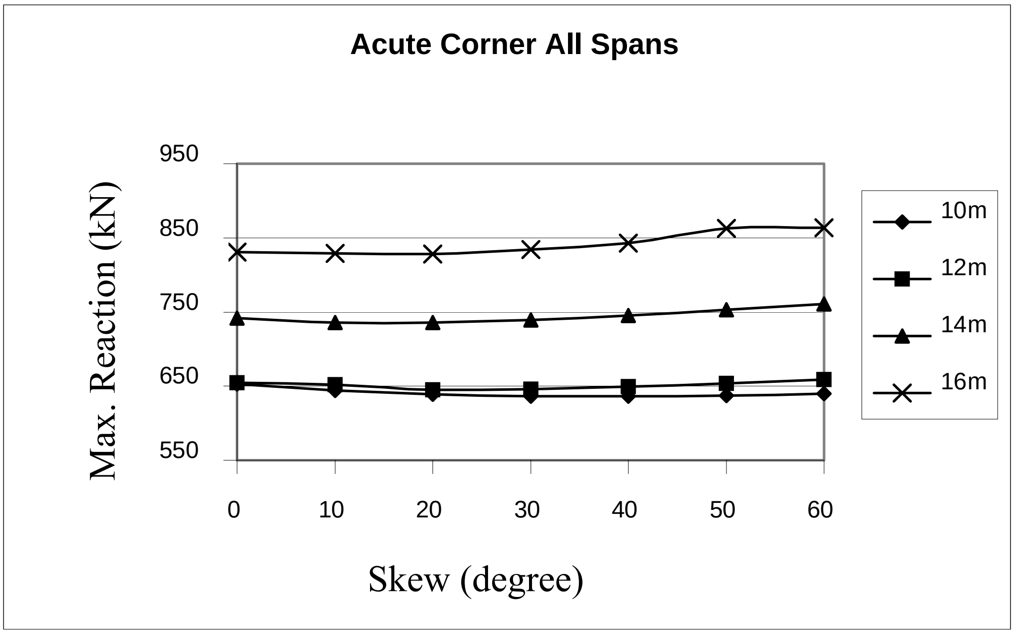


Figure 32. The relation between the Max. Reaction at the acute corner and the skew angle for each span (10,12,14, and 16m)

By comparing the two figures, it can be noticed that for the obtuse corner, the vertical reaction increases when increasing the skew angle form zero degree to 60 degrees for all the spans studied about (8.5%). For the acute corner, the behavior was different, increasing the skew angle will cause reduction in the vertical reaction at the support, and then it will increase with a different rate from a length span to another .For a span length of 10m's, the increase will start from 50 degrees, but for 12, 14, and 16 m it starts from 30 degrees. The maximum increase in the support reaction is about 3.5%.

7. Effect of diaphragm on the results for spans 10m and 12m

Tables 8 through 10 show the results of spans 10m and 12m with and without intermediate diaphragms at the mid span.

Table 8. Results for max. Forces for spans 10 and 12m with and without intermediate diaphragms at the mid span.

	Skew (Degree)	With a Diaphragm		Without a Diaphragm	
		10m	**12m**	**10m**	**12m**
M+ve Max. (kN.m)	0	340	451	418	577
	10	348	458	425	581
	20	359	467	430	585
	30	379	485	434	586
	40	385	504	436	587
	50	397	518	438	586
	60	407	528	438	584
M-ve Max. (kN.m)	0	450	614	497	706
	10	456	620	498	708
	20	462	629	500	708
	30	471	639	500	708
	40	475	648	499	706
	50	482	656	498	704
	60	488	663	497	701
Max. Shear (kN)	0	358	408	387	433
	10	361	411	387	433
	20	363	411	387	433
	30	373	412	387	432
	40	368	413	387	431
	50	370	414	386	430
	60	371	414	386	430

Table 9. Results for External and Internal edge girder (Span = 10m) with and without intermediate diaphragms at the mid span.

	Skew (degree)	With a Diaphragm				Without a Diaphragm			
		36+59	37+58	38+57	39+56	36+59	37+58	38+57	39+56
M^{+ve} Max. (kN.m)	0	290	291	291	290	418	418	418	418
	10	315	279	308	266	416	416	425	425
	20	338	270	325	245	414	414	430	430
	30	357	264	341	226	411	411	434	434
	40	373	261	354	211	407	407	436	436
	50	384	260	365	201	403	403	438	438
	60	392	260	375	195	400	400	438	438
M^{-ve} Max. (kN.m)	0	20	414	414	20	149	497	497	149
	10	42	413	416	17	169	498	495	149
	20	69	415	419	18	191	500	493	151
	30	97	419	424	24	213	500	493	155
	40	124	422	428	31	233	499	492	161
	50	147	426	430	39	252	498	492	168
	60	167	428	432	46	268	497	491	174
Shear Max. (kN)	0	281	358	358	281	320	387	387	320
	10	289	355	361	274	322	386	387	318
	20	297	354	363	268	325	386	387	317
	30	304	353	366	263	328	385	387	317
	40	311	353	368	259	330	384	387	317
	50	316	354	370	257	332	384	386	317
	60	320	355	371	256	334	383	386	317

Table 10. Results for External and Internal edge girder (Span = 12m) with and without intermediate diaphragms at the mid span.

	Skew (degree)	With a Diaphragm				Without a Diaphragm			
		36+59	37+58	38+57	39+56	36+59	37+58	38+57	39+56
M^{+ve} Max. (kN.m)	0	401	402	402	401	577	577	577	577
	10	433	390	418	370	575	575	581	581
	20	461	381	434	341	573	573	585	585
	30	485	375	448	315	570	570	586	586
	40	504	372	461	293	565	565	587	587
	50	518	372	471	277	562	562	586	586
	60	528	373	480	266	558	558	584	584
M^{-ve} Max. (kN.m)	0	36	582	582	36	214	706	706	214
	10	59	579	587	32	232	708	706	213
	20	87	579	592	33	252	708	706	215
	30	115	581	596	37	273	708	706	220
	40	144	583	600	44	294	706	705	227
	50	170	584	602	52	313	704	703	237
	60	192	585	603	61	329	701	700	247
Shear Max. (kN)	0	319	408	408	319	354	433	433	354
	10	327	407	409	313	356	433	433	353
	20	334	407	411	308	358	433	432	352
	30	341	408	412	303	360	432	432	352
	40	347	408	413	301	363	431	431	352
	50	352	409	414	299	365	430	430	353
	60	356	410	414	298	367	430	429	353

7.1 Effect of diaphragm on the Max. Positive Moment

Figures 33 to 34 show the effect of changing the skew angle on the 10-m span and12-m span length with and without intermediate diaphragms, respectively.

Reference to Figures 33 and 34, the effect of diaphragm on the Max. Positive Moment is clear. It reduces the value of the Max Positive Moment for the girders (i.e. for span length of 10m,the Max. Positive Moment for zero skewness with a diaphragm is about 290 kN.m, but without a diaphragm the value is 418 kN.m) the same for span 12m. The effect of the skew angle on the Max. Positive Moment on the span is less significant on the bridges without a diaphragm.

In Figure 35, the effect of increasing the skew angle on the Max. Positive Moment is less significant on bridges without intermediate diaphragms than those with intermediate diaphragms.

In general the existence of diaphragm decrease the Max. Positive Moment in girders. But when designing a span without diaphragm, the assumption of normal skew bridge in design could be applicable.

7.2 Effect of diaphragm on the Max. Negative Moment.

The effect of introducing an intermediate diaphragm in the bridge on the Max. Negative Moment at the edge girder is shown on Figures 36 and 37.

In both figures the effect of the skew angle is negligible on the Max. Negative Moment but the existence of diaphragm decreases the value of Max. Negative Moment at the edge girders.

The same behavior appears in Figure 38 for the Max. Negative Moment at all bridge girders, the effect of skew angle is negligible and existing of a diaphragm will decrease the value of Negative Moment.

7.3 Effect of diaphragm on the Max. Shear.

The Figures 39 and 40 show the effect of changing the skew angle on the Max. Shear on edge girder for spans 10m and 12m with and without interior diaphragm.

The effect of increasing the skew angle for girders without diaphragm is insignificant, but adding a diaphragm to the bridge will decrease the value of the Max. Shear at the edge girders.

For Max. Shear for all bridges the effect of increasing the skew angle is negligible for bridges without diaphragms for both spans adding diaphragms to the bridge will decrease the Max. Shear Force at the bridge girder.

Figure 41 shows the effect of adding a diaphragm on the Max. Shear.

8. Effect of skew angle on the Internal forces of the diaphragm

Table 11 shows the Max. Moment and Shear of the intermediate diaphragm for the spans 10m, 12m, 14m,and 16m.

Table 11. Results for max. Internal Forces on diaphragms for spans 10, 12, 14, and 16 m

	Skew (degree)	Span (m)			
		10	12	14	16
M+ve Max. (kN.m)	0	286	337	390	429
	10	294	346	402	454
	20	303	357	418	464
	30	312	368	435	487
	40	321	379	452	511
	50	328	387	466	531
	60	334	393	477	548
Shear Max. (kN)	0	136	151	162	168
	10	141	156	168	175
	20	146	162	175	183
	30	150	167	182	191
	40	154	171	188	199
	50	157	174	193	205
	60	158	176	196	210

The effect of increasing the skew angle on the Max. Moment at the diaphragm is shown in Figure 42. Increasing the skew angle will increase the value of the Max. Moment. This increase depends on the span length; for 10m span the percentage increase is (14%) and for the 16m span it is (22%).

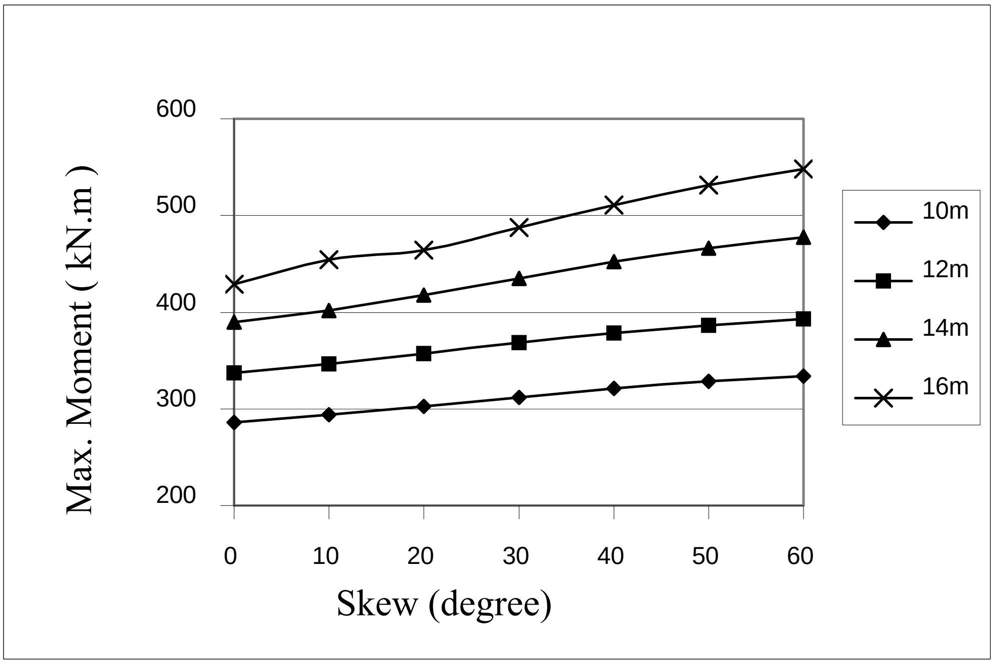


Figure 42. Max. Moment on diaphragm

The same behavior appeared in Figure 43 for the Max. Shear force. Increasing the skew angle will increase the Shear force at diaphragm; for 10m span the percent of increase was (13%) but it was (20%) for 16m span.

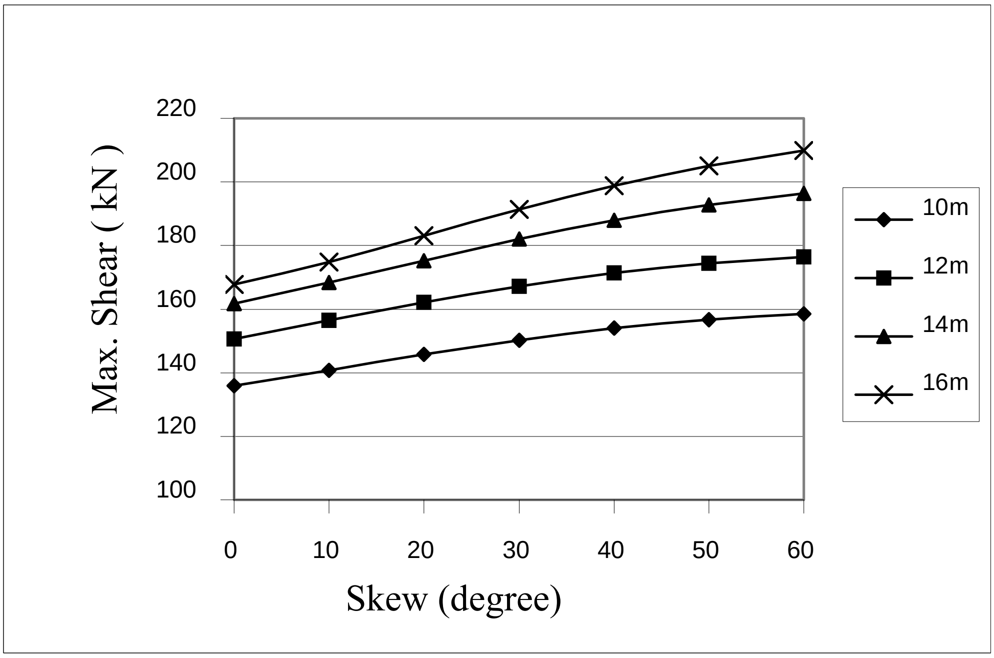
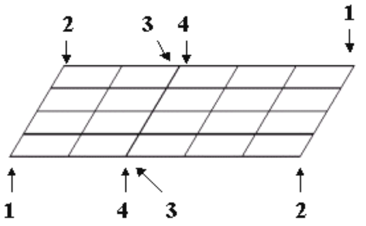


Figure 43. Max. Shear on diaphragm

9. Effect of skew on Deck Slab

Table 12 shows the results for Internal forces (kN.m/m) at deck slap due to increasing skew angle at the corner of deck at abutment and pier. Node (2) indicate the obtuse corner at abutment, node (3 and 4) indicate the point of pier (acute and obtuse corner), and node (1) indicate the acute corner at abutment.

Table 12. Results for Internal forces at Deck Slab. (kN.m/m)



	Skew (Degree)	Span (m) 16			Span (m) 14			Span (m) 12		
		M_{11}	M_{22}	M_{12}	M_{11}	M_{22}	M_{12}	M_{11}	M_{22}	M_{12}
Obtuse Corner Of Abutment (2)	0	6	-0.2	0.4	6.3	-0.2	0.4	6.6	-0.1	0.3
	10	5.9	-0.4	-0.3	6.2	-0.3	-0.4	6.5	-0.3	-0.5
	20	5.8	-0.4	-1	6.1	-0.3	-1.1	6.4	-0.3	-1.3
	30	5.8	-0.3	-1.6	6	-0.2	-1.7	6.3	-0.1	-1.9
	40	5.7	-0.2	-2.1	5.9	-0.1	-2.3	6.1	0	-2.4
	50	5.6	-0.2	-2.5	5.8	0	-2.7	6	0.2	-2.8
	60	5.6	-0.1	-2.8	5.7	0.1	-2.9	5.8	0.2	-3.1
Acute Corner Of Pier (3)	0	-3.9	-5.2	0	-3.9	-4.6	0	-4	-4	0
	10	-4	-5.4	0.2	-4	-4.8	0.3	-4	-4.2	0.3
	20	-4.1	-5.9	0.5	-4	-5.2	0.6	-4	-4.6	0.7
	30	-4.2	-6.5	0.7	-4.1	-5.9	0.8	-4.1	-5.2	0.9
	40	-4.3	-7.3	0.9	-4.2	-6.6	1	-4.2	-5.9	1.2
	50	-4.4	-8.1	1.1	-4.3	-7.3	1.2	-4.2	-6.6	1.3
	60	-4.5	-8.8	1.2	-4.4	-7.9	1.3	-4.2	-7.2	1.4
Obtuse Corner Of Pier (4)	0	-3.9	-5.2	0	-3.9	-4.6	0	-4	-4	0
	10	-3.9	-5.3	0.3	-3.9	-4.7	0.3	-4	-4.1	0.4
	20	-4	-5.7	0.6	-4	-5	0.6	-4	-4.4	0.7
	30	-4.1	-6.3	0.8	-4	-5.6	0.9	-4	-5	1
	40	-4.2	-7	1	-4.1	-6.2	1.1	-4.1	-5.6	1.2
	50	-4.3	-7.7	1.2	-4.2	-6.9	1.3	-4.1	-6.2	1.4
	60	-4.4	-8.3	1.3	-4.3	-7.5	1.4	-4.1	-6.8	1.5
Acute Corner Of Abutment (1)	0	6	-0.2	-0.4	6.3	-0.2	-0.4	6.6	-0.1	-0.3
	10	6.1	0.2	-1.1	6.4	0.2	-1.2	6.7	0.2	-1.2
	20	6.2	0.9	-1.9	6.5	0.9	-1.9	6.8	0.9	-2
	30	6.4	1.6	-2.5	6.6	1.7	-2.6	6.8	1.7	-2.8
	40	6.5	2.5	-3.1	6.7	2.5	-3.3	6.9	2.5	-3.4
	50	6.6	3.5	-3.6	6.8	3.4	-3.8	6.9	3.3	-3.9
	60	6.7	4.1	-4	6.8	4.1	-4.1	6.9	4.1	-4.3

Moments on deck slap are, M_{11} (kN.m/m) for bending moment in the longitudinal direction, M_{22} (kN.m/m) bending moment in the transverse direction, M_{12} (kN.m/m) for torsional moment in the slap.

AASHTO specification for highway bridges indicated the following: (the bending moment per foot width of slab shall be calculated according to the method given under case A and B, unless more exact method are used (3.24.4)). Case A deals with main reinforcement perpendicular to traffic where the following formula shall be used to calculate the live loud moment:

HS20 loading:

$$\left(\frac{S+2}{32}\right)p_{20} \quad = (Ib.ft/ft)$$

$$\left(\frac{S+0.61}{9.74}\right)p_{18} = (\text{kN.m/m})$$

Were:

S	=	effective span length.
P_{20}	=	16,000 pounds H20 loading (71.17 kN)

$$\left(\frac{(2-0.4)+0.61}{9.74}\right)(71.17)(0.8) \approx 13 \text{ (kN.m/m)}$$

Using this formula in the case study will give (13 kN.m/m) for all span cases regarding the skew angle.

Figure I show the effect of increasing the skew angle on M_{11} at the obtuse corner at abutment. It can be noted that increasing the skewed will decrease the value of M_{11} for all spans.

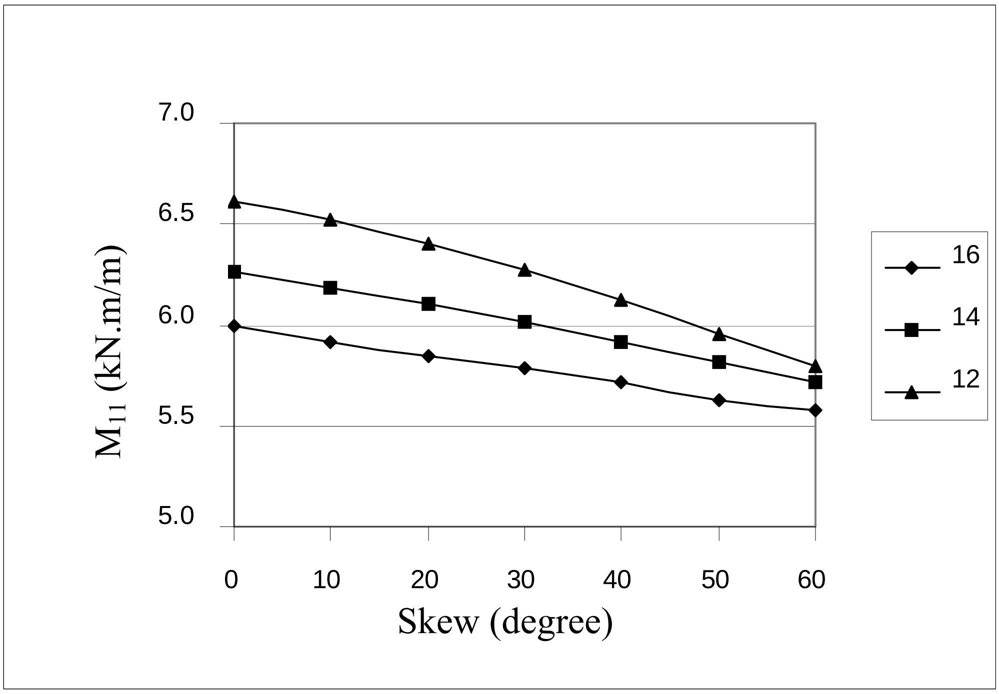


Figure 44. The relation between M_{11} and skew angles, of obtuse corner at abutment.

The opposite behavior appears in Figure 45, which shows the effect of increasing the skew angle on M_{11} at the acute corner at abutment. Increasing the skew angle will increase M_{11}.

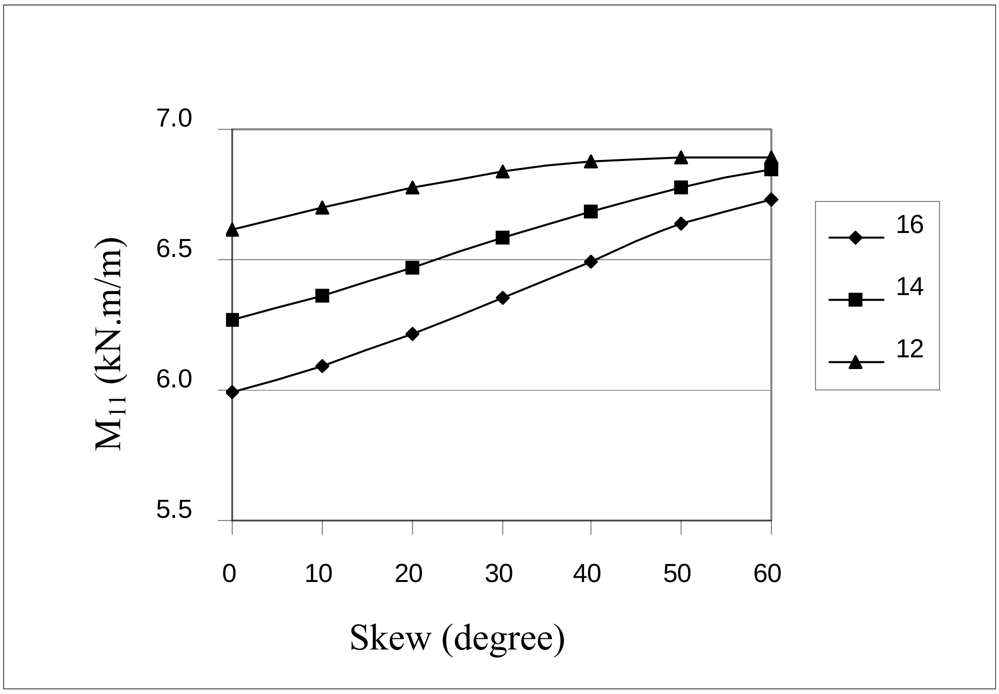


Figure 45. The relation between M_{11} and skew angles, at acute corner at abutment.

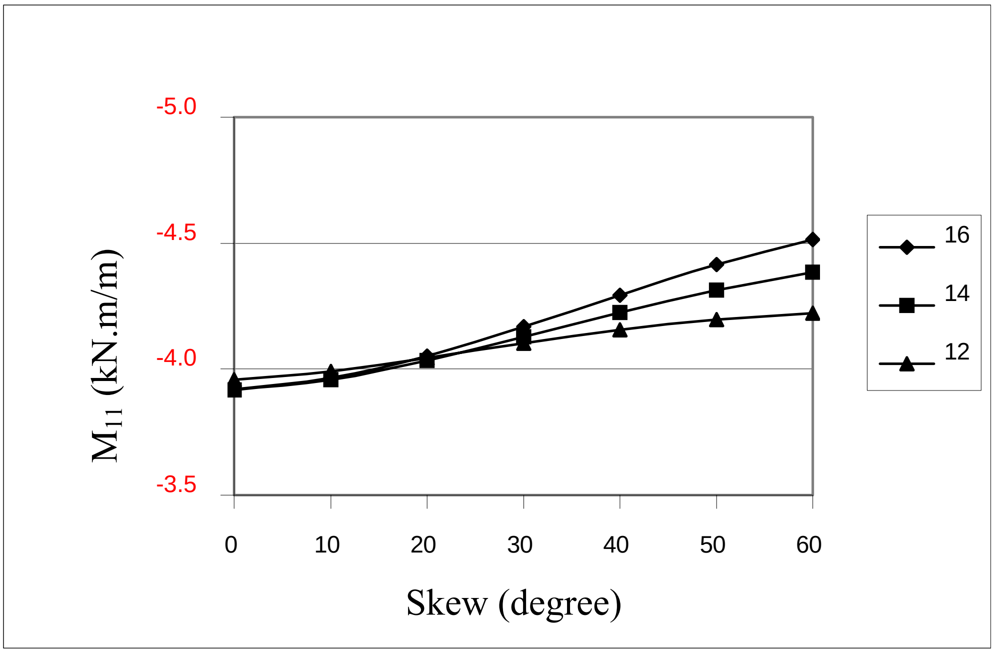


Figure 46. The relation between M_{11} and skew angles, at acute corner at pier.

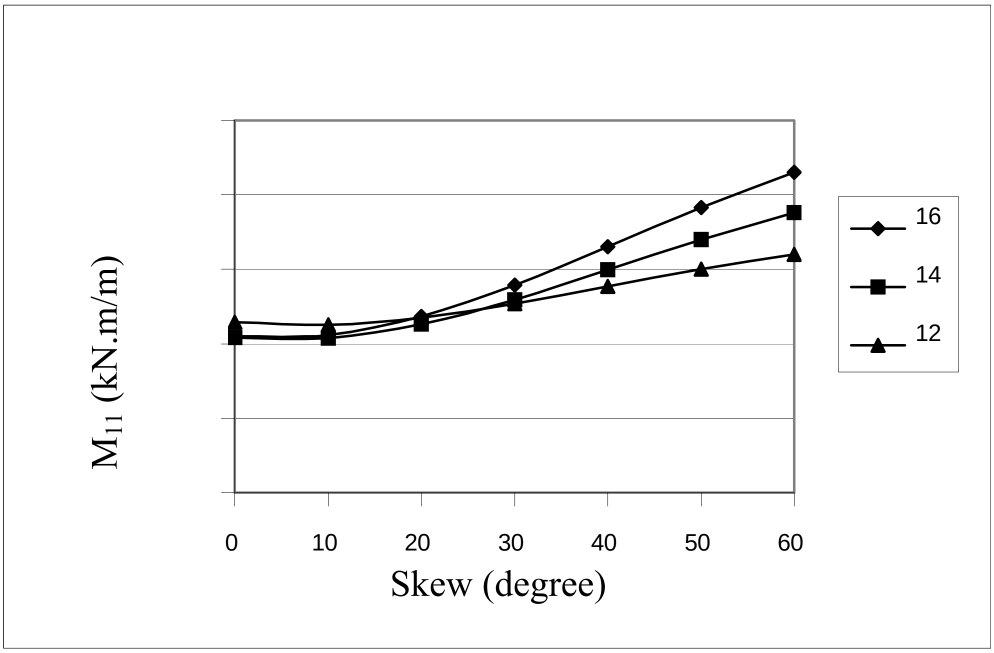


Figure 47. The relation between M_{11} and skew angles, at obtuse corner at pier

Figures 46-47 show the effect of increasing the skew angle on M_{11} at the acute and obtuse corner at pier. The two figures gave the same behavior. Increasing the skew angle will increase the value of M_{11} at this point. For higher span the effect was higher, for 12m span the value increased by (10%) but for 16m span increased by about (16%).

Effect of skewed on M_{22} are shown in Figures 48 through 51.

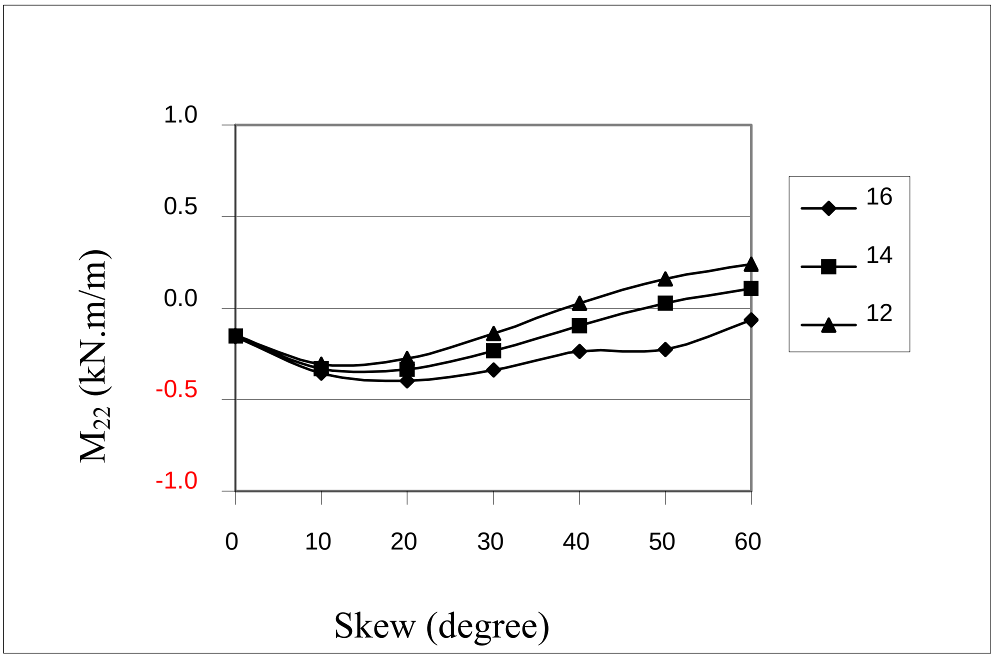


Figure 48. The relation between M_{22} and skew angles, of obtuse corner at abutment.

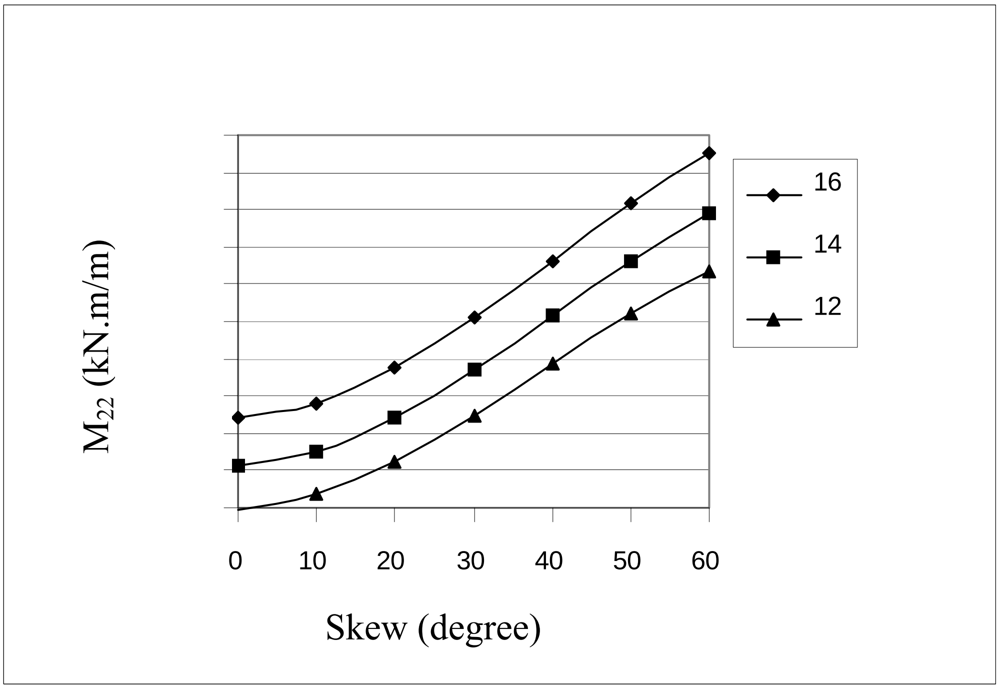


Figure 49. The relation between M_{22} and skew angles, at acute corner at pier.

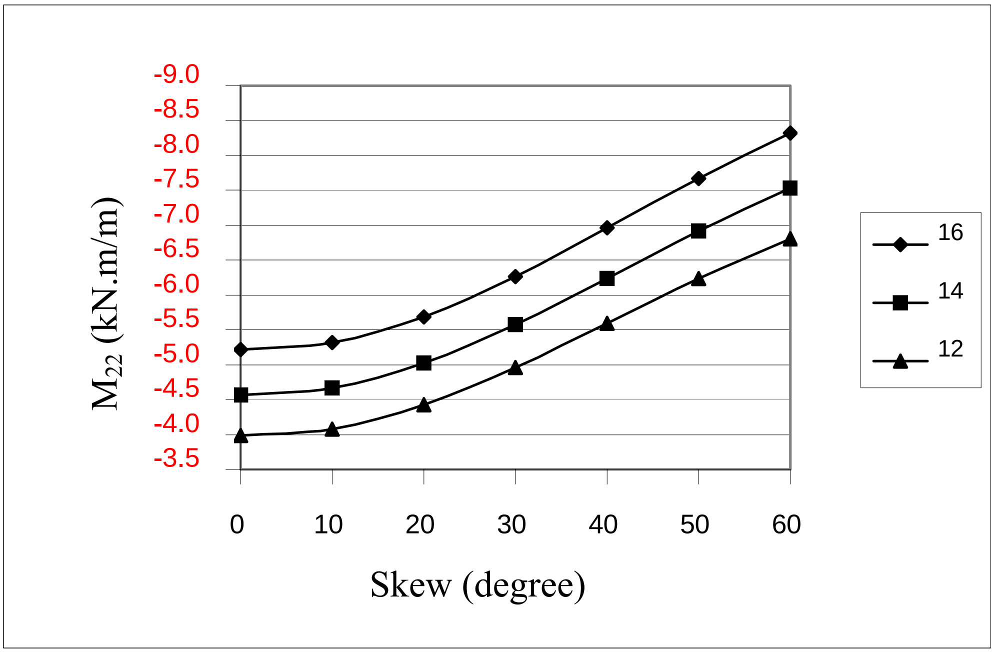


Figure 50. The relation between M_{22} and skew angles, at obtuse corner at pier.

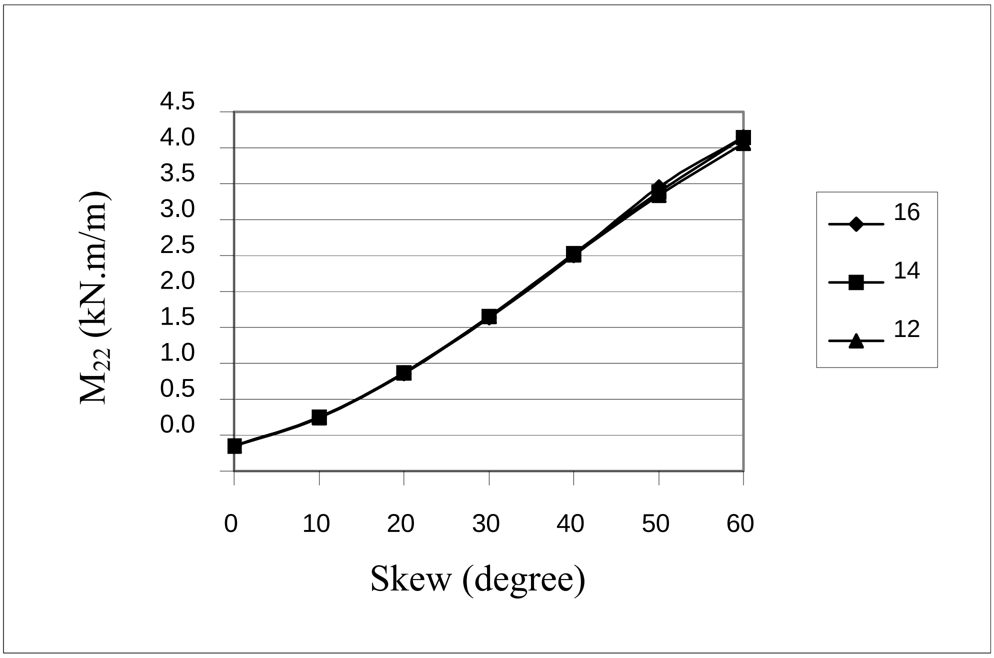


Figure 51. The relation between M_{22} and skew angles, at acute corner at abutment.

M_{22} values were very small for the obtuse corner at abutment and the effect of skewed was insignificant. This can be shown in Figure 48 the values were in the range of (-0.5 to 0.25) kN.m/m for all span.

Figure 49 shows the effect of skewed on the acute corner at piers, it can be noted that increasing the skew angle will increase the value of M_{22} at this point. Also increasing the slap length will increase the value of M_{22}.

The same behavior appears for the obtuse corner at piers Figure 50. In Figure 51 increasing the skew angle increased the values of M22 of the acute corner at abutment. But the effect of span length was negligible.

Effect of increasing skewed on M_{12} are shown in Figures 52 through 55.

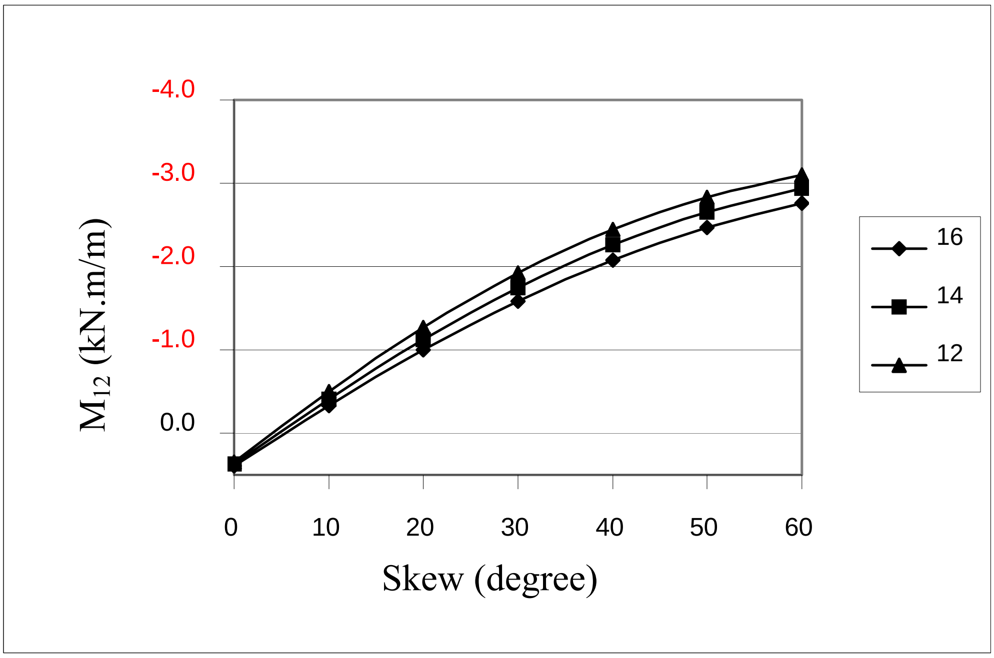


Figure 52. The relation between M_{12} and skew angles, of obtuse corner at abutment.

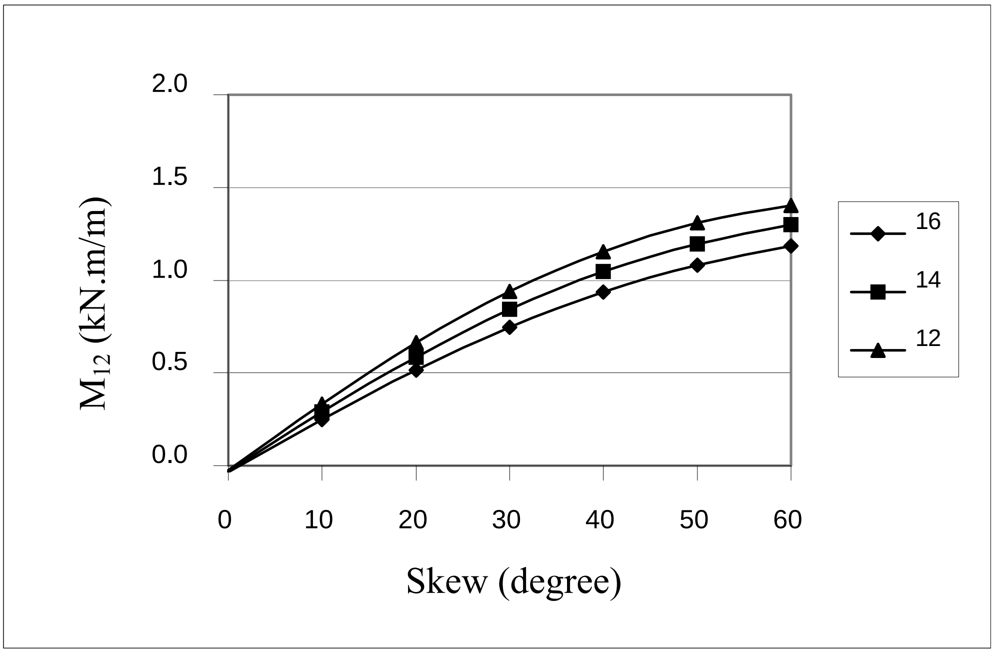


Figure 53. The relation between M_{12} and skew angles, at acute corner at pier.

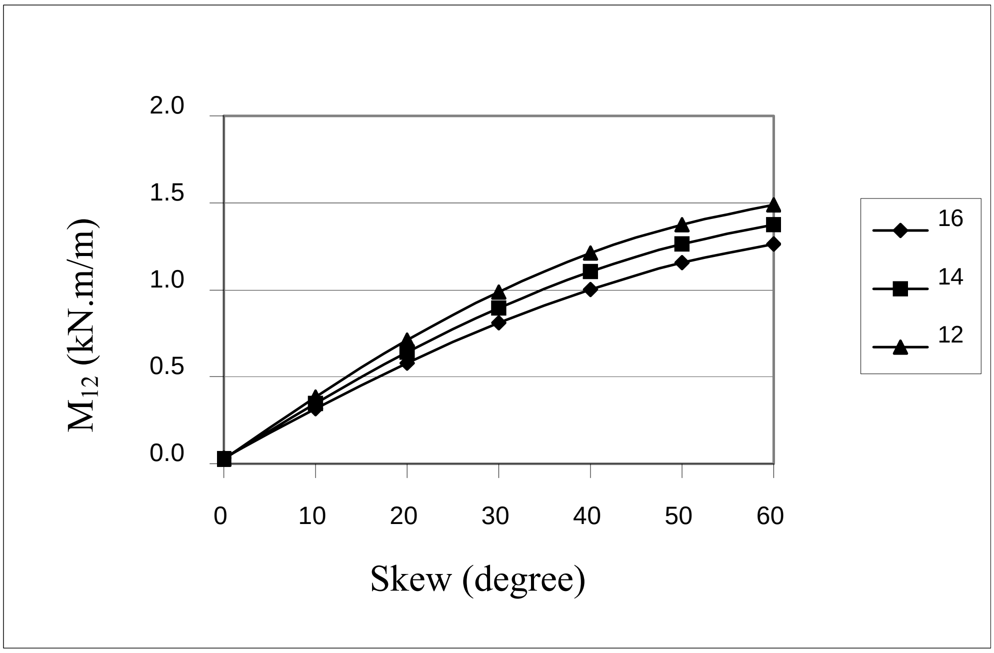


Figure 54. The relation between M_{12} and skew angles, at obtuse corner at pier.

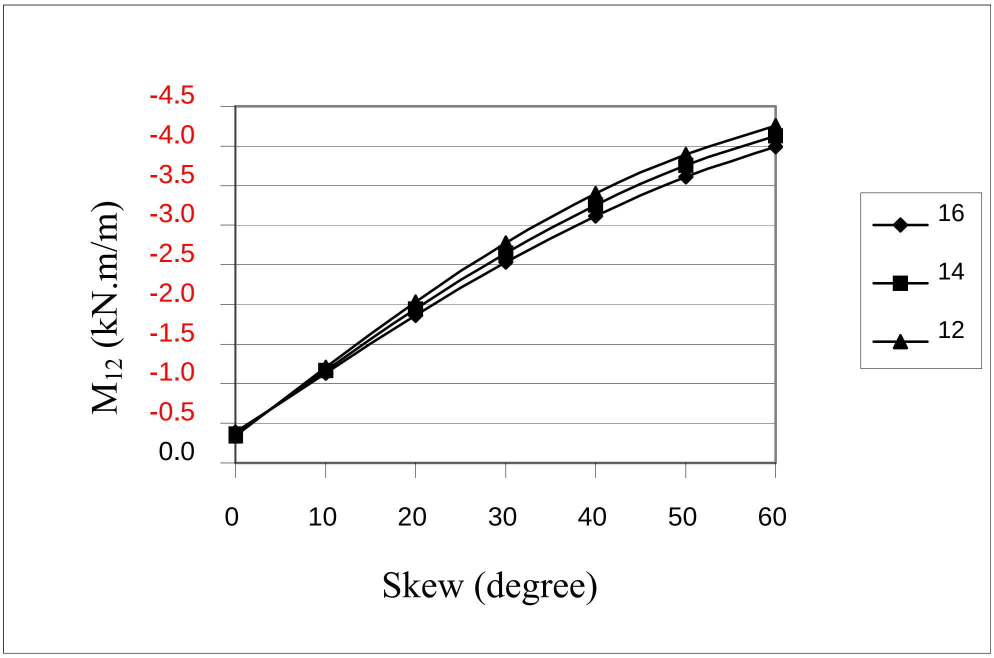


Figure 55. The relation between M_{12} and skew angles, at acute corner at abutment.

The behavior in all corners was the same. Increasing the skew angle will increase the value of M_{12}. But increasing the span length will decrease the value of M_{12}.

Comparing the value of M_{11}, M_{22} with the value of the empirical formula of AASHTO shows that the value of the formula was higher than all the value obtained by analysis for longitudinal and transverse moment of the deck slab.

So, in designing the deck slab it is easier to use AASHTO formula and ignoring the effect of the skew angle.

CONCLUSIONS AND RECOMMENDATIONS

1. Conclusions

From this research, the following points can be concluded:

1) Increasing the skew angle will move the point of Max. Positive Moment near to the middle support of the bridge.

2) Increasing the skew angle will increase the value of the Max. Positive Moment of the girder for the same span length. This increase was more significant for spans of 12, 14, and 16 m.

3) The Max. Negative Moment did not affected significantly by increasing the skew angle for the span of 10m. But for the other spans the percent of effects was about 7% for 12m and 4.5% for 14 and 16 m spans.

4) The Shear force near the obtuse corner was decreasing for increasing of the skew angle while near the acute corner it was increasing for all spans. The max. percent of increasing in the shear force was about 1.5%.

5) The vertical reaction at the support near the obtuse corner was increasing for increasing the skew angle for all spans with a percent of 8.5%.

6) At acute corner, increasing the skew angle will decrease the vertical reaction first then the values will increase. For span of 10 m, the values will start to increase after 50º, but for other spans after 30^{o}. The max. percent of increase was 3.5%.

7) Adding the diaphragm to the bridge will increase the overall stiffness of the bridge. This will decrease the values of positive and negative moment and shear force for the girders.

8) Increasing the skew angle in bridges without intermediate diaphragms will not effect on the stresses of the girders. The assumption of normal skewed bridge can be applied for analysis and design of the bridge.

9) Increasing the skew angle will increase the moment and shear force on the intermediate diaphragm of the bridge. This should be taken into consideration when designing high skew angle bridges.

10) At Abutment increasing skew angle of the bridge will decrease M11 at the obtuse corner, but it will increase at the acute corner.

11) At the intermediate pier, increasing the skew angle will increase M11

12) Increasing the skew angle will increase the value of M22 at any corner of the deck slab.

13) Effect of increasing skew angle on the max. Torsion on girder is insignificant. Maximum effect was about (4%).

14) Increasing skew angle will effect on the girder near the piers, but it will decrease the Torsion on the girder near abutment.

2. Recommendations

Based on the given conclusions, the following recommendations are suggested:

1) Bridges with small skew angles (up to 10^{o}) may design as a right angle skew bridge, but for higher skew angles, more detailed analysis must be done to study the extra effect of the skew angle on the Moment. Also, this should be applied when selecting the bearing pads at the girder's supports.

2) Other types of bridges are recommended to study, such as Voided Slab bridges, Box Girder bridges, Post-Tensioned bridges, and Steel bridges.

3) The effect of seismic forces on the normal skew and skewed bridges is recommended to study.

4) Other methods of analysis are also recommended to use and compared with the SAP2000 analysis. Software that use other methods of analysis are recommended such as STAAD-Pro.

5) Economical study for different types of bridges is recommended to find the effect of skew angle on the total cost of the bridge.

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توزيع الإجهادات على زوايا الجسور المستمرة المنحرفة

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المشرف
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ملخص

لقد ازدادت في السنوات الأخيرة أعداد الجسور التي تبنى بصوره منحرفة عـن محـور الطريق وخاصة في الدول المتقدمة، حيث أصبح شائعا بناء هذه الجسور في المـدن الرئيـسية وذلك لافتقارها للمساحات الواسعة التي عادة ما تحتاجها عملية إنشاء الجسور المستقيمة .

ومع هذا الازدياد في أعداد الجسور التي تبنى بصوره منحرفة عن محور الطريق فلقـد ازدادت الدراسات والأبحاث التي تدرس تأثير هذا الانحراف على توزيع الاجهادات على زوايا هذه الجسور وان كانت هذه الدراسات قليلة مقارنة بالتي بحثت في بناء الجسور المـستقيمة . هذه الرسالة هي دراسة نظرية تدرس تأثير تغيير زاوية الانحراف على توزيع الاجهادات على البنية الفوقية للجسر.

من أهداف هذه الرسالة : دراسة حجم مشكلة الانحراف وتأثيرها في تصميم الجـسور ، دراسة طبيعة التغيير في اجهادات الانحناء وقوى القص في العناصر الرئيسية لجـسم الجـسر العلوي وذلك لزوايا انحراف مختلفة وبحور مختلفة ،دراسة تأثير درجة الانحراف علـى ردود الأفعال الناتجة من البنية الفوقية على الركائز.

لدراسة تأثير درجة الانحراف على طبيعة التغيير في اجهادات الانحناء وقوى القص في العناصر الرئيسية لجسم الجسر العلوي كنموذج للدراسة، تم اخذ جـسر مـستمر ذو بحـرين متـساويين وبفـارق ٢م فـي كـل حالـه (١٠،١٢،١٤،١٦)م، وذلـك لزوايـا انحـراف

(٠،١٠،٢٠،٣٠،٤٠،٥٠،٦٠) درجة، ثم تمت عملية المقارنة بين النموذجين ١٠م و١٢م وذلك مع وبدون استخدام جوائز الربط الوسطية (الحجب) (diaphragms) علما بأنه تـم اسـتخدام برنامج (SAP2000) واعتمدت متطلبات الـ(AASHTO) للتحميل.

هذا ويمكن تلخيص نتائج هذه الدراسة بمايلي:

- إن زيادة زاوية الانحراف ستؤدي إلى زيادة عزم الانعطاف الاعظم الموجب وتؤدي إلى تحريك موقع هذا العزم باتجاة منتصف الجوائز.
- إن تاثير زيادة زاوية الانحراف قليلة على قوة القص الناتجة في الجيزان و على عـزم الإنعطاف الأعظم السالب.
- زيادة زاوية الانحراف تعمل على زيادة رد الفعل عند الركائز عند الزاوية المنفرجـة و لكن تقلل من رد الفعل عند الزاوية الحادة.
- وجود الحجب (diaphragms) يعمل على تقليل العزوم و قوى القص في الجوائز.

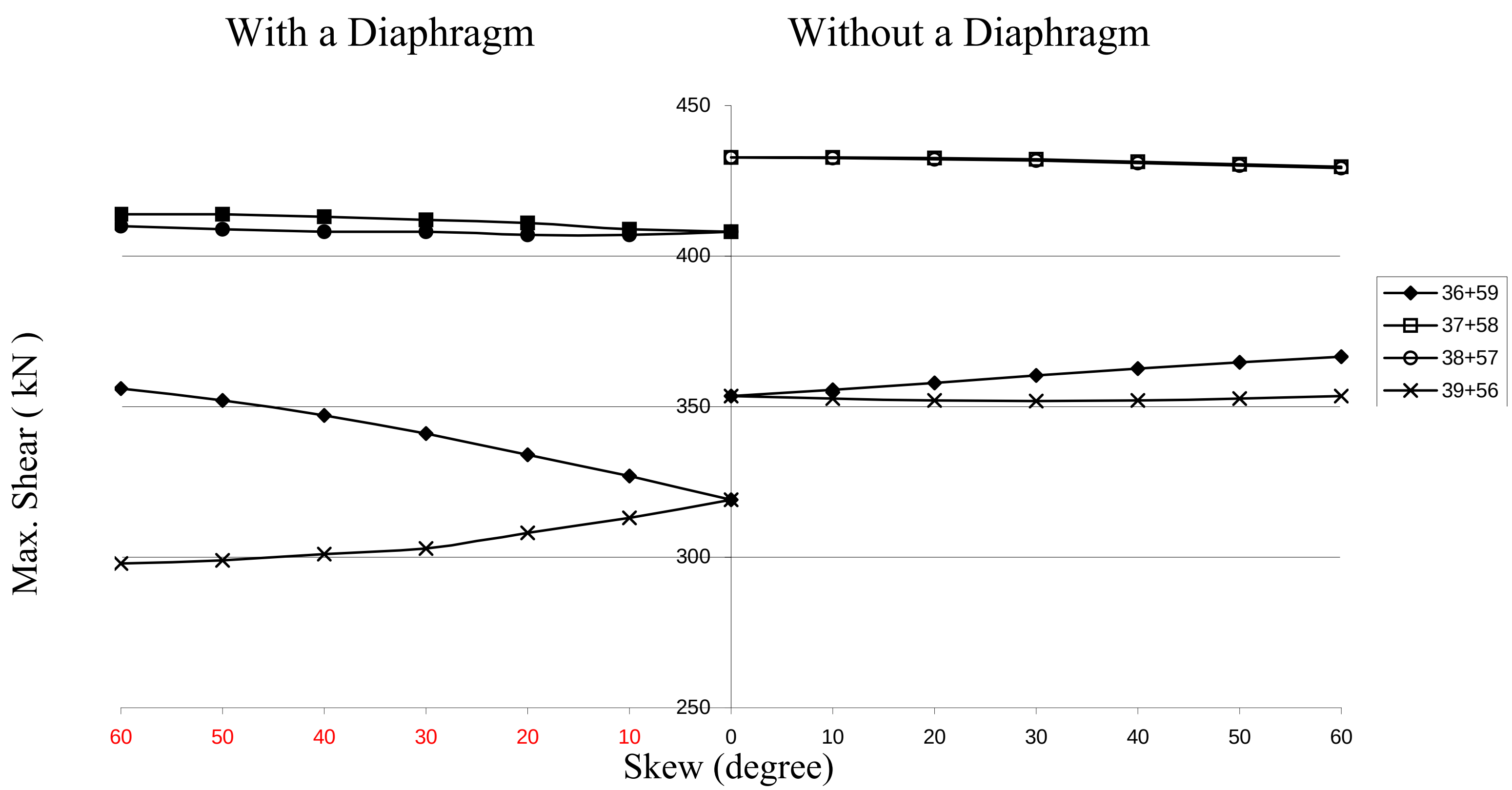

Figure 40. Max. Shear for span = 12 m (with and without a diaphragm)

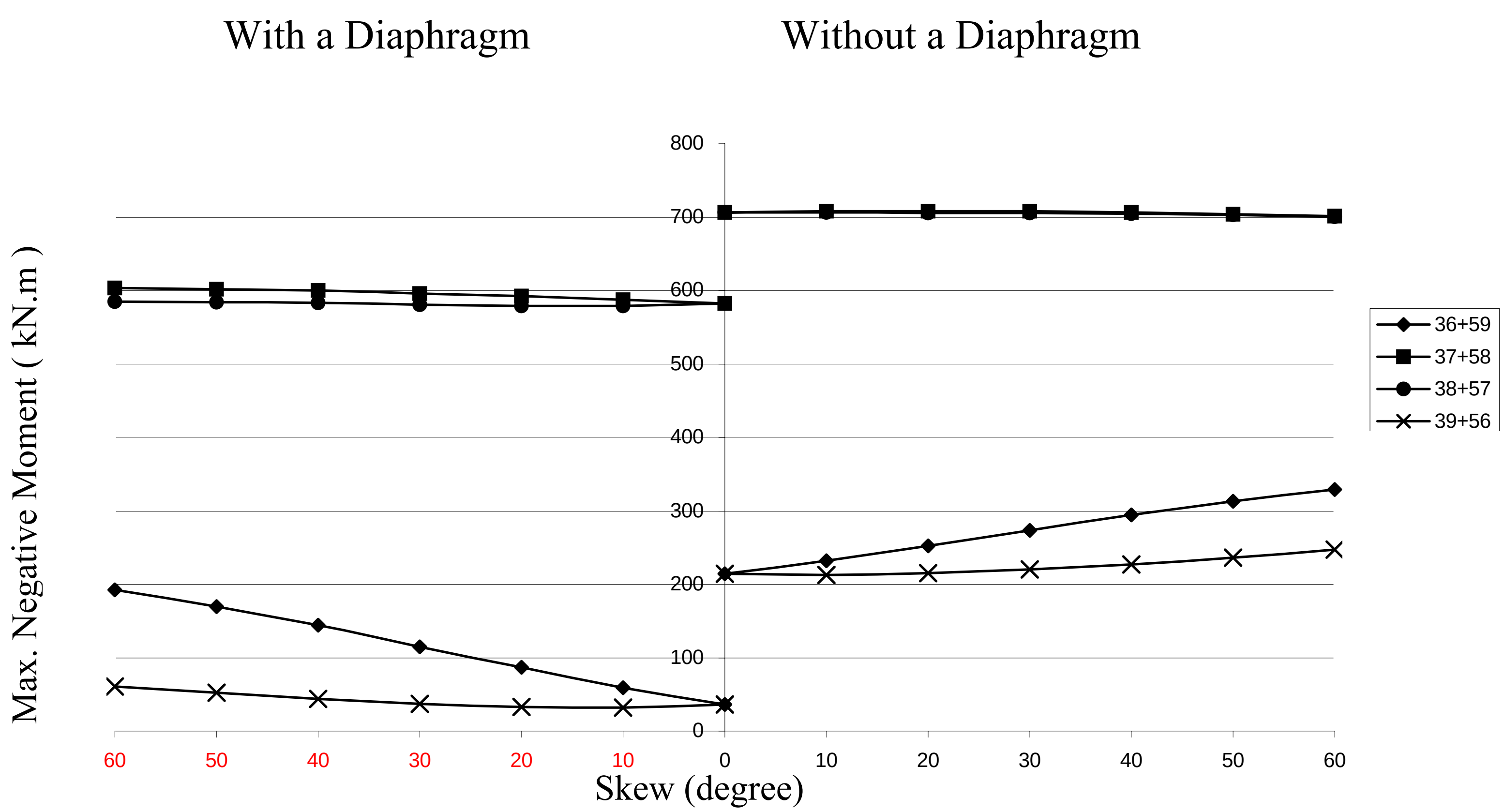

Figure 36. Max. Negative Moment for span = 12 m (with and without a diaphragm)

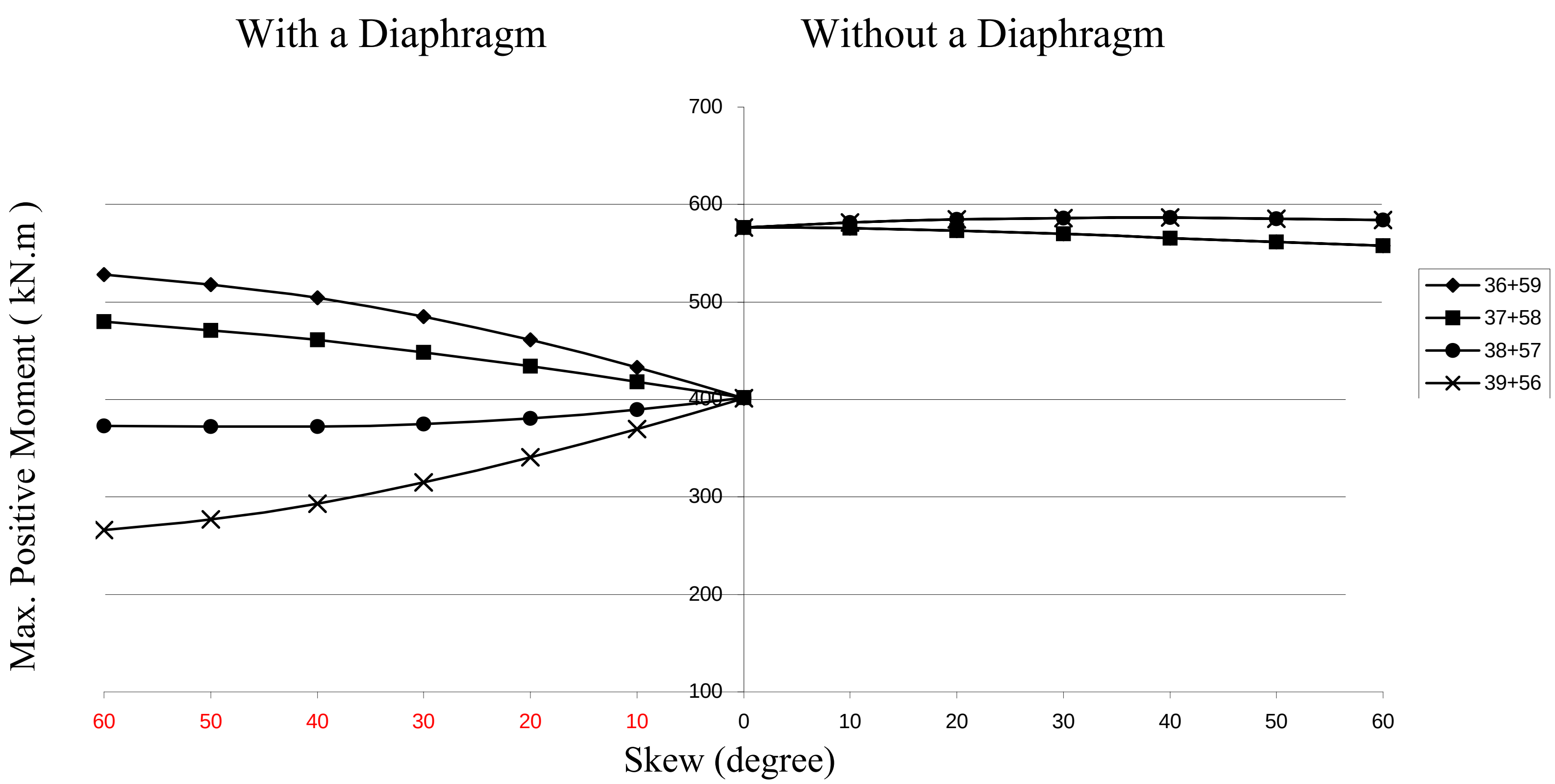

Figure 34. Max. Positive Moment for span = 12 m (with and without a diaphragm)

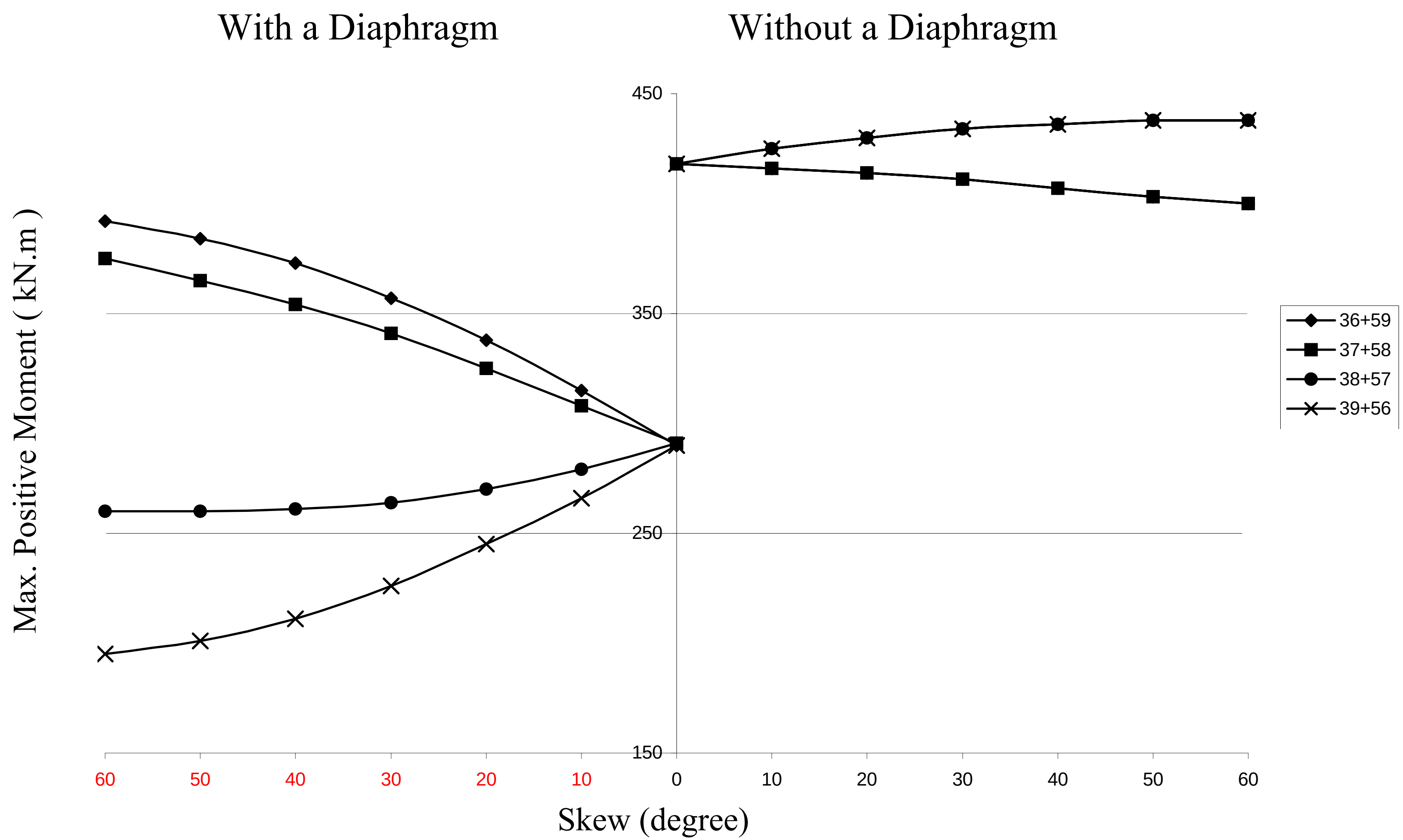

Figure 33. Max. Positive Moment for span = 10 m (with and without a diaphragm)

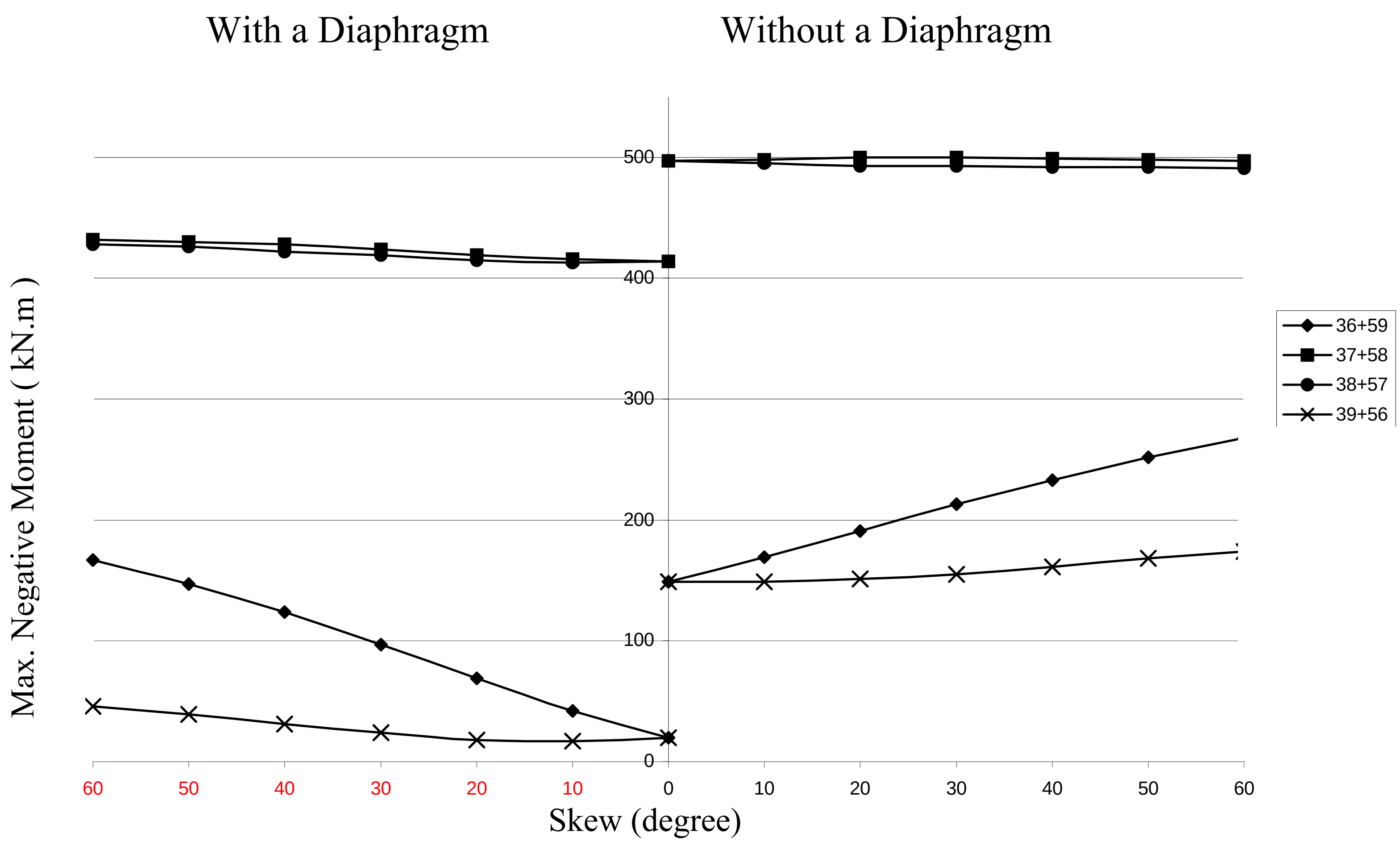

Figure 37. Max. Negative Moment for span = 12 m (with and without a diaphragm)

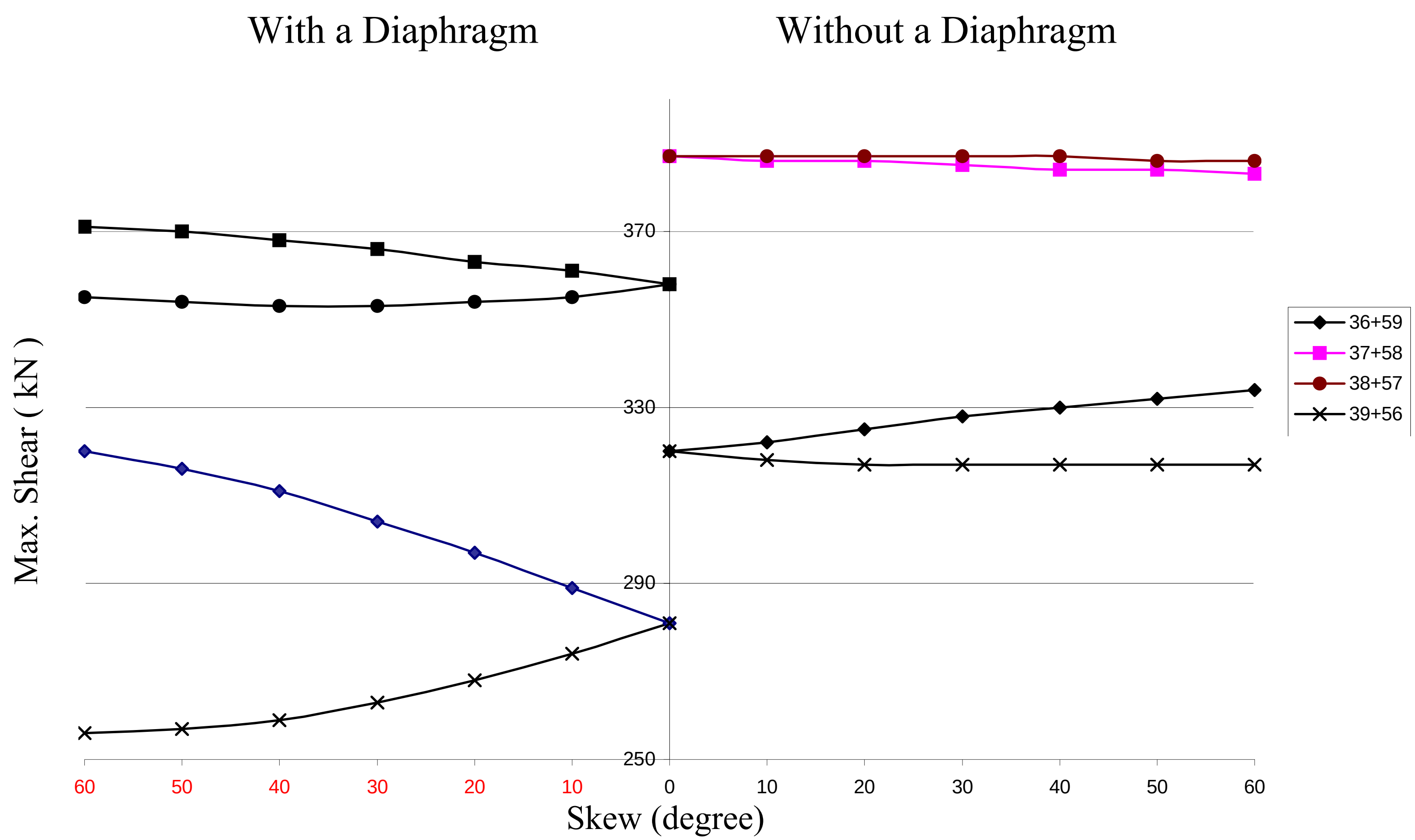

Figure 39. Max. Shear for span = 10 m (with and without a diaphragm)

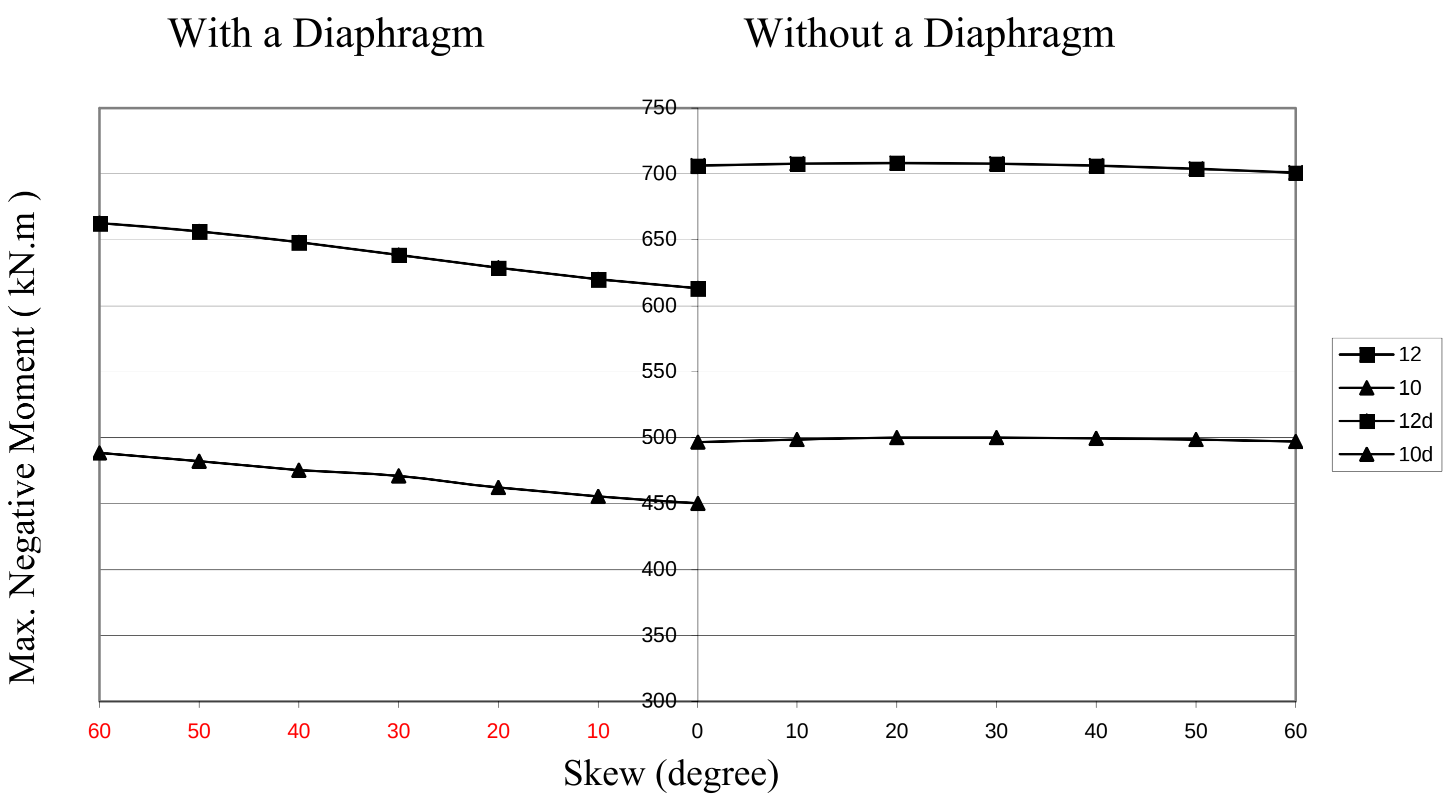

Figure 38. Max. Negative Moment with and without a diaphragm for 10, and 12m spans

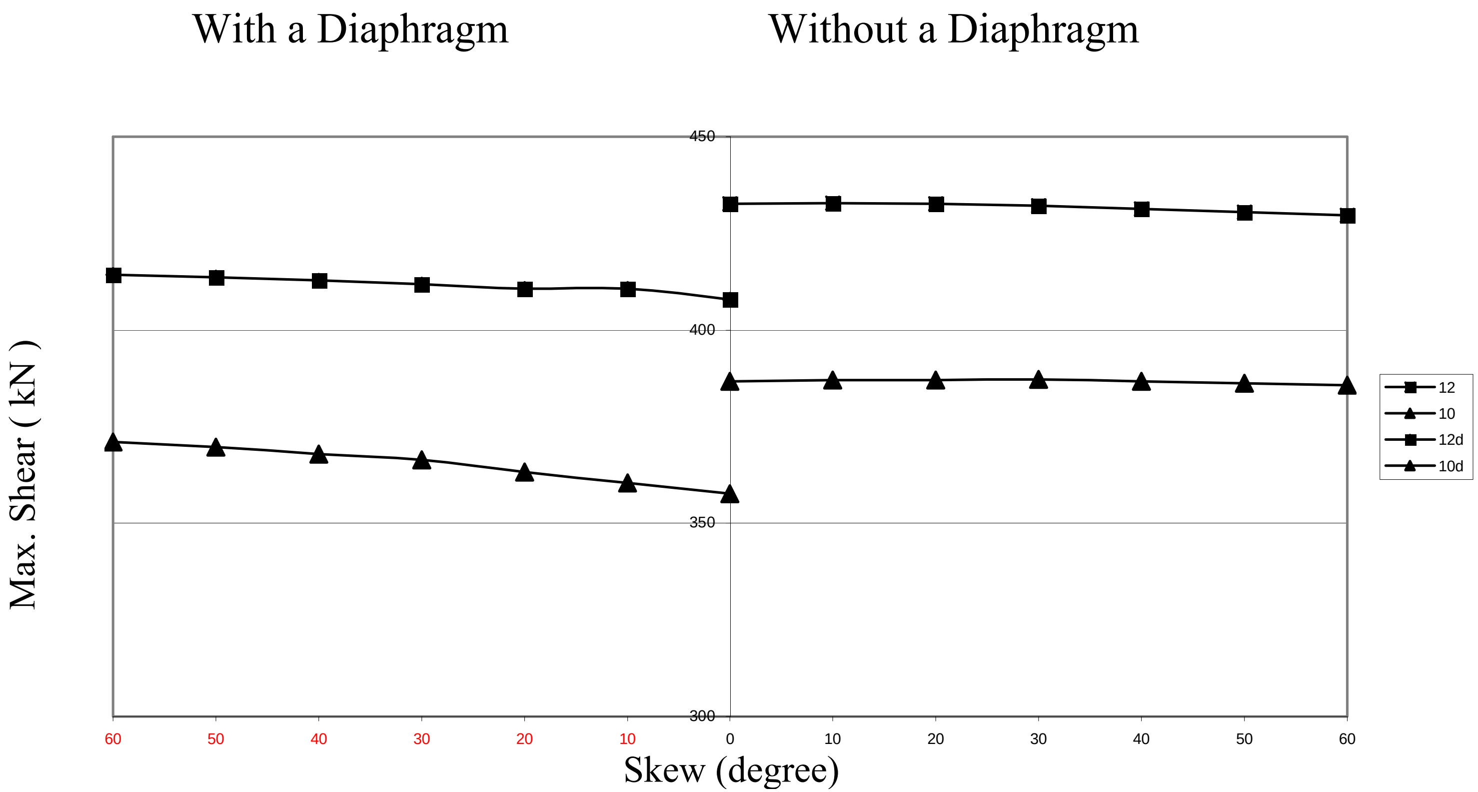

Figure 41. Max. Shear with and without a diaphragm for 10, and 12 m spans

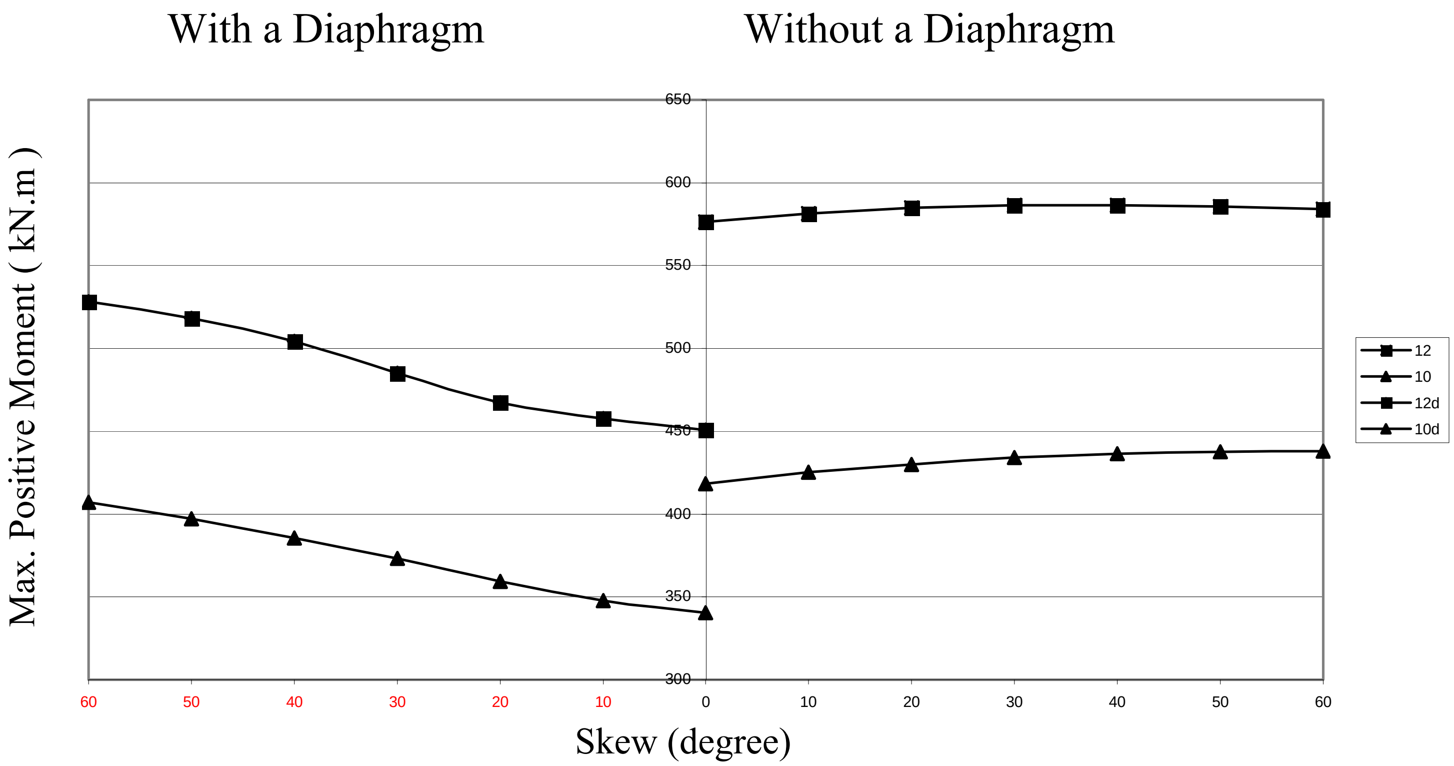

Figure 35. Max. Positive Moment with and without a diaphragm for 10, and 12m spans