Project Title

Behavior and Design of Buried Concrete Pipes

Submitting Principal Investigators

Principal Investigator: Ece Erdogmus, PhD

Co-Principal Investigator: Maher Tadros, PhD, PE

June 2006

Table of Contents

Acknowledgements

The authors would like to thank the following institutions and individuals for their contributions to this study:

- The Nebraska Department of Roads
- Mr. Josh Beakley and the American Concrete Pipe Association
- Mr. Doug Mohrman and the Concrete Industries staff
- Mr. Rick Phillips and the Rinker Materials- Hydroconduit staff
- Mr. Mike Beecham
- Mr. Rich Decca and the Ivy Steel staff

List of Figures

List of Tables

1. INTRODUCTION

This report presents the findings of a year-long research project, where an in-depth review of the available concrete pipe design methods and the Nebraska Department of Roads (NDOR) pipe design policy is conducted. In this chapter, the project's significance, objectives, and tasks are presented.

1.1. Problem Statement & Research significance

Figure 1 summarizes the history of reinforced concrete pipe design methods and development. Currently, two methods are available for the design of reinforced concrete pipes: ***the indirect design method*** and ***the direct design method***. Both of the available design methods are proven to be reliable, yet as a result of recent advancements in manufacturing and construction, practical questions about the economy and state-of-the-art of the existing methods have developed.

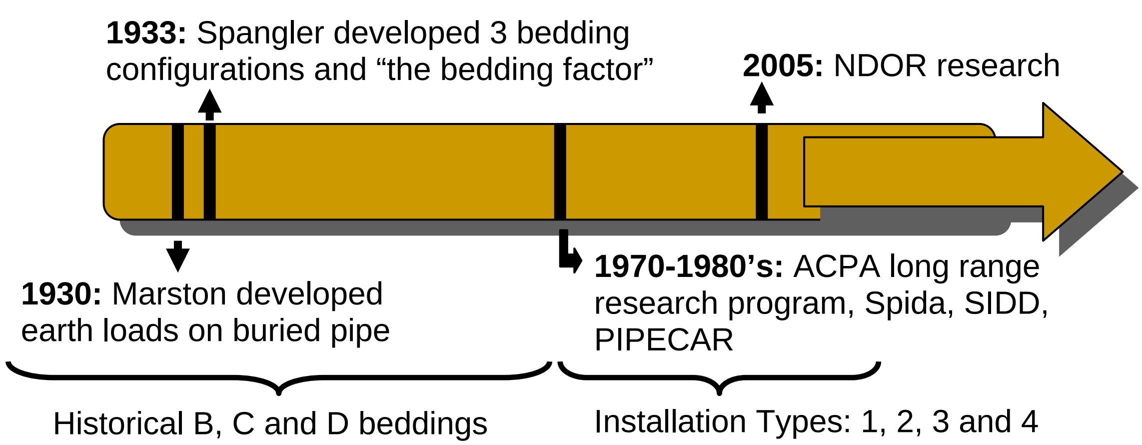


Figure 1 *History of Pipe Research*

Until the 1970's, the *indirect design method,* an empirical method developed in the early 1900's, was the only choice for the design of concrete pipes. According to this method, for circular pipes, ASTM C 655 defines the three-edge bearing ultimate load and an observed 0.01-inch crack width in terms of D-Loads multiplied by a strength factor. ASTM C 76 specification contains tables for steel reinforcement requirement, maximum spacing, and minimum wall thickness. These tables present design requirements for classes of reinforced concrete pipes based on test data. All pipe in a given strength class have the same ultimate D-Load requirement and the same 0.01-inch-crack-width requirement regardless of the pipe diameter.

In the 1970’s, a new procedure for the design of precast concrete pipes was developed. In this procedure, referred to as the *direct design method,* strength and serviceability limits are considered. Data from previous test programs and routine testing for quality control including the three-edge-bearing test results were used to develop the new design procedure. All tests were performed using a design concrete strength of less than 7,000 psi, and the measured compressive strengths of concrete cylinders were between 2,500-7,500 psi. The yield strength of the steel reinforcement used was less than 60,000 psi.

Both methods of design for reinforced concrete pipes have proven to be conservative and reliable through the years, however, questions regarding the correlation between three-edge-bearing test results and the installed condition remain unanswered. Furthermore, due to the increasing use of high performance concrete, high strength reinforcing steel, larger diameter pipes and the advancements in analysis methods, previous empirical evaluations of the structural behavior of reinforced concrete pipe must be revisited. Both the indirect and direct design methods need to be verified for their adaptability to these advancements in construction technology and structural analysis. A detailed study where the available design methods are critically reviewed and possibilities of incorporating these advancements are investigated, will lead to a better understanding of the pipe behavior and the refinement of existing design methods.

1. 2. Objectives

The objectives of this research are:

- To evaluate the methods used by NDOR for design of concrete pipe culverts by comparing these methods with methods used in other states, recent research results and national recommendations.
- To evaluate the design criteria and the design practice for reinforced concrete pipes, and suggest changes for consideration.
- To evaluate current NDOR concrete pipe construction specifications and suggest changes for consideration.

The long term objective of the study is:

- To improve the design criteria, construction specifications, and the theoretical understanding of the structural behavior of concrete pipes to achieve more rational, economical, and safer design methods.

1.3. Research Plan & Tasks

To accomplish the research objectives, the following tasks are carried out:

1. Review of the traditional pipe design practice (both indirect and direct design methods): Performance data, recent research findings, national recommendations, and other information relating to the construction and design practice of buried concrete pipes are reviewed and evaluated on the basis of applicability and usefulness for the improvement and development of NDOR specifications.

2. Preparation of recommendations for the improvement of design criteria and current design specifications: Possible revisions to NDOR pipe specifications are identified based on the available information, and preliminary additions and changes for the design tables are suggested.

3. Preparation of a report including a detailed plan for the tasks of Phase 2: The tasks to be completed for the implementation of the necessary revisions to the NDOR specifications regarding concrete pipe design are listed, i.e. the necessary work to be done in Phase 2 of the project are determined. (See Phase 2 proposal)

2. REVIEW OF CONCRETE PIPE DESIGN PROCEDURES

The main objective of this study is to review the available reinforced concrete pipe design methods and examine the NDOR pipe policy to determine whether or not updates, refinements, or improvements are needed to match advancing technology. This section discusses the findings of the study.

2.1. Review of Available Pipe Design Methods

In the late 1920's a research project at Iowa Experiment Station was conducted with the objective of determining the supporting strength of buried rigid pipes in embankment installation when subjected to earth pressures, using Marston's theories. The results of this research were given in a comprehensive paper by M.G. Spangler (1933), where, a general equation for the bedding factor is presented. His work included the definition of four standard bedding types that are similar to those defined earlier by Marston. Marston and Spangler's research is the basis of the currently used *indirect design method*. In 1983, the indirect design method developed by Marston-Spangler was included in a new section of the American Association of State Highway and Transportation Officials Bridge Design Specifications (AASHTO).

According to the indirect design method, the required supporting strength of the pipe is a function of the magnitude of the earth pressure and its distribution around the pipe and it is obtained either from empirical evaluation of former tests or from actual results of three-edge-bearing tests (TEB). The required strength (design TEB) is then defined in terms of the ratio of the total load to the bedding factors that were calculated based on the Marston-Spangler soil-structure interaction analyses.

Using this D-Load, wall thickness, concrete strength and reinforcement requirements are determined using the previously established standard values (ASTM C 76, ASTM C 655, AASHTO M 170, and M 242).

In summary, the indirect design method is an empirical method developed in the early part of the last century. The method is empirical in nature because it uses the 0.01-inch crack criterion developed in a three-edge-bearing test to evaluate the supporting strength of reinforced concrete pipe. The indirect design method is still widely used today and documents such as ASTM C76 and the ACPA fill height tables are published as specifications and design aids. The empirical nature of the indirect design method does not provide flexibility in design and specification of reinforced concrete pipe.

Although the indirect design method has been a generally accepted and satisfactory procedure in the recent past, the developments on the knowledge of soil properties, as well as the advancements in the structural analysis techniques have led to significant improvements in the design of concrete pipes. In the 1970's, American Concrete Pipe Association (ACPA) instituted a long-range research program with the objective of evaluating the performance of concrete pipe-soil installations and improving the design practice. In this research, the structural behavior of concrete pipes and the structure-soil interactions were examined. As a result of this research program, new standard installation types and the Heger earth pressure distribution (Figures A-1 through A-3 and Table A-1) were recommended, which differ considerably from those originally developed by Marston-Spangler. Consecutively, four new standard installations, Heger earth pressure distribution and the *direct design procedure* were incorporated in a 1993 American Society of Civil Engineers Standard entitled "ASCE Standard Practice for Direct Design of Buried Precast Concrete Pipe in Standard Installation (SIDD)".

The direct design method is a more rational semi-empirical approach to reinforced concrete pipe design. Direct design is a limit states design procedure that allows for the design of reinforcing for concrete pipe based on five limit states: 1) reinforcement tension, 2) concrete compression, 3) radial tension, 4) diagonal tension, and 5) crack control. Thus, direct design is much more flexible than indirect design provided that it is used efficiently.

According to the direct design method, the required strength of the concrete pipe is determined from the effects of the bending moment, thrust and shear. Wall thickness, concrete strength and reinforcement design are evaluated using rational procedures based on strength and crack width limits that were developed in the long-range research program of the ACPA.

In the next section, a general comparison of the two methods are presented, while further details of the indirect and direct design method procedures are presented in sections 2.1.2. and 2.1.3., respectively.

2.1.1. Indirect and Direct Design Methods: A General Comparison

Currently, depending on the designer's preference, either the indirect or the direct design method is used for the design of RCP, and both methods have some common elements. The modern standard installations which eliminate the limitations of the historic installations were developed mainly under the scope of the direct design method. However, today they are also used in the indirect design method with acceptable performance. Vertical arching factor (VAF), as shown in Table A-1, generated by Heger earth pressure distribution is also applied to the calculation of earth pressures in the indirect design method. On the other hand, the crack width limit that is used for predicting the strength of reinforced concrete pipe in the direct design method was developed based on the results of three-edge-bearing testing, which was originally developed within the scope of the indirect design method.

Figure 2 illustrates the difference between designs by the direct and indirect methods for a 48-inch-diameter pipe installed in Type III bedding. The indirect design method is characterized by discrete steps that represent changes in ASTM specified pipe classes. The data points for the direct design method are generated from the direct design software package PipeCar and are characterized by a linear curve. The comparison shows that direct design can be both more conservative and less conservative than indirect design depending on the required fill height and the class of pipe specified by the indirect design method.

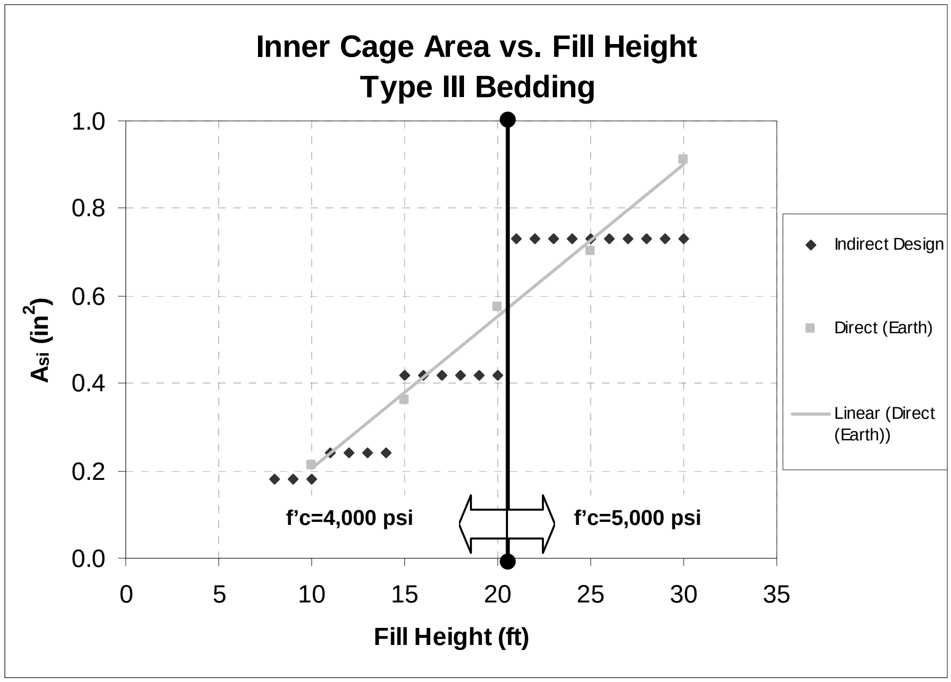


Figure 2 *Comparison of Indirect and Direct Design*

Table 1 provides a general outline of the design procedures for both methods. As can be seen, the first three items are common in both methods, while major differences exist for the determination of the pipe's supporting strength. These similarities are detailed further in this section, while the details of the supporting strength calculations for indirect design methods are given in sections 2.1.2. and 2.1.3., respectively.

Table 1 *Comparison of the indirect and direct design procedures*

Indirect Design	Direct Design	
1. Determine earth load (PL x VAF, Table 1)	1. Determine earth load (PL x VAF, Table 1)	Common Elements
2. Determine live load (AASHTO live load)	2. Determine live load (AASHTO live load)	
3. Select Standard Installation Type (Standard Installation Fig. 1, 2, 3)	3. Select Standard Installation Type (Standard Installation Fig. 1, 2, 3)	
4. Determine bedding factor	4. Determine moments, thrusts, and shear forces	Distinctive Elements
5. Determine required D-load (TEB-test)	5. Determine wall thickness, concrete strength, reinforcement based on an analysis of five limit states	
6. Determine wall thickness, concrete strength, reinforcement based on D-load (ASTM C76, C 655 or AASHTO M170, M242)		

Elements common to both design methods include: the earth and live load calculations, and the use of the standard installation types. These common elements are briefly discussed below:

1. Earth Load: In both the indirect and direct design methods, the earth load is determined by using the Marston-Spangler theory. The magnitude of earth load is the weight of the column of earth above the pipe defined in terms of the prism load (PL) multiplied by the arching factor (VAF).

$$W_e = PL \times VAF \qquad (1)$$

$$PL = \left[\frac{wD_o}{12}\right]\left[H + \left(\frac{0.107\,D_o}{12}\right)\right] \qquad (2)$$

Where:

W_e	=	*unfactored earth load, lbs/ft*
PL	=	*the prism load*
VAF	=	*vertical arching factor*
W	=	*unit weight of soil, lbs/ft^3*
D_o	=	*outside diameter of pipe, in.*
H	=	*design height of earth above top of pipe, ft*

The arching factors (VAF) given in Heger earth pressure distribution are adopted later for calculating the earth load in each standard installation type; further details of arching coefficients are shown in Table A-1. The simplified formula for calculating the earth load given in AASHTO-LRFD Specifications is given in Equation (3).

$$W_E = F_e \, w \, B_c \, H \qquad (3)$$

Where:

W_E	=	*unfactored earth load (KIP/FT)*
F_e	=	*soil-structure interaction factor for the specified installation*
B_c	=	*out-to-out horizontal dimension of pipe (FT)*
H	=	*height of fill over pipe (FT)*
W	=	*unit weight of soil (PCF)*
F_e	=	*VAF when standard installation and Heger earth pressure distribution are used. It is noted that the appropriate soil structure interaction analysis should be determined for calculating the earth load and the pressure distribution when nonstandard installations are used.*

2. Live load: As specified in Article 3.6 of the AASHTO-LRFD Specifications *the standard installation* and *Heger earth pressure distribution* are used for both the indirect and direct design methods.

3. Standard Installation Types: As mentioned in the previous section discussing the historical development of the two methods, as a result of ACPA's research program during 1970-80's, new standard installation types and the Heger earth pressure distribution (Figures A-1 through A-3 and Table A-1) were developed, which differ considerably from those originally developed by Marston-Spangler. Today, these installations are used, regardless of the chosen method of design (i.e. indirect or direct design procedure).

2.1.2. Indirect Design Method: Pipe's Supporting Strength

In the indirect design method, supporting strength is determined by using an equivalent three-edge bearing load (TEB), which is defined as the ratio of total field load to bedding factors (B_f). Bedding factors based on the Marston-Spangler design procedures are applied to obtain the required minimum TEB load.

$$Design\ TEB = \frac{W_e + W_L + W_f}{B_f} \qquad (4)$$

For convenience, three-edge bearing strength requirement is expressed in terms of D-Load. D-Load is defined as the ratio of the TEB load per foot to the inside diameter (D_i) of pipe.

$$D - Load = \frac{TEB}{Di} \qquad (5)$$

Based on the required D-load, concrete strength, reinforcement requirement and pipe wall thickness are given in ASTM C 76, ASTM C 655, AASHTO M 170, and M242.

2.1.3. Direct Design Method: Pipe's Supporting Strength

In the direct design method, the supporting strength is determined by the effect of pressure distribution around the pipe defined in terms of moment, thrust, and shear. The moment, thrust and shear can be computed by using either a computer program or hand calculations with the appropriate coefficients.

$$\begin{aligned} &\textit{Moment} \quad M_i = C_{mi}\, W_i\, D_m/2 \qquad (6)\\ &\textit{Thrust} \quad N_i = C_{ni}\, W_i\\ &\textit{Shear} \quad V_i = C_{vi}\, W_i \end{aligned}$$

The coefficients, C_{mi}, C_{ni}, C_{vi}, shown in Table A-5 are derived from the results obtained using computer analysis.

Concrete strength, reinforcement requirement and wall thickness are determined using rational procedures based on strength and crack width limit states. The design procedures are given in section 12.10.4.2 of the AASHTO- LRFD Bridge

Specification and in ASCE Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD).

2.2. In-Depth Evaluation of Direct Design Method: Parametric Study

A parametric study was performed to evaluate the influence of concrete strength, reinforcing steel strength, and crack control on the supporting strength of a pipe designed using the direct design method. Figure 3 illustrates how pipe strength is controlled by flexure, crack control, or shear depending on the depth of fill height above the pipe as presented in ASCE15(1998). For low fill heights, flexure controls, for medium depths of fill there is a small region where crack control governs, and at deeper fill heights, shear strength controls the pipe design. This plot is non-dimensional and therefore does not reveal the boundaries of these controlling criterion changes. The parametric study conducted in this study aims at generating similar plots for a 48-inch pipe diameter and fixed ranges of fill height and reinforcing steel area.

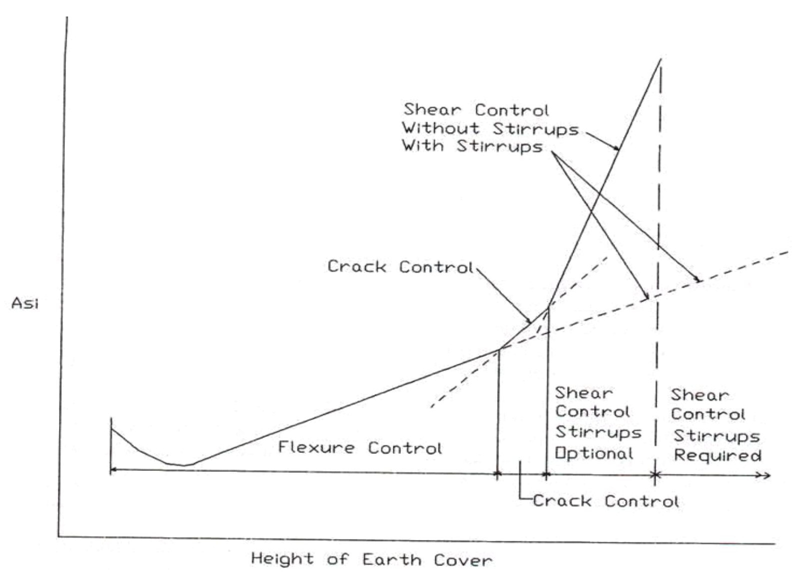


Figure 3 *Controlling Criteria* (ASCE 15-98)

Since the direct design method is flexible in the selection of input parameters, a search for efficiency in design necessitates a parametric study. The effects of varying several parameters on governing design criteria were studied. The parameters studied are:

- concrete strength (4,000-8,000 psi)
- steel reinforcement strength (65,000-80,000 psi)
- crack control factor (0.7, 0.9, 1.3)

The results of this parametric study are presented in this section.

Parametric Study for Concrete Strength

Figure 4 summarizes the results of varying concrete strength from 4,000 psi to 8,000 psi in 1,000-psi increments. As concrete strength increases, flexural capacity increases. At deeper fill heights, where crack control and shear govern pipe design, increasing concrete strength allows a reduction in the required steel area.

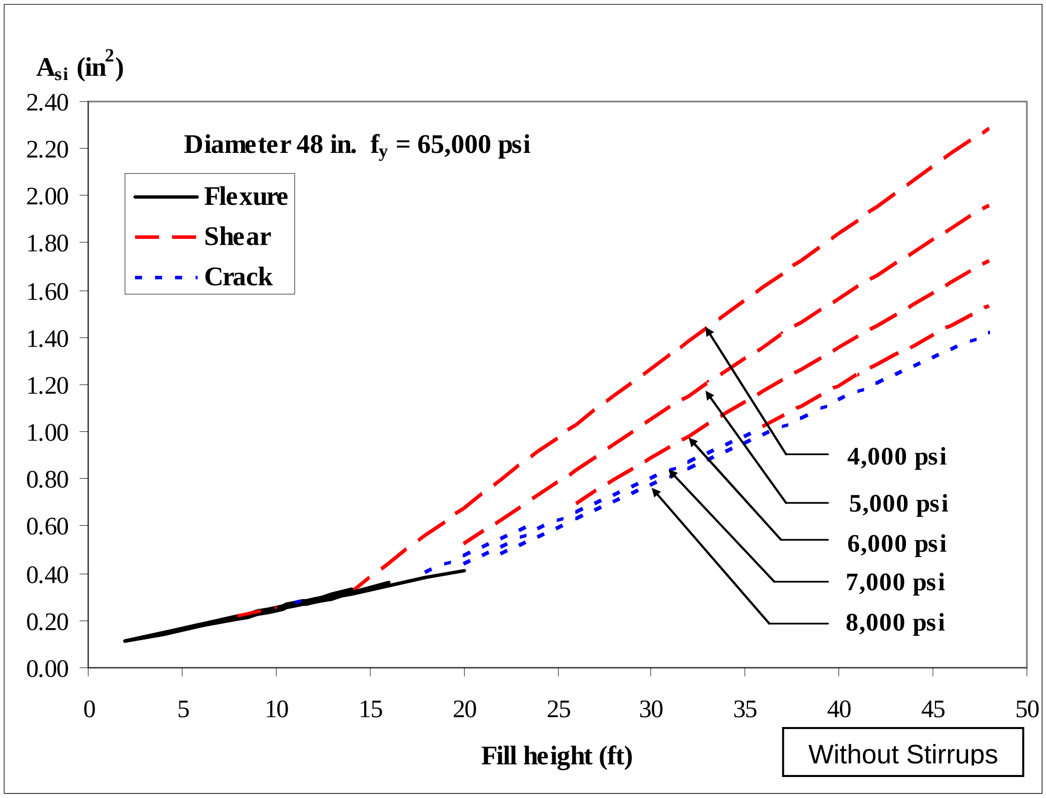


Figure 4 *Parametric Study - Concrete Strength*

Parametric Study for Reinforcing Steel Strength

Figure 5 summarizes the results of varying reinforcing steel strength from 65,000 psi to 80,000 psi in 15,000-psi increments. Increasing the steel strength has a very small effect on the capacity of the pipe. This is because with the current NDOR practice where stirrups are considered special design, there is no effect for deeper fill heights where shear controls. Since there are no stirrups, the steel strength is not a variable in the design equations.

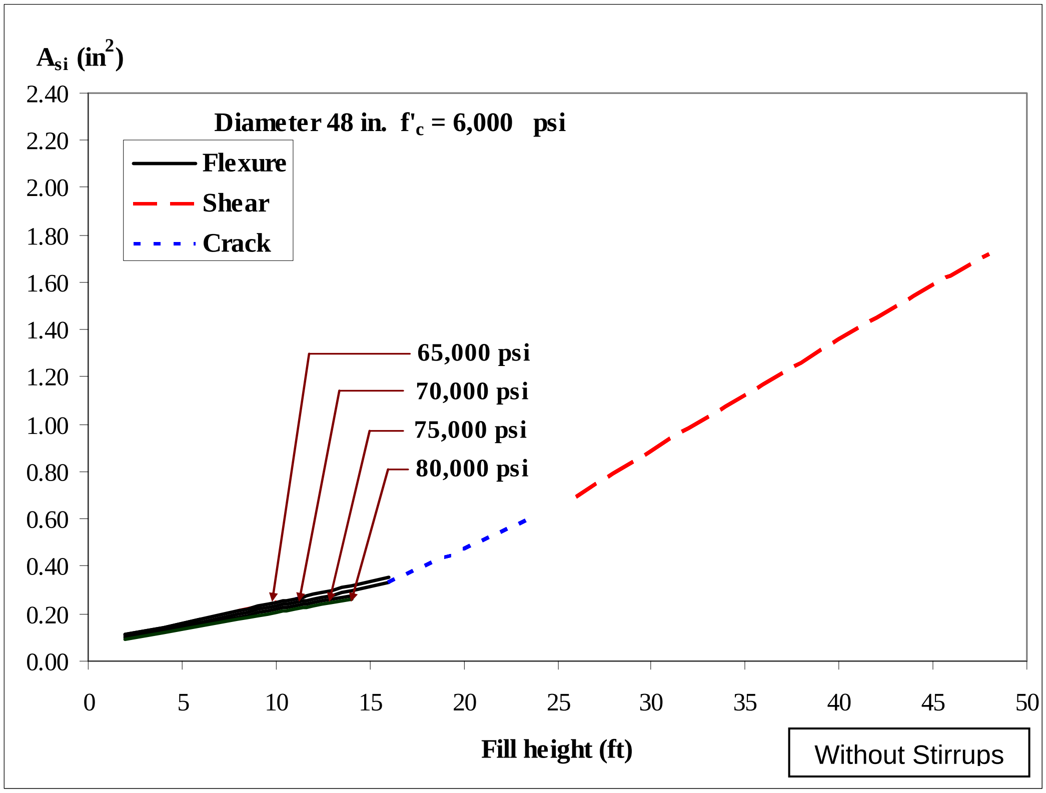


Figure 5 *Parametric Study – Reinforcing Steel Strength*

Figure 6 is a modified version of Figure 5 where the horizontal axis representing fill height has been reduced to illustrate the effect of reinforcing steel strength in the region of pipe behavior controlled by flexure. Increasing the reinforcing steel strength allows for a reduction in the required amount of reinforcing steel area however, the effect is not significant. Therefore, increasing reinforcing steel strength may not be economical for improving the supporting capacity of reinforced concrete pipe.

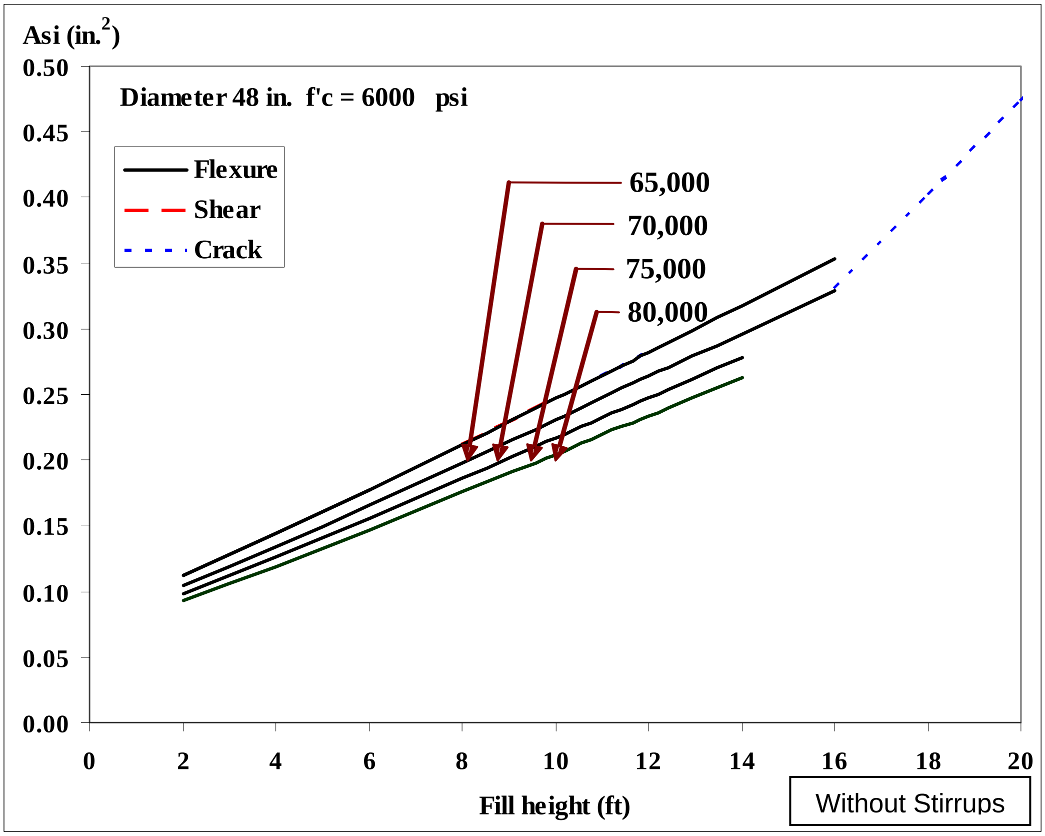


Figure 6 *Parametric Study – Reinforcing Steel Strength*

Parametric Study for Crack Control Factor

Figure 7 summarizes the results of varying the crack control factor, F_{cr}, through the SIDD acceptable range from a minimum of 0.7 to a maximum of 1.3. The crack control factor becomes more conservative as its value approaches the minimum. If F_{cr}=1.0, there is a 50% probability that cracks larger than 0.01-inches in width will occur at the design service load (ASCE15-98). In Figure 7, minimum and maximum values are plotted in addition to the commonly used value of 0.9. At the minimum value, crack control will govern design for medium to deep fill heights. Increasing F_{cr} reduces or eliminates the region of fill heights where crack control governs pipe design. From Figure 7, it can be seen that if the minimum value for F_{cr} (0.7) were used, crack control would govern the design of pipe for all fill heights above 14 feet. However, the difference between the maximum value of 1.3 and the commonly used value of 0.9 is very subtle. Therefore, even though the selection of the conservative value of 0.9 is somewhat arbitrary, it does not affect efficient design of concrete pipes.

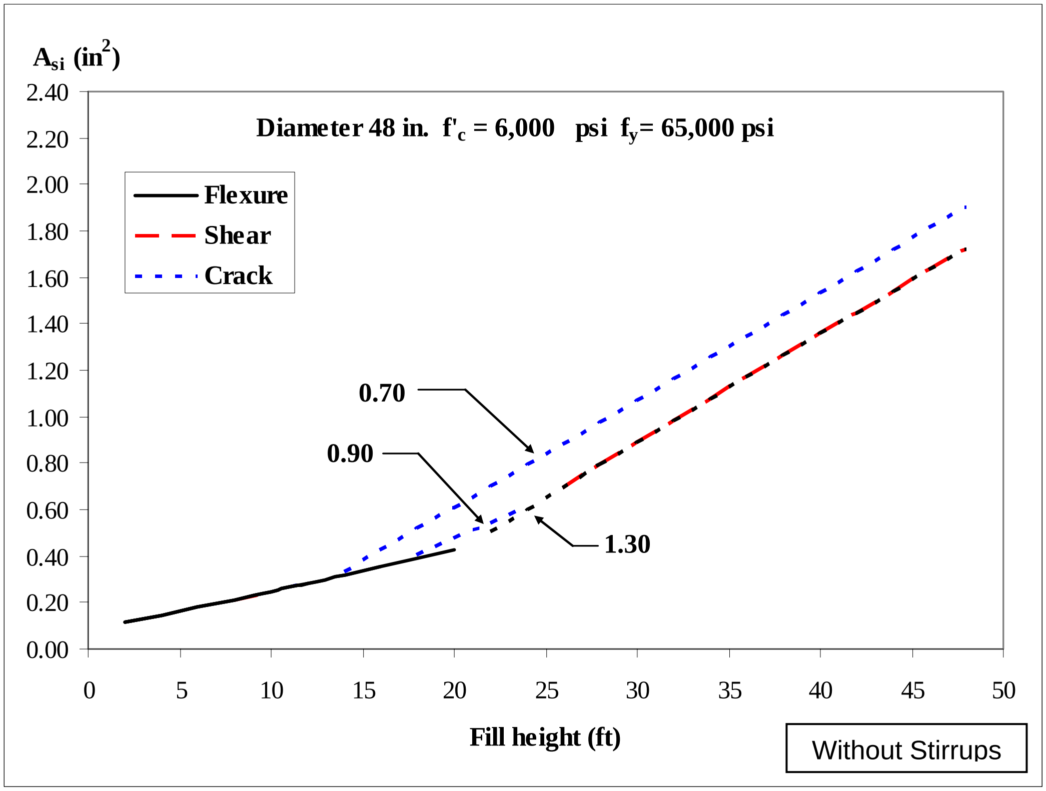


Figure 7 *Parametric Study – Crack Control Factor*

The controlling criteria examined in the parametric study were: flexure, shear, and crack control. The parameters studied were: concrete strength, steel reinforcing strength, and crack control factor. Stirrups were not considered for any designs evaluated in the parametric study. Increasing concrete strength results in a substantial decrease in the amount of steel reinforcing required. Increasing the reinforcing steel strength has a minor effect on supporting strength. The selection of the crack control factor is not clear, yet it can control the design in some cases.

3. REVIEW OF NDOR PIPE POLICY

A review of the current NDOR Pipe Policy was performed to evaluate its efficiency and identify possible suggestions for improvement. Based on the research carried out, and the discussions held with the NDOR Technical Advisory Committee (TAC), the methodology used to generate the NDOR fill height tables is determined. The procedure for creating the NDOR fill height tables is based on executing designs using a computer program PipeCar. PipeCar is a direct design-based software program published by the ACPA. The user specifies pipe geometry, loading data, material properties, and design data. The minimum required user input is: pipe geometry, depth of fill, and loading type. It must be noted that depth of fill is a user input parameter. A default parameter file supplies the remaining data. This file may be user modified to provide a unique set of default values. The user runs the PipeCar software with input parameters based on NDOR specifications. Therefore, NDOR starts the design process with direct design. The software performs a structural analysis and the required reinforcement area is generated as output. Then, the designer compares the required steel area determined by PipeCar to the ASTM C76 tables. These tables present the required reinforcement area based on a given D-load, pipe diameter, pipe wall thickness, and concrete strength. If the PipeCar reinforcing area is equal to the reinforcing area specified by ASTM C76, the input fill height is acceptable. If not, the fill height input is reduced and an iterative process is used to determine an acceptable fill height. If the PipeCar reinforcing area is less than the ASTM C76 specification, the fill height input is increased and an iterative process determines the maximum acceptable fill height. Thus, the procedure is limited by ASTM specifications. It should be noted here that ASTM tables are mainly for use with the ACPA fill height tables or indirect design methods. Although direct design is identified as the preferred design method, if at this stage, the reinforcement area designed by the PipeCar software is larger than that given by the ASTM table, the user needs to go back to PipeCar and reduce the fill height.

It should be noted and emphasized that the ASTM tables are indirect design based. It is not clearly stated anywhere that ASTM C76 is an indirect design-based document, however, the tables present reinforcement areas for a given D-load, pipe diameter, pipe class, and wall type. These are empirical tables based on three-edge-bearing test results, i.e. the basis of indirect design method. It probably is clear by now to an experienced reinforced concrete pipe designer that the ASTM tables would work best in conjunction with the ACPA fill height tables, and not the PipeCar software.

Moreover, PipeCar allows designs with shear reinforcement, which are considered a special design by NDOR. Therefore, some possibilities that satisfy the ASTM C76 criteria are not included in the NDOR tables. This results in additional tasks for the NDOR staff if the designers or owners need to use shear reinforcement, such would be necessary for deeper fill heights. Although NDOR staff reports that such jobs constitute a small percentage of all pipe installations, an expanded table would provide more options to the users.

Based on these observations of the NDOR pipe policy, the research team’s main suggestion is to perform future work aiming at eliminating the interdependence between the direct and indirect design methods and provide an expanded table with experimentally validated entries. However, the first task is to regenerate the existing fill height tables using the same method as NDOR, yet making the implicit assumptions buried in the table clearer and more up-to-date.

Table 2 illustrates the comparison between fill heights generated with different assumptions. The table gives fill heights for Class III, IV, and V pipe based on NDOR standards (NDOR), AASHTO STD (STD), AASHTO LRFD (LRFD), and the ACPA indirect design (ID) fill height tables. The STD and LRFD columns were generated by the University of Nebraska research team. The fill heights were verified in an iterative process following the NDOR procedure which limits the reinforcing areas and concrete strength to the ASTM C76 specification. The additional fill height in the STD column is a result of assuming that the welded wire fabric making up the reinforcing cages has a wire-to-wire spacing of no more than four inches. A more conservative assumption would follow the ASTM guidelines which allow a maximum spacing equal to the smaller of the thickness of the pipe wall or six inches. The assumption for a four inch spacing results from discussion of fabrication methods with industry pipe producers. The additional fill height in the LRFD column is a result of the same assumption regarding wire spacing and the change in design criteria from STD to LRFD. Generally, the NDOR specifications are the most conservative, while the ACPA indirect design fill height tables are the least conservative.

***Table 2** Fill Height Table Comparison*

	Class III				Class IV				Class V			
pipe diameter (inches)	fill height (feet)				fill height (feet)				fill height (feet)			
	NDOR	**STD**	**LRFD**	**ID**	**NDOR**	**STD**	**LRFD**	**ID**	**NDOR**	**STD**	**LRFD**	**ID**
15	12	12	13	14	15	15	16	22	21	21	22	33
18	12	12	13	15	17	17	18	22	24	24	25	34
21	13	13	13	15	19	19	20	22	26	26	27	34
24	13	13	12	15	19	19	20	22	26	26	27	34
27	13	13	13	14	17	17	17	22	26	26	27	34
30	12	12	12	14	14	14	15	22	25	25	25	33
36	10	10	11	14	16	16	17	22	24	24	25	33
42	10	10	11	14	15	15	16	22	23	23	24	33
48	10	10	11	14	14	15	15	21	22	23	24	33
54	10	10	11	14	14	15	15	21				
60	9	10	10	14	14	15	16	21				
66	9	10	10	14	14	16	16	21				
72	9	10	10	13	14	16	16	21				
78	9	10	11	13								
84	9	10	10	13								
90	9	10	11	13								
96	9	10	11	13								
102	10	11	11	-								
108	10	11	11	-								

Based on this comparative study, the research team suggests, at the very least, the fill height values generated using AASHTO LRFD (highlighted). As mentioned before, the team also suggests that further improvements are possible if the direct design method is used more effectively. Figure 8 presents a simplified flowchart describing the findings of this project and suggestions for future work. The research team's review of existing procedures indicates that based on current standard inventory of concrete pipe available from Industry, NDOR is appropriately using the Direct Design to determine fill heights, as shown in Table 2 Fill Height Table Comparison.

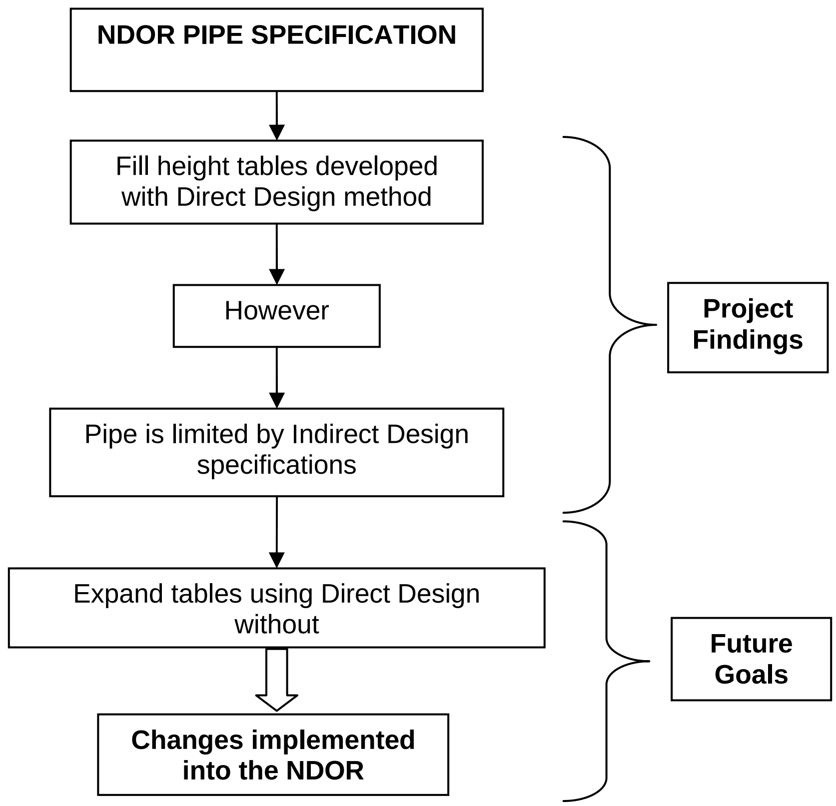


Figure 8 *NDOR Pipe Policy Review*

4. ONLINE SURVEY FOR DOT PIPE DESIGN

An online survey was administered to discover similarities and differences among department of transportation specifications for reinforced concrete pipe design nationwide. The survey itself and results are presented in this chapter. The preliminary results of the survey indicate that among respondents, the indirect design method is more common than the direct design method. This suggests that the development and introduction of a unified, efficient, and state-of-the-art method for the design and specification of reinforced concrete pipe could be utilized nation-wide.

Online Survey and Results

1. Please provide your contact information	Number of Responses	Response Ratio
AK, AZ, AR, CT, IL, IA, KS, LA, MI, NY, OR, TX, VA, WA Alberta, New Brunswick	16	N/A

2. Which method of concrete pipe design does your State DOT utilize?	Number of Responses	Response Ratio
Direct Design	6	43%
Indirect Design	9	64%

3. What is the pipe design procedure used in your state? (Please check all that apply)	Number of Responses	Response Ratio
We have our own design standards based on the design method used	5	31%
We approve concrete pipe designs on a case by case basis	3	19%
Our designers specify pipe manufacturing to meet ASTM C-76, regardless of design method used	8	50%
Our designers use AASHTO LRFD Specifications for load and resistance factors	0	0%
Our designers use AASHTO STD Specifications for load and resistance factors	1	6%
Other, please specify	8	50%

1. AASHTO M_170 or ASTM C-76
2. Canadian Hwy Bridge Design Code (CAN/CSA-S6-00)
3. CSA A257
4. We rarely use concrete pipe, prb LRFD now
5. Our specs. Have a class of pipe vs. size & fill ht.
6. Please contact gdouglas@dot.state.ny.us
7. SIDD (Std. Installation Direct Design) and PipeCar
8. AASHTO M170

Online Survey and Results (Cont'd)

4. Is the use of shear reinforcement common practice?	Number of Responses	Response Ratio
Yes	5	36%
No	9	64%
Total	14	100%

1. WSDOT allows either un-reinforced or reinforced concrete pipe, depending on the height of cover for the specific application.
2. Alberta Transportation has outsourced all design and construction super-vision of bridge structures. We employ very little concrete pipe in bridge construction, and when we do it's usually a jacking/tunelling project.
3. Considered with 40 ft. of fill or more.
4. Not sure. Our standard specifications for concrete pipe are provided online at http://www.dot.state.ny.us/specs/2002specbook.html 706-02
5. In all Indirect Design of large diam. with high strength class of pipes are governed in shear in TEB tests; however, in Direct Design method, the design area of the inner reinf. is increased to provide increased shear without the use of stirrups.

5. What is the concrete strength most commonly used for pipes in your state?	Number of Responses	Response Ratio
4,000 psi	10	63%
5,000 psi	3	19%
6,000 psi	2	13%
7,000 psi	0	0%
Other, please specify	5	31%

1. As per AASHTO M-170
2. As specified by ASTM C 76 for the class of pipe
3. ASTM Class II furnished unless stronger is specified
4. Please see 4
5. Higher conc. strength if designed to ASTM C 655.

5. COLLABORATION WITH INDUSTRY

Throughout the project's duration, the research team was active in meeting with the concrete pipe producers and industry members. The team visited two pipe production plants in Nebraska, met with ACPA representative Josh Beakley several times to discuss the project, and attended the Rigid Pipe Committee Meetings during the Annual Transportation Research Board Conference in Washington, D.C.

The research team visited the *Rinker Materials Hydro Conduit* plant and the *Concrete Industries* plant. Details of construction, including fabrication of steel reinforcing cages and reinforced pipe were noted and recorded. The methods of identifying and testing the finished pipe were noted. One of the important findings from these observations is the fact that the pipes currently produced are typically made of concrete with an inherent strength exceeding 4,000 psi, even though the reported pipe capacity is always based on 4,000 psi.

The research team attended the TRB conference in Washington, D.C. The committee listened to the project progress and as a result shared the team's concerns regarding the current pipe design practice, the discrepancies between the two available methods, and the lack of recent developments and research in the design methods. ACPA created a Technical Resources task group to perform a study parallel to this NDOR-University of Nebraska study, regarding the discrepancies between indirect and direct design methods.

6. REVIEW OF LITERATURE ON BEDDING FACTOR

The indirect design method is still commonly used by consulting engineers both in the state of Nebraska and around the nation. One indirect design parameter, the bedding factor, strongly affects the design results. However, the selection of the factor and the concerns regarding the control of construction for the selected bedding factor usually leads to overly conservative designs. Thus, the research team started a detailed review of the literature to understand the basis of the bedding factor phenomenon, its development and applications, and possibilities for improvements on the assumptions and use. A journal article including the findings of this review is in progress, which will serve to educate the practicing engineers regarding the implications of installation types and bedding factors. This study and the journal article will benefit indirect design users.

The bedding factor is used in indirect design to relate the strength of pipe in the three-edge-bearing test to the strength of pipe in the installed condition. The major references reviewed in this study are: “The Supporting Strength of Rigid Pipe Culverts” by Spangler (1933), Concrete Pipe Info #12 by ACPA (1991), and Design Data #40 by ACPA (1996).

Marston Spangler developed the concept of bedding factor during pioneering work performed at Iowa State University in the 1930’s and published in the report, “The Supporting Strength of Rigid Pipe Culverts” (1933). The fundamental definition of the bedding factor is the ratio of the vertical load which causes cracking in the field to the vertical load which causes cracking in a three-edge-bearing test. Early bedding factors were evaluated from experimental work. The fundamental bedding factor relationship is expressed in Equation (7).

$$B_f = \frac{W_e}{TEB} \qquad (7)$$

Spangler concluded that the bedding factor is a function of the width and quality of contact between the pipe and bedding material. The bedding factor is also dependent on the magnitude of lateral pressure and the portion of the vertical height of the pipe over which this pressure acts. Lateral pressure causes bending moments in the pipe wall, which act opposite to the bending moments resulting from vertical soil pressure. An analytical expression for the bedding factor was developed from these statements and is presented as Equation (8). The moments produced by lateral soil pressure are therefore beneficial to the supporting strength of the pipe.

$$B_f = \frac{1.431}{N - xq} \qquad (8)$$

Where:

B_f	=	*bedding factor*
N	=	*constant depending on distribution of vertical loading and vertical reaction*
x	=	*function of distribution of lateral pressure*
q	=	*ratio of total lateral pressure to total vertical pressure*

Concrete Pipe Info #12 (1991) is an ACPA publication that updates the concept and calculation of bedding factors. The bedding factor is inversely proportional to the required D-load, Equation (9).

$$D_{load} = \frac{W}{B_f} \times \frac{F.S.}{D} \qquad (9)$$

Where:

B_f	=	*bedding factor*
W	=	*total load (lbs/ft)*
F.S.	=	*factor of safety*
D	=	*pipe diameter (feet)*

Concrete pipe does not experience significant deflections under service loading and therefore, passive earth pressure is not considered. Axial thrust is not considered although bending moments caused by lateral pressure are considered. Axial thrust has a positive effect on pipe capacity and it is therefore conservative to neglect these effects in the calculation of the bedding factor.

Design Data 40 (1996) is the newest ACPA publication pertaining to bedding factors. In this document, the bedding factors are re-developed for the latest standard installations and Heger pressure distributions, and axial thrust is considered in the development of the updated bedding factors. The conclusions from this study up to date are:

- bedding factors are conservative with respect to the actual supporting strength of concrete pipe
- lateral pressure acting on the pipe produces bending moments in the opposite direction of the bending moments produced by vertical loading and should be accounted for in the formulation of the bedding factor
- axial thrust has a positive effect on flexural stresses in the pipe wall and should therefore be considered when calculating the bedding factor.

This study is ongoing and a journal article is being prepared on the topic.

7. DEVELOPMENT OF EFFICIENT SHEAR REINFORCING

According to direct design, for deeper fill heights and large pipe diameters, shear capacity will control design. Research indicates that shear reinforcement may considerably improve pipe capacity. However, it is usually considered special design due to the fact that currently available shear reinforcement is expensive as it needs to be ordered specially and cannot easily be produced at the pipe manufacturing plants. Therefore it is possible if an inexpensive, easy-to-manufacture stirrup ring is designed; the shear capacity of pipe can be economically increased resulting in more efficient installations, where deep fill heights and large diameters are required. This study is outside the scope of the current project; however, the team suggests that it will provide important input for future tasks.

A proposed method for the efficient fabrication of such reinforcing is illustrated in Figure 9. Diagonal strips are cut from wire fabric and then formed into rings that are welded to a central reinforcing ring. These rings are assembled to form the reinforcing cage and the concrete is cast around the cage to complete fabrication.

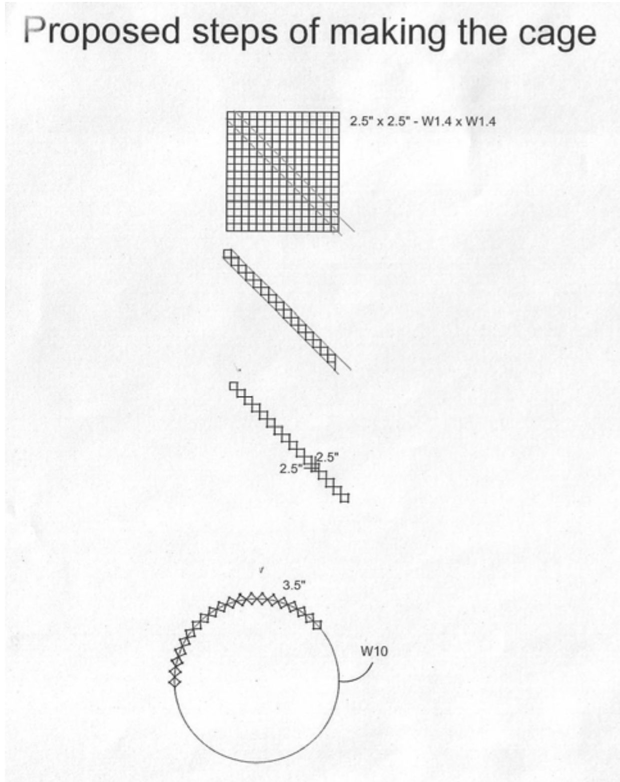


Figure 9 *Fabrication of Shear Reinforcing*

The following tasks remain for this pilot experimental study:

- prepare the reinforcing sample
- send the cages to plant for production
- gather and test the specimens
- compare shear capacities of the pipe to evaluate feasibility of shear reinforcing
- develop suggestions for stirrup use and design

Three specimens will be developed:

- one with the proposed shear reinforcing
- one with no shear reinforcing
- one unreinforced specimen.

8. SUMMARY AND DISCUSSION

The findings from this study suggest that the NDOR policies can be updated as follows: The existing fill height tables can be updated (Chapter 3) and then expanded to include more design options for the pipe designer and manufacturer. All proposed changes would be validated through laboratory experiments. The research team also envisions substantial contributions can be made to the current pipe design practice at a more fundamental and nation-wide level. This research could develop a unified, efficient design method that eliminates the confusion and discrepancies between the current design methods. The research team's review of existing procedures indicates that based on the current standard inventory of concrete pipe available from Industry, NDOR is appropriately using the Direct Design to determine fill heights, as shown in Table 2 Fill Height Table Comparison.

9. FUTURE WORK (Phase II)

The objective of Phase II is to develop a unified and efficient design procedure for reinforced concrete pipe that satisfies both designers and pipe producers in the state of Nebraska. This will be achieved by updating and expanding the NDOR fill height table based on the results of the University of Nebraska parametric study, and evaluation and validation of suggested changes through laboratory experiments. Training sessions and seminars will be developed to introduce the unified, user-friendly design criteria and procedures based on the results of Phase II. To accomplish these objectives, the following tasks are proposed for Phase II:

1. Experimental program to validate the suggested design procedures:

An experimental program for the empirical and rational evaluation of the suggested design criteria will be carried out. Three-edge bearing tests will be carried out to examine possible improvement of the design parameters as a result of advancements in the material properties and production technology.

2. Analysis of the experimental results.

The structural behavior of reinforced concrete pipes based on experimental results will be compared to the national standards, other state specifications, as well as previous and suggested NDOR specifications. These analyses will include evaluation of resistance of concrete pipes constructed with high performance concrete for combined flexure, shear, axial load, radial tension and crack control.

3. Preparation of a detailed set of design criteria and specifications validated by the experimental results.

Revisions to the NDOR pipe specifications suggested based on the literature survey and a theoretical study carried out in Phase I will be evaluated using the experimental results in Phase II. As a result, a complete set of revisions validated through experimental results will be developed. Load charts and tables of standard design including construction specifications will be proposed for the adoption of the State of Nebraska.

4. Development of detailed examples.

The examples will cover the new design procedures for easy adoption by the NDOR designers and consultants.

5. Organization of a workshop for NDOR designer and consultants.

A PowerPoint presentation summarizing the research and its findings will be prepared and submitted to NDOR. This presentation along with new NDOR standards and numerical examples will be used in the workshop.

6. Preparation of the final report.

The entire research effort, recommended specifications, example and analysis guidelines will be documented in a final comprehensive report. An executive summary, or technical brief will also be submitted for possible posting on the NDOR website.

10. REFERENCES

1. AASHTO, LRFD Bridge Design Specifications, Second Edition 1998 and Interims 1999 and 2000, American Association for State Highway and Transportation Officials, Washington, D.C.
2. AASHTO, Standard Specifications for Transportation Materials and Methods of Sampling and Testing, Seventeenth Edition 1995
3. American Concrete Institute (ACI), Building Code Requirement for Reinforced Concrete, ACI 318-02/318R-02
4. American Society of Civil Engineers (ASCE), Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD), 1993
5. American Society of Civil Engineers (ASCE), Standard Practice for Direct Design of Buried Precast Concrete Pipe Using Standard Installations (SIDD), 1998
6. American Concrete Pipe Association, Concrete pipe Design Manual, Fourteenth Printing, 2001
7. American Concrete Pipe Association, Concrete Pipe Info #12, 1991
8. American Concrete Pipe Association, Design Data 40, 1996
9. American Concrete Pipe Association, Concrete pipe Design Manual, Fourteenth Printing, 2001
10. Heger, Frank J., “Structural Behavior of Circular Reinforced Concrete Pipe-Development of Theory, ACI Journal,” Proceeding V. 60, No. 11, Nov. 1963, pp. 1567-1614

11. Heger, Frank J.; Nawy, Edward L.; and Saba, Robert B., “Structural Behavior of Circular Concrete Pipe Reinforced with Welded Wire Fabric,” ACI Journal, Proceedings V. 60, No. 10, Oct. 1963, pp. 1389-1414
12. Spangler, M.G. (1933). “The Supporting Strength of Rigid Pipe Culverts,” Bulletin 112, Iowa State College
13. Tadros, M.K.; Benak, J.V.; Gilliland, M.K. (1989). “Soil Pressure on Box Culverts,” ACI Structural Journal, V. 86, No. 4, July-August 1989, pp. 439-450

11. APPENDIX

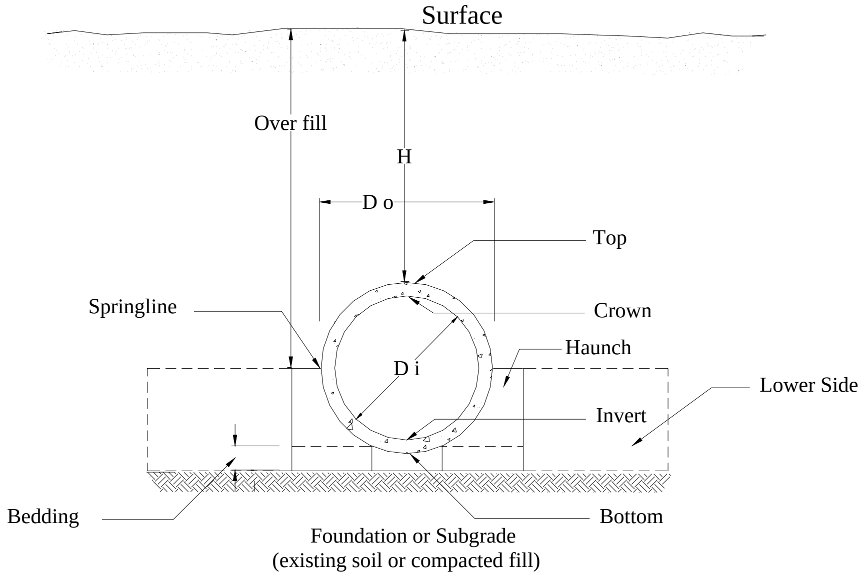


Figure A-1 *Standard Installation Terminology*

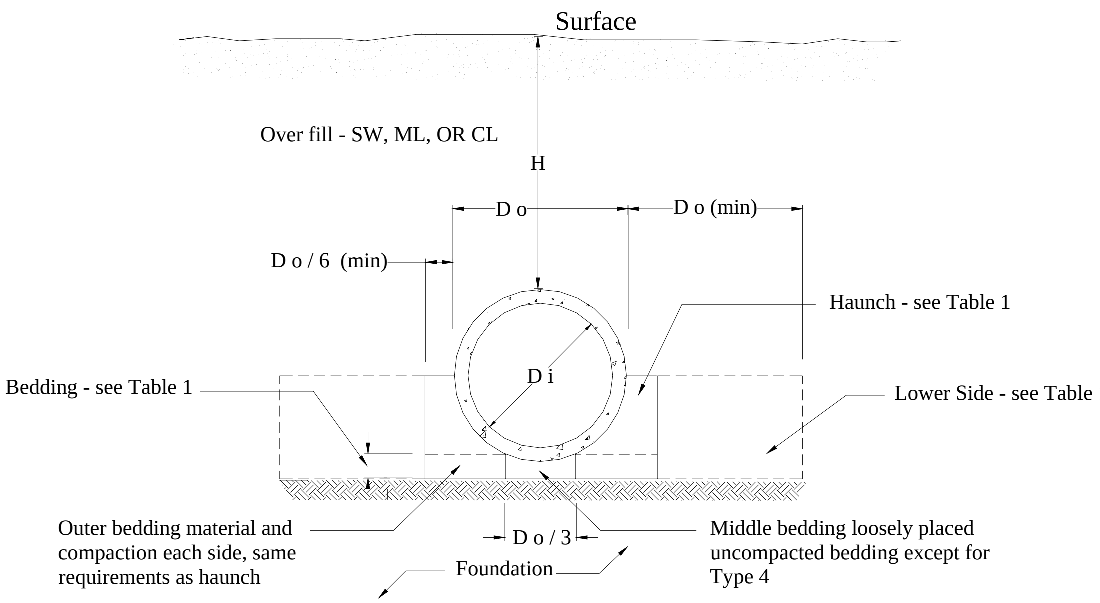


Figure A-2 *Standard Embankment Installations*
Note: See Table A-2

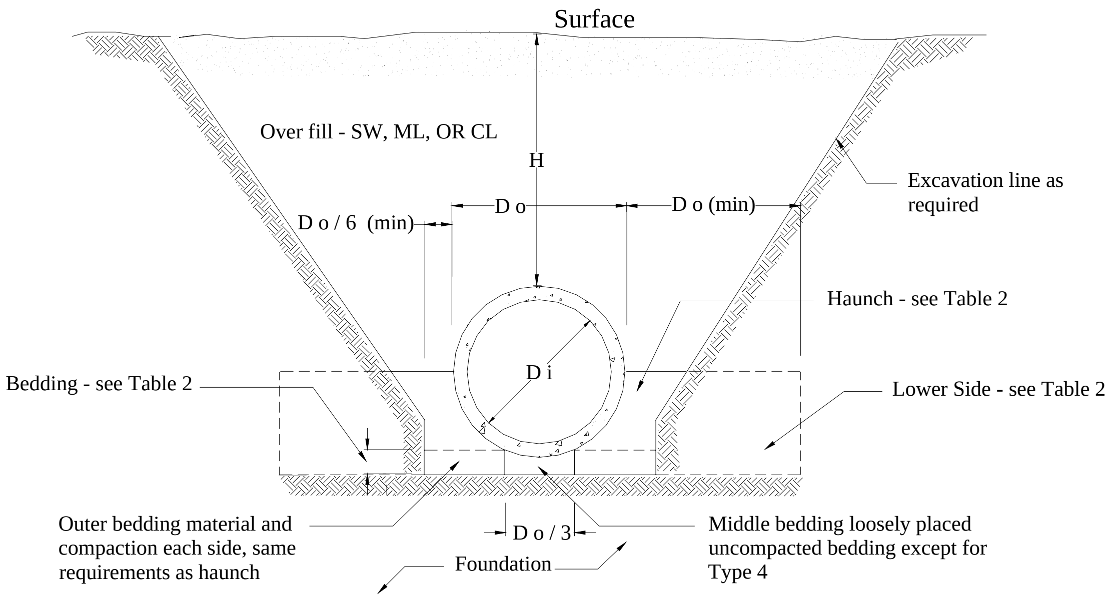
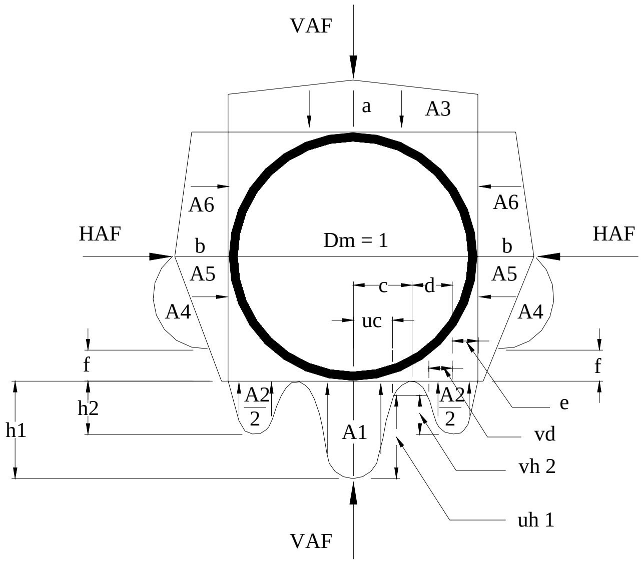


Figure A-3 *Standard Trench Installations*
Note: See Table A-3

Table A-1 *Arching Coefficients and Heger Earth Pressure Distribution*



Installation Type	VAF	HAF	A1	A2	A3	A4	A5	A6	a	b	c	e	f	u	v
1	1.35	0.45	0.62	0.73	1.35	0.19	0.08	0.18	1.40	0.40	0.18	0.08	0.05	0.80	0.80
2	1.40	0.40	0.85	0.55	1.40	0.15	0.08	0.17	1.45	0.40	0.19	0.10	0.05	0.82	0.70
3	1.40	0.37	1.05	0.35	1.40	0.10	0.10	0.17	1.45	0.36	0.20	0.12	0.05	0.85	0.60
4	1.45	0.30	1.45	0.00	1.45	0.00	0.11	0.19	1.45	0.30	0.25	0.00	-	0.90	-

Notes:

1. VAF and HAF are vertical and horizontal arching factors. These coefficients represent non-dimensional total vertical and horizontal loads on the pipe, respectively. The actual total vertical and horizontal loads are (VAF) x (PL) and (HAF) x (PL), respectively, where PL is the prism load.
2. PL, the prism load, is the weight of the column of earth cover over the pipe outside diameter and is calculated as:

$$\text{English units} \quad PL = \left[\frac{wD_o}{12}\right]\left[H + \left(\frac{0.107\,D_o}{12}\right)\right]$$

$$\text{SI units} \quad PL = \left[\frac{wD_o}{1{,}000}\right]\left[H + \left(\frac{0.107\,D_o}{1{,}000}\right)\right]$$

3. Coefficients A1 through A6 represent the integration of non-dimensional vertical and horizontal components of soil pressure under the indicated portions of the component pressure diagrams (i.e., the area under the component pressure diagrams). The pressure are assumed to vary either parabolically or linearly, as shown, with the non-dimensional magnitudes at governing points represented by h1, h2, uh1, vh2, a, and b. Non-dimensional horizontal and vertical dimensions of component pressure regions are defined by c, d, e, uc, vd, and f coefficients.
4. d is calculated as (0.5-c-e)
   h1 is calculated as (1.5 A1)/(c) (1+u).
   h2 is calculated as (1.5 A2)/ ((d) (1+v) + (2e)).

Table A-2 *Standard Embankment Installation Soils and Minimum Compaction Requirements*

Installation type	Bedding thickness	Haunch and outer bedding	Lower side
Type 1	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	95% SW	90% SW, 95% ML, or 100% CL
Type 2	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	90% SW or 95% ML	85% SW, 90% ML, or 95% CL
Type 3	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL
Type 4	No bedding required, except if rock foundation, use D_o/12 minimum, not less than 6 in. (150 mm)	No compaction required, except if CL, use 85% CL	No compaction required, except if CL, use 85% CL

Notes:

1. Compaction and soil symbols, i.e., 95% SW, refer to SW soil material with a minimum Standard Proctor compaction of 95%. See Table 3 for equivalent Modified Proctor values.
2. Soil in the outer bedding, haunch, and lower side zones, except within Do/3 from the pipe springline, shall be compacted to at least the same compaction as the majority of soil in overfill zone.
3. Sub-trenches

3.1 A sub-trench is defined as a trench with its top below finished grade by more than 0.1H or, for roadways; its top is at an elevation lower than 1 ft (0.3 m) below the bottom of the pavement base material.

3.2 The minimum width of a sub-trench shall be 1.33 Do, or wider if required for adequate space to attain the specified compaction in the haunch and bedding zones.

3.3 For sub-trenches with walls of natural soil, any portion of the lower side zone in the sub-trench wall shall be at least as firm as an equivalent soil placed to the compaction requirement specified for the lower side zone and as firm as the majority of soil in the overfill zone or shall be removed and replaced with soil compacted to the specified level.

***Table A-3** Standard Trench Installation Soils and Minimum Compaction Requirements*

Installation type	Bedding thickness	Haunch and outer bedding	Lower side
Type 1	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	95% SW	90% SW, 95% ML, or 100% CL, or natural soils of equal firmness
Type 2	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	90% SW or 95% ML	85% SW, 90% ML, or 95% CL, or natural soils of equal firmness
Type 3	D_o/24 minimum, not less than 3 in. (75 mm). If rock foundation, use D_o/12 minimum, not less than 6 in. (150).	85% SW, 90% ML, or 95% CL	85% SW, 90% ML, or 95% CL, or natural soils of equal firmness
Type 4	No bedding required, except if rock foundation, use D_o/12 minimum, not less than 6 in. (150 mm)	No compaction required, except if CL, use 85% CL	85% SW, 90% ML, or 95% CL, or natural soils of equal firmness

Notes:

1. Compaction and soil symbols, i.e., 95% SW, refer to SW soil material with a minimum Standard Proctor compaction of 95%. See Table 3 for equivalent Modified Proctor values.
2. The trench top elevation shall be no lower than 0.1 H below finished grade or, for roadways; its top shall be no lower than an elevation of 1 ft. (0.3 m) below the bottom of pavement base material.
3. Earth loading shall be based on embankment conditions.
4. Soil in bedding and haunch zones shall be compacted to at least the same compaction as specified for the majority of soil in the backfill zone.
5. The trench width shall be wider than shown if required for adequate space to attain the specified compaction in the haunch and bedding zones.
6. For trench walls that are with 10 degrees of vertical, the compaction or firmness of the soil in the trench walls and lower side zone need not be considered.
7. For trench walls with greater than 10-degree slopes that consist of embankment, the lower side shall be compacted to at least the same compaction as specified compaction as specified for the soil in the backfill zone

Table A-4 *Equivalent USCS and AASHTO Soil Classification for SIDD Soil Designations*

SIDD Soil	**Representative Soil Types**		**Percent Compaction**	
	USCS	**AASHTO**	**Standard Proctor**	**Modified Proctor**
Gravelly sand (SW)	SW, SP, GW, GP	A1, A3	100 95 90 85 80 61	95 90 85 80 75 59
Sandy silt (ML)	GM, SM, ML; also GC, SC with less than 20% passing #200 sieve	A2, A4	100 95 90 85 80 49	95 90 85 80 75 46
Silty clay (CL)	CL, MH, GC, SC	A5, A6	100 95 90 85 80 45	90 85 80 75 70 40
	CH	A7	100 95 90 45	90 85 80 40

Table A-5 *Coefficients for Analysis of Pipe in Standard Installation*

Installation Type 1					Installation Type 2				
Location	Load Type	Coefficients			Location	Load Type	Coefficients		
		Cm_i	Cn_i	Cv_i			Cm_i	Cn_i	Cv_i
	Wp	.225	.077		Invert	Wp	.227	.077	
	We	.091	.188			We	.122	.169	
	Wf	.088	-.445			Wf	.111	-.437	
	WL1	.075	.250			WL1	.107	.205	
	WL2	.165	-.046			WL2	.189	-.035	
Crown	Wp	.079	-.077		Crown	Wp	.079	-.077	
	We	.083	.157			We	.094	.126	
	Wf	.057	-.187			Wf	.062	-.204	
	WL1	.068	.200			WL1	.080	.171	
	WL2	.236	.046			WL2	.241	.035	
Springline 90 degree	Wp	-.091	.249		Springline 90 degree	Wp	-.091	.249	
	We	-.077	.500			We	-.090	.500	
	Wf	-.064	-.068			Wf	-.070	-.068	
	WL1	-.065	.500			WL1	-.078	.513	
	WL2	-.154	.500			WL2	-.160	.500	
Critical shear invert	Wp		.174	.437	Critical shear invert	Wp		.177	.437
	We		.219	.143		We		.218	.198
	Wf		-.408	.141		Wf		-.386	.193
	WL1		.270	.150		WL1		.256	.188
Critical shear invert	Wp		-.055	.083	Critical shear invert	Wp		-.050	.088
	We		.205	.117		We		.185	.136
	Wf		-.176	.062		Wf		-.181	.074
	WL1		.250	.100		WL1		.205	.137

Installation Type 3					Installation Type 4				
Location	Load Type	Coefficients			Location	Load Type	Coefficients		
		Cm_i	Cn_i	Cv_i			Cm_i	Cn_i	Cv_i
Invert	Wp	.230	.077		Invert	Wp	.235	.077	
	We	.150	.163			We	.191	.128	
	Wf	.133	-.425			Wf	.160	-.403	
	WL1	.136	.199			WL1	.185	.152	
	WL2	.211	-.023			WL2	.237	-.004	
Crown	Wp	.079	-.077		Crown	Wp	.079	-.077	
	We	.103	.107			We	.118	.079	
	Wf	.068	-.215			Wf	.076	-.232	
	WL1	.091	.149			WL1	.110	.114	
	WL2	.247	.023			WL2	.255	.004	
Springline 85 degree	Wp	-.097	.271		Springline 80 degree	Wp	-.101	.287	
	We	-.103	.500			We	-.127	.504	
	Wf	-.081	-.063			Wf	-.095	-.057	
	WL1	-.126	.497			WL1	-.121	.495	
	WL2	-.155	.496			WL2	-.168	.492	
Critical shear invert	Wp		.177	.437	Critical shear invert	Wp		.188	.431
	We		.224	.249		We		.211	.309
	Wf		-.363	.238		Wf		-.323	.284
	WL1		.273	.224		WL1		.229	.305
Critical shear invert	Wp		-.044	.094	Critical shear invert	Wp		-.044	.100
	We		.173	.150		We		.151	.169
	Wf		-.193	.085		Wf		-.210	.096
	WL1		.224	.124		WL1		-.171	.152

Moment $M_i = C_{mi} W_i D_m/2$

Thrust $N_i = C_{ni} W_i$

Shear $V_i = C_{vi} W_i$