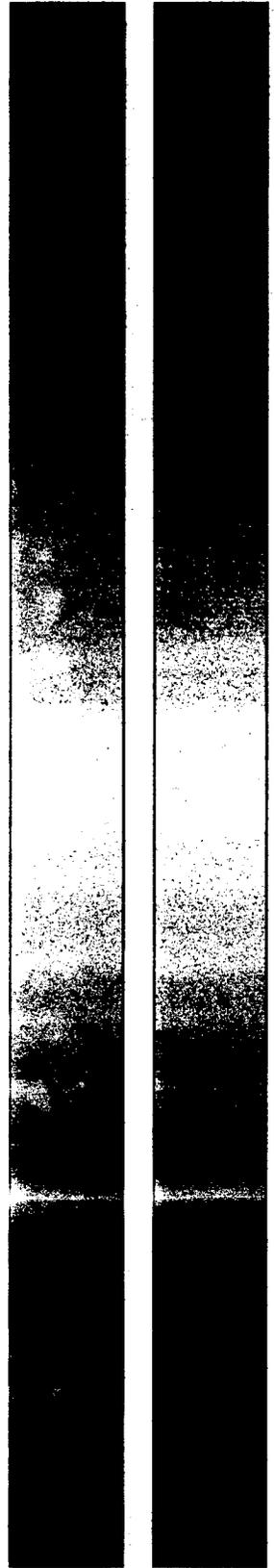




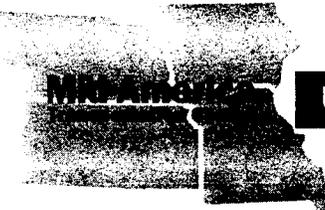
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Torsional Analysis for Exterior Girders

April 1999
Final Report



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Torsional Analysis for Exterior Girders

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Final Report**

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PREFACE

This research project was funded by the Kansas Department of Transportation K-TRAN research program and the Mid-America Transportation Center (MATC). The Kansas Transportation Research and New-Developments (K-TRAN) Research Program is an ongoing, cooperative and comprehensive research program addressing transportation needs of the State of Kansas utilizing academic and research resources from the Kansas Department of Transportation, Kansas State University and the University of Kansas. The projects included in the research program are jointly developed by transportation professionals in KDOT and the universities.

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Abstract

Concrete deck placement imposes eccentric loading on exterior steel bridge girders. This report describes a design tool that aids bridge engineers in evaluating the response of the exterior girder due to this eccentric loading.

Computer analyses are conducted in order to gain a detailed understanding of the factors influencing the response of the girder. It is shown that the "flexure analogy" is correct and can be used in the design tool. The "flexure analogy" is the assumption that torsional loads on the girder are mainly carried by the flanges in minor axis bending. Top and bottom flanges need to be analyzed independently since the boundary conditions for them vary significantly. Furthermore, analyses indicate that a substantial improvement in accuracy can be achieved if the boundary conditions on the local system used to analyze the behavior of the girder are changed. The influence of dynamic loads, such as the movement of the finisher and the impact of concrete during the placement process, is investigated and found to be negligible.

Based on these findings, a design tool in the form of a Visual Basic © application, TAEG (Torsional Analysis of Exterior Girders), for Windows 95/NT © has been created. It uses the stiffness method to calculate the stresses and deflections of the flanges due to torsional loads. Results for bracket forces and diaphragms are also calculated. TAEG can be used to evaluate the effect of temporary support in the form of tie rods and blocking.

Three examples are provided to justify the results and are compared with existing methods or field data. TAEG uses a 3-span fixed end continuous beam analysis model for finding torsional stresses while the AISC Design Guide method uses a less accurate single span fixed end model. Therefore, in comparison to the AISC Design Guide method stress results calculated with TAEG are approximately 20% higher for the positive moment region and approximately 20% lower for the negative moment region. Generally, stresses at the negative moment region govern.

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1. Introduction

1.1 Problem statement

Problems that occur due to eccentric loading of exterior steel bridge girders during concrete deck placement are the subject of the research project KTRAN KU-96-3, (refer to appendix C). This project is being conducted at the University of Kansas (KU) as a part of the Kansas Transportation Research and New Developments (KTRAN) Program of the Kansas Department of Transportation (KDOT).

Exterior girders of KDOT steel girder bridges are loaded with an eccentric load applied by cantilever overhang brackets. These brackets support the concrete overhang and the screed rail for the concrete finishing machine as well as the walkway for the worker. If deflection due to this loading is excessive it causes a thinner deck, insufficient concrete cover, concrete leakage through formwork, and, in severe cases, slip or even buckling of bent plate diaphragms.

KDOT currently uses an in-house computer spreadsheet (TORSION.WK4), [Jones and LaTorella, 1994] to predict the torsional response of the fascia girders. This spreadsheet follows the AISC Design Guide "DESIGN FOR CONCRETE DECK OVERHANG LOADS" [AISC, 1990]. Due to the inherent simplifications of the design guide and a lack of information about both the loads and the restraint, the method is not as accurate as desired and may lead to possible over or under design.

In order to achieve a better understanding of the problem and to develop a design aid that leads to an easier and more accurate design, this research project has been divided into the following three tasks:

- 1.) Gathering information on equipment types, construction loads, screed loading, and temporary bracing schemes.
- 2.) Performing field tests to determine the girder torsional response to the concrete and screed rail loads and to compare these results with an analytic model. The results will be used in calibrating and verifying the design aid that is part of task 3.
- 3.) Developing a design aid based on the information gathered in achieving tasks 1 and 2. It will evaluate stresses and deflections due to torsional loading as well as determine whether the proposed girder and bracing scheme are sufficient or not. If the proposed bracing scheme proves inadequate, the design aid will calculate the effects of revised temporary bracing as specified by the KDOT engineers. This design aid will be the primary deliverable item from this project, with an accompanying final project report.

Objective 3 is the subject of the present investigation.

1.2 Background and Scope of the Design Aid

Both the AISC Design Guide [AISC, 1990] and the KDOT spreadsheet [Jones and LaTorella, 1994] use an approach to torsion called the flexure analogy. This approach considers the flanges of the girder to act independently under torsion and

treats them as single spans loaded laterally by a horizontal couple statically equivalent to the torsion imposed by the overhang brackets. It is assumed that the spans are totally fixed by the cross-frames or diaphragms of the bridge.¹ This assumption results in higher stresses at the ends (at diaphragms) and lower stresses at the center-span (between diaphragms) than for the case of continuous support conditions. The resulting stresses in the flange tips are superimposed with the dead load stresses due to non-composite action. The rotation of the whole girder is derived geometrically from the deflection of the two flanges. The investigation of the spreadsheet program [Jones and LaTorella, 1994] revealed that a considerable error enters the calculations with the accumulation of conservative values used by the AISC Design Guide. Throughout the AISC Design Guide and the spreadsheet only the governing values for loads, moments, and other parameters are carried forward for further computations, accumulating to an error of about 10 to 15% [Zhao and Roddis, 1996]. Examples of such conservative assumptions that accumulate in sequence are 1) the usage of maximum values of lateral moment in deriving equivalent uniform loads, and 2) selecting the worst case value of 0.53 for distributed loads and 0.60 for screed loads for the ratio of fixed end to mid-span moment (M_+) max.

¹ The KDOT Design Manual [KDOT 97] page 5-43 uses a different approach for estimating the angle of rotation. Since the beam fixity at the diaphragms is somewhere between a fixed and a pinned condition, the angle of rotation is computed for both end conditions and then the results are averaged.

In development for the AISC Design Guide method, the maximum lateral moments at the cross frames, $(M_{fw})_{max}$, due to the finishing machine are calculated for two different machines, but only the larger values for the heavier Bidwell 3600 Series machine are used. To quote from the AISC Design Guide [AISC, 1990], page 9: "...the maximum moments (M_{fw}) do not vary widely with the machine type.... The governing values of $(M_{fw})_{max}$ for the 3600 Series machine were then used to derive equivalent uniform loads...."

The maximum lateral flange moments between cross frames, $(M+)_{max}$, due to the distributed overhang loads are simply derived from $(M_{fw})_{max}$ by the multiplication of 0.53. Quote from [AISC, 1990], page 7: "A conservative value of 0.53 was selected for this ratio $((M+)_{max}$ to $(M_{fw})_{max}$)". A value of 0.60 was selected for the moments due to the finishing machine. Thus, to compute the torsional moments and stresses, the KDOT Design Manual page 5-40 uses full fixed end moments at the diaphragms and center-span moments between diaphragms of $0.53 \times (DL+LL)FEM$ and $0.6 \times (Screed \text{ Load}) FEM$.

Furthermore, the research done by Zhao and Roddis [1996] provides evidence that the flexure analogy is highly accurate and represents a feasible alternative in determining normal stresses at the flange tips. It also suggests that substantial improvement can be achieved by adjusting the boundary conditions of the considered simple span model from fixed end to continuous or spring supported. This improvement may be as high as 35% [Zhao and Roddis, 1996].

In addition to the main goal of achieving more accurate stress and deflection results for the exterior girders, KDOT expressed the need for consideration of issues related to secondary members, dynamic loads, temporary supports, and bracket forces in the design aid. For the secondary member bent-plate diaphragms and cross frames it is necessary to determine: 1) whether the secondary members are adequate to resist the loading by the exterior girder, and 2) for bolted connections whether slip will occur. Significance of dynamic loads generated by the finishing machine and the placement process should be evaluated. Moment reduction due to temporary transverse supports such as tie-rod at top flange and timber blocking near bottom flange must be included also. Finally, the determination of the internal bracket forces would facilitate the work of the engineer and can be included easily in the design aid.

1.3 Overview

To investigate torsional girder response various analyses are done both by Zhao and Roddis [Zhao and Roddis, 1996] and within this document. Computer analyses in this document are used to clarify 1) the support the diaphragms provide for the bottom flange (Section 2.1.2.1), and 2) the required number of spans for a continuous beam model (Section 2.1.2.2). Hand analyses are used to resolve 1) the influence of dynamic loads due to the movement of the motor carriage (Section 2.2.1), and 2) the impact of concrete during the placement process (Section 2.2.2). Dynamic effects

were found to be insignificant. Given the findings of these investigations and the results of the field testing, a static system and boundary conditions for the design aid program use are selected. The following basic assumptions can be justified with the conducted research and are carried over to the design aid program.

- The flexure analogy is accurate for the purpose of the analysis.
- Three continuous spans with fixed ends are sufficient to achieve good improvement over the AISC design aid assumptions.
- The amount of support for the bottom flange depends on the type of lateral support (cross frames vs. diaphragms) used and needs to be considered.
- Dynamic loads due to the movement of the motor carriage are negligible.
- Loads due to the impact of concrete during the placing process are negligible.

The program "Torsional Analysis for Exterior Girders – TAEG" unites the above findings in a Windows 95 ©/ Windows NT 4.0 © application. Divided into input and output sections, TAEG provides a structured step by step procedure to help design overhang dimensions and cross frame spacing as well as to check proposed falsework schemes, giving the engineer information on stresses, deflections, and diaphragm response. It also checks for slip in the case of bolted connections between girder and diaphragms and calculates internal overhang bracket forces.

Changed boundary conditions² used by TAEG give greater accuracy compared to the methods used before. With its simple to use interface and help option, the program TAEG is a valuable tool in the design of this type of common highway bridge.³

² TAEG uses a 3 span analytic model with fixed ends and pinned intermediate supports in contrast to the AISC Design Guide 1 span fixed end model.

³ Bridges that are multi-girder, medium span, simple or continuous, composite or non-composite, steel rolled beam or plate girder.

2. Analytic Investigation of Torsional Responses of Exterior Girders

2.1 Analytic Investigation of Girder Boundary Conditions

2.1.1 Previous Finite Element Analysis

The research conducted by Zhao and Roddis as part of the overall research project consisted of a parametric study of the torsional response of the exterior girder using Finite Element Method and structural analysis using stick-frame modeling. The analyses were conducted on an artificial set of bridge girders with parameters varied over assumed ranges and actual KDOT bridges. Parameters included girder depth, bridge width, bridge skew, and temporary support schemes. Results were compared to those obtained with the AISC Design Guide.

In the final report [Zhao and Roddis, 1996] of the investigation by Zhao and Roddis the following main conclusions are drawn, which influence the design aid development:

1. The flanges of the I-shaped girder carry the lateral load independently. The flexure analogy is valid for the described conditions.
2. The effects of temporary supports are considerable and need to be included. The effect of the temporary transverse supports depends on the stiffness they provide.

3. The assumption used by the AISC Design Guide of torsionally fixed boundary conditions at the cross-frames is inaccurate. The error may be as high as 35%.

Conclusion number 3 and the fact that the analysis was done on a model where bottom and top flanges are equally restrained by the diaphragms sets the stage for a more detailed analysis of the actual boundary conditions the diaphragms provide.

Although the report [Zhao and Roddis, 1996] concludes that bridge skew increases torsional loading, this occurs due to the finishing machine traversing the bridge perpendicular to the girders and diaphragms. This is typical for bridges with skews of 20% or less. Skew is not considered for the design aid.

2.1.2 Analytic Research Within this Investigation

2.1.2.1 ANSYS® Analysis of Boundary Conditions

In order to answer the question whether the diaphragms provide equal support for top and bottom flange, a Finite Element analysis has been conducted. This analysis compares a 3 span model with fully restrained end-supports and a stiffener/diaphragm support at the intermediate supports (stiffener model) with the same model only altered at the diaphragm supports in a way that the web is laterally fixed (no-stiffener model). The stiffener model considers a diaphragm that is only two thirds of the depth of the girder and is connected towards the top of the girder.

Refer to Appendix A for more detailed information on the boundary conditions and the modeling assumptions.

The girder dimensions are chosen within usual KDOT practice [Zhao and Roddis, 1996]. The largest section used for this analysis with a girder depth of 2130 mm (84 in.) is identical to section number 6 in table 1 of [Zhao and Roddis, 1996].

Only two runs with a height of 1130 mm (45 in.) and 2130 mm (84 in.) each are conducted for the no-stiffener model since, as confirmed by later observations, no influence of the girder height on this model is expected due to equal support for both flanges as described in Appendix A. In order to get a better picture of the behavior of the stiffener model, 5 runs with heights of 1130 mm (45in.), 1530 mm (61 in.), 1730 mm (69 in.), 1930 mm (77 in.), 2130 mm (84 in.) are performed.

Using the nodal solutions generated by ANSYS® the deflections, support reactions, and flange moments are determined and analyzed with a spreadsheet program. These results are obtained for various significant locations along the girder axis, for the bottom as well as the top flange (see Appendix A).

The results for the top flange of the two models are always within a negligible margin of each other. The bottom flange exhibits a considerable difference in moments and deflection. With the maximum moment deviation of 33 % and the deflections at the center of the middle span increasing by a factor of 2.8 comparing the no-stiffener model with the stiffener model, it becomes necessary to include these

effects for a thorough analysis. Although one would most likely see X-frame assemblies for the deeper girders tested, thus providing equal support to top and bottom flange, it is deemed necessary to include a procedure that incorporates the actual support conditions.

Good agreement of the ANSYS© top flange results with a simple beam model is found. This gives confidence in the chosen model and type of analysis. It also again confirms the validity of the flexure analogy.

2.1.2.2 Required Model Size

As already mentioned in Section 1.2, a substantial improvement to the accuracy of the analysis can be achieved by adjusting the support conditions for the flexure analogy model to continuous or spring support versus the fixed-end conditions used in the AISC Design Guide [AISC, 1990]. In order to apply the flexure analogy more accurately, it is necessary to determine how many spans a continuous beam approach should include.

Three different models (1-span, 3-span, and 5-span) are analyzed to determine the number of spans needed to accurately simulate continuous boundary conditions.⁴ The actual boundary conditions at the diaphragms are considered continuous over a larger number of spans. With the intention of getting as close as possible to this state

⁴ The design aid assumes diaphragms are placed in line across the full bridge width. This is the same assumption as is used by the AISC Design Guide and the KDOT Bridge Manual.

without an excessive problem size the three models are compared for certain load and deflection values at their center spans (refer to Appendix B).

Two load cases with a distributed and a concentrated load are analyzed. These load cases resemble the actual loading conditions that are mainly the distributed load of the concrete and the concentrated load of the finishing machine. Influence lines for various parameters of interest at the center span are analyzed (Figures B.2 through B.13).

Those influence lines for the center-span moment (M_2), support moments (M_B , M_C), center-span deflection (W_2), and support reactions (B, C) exhibit for both load cases one similar trend. Differences between the 1-span and the 3-span model are pronounced and considered important for this analysis. Differences between the 3-span and the 5-span are smaller by about an order of magnitude and are considered negligible for this analysis. This shows that a substantial gain in accuracy is made by changing from a 1- to a 3-span model, while very little would be gained by changing from a 3- to a 5-span model. Therefore, a 3-span model which is fully fixed at the end supports and pin-supported at the intermediate diaphragm supports is selected for the design aid. This model is shown in Figure D.1 with the intermediate diaphragm supports labeled as support A and support B.

2.2 Data Collection

2.2.1 Dynamic Loads due to the Movement of the Motor-carriage

As the motor carriage of the finishing machine moves across the truss of the machine, it stops at certain locations to allow the rollers and vibrators to smooth and compact the concrete. These stops and the following accelerations are more or less abrupt and apply lateral dynamic forces to the supports of the finishing machine. The magnitude of these forces is not available from the machine manufacturers [Bidwell, 1997]. However, from observation during deck placement, dynamic loads from motor carriage movement would not be expected to be greater in order of magnitude than concrete impact loads. As determined in the next section, it can be clearly calculated that concrete impact loads are negligible. Thus, dynamic loads due to motor carriage movement are also neglected.

2.2.2 Concrete Impact Loads.

Concrete that is freely discharged onto the forms during the placement process exerts an additional load on the entire system of formwork and bracing. While most authors of formwork related literature acknowledge the effects of concrete impact loads, such as Richardson [1977], Waddell and Dobrowolski[1993], Peurifoy and Oberlender [1995], ACI Committee 347 [1995], et al., no source of detailed analysis of the phenomenon is given nor are any rule-of-thumb methods suggested.

Generally, the loads due to the impact of concrete on the forms are considered small compared to the main dead and live loads and are neglected. The British CERA Report No.1 [Kinneer et al., 1965] neglects the loads in question and is therefore cited in Richardson [1977]. The author states later in chapter 6: "This final simplification

ignores the relatively small increase in pressure resulting from concrete being discharged freely from a height into the form.” It can safely be concluded that concrete impact loads are negligible. A rough estimate can be conducted in order to give a quantified understanding of the loads in question as follows.

The general approach is to find the beam deflection due to the concrete impact and derive then the equivalent static load that creates the same deflection. If this static load small is compared to the rest of the construction loads, it can be neglected.

In order to find the deflection due to the concrete impact a linear momentum approach is chosen. The system is a simple span that is hit by a concrete particle in its center. The size of the particle is equivalent to the amount of concrete that can be placed within one natural period of the beam. Two cases have been investigated. One with a light beam (no concrete placed) and one with a heavy beam (all concrete in place).

Using the conservation of the momentum, mass times speed of the concrete before the impact has to be equal to the mass times speed of the concrete and the considered beam section after impact. Furthermore, the kinetic energy of concrete plus beam is transferred into potential energy while the spring stiffness of the beam slows down the system. This gives the necessary equations to calculate the resulting deflection. The equivalent static force is easily attained from the spring stiffness of the beam and the calculated deflection.

Numerous assumptions enter this calculation, all of which can be found on the conservative side. Numbers for beam parameters have been taken from Synder Bridge, values for concrete flow rate and drop height are taken from "Concrete Pumping and Spraying" [T.H. Cooke, 1990]. Even if a dynamic impact factor of 2 is considered the resulting load is far smaller than the remaining construction loads and is therefore negligible.

In any event, limiting the height of the fall should decrease impact. To avoid segregation, deck concrete should usually not be discharged from heights greater than 4-6 feet, as noted in most construction specifications.

3. Approach Selected Based on Research Results

The preceding sections established that the flexure analogy with a three-span model of the girder flanges can be used to accurately analyze the torsional behavior of the exterior girders. The analysis on the boundary conditions at the diaphragms further suggests that a differentiation between the upper and the lower flange has to be made regarding their support. This differentiation and the need for the implementation of temporary tie rods and timber blocking can be realized best in treating these supports as static springs.

The requirements of this problem are well matched with the stiffness method. Therefore, the design-aid analysis is based on the stiffness method implementing the permanent and temporary lateral supports as springs where necessary. The analyzed model extends over three equal-length lateral spans with fully fixed end supports and pinned intermediate supports. These intermediate supports are springs for the bottom flange in the presence of diaphragms and lateral-rigid in the case of cross-frames. This is implemented with an option-button arrangement in the Visual Basic application. The spring stiffness of the temporary supports is based on the assumption that the tie rods and timber blocking are fixed at the middle of the bridge.⁵ This concept was already mentioned in Zhao and Roddis [1996], page 13: “because

⁵ The design aid assumes tie rods and blocking are placed in line across the full bridge width. This is the same assumption used by the AISC Design Guide and the KDOT Bridge Manual.

the wheel loads were almost symmetrical about the centerline of the bridge, the tie rods are assumed to be fixed at the center between the two exterior girders.”

Values for stresses and deflection (rotation) of the exterior girder due to the torsional load are derived from the results for the center-span of the analyzed three-span model. These results are superimposed with the major axis stresses and deflections due to the non-composite dead load required as program input.

Because the location of the overhang brackets is not known at an early stage of the project when the analysis is performed and the fact that their exact placement is not of great influence to the final result,⁶ loads are not converted into bracket forces. Dead loads of the concrete and forms and live loads on the deck and the walkway are treated as distributed loads. Loads due to the finishing machine are concentrated loads at the location of the wheels. No dynamic or impact loads need to be included in the design aid since it has been shown that loads due to the impact of concrete during the placing process are negligible (Section 2.2.2) and dynamic loads due to the movement of the motor carriage are negligible (Section 2.2.1).

⁶ [Zhao and Roddis, 1996], page 7: “Each unit load represented a lateral flange force due to Pd1 (Concrete D.L.), Pd2 (Form weight), Pw (Walkway L.L.), and Ph (hydrostatic force from wet concrete) applied at a cantilever bracket. (...) The moments did not vary widely with the location of the unit loads.”

4. Design Aid “Torsional Analysis for Exterior Girders” – TAEG

4.1 Scope of Program

The computer program “Torsional Analysis for Exterior Girders” – TAEG – is designed for two uses: first, as a quick tool during the design phase of a bridge project to check overhang and diaphragm/cross-frame dimensions and spacing;⁷ and second, to be used by KDOT to check falsework schemes submitted by the contractor. TEAG is a Windows95/NT© application that combines the familiar user-interface of Windows © with a custom made problem solution for the overhang problems encountered by KDOT.

A typical program run requires three basic steps: Input, Calculation, and Output. The input portion of the program consists of forms on 1) dimensions of girder, 2) the overall bridge and lateral support, 3) brackets, 4) loads, 5) connection girder - diaphragm, 6) falsework, and an additional form for general project information such as project number and description. All input information can be saved in a project file (*.prj) for later use. The calculation portion of the program is run by button selection. Following this, the output is displayed on forms divided into 1) stresses in the girder, 2) deflections of the flanges and the bracket, 3) loads of the

⁷ The AASHTO LRFD bridge specification does not set a 25-foot maximum diaphragm spacing.

brackets, and 4) diaphragm reactions. A fifth form summarizes all input and output information and can be saved to a file or printed. All forms can also be printed independently as they appear on the screen. Calculations can be done in SI metric or U.S. Customary units, with units matching the form labels, or even without predetermined units, in which case the input has to be self-consistent in units.

The program calculates stresses due to torsional load for the positive and negative moment region for both top and bottom flanges. Torsionally induced stresses are added to the stresses due to non-composite dead load and then compared to the yield stress. Lateral deflections of the flanges and the resulting rotation of the girder are given. The deflection of the screed rail is given also. Bracket forces are calculated in order to facilitate the task of approving falsework schemes in later stages of a bridge project. Diaphragm reactions include the support reactions of the flanges at the lateral supports, the resulting moment, and stress in the diaphragm due to this moment. If a bolted connection between girder and diaphragm is used, bolt load and critical bolt load are given based on AASHTO Table 10.32.3C. The program also determines whether slip in the connection occurs or not.

4.2 TAEG Users Manual

4.2.1 Installation

TAEG comes with its own setup program, which places all necessary files in the computer's system directory and registers them as needed. To install TAEG execute "Setup.exe" and follow the instructions on the screen. Help is provided in all the forms of the program as described in Section 4.2.5. These files may be viewed by press the functional key, 'F1' or select help press-button in some forms.

4.2.2 General use of TAEG

The application opens a main window where all actions are monitored and invoked. All actions are started using the pull down menu on the top of the window. The toolbar below provides additional user-friendliness without adding functionality.

Input information for a particular analysis is stored in project files (*.prj). The pull down menu "File" offers functions to open, save, print, and close existing and new project files. It also brings a window on the screen that holds general project and file information of the currently opened project file. The main window's caption displays the file name of the currently opened project file.

Creation of a new project file begins with the choice of units from the menu "Units". TAEG offers three choices: SI metric units (kN, mm), U.S. Customary units (kips, in.), and unit independent calculation. Whereas the first two choices provide the user with labels for the units to be used for every input, the third choice requires the user to enter consistent input that is self-consistent in terms of units. Notice that choosing different units will always reset a project and delete all input and output information. Internally TAEG uses metric units, namely kN and mm.

Input information is entered in 6 different input windows (Section 4.2.3) that are accessed from the “Input” menu. Note that closing the windows with the window button in the top right corner of the window will unload the window and thus erase all entered information. The windows should always be closed with the “Close” button on the window.

After all necessary information for the analysis has been entered, the user should save it to the project file using the pull down menu or the toolbar. After saving the project-input file, the calculations are ready to be run. This is initiated from the menu “Calculate” or, again, from the toolbar. The results can be viewed using the “Output” menu to invoke 5 output windows (Section 4.2.4). The “Help” menu (Section 4.2.5) gives access to the help system.

4.2.3 Entering Data

Project Information. (Location: Main menu - File – Project. See Figure 4.1)

General information of the project that the analysis belongs to can be entered in the project information form. Text fields are provided for project number and title, name of the engineer running the TAEG program, and space for notes. This form also keeps track of the units used, the date of the last modification, and the creation data of the project file. These last three items can not be altered by the user and are generated by the program. The date of creation reflects the time the project file was last assigned a new name, whereas the date of last modification presents the time the file was last saved.

Girder Data. (Location: Main menu – Input – Girder. See Figures 4.2 a – 4.2.b)

This form contains geometric and material properties of the exterior girder. Fields are provided for top flange width and thickness, web height and thickness, bottom flange width and thickness, steel modulus, and yield stress. The help displays a simple cross section of a typical girder and shows the dimensions mentioned. Rolled or plate girders may be used. Since the torsional response is about the same for rolled and plate girders, the program does not distinguish between them. Use the distance between the bottom of the top flange and the top of the bottom flange as the web height.

Bridge and Lateral Support Data (Location: Main menu – Input – Bridge. See Figures 4.3 a – 4.3.c)

The form is divided into two areas. The first area contains information on the overall bridge consisting of distance between lateral supports (diaphragm spacing) and distance between the two exterior girders. The width of the bridge is used to determine the spring stiffness of possible temporary tie-rods or timbers as mentioned in Appendix D.

The second area defines the type and properties of the lateral support. A choice between cross frames and diaphragms is offered with a set of option buttons. If diaphragms are used information on the diaphragm properties has to be entered. This information includes the diaphragm height, moment of inertia, yield stress, modulus of elasticity, top offset, and web stiffener dimensions of width and thickness. The top offset denotes the distance from the top of the exterior girder to the top of the diaphragm and is used in the determination of the spring stiffness of the web stiffeners. The width and thickness of the web stiffener that connects the exterior girder with the diaphragm are also used in the calculation of the spring stiffness of the stiffener.

Bracket Data (Location: Main menu – Input – Bracket. See Figures 4.4.a – 4.4.b)

Detailed information has to be given on the bracket dimensions. This includes the walkway width, bracket spacing, bracket weight, and all major bracket measures. The bracket weight and the bracket dimensions are used to calculate the bracket forces and the lateral forces to be carried by the exterior girder flanges.

Load Data (Location: Main menu – Input – Loads. See Figures 4.5a – 4.5c and 4.4b)

The load data form is divided into sections covering 1) live loads, 2) dead loads, 3) non-composite dead loads, and 4) finishing machine loads. The loads of the overhang consist of the live loads of the walkway and the slab, and the dead loads of the formwork and the concrete (Figure 4.4 b). Stresses due to non-composite dead loads are optional and can be entered for the positive and negative moment region, top and bottom flange respectively. They are added to the torsionally induced stresses in the output window for stress and compared against yield as is done in the AISC Design Guide. Advanced load options can be specified in a separate window that can be activated with the “Advanced” button. These options are only used if the check button “Use advanced options” is checked. Advanced load options specify where the positive and negative moment region is located in relation to the diaphragms / cross frames. This defines how the torsionally induced stresses are superimposed with the stresses due to non-composite dead loads. If the advanced options are not used, the program assumes that the worst case is present and adds the maximum torsional stress of either diaphragm or between diaphragm location to the stresses due to non-composite dead loads.

The maximum single wheel load of the finishing machine and wheel spacing are required input parameters. Assuming 4 wheels, the user has to specify 3 distances between them to provide the length over which the loads are spread (Figure 4.5 c). In case of more than four wheels, the user should condense the loads of the wheels to

four loads and use the wheel spacing of the four inner wheels of the machine as input making a conservative assumption.

Connection Girder – Diaphragm Data (Location: Main menu – Input – Connection. See Figures 4.6.a – 4.6 b)

A choice of a welded and a bolted connection is offered for the girder – diaphragm connection. The option of a bolted connection is only available if the use of diaphragms is specified in the bridge information window. If a bolted connection is used, the user has to specify the number of bolts, the bolt placement, the bolt diameter, bolt material, bolt hole size, and slip category according to AASHTO Table 10.32.3C. Bolt material is limited to A325 and A490 steel whereas the choices of hole sizes are standard and oversize/short slots.

Temporary Support Data (Location: Main menu – Input – Temp. Support. See Figures 4.7.a – 4.7 b)

Two kinds of temporary support can be applied: tie rods for the top flange, and timber blocking for the bottom flange. The number of supports between diaphragms can be entered for both flanges individually with a maximum of 3 tie rods and 3 timbers. The spacing of the temporary support can be entered in the text boxes provided below, which are enabled after the number of supports has been chosen. The user should pay close attention to the fact that these distances need to add up to

the distance between lateral support as specified in the bridge information window. The distances specified here are used to locate the springs representing the supports in the system. The cross sectional area of both tie rod and timber has to be specified as well. The program uses 200 GPa for the elastic modulus of steel and 12 GPa for the elastic modulus of wood when calculating the equivalent spring stiffness of the supports.

The project is ready for calculation after all the above information has been entered.

4.2.4 Viewing Results

Stress Results (Location: Main menu – Output – Stresses. See Figures 4.8 a – 4.8.b)

The stress results are given for the negative moment region and the positive moment region separately.

Torsionally induced stresses vary linearly across the top and bottom flange, changing from positive to negative sign (minor axis bending). Stresses due to non-composite DL are nearly constant within the top and bottom flanges (major axis bending). To find the governing maximum values, the stresses from minor and major axis bending are superimposed. See Figure 4.8b for a graphical display of the stress distribution due to the torsional load and non-composite DL.

In case the advanced load options are not used, the torsionally induced stress given are the maximum of either the stress at the diaphragm or between diaphragms. In case the advanced options are used, the stresses given are as specified in the advanced load form. Not using the advanced options is always on the safe side whereas using them gives a better picture of the stress distribution at the cost of the need to enter more detailed information. Specifying the advanced load options will superimpose the stresses correctly according to their location along the girder. This can yield a substantial improvement because the stresses between diaphragms are usually around 50 – 60 % of the stresses at the diaphragm.

The text boxes for the sum of stresses will turn red if overall stresses exceed yield stress in order to give a direct visual check when optimizing a project.

Note that the maximum stresses due to torsion for the top and bottom flange may not correspond to the same wheel load location in accordance with the analysis approach described in appendix D. However, they represent the maximum values as the wheel load moves along the girder and are independent of each other.

Ultimate Stress Check (Location: Main menu – Output – Ultimate Stress. See Figures 4.9)

A check of ultimate stresses in the top flange is conducted and the results are presented in this form. The result is the value of the interaction equation (10-155), [AASHTO, 1996], which has to be smaller than 1. Calculations and assumptions

follow the example in the KDOT Bridge Design manual on page 5-35, [KDOT, 1997].

Deflection Results (Location: Main menu – Output - Deflection. See Figures 4.10)

The stiffness method used to analyze both flanges calculates horizontal deflections of the flanges. They are presented on the deflection results form. The rotation of the girder, measured in degrees, and the vertical deflection of the bracket at the screed rail are derived geometrically assuming rigid body motion of the bracket. The calculations assume that the rotation of the girder is small and therefore the horizontal deflection at screed rail is negligible. A sketch of the deflections is included for easy understanding.

Note that the top and bottom flange deflection pertain to the same wheel load location. The results are reported for the largest screed rail deflection.

Bracket Results (Location: Main menu – Output – Bracket Forces. See Figure 4.11)

Internal forces of the brackets are given in the bracket results window. The bottom portion presents the loads one bracket has to carry. All loads are calculated assuming simple spans between brackets. Loads entered in the load form in units of force per unit area are multiplied by the bracket spacing and given in units of force

per unit length. The maximum wheel load that acts on a bracket may be due to more than one or even two wheels and therefore can exceed the wheel load entered in the load form. Note that this load is not equivalent to the wheel load used to derive the stresses and deflections.

The top portion of the form displays a schematic view of the bracket with the forces in each member.

Diaphragm Results (Location: Main menu – Output - Diaphragm. See Figure 4.12)

Results for the diaphragm that supplies the lateral support for the exterior girder include the reaction force couple due to torsion that is applied to the diaphragms by the flanges of the girder, the resulting moment, and the stresses due to this moment.

If bolts are used to connect the diaphragm to the girder the critical load per bolt according to AASHTO Table 10.32.3C, and the actual load of the bolts are given and compared to make a statement as to whether slip in the connection occurs.

A sketch is located on the form for easy understanding of the calculated values.

Summary of Results (Location: Main menu – Output - Summary.)

All results and input information are summarized on a final form. The main text box starts with all the information gathered from the user, then lists all results that have been calculated. The text within the text box can be saved to a file in either

text (.txt) or rich text (.rtf) format. It is recommended to save in rich text format since this preserves the formatting better than the plain text format. All information can also be printed directly from the form using the print option if the printer is set to standard format (TAEG cannot use the print option if the printer is set to postscript format).

4.2.5 Obtaining Help

Help is provided according to Microsoft Help documents that can be viewed within TAEG by using the functional key F1 or other Help options. Help for individual forms can directly be obtained. This brings up the associated help file in a separate browser window. Help can also be obtained from the main menu by choosing Help. This displays the index of available help topics and provides links to the individual subjects.

4.3 Assumptions and Requirements

4.3.1 System Requirements

TAEG is a Visual Basic © application designed for Windows 95 © and Windows NT 4.0 ©. The basic requirements to run these operating systems will be sufficient to run TAEG. The layout of the program has been designed for a screen resolution of 1024 by 768 pixels, but smaller or larger resolutions may also be used. Other restrictions and requirements are not known.

4.3.2 Assumptions within TAEG

4.3.2.1 System

To analyze the torsional response of the exterior girder certain assumptions are made. The main assumption is the use of the flexure analogy as discussed in chapter 3. Another very important assumption is the choice of the loading and beam model and the boundary conditions as described in Appendix D. The program uses three spans, each of the length of the diaphragm spacing, as a continuous beam with the ends fixed and intermediate pinned supports to analyze the torsional loading of the flanges. Refer to Figure D.1 for details. Loads on this beam are the eccentric loads on the overhang converted to a force couple applied at top and bottom flange.

4.3.2.2 Loads

The loads along the beam are divided into three sections. Section 1 includes all overhang loads (DL and LL) and concrete that has been placed, section 2 includes all these loads plus the wheel loads of the finisher, and section 3 that includes only the overhang loads (DL and LL) minus the concrete DL. This is to simulate the placement process as the finisher moves along the bridge. The location of load section 2 is varied within the second span to find the position where it generates the maximum stresses and deflection of the flanges. The individual results for stress, flange deflection, and diaphragm response are maximized independently. This means that the results may relate to different wheel load locations on the beam.

In more detail, this means:

- Torsionally induced stresses for top and bottom flange at the positive and negative moment region may relate to different wheel load locations (See Figure D.1).
- Flange deflections for top and bottom flange are due to one single wheel load location, but this location may differ from the one for the stresses or support reactions.
- Support reactions from top and bottom flange at the diaphragm pertain to the same wheel load location, but this location may differ from the one for the stresses or flange deflections.

All loads are treated as distributed loads including the wheel loads of the finisher. This is in disagreement with the actual conditions where the loads are applied concentrated to the flanges by the brackets. The assumption is justified with a number of insights. First, although bracket spacing is assumed, the actual location and spacing of the brackets are not known or may vary from plans in the field. Second, it is much more complex to analyze the beam with discrete bracket locations and to optimize the location of the wheel load and the brackets than to use a “smeared” load that simulates the discrete bracket forces. Third, the error that enters the calculations is small and on the conservative side. Figure 4.12 compares two identical systems differing only in the load type they carry. One with four concentrated loads in the center (system 1), spaced like the finisher wheels in example

1, and one with the same load distributed over the total width of the before mentioned wheels (system 2). The maximum moment in mid-span for system 1 is 82 kNm and for system 2 is 88 kNm. This represents a 5.3 % difference and indicates that this simplification is justifiable.

4.3.2.3 Lateral Support

The kind and amount of lateral support that is provided to the flanges is specified in the input section. For the top flange, pinned rigid lateral supports are used for both cross frames and diaphragms. For the bottom flange, pinned rigid lateral supports are used for cross frames while for diaphragms these supports are replaced with springs as described in Appendix D.⁸

Temporary support can be specified for both flanges. Tie rods and timber blocking is assumed to continue across the whole bridge. They are treated as springs that support the flanges laterally. Spring stiffness is derived as described in Appendix D where half the width of the bridge is used to determine the effective spring stiffness. This follows the notion that most torsional loads on the exterior girders are symmetrical over the center of the bridge. A result of this is that the tie rods and timbers do not move over the center of the bridge and therefore only one half of the length of them is effective.

⁸ The spring stiffness only accounts for the stiffener below the diaphragm as shown in Fig. D.2. The rotational stiffness of the diaphragm loaded in its major axis is not considered.

In case of one-sided bracket, e.g. for bridge extensions or phased construction, the engineer should consider the effect of the length of the tie rods and timbers. Half of the value that is entered for the bridge width in the bridge data form is used to determine the spring stiffness of the temporary supports. The assumption of symmetric loading and of no movement of the tie-rods/timber over the centerline of the bridge is no longer valid for one-sided brackets. One-sided bracket will weaken the effect of temporary support. Double the length of the actual temporary support should be used for the bridge width in order to account for this.

One-sided brackets also have other effects. In the case eccentric loads of the overhang on one side of the bridge are not equaled out by the eccentric loads of the other side of the bridge, the system receives a total torsional load that has to be carried by the overall structure.

Another issue might be the bracing of the girder top flange. It is necessary to provide adequate bracing to prevent lateral torsional buckling. Detailed information on the lateral bracing can be found in "Fundamentals of Beam Bracing", [Yura, 1993].

Both the total torsional loading of the system and the lateral bracing of the girder are beyond the scope of this research project and are not considered in this report or TAEG.

4.3.2.4 Bolt Slip

Table 10.32.3C of the AASHTO Bridge Specifications, [AASHTO, 1996] is used calculating the critical bolt load in the connection between girder and diaphragms. The actual load of the bolt is calculated assuming that all bolts carry the same load regardless of their distance to the center of the bolt pattern. This is because the connection is considered slip critical and therefore an elastic theory can not be used since this would require slip to occur. Details on this approach can be explored in Salmon & Johnson [Salmon, Johnson, 1996]. An analytical example can be seen in Appendix G.

4.3.2.5 Deflection

The rotation of the girder is derived from the top and bottom flange deflection. A lateral movement of the shear center of the girder is neglected, which would occur in case of unequal deflection of the flanges. The vertical deflection of the screed rail is directly derived from the rotation, assuming rigid body motion of the bracket.

4.3.2.6 Bridge Skew

Bridge skew is not considered in the design aid as discussed in Section 2.1.1.

4.4 Examples and Verification

4.4.1 Example 1. Sample Calculation in the Kansas Department of Transportation Design Manual

General: The KDOT Design Manual, [KDOT Bridge Office, 1997], section 1a (Pages 5-35 to 5-46) features an example calculation for the response of an exterior girder under torsional loads. Information from this example is used to run an analysis with TAEG and to compare the two approaches. Appendix E summarizes the input information and the results using the summary windows listing. Input information has been taken directly from the design manual where possible and derived from the given values where direct transferal was not possible. This refers mainly to the loads that are given as total loads per bracket in the design manual and need to be entered in TAEG as loads per unit area of deck surface. Refer to the footnotes in Appendix E for details on conversion of loads.

Results: Refer to Table 1 for a comparison of results.

Not all results computed by TAEG can be compared to the results gained by the KDOT Design Manual. The results available are restricted to the stresses in top and bottom flange for positive and negative moment regions, and the rotation and top flange deflection of the girder. These results exhibit generally good agreement between the two approaches.

To compare the stresses calculated by TAEG to those calculated using the KDOT design manual, we first divide the results of the KDOT design manual by the

load factor 1.3 which is not considered by TAEG. The difference between them then is that for both top and bottom flanges, in the positive moment region, the stresses found by TEAG will be about 20% more than those found by the KDOT design manual. In the negative moment region, the stresses found by TEAG will be about 20% less than those found by the KDOT design manual. The main reason for the differences in stresses, and therefore in moments, is certainly found in the changed boundary conditions as described in Section 2.1.2.2. Figure B.4 displays the decrease in negative moment calculated using a 3 span versus a 1 span analytic model. Figure B.5 displays the increase in positive moment calculated using a 3 span versus a 1 span analytic model.

The top flange lateral deflection is lower in TAEG than in the KDOT manual. Whereas the KDOT manual derives the top flange deflection from the rotation of the girder using a standard torsion deflection formula, TAEG derives these deflections directly from the stiffness method. The results are nevertheless in good agreement. The rotations are in close proximity of each other, 0.25 for TEAG and 0.17 to 0.46 (with an average of 0.315) for KDOT design manual.

4.4.2 Example 2. Swartz Road Bridge

General: Example 2 is an analysis of the bridge extension of Swartz Road. This bridge's performance has been measured as part of the field testing included in this research project. Stresses for the field-testing are derived from the strain measurements taken during the testing. Temporary support at Swartz Road was

provided only in the form of one 4" by 4" timber in the center of the first span of the 3-span bridge. Analyses for the bridge with and without this timber are run with TAEG and compared with the stress information gathered during field-testing. Appendix F gives detailed information on the parameters used and results.

Results: Close agreement between the stress results generated by TAEG and the field measurements could not be found. Whereas the measured stresses do not exceed approximately 640 psi for positive and negative moments (top and bottom flanges), the stresses calculated by TAEG depend strongly on the location and type of temporary support used.

- If no temporary support is used, the maximum results for top and bottom flanges are approximately identical around 6233 psi (located on the top flange).
- If the timber is used, results for the top flange are almost not effected, whereas the stresses in the bottom flange are decreased significantly to 893 psi (decrease by 86%) for the positive moment and 3015 psi (decrease by 51%) for the negative moment.

Refer to Table 2 for a comparison of the obtained results. A copy of the KDOT spreadsheet with this example's data is enclosed with the program installation under "Example2.xls" for comparison. Stress results compare as predicted by the research. Stresses at the negative moment region predicted with the

spreadsheet are 7600 psi (divided by factor 1.3 is 5846 psi), not considering any temporary support. Stresses calculated by TAEG are 5989 psi for the bottom flange and 6233 psi for the top flange without any tie rods and timbers.

Interpretation:

1. Top Flange: Stress results for the top flange calculated by TAEG are off by a great amount. There are two possible reasons that have to be considered. First, the load conditions may differ from the assumed 10 psf live load on the walkway and slab. Second and more important, the lateral support provided by the formwork may have a substantial impact on the loading of the top flange, leading to much lower stresses than predicted by TAEG.

2. Bottom Flange: The use of timber blocking as temporary support for the bottom flange strongly influences the stress results calculated by TAEG. Considering the timber in mid-span brings the stress results closer to those measured in the field, but they are still off by a significant amount. See Table 2 for details. These differences can only be explained with the uncertainties regarding the load and lateral support not accounted for.

4.4.3 Example 3. K10 Bridge

General: Several test runs with different temporary support schemes were performed. Unfortunately, bad weather invalidated strain gauge readings at the girder

and thus prevented stress data for the torsional loading to be calculated. Deflections that have been measured with prisms are the only data that can be compared with results calculated by TAEG. The temporary support scheme for pass 1 of the tests performed is used to make a comparison with TAEG. Refer to “Field-Testing Of K-10 Bridge Over I-70”, [North, Roddis, 1996] for details on the performed tests. Two tie-rods and timbers are placed between cross frames.

Results: Maximum deflection for the top flange for pass 1 is recorded as about 0.1 in. TAEG calculates the same value as 0.05 in. The bottom flange deflection was measured as 0.12 in. and must be compared to the value of 0.011 in. calculated by TAEG. This difference between measured and calculated deflection for the bottom flange may be due to the fact that the timbers were not wedged tight so that measured bottom flange deflection should be compared to the unblocked case. Using TAEG to find bottom flange deflection with no blocking and 2 tie-rods gives a top flange deflection of 0.58 in. and a bottom flange deflection of 0.178 in. The bottom flange deflection for this case compares only fairly with the one measured in the field. This indicates that the amount of lateral support provided by the timbers is of importance and the uncertainty about timber blocking in this example is the cause of differential results.

A copy of the KDOT spreadsheet with this example's data is enclosed with the program installation under “Example3.xls” for comparison. The spreadsheet predicts

19.4 ksi (divided by 1.3 is 14.923) stress at the negative moment region where TAEG yields 13.454 ksi stress, both without temporary support. This is in good agreement with the conducted research.

5. Conclusions

Research: The conducted research shows that an increase in accuracy of up to 35 % over the AISC Design Guide can be achieved by using a 3-span continuous beam instead of a single span fixed end beam to analyze the torsional behavior of the girder. The use of a 5- span beam will yield no further enhancement over the 3-span model. Furthermore, the research indicates that the flexure analogy is correct and can be used for further computations. The need to handle top and bottom flanges independently became apparent. Dynamic loads due to the movement of the motor carriage or the impact of the concrete during placing are found to be negligible.

Design aid: A design aid based on these findings is established in the form of the Visual Basic © program TAEG (Torsional Analysis of Exterior Girders). Using the 3-span continuous beam system and the stiffness method, the program calculates results for

- 1) Stresses in the flanges for significant locations,
- 2) Ultimate stress check for the top flange,
- 3) Deflections of the flanges,
- 4) Rotation of the girder and deflection at the screed rail,
- 5) Internal forces of the overhang brackets,
- 6) Support reactions and stresses in the diaphragm,

- 7) Bolt load and critical bolt load of the connection between girder and diaphragm, and
- 8) Top flange buckling check for the interaction of a compressive force in the top flange and lateral moment due to torsional loads.

Functionality: The program can be used during the design phase of a project or to easily review falsework schemes submitted by contractors.

Parameters that influence the performance of the exterior girder can be changed and easily evaluated during the design phase without extensive calculations. Proposed falsework schemes are evaluated and improved in a more effective and accurate way using the program. With TAEG, it is now possible to include and easily evaluate temporary support. This function was not available in the KDOT spreadsheet and the AISC Design Guide only gives moment reduction factors for top flange tie-rods, neglecting timber blocking. The program will greatly facilitate and accelerate the work of the engineers at KDOT.

Results: TAEG uses a 3 span fixed end continuous beam analysis model for finding torsional stresses while the AISC Design Guide method uses a less accurate single span fixed end model. Therefore, in comparison to the AISC Design Guide method stress results calculated with TAEG are approximately 20% higher for the positive moment region and approximately 20% lower for the negative moment

region (compare Figure B.4 and B.5). Generally, stresses at the negative moment region govern.

Deflections are in good agreement with both the previous approach and data obtained from field tests.

Examples are run to compare the program with the previous approach and data collected in field-tests. Differences between the program results and the approach based on the AISC Design Guide can be explained with the changes made to the static system used to analyze the girder. Data obtained in field tests is in poor agreement with results calculated by TAEG. This is due to uncertainties in load conditions and to unaccounted lateral support provided by falsework, etc.

General: The research project yields a design tool that gives more accurate results on the behavior of exterior girders due to torsional loads. The program can be used to design and check falsework and diaphragm dimensions for standard composite highway bridges. It brings new functionality in form of the temporary support evaluation.

The increase in accuracy will save cost by eliminating problems due to excessive deflection of the exterior girder that could lead to, in severe cases, the need for a costly deck overlay of the bridge.

Appendix A - Analysis of Boundary conditions

When selecting boundary conditions, the question arises as to whether the diaphragms provide equal support to the top and the bottom flange of the exterior girder for torsion. This appendix investigates two sets of boundary conditions for a typical bridge girder and compares the resulting girder response.

A portion of a typical exterior girder section consisting of three equal lateral spans between diaphragms is modeled using ANSYS 5.3© finite element software. The girder is analyzed with two different sets of support conditions at the intermediate supports. The first set (no-stiffener model) includes 3- dimensional fixed nodes at the web next to top and bottom flange and represents the case of equal support for both flanges. The second set (stiffener model) incorporates a stiffener with nodes fixed at the top flange and two-thirds down from the top flange along the web. This is to approximate the actual support offered by a diaphragm. Refer to Figure A.1, A.2, and A.3 for model geometry and support conditions.

The following describes the modeling assumptions, geometry, boundary conditions, and loads used as well as the results.

Modeling assumptions: The 3-D model is composed of volumes glued together and meshed with the SOLID 45 element. This element has been shown to capture the effects of shear stresses typical for torsional loading [Appendix -I, [Zhao and Roddis, 1996]]. SOLID 45 has eight nodes with translation degrees of freedom in x, y, z directions but no rotational freedom. The mesh is created with user predetermined

element sizes in order to gain equal levels of error for both models and to control the problem size since only a limited version of ANSYS 5.3© is available. The stiffeners in the second model are also modeled as volumes. This gives the two models slightly different meshes. The material properties are set to $E=200 \text{ kN/mm}^2$ ($\approx 29000 \text{ ksi}$), $\nu=0.3$, $G=81 \text{ kN/mm}^2$ ($\approx 11700 \text{ ksi}$).

Geometry: Three spans of equal length of 7.32 m ($\approx 24 \text{ ft.}$) make up the girder with the total length of $L=21.96 \text{ m}$ (72 ft.). The girder section is chosen from section 6 of Renjun Zhao's investigation (see Table 1, [Zhao and Roddis, 1996]). Both flanges are 35 x 413 mm (1.4 x 16 in.) with a web that is 13 mm (0.5 in.) thick. The height of the girder varies from 1130 mm (45 in.) to 2130 mm (84 in.), where the later coincides with the actual height of section 6. In the stiffener model, one stiffener with the thickness of 10 mm and the height of the web is placed on one side of the girder at both third points. These stiffeners are fully attached to the girder (see Figure A.2, Stiffener model).

Boundary conditions: At the end sections ($X=0$, $X=L$) all nodes are completely restrained in all three degrees of freedom in both models. The difference in the two models lies in the support at the third points where the diaphragms frame into the girder. The no-stiffener model is created by fixing the web at the flanges in all three directions (see Figure A.2), thus providing equal support for top and bottom flange. The nodes in the stiffener model are fixed at the stiffener at the top flange and 2/3

down from the top, thus providing support for the bottom flange only via the stiffener (see Figure A.2).

Loads: A torsional load is applied in the form of two lateral pressure loads on the outside face of the flanges. The top flange receives a load on its right face directed in the positive z orientation. A load directed in negative z orientation is applied to the left face of the bottom flange (Figure A.1). The two loads of $p=0.0002$ kN/mm² are continuous over the full length of the girder and generate a torsional moment of 14.70 kN•m/m for the 2130 mm high girder and 7.7 kN•m/m for the 1130 mm high girder. These loads are found to be within the range of loads for actual girders of these sizes. Furthermore, they guarantee that the resulting stresses and strains stay within the elastic region and that the material law is valid. The actual configuration and magnitude of the load is not of further interest since this study only compares the results between different boundary conditions and does not focus on the numerical outcome of the results.

Summary of FE-model data:

ANSYS 5.3©, 3-D analysis with Solid 45, 8-node brick element,

$E=200$ kN/mm², $\nu=0.3$, $G=81$ kN/mm²

Three span I-section, $L=21.96$ m, Flanges: 35 x 400 mm, Web 13 mm x h mm, h varying between 1130 and 2130 mm.

No-stiffener model: all nodes restrained at end sections, web fixed at the third points

Stiffener model: all nodes restrained at end sections, stiffener at third points fixed at top flange and two thirds down from top.

Lateral load of 0.0002 kN/mm^2 in opposite directions at top and bottom flange outer faces

Results:

Only two runs with girder heights of 1130 mm (45 in.) and 2130 mm (84 in.) are performed for the no-stiffener model after it became apparent that little or no change of the results occurs with the variation of the girder height for this model. This can be expected due to the boundary and load conditions and considering the before mentioned flexure analogy. The stiffener model is investigated with girder height between 1130 mm (45 in.) and 2130 mm (84 in.) using 200-mm increments.

Analysis focuses on the flange moments over the y-axis in both top and bottom flanges at the end-section ($X=0$), the center of the outside span ($X=L/6$), the third points (diaphragm support, $X=L/3$), and the center of the middle span ($X=L/2$). The maximum deflection of the flanges in the outside and middle span and the support reactions at the third points are also investigated in detail (Refer to Figure A.1).

The flange moments are computed using the nodal stress solution compiled by ANSYS©. A spreadsheet program is used to convert these nodal results into moments. The deflections and reactions are also derived from the nodal solution.

Flange moments: Figure A.3 shows the distribution of the bottom flange moments for the different girder heights over the X-axis. As the support at the third point softens due to the increasing girder height and distance to the fixed node on the stiffener, the following observations can be made:

- (a) The fixed-end moment at $X=0$ decreases from -30 to -37.5 kN•m.
- (b) The support moment at $X=L/3$ increases by 3.5 kN•m raising the center-span moment at $X=L/2$ by approximately the same magnitude.
- (c) The center-span moment for the outer span at $X=L/6$ decreases by 1.3 kN•m where the effect of the decreasing fixed-end moment overpowers the effect of the increasing support moment.
- (d) The enclosed graph (w/o stiffener 113) for the no-stiffener model exhibits almost identical fixed-end and support moments as well as identical span moments. Furthermore, it is always the lower or higher limit for the values respectively.

All these observations comply with the known stiffness-load distribution relationship. Using the no-stiffener results as a basis, the difference between the no-stiffener model and the model with stiffener is as high as 32.2% for the mid-span moment and 12.5 % for the support moment at the third point.

The top flange moments, shown in Figure A.4, are almost indifferent to the variation of the girder height. No or little difference is observed between the no-

stiffener model and the stiffener model. This is due to the fact that the support conditions for the top flange are identical for both models. This is also another indication that the top flange is not influenced by the bottom flange and that the flexure analogy is correct. The existing differences of up to 3.3% are within the solutions margin of error and can be neglected.

Deflections: The bottom flange deflections in Figure A.5 increase with increasing girder height. This phenomenon is driven by the decreasing stiffness of the third point support. With the maximum increase of deflection at $X=L/2$ at over 280% (again, using the no-stiffener model as the basis), this effect is the most pronounced within the investigated parameters.

The top flange deflections vary only within 1% and verify that the gained understanding of the model is correct. This understanding leads to the reasoning that, within this model, the response of the top flange is indifferent to the girder height. Furthermore, the outer span deflection is about 2% smaller than the center span deflection due to the fully fixed end section at $X=0$, and the boundary conditions for the top flange in the no-stiffener model are equal to those in the stiffener model.

Support reactions: Support reactions at the third points do not exhibit changes greater than the solution variation of the FE analysis over the girder height. The difference between the no-stiffener model and the stiffener model reactions is due to the changed location of the support nodes. The ratio between the distance from top to bottom support is about two thirds and so is the ratio between the support reactions.

The differences between the top and bottom support of the stiffener model of about 2 kN is at around 2% and is negligible.

Comparison to beam results for top flange: In order to assess the ANSYS© results and their accuracy in relation to the flexure analogy, a 3-span beam model for the top flange has been analyzed with the software package RISA 2-D©. The system for the analysis was the before mentioned 3-span with fixed moment conditions at the end sections and pin-support at the third points (see Figure A.8). The section of the flange was subjected to 0.007 kN/mm, the equivalent of the ANSYS© analysis load of 0.0002 kN/mm². Span length and material properties were equal to the parameters used with ANSYS©.

The results of this analysis are displayed in figure A.4 and figure A.6 for moments and displacement respectively. Comparing the results for the no-stiffener model with those obtained by RISA 2-D©, it becomes obvious that good agreement between those two has been found. The differences for the deflections are less than 0.1% and for the moments always less than 5.5%. This confirms the flexure analogy once more and gives confidence for the pursued analysis with ANSYS©.

Appendix B – Determination of the Required Model Size

The decision to abandon the currently used 1-span model with fixed ends in favor of a continuous beam analysis leads to the question of how comprehensive the new multiple span model should be. The choice of the number of spans is based on increasing accuracy while keeping the problem size small. The following analysis is carried out to select the appropriate number of spans.

System: Three models with 1, 3, and 5 spans are analyzed with RISA 2-D © [RISA 2-D, 1993] in order to obtain results for the center span of the respective models. Refer to Figure B.1 for information on section location, system, and support conditions. The mid-span moment (M_2) and deflection (W_2), the support moment for the left (M_B) and right (M_C) side of the center span, and the support reactions left and right of the center span (B, C) are the parameters of interest. The models are fully fixed at their ends and all bearings between have are chosen to pin supports.

The cross section is a rectangle 413 by 35 mm (16 by 1.4 in.) with an elastic modulus of 200 kN/mm² (29,000 ksi) The span length L is 10 000 mm (32.8 ft.).

Loads: The systems are subjected to two different load cases. The first case is a distributed load q extending from X=0 to the coordinate a. The second is a single load P located at a. Influence lines for the result parameters are obtained by varying the coordinate a, in increments of L/5, from X=0 to X=3L (Figure B.1). Both loads are set to unit values of 1 kN/m and 1 kN respectively. The results are entered into a

spreadsheet program for visualization and analysis and are shown in Figure B.2 through Figure B.13.

Results: Multi span beam response values for selected load conditions are tabulated in widely available structural analysis textbooks such as (Schneider, 1996). Results from the RISA 2-D © [RISA 2-D, 1993] model were compared to tabulated values for selected loading to verify the accuracy of the conducted analysis. Exact match was found.

Figures B.2 –B.12 show that in general the 3 and 5-span graphs differ by 5.0 % or less for the center span of interest, while the 1-span model exhibits considerable differences of up to an order of magnitude from the 3-span model. The percentile difference, shown in the data table of each figure, uses the model with the smaller number of spans as a basis. Note that the percentage difference may be misleading where the curves cross the x-axis. For values close to zero, the percentage difference may be a high value suggesting a large difference although the values are actually in close proximity close to the x-axis.

It is important to notice that the values for the reaction B due to the distributed load q for the 1-span (Figure B.2) are not really comparable to those for the other spans. This is due to the fact that the values for the 3 and 5-span represent the shear forces from the right and the left side of the support. The 1-span values represent right side shear forces only. Results for reaction C can be compared since the load is only on the left side.

The results show that for both load cases a considerable difference between 1 and 3-span exists, but that the difference between 3 and 5-span models are smaller by about an order of magnitude. It shows that no substantial improvement for the future analysis in the design aid is gained by using a 5-span over a 3-span model since only small change occurred between the 3 and 5-span models. Even smaller change would be expected between a 5 and 7-span model.

Appendix C – Problem Description

I. Project Title

Torsion of Exterior Girders of a Steel Girder Bridge During Concrete Deck Placement.

II. Principal Investigator

W. M. Kim Roddis, Associate Professor, Civil Engineering, University of Kansas

III. Research Objectives

Deflection of the exterior girders due to torsional loading caused by the screed and concrete load during deck placement lowers the screed rail and screed, thereby producing a thinner deck and insufficient concrete cover to the top of the reinforcing steel. KDOT currently uses an in-house computer program to predict the torsional response of fascia girders to concrete placement loads, however this method is not as accurate as is desired due to a lack of information about both the loads and the restraint. Design information is not available on the loads applied by the screeds commonly used on KDOT bridges, resulting in major uncertainty with respect to magnitude and location of applied loads. Structural information on degree of torsional restraint provided by fascia girder abutment and pier supports, cross frames, and bent plate diaphragms are also known only very approximately. These two sources of uncertainty result in possible major variation between current design assumptions and actual conditions, leading to uncertainty in design and possible unintended over or under design.

The objectives of this project are to:

- Document the typical screed being used by contractors, clarifying the load side of the equation with respect to magnitude, location, and range.
- Determine the torsional restraint and structural response, analyzing the behavioral response of the fascia girder to concrete placement loads.
- Establish the proper transverse support system relative to girder sizes, girder spacing, span lengths and deck thickness and overhang, providing improved fascia girder torsional design.

IV. Workplan Summary

Objective 1 will be accomplished by surveying contractors to determine equipment types, gathering contractor and manufacturer information on screed loading, and performing testing to measure screed loads in the field.

Objective 2 requires the acquisition of field data on several KDOT bridges during construction to measure torsional response to concrete placement loads. Since the torsional response of fascia girders is of interest to KDOT for both rolled beams and plate-girder bridges, the proposed testing program will involve a minimum of 3 bridges: one rolled beam and two plate-girders. All test bridges are to be selected by KDOT depending on construction schedule and should be representative of typical KDOT design. The results of the field testing will be used to verify and calibrate an analysis program written to determine the fascia girder's torsional response to concrete placement loads. The program development will begin prior to the field-testing. Use can be made of rotational stiffness data already collected by KDOT on one structure (I-470 west of Topeka), for preliminary program testing.

Objective 3 will take the analytic program and extend it to perform the design function of calculating the proper size, spacing, and connection details for transverse attachments to the fascia girder. This Design program will be the primary deliverable item from this project, with an accompanying final project report. The report will include recommendations on when and how to use provided design aids. The report will be delivered to KDOT unbound and camera ready. Reproduction will be done by KDOT. Twenty (20) copies of the bound report will be provided to the investigator by KDOT. User needs will be taken into account so that the final program will dovetail with the design needs of the KDOT Bridge Design Department.

Appendix D – Stiffness Method used in TAEG

The following shows the principal use of the stiffness method within the design aid TAEG. The analytic background of the procedures used in TAEG is described and discussed to give a general understanding of the program and the ability to judge the results adequate. This appendix contains information on the static system used in TEAG, the derivation of spring constants of tie-rods, timber blocking, and the diaphragm stiffener. It also explains some special aspects of the use of the stiffness method, the methods used to solve the equation system, and how the results are determined.

Using the stiffness method, the following equation system has to be solved

$$\underline{\mathbf{K}} * \underline{\mathbf{V}} = \underline{\mathbf{P}}$$

$\underline{\mathbf{K}}$ = Stiffness matrix

$\underline{\mathbf{V}}$ = Vector of Deformations

$\underline{\mathbf{P}}$ = Vector of Loads

for the system shown in Figure D.1.

System: Figure D.1 shows the 3-span system with all the applied loads and the boundary conditions as it is used within the design aid TAEG. This system is used to investigate the top flange as well as the bottom flange. In the case of the bottom flange, diaphragm springs are substituted for the supports A and B. These springs represent the stiffness of the web stiffener that connects the diaphragm to the girder as mentioned in section 3. Figure D.1 shows also springs between the supports. These springs represent a case with one tie-rod/timber between lateral supports.

The distributed loads shown are divided into 3 sections. They are chosen to simulate the load distribution during the placing process. The center section includes all loads, which are formwork DL, walkway LL, slab LL, concrete DL and wheel loads. The loads on the left side consist of the all above loads but the wheel loads. The loads on the right are the before mentioned minus the wheel load and concrete DL.

In order to find the position of the wheel loads that generate the maximum stresses and deflections they are moved along the beam between support A and B. Results include the maximum stresses of either support A and B, stresses at the center span, deflection at around the center span and support reaction at support A and B.

Derivation of the spring stiffness C_t , C_w , C_s :

- C_t , spring stiffness of the tie rods attached to the top flange.

$$C_t = \frac{E_t * A_t}{l_b}$$

with

E_t = Modulus of elasticity of steel (200 GPa)

l_b = half the distance between exterior girders

A_t = Cross-section of the tie rod

- C_w , spring stiffness of the timber blocking at the bottom flange.

$$C_w = \frac{E_w * A_w}{l_b}$$

with

E_w = Modulus of elasticity of wood (12 GPa)

l_b = half the distance between exterior girders

A_w = Cross-section of the timber

The length l_b is chosen as half the distance between the two exterior girders. The assumption that the tie-rods and timbers do not move over the centerline of the bridge implies that the loads applied on the system are mostly symmetrical. This seems reasonable since only the weight of the motor carriage is unsymmetrical depending on its location.

- C_s , spring stiffness of the stiffeners at the diaphragms

A simplified model that represents the conditions at the girder – diaphragm connection can be used to determine the spring stiffness of the diaphragm stiffener. Refer to Figure D.2 for detailed information on this system and the assumptions made.

Using the integral:

$$\Delta = \int [(M_1 M_0) / (EI)] dx$$

the deflection of the tip of the stiffener due to a unit force can be found as:

$$\Delta = l^3 / (3EI) \quad \text{In units of length per force.}$$

Conversion to the force necessary to deflect the tip one unit length will yield:

$$C_s = \frac{3EI}{l^3} . \quad \text{In units of force per length.}$$

Solving the equation system: Since the system is a one-dimensional beam with no longitudinal loads the axial deformation elements of the stiffness matrix are neglected. The individual element stiffness matrices are of the dimension 4 X 4. The

one-dimensionality of the system also means that the stiffness matrix \underline{K} is a symmetrical matrix with a maximum bandwidth of 7 and a maximum dimension of 2 times n , where n represents the number of nodes of the system. The nature of the stiffness matrix holds also that no zero elements are placed on the diagonal, which makes it possible to use a simple Gauss-Jordan elimination without pivoting.

Results: The results gathered from the procedure include stresses at the positive and negative moment region, deflection inside the center span, and support reaction at the supports A and B. Results are given for top and bottom flanges. They are maximized by moving the wheel load section from support A to support B in increments of 1/10 of the diaphragm spacing. The other loads, namely the loads of the sections left and right of the wheel load, are extended or shortened accordingly. Refer to Figure D.1 for load position and maximization scheme. Stress results at the negative moment region are the maximum of either support A or B. The resulting maximum stresses for top and bottom flanges do not necessary belong to the same wheel load position as opposed to the deflection and support reaction (diaphragm load) results where top and bottom flange results always belong to the same wheel load position. The deflection of the overhang at the finishing machine rail is geometrically derived from the two lateral flange deflections assuming rigid body motion of the bracket.

Appendix E – Example 1 Data Summary

TEAG - Torsional Analysis for Girders Version 1.0 - Summary Report for
EXAMPLE1.prj

////////////////////////////////// INPUT DATA //////////////////////////////////////

PROJECT AND FILE INFORMATION

Input file name :EXAMPLE1.prj
Location :C:\teag\EXAMPLE1.prj
KDOT Project # :KANSAS DEPARTMENT OF TRANSPORTATION

DESIGN MANUAL

Engineer :MARK KRIESTEN
Project Title :EXAMPLE 1
Last Modified :7/4/99 12:12:16 AM
Created :6/24/99 2:32:23 PM
System of Units :S.I.
Notes :

GIRDER DIMENSIONS AND MATERIAL PROPERTIES

Top Flange (Width X Thickness) :300 X 25 [mm]
Bottom Flange (Width X Thickness) :400 X 40 [mm]
Web (Width X Height) :1300 X 10 [mm]
Yield Stress :250 [MPa]
Modulus of Elasticity :200000 [MPa]

BRIDGE AND LATERAL SUPPORT DATA

Distance between lateral Supports :6300 [mm]
Bridge Skew :0 [Degrees]
Bridge Width :13400 [mm]
Diaphragms :NO
Diaphragm Moment of Inertia : [mm⁴]
Diaphragm Height : [mm]
Diaphragm Yield Stress : [MPa]
Diaphragm Modulus of Elasticity : [MPa]
Diaphragm Delta :0 [mm]
Stiffener Width : [mm]
Stiffener Thickness :0 [mm]

BRACKET DIMENSION

Walkway Width (Bracket Dim. B)	:760 [mm]
Bracket Spacing	:900 [mm]
Bracket Weight	:22.6 [kg]
Bracket Dimension A	:1920 [mm]
Bracket Dimension C	:900 [mm]
Bracket Dimension D	:29.7 [Degree]
Bracket Dimension F	:130 [mm]
Bracket Dimension G	:915 [mm]

LOAD DATA

LL Walkway	:2.4 [KPa]
LL Slab	:2.4 [KPa]
DL Formwork	:0.244 [KPa]
DL Concrete	:5.413 [KPa]
Top flange stress/positive moment	:0 [MPa]
Bottom flange stress/positive moment	:0 [MPa]
Top flange stress/negative moment	:0 [MPa]
Bottom flange stress/negative moment	:0 [MPa]
Maximum Wheel Load	:3.75 [kN]
Wheel Spacing [1 - 2 - 3]	:600 - 1200 - 600 [mm]

ADVANCED LOAD OPTIONS

Use advanced load options? : YES
 Stresses given for the positive moment region at centers between diaphragms.
 Stresses given for the negative moment region are at a diaphragm

CONNECTION DETAILS

Bolted Connection	: NO
Number of Bolts	: []
Distance Top of Diaphr. to 1 st Bolt	: [mm]
Bolt Spacing	: [mm]
Bolt Diameter	: [in.]
Bolt Material	: A325
Bolt Hole Size	: Standard
Slip Category	: Class A

TEMPORARY SUPPORT INFORMATION

Number of Tie Rods : NONE

Number of Timbers :NONE

//////////////////////////////////OUTPUTDATA //////////////////////////////////////

STRESS OUTPUT
POSITIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [MPa]
Top Flange Stresses due to torsion : - 48.07 [MPa]
Top Flange Stresses total Sum : - 48.07 [MPa]

Bottom Flange Stresses due to non-comp DL : 0.00 [MPa]
Bottom Flange Stresses due to torsion : 16.90 [MPa]
Bottom Flange Stresses total Sum : 16.90 [MPa]

NEGATIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [MPa]
Top Flange Stresses due to torsion : 64.91 [MPa]
Top Flange Stresses total Sum : 64.91 [MPa]

Bottom Flange Stresses due to non-comp DL : 0.00 [MPa]
Bottom Flange Stresses due to torsion : - 22.82 [MPa]
Bottom Flange Stresses total Sum : - 22.82 [MPa]

Compare to yield at : 250.00 [MPa]

ULTIMATE STRESS CHECK

Check for ultimate strength using eq. (10-155) AASHTO. $0.09 < 1$!

DEFORMATION OUTPUT

Lateral Top Flange Deflection (VT) : 4.533 [mm]
Lateral Bottom Flange Deflection (VB) : 1.195 [mm]
Vertical Deflection of the Rail (UR) : 4.15 [mm]
Rotation of the Girder (Theta) : 0.25 [Degree]

DIAPHRAGM OUTPUT

Lateral Support Reaction at the Top Flange (Ft) : 44.05 [kN]

Lateral Support Reaction at the Bottom Flange (Ft) : 44.05 [kN]
 Resulting Moment acting on the Diaphragm : 58.69 [kNm]
 Max. Stress in the Diaphr. due Torsional Load (M) : [MPa]
 Max. Bolt Load due Torsional Load (M) : [kN]
 Critical Bolt Load acc. to AASHTO Table 10.32.3 C : [kN]
 Slip of Bolts :

BRACKET FORCES OUTPUT

Hanger Force	: 40.96 [kN]
Horizontal Force (Top)	: 27.70 [kN]
Horizontal Force (Bottom)	: 27.70 [kN]
Vertical Force	: 28.96 [kN]
Diagonal Force	: 31.89 [kN]
LL Walkway	: 2.16 [kN/m]
DL Form Work	: 0.22 [kN/m]
LL Slab	: 2.16 [kN/m]
LL Concrete	: 4.87 [kN/m]
Wheelload	: 20.35 [kN/m]

Appendix F – Example 2 Data Summary

TEAG - Torsional Analysis for Girders Version 1.0 - Summary Report for
EXAMPLE2.prj

//////////////////////////////////INPUTDATA //////////////////////////////////////

PROJECT AND FILE INFORMATION

Input file name :EXAMPLE2.prj
Location :C:\teag\EXAMPLE2.prj
KDOT Project # :SWARTZ ROAD EXTENSION
Engineer :MARK KRIESTEN
Project Title :EXAMPLE 2
Last Modified :6/24/99 8:34:59 PM
Created :6/24/99 8:33:23 PM
System of Units :U.S. Customary
Notes :

GIRDER DIMENSIONS AND MATERIAL PROPERTIES

Top Flange (Width X Thickness) :16 X 1.25 [in.]
Bottom Flange (Width X Thickness) :16 X 1.25 [in.]
Web (Width X Height) :32 X 0.312 [in.]
Yield Stress :36 [ksi]
Modulus of Elasticity :29000 [ksi]

BRIDGE AND LATERAL SUPPORT DATA

Distance between lateral Supports :218 [in.]
Bridge Skew :42.4 [Degrees]
Bridge Width :312 [in.]
Diaphragms :YES
Diaphragm Moment of Inertia :243 [in.^4]
Diaphragm Height :18 [in.]
Diaphragm Yield Stress :36 [ksi]
Diaphragm Modulus of Elasticity :29000 [ksi]
Diaphragm Delta :8 [in.]
Stiffener Width :4.344 [in.]
Stiffener Thickness :0.375 [in.]

BRACKET DIMENSION

Walkway Width (Bracket Dim. B) :24 [in.]
Bracket Spacing :48 [in.]

Bracket Weight	:50 [lb]
Bracket Dimension A	:76.5 [in.]
Bracket Dimension C	:52.5 [in.]
Bracket Dimension D	:21.413 [Degree]
Bracket Dimension F	:0 [in]
Bracket Dimension G	:30 [in.]

LOAD DATA

LL Walkway	:10 [psf]
LL Slab	:10 [psf]
DL Formwork	:13.4 [psf]
DL Concrete	:106.25 [psf]
Top flange stress/positive moment	:0 [psi]
Bottom flange stress/positive moment	:0 [psi]
Top flange stress/negative moment	:0 [psi]
Bottom flange stress/negative moment	:0 [psi]
Maximum Wheel Load	:1.337 [kips]
Wheel Spacing [1 - 2 - 3]	:21 - 64 - 18 [in.]

ADVANCED LOAD OPTIONS

Use advanced load options? : YES
 Stresses given for the positive moment region at centers between diaphragms
 Stresses given for the negative moment region are at a diaphragm

CONNECTION DETAILS

Bolted Connection	: NO
Number of Bolts	: []
Distance Top of Diaphr. to 1 st Bolt	: [in.]
Bolt Spacing	: [in.]
Bolt Diameter	: [in.]
Bolt Material	: A325
Bolt Hole Size	: Standard
Slip Category	: Class A

TEMPORARY SUPPORT INFORMATION

Number of Tie Rods	: NONE
Number of Timbers	: ONE

Distance between Diaphragms and the tie rod :109 - 109[in.]

Crosssectional Area of the Timbers :16 [in.^2]

////////////////////////////////OUTPUTDATA////////////////////////////////////

STRESS OUTPUT

POSITIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [psi]
Top Flange Stresses due to torsion : - 5816.83 [psi]
Top Flange Stresses total Sum : - 5816.83 [psi]

Bottom Flange Stresses due to non-comp DL : 0.00 [psi]
Bottom Flange Stresses due to torsion : 893.11 [psi]
Bottom Flange Stresses total Sum : 893.11 [psi]

NEGATIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [psi]
Top Flange Stresses due to torsion : 6233.12 [psi]
Top Flange Stresses total Sum : 6233.12 [psi]

Bottom Flange Stresses due to non-comp DL : 0.00 [psi]
Bottom Flange Stresses due to torsion : - 3015.43 [psi]
Bottom Flange Stresses total Sum : - 3015.43 [psi]

Compare to yield at : 36000.56 [psi]

ULTIMATE STRESS CHECK

Check for ultimate strength using eq. (10-155) AASHTO. $0.07 < 1$!

DEFORMATION OUTPUT

Lateral Top Flange Deflection (VT) : 0.092 [in]
Lateral Bottom Flange Deflection (VB) : 0.043 [in]
Vertical Deflection of the Rail (UR) : 0.21 [in]
Rotation of the Girder (Theta) : 0.23 [Degree]

DIAPHRAGM OUTPUT

Lateral Support Reaction at the Top Flange (Ft) : 16.84 [Kips]

Lateral Support Reaction at the Bottom Flange (Ft) : 12.24 [Kips]
 Resulting Moment acting on the Diaphragm : 40.28 [Kip-ft]
 Max. Stress in the Diaphr. due Torsional Load (M) : [psi]
 Max. Bolt Load due Torsional Load (M) : [Kips]
 Critical Bolt Load acc. to AASHTO Table 10.32.3 C : [Kips]
 Slip of Bolts :

BRACKET FORCES OUTPUT

Hanger Force : 10.01 [kips]
 Horizontal Force (Top) : 10.44 [kips]
 Horizontal Force (Bottom) : 10.44 [kips]
 Vertical Force : 7.08 [kips]
 Diagonal Force : 11.21 [kips]

LL Walkway : 0.04 [Kips/ft]
 DL Form Work : 0.05 [Kips/ft]
 LL Slab : 0.04 [Kips/ft]
 LL Concrete : 0.42 [Kips/ft]
 Wheelload : 1.39 [Kips/ft]

Appendix G – Example 3 Data Summary

TEAG - Torsional Analysis for Girders Version 1.0 - Summary Report for
EXAMPLE3.prj

//////////////////////////////////INPUTDATA //////////////////////////////////////
PROJECT AND FILE INFORMATION

Input file name :EXAMPLE3.prj
Location :C:\teag\EXAMPLE3.prj
KDOT Project # :K-10 BRIDGE
Engineer :MARK KRIESTEN
Project Title :EXAMPLE 3, K-10 BRIDGE
Last Modified :6/24/99 9:50:37 PM
Created :6/24/99 8:36:43 PM
System of Units :U.S. Customary
Notes :

GIRDER DIMENSIONS AND MATERIAL PROPERTIES

Top Flange (Width X Thickness) :12 X 1 [in.]
Bottom Flange (Width X Thickness) :16 X 1.375 [in.]
Web (Width X Height) :54 X 0.4375 [in.]
Yield Stress :50 [ksi]
Modulus of Elasticity :29000 [ksi]

BRIDGE AND LATERAL SUPPORT DATA

Distance between lateral Supports :288 [in.]
Bridge Skew :0 [Degrees]
Bridge Width :108 [in.]
Diaphragms :NO
Diaphragm Moment of Inertia :5345 [in.^4]
Diaphragm Height :5 [in.]
Diaphragm Yield Stress :5 [ksi]
Diaphragm Modulus of Elasticity :5 [ksi]
Diaphragm Delta :5 [in.]
Stiffener Width :5 [in.]
Stiffener Thickness :50 [in.]

BRACKET DIMENSION

Walkway Width (Bracket Dim. B)	:24 [in.]
Bracket Spacing	:48 [in.]
Bracket Weight	:50 [lb]
Bracket Dimension A	:64 [in.]
Bracket Dimension C	:40 [in.]
Bracket Dimension D	:25.1148 [Degree]
Bracket Dimension F	:0 [in]
Bracket Dimension G	:30 [in.]

LOAD DATA

LL Walkway	:10 [psf]
LL Slab	:10 [psf]
DL Formwork	:13 [psf]
DL Concrete	:87.5 [psf]
Top flange stress/positive moment	:0 [psi]
Bottom flange stress/positive moment	:0 [psi]
Top flange stress/negative moment	:0 [psi]
Bottom flange stress/negative moment	:0 [psi]
Maximum Wheel Load	:3.05 [kips]
Wheel Spacing [1 - 2 - 3]	:24 - 48 - 24 [in.]

ADVANCED LOAD OPTIONS

Use advanced load options? : YES
 Stresses given for the positive moment region at centers between diaphragms
 Stresses given for the negative moment region are at a diaphragm

CONNECTION DETAILS

Bolted Connection	: NO
Number of Bolts	: []
Distance Top of Diaphr. to 1 st Bolt	: [in.]
Bolt Spacing	: [in.]
Bolt Diameter	: [in.]
Bolt Material	: A325
Bolt Hole Size	: Standard
Slip Category	: Class A

TEMPORARY SUPPORT INFORMATION

Number of Tie Rods :TWO
 Distance between Diaphragms and the tie rod :115 - 58 - 115[in.]
 Crosssectional Area of the Tie Rods :0.2 [in.^2]

Number of Timbers :TWO
 Distance between Diaphragms and the tie rod :115 - 58 - 115[in.]
 Crosssectional Area of the Timbers :16 [in.^2]

//////////////////////////////////// OUTPUT DATA
 //////////////////////////////////////

STRESS OUTPUT

POSITIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [psi]
 Top Flange Stresses due to torsion : - 2173.75 [psi]
 Top Flange Stresses total Sum : - 2173.75 [psi]

Bottom Flange Stresses due to non-comp DL : 0.00 [psi]
 Bottom Flange Stresses due to torsion : 715.85 [psi]
 Bottom Flange Stresses total Sum : 715.85 [psi]

NEGATIVE MOMENT REGION

Top Flange Stresses due to non-comp DL : 0.00 [psi]
 Top Flange Stresses due to torsion : 4883.08 [psi]
 Top Flange Stresses total Sum : 4883.08 [psi]

Bottom Flange Stresses due to non-comp DL : 0.00 [psi]
 Bottom Flange Stresses due to torsion : - 1850.70 [psi]
 Bottom Flange Stresses total Sum : - 1850.70 [psi]

Compare to yield at : 50000.78 [psi]

ULTIMATE STRESS CHECK

Check for ultimate strength using eq. (10-155) AASHTO. $0.01 < 1$!

DEFORMATION OUTPUT

Lateral Top Flange Deflection (VT)	: 0.050 [in]
Lateral Bottom Flange Deflection (VB)	: 0.011 [in]
Vertical Deflection of the Rail (UR)	: 0.04 [in]
Rotation of the Girder (Theta)	: 0.06 [Degree]

DIAPHRAGM OUTPUT

Lateral Support Reaction at the Top Flange (Ft)	: 8.77 [Kips]
Lateral Support Reaction at the Bottom Flange (Ft)	: 8.54 [Kips]
Resulting Moment acting on the Diaphragm	: 39.80 [Kip-ft]
Max. Stress in the Diaphr. due Torsional Load (M)	: [psi]
Max. Bolt Load due Torsional Load (M)	: [Kips]
Critical Bolt Load acc. to AASHTO Table 10.32.3 C	: [Kips]
Slip of Bolts	:

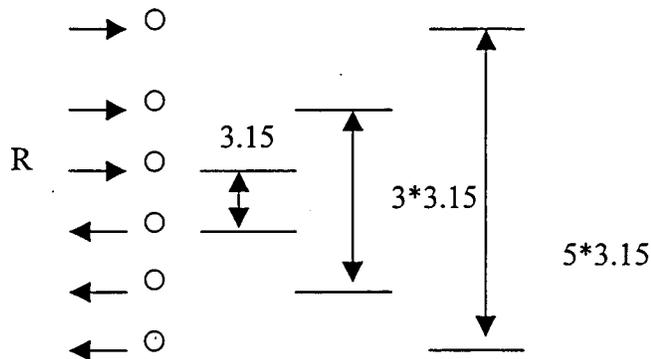
BRACKET FORCES OUTPUT

Hanger Force	: 8.88 [kips]
Horizontal Force (Top)	: 7.44 [kips]
Horizontal Force (Bottom)	: 7.44 [kips]
Vertical Force	: 6.28 [kips]
Diagonal Force	: 8.21 [kips]
LL Walkway	: 0.04 [Kips/ft]
DL Form Work	: 0.05 [Kips/ft]
LL Slab	: 0.04 [Kips/ft]
LL Concrete	: 0.35 [Kips/ft]
Wheelload	: 1.39 [Kips/ft]

Appendix H –Analytical Example for High Stress Bolt Slip

Table 10.32.3C of the AASHTO Bridge Specifications, [AASHTO, 1996] is used calculating the critical bolt load in the connection between girder and diaphragms. The actual load of the bolt is calculated assuming that all bolts carry the same load regardless of their distance to the center of the bolt pattern. This is because the connection is considered slip critical and therefore an elastic theory can not be used since this would require slip to occur. Details on this approach can be explored in Salmon & Johnson [Salmon, Johnson , 1996].

Given six 3/4 in. bolts in one row connecting a diaphragm and girder, with a bolt spacing of 3.15 in., and an applied moment of 40 k-ft, check for bolt slip.



From AASHTO Table 10.32.3.C, for class A, ASTM A325, oversized or slotted holes, the slip stress is $F_s=13$ ksi

Because for $\phi=3/4$, the sectional areas $A = 0.4418$ in², thus the bolt capacity is

$$P_s = F_s \cdot A_b = 13 \text{ ksi} \cdot 0.4418 \text{ in}^2 = 5.7434 \text{ k}$$

On the other hand, the external stress for all bolts is

$$R = 40 \text{ k-ft} \cdot 12 / (3.15 + 3.15 \cdot 3 + 3.15 \cdot 5)$$

$$R = 16.9312 \text{ k} \gg 5.7434 \text{ k}$$

So, slip will occur.

References

- [1] Kansas Department of Transportation, *Torsion of Exterior Girders of a Steel Girder Bridge During Concrete Deck Placement*. Problem Description. (Enclosed as Appendix C).
- [2] John Jones and Todd M. LaTorella. *Torsional Analysis Of Exterior Girders Subjected To Construction Loading (TAOEGSTCL)*, Version 1.3, KDOT, Topeka, December, 1990, revised December, 1994.
- [3] AISC Marketing, Inc., *Design for Concrete Deck Overhang Loads*. Final Report, AISC Marketing, INC., June 15, 1990.
- [4] Renjun Zhao and W. M. Kim Roddis, *Steel Bridge Exterior Girder Response To Concrete Deck Placement Loads*. SL Research Report 96-4, The University of Kansas, Center for Research, Inc., November 1996.
- [5] Multiple Authors, Editor Klaus-Jürgen Schneider, *Schneider Bautabellen (in German)*, 12. Edition, Werner-Verlag, 1996.
- [6] John G. Richardson, *Formwork Construction and Practice*. Viewpoint Publications by the Cement and Concrete Association, Wexham Springs, Great Britain, 1977.
- [7] Joseph J. Waddell, Joseph A. Dobrowolski, *Concrete Construction Handbook*.

- 3rd Edition, McGraw-Hill, New York, 1993.
- [8] M. K. Hurd, ACI Committee 347, *Formwork For Concrete*. Sixth Edition, American Concrete Institute, Detroit, 1995.
- [9] Robert L. Peurifoy, Garold D. Oberlender, *Formwork for Concrete Structures*. Third Edition, McGraw-Hill, New York, 1995.
- [10] R. G. Kinnear et al., *The pressure of Concrete on Formwork*. CERA Report No 1. April 1965, pp. 44.
- [11] *RISA 2-D Version 3.00, Two Dimensional Structural Analysis Program*. RISA Technologies, Forest Lake CA, USA, 1993.
- [12] Bidwell Corporation, *Concrete Finishing Machines*, A Division of MMI Corporation, Canton, SD 57013, USA.
- [13] Tom. H. Cooke, *Concrete pumping and spraying*. Thomas Telford Ltd., London, 1990.
- [14] AASHTO, *Standard Specifications for Highway Bridges*. AASHTO, Washington, 16th Edition, 1996.
- [15] KDOT Bridge Office, *Design Manual Bridge Section – Metric*, Kansas Department of Transportation Internal Document, Volume 3, Topeka, April 1997

- [16] Salmon, Charles G., Johnson, John E. *Steel Structures: Design and behavior*. 4th Edition, Harper-Collins college Publications, New York, 1996.
- [17] Roddis, W.M. Kim, North, Thomas L., Zhao, Renjun. *Field Testing Of K-10 Bridge Over I-70*. The University of Kansas, unpublished
- [18] Joseph A. Yura, *Fundamentals of Beam Bracing, Is Your Structure Suitably Braced?*, Structural Stability Research Council, 1993 Conference Milwaukee, WI, April 6-7, 1993, 20 pp.

Figures

The image shows a software dialog box titled "Project Information". It contains several input fields and control elements:

- KDOT Project #:** A single-line text input field.
- Engineer:** A single-line text input field.
- Project Title:** A single-line text input field.
- Last Modified:** A single-line text input field.
- Created:** A single-line text input field.
- Units:** A label with the text "SI metric" next to it.
- Notes:** A multi-line text area with scroll bars on the right side.
- Buttons:** Three buttons are stacked vertically on the right side: "Print Form", "Cancel", and "O.K.". Above the "Print Form" button is the text "F1 for Help".

Figure 4.1 Project Data Form

Girder Data

Use this form to enter girder dimensions and grade of steel

Top Flange Dimensions
 Width: [mm] Thickness: [mm]

Web Dimensions
 Height: [mm] Thickness: [mm]

Bottom Flange Dimensions
 Width: [mm] Thickness: [mm]

Steel Grade
 Yield stress: [MPa] Modulus, E: [MPa]

F1 for Help

Print Form

Cancel

O.K.

Figure 4.2 a Girder Data Form

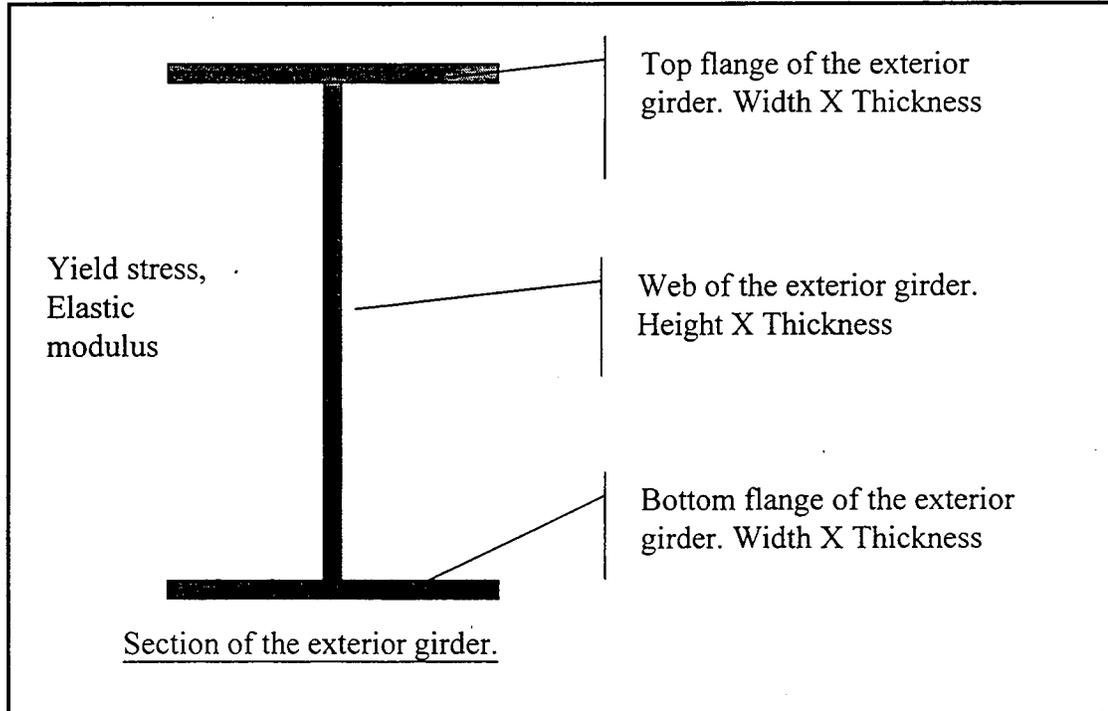


Figure 4.2 b Girder Properties

Bridge and Lateral Support Data

General Bridge Dimensions

Dist. between lateral supports: [mm]

Distance between the two ext. girder: [mm]

Lateral Support

Cross Frames Choose the permanent lateral bracing system

Diaphragms

Diaphragm Ix: [mm⁴] Stiffener Width: [mm]

Height, h: [mm] Stiffener Thickness: [mm]

Yield stress, F_y: [MPa]

Modulus, E: [MPa]

Top Offset: [mm]

Use this form to enter general bridge dimensions and information on lateral support

F1 for Help

Print Form

Cancel

O.K.

Figure 4.3 a Bridge Data Form

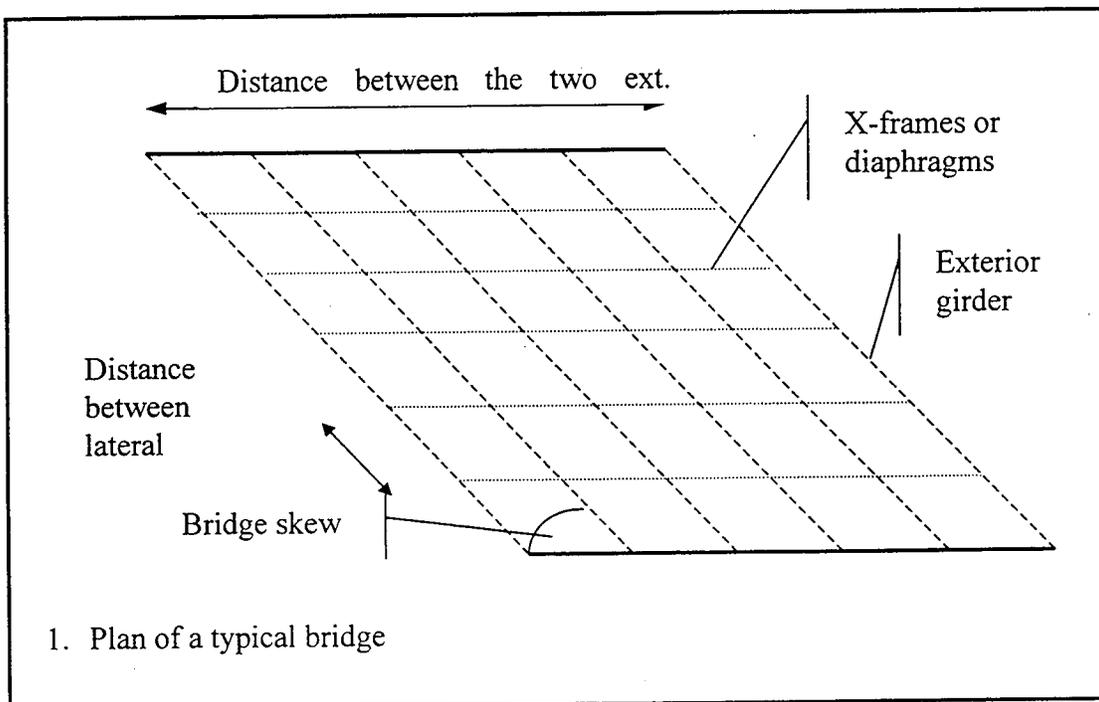


Figure 4.3.b Bridge Measures

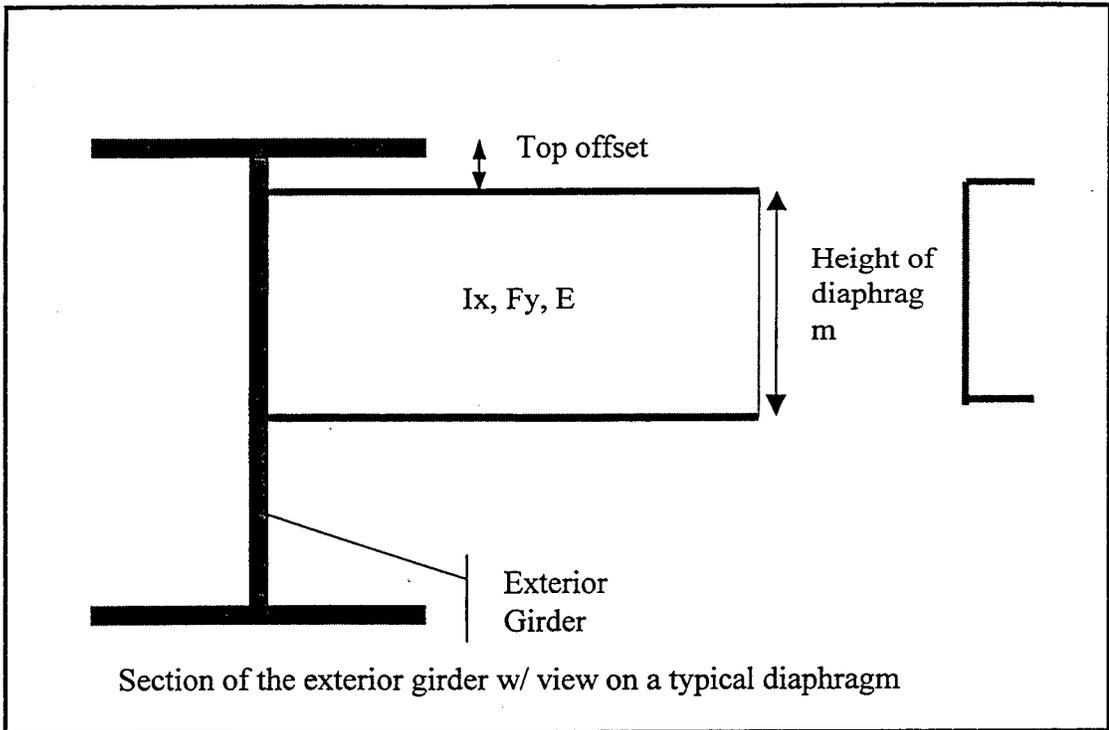


Figure 4.3.c Diaphragm Details

Bracket Data			Use this form to enter bracket dimensions and weight. F1 for Help <input type="button" value="Print Form"/> <input type="button" value="Cancel"/> <input type="button" value="OK"/>
Walkway width (B):	<input type="text"/>	[mm] Distance of walkway.	
Bracket spacing:	<input type="text"/>	[mm] Distance c-c of brackets along the girder.	
Bracket weight:	<input type="text"/>	[kg] Use manufacturers information for bracket weight.	
Bracket dimension A:	<input type="text"/>	[mm] Distance from centerline of girder to end of bracket.	
Bracket dimension C:	<input type="text"/>	[mm] Distance from centerline girder to sreed rail.	
Bracket dimension D:	<input type="text"/>	[Deg] angle between slab and strut.	
Bracket dimension F:	<input type="text"/>	[mm] width of safe forms along the walkway.	
Bracket dimension G:	<input type="text"/>	[mm] Vertical distance from the strut point to top of girder.	

Figure 4.4 a Bracket Data Form

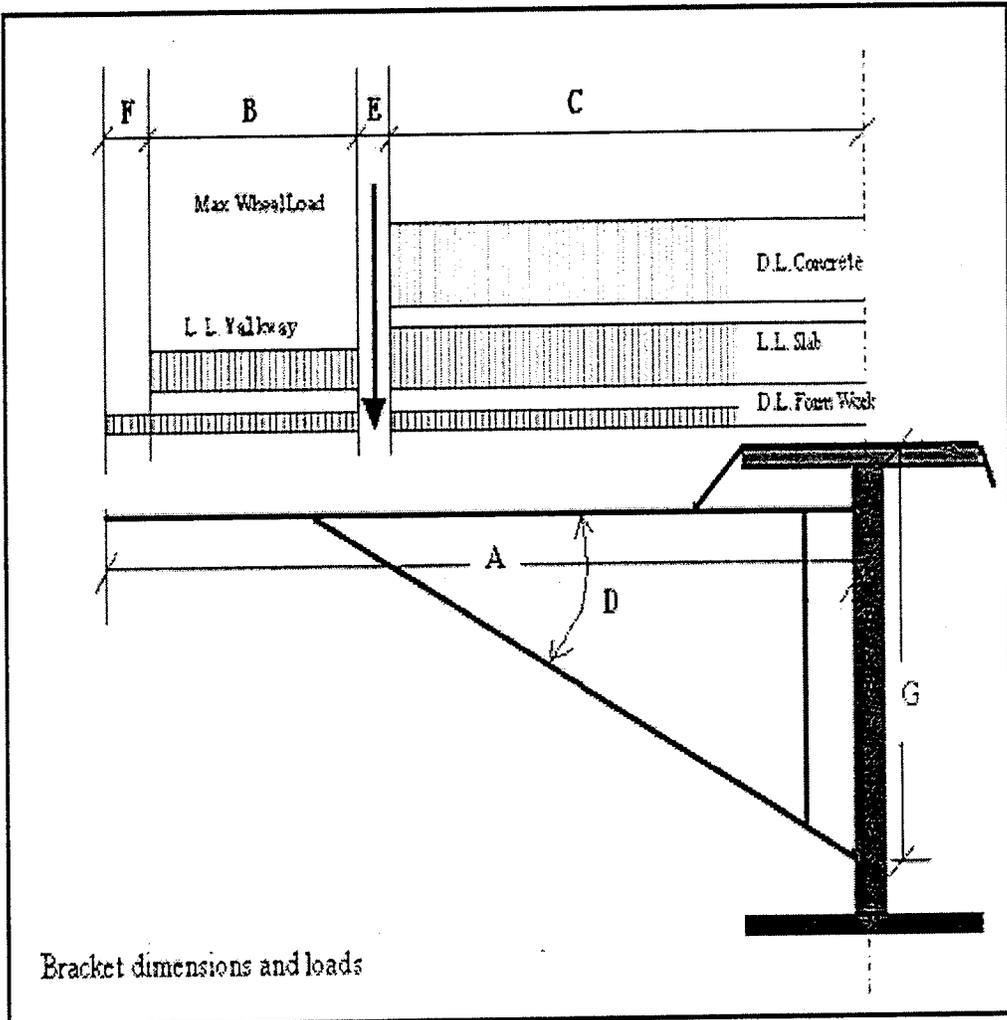


Figure 4.4 b Bracket Dimensions

Load Data

Live Loads
 L.L. Walkway: [] [kPa] L.L. Slab: [] [kPa]

Dead Loads
 D.L. Formwork: [] [kPa] D.L. Concrete: [] [kPa]

Stresses Due To Noncomposite Dead Load
Due to the maximum positive moment Due to the maximum negative moment

At the top flange: [] [MPa] At the top flange: [] [MPa] Use advanced options

At the bottom flange: [] [MPa] At the bottom flange: [] [MPa]

Finishing Machine Loads
 Max. wheel load: [] [kN] Assuming four wheels, give three distances between them

Wheel spacing: [] - [] - [] [mm]

Use this form to enter all load data
 F1 for Help

Figure 4.5 a Load Data Form

Advanced Load Options

Positive Moment Region

Stresses given for the positive moment region are at a diaphragm / crossframe

Stresses given for the positive moment region are at the center between diaphragms / crossframes

Negative Moment Region

Stresses given for the negative moment region are at a diaphragm / crossframe

Stresses given for the negative moment region are at the center between diaphragms / crossframes

Figure 4.5.b Advanced Load Options

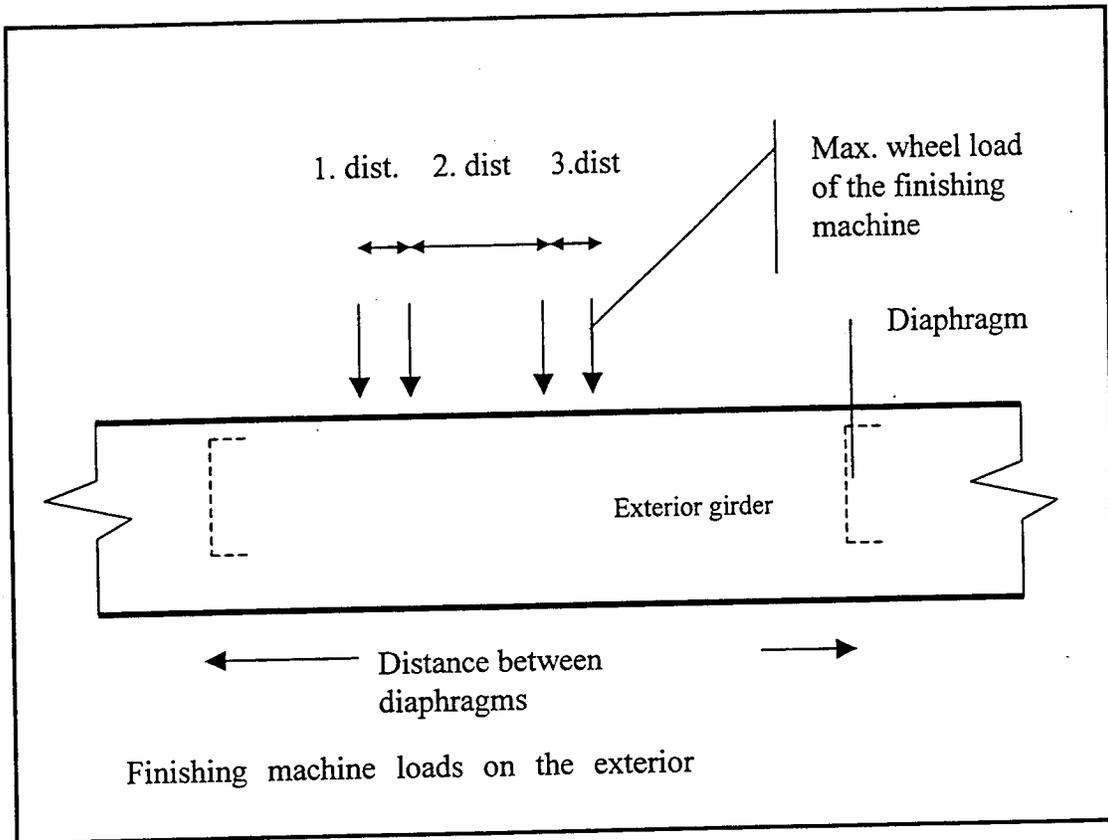


Figure 4.5.c Wheel Loads

--- Connection Girder - Diaphragm Data

Connection Details

Welded connection Choose the connection type

Bolted connection: Only enabled if diaphragms are used!

Number of bolts: [] Bolt diameter: [in.]

Top to first bolt: [mm]

Bolt spacing: [mm]

Bolt Material: A325 A490

Bolt Hole Size: Standard Oversize and short slot

Slip Category based on AASHTO: Class A Class B Class C

Use this form to enter details on the connection between girder and diaphragm

Help

Print Form

Cancel

OK

Figure 4.6 a Connection Data Form

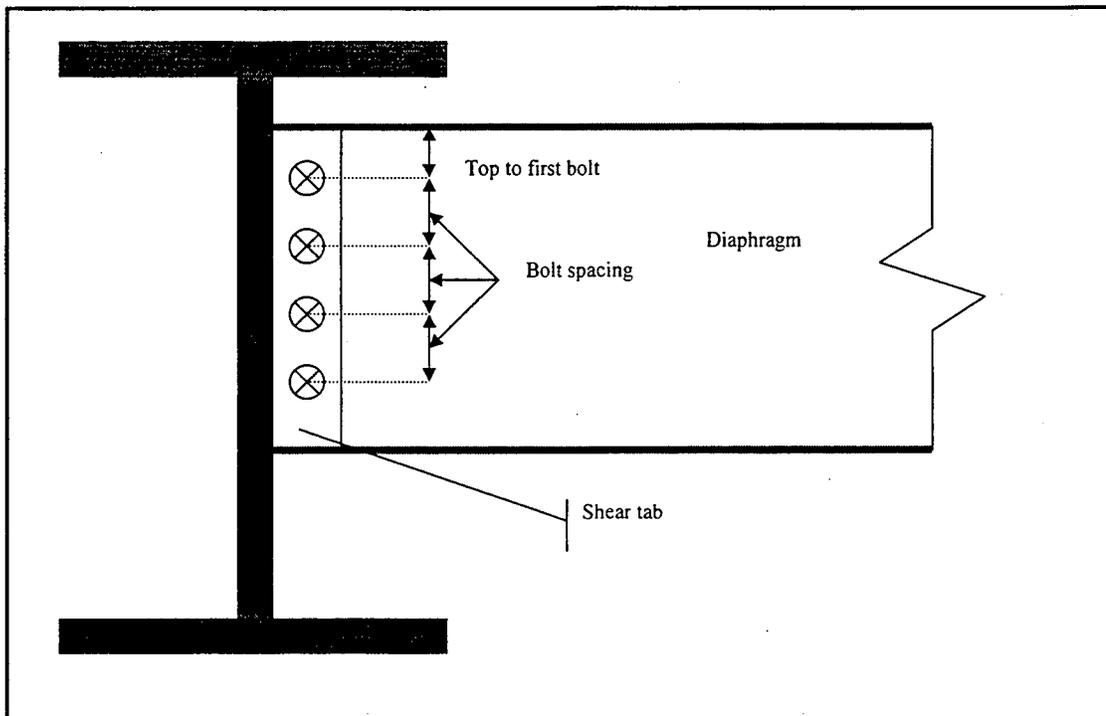


Figure 4.6 b Connection Details

Temporary Support Data

Tie Rods

of tie-rods: none one two three

Spacing between lateral supports: [mm]

Cross-sectional area: [mm²] Enter the spacing between diaphragms / crossframes and temporary steel tie-rods.

Timber Blocking

of timber blks: none one two three

Spacing between Lateral Supports: [mm]

Cross-sectional area: [mm²] Enter the spacing between diaphragms / crossframes and temporary wood timber blocking

Use this form to enter number and location of temporary supports

F1 for Help

Print Form

Cancel

OK

Figure 4.7 a Temporary Support Form

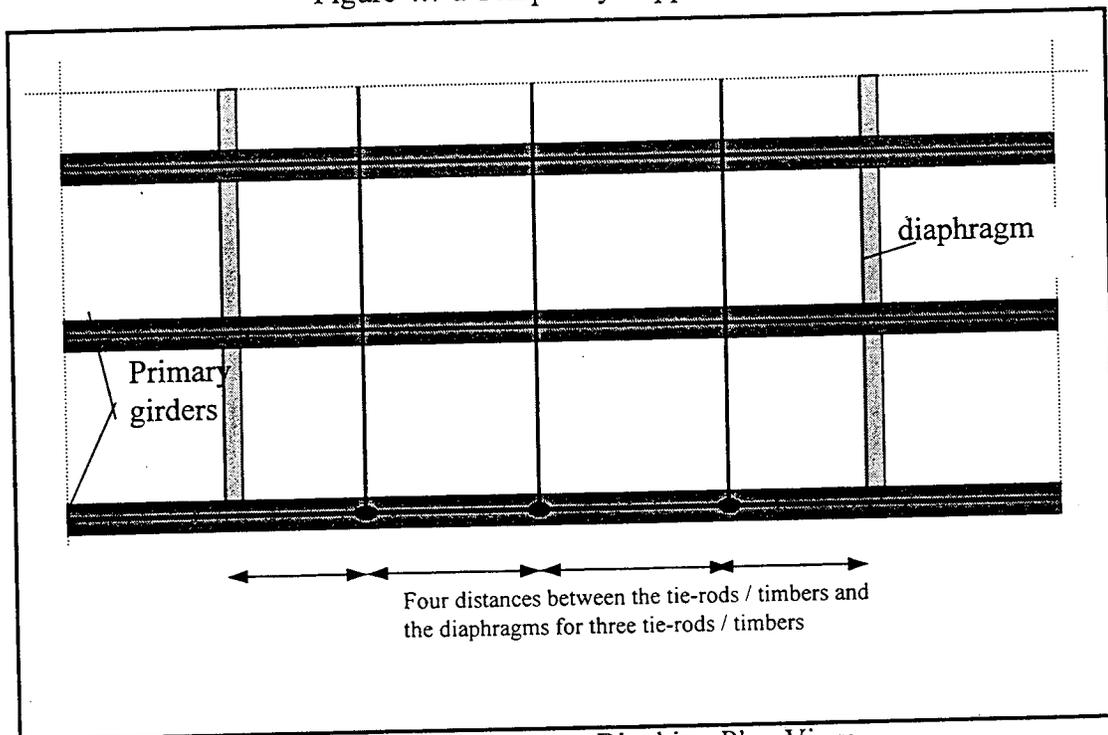


Figure 4.7 b Temporary Blocking Plan View

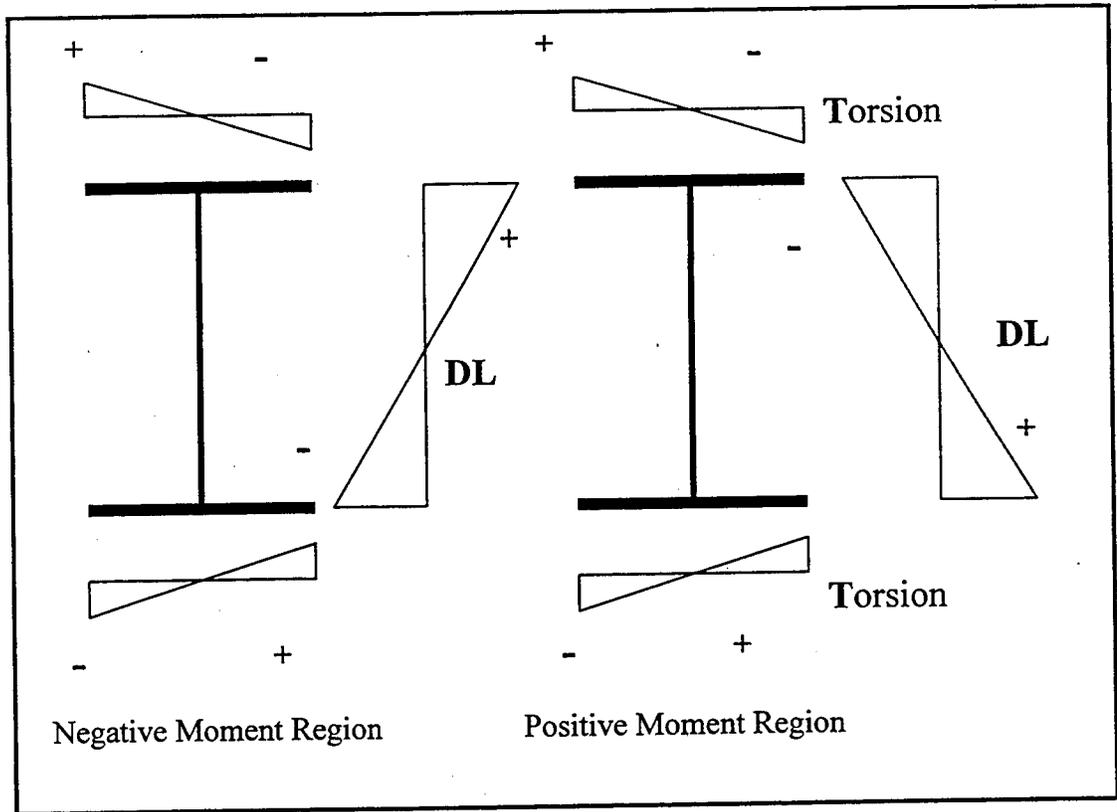


Figure 4.8 a Stress Results Form

Calculated Stresses
☰ □ ✕

Stress Distribution at the Positive Moment Region

Top Flange:

Due to non-comp. DL: [MPa]

Due to torsion: [MPa]

Sum: [MPa]

Bottom Flange:

Due to non-comp. DL: [MPa]

Due to torsion: [MPa]

Sum: [MPa]

Compare to yield at: [MPa]

Stress Distribution at the Negative Moment Region

Top Flange:

Due to non-comp. DL: [MPa]

Due to torsion: [MPa]

Sum: [MPa]

Bottom Flange:

Due to non-comp. DL: [MPa]

Due to torsion: [MPa]

Sum: [MPa]

Compare to yield at: [MPa]

Torsional stresses are the maximum of support and mid-span location

F1 for Help

Figure 4.8 b Stress Distribution due to torsion and non-composite DL

Ultimate Strength Check

Ultimate Strength Of The Top Flange

Check for ultimate strength using equation $\leq 1!$

Print Form

Cancel

OK

Figure 4.9 Ultimate Stress Check Form

Calculated Deflections

Deflection of the Girder and the Bracket Top

Lateral Top Flange Deflection (VT) [mm]

Vertical Deflection of the Bracket at the Rail (UR) [mm]

Lateral Bottom Flange Deflection (VB) [mm]

Rotation of the Girder (Theta) [Degree]

Machine Rail

U_R

V_T

θ

V_B

Sub

Print Form

OK

Figure 4.10 Deflection Results Form

Bracket Forces and Loads per Bracket

Bracket Forces

Hanger Force: [kN]

Vertical Force: [kN]

Diagonal(strut) Force: [kN]

Hor. Force: [kN]

Hor. Force: [kN]

Internal Forces of the Brackets under the maximum loads given below

F1 for Help

Loads per unit Bracket Width:

LL Walkway: [kN/m] DL Concrete: [kN/m]

DL Form Work: [kN/m] Wheel Load: [kN]

LL Slab: [kN/m]

Print Form

Cancel

OK

Figure 4.11 Bracket Forces Result Form

Calculated Diaphragm Response

Diaphragm Response

Lateral support reaction of the top flange (Ft): [kN]

Lateral support reaction of the bottom flange (Fb): [kN]

Resulting moment acting on the diaphragm (M): [kN.m]

Maximum stress in the diaphragm due to torsional load (M): [MPa]

Max. load of one bolt due to torsional load (M): [kN]

Critical bolt load acc. to AASHTO Table 10.32.3 C: [kN]

Sip of the bolts due to (M):

Girder-Diaphragm Connection

Ft

Fb

Stress due to M

F1 for Help

Print Form

OK

Figure 4.12 Diaphragm Results Form

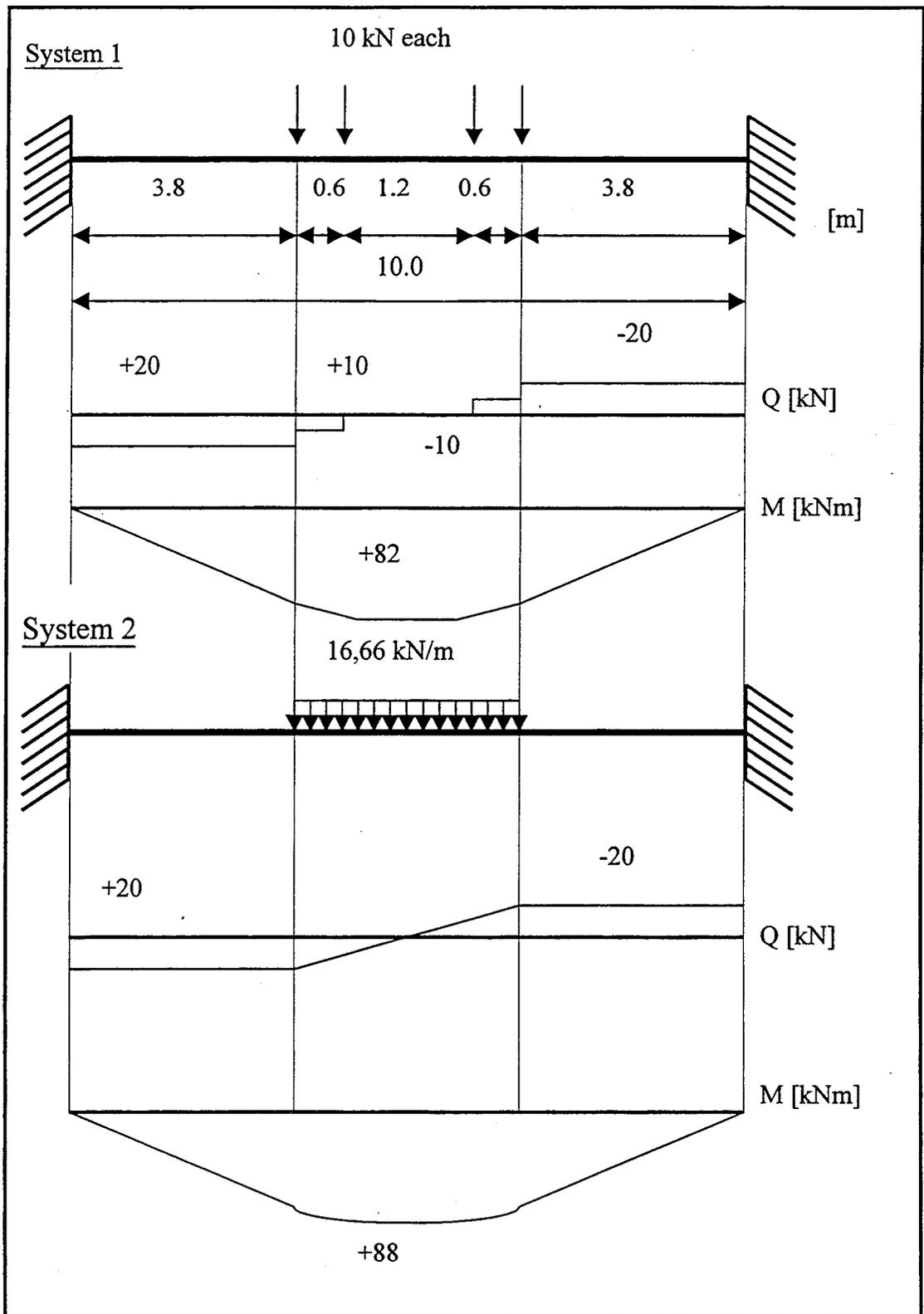


Figure 4.13 Load Type Comparison

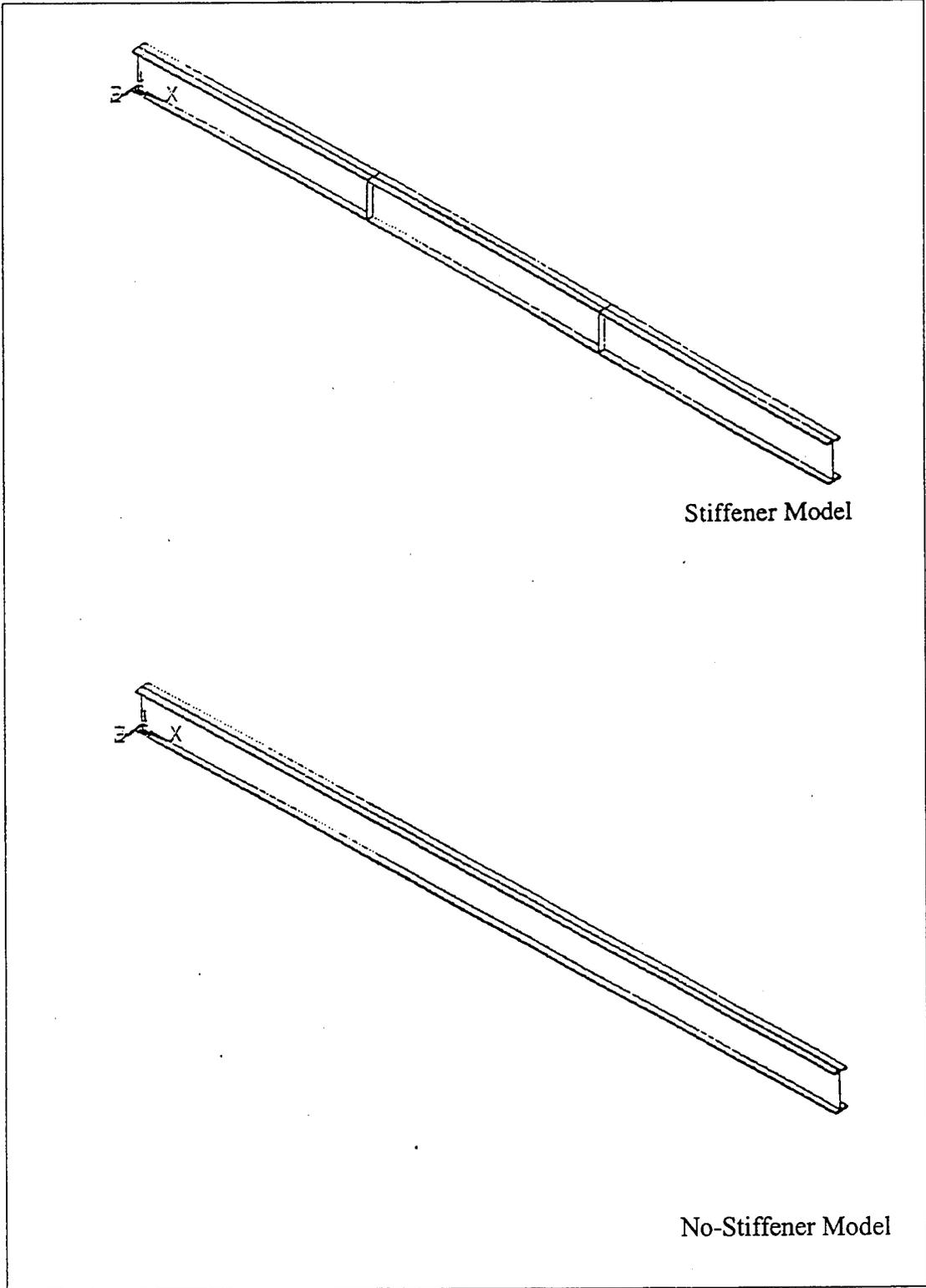


Figure A.1 3-D ANSYS ©Models

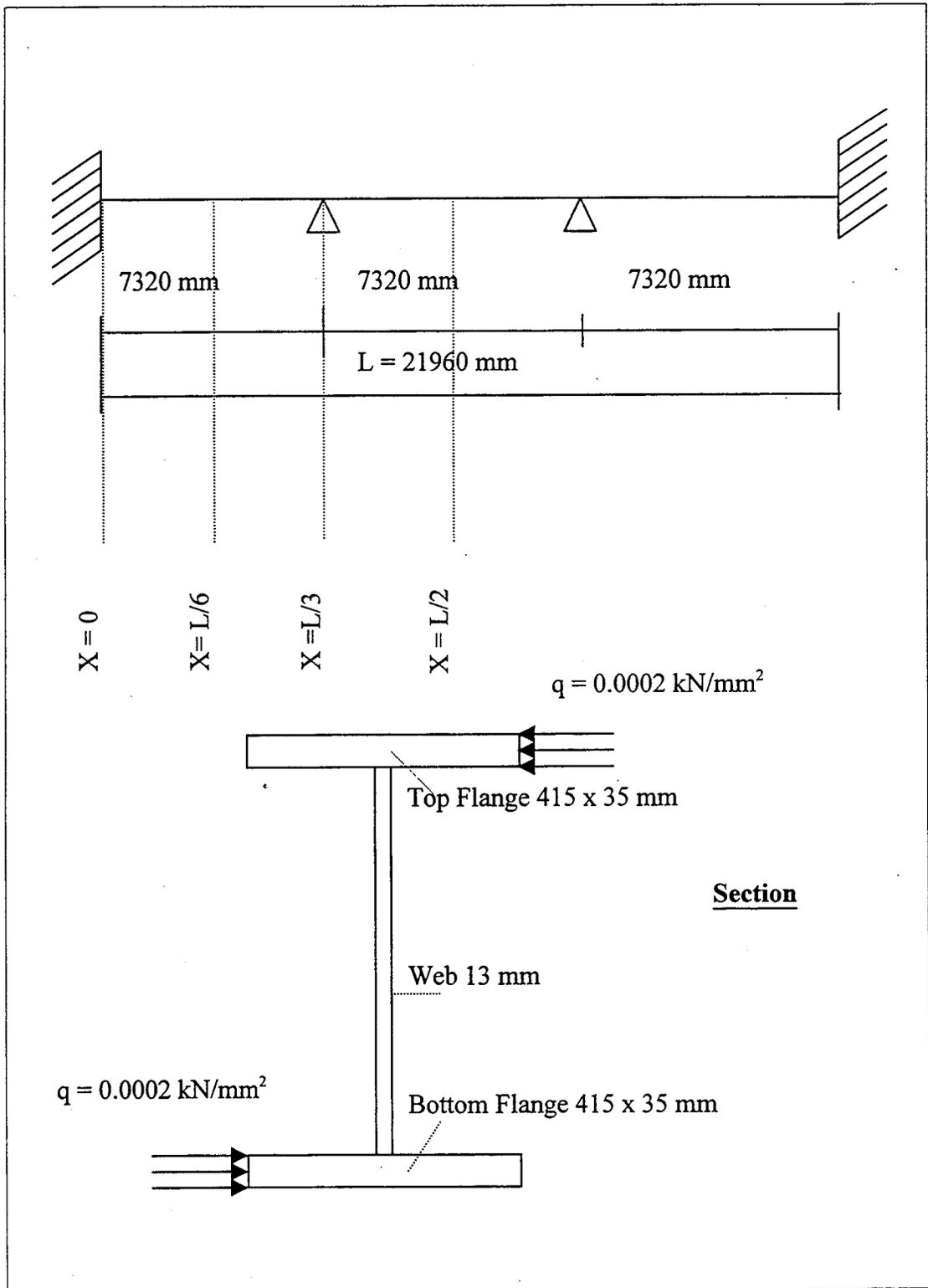


Figure A.2 System for ANSYS ©Analysis and Section Locations

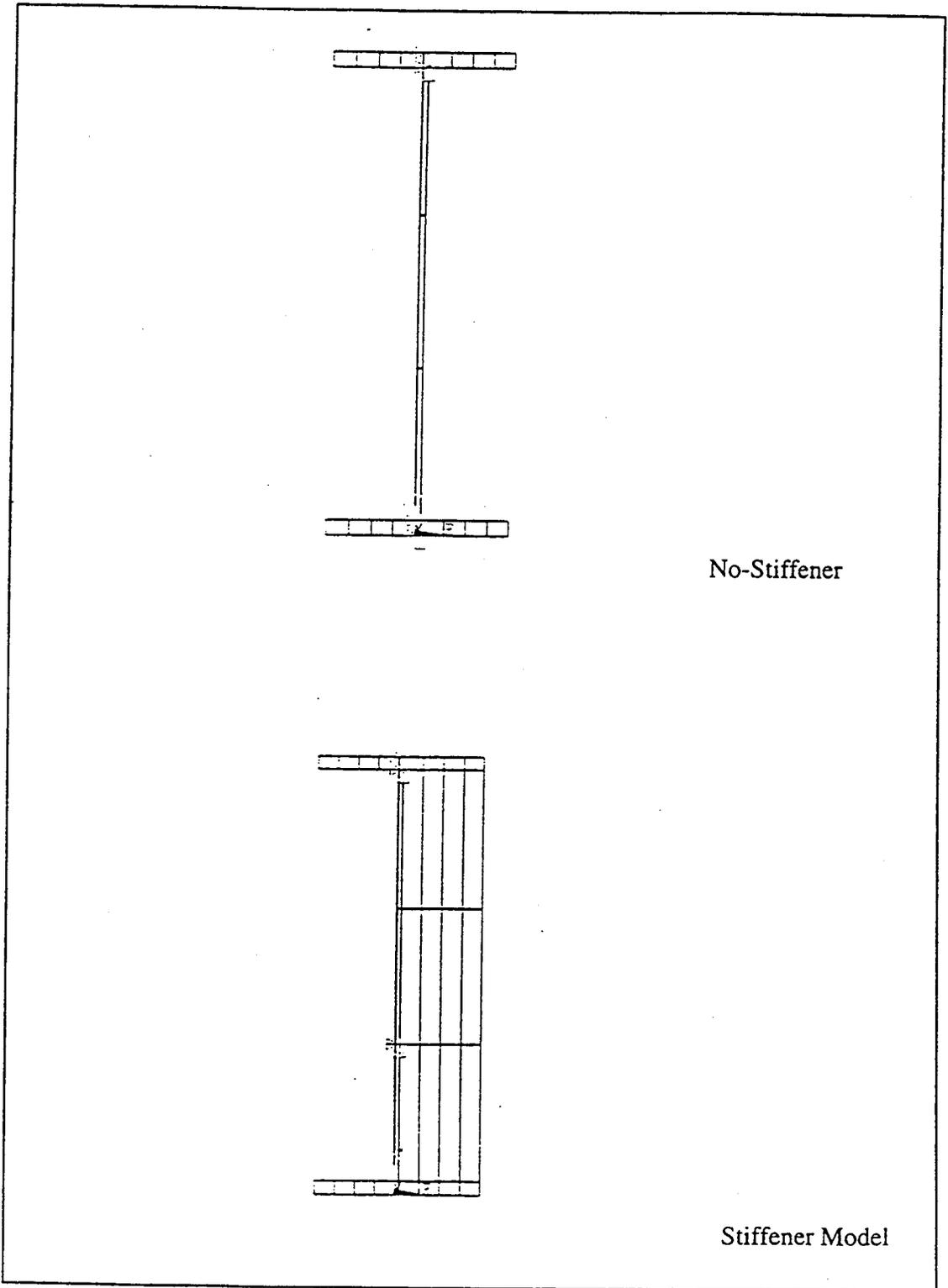


Figure A.3 Boundary Conditions at Third Points

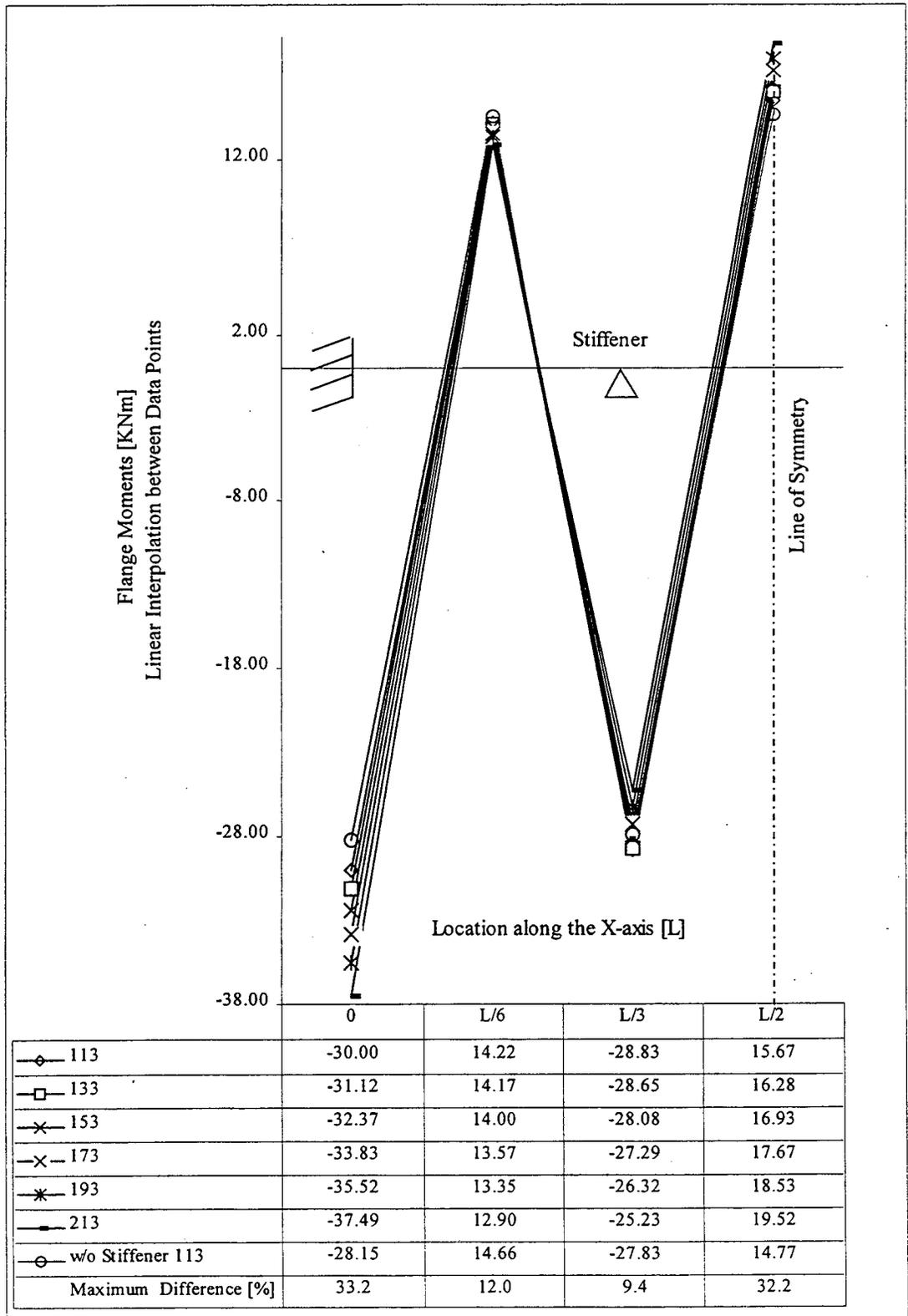


Figure A.4 Bottom Flange Moments due to Distributed Load-Couple q

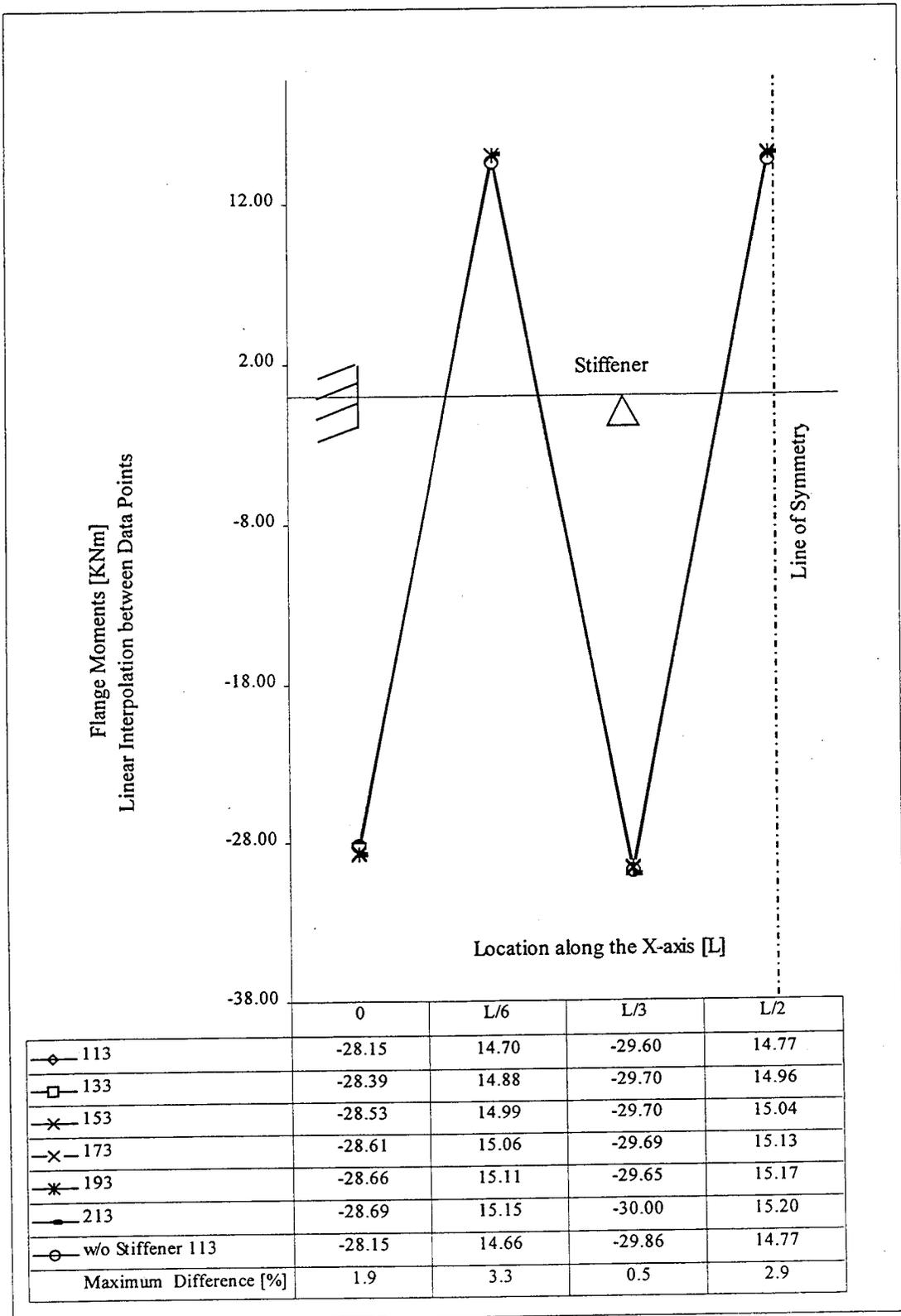


Figure A.5 Top Flange Moments due to Distributed Load-Couple q

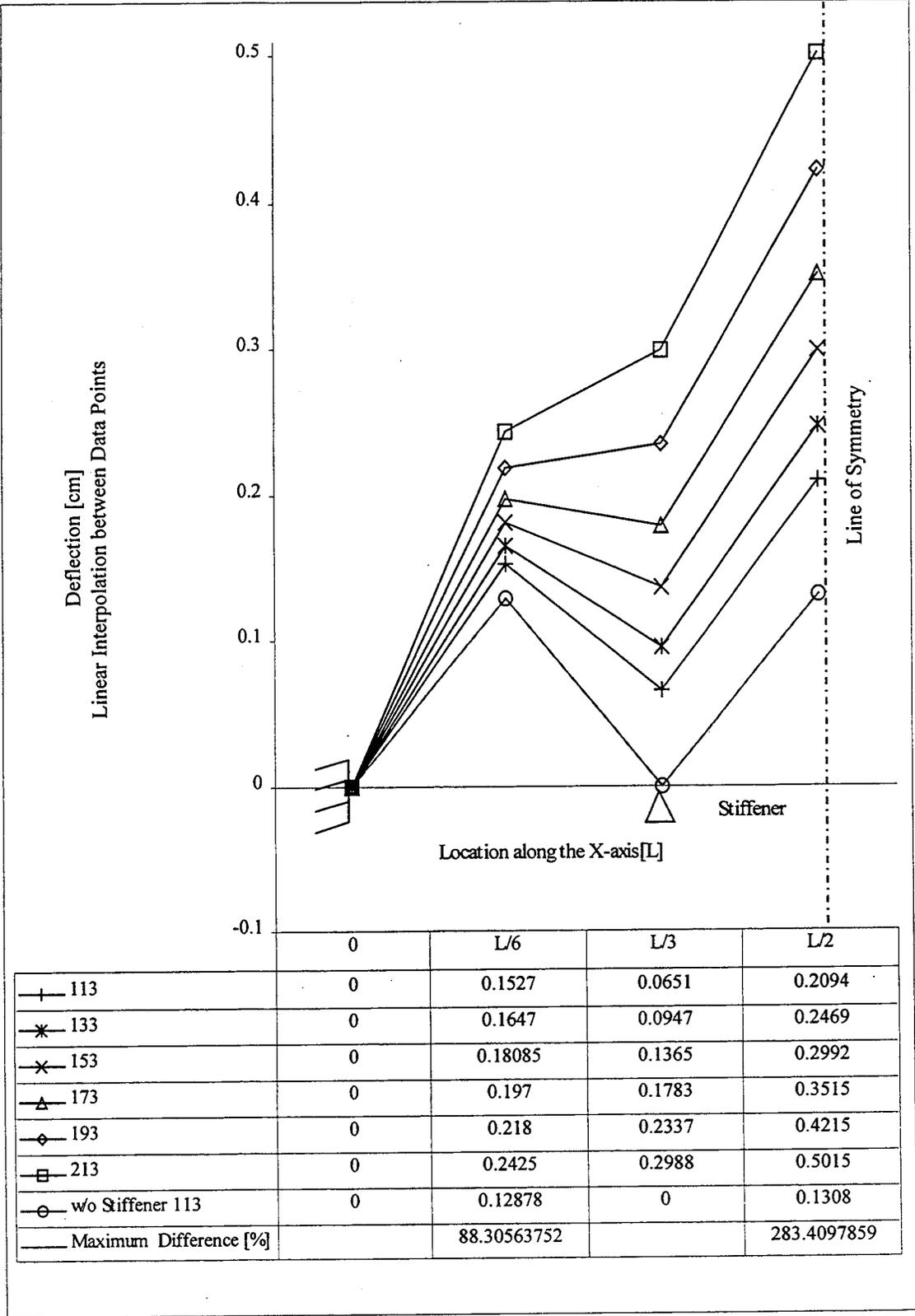


Figure A.6 Bottom Flange Deflections due to Distributed Load-Couple q

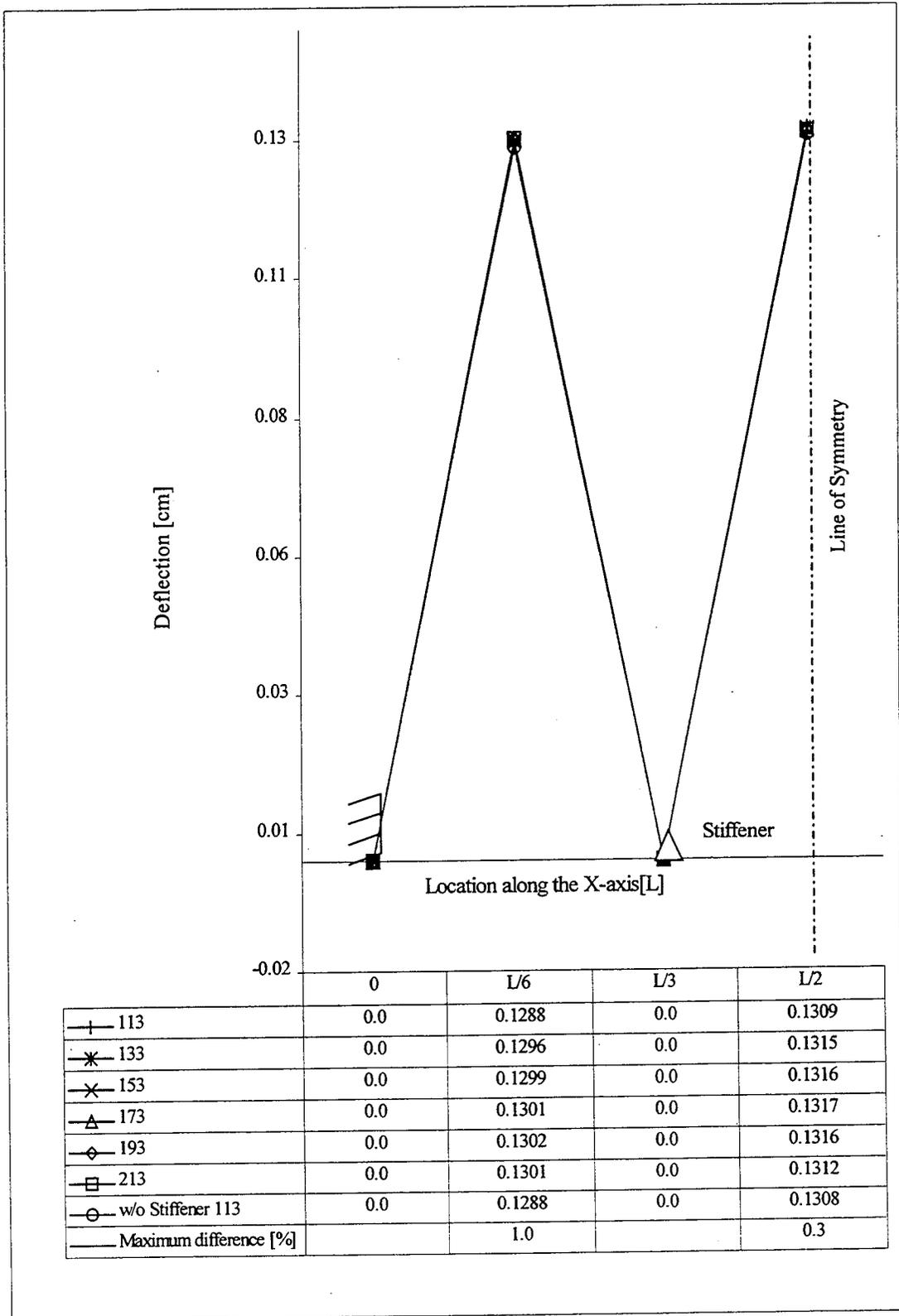


Figure A.7 Top Flange Deflections due to Distributed Load-Couple q

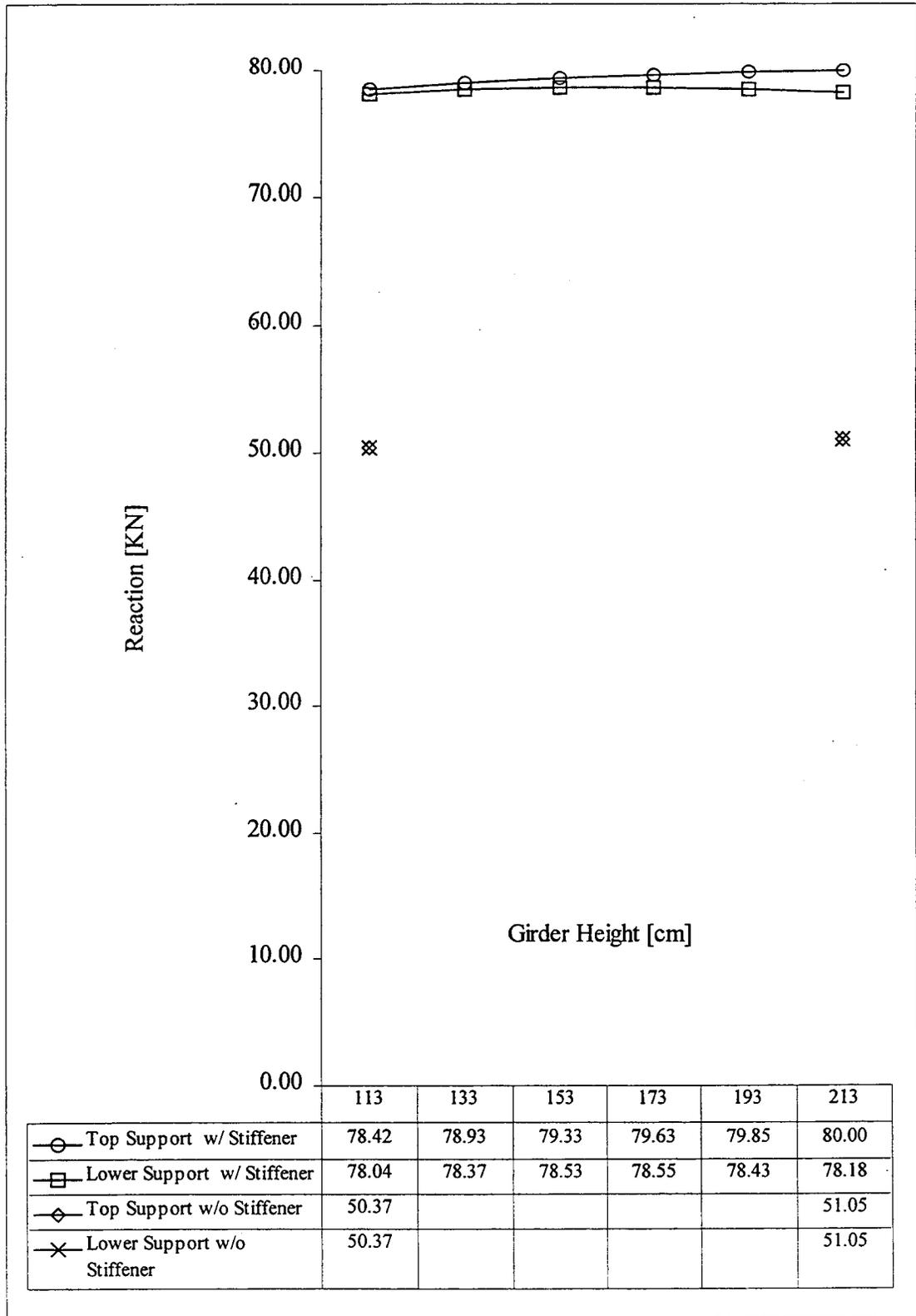


Figure A.8 Support Reactions at Middle Support due to Distributed Load-Couple q

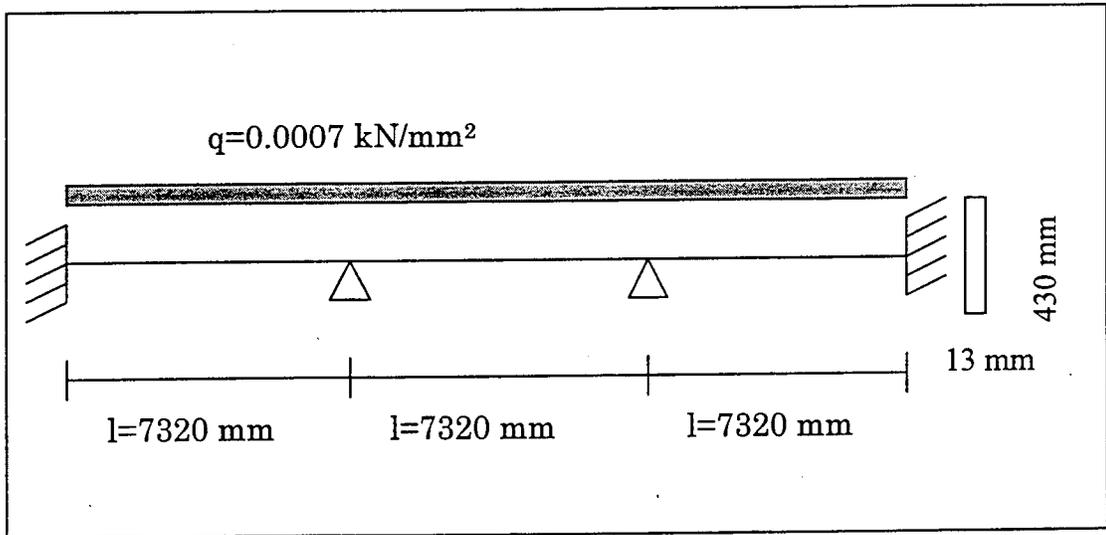


Figure A.9 System for RISA © 2-D Analysis

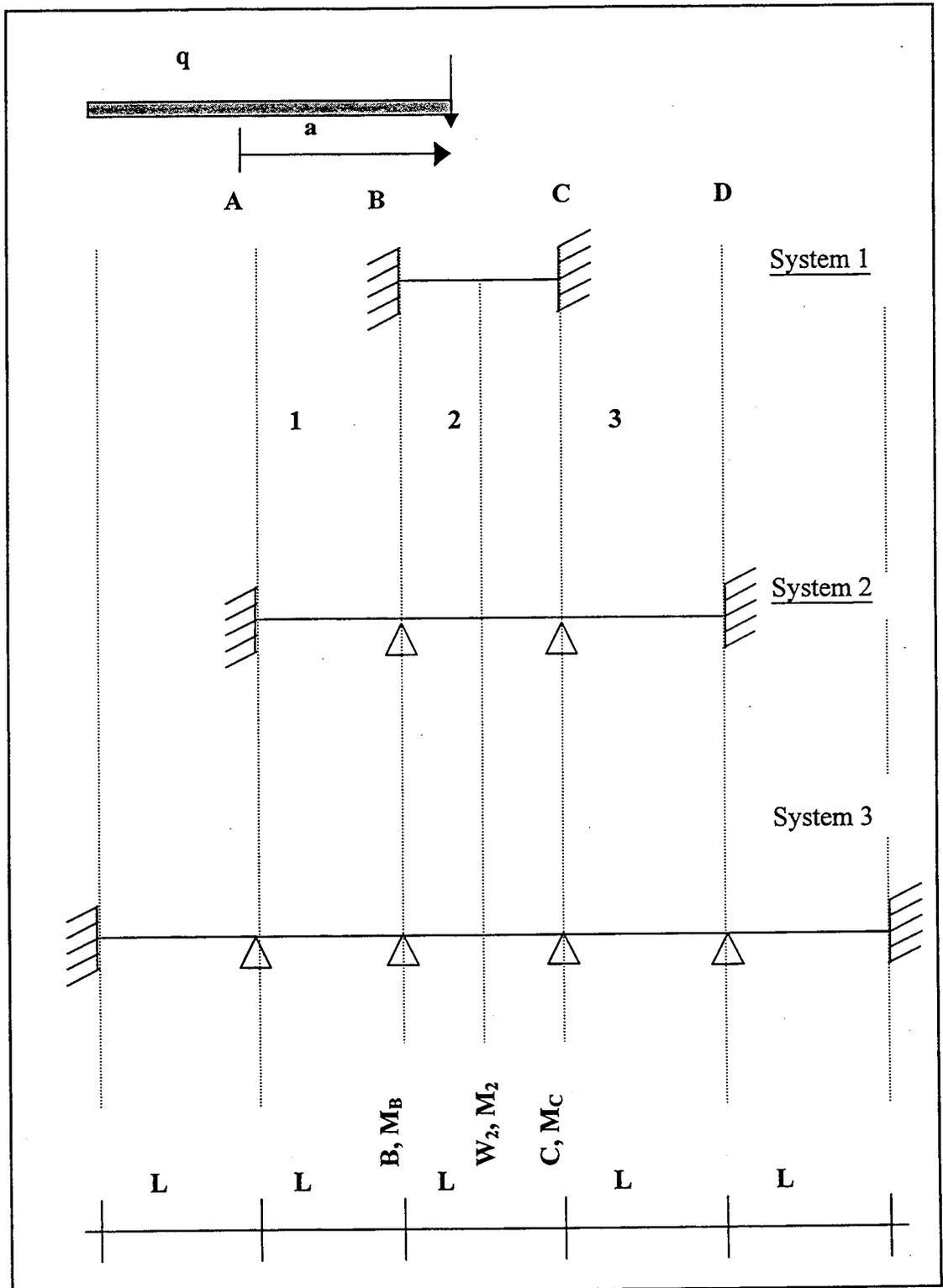


Figure B.1 System for 1, 3, and 5-span Models and Section Location

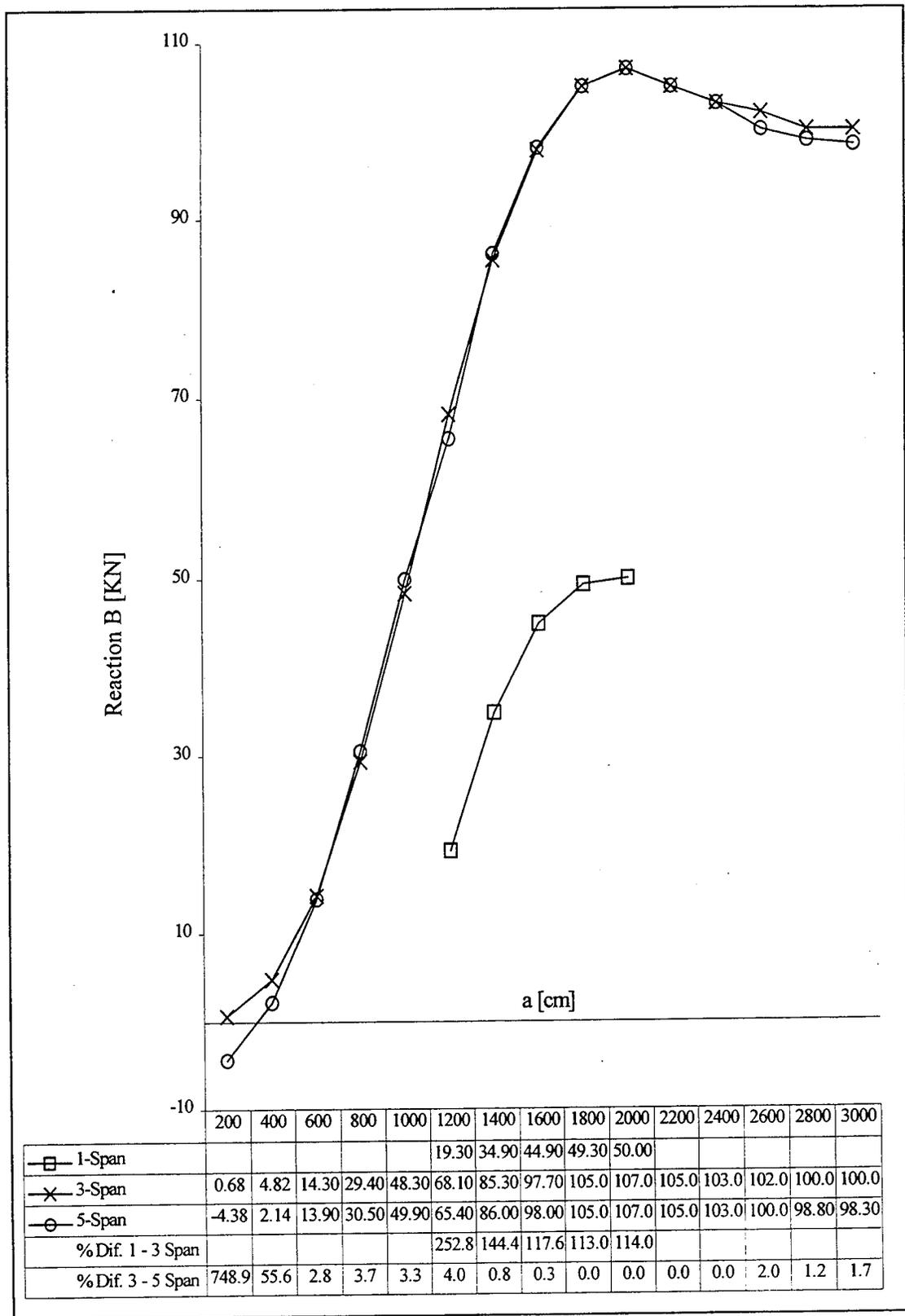


Figure B.2 Influence Line for Reaction B due to Distributed Couple q

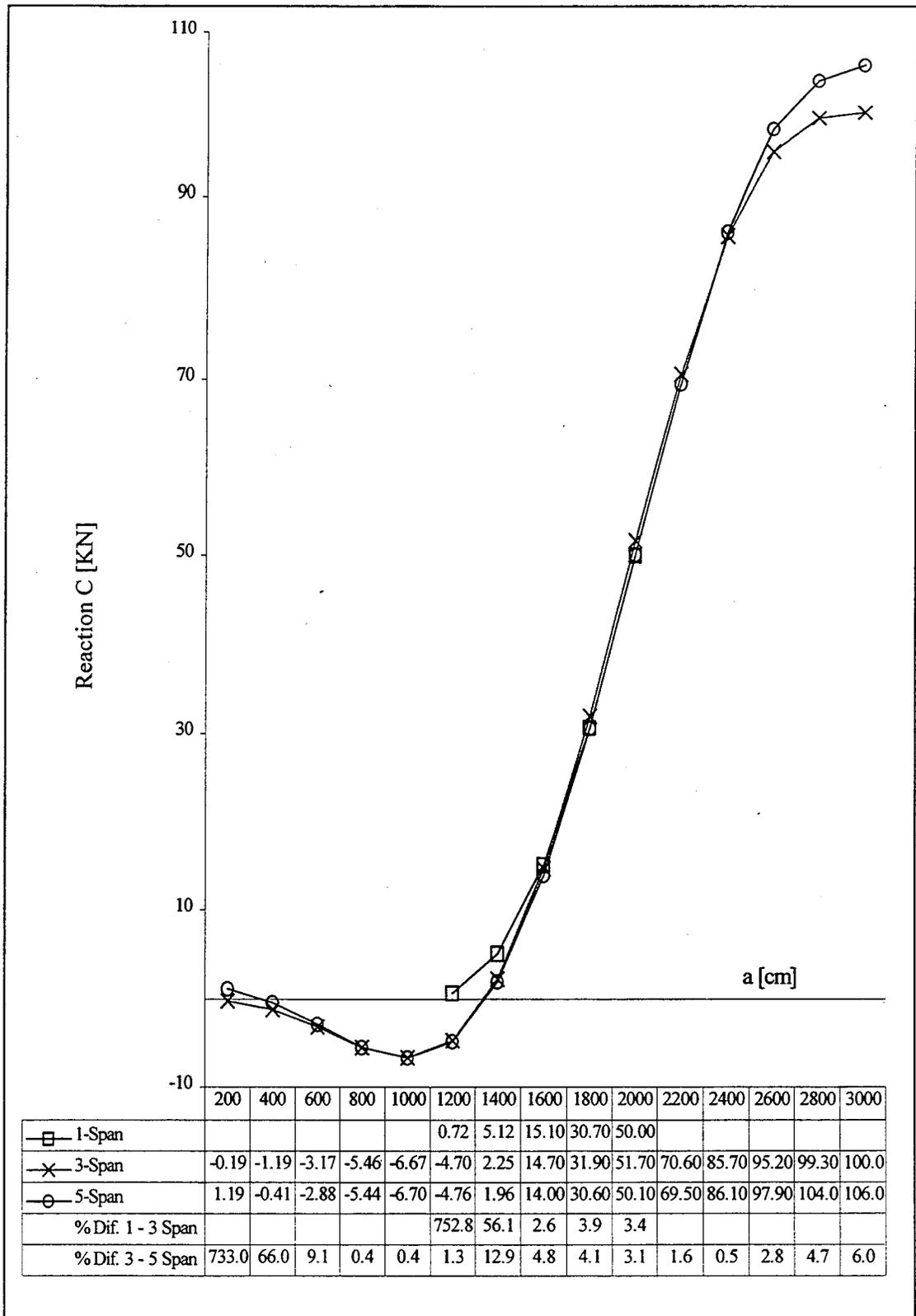


Figure B.3 Influence Line for Reaction C due to Distributed Couple q

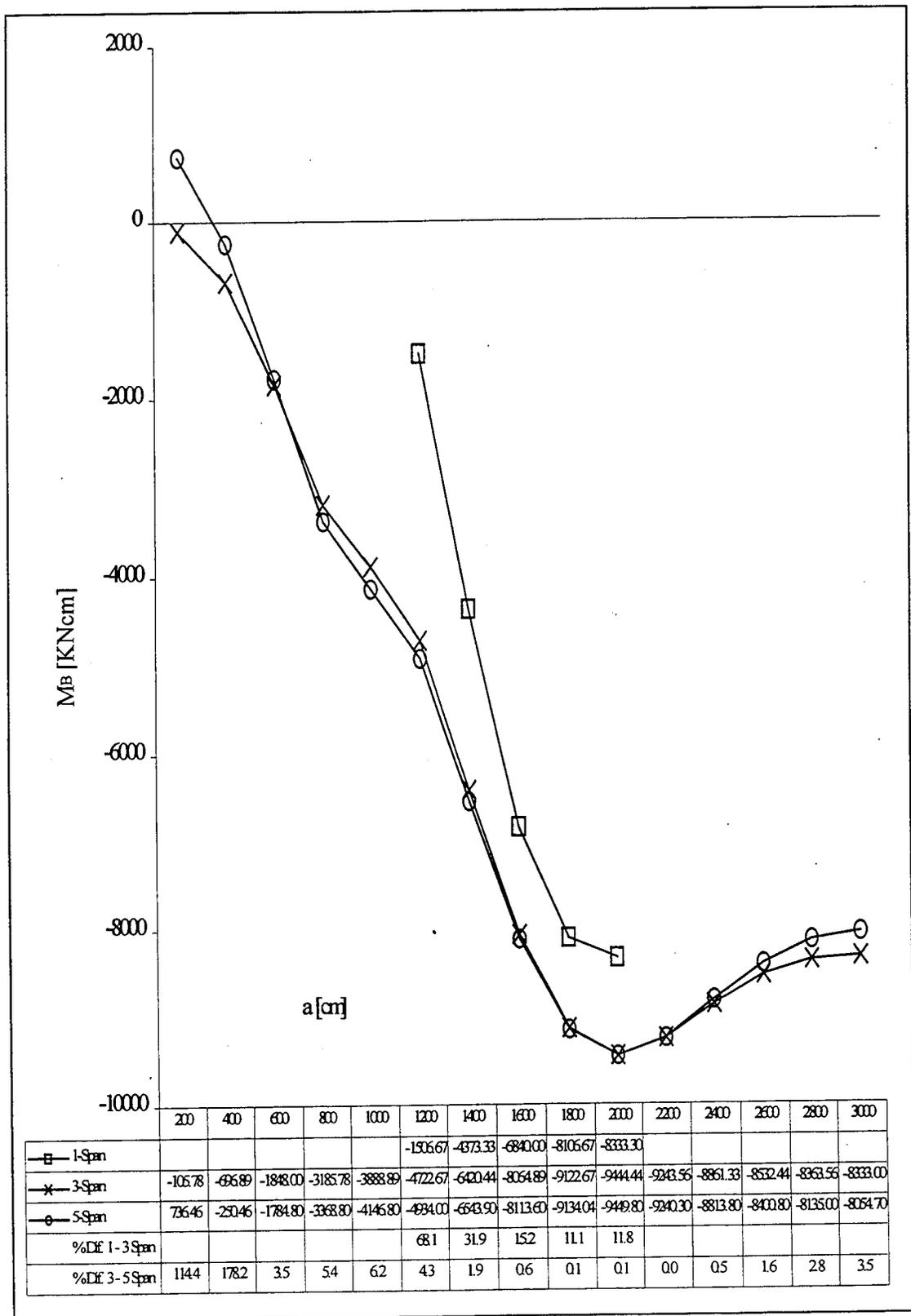


Figure B.4 Influence Line for Moment Mb due to Distributed Couple q

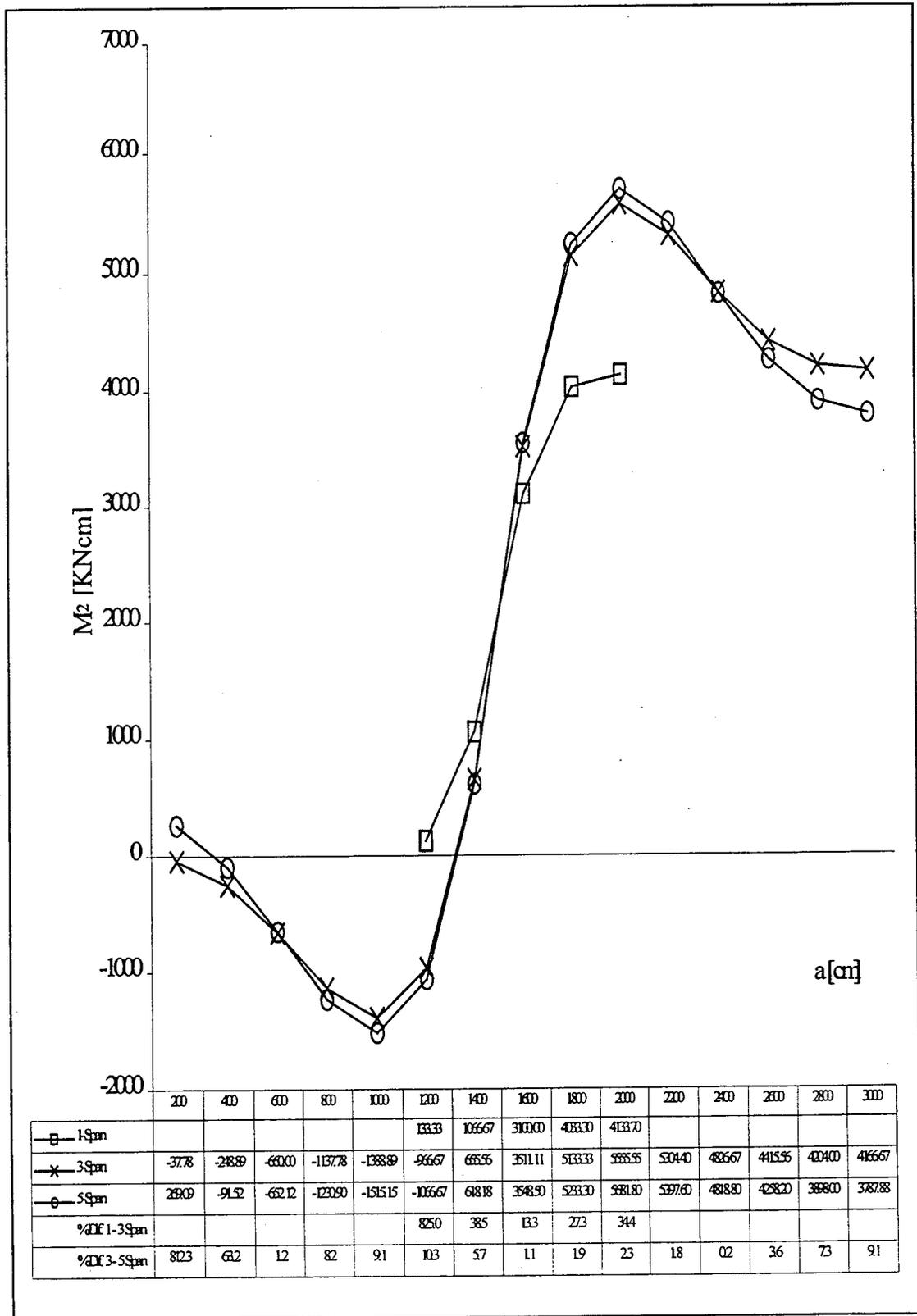


Figure B.5 Influence Line for Moment M_2 due to Distributed Couple q

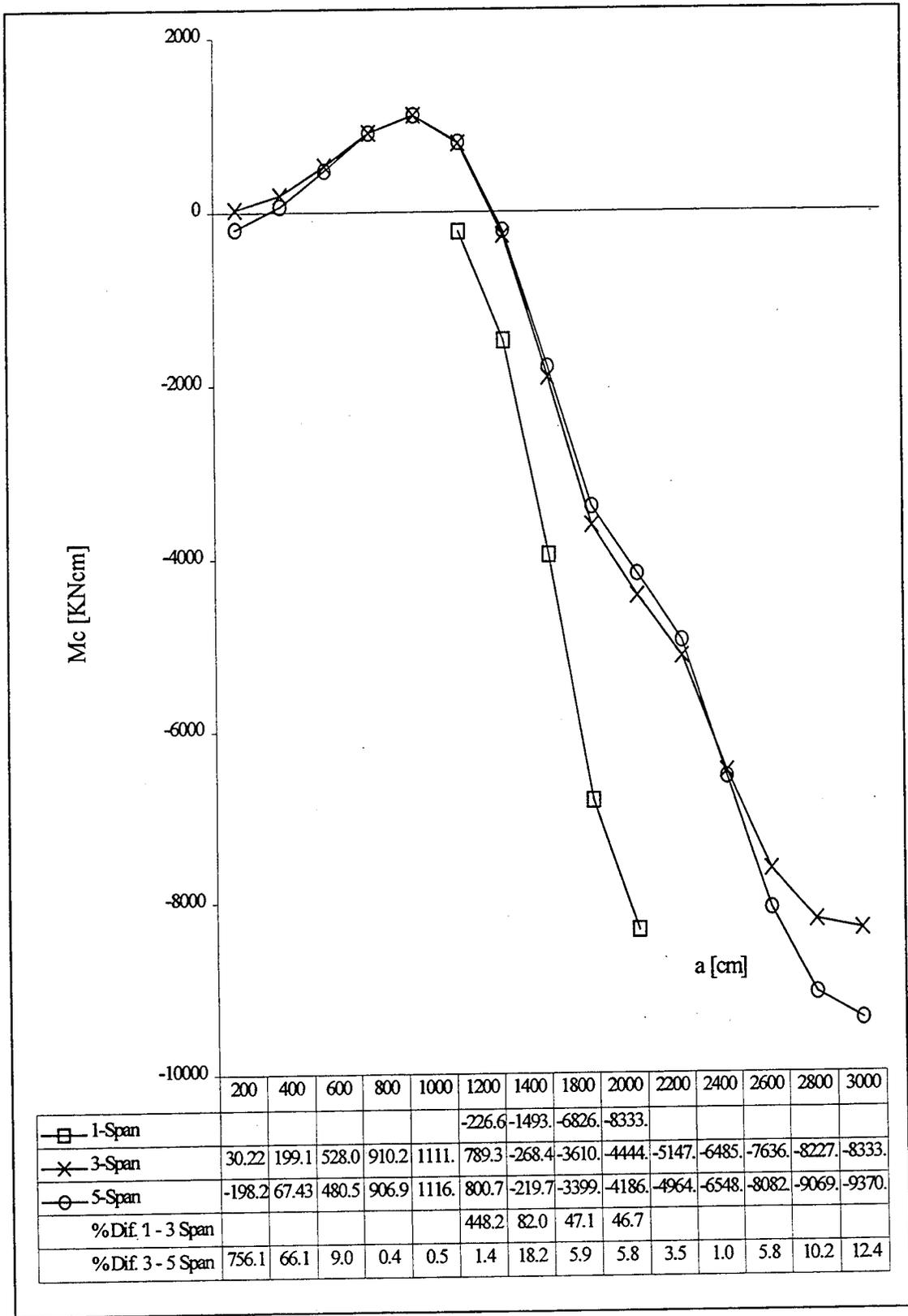


Figure B.6 Influence Line for Moment M_c due to Distributed Couple q

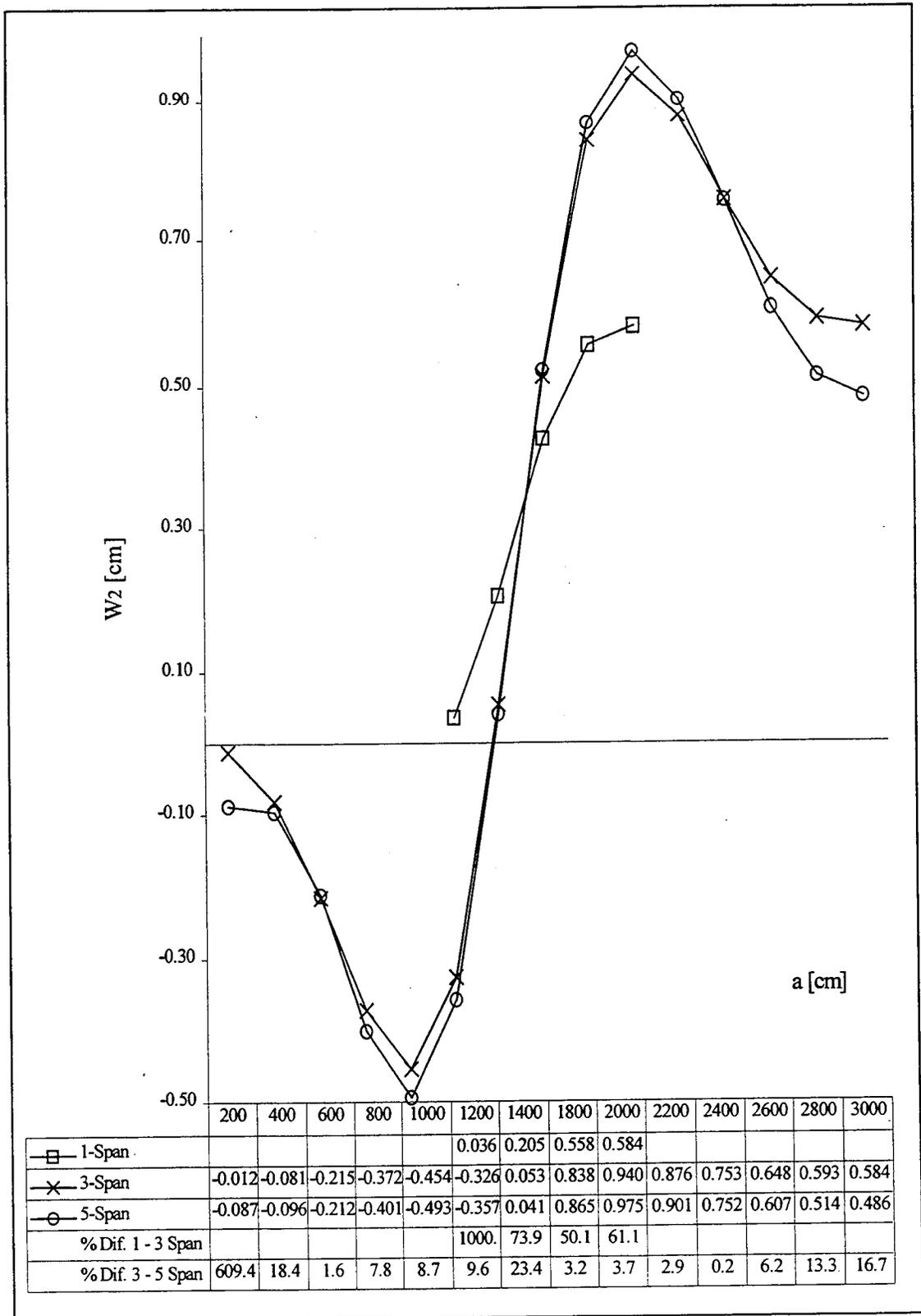


Figure B.7 Influence Line for Deflection W_2 due to Distributed Couple q

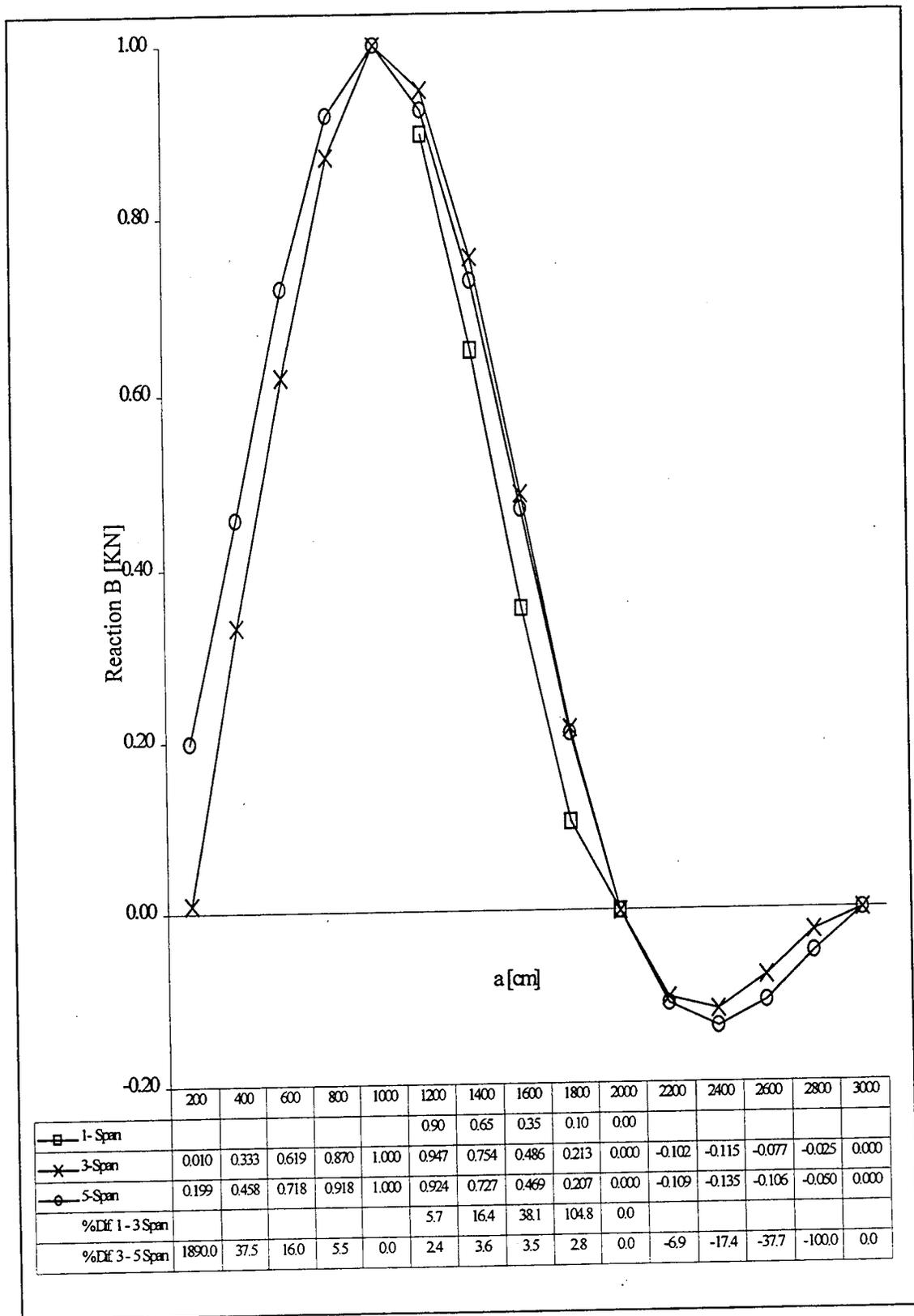


Figure B.8 Influence Line for Reaction B due to Concentrated Couple P

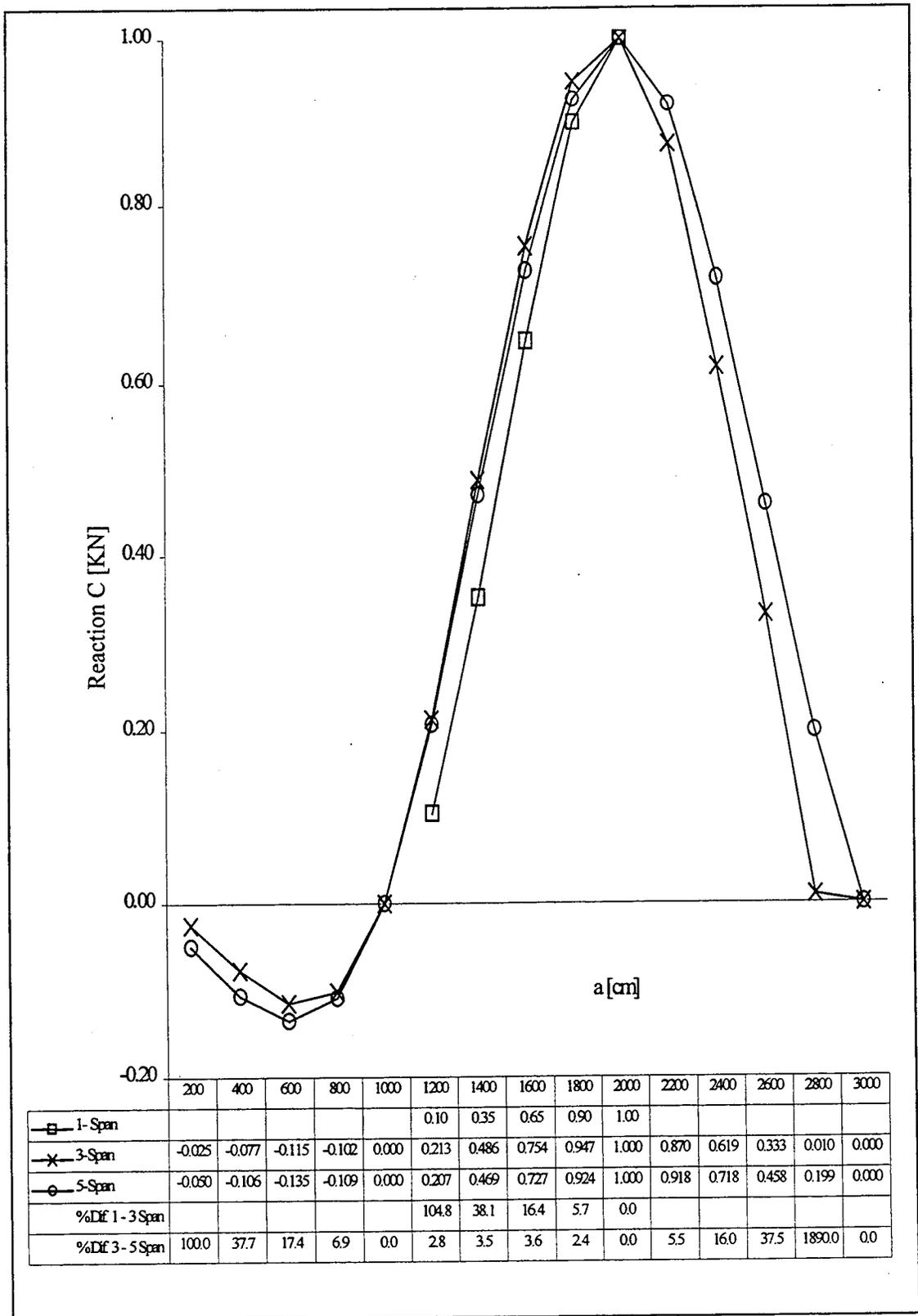


Figure B.9 Influence Line for Reaction C due to Concentrated Couple P

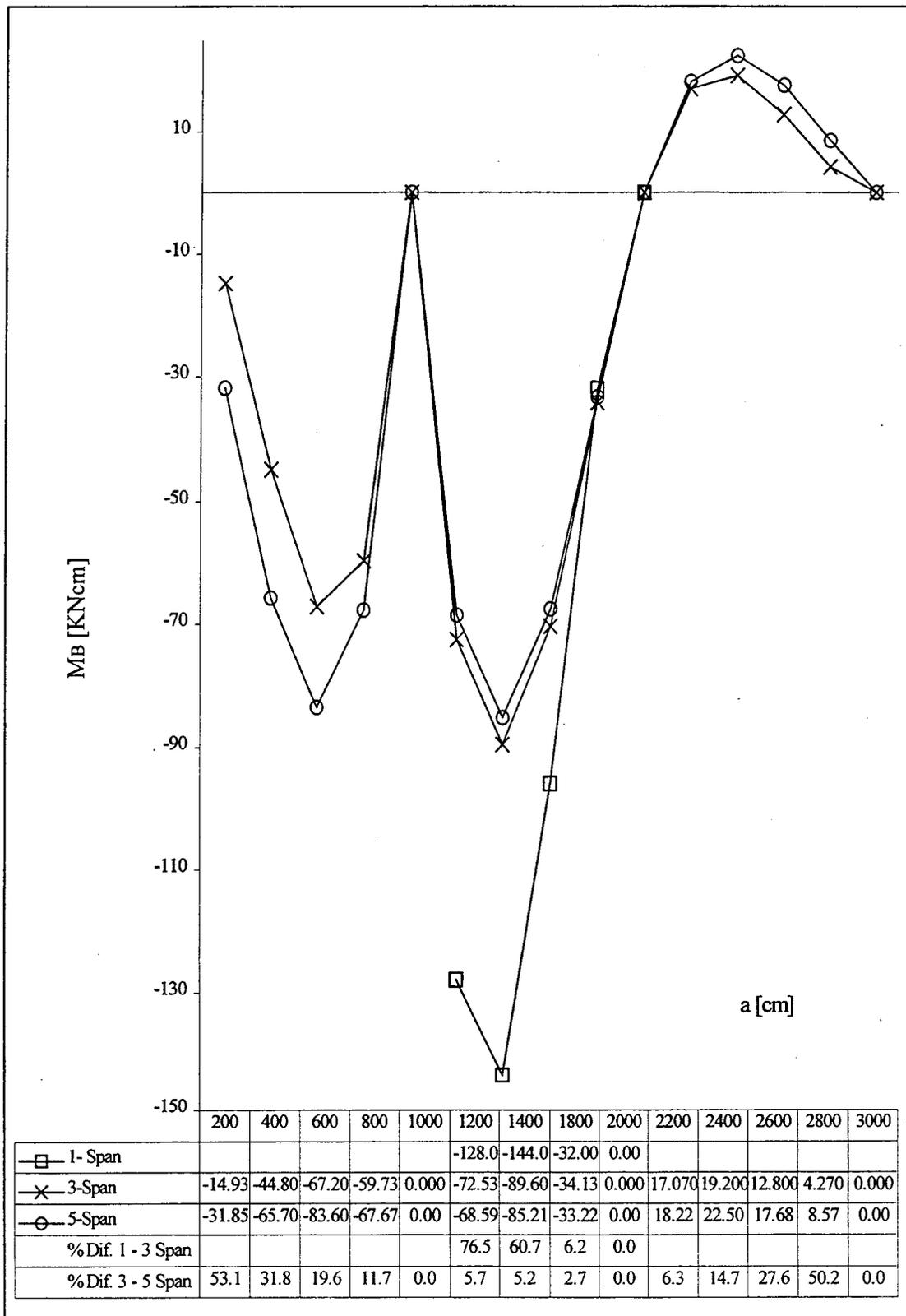


Figure B.10 Influence Line for Moment Mb due to Concentrated Couple P

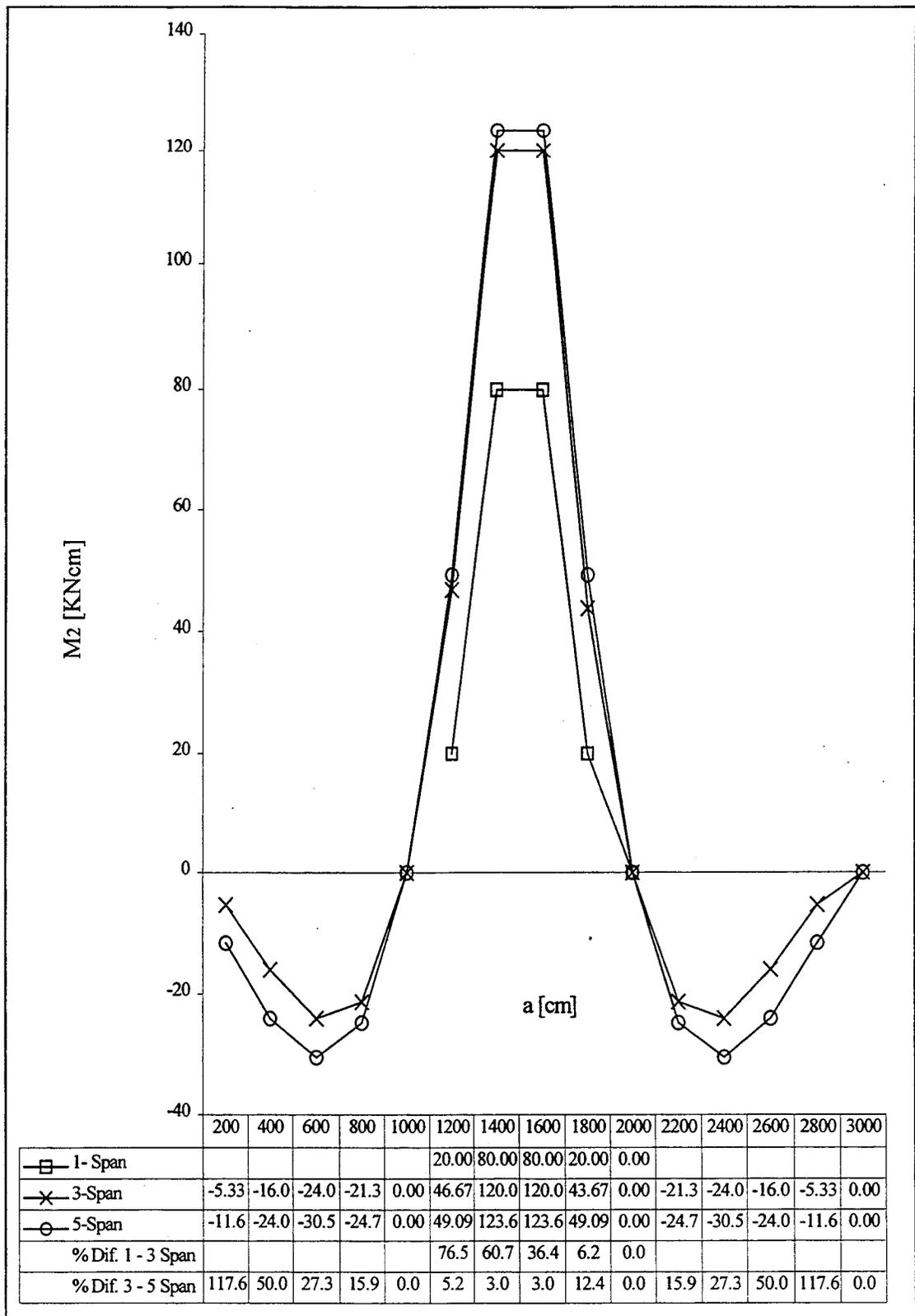


Figure B.11 Influence Line for Moment M_2 due to Concentrated Couple P

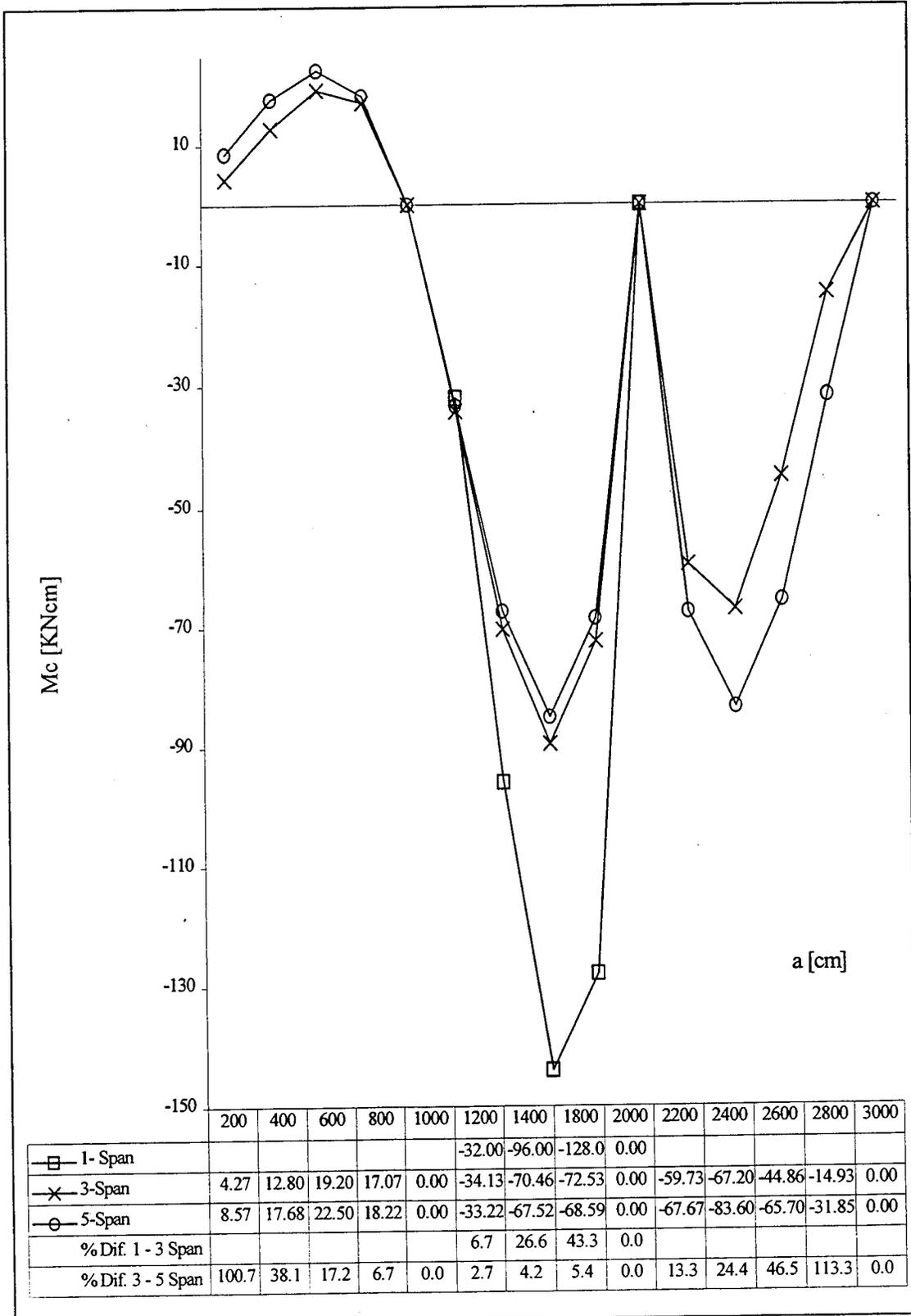


Figure B.12 Influence Line for Moment M_c due to Concentrated Couple P

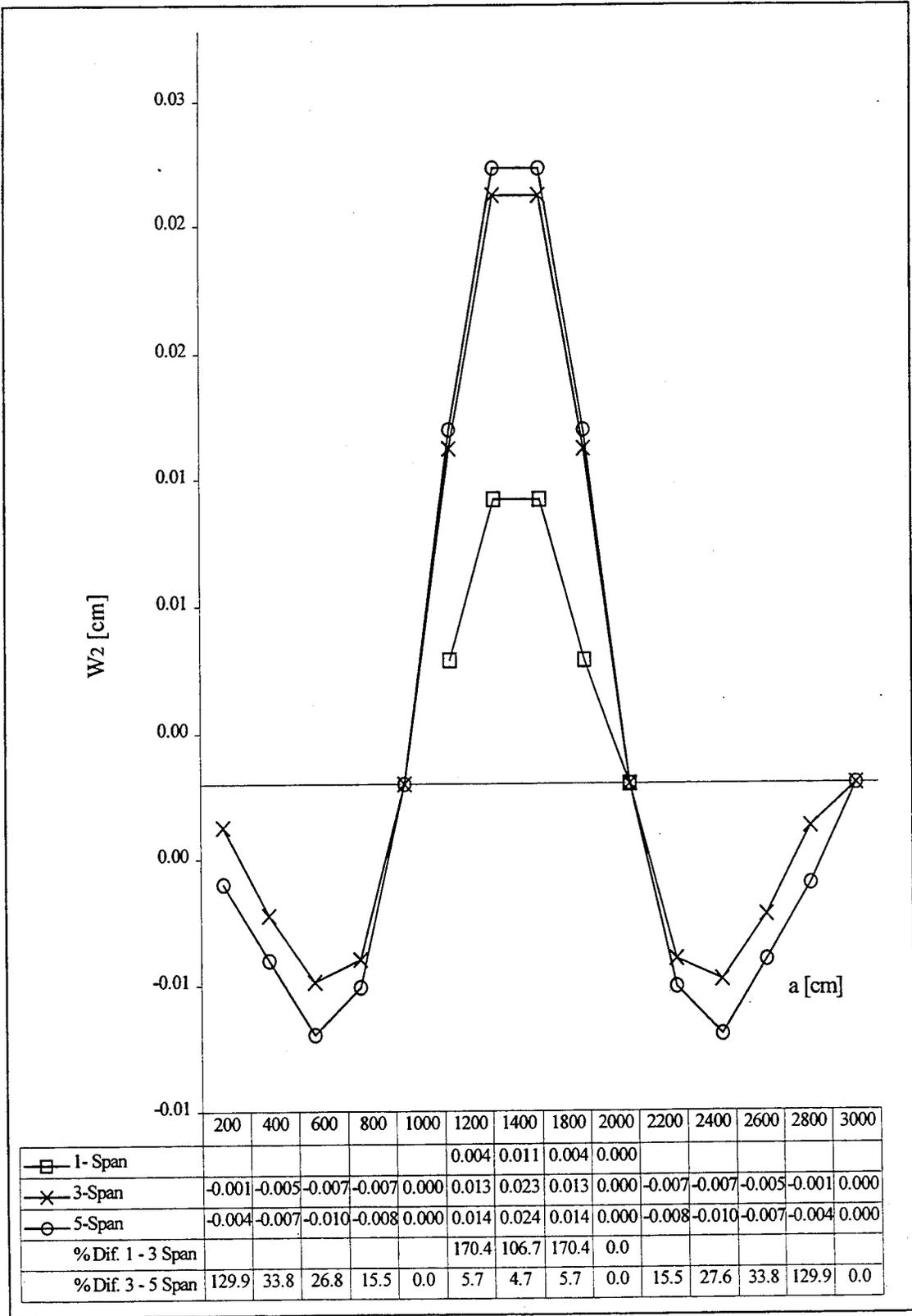


Figure B.13 Influence Line for Deflection W2 due to Concentrated Couple P

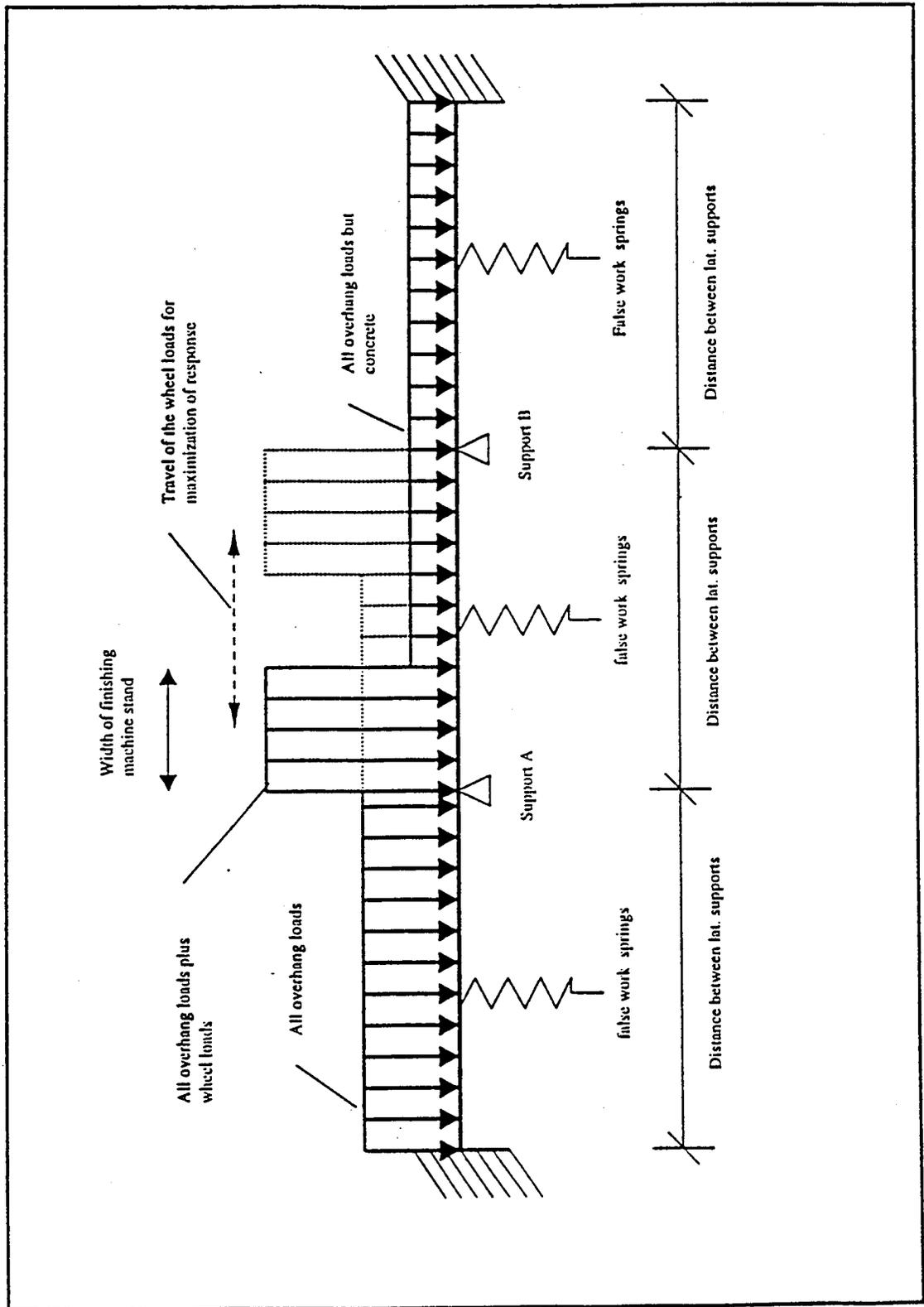


Figure D.1 Static System used by TAEG

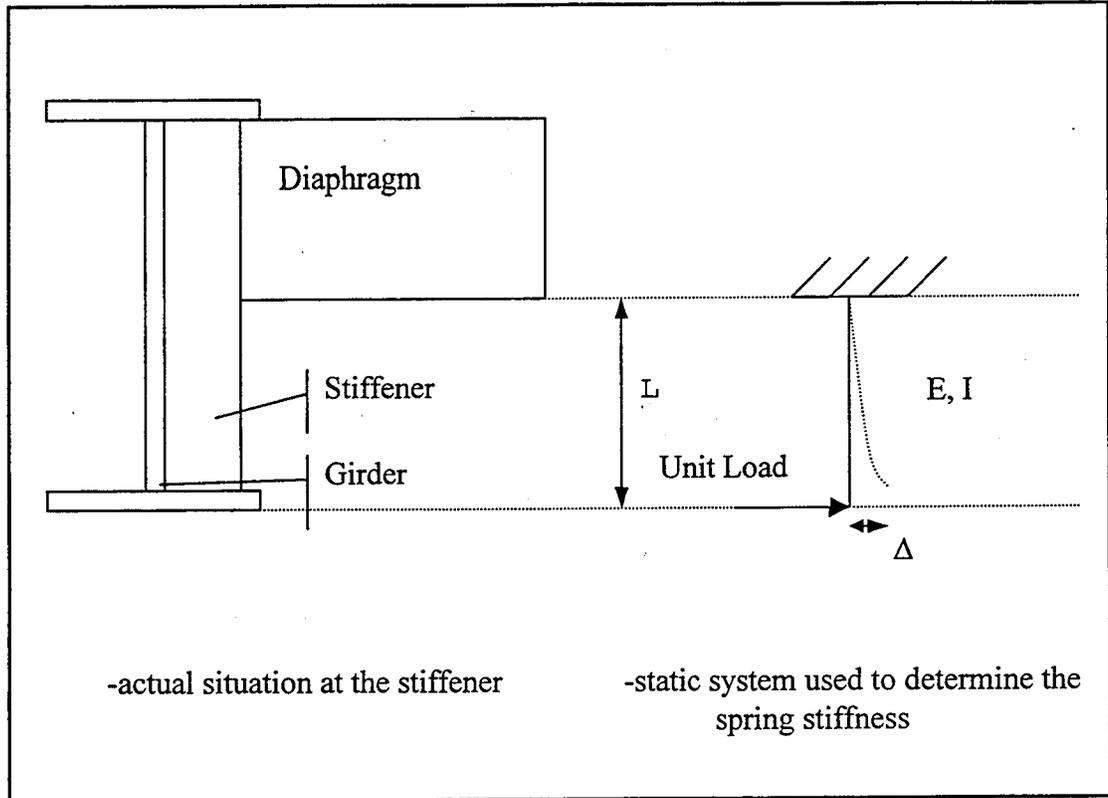


Figure D.2 Stiffener Spring Model

Tables

	KDOT Design Manual (divided by 1.3)	TAEG	
Stresses In The Positive Moment Region			
Top Flange	-46.42 (-35.71)	-48.07	[MPa]
Bottom Flange	16.32 (12.55)	16.90	[MPa]
Stresses In The Negative Moment Region			
Top Flange	87.10 (67.0)	64.91	[MPa]
Bottom Flange	-30.10 (-23.15)	-22.82	[MPa]
Girder Deflection and Rotation			
Lateral Top Flange Deflection	9.00	4.533	[mm]
Rotation of the Girder	0.17 ~ 0.46	0.25	degrees

Table 1. Results Comparison for Example 1

		Measured [psi]	TAEG w/o timber [psi]	TAEG with timber [psi]
Negative Moment	top	640.00	6233.12	6233.12
	bottom	563.38	5899.74	3015.43
Positive Moment	top	642.25	5816.83	5816.83
	bottom	72.112	5989.95	893.11

Table 2. Results Comparison for Example 2

		KDOTmanul W/o timber (divided by 1.3) [psi]	TAEG w/o timber [psi]	TAEG with timber [psi]
Negative Moment	top		13266.29	4883.08
	bottom		5427.12	1850.70
Positive Moment	top	19400 (14923)	15786.95	2173.75
	bottom		6458.30	715.85

Table 3. Results Comparison for Example 3