Simplified Shear Design of Structural Concrete Members

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

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The work was done under the general supervision of Neil M. Hawkins. The work at the University of Illinois was done under the supervision of both Neil M. Hawkins and Daniel A. Kuchma, with the assistance of Kang Su Kim, Sang-Ho Kim, and Shaoyun Sun.

FOREWORD

By David B. Beal
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This report contains the findings of research performed to develop practical equations for design of shear reinforcement in reinforced and prestressed concrete bridge girders. Recommended specifications, commentary, and examples illustrating application of the specifications were also developed. The material in this report will be of immediate interest to bridge designers.

Applying the LRFD shear provisions is difficult for designers. The sectional design model is not intuitively related to physical behavior, and the strut-and-tie model requires several trials to produce an efficient model and does not provide a unique solution. Mechanistic models that can be applied to shear design of conventional structures and to estimate shear reinforcement requirements in more complex structural configurations are needed. Such tools would permit designers to develop a more intuitive feel for shear reinforcement needs and permit verification of solutions developed from automated design software.

The objective of this research was to supplement the LRFD methods for shear design with procedures providing a direct solution for transverse and longitudinal reinforcement of concrete structures of common proportions. This work focused on development of resistance equations that yield unique solutions with defined limits of applicability. The recommended equations are similar in format and application to the resistance equations currently found in the AASHTO Standard Specifications. The equations apply to conventional structure types such as reinforced concrete T-beams, prestressed concrete I girders continuous for live load, prestressed concrete box beams, cast-in-place post-tensioned box girders, hammerhead piers and footings, and multi-post reinforced concrete bents and footings. The recommendations for additions to the LRFD specifications apply to precast concrete strengths up to 18 ksi and cast-in-place concrete strengths up to 10 ksi.

This research was performed by the University of Illinois at Urbana-Champaign. The report fully documents the research leading to the recommended shear design procedures and includes design examples. *NCHRP Web-Only Document 78* contains extensive supporting information, including a database that can be used to compare the predictions from the recommended procedures to existing design procedures. AASHTO is expected to consider these recommendations for adoption in 2007.

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SIMPLIFIED SHEAR DESIGN OF STRUCTURAL CONCRETE MEMBERS

SUMMARY

With the issuance of the *AASHTO LRFD Bridge Design Specifications* in 1994 (*1*), a new shear design method for reinforced concrete structures was introduced into U.S. bridge design practice. This method, known as the Sectional Design Model, is based on the Modified Compression Field Theory (MCFT) (*2*). That theory provides a complete behavioral model for the response of diagonally cracked concrete to in-plane shear and membrane stresses. In using the Sectional Design Model, the designer evaluates the axial strain in the member at mid-depth considering the combined actions of axial load, moment, prestressing, and shear, and then uses this strain and the shear design stress level (or cracking spacing) to select values for coefficients β and θ from tables. These values control the concrete and steel contributions to shear resistance. Although this method provided a unified treatment for the design of reinforced and prestressed concrete structures and offered some significant performance advantages, the procedure was unfamiliar to design engineers, more complicated than the shear design procedure in the AASHTO Standard Specifications, and often required an iterative solution. The objective of NCHRP Project 12-61 was to develop simplified shear design provisions that would provide an alternative shear design method to that of the LRFD Sectional Design Model.

There were many options for the structure of these new simplified provisions because there is considerable disagreement in the research community about the factors that most influence shear capacity. For this reason, the research approach taken on this project was to begin with a review and evaluation of some of the most prevalent methods for calculating shear capacity, including those of

- *ACI 318R-02* (*3*);
- *AASHTO Standard Specifications for Highway Bridges* 16th Edition (*4*);
- AASHTO 1979 provisions (*5*);
- CSA A23.3-94 (Canadian Standards Association: Design of Concrete Structures, 1994) (*6*);
- *AASHTO LRFD Bridge Design Specifications* 2nd Edition with 2003 Interim Revisions (*7*);
- CSA A23.3-04 (*8*);

- Eurocode EC2 (*9,10*);
- German Code (DIN, 2001) (*11*);
- *AASHTO Guide Specification for Design and Construction of Segmental Bridges* (ASBI) (*12*); and
- The Japanese Code (JSCE Standards, 1986) (*13*) and the shear design procedure recently developed by Tureyen and Frosch (*14*).

The structure and underlying bases for these methods were examined and their accuracies assessed using the results of a large experimental database. In addition, a survey was conducted of practitioners in 26 different state DOTs and federal lands bridge design agencies on the use of the LRFD Sectional Design Model and of the AASHTO standard shear design method.

These assessments resulted in the following findings subsequently used for developing change proposals and simplified provisions:

- The survey of the design practice showed that (1) few organizations had experience in the use of the LRFD shear design specifications. Some were reasonably comfortable with these provisions while others viewed them as a significant hurdle to be surmounted; (2) All agreed that the LRFD provisions must be automated with software if they are to be used in production design. This limitation naturally leads to loss of comfort with respect to the checking of designs, because the method cannot be readily executed by hand. Most designers also agree that the standard specification method for prestressed design that includes V_{ci} and V_{cw} must also be automated to be effective in production work, even though that method is executable by hand; (3) One of the most common concerns was that designers were losing their physical "feel" for shear design, owing to the increasing complexity of the design provisions and the resulting automation; and (4) The primary simplification that designers were seeking was an elimination of the iterative process required to determine the angle of diagonal compression.
- The changes incorporated in the 2004 Canadian Standards Association Code for the Design of Concrete Structures, CSA A23.3-04, greatly simplify the MCFT procedure for the design of concrete structures, using an approach that is functionally identical to the LRFD Sectional Design Model. In the CSA A23.3-04, the tables for evaluating β and θ were replaced by the following simple algebraic expressions:

$$\beta = \frac{4.8}{(1+1500\epsilon_x)}\frac{51}{(39+s_{xe})} \quad \text{where for members with } A_v < A_{v,min}$$

$$\beta = \frac{4.8}{(1+1500\epsilon_x)} \text{ for members with } A_v \geq A_{v,min}\text{, note } s_{xe} = 12 \text{ inches}$$

$$\theta = 29 + 7000\epsilon_x$$

Furthermore, the CSA procedure for evaluating β and θ in a design was made non-iterative by removing the dependency on the angle θ when calculating the longitudinal strain at mid-depth.

1. Traditional U.S. bridge and building design specifications use the diagonal cracking strength, V_c, as an estimate of the concrete contribution to shear resistance at the ultimate limit state and the 45-degree parallel chord truss model for calculating the contribution of shear reinforcement to shear capacity. These are empiri-

cal design approaches that are supported by test data. They were found to provide reasonably accurate and conservative estimates of the shear capacity of the members with shear reinforcement in the experimental database of shear test results. However, these methods were unconservative and poor at predicting the shear capacity of non-prestressed (reinforced) concrete members that did not contain shear reinforcement.

2. Basing the concrete contribution at ultimate on a conservative value of the diagonal cracking strength enables the designer to check whether or not a member will be cracked in shear under service load levels as well as helps in assessing the condition of structures in the field. It was also thought that characterizing the two types of diagonal cracking, web-shear and flexure-shear, as used in ACI 318-02 and the AASHTO Standard specifications, was useful for describing shear behavior.
3. The LRFD Sectional Design Model and the CSA Method produced very similar estimates of the shear capacity of the members in the experimental database of shear test results. From the various design methods considered, the LRFD and CSA methods produced the most accurate estimates of capacity and overall had only about a 10 percent probability of being unconservative.
4. Researchers have not tested the broad range of structures built with design provisions and thus experimental test data alone cannot provide a complete assessment of the suitability of provisions. For example, most members in the experimental database were small, simply-supported, stocky, did not contain shear reinforcement, and were loaded by point loads at small shear span to depth ratios. In addition, nearly all members were designed to be shear critical near an end support and thus test results are particularly ineffective at evaluating the appropriateness of provisions for regions away from supports.
5. Comparing the required strength of shear reinforcement ($\rho_v f_y$) by different design provisions with each other and with the required amounts determined by the analysis program, Response 2000 (R2K) (*15*), was a useful way of evaluating the relative conservatism of the different approaches.
6. The AASHTO LRFD Specifications require a larger minimum amount of shear reinforcement than most other codes. This higher requirement was found to be desirable for reliable behavior based on an examination of the experimental database of test results.
7. The CSA A23.3-04 (*8*), AASHTO (1979) (*5*), AASHTO LRFD (*1*, *7*), Truss Model with Crack Friction (TMwCF) (*16*), Eurocode 2 (*9*, *10*), JSCE (*13*), and DIN (*11*) all enable the designer to use an angle of diagonal compression, θ, flatter than 45 degrees when evaluating the contribution of shear reinforcement to shear capacity.
8. AASHTO LRFD, DIN, and Eurocode 2 allow the engineer to design members to support much larger shear stresses than permitted in other codes of practice. Any shear stress limit is principally intended to guard against diagonal compression failures. In AASHTO LRFD, the shear design stress limit is $0.25f_c'$ plus the vertical component of the prestressing while in ACI 318-02 or AASHTO Standard specifications the limit is approximately $12\sqrt{f_c'}$. The LRFD stress limit is adequate to prevent web crushing in regions where there is a uniform field of diagonal compression. However, this limit may be unconservative near supports where there is a significant magnification of the stress as the diagonal compression funnels into the support.

Based on these findings, two proposed changes to the LRFD specifications were developed. The first change is the introduction of proposed simplified provisions that are a modified version of the AASHTO Standard Specifications for prestressed concrete. These simplified provisions differ from the standard specifications in four principal ways:

1. The expression for calculating the web-shear cracking strength is made more conservative and applicable for partially prestressed as well as prestressed members;
2. A variable angle truss model is introduced in which the calculated angle of diagonal cracking is used for evaluating the contribution of the shear reinforcement in web-shear regions. In flexure-shear regions, and all regions where $M_u > M_{cr}$, the 45-degree truss model is used;
3. The maximum shear design stress is substantially increased; and
4. Minimum shear reinforcement requirements are made the same as those for the Sectional Design Model. Comparisons with the shear database showed the proposed simplified shear provisions to have a six percent probability of being unconservative.

The second change is that the LRFD Sectional Design Model be modified to use the relationships of the CSA Method for calculating β, θ, and ϵ_x.

The primary relationships in the proposed simplified provisions are expressed below in psi units:

$$V_{cw} = (1.9\sqrt{f_c'} + 0.30 f_{pc}) b_v d_v + V_p$$

$$V_{ci} = 0.632\sqrt{f_c'} b_v d_v + V_d + \frac{V_i M_{cr}}{M_{\max}} \geq 1.9\sqrt{f_c'} b_v d_v$$

$$V_s = \frac{A_v f_y d_v \cot(\theta)}{s} \text{ where } \cot(\theta) = 1.0 + 0.095\frac{f_{pc}}{\sqrt{f_c'}} \leq 1.8$$

$$\cot(\theta) = 1.0 \text{ in flexure-shear regions}$$

$$V_c + V_s \leq 0.25 f_c' b_v d_v + V_p \text{ where } V_c \text{ is lesser of } V_{cw} \text{ and } V_{ci}$$

The effect of the proposed changes on bridge design practice, if implemented, depends on which approach is used currently by designers (i.e., the AASHTO Standard or the AASHTO LRFD Sectional Design Method) and on which of the two proposed methods is selected for use. Switching from the AASHTO Standard procedure to either of the proposed design methods will allow for the design of members for considerably higher levels of shear stress and thereby enable the same size section to be used to span longer distances or support heavier loads. It will also involve an increase in the minimum required amounts of shear reinforcement which will improve safety. Adopting the equations for β, θ, and ϵ_x from the CSA Method into the LRFD Sectional Design Model will greatly improve the simplicity of designing by the Sectional Design Model. The CSA method can be used for the design of sections for shear that are subjected to any combination of axial load, moment, and level of prestressing. Adopting the proposed simplified provisions will result in a somewhat more uniformly conservative design procedure for the range of members that will be designed with the LRFD specifications.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

The goal of this project was to develop proposed simplified shear design provisions for the *AASHTO LRFD Bridge Design Specifications* that would overcome perceived difficulties with using the current shear design provisions, which are the provisions of the Sectional Design Model (A5.8.3). This Sectional Design Model constitutes the general shear design requirements in the first three editions of the *AASHTO LRFD Bridge Design Specifications* (*1, 7,* and *17*).

Section 1.1 describes the problem that led to this project and begins with a summary of the LRFD Sectional Design Model (A5.8.3), followed by a brief description of the basis of this model, and a discussion of the differences between the AASHTO LRFD and Standard Specifications (AASHTO, 2002) shear design provisions. Section 1.2 summarizes the information that was available to develop the proposed simplified provisions. This information consists of an overview of what is known about the mechanisms of shear resistance, a summary of code provisions, and descriptions of available experimental test data and analysis methods for shear. Section 1.3 defines project objectives, the approach used for meeting these objectives, and project tasks.

1.1 THE AASHTO LRFD SHEAR DESIGN SPECIFICATIONS

1.1.1 Summary of the LRFD Sectional Design Model (S5.8.3)

The AASHTO LRFD Section Design Model for Shear (A5.8.3) is a hand-based shear design procedure derived from the Modified Compression Field Theory (MCFT). Prior approaches focused on expressions for shear strength that were then modified for the effect of other forces. This is a comprehensive design approach for structural concrete members in which the combined actions of axial load, flexure, and prestressing are taken into account when completing the shear design of any section of any member. In this approach, the nominal shear capacity is taken as a sum of a concrete component, a shear reinforcement component, and the vertical (or transverse) component of the prestressing:

$$V_n = V_c + V_s + V_p \qquad \text{(Eq. 1)}$$

The concrete contribution is controlled by the value of the coefficient β as follows:.

$$V_c = 0.0316\beta\sqrt{f'_c}\,b_v d_v \quad \text{where } f'_c \text{ is in ksi units} \qquad \text{(Eq. 2)}$$

The coefficient of 0.0316 is $1/\sqrt{1000}$ and is used to convert the relationship for V_c from psi to ksi units.

A variable angle truss model is used to calculate the contribution of the shear reinforcement. See Equation 3 where the angle of the field of diagonal compression, θ, is used in calculating how many stirrups, $[d_v\cot(\theta)/s]$, are included in the transverse tie of the idealized truss.

$$V_s = \frac{A_v f_y d_v \cot(\theta)}{s} \qquad \text{(Eq. 3)}$$

where $d_v \geq 0.9d$ or $0.72h$, whichever is greater. (Eq. 4)

The values for β and θ are obtained from Table 1 for members that contain at least the minimum required amount of shear reinforcement (See Equation 5) and from Table 2 for members that contain less than that amount.

$$A_{v,\min} \geq 0.0316\sqrt{f'_c}\,\frac{b_v s}{f_y} \quad \text{where } f'_c \text{ and } f_y \text{ are in ksi units} \qquad \text{(Eq. 5)}$$

To obtain values for β and θ from Table 1 ($A_v < A_{v,min}$), the designer selects the row in which to enter the table from the shear design stress ratio (v/f'_c) and the column by the longitudinal strain ϵ_x at mid-depth, which may be taken as one-half of the strain in the longitudinal tension reinforcement, ϵ_t. This strain is equal to the force in the longitudinal tension reinforcement divided by the axial stiffness of the tension reinforcement. As shown in Equation 6 and illustrated in Figure 1, the effects of all demands on the longitudinal reinforcement are taken into account:

$$\epsilon_x = \frac{\epsilon_t}{2} = \frac{M_u/d_v + 0.5N_u + 0.5(V_u - V_p)\cot(\theta) - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \qquad \text{(Eq. 6)}$$

Equation 6 assumes that the member is cracked and, therefore, only the axial stiffness of the reinforcement need be

TABLE 1 Values of β and θ for members with at least minimum shear reinforcement

$\frac{v^*}{f'_c}$		Longitudinal Strain, $\varepsilon_x \times 1000$										
		≤ −0.20	≤ −0.10	≤ −0.05	≤ 0	≤ 0.125	≤ 0.25	≤ 0.50	≤ 0.75	≤ 1.00	≤ 1.50	≤ 2.00
≤ 0.075	θ	22.3°	20.4°	21.0°	21.8°	24.3°	26.6°	30.5°	33.7°	36.4°	40.8°	43.9°
	β	6.32	4.75	4.10	3.75	3.24	2.94	2.59	2.38	2.23	1.95	1.67
≤ 0.100	θ	18.1°	20.4°	21.4°	22.5°	24.9°	27.1°	30.8°	34.0°	36.7°	40.8°	43.1°
	β	3.79	3.38	3.24	3.14	2.91	2.75	2.50	2.32	2.18	1.93	1.69
≤ 0.125	θ	19.9°	21.9°	22.8°	23.7°	25.9°	27.9°	31.4°	34.4°	37.0°	41.0°	43.2°
	β	3.18	2.99	2.94	2.87	2.74	2.62	2.42	2.26	2.13	1.90	1.67
≤ 0.150	θ	21.6°	23.3°	24.2°	25.0°	26.9°	28.8°	32.1°	34.9°	37.3°	40.5°	42.8°
	β	2.88	2.79	2.78	2.72	2.60	2.52	2.36	2.21	2.08	1.82	1.61
≤ 0.175	θ	23.2°	24.7°	25.5°	26.2°	28.0°	29.7°	32.7°	35.2°	36.8°	39.7°	42.2°
	β	2.73	2.66	2.65	2.60	2.52	2.44	2.28	2.14	1.96	1.71	1.54
≤ 0.200	θ	24.7°	26.1°	26.7°	27.4°	29.0°	30.6°	32.8°	34.5°	36.1°	39.2°	41.7°
	β	2.63	2.59	2.52	2.51	2.43	2.37	2.14	1.94	1.79	1.61	1.47
≤ 0.225	θ	26.1°	27.3°	27.9°	28.5°	30.0°	30.8°	32.3°	34.0°	35.7°	38.8°	41.4°
	β	2.53	2.45	2.42	2.40	2.34	2.14	1.86	1.73	1.64	1.51	1.39
≤ 0.250	θ	27.5°	28.6°	29.1°	29.7°	30.6°	31.3°	32.8°	34.3°	35.8°	38.6°	41.2°
	β	2.39	2.39	2.33	2.33	2.12	1.93	1.70	1.58	1.50	1.38	1.29

$*v = V_u/b_v d_v$

considered when evaluating ϵ_t and ϵ_x. If ϵ_x is negative, then the member is uncracked and the axial stiffness of the uncracked concrete needs to be considered per Equation 7.

$$\epsilon_x = \frac{\epsilon_t}{2} = \frac{M_u / d_v + 0.5N_u + 0.5(V_u - V_p)\cot(\theta) - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps} + A_{ct} E_c)} \quad \text{(Eq. 7)}$$

where A_{ct} is the area of the concrete beneath mid-depth.

Alternatively, the designer can conservatively take $\epsilon_x = 0$ if Equation 6 yields a negative value.

Table 1 shows that as the longitudinal strain becomes larger, the values for β decrease and the values for θ increase. This means that as the moment and longitudinal strain increase, both the magnitude of the concrete and shear reinforcement contributions to shear resistance decrease.

To obtain values for β and θ when $A_v < A_{v,min}$, Table 2 is used. As for members containing at least minimum shear reinforcement, the column by which the designer enters Table 2 is based on the value of the longitudinal strain at mid-depth, ϵ_x. To determine the row, the spacing of the layers of crack control reinforcement is used, s_{xe} (see Equation 8 and Figure 2).

$$s_{xe} = \frac{1.38 s_x}{0.63 + a_g} \quad \text{(Eq. 8)}$$

where a_g is the maximum aggregate size in inches and taken equal to 0 when $f'_c \geq 10$ ksi.

Table 2 shows that as s_{xe} and ϵ_x increase, the value of β decreases and θ increases. The result is that, as the member becomes deeper and the value of the moment increases, the contributions of the concrete and shear reinforcement decrease.

The LRFD Sectional Design Model introduced a new requirement into shear design provisions—the direct consideration of shear in determining the required capacity of the longitudinal reinforcement at any point along the length of the member (see Equation 9).

$$T_{\min} \geq 0.5N_u + 0.5V_u \cot\theta + M_u/d_v - A_{ps} f_{ps} \quad \text{(Eq. 9)}$$

In the end regions of prestressed concrete members, the development length of the strands at the location of the first diagonal crack must be taken into consideration when satisfying the requirements of Equation 9.

In the design of a member by the LRFD Sectional Design Model, the member can be considered to be divided into design spans of length $d_v\cot(\theta)$ as shown in Figure 3. Each design span can be designed for the shear force midway along the length of the span. If the load is applied to the top of the member, then a staggered shear design concept may be used in which each design span is designed for the lowest value of shear occurring within the design span.

TABLE 2 Values of β and θ for members with less than minimum shear reinforcement

s_{xe} * (in)		Longitudinal Strain, $\varepsilon_x \times 1000$										
		≤ −0.20	≤ −0.10	≤ −0.05	≤ 0	≤ 0.125	≤ 0.25	≤ 0.50	≤ 0.75	≤ 1.00	≤ 1.50	≤ 2.00
≤ 5	θ	25.4°	25.5°	25.9°	26.4°	27.7°	28.9°	30.9°	32.4°	33.7°	35.6°	37.2°
	β	6.36	6.06	5.56	5.15	4.41	3.90	3.26	2.86	2.58	2.21	1.96
≤ 10	θ	27.6°	27.6°	28.3°	29.3°	31.6°	33.5°	36.3°	38.4°	40.1°	42.7°	44.7°
	β	5.78	5.78	5.38	4.89	4.05	3.52	2.88	2.50	2.23	1.88	1.65
≤ 15	θ	29.5°	29.5°	29.7°	31.1°	34.1°	36.5°	39.9°	42.4°	44.4°	47.4°	49.7°
	β	5.34	5.34	5.27	4.73	3.82	3.27	2.64	2.27	2.01	1.68	1.46
≤ 20	θ	31.2°	31.2°	31.2°	32.3°	36.0°	38.8°	42.7°	45.5°	47.6°	50.9°	53.4°
	β	4.99	4.99	4.99	4.61	3.65	3.09	2.46	2.09	1.85	1.52	1.31
≤ 30	θ	34.1°	34.1°	34.1°	34.2°	38.9°	42.3°	46.9°	50.1°	52.6°	56.2°	59.0°
	β	4.46	4.46	4.46	4.43	3.39	2.82	2.19	1.84	1.61	1.30	1.10
≤ 40	θ	36.6°	36.6°	36.6°	36.6°	41.1°	45.0°	50.2°	53.7°	56.3°	60.2°	63.0°
	β	4.06	4.06	4.06	4.06	3.20	2.62	2.00	1.66	1.43	1.14	0.95
≤ 60	θ	40.8°	40.8°	40.8°	40.8°	44.5°	49.2°	55.1°	58.9°	61.8°	65.8°	68.6°
	β	3.50	3.50	3.50	3.50	2.92	2.32	1.72	1.40	1.18	0.92	0.75
≤ 80	θ	44.3°	44.3°	44.3°	44.3°	47.1°	52.3°	58.7°	62.8°	65.7°	69.7°	72.4°
	β	3.10	3.10	3.10	3.10	2.71	2.11	1.52	1.21	1.01	0.76	0.62

$* s_{xe} = \dfrac{1.38 s_x}{0.63 + a_g}$ (*in. units*), where s_x = the lesser of d_v and the maximum distance between layers of crack control reinforcement (*in.*), a_g = maximum aggregate size (*in.*).

The Sectional Design Model was developed for regions in which engineering beam theory applies and there is a uniform flow of the diagonal compressive stresses. However, the LRFD specifications also permit the end region of members (the distance between the support and $d_v\cot(\theta)/2$ from the support) that are subject to a complex state of stress to be designed by the Sectional Design Model for the shear force at $d_v\cot(\theta)/2$ from the support.

Figure 4 is a flowchart of the entire procedure for use of the LRFD Sectional Design Model. To further illustrate this procedure, a brief example is given for the design of a section of the 72-inch-deep bulb-tee girder in Figure 5. (This

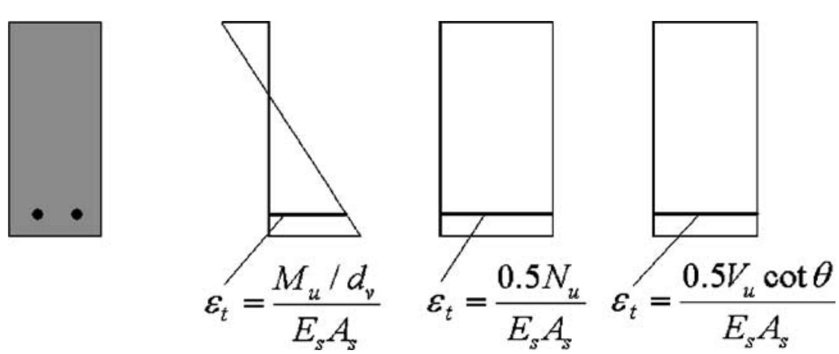


Figure 1. Effects of axial load, moment, shear, and prestressing on longitudinal strain in non-prestressed member.

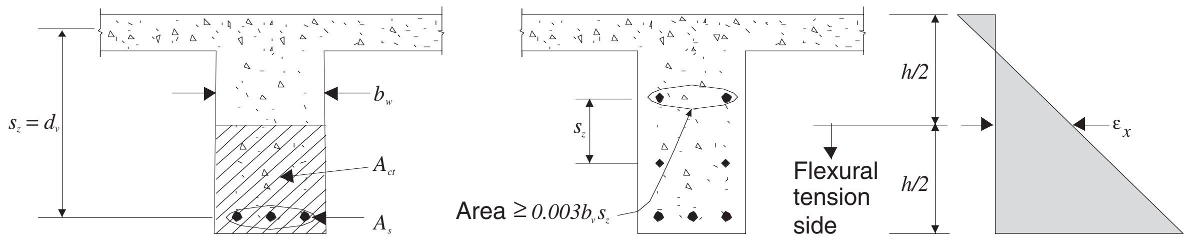


Figure 2. Evaluation of crack spacing parameter S_x.

example was extracted from a design of a 120-foot single-span AASHTO-PCI bulb-tee beam bridge with no skew. The example briefly illustrates the shear design procedure in LRFD specifications. The critical section is taken at 0.06L from centerline of a support.)

$V_u = 316.2$ kips, $M_u = 2134.0$ ft-kips, $N_u = 0$ kips, $V_p = 23.4$ kips

$A_s = 0$, $A_{ps} = 5.508$ in^2, $f_{po} = 189.0$ ksi, $f_c' = 6.5$ ksi, $E_p = 28500$ ks

$b_v = 6$ in, $d_v = 73.14$ in, $\phi = 0.9$

1. Compute shear stress ratio v_u/f_c'

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = 0.7473 \text{ ksi}$$

$$v_u/f_c' = 0.115$$

2. Assume ϵ_x as $-0.10 \times 10^{-3} \leq \epsilon_x \leq -0.05 \times 10^{-3}$, then obtain

$\theta = 22.8°$ and $\beta = 2.94$ from Table 1 (S5.8.3.4-1).

3. Compute ϵ_x

$$\epsilon_x = \frac{M_u/d_v + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})}$$

$$= -1.091 \times 10^{-3} \leq 0.002$$

Given that ϵ_x is negative, recalculate

$$\epsilon_x = \frac{M_u/d_v + 0.5N_u + 0.5(V_u - V_p)\cot\theta - A_{ps}f_{po}}{2(E_cA_c + E_sA_s + E_pA_{ps})}$$

$$= -0.080 \times 10^{-3}$$

if ϵ_x satisfies the assumed range, then $\theta = 22.8°$ and $\beta =$ 2.94 are O.K.

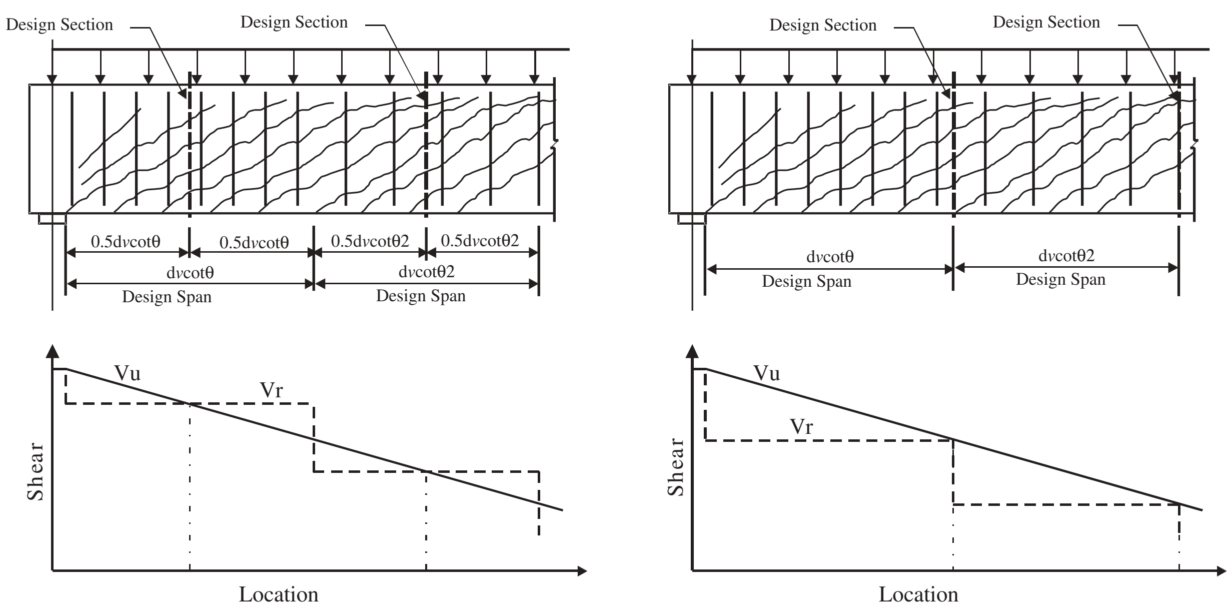


Figure 3. Design regions and shear demand using the sectional design model.

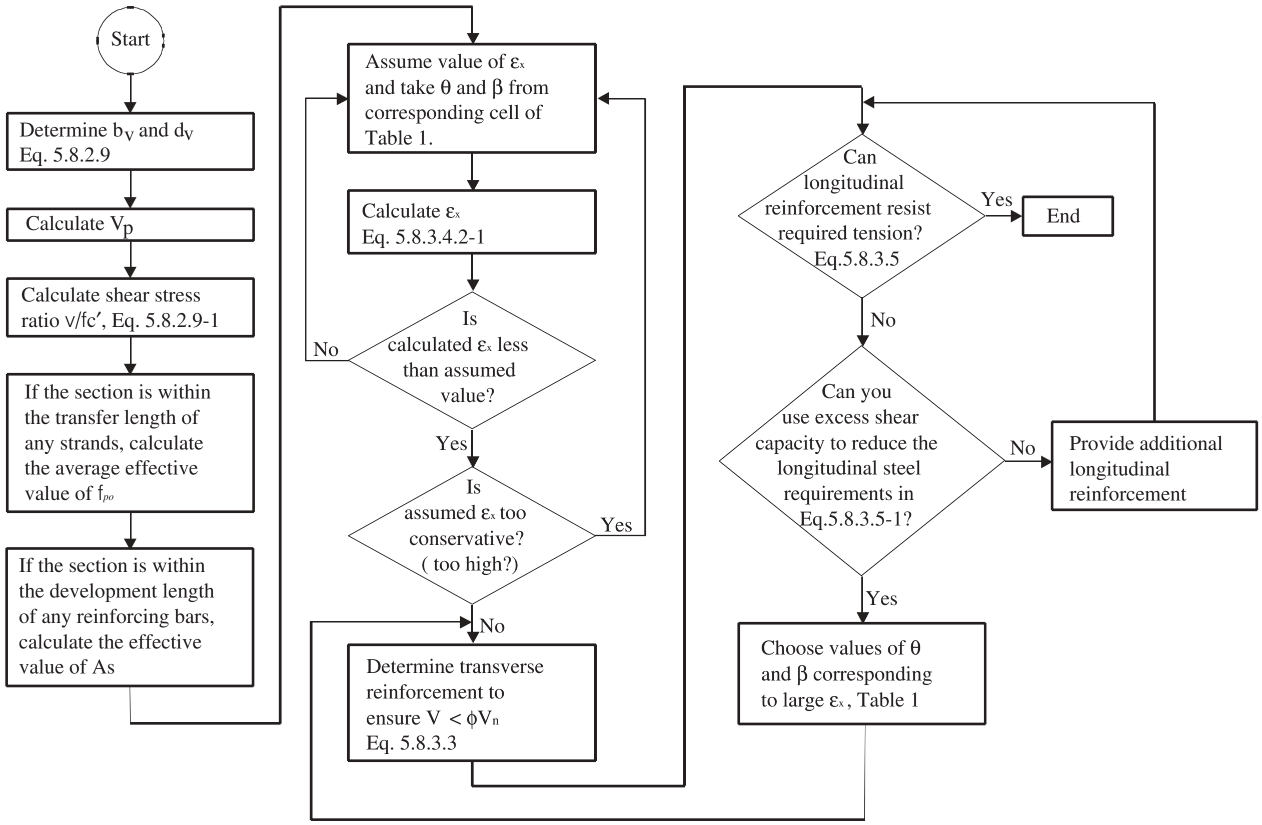


Figure 4. Flowchart for LRFD design procedure.

4. Determine shear reinforcement

$$V_c = 0.0316\,\beta\sqrt{f_c'}b_v d_v = 103.9 \text{ kips},$$

$$V_s = \frac{V_u}{\phi} - V_c - V_p = 224.0 \text{ kips}$$

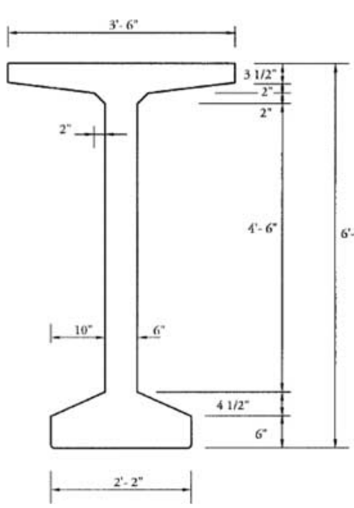


Figure 5. Design example implementing the LRFD sectional design model.

From $V_s = \dfrac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$, and set $f_y = 60.0$ ksi, then

$$\frac{A_v}{s} = \frac{V_s}{f_y d_v \cot\theta} = 0.021 \text{ in}^2/\text{in}$$

Use #4 bar double legs @12 in., $\dfrac{A_v}{s} = 0.033 \text{ in}^2/$ in $> 0.021 \text{ in}^2/\text{in}$

This provides $V_s = \dfrac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} = 344.6$ kips

5. Compute maximum limit check: $V_c + V_s \le 0.25 f_c' b_v d_v$

   $V_c + V_s = 448.5 \text{ kips} \le 0.25 f_c' b_v d_v = 713.1$ kips, O.K.

6. Compute longitudinal reinforcement check at the end of beam

$$A_s f_y + A_{ps} f_{ps} \ge \frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

$$\frac{M_u}{d_v\phi} + 0.5\frac{N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta = 456.4 \text{ kips}$$

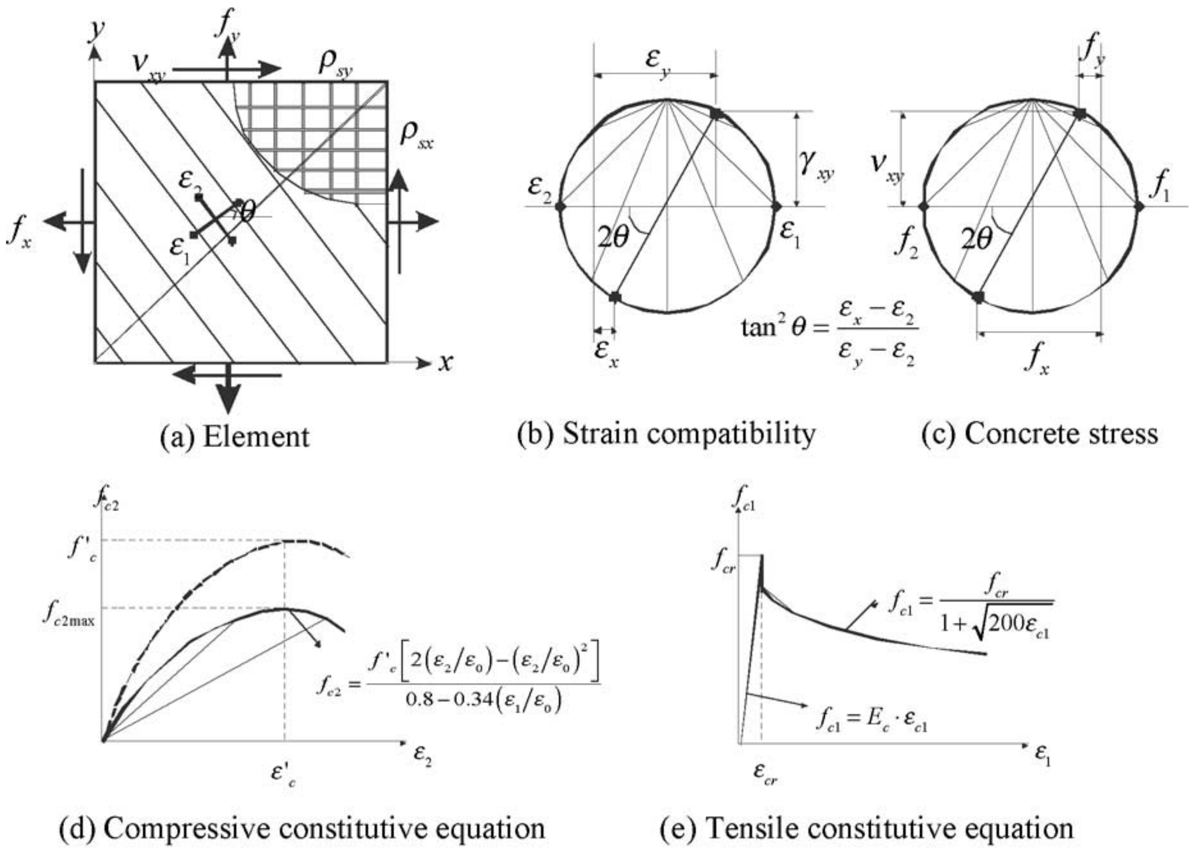
$$A_s f_y + A_{ps} f_{ps} = 460.1 \text{ kips} \ge 456.4 \text{ kips, O.K.}$$

1.1.2 Basis of the LRFD Sectional Design Model

The LRFD Sectional Design Model is derived from the MCFT, a behavioral model that can be used to predict the shear-stress versus shear-strain response of an element subjected to in-plane shear and membrane forces. The theory consists of constitutive, compatibility, and equilibrium relationships that enable determination of the state of stress (f_x, f_y, v_{xy}) in structural concrete corresponding to a specific state of strain (ϵ_x, ϵ_y, ϵ_{xy}) as shown in Figure 6.

The full implementation of the MCFT is possible in a two-dimensional continuum analysis tool, such as that done in program VecTor2 (*18*). The MCFT is also implemented in Response 2000, a multilayer sectional analysis tool that can predict the response of a section to the simultaneously occurring actions of axial load, prestressing, moment, and shear. In Response 2000, the plane section assumption is used which constrains the distribution of shear stress over the depth of the section. For each layer, an equivalent dual section analysis is performed that uses the MCFT to solve for the angle of diagonal compression, longitudinal stress, and shear stress in each layer (*19*). In a typical analysis, the cross section will be divided into more than 100 layers. The LRFD Section Design Model is also derived from the MCFT, but developing this hand-based general shear design method (*20*) required several additional simplifications and assumptions to be made. The most significant of these was that the distribution of shear stress over the depth of the section was taken as the value at mid-depth as calculated by the MCFT using the designer-calculated longitudinal strain, ϵ_x, at mid-depth.

Additional assumptions that were made in the development of the LRFD Sectional Design Model were that the shape of the compressive stress-strain response of the concrete was parabolic with a strain at peak stress of –0.002, and, for members with $A_v \geq A_{v,min}$, that the spacing of the cracks was 12 inches and the size of the maximum aggregate was 0.75 inches.



$$f_x = f_{cx} + \rho_{sx} \cdot f_{sx}, \quad f_y = f_{cy} + \rho_{sy} \cdot f_{sy}, \quad v_{xy} = v_{cxy\ y}$$

$$f_{cx} = f_{c1} - v_{xy} / \tan\theta, \quad f_{cy} = f_{c1} - v_{xy} \cdot \tan\theta, \quad f_{c2} = f_{c1} - v_{xy} \cdot \left(\tan\theta + 1/\tan\theta\right)$$

(f) Equilibrium Relationships

Figure 6. MCFT for predicting shear response of an element.

Although the LRFD specifications were derived from the MCFT, because of the significant simplification and assumptions used in developing this method, the shear capacity determined using the LRFD Sectional Design Model should not be considered equivalent to the shear capacity calculated by the MCFT.

1.1.3 Comparison of AASHTO LRFD and AASHTO Standard Specifications

The LRFD Sectional Design Model provides a complete shear design approach for structural concrete in which the actions of axial loading, moment, and prestressing are considered explicitly. This approach is a significant departure from the shear design procedures of the AASHTO Standard Specifications and ACI318-02.

The key differences between the AASHTO LRFD and Standard Specifications are as follows:

- *LRFD Eliminates Approach of Evaluating V_c Based on the Diagonal Cracking Load*

In the AASHTO Standard Specifications, the concrete contribution to shear resistance, V_c, is taken as the load at which diagonal cracking is expected to occur. In this approach, V_c is taken as the lesser of the force required to cause web-shear cracking, V_{cw}, or flexure-shear cracking, $V_{ci.}$

In the LRFD approach, V_c is taken as a measure of the concrete contribution at ultimate. A significant effect of this difference is that with LRFD the state of shear cracking in a member cannot be used to estimate the force that it has supported nor can the designer evaluate whether or not the member is likely to be cracked in shear under service loads.

- *LRFD Introduces Use of a Variable Angle Truss Model*

In the Standard Specifications, the contribution of the shear reinforcement to capacity is determined using a 45-degree parallel chord truss model. In this way, the number of stirrups considered to lift the diagonal compression across a single shear crack is taken as *d/s* where *d* is the depth of the member and *s* is the spacing of the shear reinforcement.

In the LRFD Sectional Design Model, the angle of diagonal compression can be taken as ranging from 18.1 to 43.9 degrees and where the number of stirrups considered to lift the diagonal compression force is taken as $d_v \cot\theta/s$. Because cot(18.1 degrees) is 3.06, a given number of stirrups can be calculated by the LRFD Specifications to provide about three times as much shear capacity as would be calculated by the Standard Specifications.

- *Evaluation of Shear Depth*

In the LRFD Specifications, the shear depth is taken as $d_{v,}$ rather than *d,* to overcome a previous simplification in the Standard Specifications. In accordance with the parallel chord truss model, the shear depth is equal to the distance from the centroid of the longitudinal tension reinforcement to the centroid of the compression block (i.e., the flexural level arm). In developing the Standard Specifications, *d* was used rather than the flexural lever arm for the sake of simplicity and also because the provisions still proved to be conservative with the use of *d*. In the LRFD specifications, d_v is used as the flexural lever arm and is typically taken as 0.9*d*.

- *LRFD Raises Minimum Shear Reinforcement Requirement*

The LRFD shear design provisions require a substantially larger amount of minimum shear reinforcement (typically 50 percent more), than do the Standard Specifications, as shown in Figure 7. This difference is particularly important for prestressed concrete members for it is common that large portions of the length of prestressed concrete members require minimum shear reinforcement only.

- *LRFD Introduces Longitudinal Reinforcement Requirement Check*

In the Standard Specifications, anchorage rules for longitudinal reinforcement have been used to account for the demands that shear imposes on the longitudinal reinforcement requirements. In the LRFD Specifications, the demand that shear imposes on longitudinal reinforcement requirements is taken into account directly. The difference between these approaches is particularly significant at the ends of simply supported prestressed members where the horizontal component of the diagonal compression force can be large and yet, by the LRFD Specifications, only the developed portion of the strands may be considered to provide the required resistance (see Figure 8).

- *LRFD Enables Design for Much Higher Shears*

One of the greatest differences with the LRFD Specifications is that it enables members to be designed for shear stresses that can exceed 2.5 times those permitted by the Standard Specifications. In the Standard Specifications, the contribution of the shear reinforcement is limited to $8\sqrt{f_c'}b_w d$ so as to guard against

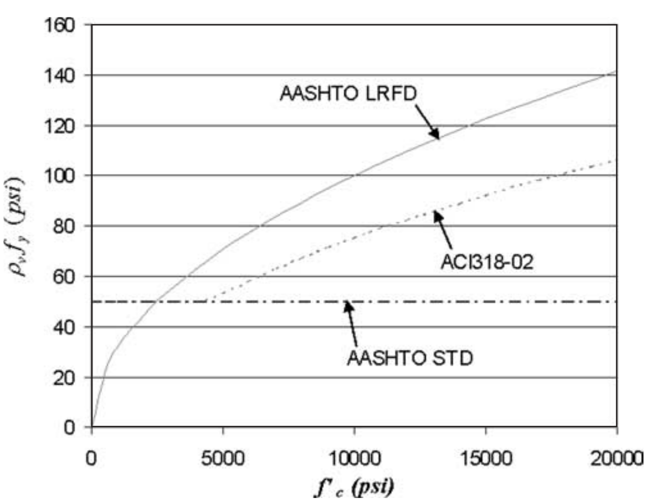
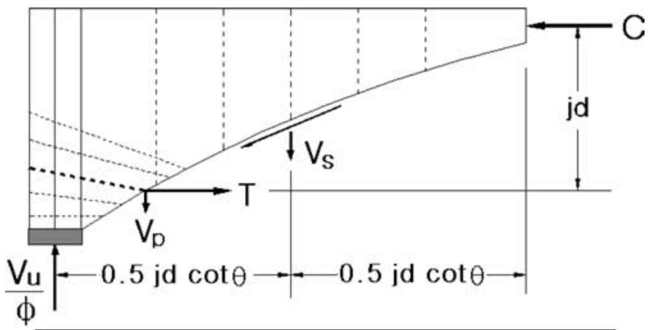


Figure 7. Minimum required amount of shear reinforcement.



$$A_s f_s + A_{ps} f_{ps} = \frac{M_u}{\phi jd} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

Figure 8. Shear demands on longitudinal reinforcement at end of prestressed girder.

the member being overly reinforced in shear and failing by diagonal crushing of the concrete or another means before yielding of the shear reinforcement. According to the MCFT, and based on the results of shear tests on elements (*21, 22*), such failure mechanisms do not occur until design shear stresses are in excess of 0.25 f'_c. The difference between these limits is shown in Figure 9.

- *LRFD Requires an Iterative Shear Design Procedure*

The LRFD shear design procedure requires the evaluation of the longitudinal strain at mid-depth, ϵ_x, in order to obtain values for β and θ from Table 1 and Table 2. Because ϵ_x is a function of θ (see Equations 1-6 and 1-7), the design procedure is iterative. The angle θ is first assumed and then ϵ_x is evaluated for the given value of θ. The value of θ is obtained from Table 1 or Table 2, and then ϵ_x is checked to confirm that is not significantly changed by using the new value of θ. If it is, then it may be necessary for a different column to be used for obtaining β and θ.

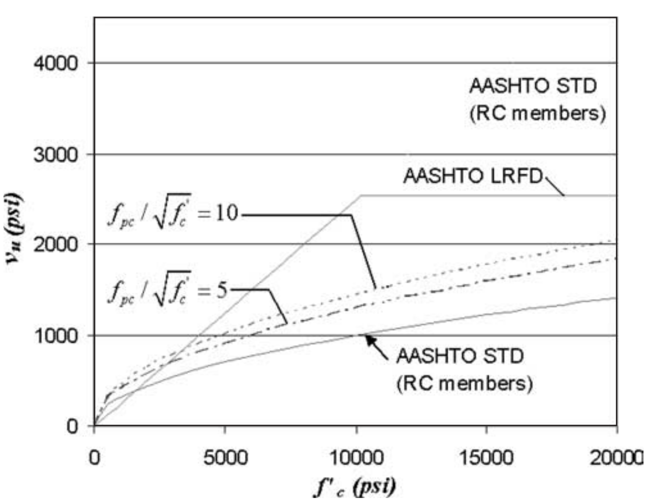


Figure 9. Maximum allowable design shear stress.

- *Further Iteration Required for Capacity Evaluation*

In the LRFD Sectional Design Model, ϵ_x and thus β and θ are functions of V_u. Thus, the shear design force must be known in order to evaluate V_c, V_s, and the nominal shear strength. As a result, the procedure for evaluating capacity is iterative and requires the engineer to guess the capacity, evaluate model parameters and V_n, and then check that the calculated capacity is close to the factored load.

- *Empirical versus Model-Based Justification*

The Standard Specifications justify the relationship for V_c by experimental test data (*23*) which indicates that the measured shear capacity of prestressed and non-prestressed test beams is conservatively predicted by the sum of V_c (lesser of V_{ci} and V_{cw}) and the contribution of the shear reinforcement, V_s, as calculated using a 45-degree parallel chord truss model.

The LRFD Sectional Design Model shear provisions are derived from a comprehensive behavioral model (the MCFT); therefore, the basis of this model is the MCFT. The calculated capacities by the LRFD Sectional Design Model were illustrated by experimental test data (*24*) to provide conservative estimates of shear capacity.

- *Difference in Shear Reinforcement Requirements and Capacity Ratings*

The LRFD shear design requirements different considerably from those of the Standard Specifications. This leads to significant differences in required amounts of shear reinforcement and rated capacities of existing structures. Because the structure of the design provisions is so different, it cannot be readily said when one set of provisions will be more conservative than the other. Further, with use of the Standard Specifications it is easy to perform independent checking of designs. However, the opposite is true with use of the LRFD Specifications.

1.2 INTRODUCTION TO SHEAR BEHAVIOR AND DESIGN PRACTICES

This section summarizes the resources considered and used to develop the proposed simplified provisions. This subsection presents the development of U.S. code provisions and compression field approaches for shear design and discusses the factors that influence the primary mechanisms of shear resistance; lists other code provisions warranting consideration; and presents an overview of available experimental test data, analysis tools, and design data.

1.2.1 Development of Traditional U.S. Code Provisions for Shear

The basic model for how shear is carried in structural concrete is the parallel chord truss model that was first proposed by Ritter in 1899 (*25*). In this model, the load is carried in reinforced concrete in the same manner as load flows in a truss with the load zigzagging its way to the support. The load flows down the concrete diagonal struts and then is lifted to the compression chord by transverse tension ties on its way to the support. Equilibrating the flow of forces puts tension in the bottom chord and compression in the top chord of the truss. Although the model is traditionally shown as one truss with stirrups at a longitudinal spacing of "*d*," such as given in Figure 10a, it was correctly understood by Ritter that there was a continuous band of diagonal compression carried up and over cracks by a band of stirrups, Figure 10b. For a 45-degree truss, the capacity provided by the shear reinforcement is equal to the capacity of an individual stirrup multiplied by the number of stirrups over the length, "*d*" which is approximately equal to "*d/s*." See Equation 10.

$$V_s = \frac{A_v f_y d}{s} \quad \text{(Eq. 10)}$$

When the 45-degree parallel chord truss model was introduced in the United States in the early 1900s, researchers at the University of Illinois (*26*) and the University of Wisconsin (*27, 28*) observed through experimental research that the shear capacity of beams was greater than that predicted by this truss model by nearly a constant amount (see Figure 11). Thus, the idea of a concrete contribution to shear resistance was introduced. This contribution was originally taken as equal to a shear stress of between 2 and 3 percent of f'_c multiplied by the shear area ($b \times d$). However, over time that contribution became linked to the diagonal cracking strength because this provided a better fit with test data. The most commonly used relationship in U.S. design practice for the diagonal cracking load, and thus the concrete contribution to shear resistance in reinforced concrete members, is given by Equation 11:

$$V_c = 2\sqrt{f'_c}\,b_v d \text{ where } f'_c \text{ is in psi units} \quad \text{(Eq. 11)}$$

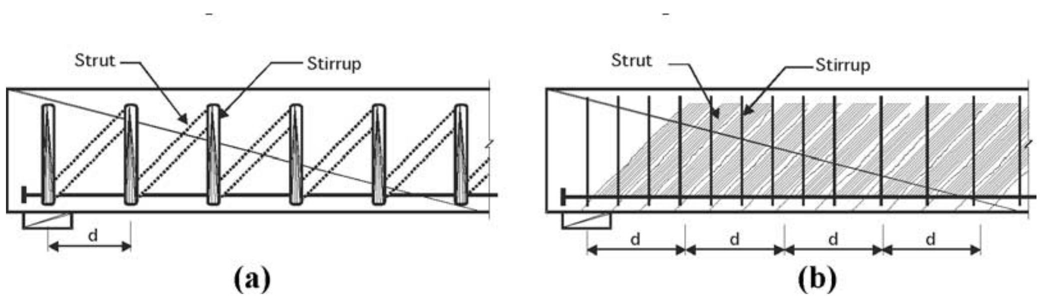


Figure 10. Parallel chord truss model.

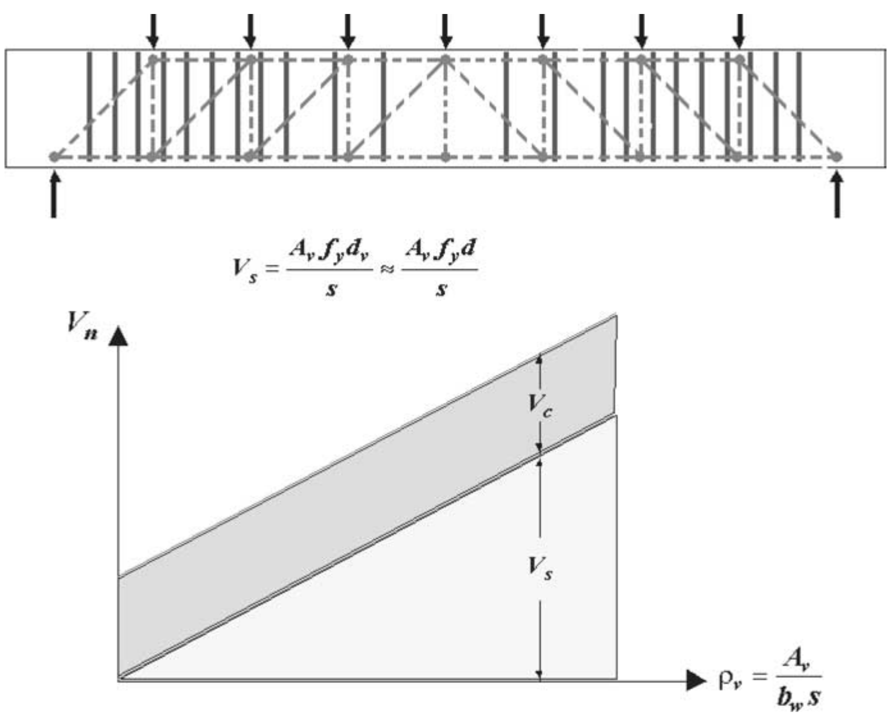


Figure 11. Shear strength of RC beams with shear reinforcement.

There is no mechanical reason to suggest that the concrete contribution to shear resistance at ultimate is equal to the diagonal cracking load, but experimental test data supported the argument that the sum of the diagonal cracking strength plus a shear reinforcement contribution calculated using a 45-degree truss provided a reasonably conservative estimate of shear capacity. Over time, additional expressions for the diagonal cracking strength were developed to account for the influence of prestressing, flexure, and other axial loads. However, as noted in University of Illinois Bulletin No. 493 (*29*), where the data that forms the basis for the prestressed concrete shear design concepts of the Standard Specifications and ACI 318-05 (*30*) are reported, the equating of the concrete contribution at ultimate to the shear at inclined cracking is a convenience justified by the simplicity of the result and not by a rational theoretical model.

1.2.2 Compression Field Approaches for Modeling Shear Behavior

When the parallel chord truss model was developed, Mörsch (*31, 32*) argued in 1920 and 1922 that it was not possible to calculate the angle of diagonal compression for there were four unknowns and only three equations (see Figure 12). This dilemma was overcome by Mitchell and Collins in the Compression Field Theory (*33*) through the introduction of a compatibility relationship made possible by the assumption that the direction of principal compressive stress was equal to the direction of principal compressive strain. In addition, within the compression field theory, the concept of compression softening was introduced. The principal tensile strain, ϵ_1, is considered to decrease the stiffness and strength of concrete in compression. In the MCFT, the average tensile stress in the concrete after cracking was considered. The MCFT can predict the complete response of an element subjected to shear and member forces as described in Figure 6 and more fully explained in Appendix A (Appendix A is available on line as part of *NCHRP Web-Only Document 78*).

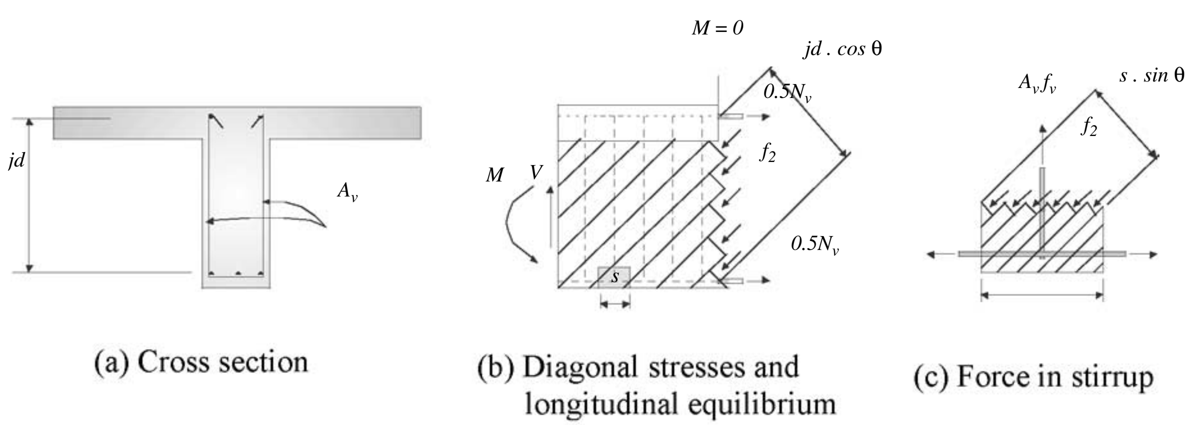
Since the development of the MCFT, three other compression field behavioral models developed worth noting have been developed: the variable-angle softened truss model introduced by Belarbi and Hsu (*34–37*), the fixed-angle softened truss model by Pang and Hsu (*38*), and the disturbed stress field model by Vecchio (*39*).

1.2.3 Other Approaches and Design Provisions

The MCFT provides a clear model for the flow of forces in both prestressed and non-prestressed (reinforced) concrete members and for calculating the angle of diagonal compression and the concrete contribution based on the average tensile stress in the concrete. However other ways of looking at shear resistance remain.

Another approach for evaluating the angle of diagonal compression is based on plasticity theory and an assumption that the diagonal compressive stress is limited to a fraction of the uniaxial compressive strength; $0.6f'_c$ is common. This model is used in some European design approaches.

Methods for calculating the concrete contribution to shear resistance are far more varied because the concrete contribution at ultimate is really the sum of several mechanisms of resistance as described in Figure 13. These mechanisms are shear in the uncracked compression zone, aggregate interlock or interface shear transfer across cracks, dowel action, and residual tensile stresses normal to cracks. In prestressed concrete members, such as bulb-tee girders, the bottom bulb



Equilibrium equations: $f_2 = \dfrac{V}{b_w jd}\left(\tan\theta + \cot\theta\right)$, $\quad N_v = V\cot\theta$, $\quad \dfrac{A_v f_v}{s} = \dfrac{V}{jd}\tan\theta$

Unknowns: f_2, N_v, f_v, θ

Figure 12. Free body diagrams for development of shear relationship.

may also provide significant shear capacity. Additional components are the vertical component of the force of draped prestressing strands and the shear transmitted directly to the support by arch action. The relative magnitude of each of these components to the total resistance depends on many factors but it is generally agreed that the dominant concrete components to shear resistance in beams with transverse reinforcement are shear in uncracked compression zones and interface shear transfer.

Although researchers agree on the foregoing mechanisms of shear resistance, the structure of code provisions and the amount of shear reinforcement required by different codes for the same design situation vary because of the complexity of shear resistance mechanisms, the factors that influence these mechanisms, and the different methods used to evaluate the contributions of the shear reinforcement.

The discussion presents some of the complexities of developing a model for shear resistance and to show how different codes have chosen dramatically different approaches. Those approaches have then lead to the development of different infrastructures for design equations and different ways of thinking about shear. For this development of proposed AASHTO simplified shear design provisions, primary resources were underlying models for shear resistance and behavior, shear design equations in current national codes of practices, and expressions for calculating shear capacity that are promoted by individual researchers.

1.2.4 Factors Influencing Shear Resistance

Different factors can have surprising effects on shear resistance. Shear is complex, there are potential safety concerns with traditional approaches, and developing simplified provisions may require making conservative assumptions.

Influence of Depth

A core assumption in the *ACI 318* and AASHTO Standard Specifications is that the shear capacity is proportional to the depth of the member. This assumption was investigated in a landmark study conducted by Shioya et al. (*40*) in which they tested reinforced concrete members that ranged in depth from 4 to 118 inches. All members were simply supported, did not contain shear reinforcement, were lightly reinforced in flexure (0.4%), and subjected to a uniformly distributed load. In Figure 14, the normalized shear stress at failure is plotted versus the depth of the member. The horizontal line corresponds to the shear strength calculated using the traditional shear design expression of the ACI and AASHTO Standard Specifications. The results show that the shear stress at failure decreases as the depth of the member increases. Of particular concern is that members greater than 36 inches deep failed under stresses approximately one-half of the strength calculated by these codes of practice. However, although this depth effect is marked for beams without transverse reinforcement, available test data show little if any depth effect for beams with transverse reinforcement (*41*).

Influence of Concrete Strength

In traditional U.S. design practice, and in the LRFD Sectional Design Model, the contribution of the concrete to

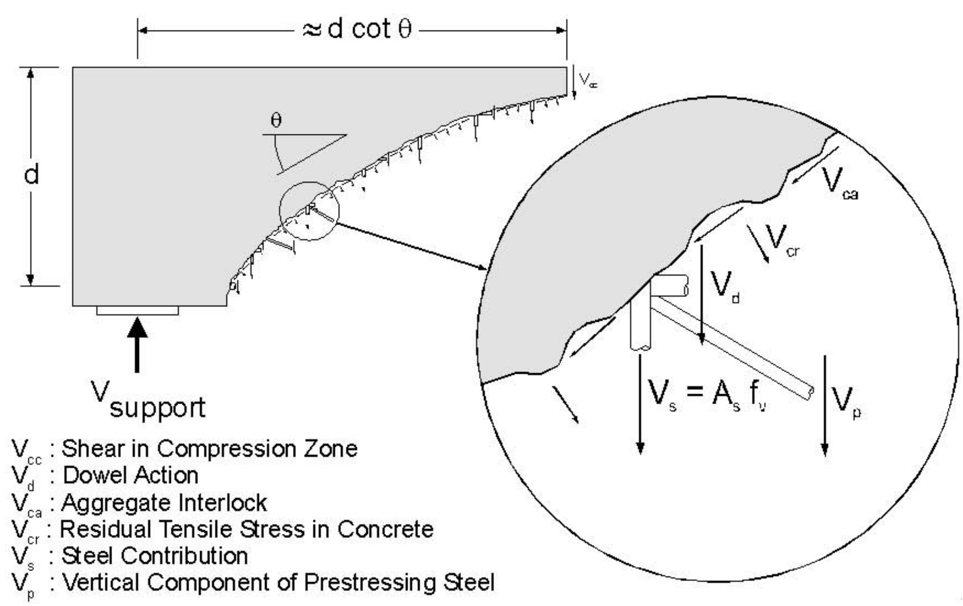


Figure 13. Mechanism of shear resistance.

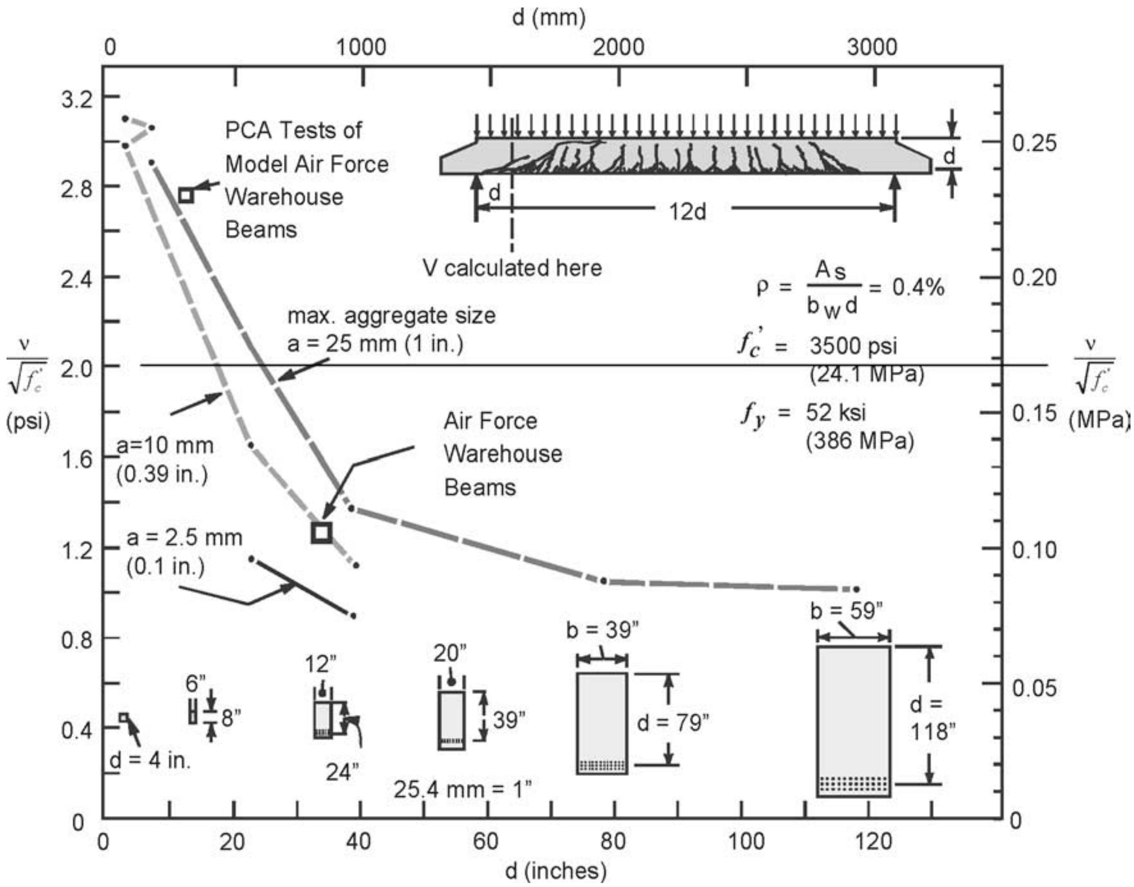


Figure 14. Influence of depth on shear capacity.

shear resistance is taken as proportional to the square root of the cylinder compressive strength f_c'. Figure 15 presents some of the test data by Moody et al. in 1954 (*42*) from which the permissible design stress limit of $2\sqrt{f_c'}$ was developed. The test beams were typically around 14 inches deep, overly reinforced in flexure, and contained large aggregates. Also shown in this plot are the results from a series of tests by Angelakos in 2001 (*43*) conducted at the University of Toronto on larger and more lightly reinforced members cast using smaller size aggregates. As the results in Figures 14 and 15 show, the apparent safety of the traditional equation for $2\sqrt{f_c'}$ as used in U.S. practice for beams without shear reinforcement is also dependent on the parameters of beam depth, concrete strength and maximum aggregate size, not considered in that expression.

Influence of Axial Loads

The influence of axial compression and tension on shear capacity is examined in Figures 16 (*44*) and 17 (*45*). As shown, traditional U.S. design practice expressions can be both conservative and unconservative. Part of the explanation for these shortcomings is the assumption that the angle of diagonal compression is at 45 degrees whereas, as these figures illustrate, axial compression increases the number of stirrups that carry the shear across diagonal cracks while axial tension decreases the number of stirrups that are available to carry the shear across cracks.

1.2.5 Experimental Test Data

The previous examples illustrate the importance of evaluating and calibrating any potential simplified provisions with extensive experimental data. Professors Reineck and Kuchma (*46*), and their research assistants have assembled what is probably the largest available database of results from shear tests on structural concrete members. The database contains more than 2000 test results. This database can be mined to assess the accuracy and limitation of all prospective code approaches.

1.2.6 Analysis Tools

In addition to experimental test data, analytical tools can be used to predict the capacity of prestressed and nonprestressed concrete members. These tools are particularly

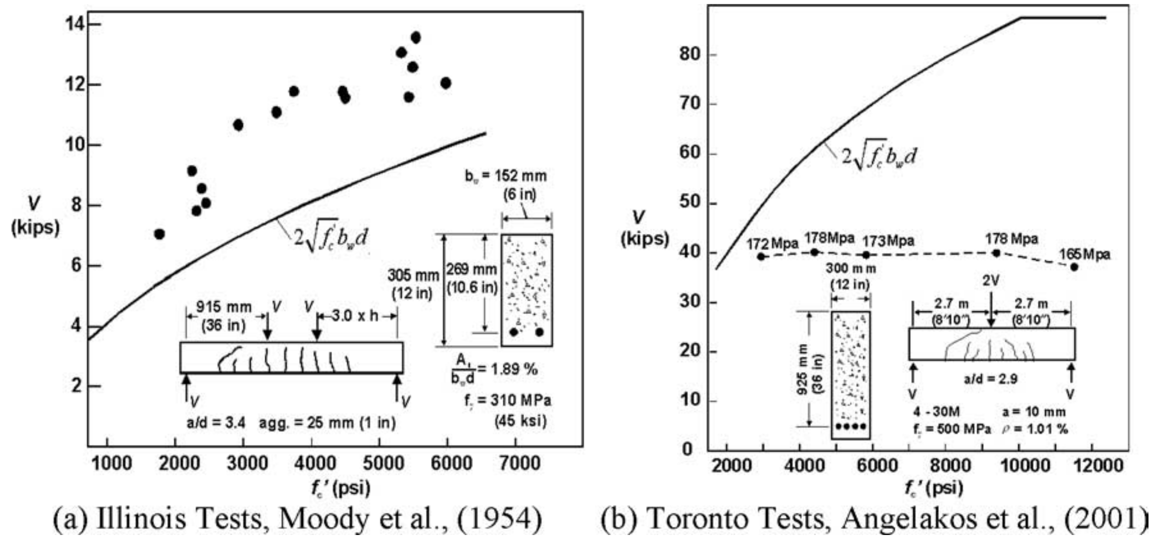


Figure 15. Influence of concrete strength on shear capacity.

useful for predicting the capacity for the types of members for which no experimental test data is available. Before the use of any analytical tool, the accuracy and reliability of the tool must first be assessed by making comparisons with existing experimental test data. A further consideration is the effort required to use these tools to obtain an evaluation of the shear capacity. Some of the most promising available tools are Response 2000 (*15*), ABAQUS (*47*), VecTor2, DIANA (*48*), and ATENA (*49*).

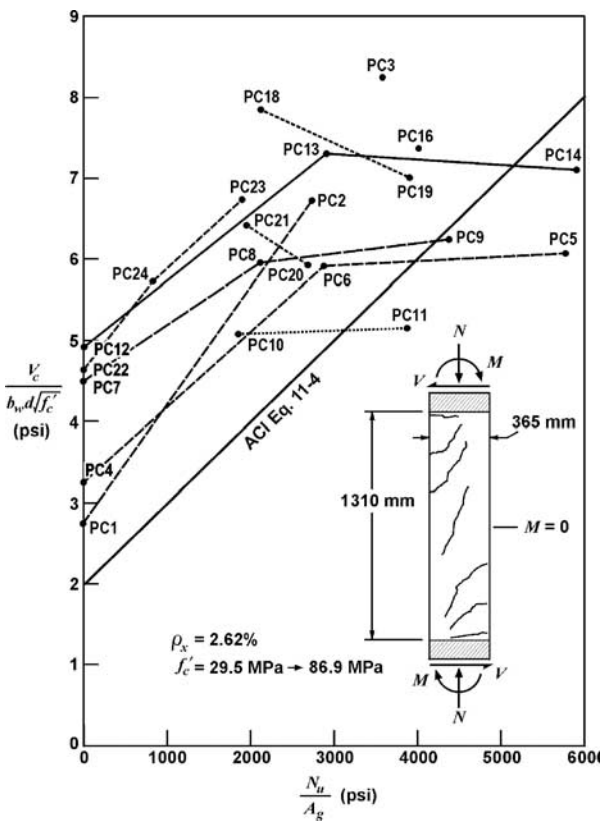


Figure 16. Influence of axial compression on shear capacity.

1.2.7 Design Cases

A further way to evaluate design methods is to compare the required strengths of shear reinforcement ($p_v f_y \equiv A_v f_y / b_v s$) by the different design methods for a large database of design cases. Ideally, these cases would represent the range and frequency of members built using the given design provisions. Comparing the required amount of shear reinforcement by different design approaches for each design case can reveal where prospective provisions may be unconservative or overly conservative. It is also useful to compare these required strengths of shear reinforcement ($p_v f_y$) with the strength determined using analysis tools such as Response 2000.

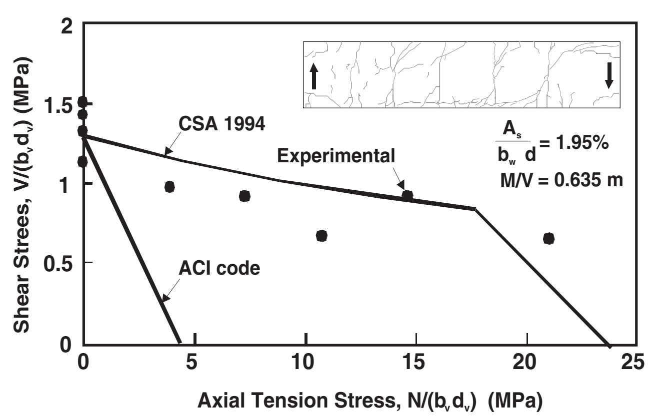


Figure 17. Influence of axial tension on shear capacity.

1.3 PROJECT OBJECTIVES AND TASKS

1.3.1 Project Motivation and Objectives

The LRFD shear design provisions provide some definite advantages over the methods in the AASHTO Standard Specifications. The Sectional Design Model provides a comprehensive approach for shear design of sections subjected to the actions of axial load, prestressing, and moment while the strut-and-tie method provides a completely general design method for regions in which the flow of forces is more complex, such as near geometric discontinuities or near concentrated forces and reactions. However, the Sectional Design Model requires an iterative design procedure that involves selecting β and θ values from tables. Some designers consider this procedure complex to use and difficult to understand, with the effect that some design engineers lose a feel for what they are evaluating. With the strut-and-tie method, concerns have been expressed that solutions require an iterative approach and are non-unique.

The overall objective of this research was to provide simplified procedures for the shear design of the most common concrete structures, including reinforced concrete T-beams; prestressed concrete I girders continuous for live load; prestressed concrete box beams; and cast-in-place post-tensioned box girders, hammerhead piers, and concrete bents. These simplified provisions were expected to be in a form similar to the standard specifications and to be applicable for both prestressed and precast members up to concrete strengths of 18 ksi and cast-in-place concrete strengths up to 10 ksi. Although there are recognized challenges to the application of the strut-and-tie method, there was no project objective to refine the strut-and-tie design provisions.

1.3.2 Research Approach and Project Tasks

There are many approaches for shear design, underlying theories for explaining how shear is carried in structural concrete, and the primary factors that influence the mechanisms of resistance. The approach on this project was to investigate and then select the most suitable simplified shear design provisions based on a detailed review of existing shear design approaches and an evaluation of these approaches by comparison with both experimental test data and with the predictions from numerical methods. The members of the research team were selected so that different points of view and experiences were represented. Several of the important, if not essential, attributes of the research team were as follows:

- Leadership experience in the developing code provisions for shear, including the AASHTO LRFD specifications, the AASHTO Standard Specifications and the ACI 318-05 provisions;
- Detailed knowledge of a broad base of mechanistic models for shear including U.S., Canadian, and European approaches;
- Detailed understanding of the Modified Compression Field Theory and how the LRFD provisions were derived from this theoretical model for behavior;
- Custodians of the largest and more detailed shear test database yet assembled;
- Not committed to a single line of thinking on the final structure of the simplified provisions or on the mechanistic model (or models) on which these provisions should be based; and
- Familiarity with the use of non-linear numerical tools for predicting the capacity of members in a design testbed.

There were 8 tasks listed in the request for proposals that were required for meeting project objectives. The researcher's approach on each of these tasks follows the description of each of these individual tasks.

The researchers conducted a survey of practicing engineers concerning their experiences in the use of AASHTO Standard and LRFD specifications, collected codes of practice, and then conducted a preliminary review and assessment of different shear design approaches using an extensive experimental database of shear test results.

A refined work plan was established for developing and refining the selected proposed simplified provisions. This plan included the use of a design database to assess the effect of different potential approaches on the efficiency and conservatism of codes. The researchers produced a tentative list of design examples for consideration by the project review panel from which the final design examples were selected.

An Interim Report was submitted and then, following a request by the Project Panel, a more comprehensive interim report was submitted containing an initial proposal by the contractor for the simplified provisions. These proposed simplified provisions were essentially the same as those developed by Michael Collins, a developer of the MCFT, for the 2004 Canadian Standards Association "Design of Concrete Structures." These simplified provisions consisted of equations for β and θ and an elimination of the dependency of ϵ_x on θ, thereby eliminating the iterative nature of the LRFD design procedure.

The researchers conducted the plan approved by the project panel. This plan consisted of:

- Reviewing shear design provisions additional to those examined in the Interim Report;
- Developing a refined experimental database of shear test results of large members with shear reinforcement;
- Developing an expanded member design database;
- Developing alternative provisions to the CSA method proposed in the Interim Report; and
- Developing detailed criteria for selection and verification of the simplified specifications.

Based on the results of their analytical and design investigations, the researchers (1) developed a new simplified shear design procedure for members with minimum shear reinforcement, (2) verified the need for the existing limit on the required minimum amount of shear reinforcement, (3) verified the need for a new lower limit on the maximum shear stress that can be used in design if members are not supported over their full depth at the ends, and (4) developed modifications to simplify the existing General Procedure for sectional shear design of Article 5.8.3.4.2 of the LRFD Specifications.

Based on the final form of the proposed simplified specifications, the goal of the regression testing was the setting of only a few parameters and limits. The tuning of these parameters was performed by considering the fit of the proposed simplified provisions with the test results in the refined experimental database and by comparing the required amounts of shear reinforcement for members in the design database with the requirements by other design methods, including the current LRFD Sectional Design Model, the AASHTO Standard Specifications, and Response 2000.

The research team prepared eight design examples that covered both prestressed and non-prestressed members, simple span and continuous members, different types of structural components and both stocky and slender members.

CHAPTER **2**

FINDINGS

In accordance with the research approach, a review and evaluation was conducted of existing models and approaches for shear design. This study revealed that there are dramatically different methods and bases for shear design provisions. In Section 2.2, a comparison of relationships used in codes and suggested by researchers is made. This led to the identification of positive attributes of different shear design methodologies. Section 2.3 presents an evaluation of the accuracy of prominent shear design provisions. Section 2.4 presents the results of a survey conducted to evaluate the experience of practitioners in using the LRFD Sectional Design Model and the AASHTO shear design provisions. Using the findings from Sections 2.1 through 2.4, criteria were developed for the simplified provisions. See Section 2.5. This led to the development of the proposed changes to the LRFD Sectional Design Model and the Proposed Simplified Provisions presented in Chapter 3.

Chapter 2 summarizes the findings. More comprehensive results are presented in the following appendixes:

- Appendix A: Models for Shear Behavior
- Appendix B: Shear Design Provisions
- Appendix C: Shear Database
- Appendix D: Evaluation of Shear Design Provisions
- Appendix E: Field Performance Data and Practitioner Experience
- Appendix F: Recommended Revisions to Shear Provisions of AASHTO LRFD Concrete Provisions
- Appendix G: Evaluation of the Proposed Simplified Provisions with Selected Shear Database
- Appendix H: Examination of Proposals Using Design Database
- Appendix I: Utilization of NCHRP Process 12-50
- Appendix J: Examples of Shear Design

These appendixes are available in *NCHRP Web-Only Document 78*.

2.1 DIFFERENCES IN UNDERLYING BASES OF CODE PROVISIONS

As discussed in Chapter 1, the 100-year-old parallel chord truss model is the predominant model for describing the flow of shear forces in a reinforced or prestressed concrete beam. There is also general agreement in the research community that the concrete contribution to shear resistance results principally from a combination of interface shear transfer across cracks in the body of the beam and shear in the compression zone. However, because of the many different ways used to calculate the angle of diagonal compression and the many factors influencing interface shear transfer and shear transfer in the compression zone, the existing forms of shear design provisions differ greatly.

For example, in determining the angle of diagonal compression it is traditional U.S. design practice to assume a 45-degree angle because this approach has been considered to always lead to conservative designs. By contrast, in European practice the angle of diagonal compression is taken as low as 18 degrees while in the LRFD Sectional Design Model this angle is determined by considering the calculated longitudinal strain at mid-depth of the member. These different approaches for determining the contribution of the shear reinforcement then lead to different approaches in calculating the concrete contribution to shear resistance because $V_c = V_{test} - V_s$.

Before presenting and discussing the different shear design relationships in codes of practice, it is useful to further classify shear design approaches by the information on which they are based: empirical test data, an equilibrium model for the condition of a beam in its ultimate limit state, a comprehensive behavioral model for shear resistance, or some combination of the above. Relying on each of these three types of information has its advantages and limitations as discussed below.

2.1.1 Type 1: Empirical Relationships Designed to Fit Test Data

Empirical provisions are those based primarily on experimental test data. Because of the complexity of how shear is carried in structural concrete and the lack of a universally accepted model for shear behavior, this approach has many clear advantages. No consensus is needed from any committee and no selected model for behavior will bias the resulting provisions from accounting for the complexity of shear behavior.

The primary problem with this empirical approach is the deficiencies in the experimental test data that are available and therefore used in developing the resulting empirical approaches. As will be discussed in Section 2.3, there are large deficiencies in what has been tested experimentally; most experiments have been on small, rectangular, simply supported members that are over-designed in flexure, loaded by one or two point loads, and supported on bearings positioned underneath the member. In addition, most tests have been on members that do not contain shear reinforcement. By contrast, most members in practice are continuous and large, have top flanges, contain shear reinforcement, are acted on by distributed loads, and are built integrally into supports at their ends. Because what has been tested does not represent what is designed with provisions, there is no reason to believe that empirically derived provisions will provide a reasonable and conservative design procedure for members that fall outside the range of the experimental database used in developing the empirical provisions. This fact was illustrated in Section 1.2.4 where new types of tests illustrated that the effect of depth, concrete strength, and axial effects were not reasonably accounted for in traditional U.S. design practice.

A further complication is that only a limited selection of experimental test data has previously been available to code committees in developing or validating empirical design approaches. The database effort being led by Professors Reineck and Kuchma is attempting to overcome this problem by assembling most of the published test results. A remaining challenge is in selecting which test results to use in evaluating provisions because even within the narrower range of what has been tested there is a bias toward members of particular types. Furthermore, not all tests are equally reliable and those classified as shear tests may actually have included beams failing in flexure, because of anchorage failures, or tests deficient in their setups or members deficient in their detailing. Therefore, to use this database effectively for developing shear provisions, a means of selecting and weighting test data still needs to be developed.

An example of provisions that are effectively empirical is the AASHTO standard provisions for reinforced concrete members. These provisions are empirical because the angle of diagonal compression is assumed to be 45 degrees and because the concrete contribution is taken as the diagonal cracking strength which is not physically related to the concrete contribution at the ultimate limit state. It is only through validation with experimental test data that these provisions can be justified as effective. The AASHTO standard provisions are not based on a fully consistent mechanistic model of shear behavior

2.1.2 Type 2: Relationships Based on Specific Condition of Member in Its Ultimate Limit State

Design provisions may also be based on the condition of a member in its ultimate limit state. In this approach, there is one equilibrium diagram showing all of the forces that act on a given section. This is a very powerful approach because it enables the designer to consider the differing contributions of the various mechanisms of resistance to shear capacity and the factors that can influence these mechanisms of resistance.

There are two principal shortcomings with this approach. First, in developing this equilibrium diagram, many assumptions are made that cannot be fully substantiated. For example, it is typical that these approaches focus on only one of the multiple mechanisms of resistance (e.g., shear in compression zone, interface shear transfer, dowel action, arch action, and direct transmission of tensile stress across cracks) that exist. Second, these approaches then assume that mechanism is the dominant mechanism for all loading and material conditions. No single equilibrium diagram can capture accurately the critical condition for all types of members at any point along the design span and for any combination of loading.

A further complication is that the experimentally measured concrete contribution to shear resistance used to calibrate this type of model also requires an assumption for the angle of diagonal compression to be used in calculating the concrete contribution to shear resistance. Thus, the concrete contribution to shear strength V_c cannot be clearly established by this approach.

Although the angle of cracking may seem to be a clear indicator of the direction of diagonal compression, many researchers contend that substantial shear stress is transferred across these shear cracks with the effect that the true angle of diagonal compression is typically smaller than the angle of diagonal cracking. In NCHRP Project 12-56, shear tests on large bulb-tee girders were conducted from which the angle of diagonal compression was often somewhat larger than the angle of diagonal cracking near the end regions of members because of the introduction of the large anchorage force from the strands. A further complication is in counting how many stirrups cross the line of diagonal compression. Some researchers argue that cracks often do not cross stirrups and are likely to run from the top of one stirrup to the base of another. Thus, these researchers propose that the number of stirrups that should be considered to cross the plane of equilibrium in these models should be taken as $d_v \cot\theta / s - 1$.

To describe more accurately how shear is carried, some of these provisions provide two different relationships for V_c, one for members with shear reinforcement and one for members without shear reinforcement.

The truss model with crack friction is an example of a model based on an equilibrium diagram of a member in its ultimate limit state. Additional information on this method is available in Appendix A, which is included in *NCHRP Web-Only Document 78.*

2.1.3 Type 3: Relationships Derived from Comprehensive Behavioral Model

The strength of this approach is that it is based on a comprehensive behavioral model of the beam. This approach has the potential to capture the true complexity of shear behavior in which the angle of diagonal compression is calculated based on the calculated stiffness characteristics of the member, in which all mechanisms of resistance can contribute to carrying shear, and in which failure by breakdown of one or more mechanism of resistance can be considered.

There are three principal shortcomings of this approach. First, there are the shortcomings of the behavioral model itself. Second, the development of a hand-based design procedure from a comprehensive behavioral model requires many simplifications and can result in significantly reduced reliability of the model. Third, to fully understand the provisions requires an understanding of the underlying comprehensive behavioral model and that may be beyond the interests of most design engineers.

The LRFD Sectional Design Model is an example of shear provisions that have been implemented in codes of practice derived from a comprehensive model for behavior. This design procedure was described in Section 1.1. The potential shortcomings of the MCFT and the effect of assumptions made in deriving the LRFD Sectional Design Model on the effectiveness of these provisions are described below.

The MCFT is a smeared crack model for predicting the complete response of diagonally cracked concrete to in-plane shear and membrane stresses as shown in Figure 6. Because the effect of cracking is smeared, it does not attempt to model the development of individual discrete cracks. If the behavior of a member is dominated by the development of a single discrete crack, then an approach based on fracture mechanics (*50*) may be more appropriate. It is also a rotating angle crack model that assumes that the direction of cracking will rotate as the orthotropic stiffness characteristics of the element change over the loading history of the element. Research results suggest that this will only occur after very significant changes in relative stiffness characteristics; little to no crack rotation was observed in the girders tested as part of NCHRP Project 12-56. The evaluation of the angle of diagonal compression in the MCFT was made possible by the assumption that the angle of diagonal compressive stress coincided with the angle of diagonal compressive strain. This has also been experimentally observed to be an approximation and the Disturbed Stress Field Model by Vecchio in 2000 (*39*) was developed to account for the difference in these angles by considering slip deformations along crack interfaces.

Furthermore, the MCFT was derived from experiments on elements or panels in which there was a uniform distribution of stress across the width of the test specimens. By contrast, the LRFD Sectional Design Model is permitted by the LRFD specifications to be used for the design of end regions of members for which there is a very non-uniform distribution of stress and in the design of members that can have upper and lower flanges that are very stiff relative to the web and restrain the deformations of the web. These effects can lead to (1) unconservative results because of the additional stresses created by funneling the diagonal compressive stresses into the supports or (2) conservative results because of the restraint of the web deformations by the flanges.

Determining internal stresses in an element corresponding to a particular state of stress (v_{xy}, f_x, f_y) by the MCFT is a multistep and highly iterative process. By contrast, the completion of a shear design by the LRFD Sectional Design Model is a comparatively simple hand-based procedure. Developing this hand-based procedure from the MCFT required several assumptions. Predicting the full effect of these assumptions is beyond the scope of this project but a few simple observations follow:

1. In a multilayer sectional analysis, such as conducted using Response 2000, the longitudinal strain varies over the depth of the member. When the MCFT is then used to calculate the shear stress at each level, the distribution of shear stress over the depth of the member varies. By contrast, in the LRFD Sectional Design Model the shear stress is assumed to be constant over the depth of the member and only the calculated longitudinal strain at mid-depth, ε_x, is used in calculating its value. If this shear stress at mid-depth is similar to the average stress over the depth of the member, as would be predicted by a multilayer analysis, then the effect of this assumption is minimal. See Figure 18. If that is not the case, the effect can be significant.
2. In the derivation of the LRFD Sectional Design Model, the stress-strain relationship in concrete is

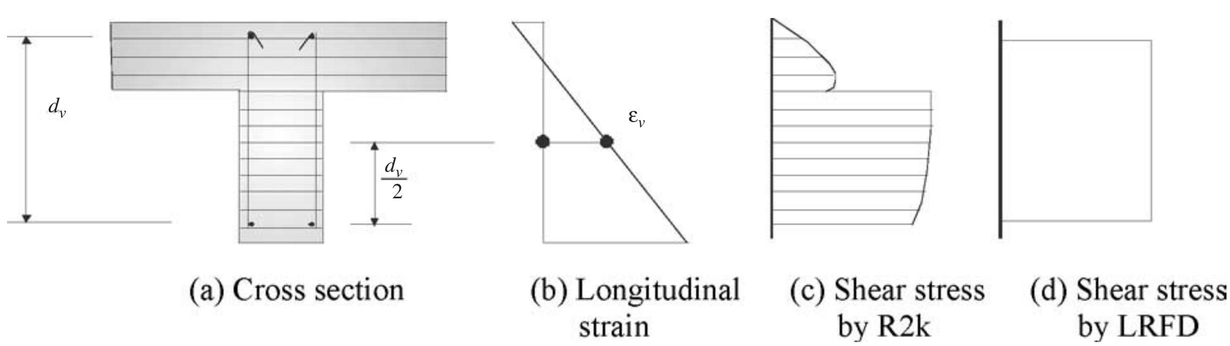


Figure 18. Shear stress distribution.

assumed to be parabolic with a strain at the peak stress of −0.002. This assumption is not consistent with the stress-strain behavior of high-strength concrete where the strain at peak stress can exceed −0.003 for an 18,000 psi concrete.

3. In the derivation of the LRFD Sectional Design Model for members with shear reinforcement, the average crack spacing was assumed to be 12 inches. This value was used in calculating the crack width (crack spacing × principal tensile strain ϵ_1) from which the resistance to crack slip was determined. This affected some of the values of β and θ in Table 1. Given that a conservative (larger than typical) value for crack spacing was assumed, this approach was considered to lead to a conservative estimate of shear capacity.

2.2 COMPARISON OF SHEAR DESIGN METHODS

The approach used in this research was to derive the simplified design provisions after a thorough review and evaluation of current code provisions and other relationships proposed by researchers. The following shear design procedures were selected as the most useful for providing potential direction for the simplified provisions to be developed in this project:

- ACI 318-02;
- AASHTO Standard Specifications 16th Edition;
- AASHTO 1979 provisions;
- CSA A23.3-94 (Canadian Standards Association: Design of Concrete Structures, 1994);
- AASHTO LRFD Bridge Design Specifications 2nd Edition;
- CSA A23.3-04;
- Eurocode EC2, Part 1(1991), Eurocode EC2 (2003);
- German Code (DIN, 2001);
- AASHTO Guide Specification for Segmental Bridges (ASBI);
- The Japanese Code (JSCE Standards, 1986); and
- The shear design approach recently developed by Tureyen and Frosch.

These shear design procedures are summarized in Appendix B, which is included in *NCHRP Web-Only Document 78*. In this section, a comparison is made of how V_c, V_s, $V_{n,max}$, and $A_{v,min}$ are evaluated. See Table 3. To compare these provisions, relationships have been modified when possible in order to use LRFD nomenclature and psi units.

Based on the comparison of design formulas presented in Table 3, and from consideration of the underlying bases for these expressions, the following observations were made. (A focus on the attributes of each design approach was considered important for the selection and development of the proposed simplified provisions.)

1. The changes incorporated in the 2004 Canadian Standards Association Code for the Design of Concrete Structures, CSA A23.3-04, greatly simplify the procedure for the design of concrete structures using an approach functionally identical to the LRFD Sectional Design Model. In the CSA A23.3-04, the following significant changes were made: (a) The tables for calculating β and θ were replaced by simple formulas that are easy to remember. See Equations 12 and 13.

$$\beta = \frac{4.8}{(1+1500\epsilon_x)}\frac{51}{(39+s_{xe})} \quad \text{where for members with } A_v < A_{v,min} \qquad \text{(Eq. 12a)}$$

$$\beta = \frac{4.8}{(1+1500\epsilon_x)} \quad \text{for members with } A_v \geq A_{v,min}\text{, note } s_{xe} = 12 \text{ inches} \qquad \text{(Eq. 12b)}$$

$$\theta = 29 + 7000\epsilon_x \qquad \text{(Eq. 13)}$$

(b) A further simplification is that the iterative means of calculating β and θ is eliminated by assuming that the angle θ is equal to 30 degrees in the evaluation of ϵ_x. Thus, Equation 1-6 is simplified to Equation 14.

$$\epsilon_x = \frac{\epsilon_t}{2} = \frac{M_u / d_v + 0.5N_u + V_u - V_p - A_{ps}f_{po}}{2(E_sA_s + E_pA_{ps})} \qquad \text{(Eq. 14)}$$

However, the procedure for analyzing the shear capacity remained iterative given that the longitudinal strain is a function of the shear design force. The combination of these two changes greatly simplifies the design procedure to the extent that the use of the Sectional Design Model in CSA A23.3-04 is at least as simple, if not simpler to use, than the AASHTO standard method. The reality of this observation is apparent in the design examples of Appendix J (included in *NCHRP Web-Only Document 78*).

2. In ACI 318-02, AASHTO standard and ASBI, the calculated value for *Vc* is an estimate of the diagonal cracking load. This approach was considered useful for both assessing the condition of a member in the field and for checking whether or not the member was expected to be cracked in shear under service loads. It was also thought that independent consideration of the two types of diagonal cracking, web-shear and flexure-shear, as used in ACI 318-02 and the AASHTO Standard Specifications, was useful for characterizing shear behavior and for visualizing the effectiveness of the shear reinforcement.

3. The AASHTO LRFD specifications require a larger minimum amount of shear reinforcement than most other codes. This requirement was examined using the results of the experimental database of test results.

TABLE 3 Comparison of different design approaches (units: psi, in, lbs)

Approaches	V_c	V_s	$V_{n,\max}$	$A_{v,\min}$
ACI318-02	- For RC members: $V_c = 2\sqrt{f_c'}b_w d$ (empirical eq.) - For PC members: The lesser of V_{ci} and V_{cw}. $V_{cw} = (3.5\sqrt{f_c'} + 0.3f_{pc})b_w d + V_p$ $V_{ci} = 0.6\sqrt{f_c'}b_w d + V_d + \frac{V_i M_{cr}}{M_{\max}}$	$V_s = \frac{A_v f_y d}{s}$	$V_c + 8\sqrt{f_c'}b_w d$	$0.75\sqrt{f_c'}\frac{b_w s}{f_y}$ $\geq \frac{50b_w s}{f_y}$ also for PC members with $f_{se} \geq 0.4f_{py}$, $\frac{A_{ps}f_{pu}s}{80f_y d}\sqrt{\frac{d}{b_w}}$
AASHTO STD (1989)	- For RC members: $V_c = 2\sqrt{f_c'}b_w d$ (empirical eq.) - For PC members: The lesser of V_{ci} and V_{cw}. $V_{cw} = (3.5\sqrt{f_c'} + 0.3f_{pc})b_w d + V_p$ $V_{ci} = 0.6\sqrt{f_c'}b_w d + V_d + \frac{V_i M_{cr}}{M_{\max}}$	$V_s = \frac{A_v f_y d}{s}$	$V_c + 8\sqrt{f_c'}b_w d$	$\frac{50b_w s}{f_y}$
CSA (1994)	$V_c = \beta\sqrt{f_c'}b_v d_v$ (derived from MCFT) β is given in Table as a function of ε_x, $(v/f_c'$ or $s_{xe})$	$V_s = \frac{A_v f_y d_v \cot\theta}{s}$ θ is given in Table as a function of ε_x, $(v/f_c'$ or $s_{xe})$	$0.25f_c'b_v d_v + V_p$	$0.72\sqrt{f_c'}\frac{b_w s}{f_y}$
AASHTO LRFD (2002)	$V_c = \beta\sqrt{f_c'}b_v d_v$ (derived from MCFT) β is given in Table as a function of ε_x, $(v/f_c'$ or $s_{xe})$	$V_s = \frac{A_v f_y d_v \cot\theta}{s}$ θ is given in Table as a function of ε_x, $(v/f_c'$ or $s_{xe})$	$0.25f_c'b_v d_v + V_p$	$\sqrt{f_c'}\frac{b_v s}{f_y}$
CSA (2004)	$V_c = \beta\sqrt{f_c'}b_v d_v$ (derived from MCFT) $\beta = \frac{4.8}{(1+1{,}500\varepsilon_x)}\frac{51}{(39+s_{xe})}$ $\varepsilon_x = \frac{M_u/d_v + 0.5N_u + V_u - \phi_p V_p - A_{ps}f_{po}}{2(E_s A_s + E_p A_{ps})}$	$V_s = \frac{A_v f_y d_v \cot\theta}{s}$ $\theta = 29 + 7000\varepsilon_x$	$0.25f_c'b_v d_v + V_p$	$0.72\sqrt{f_c'}\frac{b_v s}{f_y}$
EC2 (1991)	- Standard method: $V_c = 0.2758k\beta(f_{ck})^{2/3}(1.2+40\rho_\ell)b_w d$ (empirical eq.) where $1.0 \leq \beta = \frac{2.5d}{x} \leq 5.0$ $k = 1.6 - 0.0254d \geq 1.0$ - Variable-angle truss method: $V_c = 0$	- Standard method: $V_s = \frac{A_v f_y (0.9d)}{s}$ - Variable-angle truss method: $V_s = \frac{A_v f_y (0.9d)\cot\theta}{s}$ where $0.4 < \cot\theta < 2.5$ but for beams with curtailed longitudinal reinforcement: $0.5 < \cot\theta < 2.0$	- Standard method: $0.5\upsilon(0.67f_{ck})b_w(0.9d)$ $\upsilon = 0.7 - \frac{f_{ck}}{29000} \geq 0.5$ - Variable-angle truss method: $\frac{\upsilon(f_{ck}/1.5)b_w(0.9d)}{\cot\theta + \tan\theta}$ where $\cot\theta$ can be obtained as: $\cot\theta \leq \sqrt{\left(\frac{\upsilon(f_{ck}/1.5)}{\rho_v f_y} - 1\right)}$	for $f_c' \leq 2800\ psi$: $\rho_v f_y \geq 0.35$ for $f_c' \leq 4800\ psi$: $\rho_v f_y \geq 0.52$ for $f_c' \leq 6900\ psi$: $\rho_v f_y \geq 0.64$

TABLE 3 (Continued)

EC2 (2003)	- For members without shear reinforcement: $V_c = \left[3.31k(100\rho_l f_{ck})^{1/3} - 0.15N_u / A_c\right]b_w d$ $\geq (v_{\min} - 0.15N_u / A_c)b_w d$ (empirical eq.) where $v_{\min} = 0.4215k^{3/2} f_{ck}^{1/2}$ $k = 1 + \sqrt{\frac{7.87}{d}} \leq 2.0$ - For members with shear reinforcement: $V_c = 0$	$V_s = \frac{A_v f_y (0.9d)\cot\theta}{s}$ where $1 \leq \cot\theta \leq 2.5$	$\left[\alpha_{cw}\upsilon_1 \frac{(0.67 f_{ck})}{\cot\theta + \tan\theta}\right] b_w z$ α_{cw} : taking account of axial compressive stress, $\upsilon_1 = 0.6(1 - f_{ck} / 36250)$, $\cot\theta$ can be obtained as: $\cot\theta \leq \sqrt{\left(\frac{\alpha_{cw}\upsilon_1 (f_{ck}/1.5)}{\rho_v f_y} - 1\right)}$	$\frac{0.00664\sqrt{f_{ck}}}{f_y}$
DIN (2001)	- For members without shear reinforcement: $V_c = \left[2.76k(100\rho_l f_{ck})^{1/3} - 0.12N_u / A_c\right]b_w d$ where $k = 1 + \sqrt{\frac{7.87}{d}} \leq 2.0$ (empirical eq.) - For members with shear reinforcement: $V_c = \left[6.63 f_{ck}^{1/3}\left(1 - 1.2\frac{N_u / A_c}{(0.67 f_{ck})}\right)\right]$ $\times b_w (0.9d)$ (empirical eq.)	$V_s = \frac{A_v f_y (0.9d)\cot\theta}{s}$ where $\cot\theta =$ $\frac{\left(1.2 - 1.4\frac{N}{A_c} / (0.57 f_{ck})\right)}{(1 - V_c / V_u)}$ but $0.58 \leq \cot\theta \leq 3.0$	$\frac{(0.67 f_{ck})}{\cot\theta + \tan\theta} b_w (0.9d)$	$\frac{0.0025(f_{ck})^{2/3}}{f_y}$
JSCE (1986)	$V_c = 0.9\beta_d \beta_p \beta_n (f_c')^{1/3} b_w d$ (empirical eq.) $\beta_d = (39.4 / d)^{1/4} \leq 1.5$ $\beta_p = (100\rho_w)^{1/3} \leq 1.5$ $0 < \beta_n \leq 2.0$: taking account of moment and axial compressive stress	$V_s = \frac{A_v f_y (0.87d)}{s}$	$15\sqrt{f_c'} b_w d$	$0.0015 b_w s$
AASHTO Segmental Bridges	$V_c = 2K\sqrt{f_c'} b_w d$ (empirical eq.) where $K = \sqrt{1 + \frac{f_{pc}}{2\sqrt{f_c'}}} \leq 2.0$	$V_s = \frac{A_v f_y d}{s}$	$12\sqrt{f_c'} b_w d$	$\frac{50 b_w s}{f_y}$
AASHTO (1979)	$V_c = 0.6 f_c' b_w (0.9d) \leq 180 b_w (0.9d)$ (empirical eq.)	$V_s = \frac{2A_v f_y (0.9d)}{s}$		$100 b_w s / f_y$ and $\left(0.689\frac{b_w s}{f_y}\right)$
Frosch (2002)	$V_c = 5\sqrt{f_c'} b_w c$ (theoretically derived) $c = kd$, $k = \sqrt{2\rho n + (\rho n)^2} - \rho n$, $n = E_r / E_c$	$V_s = \frac{A_v f_y d}{s}$	$V_c + 8\sqrt{f_c'} b_w d$	$0.75\sqrt{f_c'}\frac{b_w s}{f_y}$ $\geq \frac{50 b_w s}{f_y}$

(Note) When there is more than one approach in a shear design provision, the most commonly used one is summarized in this table. More Detailed information can be found in Appendix B.

$f_{ck} \approx 0.95 f_c'$, $b_w = b_v$

4. The CSA A23.3-04, AASHTO (1979), AASHTO LRFD, Truss Model with Crack Friction, Eurocode 2, JSCE, and DIN all enable the designer to use an angle of diagonal compression flatter than 45 degrees when evaluating the contribution of shear reinforcement to shear capacity.
5. AASHTO LRFD, DIN, and Eurocode 2 allow the engineer to design a member to support a much larger shear stress than permitted in other codes of practice. This shear stress limit is intended principally to guard against diagonal compression failures. In the AASHTO LRFD, the shear design force limit is $0.25f_c'$ plus the vertical component of the prestress, while in ACI 318-02 the limit is approximately $12\sqrt{f_c'}$ plus the vertical component of the prestress when the web-shear crack-

ing shear V_{cw} governs. (Example, when f'_c = 10,000 psi, the LRFD stress limit is 2,500 psi while the AASHTO Standard Specifications limit is 1,000-1,400 psi). Although the authors have concluded that the LRFD stress level is sufficient to prevent web crushing in regions where there is a uniform field of diagonal compression, they are concerned that this limit may be unconservative near supports where there is a significant magnification of the stress as the diagonal compression funnels into the support.

6. The approach by Tureyen and Frosch is not sufficiently mature for the complete design of reinforced through prestressed concrete members of all shapes and loadings. In addition, this approach is a significant departure from how most of the research community views the transfer of shear, even in web-shear regions. Since the publication of this method, members of the research community have responded with experimental evidence that suggests that for beams with shear reinforcement most of the shear carried by the concrete could not be transmitted in the uncracked compression zone suggested by Tureyen and Frosch.

2.3 EVALUATION OF SHEAR DESIGN METHODS USING TEST DATABASE

To evaluate the accuracy of different national codes of practice, a large experimental database was used to evaluate the shear strength ratio (V_{test}/V_{code}) for six different Codes for 1,359 selected beam tests results. Following a brief description of the experimental database, the results of this evaluation are summarized. More detailed information on the database and an evaluation of Codes is presented in Appendixes C and D, which are part of *NCHRP Web-Only Document 78*.

The 1,359-member database consists of 878 reinforced concrete (RC) and 481 prestressed concrete (PC) members. These members were selected from the larger shear database so that members in which significant arch action or flexural failures were suspected were removed from the database. Most of the RC members had rectangular cross sections and were simply supported using bearings positioned underneath the member. Of these, 718 did not contain shear reinforcement and 160 did. The PC members consisted of rectangular, T-shaped, and I-shaped sections and most of the members were simply supported, again on bearings positioned underneath the member. Of these, 321 did not contain shear reinforcement and 160 did. About 80 percent of both the members in the RC and PC components of the database had depths less than 20 inches.

Table 4 presents an examination of the Shear Strength Ratios (V_{test}/V_{code}) and Coefficients of Variation (COV) for ACI 318-02, AASHTO LRFD Bridge Design Specifications (2001), CSA A23.3-04, JSCE Code (1986), Eurocode EC2 2003, and the German Code (DIN, 2001). The code calculated strengths are nominal capacities and therefore all resistance and strength reduction factors are set to 1.0. As a result, the calculated strengths by ACI 318-02 would be equivalent to the calculated strengths by the AASHTO Standard Specifications (16th edition). In this table, the mean and COV are presented for seven segments of the database, all members, all RC members, all RC members without shear reinforcement, all RC members with shear reinforcement, all PC members, all PC members without shear reinforcement, and all PC members with shear reinforcement.

From Table 4, the following observations can be made:

- The AASHTO LRFD and CSA approaches were best able to predict the capacity of the members in this database. The mean of the strength ratios for both of these approaches was very consistent across the different categories of selected members and of a value (1.19–1.46) that would be expected to result in conservative designs

TABLE 4 Code assessment for RC and PC members

Member Type		All	RC	RC	RC	PC	PC	PC
With or without Av			Both	No Av	With Av	Both	No Av	With Av
count	(#)	1359	878	718	160	481	321	160
ACI	Mean	1.44	1.51	1.54	1.35	1.32	1.38	1.21
	COV	**0.371**	**0.404**	**0.418**	**0.277**	**0.248**	**0.247**	**0.221**
LRFD	Mean	1.38	1.37	1.39	1.27	1.40	1.44	1.32
	COV	**0.262**	**0.262**	**0.266**	**0.224**	**0.261**	**0.290**	**0.154**
CSA	Mean	1.31	1.25	1.27	1.19	1.41	1.46	1.31
	COV	**0.275**	**0.274**	**0.282**	**0.218**	**0.261**	**0.287**	**0.147**
JSCE	Mean	1.51	1.36	1.35	1.38	1.80	1.85	1.70
	COV	**0.321**	**0.280**	**0.293**	**0.216**	**0.292**	**0.297**	**0.272**
EC2	Mean	1.85	1.74	1.75	1.70	2.06	2.13	1.91
	COV	**0.409**	**0.336**	**0.328**	**0.373**	**0.470**	**0.343**	**0.687**
DIN	Mean	2.05	1.95	2.10	1.25	2.25	2.59	1.58
	COV	**0.395**	**0.368**	**0.327**	**0.267**	**0.413**	**0.345**	**0.357**

that made reasonably efficient use of shear reinforcement. The small COV was particularly impressive for PC members with shear reinforcement.

- The close correlation between the means and COV for the AASHTO LRFD and CSA methods indicates that these two methods would yield similar designs in terms of the amount of required shear reinforcement. As a result, it is concluded that the equations for calculating β, θ, and ϵ_x in CSA A23.3-04 are reasonable replacements for the tables and the old equation for ϵ_x of AASHTO LRFD that could result in an iterative design approach.
- Although the overall COV for the ACI (AASHTO Standard Specifications) approach is about 40 percent greater than the least overall COV, this results principally from the poor performance of these design provisions in predicting the capacity of RC members that do not contain shear reinforcement. Both the mean and COV of the ACI (AASHTO Standard Specifications) method for RC and PC members with shear reinforcement were quite good. Regarding the prediction of RC members without shear reinforcement, given the measured shear capacity of many of the members, the ACI (AASHTO Standard Specifications) approaches would frequently have required designs with minimum shear reinforcement because of the rule that $A_{v,min}$ is required when $V_u \geq \phi V_c / 2$.
- The JSCE code was somewhat better than the ACI (AASHTO Standard Specifications) approach at calculating the shear strength of RC members, but very conservative at calculating the shear strength of prestressed concrete members. This conservatism results from a limitation in the Japanese code of only doubling V_c due to the effect of prestressing and because a 45-degree parallel chord truss model is used in calculating the contribution of the shear reinforcement.
- Both the EC2 and DIN Codes were considerably less successful in predicting the capacity of members in the shear database. There was a surprisingly large variation in the means across the different categories of members.

This large database of shear test results is also useful for examining minimum required amounts of shear reinforcement in codes of practice. In Figure 19, the shear strength ratio ($V_{test}/V_{AASHTO\text{-}STD}$) is examined for reinforced concrete beams that contain reasonably light amounts of shear reinforcement. The results illustrate that traditional amounts of minimum shear reinforcement, $\rho_v f_y$ = 40-60 psi, were insufficient to ensure that code provisions were conservative. This illustrates why the larger amounts of minimum shear reinforcement required by AASHTO LRFD Sectional Design Model are appropriate.

This database is also useful for evaluating the maximum shear stress design limit. The LRFD Sectional Design Model enables the design of members for up to 2.5 times the maximum shear design stress permitted in the AASHTO Standard Specifications. Figure 20 reveals that the AASHTO Standard Specifications are unduly conservative. However, the authors are concerned that the LRFD limit of $0.25 f_c'$ may be too high. As observed in NCHRP Project 12-56, in the end regions of prestressed concrete bulb-tee girders, the funneling of the diagonal compressive stresses into the support results in stresses that are significantly higher than those assumed in the LRFD Sectional Design Model which was developed for regions of members in which the diagonal compression field is parallel. However, most available test results are for beams simply supported on bearings positioned underneath the

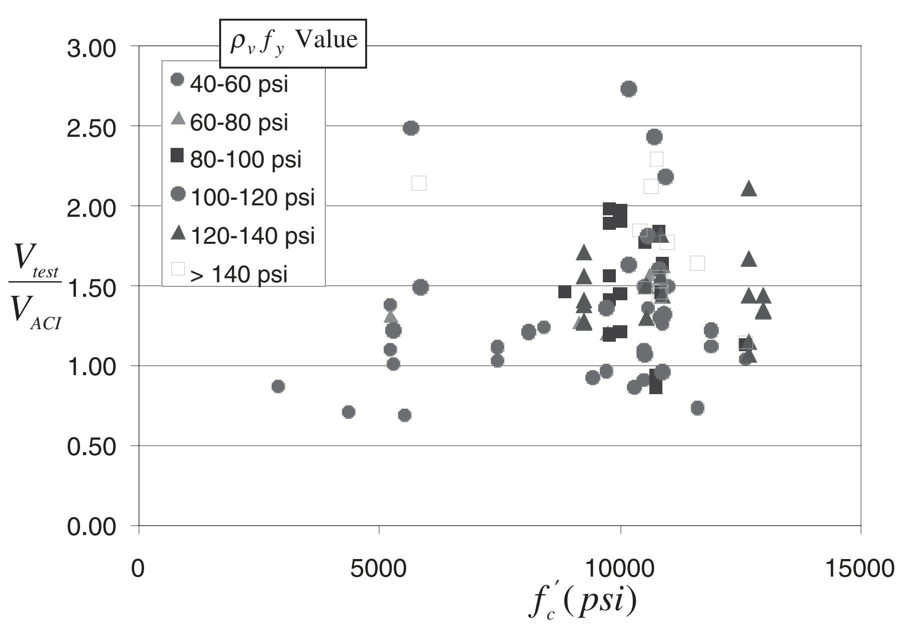


Figure 19. Strength ratio versus concrete compressive strength for RC members.

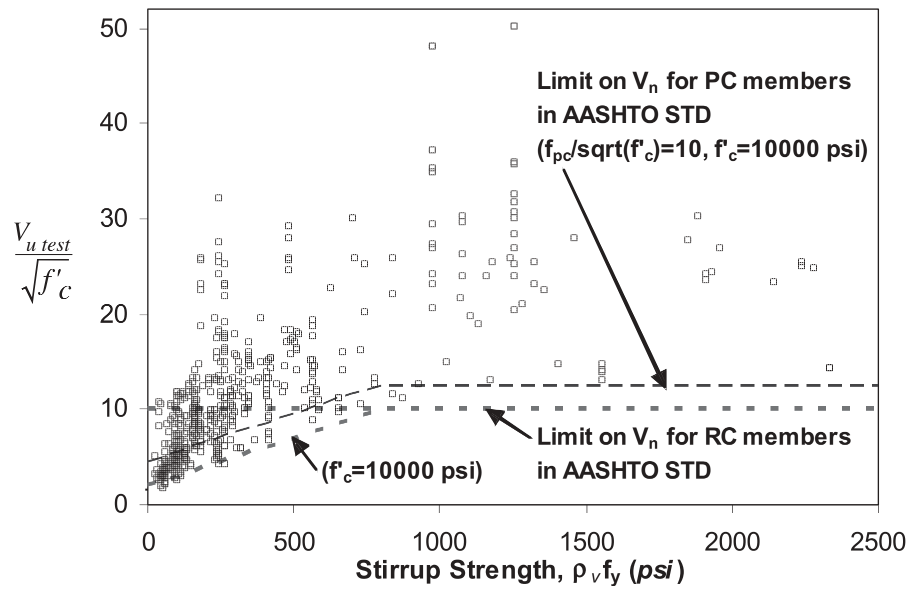


Figure 20. Test data and maximum shear stress limit.

member. In many situations in practice, the ends of beams are built integrally at their ends into piers, columns, or diaphragms. The results of the beam tests conducted for NCHRP Project 12-56 demonstrate that higher maximum shear stresses can be achieved for that situation than for beams simply supported on bearings positioned underneath the member.

2.4 RESULTS OF SURVEY OF PRACTICE

A survey of the design practices of 26 different state DOTs and federal lands bridge design agencies was conducted. This survey included both a written questionnaire and either a telephone briefing on the response to the questionnaire or a written response. Of the 26 agencies polled, 21 responded; these are listed alphabetically at the end of this section. The specific questions and the responses are included in Appendix E, which is part of *NCHRP Web-Only Document 78*. The questionnaire was to determine the status of conversion to LRFD, identify specific problems and practices with respect to concrete element shear design, ascertain preferences for shear design methodologies, and provide a vehicle for organizations to express their opinion of the current LRFD shear design methodology.

Some recurring themes and trends emerged. First, many of the organizations have not yet converted the bulk of their practice to LRFD, although most have undertaken serious in-house evaluation of the likely effects of conversion. In most cases, the in-house evaluators were interviewed and were the primary respondents to the Questionnaire. Only 7 of 21 had converted to LRFD for most of their bridges, even though a deadline for conversion has been set nationally. Consequently, many organizations are actively beginning the conversion process, and thus are on, or just beginning, the learning curve with the LRFD Sectional Design Model (Section 5.8.3.3 of the LRFD specification). This is relevant because two camps of designers seem to exist: those that have become reasonably comfortable with the production of LRFD shear designs and those who view it as a significant hurdle yet to be surmounted. Although some users have become familiar with the mechanics of the method, almost universally designers report that the method is not easily executed by hand and that one often loses sight of the relative mechanics of what is happening in the structure. All agree that the LRFD shear design provisions must be automated with software to be used in production design. This fact naturally leads to loss of comfort with respect to checking designs, because the method cannot be readily executed by hand. Most designers also agree that the Standard Specification method for prestressed design (Section 9.20) that includes V_{ci} and V_{cw} must also be automated to be effective in production work, even though that method is executable by hand. Thus, with the existing AASHTO LRFD provisions one of the most prevalent comments was that designers are losing their physical "feel" for shear design because of the increasing complexity of the design provisions and the resulting automation.

Most agencies using the LRFD shear provisions make no modifications or simplifications to the process before adoption into their design procedures, although many respondents indicated that they would if the simplifications were reasonable. The primary simplification that is being used by a few organizations is the elimination of the iterative process required to determine the crack angle, θ.

Almost all respondents indicated that there had been some difficulty in applying the MCFT provisions and that often difficulties also arose when applying the provisions to bent cap beams, columns, and footings. Difficulties in applying the strut-and-tie method were also commonly reported, particularly for indeterminate beams. Furthermore, the potential for more than one shear design solution was perceived as a problem by about one-third of the respondents, although this was not seen as a significant problem by most.

When asked whether the LRFD shear provisions produce significantly different designs than the Standard Specification method, the responses were mixed. Many were not sure yet, but about one-third indicated that more shear steel is often required, although the amounts of the increase were quite variable, with the high response being about 40 to 50 percent increases for bridges with large skews. This particular respondent also indicated that the demand side had as much to do with the increase as the resistance side, because the LRFD load combinations often produce larger design forces. Several respondents indicated that the revisions to the β and θ values incorporated into the second edition of LRFD helped bring the steel contents into parity with the Standard Specifications method.

Regarding the use of the simpler 1979 AASHTO shear design procedure allowed by the Standard Specifications in a footnote to Section 9.20, about one-third indicated that they still used this method. Furthermore, they report that no problems have apparently arisen from the continued use of the 1979 method.

Nearly all the respondents indicated that a relative simple design method would be highly desirable, even if it was used for checking only. In fact, a simple checking method is one of the most requested items. Also, many designers recognize the advantages of the LRFD provisions for a more accurate evaluation of shear resistance. The LRFD shear provisions thus can often offset the increases on the demand side that are common with LRFD, though this tends toward less conservative designs for shear. Most designers seem to prefer a palette of design approaches. If simple methods provide similar designs to more complex methods, the need to switch to the complex method is not felt warranted. Some would even prefer to include some of the older, tried and proven shear design methods as alternates. This would ease the 'transition trauma' associated with the LRFD learning curve.

The most common types of concrete bridges appear to be prestressed concrete girder bridges, including those designed as simple spans for both dead and live loads and those made continuous for live load. Only a few respondents indicated a routine use of box girders and segmental construction. Most did not know of any cases where the LRFD shear design had eliminated a bridge type from consideration for a given project. However, one case of deeper girders being required when high skews were present was cited. For PC girder bridges, standard designs have been common in the past and are still desirable. However, standard designs have been difficult, if not impossible to achieve, with the LRFD shear procedures.

Most of the LRFD users thought that the engineering time for shear design is not significantly increased over that for the Standard Specifications, provided that shear design is automated using spreadsheets, MathCAD, or other software. However, if designs are attempted by hand, the design time is often significantly increased over that required by the Standard Specifications. This is often a source of discontent when the effort is increased, but the results change very little. The discontent is reinforced by the fact that shear steel is not typically a significant cost relative to the entire bridge cost and by the fact that many designers prefer to be conservative for shear design.

For the relatively common case of girders made continuous for live load, it is widely thought that the current LRFD provisions do not adequately explain the application of the method to this case. Problems seem to be particularly common in the negative moment region. The problem of appropriate use of simultaneously acting internal forces in the resistance equations is, however, not new to LRFD, although confusion seems to be worse with the LRFD shear design procedure. This confusion stems from the fact that cases arise in practice that are more complex than those envisaged during the development of the specifications, thereby producing confusion among designers. This problem is at least in part related to definitions, where common cases are not always clearly explained. Definition problems are compounded when confusion also arises over appropriate signs to be used for internal forces.

The common themes in terms of the most important design issues can be synthesized into the following: (1) There should be a simple, logical method for performing shear design or alternately, checks of designs; (2) The method should provide a feel for the mechanics and should help designers develop a comfort level with the results; (3) The simplified method need not supplant the MCFT theory, but need only supplement the method and provide a logical alternate; and (4) The method does not need to be highly accurate, provided it is conservative.

Regarding field problems that may be related to shear design, most indicated that few, if any, problems have occurred with the more modern design procedures. Several designers outlined problems that have arisen in older bridges whose design predates the current procedures in the Standard Specifications. However, one respondent did indicate that potential problems in segmental construction have arisen when using the LRFD methods because of the lack of a principal tension check for the webs. With respect to fabrication of precast elements, many designers indicated that congestion at the ends of beams is quite common, although this stems more from the confinement steel requirements than from shear steel requirements.

Finally, the issue of bridge rating using LRFR (the LRFD rating approach) versus the current LFR rating is a source of ongoing concern, although most have not had any experience with the LRFR method yet. Whether this is a potential problem depends primarily on how the method is phased in and whether it is mandated for older bridges. This is viewed as more of a policy issue to ensure consistency, than a technical issue.

The following states responded to the questionnaire: Alaska, Arkansas, California, Delaware, FHWA CFLHD, Florida, Georgia, Illinois, Kansas, Kentucky, Mississippi, Missouri, Montana, Nevada, New Hampshire, New Jersey, Oregon, Pennsylvania, Tennessee, Texas, and Washington.

2.5 CRITERIA FOR PROPOSED SIMPLIFIED PROVISIONS

Based on the experiences of practicing engineers, the review of shear design methods in codes of practice, the analysis of experimental test data, and a comparison of the required amounts of shear reinforcement for sections in a design database (presented in Section 2.9), the following set of criteria were developed for the simplified provisions:

The simplified provisions should

- Be directly usable, without iteration, for the design of a member;
- Be directly usable, without iteration, for evaluating the capacity of a member;
- Be useful in conducting field evaluations by providing the engineer with an estimate of the loads at which shear cracking is expected to occur in the member;
- Have a basis that can be readily understood and explained by one engineer to another while still being based on a mechanistic model for strength;
- Allow for rapid and reliable hand-based designs and checks of existing designs;
- Not be a pure simplification of the existing LRFD specifications because a significant shortcoming of the current LRFD shear design provisions was considered to be the difficulty of fully understanding the MCFT and how the LRFD provisions were derived from this theory.
- Avoid the necessity of calculating the angle θ. If a simple relationship is to be suggested for calculating θ, then there needs to be a default value that can be used if the engineer does not wish to make this calculation;
- Not enable the effects of all actions (axial load, moment, shear, and prestressing) to be simultaneously considered as this is already done in the current LRFD Sectional Design Model (S5.8.3);
- Provide safe and accurate estimates of shear capacity of the members in the selected experimental test database without significant trends in the strength ratios (V_{test}/V_{code}) with design parameters (d, f'_c, $\rho_v f_y$, ρ_l, etc).
- Result in reasonable shear reinforcement amounts ($\rho_v f_y$) being required for the sections in the design database where "reasonableness" is assessed from a comparison of the required amounts of shear reinforcement by analysis methods in comparison with the requirements of other codes of practice and analysis methods.

Where the required shear reinforcement amount ($\rho_v f_y$) by the simplified specifications differs substantially from what is required by the existing AASHTO Standard Specifications, the LRFD specifications, and analytical methods, then the reasons for the required amount of shear reinforcement should be well justified and the required amount of shear reinforcement should be conservative.

CHAPTER **3**

PROPOSED CHANGES TO LRFD BRIDGE DESIGN SPECIFICATIONS

Sections 3.1 and 3.2 present the two change proposals. The first proposal is the Proposed Simplified Provisions (in a form similar to the AASHTO standard provisions) and the second is a modification to the current LRFD Sectional Design Model. Section 3.3 presents an overview of design examples using these two methods. Section 3.4 presents a justification for the proposals using experimental test data and Section 3.5 presents a justification through a comparison of required strengths of shear reinforcement for many design cases. Section 3.6 examines how change proposals, if implemented, may affect design, and then in Section 3.7 their anticipated effect on safety and economy is presented. This effect is evaluated for users of both the AASHTO Standard Specifications and LRFD Sectional Design Model who choose to use either of the two proposed shear design methods. Section 3.8 presents how design database was incorporated in NCHRP Process 12-50.

Two changes to the LRFD specifications are presented. Change Proposal 1 is the addition of alternative (or simplified) shear design provisions that reintroduce the idea of basing V_c on the lower of the calculated web-shear (V_{cw}) and flexure-shear (V_{ci}) strength but where a new and more conservative relationship is used for V_{cw} and where a variable angle truss model is introduced for evaluating the contribution of the shear reinforcement based on the angle of diagonal cracking. This report refers to this alternative as either the "proposed simplified provisions" or the "Modified Standard Approach."

Change Proposal 2 is that the current tables for determining β and θ, as well as the equation for evaluating ϵ_x in the Sectional Design Model (S5.8.3), be replaced by the relationships for β, θ, and ϵ_x that have already been incorporated in the Canadian Concrete Design Code for Concrete Structures (CSA A23.3-04). Therefore this change proposal is referred to as the CSA Method.

3.1 CHANGE PROPOSAL 1: PROPOSED SIMPLIFIED APPROACH (MODIFIED V_{CW} AND V_{CI} OR MODIFIED STANDARD)

An alternative (or simplified) shear design method is proposed to overcome the limitations of the modified LRFD Sectional Design Model as presented in change proposal 2 (CSA Method).

After considering the provisions in numerous codes of practice and of other suggested shear design approaches, the research team proposes to adopt a method that shares the approach taken in the current AASHTO Standard Specifications and in ACI 318-02 where, for shear design, the structure is considered to be divided into regions of web-shear and flexure-shear cracking. The ability to estimate a lower bound to the diagonal cracking load for the purpose of service evaluations was considered important to include in the AASHTO LRFD Specifications, particularly because the AASHTO Standard Specifications will be discontinued in time. The proposed simplified Specifications differ from the current AASHTO Standard Specifications in the expression for V_{cw}, the assumed angle for θ, the maximum shear stress permitted for design, the minimum required amount of shear reinforcement, and requirements for the amount of longitudinal tension reinforcement that must be developed at the face of the support. Furthermore, the values for V_{cw} are selected so that they are consistent with the contribution of the concrete to the ultimate shear capacity of a beam in accordance with the crack model with friction concept. The expressions for V_{cw} are developed so that they can be applied easily to beams with deformed bar reinforcement only, with prestressed reinforcement only, and all combinations of those reinforcements. A need for seamlessness between reinforced and prestressed concrete provisions for shear was not recognized when the AASHTO Standard Specifications for shear in prestressed beams were developed because, at that time, prestressed and reinforced concrete were seen as separate materials.

The basis for the proposed simplified provisions is summarized below, followed by the specific proposed relationships for the simplified (alternative) LRFD shear design specifications. The detailed explanation of the basis for the equations of the proposed simplified provision is given in Appendix F, which is included in *NCHRP Web-Only Document 78.*

3.1.1 Basis of Proposed Simplified Provisions

Web-Shear Cracking Strength, V_{cw}

The estimate of the web-shear cracking force follows directly from Mohr's Circle of stress.

$$V_{cw} = f_t \sqrt{1 + \frac{f_{pc}}{f_t}}\, b_w d + V_p \qquad \text{(Eq. 15)}$$

where d is suggested to be not less than 0.8h.

The tensile strength of the concrete, f_t, can be taken as somewhere between $2\sqrt{f_c'}$ and $4\sqrt{f_c'}$ where f_c' is in psi units. In the proposed provisions, stress is expressed in ksi but it is considered more useful to present the proposal with the stress given in psi units. A tensile cracking strength close to $4\sqrt{f_c'}$ is believed to provide a more accurate estimate of the diagonal cracking strength in the design of the end regions of a fully prestressed member in which there is no effect of flexure while a value of $2\sqrt{f_c'}$ is a better estimate of the diagonal cracking load in a reinforced concrete member or a prestressed member with a low level of prestressing. A transition between those two levels is a function of the level of the prestress and the axial load.

Flexure-Shear Cracking Strength, V_{ci}

For flexure-shear cracking of prestressed beams, the expression used in the AASHTO Standard Specification is

$$V_{ci} = 0.6\sqrt{f_c'}b_w d + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.7\sqrt{f_c'}b_w d \qquad \text{(Eq. 16)}$$

where the sum of the second and third terms is an estimate of the shear force at the time of flexural cracking while the first term is the increase in shear that has been observed in experiments for a flexural crack to propagate into a diagonal crack.

Although the concrete contribution, V_c, is taken as an estimate of the diagonal cracking load, it must also be a lower bound estimate of the concrete contribution to shear resistance at the ultimate limit state. At the ultimate limit state, the concrete contribution is the sum of the shear carried in the compression zone, the shear carried across diagonal crack due to shear-friction (aggregate interlock), direct tension across diagonal cracks, dowel action, and arch action. Many factors influence the contributions of each of these mechanisms and attempts to reasonably account for them lead to complicated expressions for V_c. Thus, the approach taken by the research team in developing this simplified approach has been to use a lower bound estimate of the diagonal cracking load that, when added to the calculated stirrup contribution to shear resistance, is shown to provide a conservative estimate of the capacity of test beams presented and discussed in Section 3.3.

Contribution of Shear Reinforcement, V_s

The contribution of the shear reinforcement to shear resistance is given in Equation 17. The angle of shear cracking can be directly calculated by Mohr's circle of stress, as shown in Figures B-4 and F-2 in the appendixes; see Equation 18.

$$V_s = \frac{A_v f_y d}{s}\cot\theta \qquad \text{(Eq. 17)}$$

where: $$\cot\theta = \sqrt{1 + \frac{f_{pc}}{f_t}} \qquad \text{(Eq. 18)}$$

When f_{pc} = 0, the axial stress is zero, or if flexure-shear cracking governs, then cot θ = 1 (θ = 45 degrees).

3.1.2 Proposed Simplified Provisions

The proposed simplified provisions are given here in both ksi and psi units. In order not to imply a greater level of precision in the procedure than can be justified, the coefficients for the expressions in ksi units, as currently used in the LRFD specifications, are rounded off.

Web-Shear Cracking Strength

$$V_{cw} = (0.06\sqrt{f_c'} + 0.30 f_{pc}) b_v d_v + V_p$$

(where stress is in ksi units) (Eq. 19)

which is equivalent to

$$V_{cw} = (1.9\sqrt{f_c'} + 0.30 f_{pc}) b_v d_v + V_p$$

(where stress is in psi units) (Eq. 20)

Flexure-Shear Cracking Strength

$$V_{ci} = 0.02\sqrt{f_c'}b_v d_v + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 0.06\sqrt{f_c'}b_v d_v$$

(where stress is in ksi units) (Eq. 21)

which is equivalent to

$$V_{ci} = 0.632\sqrt{f_c'}b_v d_v + V_d + \frac{V_i M_{cr}}{M_{max}} \geq 1.9\sqrt{f_c'}b_v d_v$$

(where stress is in psi units) (Eq. 22)

The 0.06 coefficient in Equation 19 establishes a uniform minimum V_c contribution over the length of the member independent of whether a web-shear or flexure-shear region is being designed. The coefficient of 0.06 (ksi units) is also very close to the traditional coefficient of 1.7 (psi units) when it is considered that $d_v = 0.9d$.

$$\begin{aligned} V_{ci,\min} &= 1.7\sqrt{f_c'}b_v d_v \text{ (psi units)} \\ &\equiv 1.7\sqrt{f_c'}/1000/0.9 \equiv 0.0597\sqrt{f_c'}b_v d_v \text{ (ksi units)} \end{aligned}$$

(Eq. 23)

Theta

$$\cot\theta = 1.0 + 3\frac{f_{pc}}{\sqrt{f_c'}} \le 1.8$$

when $V_{cw} < V_{ct}$ (where stress is in ksi units) (Eq. 24)
which is equivalent to:

$$\cot\theta = 1.0 + 0.095\frac{f_{pc}}{\sqrt{f_c'}} \le 1.8$$

(where stress is in psi units) (Eq. 25)

This expression was selected so that cot(θ) was equal to 1.0 (θ = 45 degrees) when $f_{pc} = 0$ (i.e. non-prestressed member). The slope of the influence of f_{pc} on θ provides a good correlation with test data.

The complete design procedure is shown in Figure 21.

3.2 CHANGE PROPOSAL 2: MODIFICATION OF LRFD SECTIONAL DESIGN MODEL (S5.8.3)

The shear design provisions in the 1994 Canadian Standards Association code for the Design of Concrete Structures (*6*) were essentially the same as the Sectional Design Model in the first three editions of the LRFD Bridge Design Specifications (*1, 7, 17*). In order to simplify the CSA shear design provisions, the 2004 code introduced equations for evaluating β and θ that replaced the tables. Furthermore, a new equation for ϵ_x was introduced by assuming that θ was 30 degrees when evaluating the influence of shear on the longitudinal strain, ϵ_x. Change proposal 2 is the adoption of the CSA relationships for β, θ, and ϵ_x. These provisions are herein referred to as the CSA Method. This method is presented below.

$$V_n = V_c + V_s + V_p \le 0.25 f_c' b_v d_v + V_p \quad \text{(Eq. 26)}$$

where: $V_c = \beta\sqrt{f_c'} b_v d_v$ (*in., psi*) : concrete contribution (Eq. 27)

and $V_s = \dfrac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s}$: steel contribution (Eq. 28)

As shown in Figure 1, the longitudinal strain, ε_x, is computed at mid-depth of the cross-section by:

$$\epsilon_x = \frac{M_u/d_v + 0.5N_u + V_u - V_p - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps})} \quad \text{(Eq. 29a)}$$

When ε_x is negative, it is taken as either zero or recalculated by changing the denominator of Equation 29a such that the equation becomes:

$$\epsilon_x = \frac{M_u/d_v + 0.5N_u + V_u - V_p - A_{ps} f_{po}}{2(E_s A_s + E_p A_{ps} + E_c A_{ct})} \quad \text{(Eq. 29b)}$$

where A_{ct} is the area of concrete in tension.

However ϵ_x shall not be taken as less than -0.2×10^{-3}.

For members having less than minimum shear reinforcement, as required by Equation 32, the equivalent crack spacing parameter, s_{xe}, is calculated as:

$$s_{xe} = \frac{1.38 s_x}{0.63 + a_g} \text{ (in. units)} \quad \text{(Eq. 30)}$$

where a_g is the maximum aggregate size (*in.*). Then, the factor accounting for the shear resistance of cracked concrete, β, can be computed from:

$$\beta = \frac{4.8}{(1 + 1500\epsilon_x)}\frac{51}{(39 + s_{xe})} \text{ (in. units)} \quad \text{(Eq. 31)}$$

The minimum area of shear reinforcement is:

$$A_{v,\min} = \sqrt{f_c'}\frac{b_v s}{f_y} \text{ (in., psi)} \quad \text{(Eq. 32)}$$

It should be noted that minimum shear reinforcement is required when the factored shear force exceeds V_c, rather than $V_c/2$ as required by the ACI 318-02 code. Furthermore, the minimum amount of shear reinforcement is greater than the minimum amount required by ACI 318-02 and the AASHTO Standard Specifications.

For members having at least minimum transverse reinforcement, the angle of the diagonal compression field, θ, is calculated as:

$$\theta = 29 + 7000\epsilon_x \quad \text{(Eq. 33)}$$

and the coefficient, β, is obtained from Equation 31 with the equivalent crack spacing parameter, s_{ze}, set to 12 inches.

In the modified LRFD design provisions presented in Appendix F, the contractor proposes that when the member is not continuous, or cast integrally with the support, the end region is designed by the strut-and-tie method in LRFD Article 5.6.3 (7) when the design shear stress exceeds $0.18f_c'$ at the first critical section from the support. This is to guard against a diagonal compression failure that could occur due to funneling of the diagonal compression above a simple support.

The complete CSA design procedure is presented in Figure 22.

3.3 DISCUSSION OF DESIGN EXAMPLES

To illustrate the use of these two proposed methods in different design situations, eight design examples were prepared. These examples were selected from existing PCI examples, suggestions from project panel members, and new examples selected by the contractor. Each example begins with the completed flexural design at a section with specified

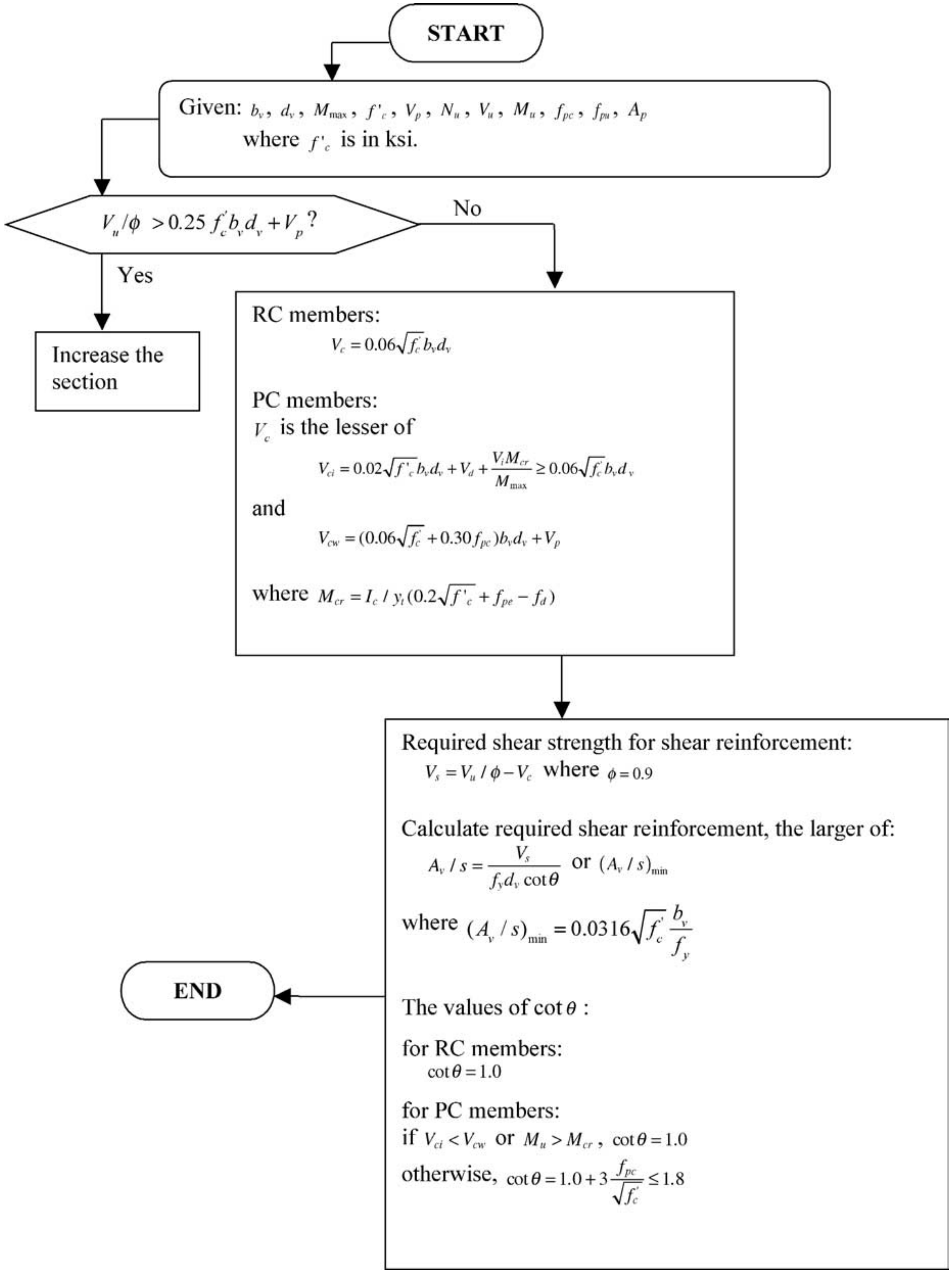


Figure 21. Flowchart for use of proposed simplified provisions.

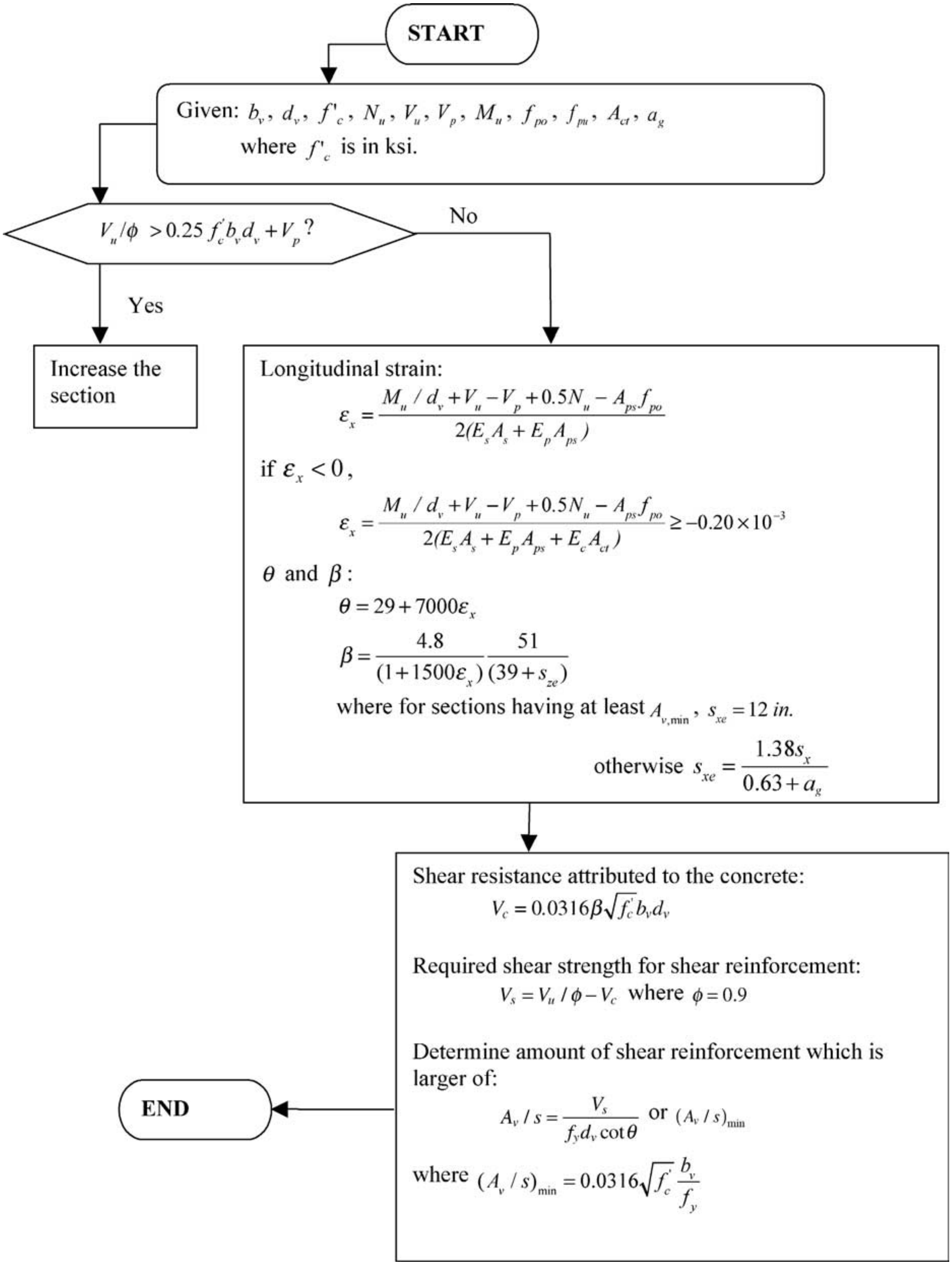


Figure 22. Flowchart for shear design in accordance with CSA.

factored sectional design forces (V_u, M_u, and N_u). In these examples, the critical sections used are those already available from the designers who provided the case studies on which these examples are based.

Example 1: Precast, Pretensioned Noncomposite Box Beam

This example demonstrates the shear design at a specific section of a 95-ft single-span AASHTO Type BIII-48 box beam bridge with no skew. The example is based on Example 9.2 of the *PCI Bridge Design Manual* [PCI, 1997]. Seven of the 29 0.5-inch diameter strands used for flexural tension reinforcement in the 39-inch-deep precast box beams are debonded.

Example 2: Three-Span Continuous Precast, Pretensioned Girders

This example is based on Example 9.6 of the *PCI Bridge Design Manual* [PCI, 1997]. The bridge uses 72-inch bulb tees with harped (draped) pretensioned strands on 110-foot end spans and 120-foot interior span. The beams are made continuous for live load by the addition of unstressed reinforcement in the deck in the negative moment region. This example illustrates the shear design in the negative moment region of a beam made continuous with nonprestressed reinforcement.

Example 3: Reinforced Concrete Cap Beam

This design example demonstrates the shear design of a section of a non-prestressed 15-ft span cap beam supported on three circular columns of 3-ft diameter. The cap beam supports a 3-lane superstructure consisting of six AASHTO Type IV beams.

Example 4: Reinforced Concrete Column and Footing

This design example demonstrates shear design for two sections of a reinforced concrete column and footing, which are part of a pier designed by Modjeski and Masters, Inc. In the shear design of the footing, only one-way action is considered for a demonstration of proposals.

Example 5: Two-Span Continuous Post-Tensioned Box Bridges in Nevada

BERGER/ABAM designed this two-span, cast-in-place, post-tensioned box girder bridge. Spans are 110 and 120 feet for the 5-foot deep box girder. Shear design for positive and negative moment regions, and in the vicinity of the inflection point, are illustrated.

Example 6: Multi-Post Bent Cap

This design example is for a multi-post bent cap beam 86 feet wide. The beam is supported on four columns distributed at 22 ft centers below the beam. Figure J-20 shows the elevation of the multi-post bent cap beam. The design section is taken at the internal face of the first pier in the first bay. Bridge details were provided by the Tennessee Department of Transportation.

Example 7: Type IV Girder

This example demonstrates the shear design of a section of a 100-ft span AASHTO Type IV beam bridge. Bridge details were provided by the Texas Department of Transportation. The bridge consists of 3 spans with each span simply supported. The composite pretensioned beams are 54 inch deep and have an 8 in. thick deck.

Example 8: Segmental Girder

This example gives the shear design calculations for a 5-span Precast Balanced Cantilever Bridge constructed using AASHTO-PCI-ASBI segmental box girders. The design section is taken from the second bay near the support.

3.4 EVALUATION OF SIMPLIFIED PROVISIONS WITH SELECTED TEST DATA

Appendix G presents a detailed evaluation of the two proposed changes to the LRFD shear design provisions using the selected experimental database. The first of these is the proposed simplified provisions which are a significant modification to the AASHTO standard approach and which introduces the use of a variable angle truss model.

The second is the proposed modification to the LRFD Sectional Design in which the equations replace the tables for evaluating β and θ and a simplified relationship is used for evaluating the strain at mid-depth, ϵ_x, that eliminates the dependency on the angle θ. This is essentially the new CSA approach.

The selected experimental database consists of 64 reinforced concrete members and 83 prestressed concrete members. All of these members contain at least the traditional ACI level of minimum shear reinforcement ($\rho_v f_y > 50$ psi), have an overall height of a least 20 inches, were cast with concretes that had cylinder compressive strengths of 4000 psi or greater, and had shear span-to-depth ratios at least 1.70 and usually considerably higher. The members were selected from the larger database described in Section 2.3.

Appendix G also presents evaluations with the selected database of the current LRFD Specifications and AASHTO standard procedures, an examination of the proposal's ability to predict cracking strengths, and a more detailed discussion of the accuracy of proposed changes. In this section, the strength ratios are evaluated briefly.

Table 5 compares the strength ratios for the 64 selected reinforced concrete and 83 selected prestressed concrete test results. From this table, the following observations can be made about how well these four design provisions predicted the capacity of members in the selected databases:

1. The LRFD and CSA methods provided the most accurate and very similar estimates of the shear capacity of both the RC and PC members. With Mean strength ratios ranging from 1.1 to 1.24 and coefficients of variation (COV) for these ratios that range from 0.13 to 0.18, the fit with the test data is considered to be excellent.
2. For the 64 RC members, the proposed simplified provisions were slightly less accurate than the LRFD and CSA approaches but far more accurate than the AASHTO standard approach. The proposed simplified provisions and the STD approach had similar Mean values, but the STD approach had a significantly larger COV. This result implies that there are likely to be many more situations in which the STD provisions are less conservative than the proposed simplified provisions.
3. For the 83 PC members, the proposed simplified provisions had a larger Mean and a slighter larger COV than the STD specifications.

Figure 23 presents the trends in the strength ratios for the proposed simplified provisions as a function of: (a) f'_c; (b) depth, d; (c) percentage of longitudinal reinforcement, ρl; and (d) strength of shear reinforcement provided, $\rho_v f_y$. The results illustrate a slight trend in strength ratio with the percentage of longitudinal reinforcement. Some of the most conservative results are for members that contain high levels of shear reinforcement. This result is to be expected as the proposed simplified provisions limit the shear strength conservatively to guard against brittle diagonal compressive failures.

Figure 24 presents the trends in the strength ratios for the CSA method as a function of: (a) f'_c; (b) depth, d; (c) percentage of longitudinal reinforcement, ρl; and (d) strength of shear reinforcement provided, $\rho_v f_y$. The results illustrate no trend in the strength ratio with any of these parameters for the range of values shown.

The results presented in Table 5, Figure 23, and Figure 24 give the impression that the proposed simplified provisions are less accurate than the current AASHTO standard methods for prestressed members and are significant less accurate than the CSA method. This may be misleading for, as previously mentioned, the types of members tested in laboratories do not represent well the types of members built in the field. Additionally, most members tested in laboratories were designed to fail near a support while a member in the field must be designed for shear over its entire length. For this reason, the fit with experimental test data should only be viewed as one evaluation metric. Another is a comparison of the required amount of shear reinforcement by multiple methods, including analytical results, for a large number of design sections that represent the types of sections designed in practice. This comparison is presented in the next section.

3.5 COMPARISON OF REQUIRED STRENGTH OF SHEAR REINFORCEMENT IN DESIGN DATABASE

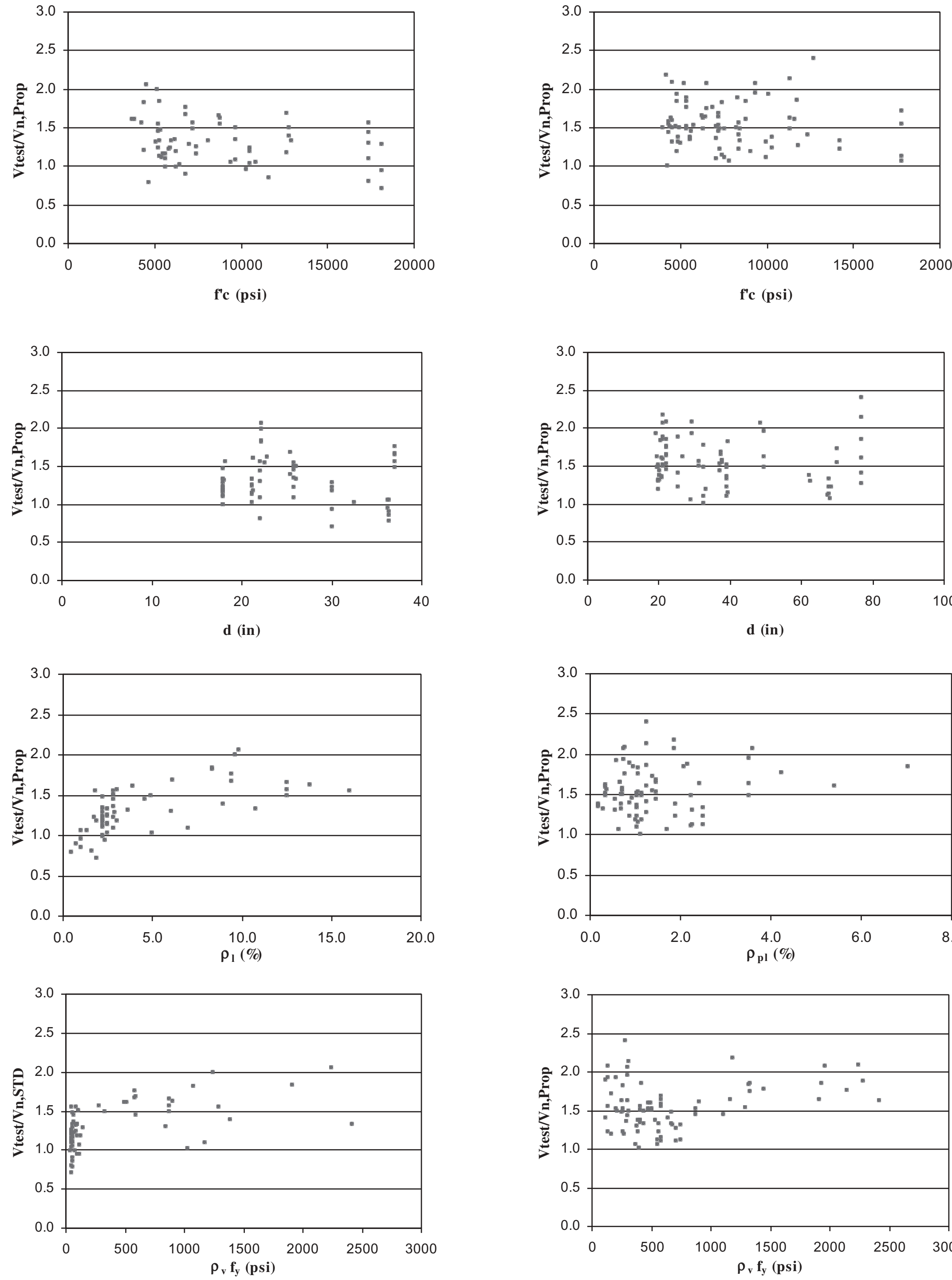
To further assess the safety and efficiency of the two change proposals, the required amount of shear reinforcement ($\rho_v f_y$) by these two methods, the LRFD and AASHTO standard methods, and program Response 2000 (Response 2000) are compared for a large number of design sections. A summary of these results is presented following a description of the design database. A more complete presentation of the results and the design database is presented in Appendix H, which is included in *NCHRP Web-Only Document 78.*

The design database was developed to cover practical design sections. The database included prestressed and non-prestressed sections, composite and non-composite, as well

TABLE 5 Evaluation of approaches for selected RC and PC members

Member type	64 RC members				83 PC members			
Statistical Values	$\frac{V_{test}}{V_{n,STD}}$	$\frac{V_{test}}{V_{n,LRFD}}$	$\frac{V_{test}}{V_{n,CSA}}$	$\frac{V_{test}}{V_{n,Prop}}$	$\frac{V_{test}}{V_{n,STD}}$	$\frac{V_{test}}{V_{n,LRFD}}$	$\frac{V_{test}}{V_{n,CSA}}$	$\frac{V_{test}}{V_{n,Prop}}$
Average	1.296	1.214	1.105	1.309	1.322	1.227	1.245	1.542
Stdev	0.431	0.217	0.172	0.291	0.211	0.177	0.167	0.292
COV	0.333	0.179	0.156	0.222	0.160	0.145	0.134	0.189
% SR < 0.9	17.9	7.3	11.7	8.0	2.2	0.9	1.9	1.4

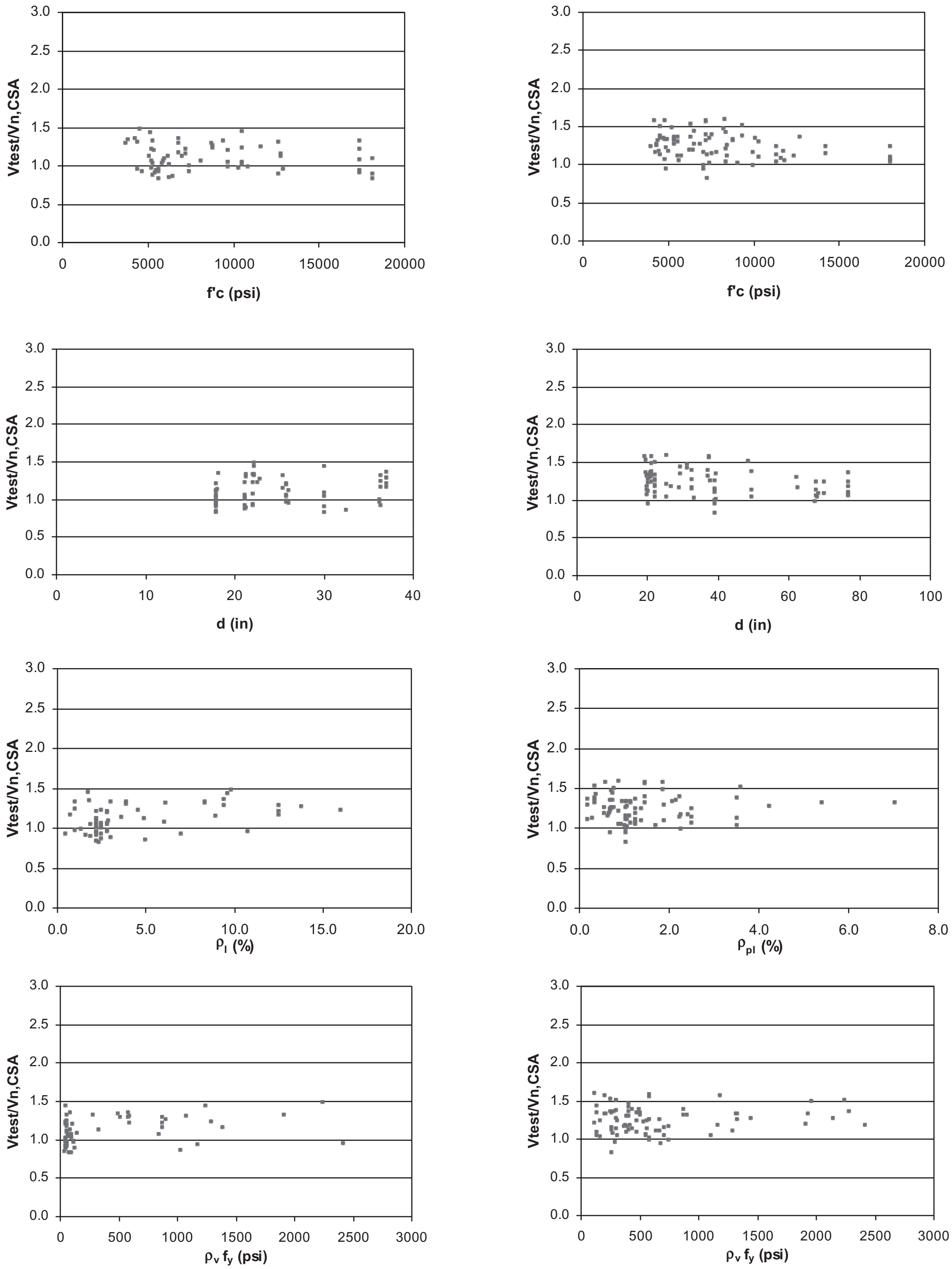
%SR < 0.9 is the percentage of the cases in which the measured strength would be expected to be less than the design strength, taken as 0.9 times the nominal capacity, based on a normal distribution of test data.



(a) 64 RCmembers (b) 83 PCmembers

Figure 23. Comparison of simplified approach predictions and test results.

as simply-supported and continuous members. All members supported a uniformly distributed load and were designed for flexure to satisfy the requirements of the LRFD specifications. The sections selected for shear design of simply-supported members were at "d", 0.1L, 0.2L, 0.3L, and 0.4L from the support. The sections selected for shear design of continuous members were at "d", 0.1L, 0.2L, 0.3L, 0.4L, 0.8L, 0.9L and L-d from the simple support. In order to obtain a range of shear design stress levels and M/V ratios at each of these sections, each member was designed for



(a) 64 RCmembers (b) 83 PCmembers

Figure 24. Comparison of CSA 2004 predictions and test results.

multiple lengths and to support loads that required different levels of flexural reinforcement (50%, 75%, or 100% of the maximum allowable flexural reinforcement). This led to some shear design stress levels larger than those commonly seen in current design but are still admissible by the LRFD specifications. The six different types of members from which the sections were selected are:

1. A 36-inch Deep Simply-Supported Prestressed I-Beam with 7.5-inch Thick Composite Slab

2. A 72-inch Deep Simply-Supported Prestressed Bulb-Tee Girder with 7.5 inch Thick Composite Slab
3. A 78-inch Deep Two-Span Continuous Post-Tensioned Box Girder
4. A 36-inch Deep Simply-Supported Rectangular Reinforced Concrete Beam
5. A 42-inch Deep Simply-Supported T-Shaped Reinforced Concrete Beam
6. A 36-inch Deep Two-Span Continuous Reinforced Concrete Beam

In comparing required amounts of shear reinforcement by the five methods, it is considered particularly important to consider the differences between the amounts required by each design method and the amounts required by using program Response 2000. While the required amount by Response 2000 is not a replacement for experimental test data, this program has successfully proven to provide accurate predictions of the shear capacity for members in the experimental database. Since this program is based on a general behavioral model (MCFT) and not calibrated by this beam test data, it is reasonable to expect that the program will provide similarly accurate estimates of the capacity of members in this design database as it did for the members in the experimental test database. In this use of Response 2000, the appropriate ratio of M/V and level of prestressing was input and then the amount of shear reinforcement was adjusted until the predicted capacity was equal to V_u/ϕ.

As stated in the previous section, most experimental test data is from small-sized member tests and nearly all failures have occurred near simple supports. Thus, an evaluation of the proposed changes in sections away from the support is required. For this evaluation, the required amount of reinforcement, $\rho_v f_y$, required by 5 methods (AASHTO Standard Specifications), the LRFD Sectional Design Model (LRFD), the proposed simplified provisions (p, change proposal 1), the CSA Method (CSA, change proposal 2) and Response 2000 (Response 2000)). This required amount of shear reinforcement by each design method is given by Equation 34.

$$\rho_v f_y = (V_u - V_s / A_{cv}) \qquad \text{(Eq. 34)}$$

where A_{cv} is the area of concrete resisting shear.

These required amounts are compared for all 473 design cases in a series of tables, charts and plots in Appendix H. All of the design sections in this database are representative of the types of situations to which the proposed simplified provisions would be applicable. In reviewing the results, the researchers were particularly interested in identifying those conditions under which any of the methods were either unconservative or particularly different than other provisions. For each of the prestressed members, the total number of design sections is further divided into web-shear regions, transition regions, and flexure-shear regions. A transition region is where both web-shear cracking and flexure shear cracking could be expected to occur and which was numerically considered to be when V_{cw} was greater than V_{ci} but $M_u > M_{cr}$. In many of the design cases, minimum shear reinforcement was required. In these cases, the ratio is shown by a hollow symbol. Otherwise a solid symbol is used.

Table 6 summarizes the results of the comparisons by presenting the mean and COV of the ratios of the requirement strength of shear reinforcement by each of the four design

TABLE 6 Comparisons of ratios of required strength of shear reinforcement by four design methods to required strength determined by program R2K

		All	I-beam	Bulb-Tee	RC Rect	RC T-Shape	Box Sect	RC
			SS*	SS*	SS*	SS*	Cont*	Cont*
STD	mean	1.26	1.36	1.14	1.50	1.55	0.77	1.11
	c.o.v.	0.31	0.28	0.29	0.24	0.15	0.26	0.22
	5% fractile	25%	17%	34%	8%	1%	87%	33%
LRFD	mean	1.42	1.98	1.48	1.23	1.21	1.06	0.85
	c.o.v.	0.37	0.33	0.30	0.24	0.12	0.14	0.24
	5% fractile	21%	7%	14%	22%	7%	34%	77%
CSA	mean	1.40	2.10	1.43	1.24	1.18	0.88	0.90
	c.o.v.	0.48	0.42	0.45	0.20	0.12	0.25	0.19
	5% fractile	28%	11%	25%	17%	10%	71%	72%
Prop	mean	1.57	1.83	1.38	1.78	1.78	1.33	1.35
	c.o.v.	0.23	0.20	0.18	0.22	0.16	0.08	0.21
	5% fractile	6%	1%	6%	2%	0%	0%	11%

* Note: SS – Simply-Supported, Cont - Continuous

methods to the required strength determined by program Response 2000. In this table, only the results from design cases in which all methods required greater than minimum shear reinforcement are considered. From this table, the following observations are made:

1. The proposed simplified provisions provided the most conservative estimate of the required amount of shear reinforcement with a mean ratio to the Response 2000 requirements of 1.57. It also had the smallest coefficient of variation of the four design methods of 0.23. If a normal distribution of data is assumed and a strength reduction factor of 0.9 is applied, then it would be expected that in only 6% of cases would sections be under-reinforced relative to the amount of shear reinforcement required by Response 2000. For each of the six design cases, the proposed simplified provisions were conservative.
2. The AASHTO Standard Specifications had the lowest mean reinforcement requirement ratio of 1.26. When coupled with a COV of 0.31, this suggests that in 25% of the cases sections would be under-reinforced relative to the amount of shear reinforcement required by Response 2000. The Standard specifications were found to be particularly unconservative for the design of the continuous box beam and somewhat unconservative for the design of the continuous RC beam and Bulb-Tee girders.
3. The LRFD Sectional Design Model and CSA methods had similar mean reinforcement requirement ratios for most of the design cases. This was somewhat expected given that the relationships for β and θ were also derived from the MCFT using the longitudinal strain at mid-depth and similar assumptions as described in Section 2.1.3 and justified in the paper by Bentz (*51*). The CSA method was somewhat less conservative for continuous members. For the LRFD Sectional Design Model, the mean reinforcement ratio for all members is 1.42 with a COV of 0.37 suggesting that in 21% of cases sections would be under-reinforced relative to the amount of shear reinforcement required by Response 2000. For the CSA method, the mean reinforcement ratio for all members is 1.40 with a COV of 0.48 suggesting that in 28% of cases sections would be under-reinforcement relative to the amount of shear reinforcement required by Response 2000.

If the results from Response 2000 are perfectly correct, then only the proposed simplified provisions come close to satisfying the general design philosophy that less than 5% of designs are unconservative.

3.6 EFFECT OF CHANGE PROPOSALS ON DESIGN PROCESS

This section is concerned solely with the effect of the change provisions on design for engineers. It addresses the effort required by the designer, the range of structures that can designed by the provisions, and the use of provisions for evaluating the condition or strength of existing structures in the field.

Two change proposals were presented and current designers may be using the shear design provisions of either the AASHTO Standard or LRFD specifications. Thus, there are four possible changes that a designer can make:

- AASHTO Standard Specifications → LRFD Modified Sectional Design Method (CSA Method)
- AASHTO Standard Specifications → LRFD Proposed Simplified Provisions (Modified Standard)
- LRFD Sectional Design Model → LRFD Modified Sectional Design Model (CSA Method)
- LRFD Sectional Design Model → LRFD Proposed Simplified Provisions (Modified Standard)

The effect of each of these changes on shear design is discussed below.

3.6.1 AASHTO Standard Specifications → LRFD Modified Sectional Design Method (CSA Method)

Differences in the design by the AASHTO Standard Specifications and LRFD Sectional Design Model were presented in Section 1.1.3. Because the CSA Method is only a simplification to the LRFD Sectional Design Model, all of these differences remain the same except item vii as the design procedure by the CSA method is non-iterative. Based on the use of the CSA and AASHTO standard method in preparing the design examples and in calculating the required amounts of shear reinforcement in the design database, it is considered that the CSA method is slightly easier to execute than the AASHTO standard method. It should be reemphasized that the CSA Method enables a section to be designed for a much higher shear design stress than permitted in the AASHTO standard Method. Furthermore, the CSA Method is a comprehensive design approach capable of designing a section for shear also subjected to the actions of axial load, moment, and prestressing and that it is derived from a complete behavior model for shear. By contrast the AASHTO standard method is an empirical approach for the design of prestressed and non-prestressed flexural members justified by a fit of design equations with experimental test data.

3.6.2 AASHTO-Standard Specifications → LRFD Proposed Simplified Provisions (Modified Standard)

Given that many states have not yet switched to using the LRFD Bridge Design Specifications, many designers probably will choose to use the proposed simplified provisions because they are more similar in structure to the AASHTO Standard Specifications than is the Modified LRFD Sectional Design Model (CSA Method). The equation for V_{cw} in the simplified proposed provisions is somewhat different from that in the Standard specifications in that it can also be used for partially prestressed members and it produces somewhat lower estimates of V_c. The larger difference between the methods is that the proposed simplified provisions introduce the use of a variable angle truss model.

3.6.3 LRFD Sectional Design Model → LRFD Modified Sectional Design Model (CSA Method)

The CSA Method was developed to provide a simplified design approach for the entire class of members for which the LRFD Sectional Design Model was developed. The equations for β and θ replace the use of more complicated tables and the new expression for ϵ_x reduces the dependence on the angle Theta making the design process non-iterative. These changes greatly simplify the design process. Although evaluating the capacity of a structure is simplified by the CSA method, it is still iterative as ϵ_x is a function of the angle θ.

3.6.4 LRFD Sectional Design Model → LRFD Proposed Simplified Provisions (Modified Standard)

This will be a more significant switch than going from the LRFD Sectional Design Model to CSA Method because it is returning to the more traditional approach of calculating V_c from the diagonal cracking strength and using an approach justified purely by experimental test data. The proposed simplified provisions are not a comprehensive design approach and thus more limited in what they can be used to design. Since the CSA Method and the proposed simplified provisions are similarly easy design procedures, the designer is more likely to use the simplified proposed provisions only if the outcome leads to more acceptable levels of required shear reinforcement.

3.7 SAFETY AND ECONOMY OF STRUCTURES DESIGNED BY SIMPLIFIED PROVISIONS

In this section the influences of changes in the minimum shear reinforcement requirements, the maximum shear stress limit, and the required amounts of shear reinforcement are examined and evaluated using both experimental test data and design case examples.

3.7.1 Minimum Shear Reinforcement Requirements

Both change proposals require the use of the same minimum shear reinforcement requirements as does the current LRFD Sectional Design Model. Therefore, designers and owners switching from the AASHTO standard method to the LRFD specifications may be required to include additional shear reinforcement. The justification for this additional required reinforcement is now discussed.

Minimum shear reinforcement is provided to ensure that a member will be able to continue to provide the calculated concrete contribution to shear resistance after diagonal cracking has developed and progressed. The larger the amount and the closer the spacing of the shear reinforcement, the smaller are the crack spacings and crack widths. Provided crack widths are kept sufficiently small, it is considered that shear stresses can be transmitted across cracks. It is this interface shear transfer (or aggregate interlock) that contributes significantly to the concrete contribution to shear resistance at the ultimate limit state and effectively eliminates the depth effect on shear strength that occurs for beams without shear reinforcement. Minimum shear reinforcement requirements consist of three components:

(i) A minimum required strength ($\rho_v f_y$) of shear reinforcement,
(ii) Rules for the spacing of the shear reinforcement, and
(iii) Rules for when it is necessary to use minimum shear reinforcement

In traditional U.S. design practice and in the AASHTO Standard Specifications, the minimum required strength of shear reinforcement is $\rho_v f_y = 50$ psi, and the maximum spacing of reinforcement is d/2. This reinforcement is required for most bridge members when $V_u > \phi V_c / 2$. The exception is wide members, including footings and one-way slabs, where minimum shear reinforcement is not required until $V_u > \phi V_c$.

In recent ACI codes and in the AASHTO LRFD specification, the amount of required minimum shear reinforcement has been increased above traditional levels, as shown in Figure 7, and made a function of concrete strength. This increase was made over the concern that with higher strength concretes both the calculated concrete contribution to shear resistance is larger and that cracks become smoother, providing less interface shear transfer resistance. The experimental test data presented in Figure 20 illustrated that additional reinforcement was required. Because the minimum reinforcement requirement governs over a large percentage of the span in prestressed concrete bridge members, this increased requirement is a significant improvement in safety at a cost in economy.

3.7.2 Maximum Shear Design Stress Limit

As presented in Section 1.1.3, the LRFD Sectional Design Model allows the design of members with shear stresses as large as $0.25f'_c$ which is a very substantial increase over the maximum shear stress permitted in the AASHTO Standard Specifications. Both change proposals suggest imposing a maximum design stress limit of $0.18\,f'_c$. A maximum design stress lower than that in the current LRFD Sectional Design Method is recommended due to the results of shear tests on large bulb-tee girders conducted in NCHRP Project 12-56 in which the funneling of diagonal compressive stresses to the support was found to substantially magnify the diagonal compressive stresses which then led to compressive failures at loads lower than LRFD predicted capacities.

For designers and owners still using the AASHTO Standard Specifications, adoption of either change proposal method will result in a significant increase in the permissible shear design stress. This increase enables the same size section to be used to span longer distances or carry heavier loads and can result in significant improvements in economy.

For designers and owners using the LRFD Sectional Design Model, this change leads to a significant decrease in the design shear stress limit. Since this change has been shown by testing to be required, it is a significant improvement in safety.

3.7.3 Evaluation of Change Proposals Using Experimental Test Results

The evaluation of the design provisions using experimental test data, as summarized in Table 5, and more fully presented in Appendix G, can be used to draw observations on the changes in safety and economy resulting from the adoption of these change proposals. Due to the limitations of experimental test data, this evaluation is restricted to commenting on the safety and economy of the regions near supports for a limited range of member types.

If the test data were representative of bridge girders in practice, then a comparison of the means of the strength ratios, as shown in the first row of Table 5, illustrates that the only potentially significant effects are slight decreases in the required amount of shear reinforcement for non-prestressed members if the CSA method is adopted and a modest increase in the amount of shear reinforcement required for prestressed concrete members if the proposed simplified provisions are adopted.

The relative safety of the provisions can be evaluated by using the means and standard deviations of the strength ratios shown in Table 5 to calculate the percentage of cases for which the measured capacity is expected to be less than the design strength. A resistance factor of 0.9 and a normal distribution are used in calculating the percentages given in the bottom row of Table 5. The results illustrate that switching from the AASHTO standard method results in a modest increase in safety for reinforced concrete members. All four methods had a very similar and very acceptable level of safety for prestressed concrete members.

3.7.4 Evaluation of Change Proposals Using Design Cases Examples

In order to evaluate the expected safety and economy for regions away from supports and for members not well represented in the experimental test database, it is useful to compare the required strengths of shear reinforcement for a large number of design cases by the four design methods and Response 2000. These design cases covered some design sections over the length of prestressed and non-prestressed members, simple and continuous structures, members with rectangular and I- or T-shaped cross-sections, and members designed to a different percentage of feasible flexural capacity. The required amounts of shear reinforcement are summarized in Table 6.

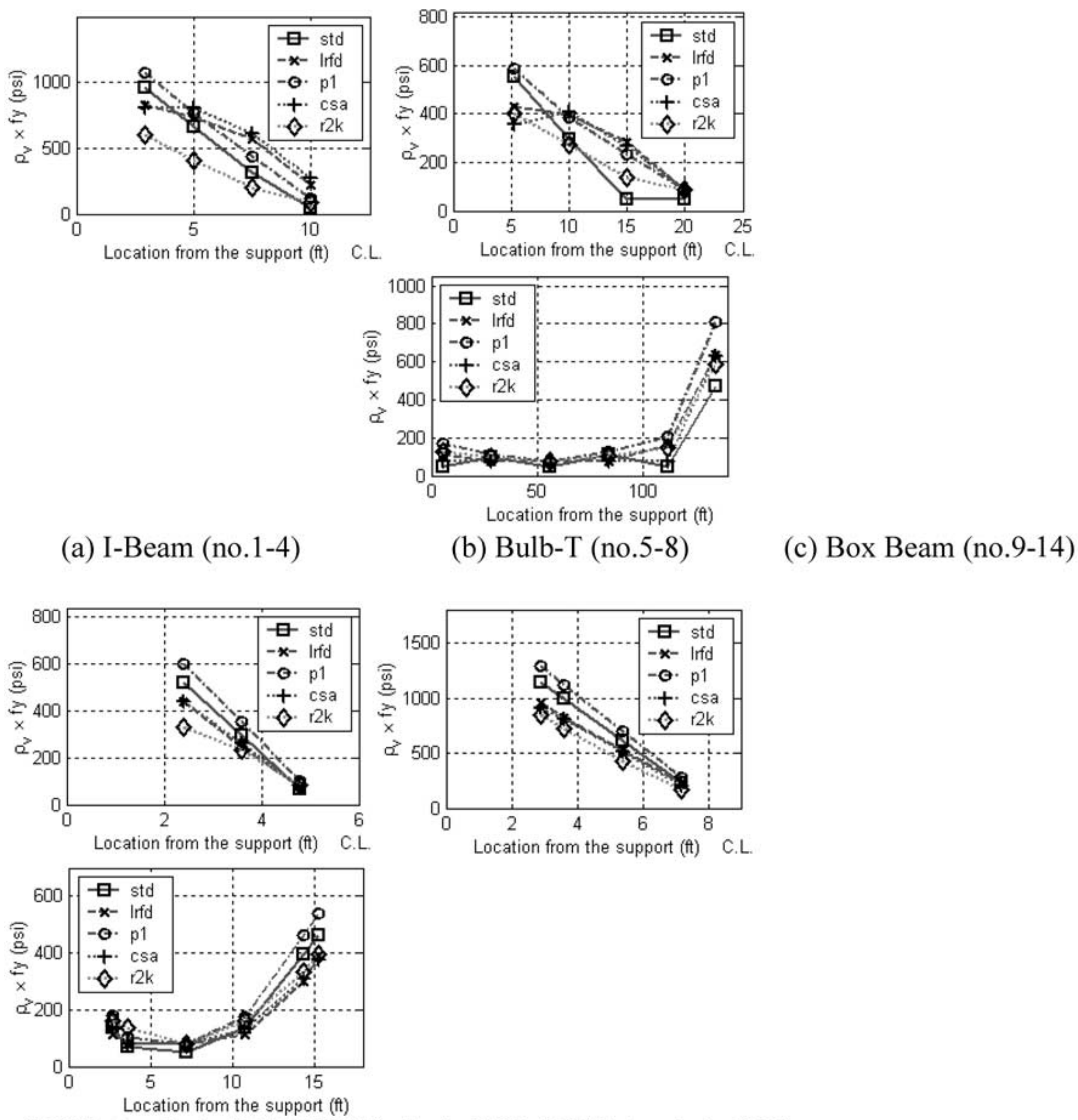
If the predictions of Response 2000 are reasonably accurate and the design database representative, then a comparison of the relative amounts of required reinforcement by each design procedure and Response 2000 is a good measure of the accuracy of each design procedure. The relative values of the means of the requirement ratios in Table 6 illustrate that the procedures, in order of increasing economy, are the AASHTO Standard Specifications, the CSA (change proposal 2), the LRFD Sectional Design Model, and the proposed simplified provisions (change proposal 1). Of course economy and safety must be examined together. For all methods but the proposed simplified provisions, the actual capacity would be expected to be less than the design strength in about 25 percent of cases. As discussed in Section 3.5, all methods but the simplified proposed specifications were particularly unconservative for continuous members. In only about 6% of the cases are the proposed simplified provisions found to be unconservative.

It is also useful to examine where the largest differences are between required amounts of shear reinforcement by the different design methods and to assess whether or not these differences are justified. For this examination, the results for a selection of the design cases is shown in Table 7 and plotted in Figure 25. Additionally, a comparison of the required amounts of shear reinforcement for the eight complete design examples is summarized in Table 8. These results illustrate that some of the largest differences, particularly as a fraction of each other, are in transition zones (between web and flexure-shear regions) and in flexure-shear regions; see cases 3, 4, 7 and 8. In these cases, the AASHTO standard method is the least conservative of the approaches and sometimes quite unconservative if the predictions of Response 2000 are accurate. The proposed simplified provisions are always conservative while the CSA method is usually conservative relative to the Response 2000 values.

Another area of significant differences is near inflection points in continuous members. Inflection point regions are a special transition region and the flexural and shear reinforcement detailing requirements for that region as a function of the inclined cracking that can develop in the region have not been adequately researched. The wide variation in shear

TABLE 7 Comparison of required transverse reinforcement

Design Examples	Required $\rho_v f_y$ (psi)			
	STD	LRFD	Prop.1 (MSTD)	Prop.2 (CSA)
Example 1: SS, Pre., 39-in. deep Box, Span: 95 ft	50	103	72	71
Example 2: Cont., Pre., 72-in. deep Bulb, Span: 110 ft	548	467	492	453
Example 3: Cont., RC, 39-in deep, Square, Span: 15 ft	137	115	171	140
Example 4a: RC, 39-in dia., Circular, Length: 18 ft	0	0	0	0
Example 4b: RC, 12 ft by 12 ft by 3 ft footing	0	0	0	0
Example 5: Cont., Post, 5-ft deep Box, Span 110 ft	50	156	207	110
Example 6: Cont., Multi-post bent cap beam	372	364	319	229
Example 7: SS, Pre., 54-in deep Type IV, Span: 100 ft	188	211	218	157
Example 8: 5-span Precast segmental box girder	100	189	153	146



(a) I-Beam (no.1-4) (b) Bulb-T (no.5-8) (c) Box Beam (no.9-14)

(d) RC Rectangular (no.15-17) (e) RC T-Section (no.18-21) (f) RC Rectangular (no.22-27)

Figure 25. Selected design database.

TABLE 8 Comparisons of selected design database

No.	Description*	Required $\rho_v f_y$ (psi)				
		LRFD	STD	CSA	Modified STD	R2k
1	Pre., SS., I-shape, v = 1.32 ksi, dist=2.9 ft (d)	820	958	801	1065	618
2	v = 1.03 ksi, dist=5.0 ft (0.2L)	729	660	804	762	412
3	v = 0.68 ksi, dist=7.5 ft (0.3L)	570	309	603	431	206
4	v = 0.34 ksi, dist=10 ft (0.4L)	225	50	271	119	89
5	Pre., SS., Bulb-, v = 1.02 ksi, dist=5.3 ft (d)	429	552	359	586	400
6	v = 0.76 ksi, dist=10 ft (0.2L)	394	297	404	382	270
7	v = 0.50 ksi, dist=15 ft (0.3L)	281	50	265	234	140
8	v = 0.24 ksi, dist=20 ft (0.4L)	89	50	89	89	89
9	Post., Cont., Box, v = 0.66 ksi, dist=5.2 ft (d)	115	50	77	167	125
10	v = 0.35 ksi, dist=28 ft (0.2L)	77	100	77	111	99
11	v = 0.02 ksi, dist=56 ft (0.4L)	77	50	77	77	77
12	v = 0.40 ksi, dist=84 ft (0.6L)	77	113	77	125	104
13	v = 0.78 ksi, dist=112ft (0.8L)	164	50	77	206	150
14	v = 1.07 ksi, dist=135 ft (L-d)	637	473	630	808	583
15	RC, SS., Rect., v = 0.67 ksi, dist=2.4 ft (d)	438	518	440	600	330
16	v = 0.45 ksi, dist=3.6 ft (0.3L)	246	295	264	352	230
17	v = 0.22 ksi, dist=4.8 ft (0.4L)	77	68	77	101	83
18	RC, SS., T-, v = 1.29 ksi, dist=2.9 ft (d)	953	1139	908	1290	850
19	v = 1.15 ksi, dist=3.6 ft (0.2L)	823	991	805	1126	720
20	v = 0.76 ksi, dist=5.4 ft (0.3L)	522	607	515	700	430
21	v = 0.38 ksi, dist=7.2 ft (0.4L)	235	226	204	276	165
22	RC, Cont., Rect., v = 0.29 ksi, dist=2.7 ft (d)	112	138	131	178	158
23	v = 0.23 ksi, dist=3.6 ft (0.2L)	77	71	80	104	138
24	v = 0.03 ksi, dist=7.2 ft (0.4L)	77	50	77	77	77
25	v = 0.29 ksi, dist=10.8ft (0.6L)	110	135	129	176	158
26	v = 0.55 ksi, dist=14.4ft (0.8L)	297	393	309	462	333
27	v = 0.62 ksi, dist=15.3ft (L-d)	391	460	377	537	396

* Note

Pre.: Prestressed, Post.: Post-tensioned

SS: Simply Supported, Cont.: Continous

I-Shape: I-Shape Section, Bulb: Bulb-T Section

v: design shear stress calculated from $v = \dfrac{V_u - 0.9 \times Vp}{\phi b_v d_v}$

dist: distance from support (ex: d – effective depth, 0.1L - 10% of span length)

reinforcement requirements for inflection point regions, as shown by the results for Number 5 in Table 8, effectively illustrates this point.

3.8 UTILIZATION OF NCHRP PROCESS 12-50

Code developers wish to be able to accurately assess the effect of changes in specifications on the safety and economy of the transportation infrastructure. NCHRP Process 12-50 is an infrastructure of tools for making this type of evaluation by providing access to databases of bridge structures. It consists of three main components: (1) Generating input data for various design programs; (2) Collecting and Displaying the output on a common viewer (post-processing); and (3) Creating access to the archived data through the World Wide Web. The NCHRP report on Process 12-50 provides sample codes written in Visual Basic, Visual C++, and FORTRAN that developers can use to generate input data. It also presents a common viewer program that enables the developers to find problems with their codes or programs by comparing results. Process 12-50 uses XML format, in which output data can be distributed.

Process 12-50 was used in this project to input the design database. The existing NCHRP bridge database was reviewed. However, it was found to not contain the range in member types and shear design stress levels suitable for comparing required amounts of shear reinforcement by different design code provisions. In addition, the information in the NCHRP database was not sufficient for shear design calculations in accordance with other than AASHTO LRFD specifications. Thus, a new set of members, suitable for shear calculations, was developed. The Design Database introduced in Appendix H includes those newly created members.

In addition, creating the database was achieved either manually or by using spreadsheets to encompass the large range of design stress levels or sectional shapes being used in practice. The NCHRP Viewer program can be used to display the output including the required amount of shear reinforcement. This post-process program allows display of basic member data and comparison of the results obtained by various design methods. The Design Database is available in the accompanying CD. Directions for viewing the data, as well as the description of the database, are provided in Appendix I.

CHAPTER 4

CONCLUSIONS AND SUGGESTED RESEARCH

4.1 CONCLUSIONS

The conclusions presented here go beyond the scope of Project NCHRP 12-61. These conclusions principally identify deficiencies that the research team considers will remain, even if the change proposals recommended in this document are adopted. The conclusions are presented in four categories: basis of design provisions; role of experimental research and field experience; role of design database and numerical tools; and differences in shear design provisions.

4.1.1 Basis of Design Provisions

1. Although researchers may agree about the components that contribute to shear resistance, there is considerable disagreement about the relative magnitude of these contributions, the factors that influence these contributions, and their significance for different design conditions.
2. The diagonal cracking strength is not a measure of the concrete contribution at ultimate for members with shear reinforcement. Thus, provisions in which V_c is related to the diagonal cracking strength for members with shear reinforcement are purely empirical and need to be validated by comprehensive test data.
3. The parallel chord truss model provides a direct means of calculating the contribution of shear reinforcement to shear capacity. That contribution can be calculated as the yield strength of a stirrup times the number of stirrups in one leg of the idealized truss (see Equation 35 where the angle θ is the angle of diagonal compression in the idealized truss relative to its longitudinal axis); however, there is no agreement on how to calculate the angle θ. In ACI 318-05 and the AASHTO Standard Specifications, θ is assumed to be 45 degrees. In the Eurocode, in which the shear design provisions are partially based on plasticity theory, the angle can be selected by the designer to be as low as 18 degrees or that when the diagonal compressive stress reaches a limit equal to about 60 percent of the concrete compressive strength. In the LRFD Sectional Design Model, this angle is calculated using the MCFT and using the longitudinal strain at mid-depth. In the proposed simplified provisions, this angle is calculated in the web-shear region using Mohr's circle of stress to find the angle of diagonal cracking. It has further been argued by researchers that the number of stirrups that should be considered to form a given leg in the idealized truss should be $d_v\cot(\theta)/s - 1$ because diagonal cracks often form from the top of one stirrup to the bottom of another.

$$V_s = \frac{A_v f_y d_v \cot(\theta)}{s} \qquad \text{(Eq. 35)}$$

4. For members with shear reinforcement, the equation developed for V_c in provisions must account for the rules used for evaluating the angle of diagonal compression. For example, in current U.S. practice where the angle θ is assumed to be 45 degrees for prestressed concrete structures, the calculated contribution of the shear reinforcement is less and V_c can afford to be larger than in other codes such as the LRFD Sectional Design Model where θ may be as low as 18 degrees.
5. For members without shear reinforcement, there is a large debate about how to evaluate the contribution of the concrete at the ultimate limit state. Some researchers argue that it should be the load required to form or propagate a diagonal crack. Others suggest that it should be based on the shear-slip resistance of the diagonal crack while others suggest that it is best evaluated by considering the shear force that can be transmitted in the uncracked compression zone. Regardless of which method is used, there is a significant depth effect in shear for members without transverse reinforcement and little depth effect for members with shear reinforcement; members without shear reinforcement and with a unit depth of three can fail at one half the stress of a geometrically similar member with a unit depth of one. However, there is significant debate over the types and sizes of members for which the depth effect in shear must be considered.

4.1.2 Role of Experimental Research and Field Experience

1. What researchers have tested and continue to test in laboratories is not representative of what is built using

design codes. The most typical laboratory test structures are small (less than 15 inches deep), have rectangular cross sections, do not contain shear reinforcement, are simply supported, are stocky, are loaded by point loads over short shear spans, and are supported on bearings positioned underneath the member. In addition, nearly all members are designed so that shear failures occur near supports. By contrast, a large fraction of the bridge members in the field are large, continuous, have top flanges, are subjected to uniformly distributed loads and are built integrally at their ends into diaphragms or piers. In addition, members in the field are designed for shear over their entire length and away from simple supports where there can be substantial effects of flexure on shear capacity.
2. Because most code provisions are ultimately validated by test data, and because most members in the experimental database do not represent what is built with provisions, there is great uncertainty about the safety, economy, and validity of these provisions for most shear design regions in most structures. A particular case in point is the region of contraflexure in a continuous beam. The wide spread in the shear requirements found in Example 5 of Appendix J for this region for different provisions is a direct reflection of the uncertainty of the safety of those provisions for that region.
3. Most experimental researchers fail to collect or report detailed information about the performance of the test members before failure. This information consists of material properties, member or test set-up geometry, crack patterns and widths, stirrups strains, measured diagonal compressive stresses, and shear deformations. Thus, most tests are not useful for assessing the condition of members under service loads or for evaluating the accuracy of complete behavioral models for resistance.
4. It is difficult to judge the overall safety of design code provisions from field experience because most structures in the field have redundant load paths, additional load resisting elements not accounted for in design, and are unlikely to be subjected to loads approaching their factored design loads. Further, many of the difficulties observed in the field are dominated by an interaction of deterioration, environmental and repeated loading effects.

4.1.3 Role of Design Database and Numerical Tools

1. In this project, a comparison was made of the required strength of the shear reinforcement ($\rho_v f_y$) by four different design approaches and by Response 2000 for about 500 design cases. The research team chose these design cases in an effort to capture the range in design cases for which shear design specifications would be applied. The results of these comparisons were considered useful for evaluating the safety and economy of design provisions, particularly for the types of structures and regions for which there is little experimental test data.
2. Although these comparisons were useful, the dataset selected by the research team may not well represent the types and frequency of structures to be designed by provisions.
3. The assessment of the effect of the proposed changes on bridge design practice would also have been more reliable if the design database well represented the types and frequencies of structures to be designed by these provisions.
4. The results would also have been more useful if there were more computational tools (other than Response 2000) for predicting the required strength of shear reinforcement in these design cases.
5. The NCHRP Process 12-50 helped establish a framework for addressing the three foregoing shortcomings, but the design example database has yet to be populated with representative types and frequencies of members designed with provisions.

4.1.4 Differences in Shear Design Provisions

1. There is a wide variation in the forms of shear design specifications used in different influential codes of practice such that the amount of shear reinforcement required by one code may be two to three times that required by another code for the same section and factored sectional forces.
2. There remains considerable disagreement in codes of practice on the minimum required amount of shear reinforcement and when this minimum reinforcement is required. There is a factor of about 2 in the minimum required amounts of shear reinforcement. Some codes required minimum shear reinforcement when the factored design shear force exceeds one half of the design strength provided by concrete alone while others do not require minimum shear reinforcement until the factored design shear force exceeds this design strength. The types of members exempt from more stringent minimum shear reinforcement requirements also vary.
3. There is a large variation in the maximum allowable shear stress by different codes of practice. The difference can be a factor of two and one-half between the AASHTO Standard Specifications and LRFD Sectional Design Model.
4. The depth effect in shear that has been strongly observed in members without shear reinforcement is captured in some codes of practice by making the allowable design stress a function of the overall depth of the member. The depth effect can change the allowable shear design stress by more than a factor of two for different sized members.

5. The bases of shear design provisions include experimental test data, the equilibrium condition of members in the ultimate limit state, and comprehensive behavioral models for capacity.

4.2 RECOMMENDED RESEARCH

Several significant shortcomings in shear design practice were presented above. Of particular concern are the large differences between codes of practice in the required amount of shear reinforcement, the maximum allowable shear design stress, minimum shear reinforcement requirements and how the depth effect in shear is addressed. Equally important are the lack of experimental validation for practical design cases and the lack of understanding and consensus on how structural concrete members carry shear. To address these concerns, the following research efforts are recommended.

1. Process 12-50 should be populated with the range and frequency of members commonly designed using the AASHTO Bridge Design Specifications. This will enable a much more accurate assessment of the effect of proposed changes to the specifications on the safety and economy of the nation's current and future inventory of bridge structures.
2. A web-based national database of shear test results should be established. This can be used by researchers and funding agencies to understand where research is most needed. It will also ensure that the resources spent in conducting experiments are used when it is time to revise code specifications. Process 12-50 developed some of the structure for creating this archive, but additional coordination and efforts are required. An example of an experimental test archive being developed by the earthquake engineering community as part of a large new initiative by the National Sciences Foundation is available at http://nees.org
3. Because provisions principally are validated by test data, shear tests are needed on the types of members built with provisions but for which there is little or no test data. This missing population principally consists of large members, continuous members, members supporting distributed loads, and members that fail in regions other than adjacent to a support.
4. Where testing of members is not practical, suitable numerical approaches should be used to obtain the best possible estimates of shear capacity and behavior.
5. Standards for shear testing should be developed so as to ensure that material test data and the detailed structural Response 2000 are measured in such ways that they can enable the evaluation of structures under service load levels, as well as ultimate load levels, and the validation of numerical methods and behavioral models for analyzing Response 2000.
6. Although the depth effect in shear has been well demonstrated, the range of applicability of this effect and its relation to minimum shear reinforcement requirements needs to be better understood. It is likely that depth rather than concrete compressive strength is a better parameter for establishing minimum shear reinforcement requirements.

NOTATION

The notation conforms to that of Section 5.3 of the AASHTO-LRFD Specifications; however, some new symbols are needed to describe terms used in various models and in several instances modifications are needed to the basic AASHTO-LRFD definition to better describe subsets of that term. Definitions for new terms and changes are shown in italics.

Main Report

A_c = *area of concrete on flexural tension side of member*
A_{ct} = *area of concrete in tension*
A_{cv} = area of concrete resisting shear transfer
A_g = gross area of *concrete* section
A_{ps} = area of prestressing steel *on flexural tension side of member at ultimate load*
A_s = area of non-prestressed tension reinforcement *on flexural tension side of member at ultimate load*
A_v = area of transverse reinforcement within a distance s
$A_{v,min}$ = *area of minimum required transverse reinforcement*
a_g = *maximum aggregate size*
b = width of compression face of member
b_v = width of interface; web width including adjustment for presence of ducts
b_w = web width
d = distance from compression face to centroid of tension reinforcement
d_v = effective shear depth
E_c = modulus of elasticity of concrete
E_p = modulus of elasticity of prestressing *steel*
E_s = modulus of elasticity of reinforcing bars
f_2 = *stress in direction 2; principal compressive stress*
f'_c = *concrete compressive strength*
f_{c1} = *concrete stress in direction 1*
f_{c2} = *concrete stress in direction 2*
f_{c2max} = *maximum value of concrete stress in direction 2 when there is tension in direction 1*
f_{ck} = *characteristic concrete cylinder compressive strength (EC2 method)*($\approx 0.9f'_c$)
f_{cr} = *concrete stress at tensile cracking*
f_{ct} = *concrete tensile stress*
f_{cx} = *concrete stress in direction x*
f_d = *stress due to unfactored dead load*
f_{pc} = compressive stress in concrete after all prestress losses have occurred either at centroid of the cross-section resisting live load or at the junction of the web and flange when the centroid lies in the flange
f_{pe} = effective stress in the prestressing steel after losses; *compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads*
f_{po} = E_p times locked in difference in strain at ultimate load between the prestressing tendons and the surrounding concrete
f_{ps} = *stress in prestressing steel*
f_{pu} = tensile strength of prestressing steel
f_{sx} = *steel stress in direction x*
f_{sy} = *steel stress in direction y*
f_t = *tensile strength of concrete*
f_v = *shear stress; stress in shear reinforcement; vertical stress*
f_x = *stress in direction x*
f_y = yield strength of reinforcing bars; *stress in direction y*
h = overall thickness or depth of member

I_c = *moment of inertia of uncracked concrete*
j = 1-k/3
K = *coefficient to define prestress effect in ASBI shear strength evaluation method*
k = *coefficient on d to define depth of compression zone for elastic behavior; parameter in EC2 (2003) and DIN shear strength evaluation method*
L = span of member *center to center of supports*
M = *moment*
M_{cr} = cracking moment
M_{max} = *maximum factored moment at section due to externally applied loads*
M_n = nominal flexural resistance *of section*
M_u = *ultimate moment*; factored moment at section
N_u = *factored axial force*
N_v = $V\cot\theta$
n = modular ratio
s = spacing of bars of transverse reinforcement
s_{max} = maximum permitted spacing of transverse reinforcement
s_x = crack spacing parameter
s_{xe} = *crack spacing parameter*
s_z = *crack spacing parameter*
s_{ze} = *crack spacing parameter*
T_{min} = *minimum tensile capacity required for reinforcement on flexural tension side of member at* $d_v\cot\theta$ from design section
V = *shear*
$V_{AASHTO\text{-}LRFD}$ = *shear capacity evaluated using AASHTO-LRFD*
V_{ACI} = *shear capacity evaluated using ACI 318*
V_c = *shear at inclined cracking; nominal shear resistance provided by concrete*
V_{ca} = *shear carried by aggregate interlock*
V_{cc} = *shear in compression zone*
V_{ci} = *shear at flexure-shear cracking*
V_{code} = *nominal shear strength of member as evaluated by a specific code method or procedure*
V_{cr} = *shear carried by residual tensile stresses in concrete*
V_{cw} = *shear at web-shear cracking*
V_d = *shear carried by dowel action; shear force at section due to unfactored dead load*
V_i = *factored shear force at section due to externally applied loads occurring simultaneously with* M_{max}
V_n = nominal shear resistance of section considered
$V_{n,max}$ = *maximum allowable nominal shear capacity*
V_p = component in the direction of the applied shear of the effective prestressing force
V_r = factored shear resistance = φV_n
V_s = shear resistance provided by transverse reinforcement
V_{test} = *shear resistance measured at ultimate capacity in test*
V_u = factored shear force at section
v = factored (*design*) shear stress
v_u = $V_u/b_v d_v$
v_{utest} = $V_{test}/b_v d_v$ or $V_{test}/b_w d$
v_{xy} = *shear stress*
y_t = distance from neutral axis to extreme tension fiber *for uncracked section*
α = angle of inclination of transverse reinforcement to longitudinal axis of member
α_p = *angle between prestressing force and longitudinal axis of member (JSCE 1986)*
β = factor relating effect of longitudinal strain on the shear capacity of concrete, as indicated by the ability of diagonally cracked concrete to transmit tension
β_d = *depth effect parameter in JSCE shear strength evaluation procedure*
β_n = *coefficient to account for prestress and axial load in JSCE shear strength evaluation procedure*
β_p = *coefficient to account for longitudinal reinforcement ratio effect in JSCE shear strength evaluation method*
γ_{xy} = *shear strain*
ϵ_1 = *strain in concrete in direction 1; principal tensile strain*
ϵ_2 = *strain in concrete in direction 2*

ϵ_t = *strain at level of longitudinal reinforcement on tension side of member*
ϵ_x = *strain in direction x; longitudinal strain at mid-depth of section*
ϵ_y = *strain in direction y; strain at yield of reinforcing steel*
θ = angle of inclination of diagonal compressive stress
ν = *parameter determining maximum nominal shear capacity for EC2 method*
φ = resistance factor
ρ_l = *longitudinal reinforcement ratio* = $[A_s + A_{ps}]/b_w d$
ρ_{sx} = *steel ratio for direction x*
ρ_{sy} = *steel ratio for direction y*
ρ_v = ratio of area of vertical shear reinforcement to area of gross concrete area of a horizontal section = $A_v/b_w s$
ρ_w = $A_s/b_w d$

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Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
NASA	National Aeronautics and Space Administration
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation