NUREG/CR-5209

Design Provisions for Tangential Shear in Containment Walls

Prepared by R.G. Oesterle

Construction Technology Laboratories, Inc.

Prepared for
U.S. Nuclear Regulatory
Commission

NUREG/CR-5209
RD

Design Provisions for Tangential Shear in Containment Walls

Manuscript Completed: April 1988
Date Published: August 1988

Prepared by
R.G. Oesterle

Construction Technology Laboratories, Inc.
5420 Old Orchard Road
Skokie, IL 60077

Prepared for
Division of Engineering
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555
NRC FIN B2077

U.S. Nuclear Regulatory Commission

Reports in Concrete Structure Research Series
Related to Tangential Shear

"Analysis of Shear Transfer in Reinforced Concrete with Application to Containment Wall Specimens," P. Leombruni, O. Buyukozturk, J. J. Connor, Massachusetts Institute of Technology, NUREG/CR-1085, October 1979.

"Safety Analysis of Nuclear Concrete Containment Structures," P. Petrina, R. Sexsmith, R. N. White, Cornell University, NUREG/CR-1097, December 1979.

"Shear Transfer in Large Scale Reinforced Concrete Containment Elements Report No. 1," R. G. Oesterle, H. G. Russell, Construction Technology Laboratories, Portland Cement Association, NUREG/CR-1374, April 1980.

"Strength and Stiffness of Tensioned Reinforced Concrete Panels Subjected to Membrane Shear, Two-Way Reinforcing," P. C. Perdikaris, R. N. White, P. Gergely, Cornell University, NUREG/CR-1602, July 1980.

"Strength and Stiffness of Reinforced Concrete Panels Subjected to Membrane Shear, Two-Way and Four-Way Reinforcing," C. H. Conley, R. N. White, P. Gergely, Cornell University, NUREG/CR-2049, April 1981.

"Shear Transfer in Large Scale Reinforced Concrete Containment Elements Report No. 2," R. G. Oesterle, H. G. Russell, Construction Technology Laboratories, Portland Cement Association, NUREG/CR-2450, December 1981.

"Behavioral Model for Reinforced Concrete Panels Under Cyclic Shear," Tsi-Ming Tseng, J. Calvo, O. Buyukozturk, J. Connor, Massachusetts Institute of Technology, NUREG/CR-2451, December 1981.

"Strength and Stiffness of Uniaxially Tensioned Reinforced Concrete Panels Subjected to Membrane Shear," S. I. Hilmy, R. N. White, P. Gergely, Cornell University, NUREG/CR-2788, June 1982.

"Design of Reinforced Concrete Containment Wall Elements Under Combined Action of Shear and Tension," J. Calvo, O. Buyukozturk, J. J. Connor, Massachusetts Institute of Technology, NUREG/CR-3157, February 1983.

"Analysis of Reinforced Concrete Containment Vessels With Nonlinear Shearing Stiffness," C. H. Conley, R. N. White, P. Gergely, Cornell University, NUREG/CR-3255, April 1983.

ABSTRACT

The purpose of the work accomplished in preparing this report was to synthesize results of available research concerning the capacity of cracked reinforced containment walls to transfer tangential shear stresses while in a state of biaxial tension from internal pressurization. A review of experimental work is presented. Results of experimental work indicate that the current ASME-ACI code provisions for tangential shear stress are very conservative.

Recommendations for redefinition and revised use of the terms V_c, "concrete contribution," and V_s "steel contribution" are provided. Results of testing programs are used to formulate revised design provisions for diagonal tensile strength. Significantly higher shear stresses can be allowed without inclined reinforcement. Also, an analytical study based on recent testing programs is used to define a conservative maximum limit for tangential shear stress. The maximum limit is dependent on the relative amounts of orthogonal reinforcement and inclined reinforcement used to provide the tangential shear strength in the containment walls.

While testing programs indicate that significant shear strength is available in cracked reinforced concrete, the testing also demonstrates that shear stiffness reduces significantly after cracking. The need to consider the reduced shear stiffness is discussed. Recommendations for revised design provisions are summarized and design examples are provided.

TABLE OF CONTENTS

LIST OF FIGURES

FIGURE NO. Page

FOREWORD

Concrete has been used extensively for containments and other safety-related structures within nuclear power plants. Because of the importance of these structures, a high degree of reliability is sought in design. Therefore, the U.S. Nuclear Regulatory Commission (USNRC) established a research program to gain improved understanding of the behavior of cylindrical concrete containment vessels. Results were intended to facilitate an improved assessment of the level of reliability being achieved by design practices.

A particular aspect of containment behavior addressed by the USNRC research concerned the capacity of reinforced concrete to transfer shear stress while in a state of biaxial tension. This shear transfer capacity is required in the walls of containments subjected to combined internal pressure and seismic loading. Internal pressure produces membrane or biaxial tension, and seismic loading produces tangential shear stresses in the plane of the containment wall.

Participants in the USNRC research program focusing on tangential shear behavior included Cornell University, Construction Technology Laboratories, and Massachusetts Institute of Technology. Cornell University was involved in testing of intermediate scale models representing elements of containment walls and in analysis of containment behavior under combined internal pressure and seismic loading. Construction Technology Laboratories was involved in testing of large scale models of elements of containment walls. Massachusetts Institute of Technology contributed with development of analytical models and interpretation of testing results. Results of testing and analysis programs accomplished by these three organizations are contained in the reports listed on Page ii of this report.

As the results of the USNRC research program became available, a Task Group on Shear was formed within the ASME/ACI Joint Committee on Concrete Pressure Components-Subgroup on Design. This subgroup is responsible for maintaining the design provision Sections CB and CC 3000 within Section III, Division 2, "Code for Concrete Reactor Vessels and Containments," of the ASME Boiler and Pressure Vessel Code. Members of the Task Group on Shear included J. A. Curtin of Stone and Webster Engineering Corporation, T. E. Johnson and P. Shunmugavel of Bechtel Power Corporation, A. Walser of Sargent and Lundy Engineers, and R. N. White of Cornell University. R. G. Oesterle of Construction Technology Laboratories served as Chairman of the Task Group.

The functions of the Task Group on Shear included synthesis of the results of the USNRC research program, along with other available research information, and formulation of recommendations for revised design provisions for the ASME Code. This report represents the results of the work of the Task Group on Shear as related to design provisions for tangential shear in containment walls. Information contained in this report provided the basis for draft revisions of ASME Code provisions that are currently under review within the ASME/ACI code committees for possible inclusion in the 1988 Winter Addenda to Section III, Division 2 of the ASME Boiler and Pressure Vessel Code. The report contained herein is published as a NUREG/CR Report to facilitate dissemination of the integrated results of the USNRC research program.

DESIGN PROVISIONS FOR TANGENTIAL SHEAR IN CONTAINMENT WALLS

INTRODUCTION

Background

Concrete structures in nuclear power plants have been used extensively since the beginning of the nuclear power industry. Initially, concrete was used for radiation shielding. However, use of reinforced and prestressed concrete structures as pressure containments was started in the 1960's. Although concrete had been used in safety-related structures for many years, use of concrete in pressure vessels was a new concept. The size, shape, and possible stress states in containments produced many unique problems for both design and construction. Because of the importance of containments, a high degree of reliability was sought in solving these problems. This philosophy has sometimes led to cumbersome designs.

A primary example of difficulty is design and construction problems produced by the questioned capacity of concrete to transfer shear stress while in a state of biaxial tension. Concrete containments in the United States are designed to resist a combination of biaxial tension caused by internal pressure, and tangential shear caused by earthquakes. To resist internal pressure, reinforcement is generally placed in an orthogonal pattern of vertical and horizontal bars. Use of the same orthogonal reinforcement to resist earthquake forces requires that tangential shear stresses be transferred across open orthogonal cracks. The shear transfer capacity across the open cracks requires experimental verification.

Review of Experimental Work

The capacity for force transfer across an open crack in reinforced concrete has been the subject of a number of experimental investigations conducted during the past three decades. These investigations can be categorized by the type of specimen used. Specimens have included cracked joints in pavements, predefined cracks in unreinforced concrete, predefined cracks in concrete with internal reinforcement crossing the cracks, and randomly induced cracks in reinforced concrete panels.

Some of the early testing programs were conducted to evaluate the effectiveness of shear transfer by aggregate interlock across open control joints in concrete pavement. Experimental work by Colley and Humphrey[1] included alternating loads on each side of an open joint which simulated a wheel load crossing the joint. Parameters studied were aggregate size and joint opening width. Results indicated that joints with opening widths up to 0.065 in. work initially 80% as effectively as a closed joint. Effectiveness was evaluated by comparison of joint deflections. With cyclic loading, joint effectiveness decreased as a function of increase in joint opening width. As an example, the effectiveness of a joint with a width of 0.035 in. was reduced to approximately 50% of that of a closed joint after 500,000 cycles. It should be emphasized, however, that effectiveness was judged by comparison of deflections across the joint. Therefore, although the results of this study indicated that cyclic load reduced the joint stiffness, results did not necessarily show strength experienced a similar decrease.

Several investigators[2,3,4] have tested unreinforced concrete specimens with predefined cracks restrained in very stiff test frames. The objective was to evaluate aggregate interlock across cracks with constant width. Results of testing by Paulay and Loeber[3] indicated that although the stiffness was definitely decreased as crack widths were increased from 0.005 in. to 0.020 in., the maximum shear stress transferred across the 0.020 in. wide crack was only slightly less than the maximum stress across crack widths of 0.005 in. and 0.010 in. Also, the maximum stress in all specimens was greater than 1000 psi. This is a very high stress for interface shear transfer and not likely to be encountered in real structures. A stress of 1000 psi is higher than allowable by the ACI Building Code.[5] Also, by maintaining a constant width, the crack was essentially provided with an unrealistic "infinite stiffness" for deformation normal to the crack.

To more realistically model the stiffness normal to cracks, specimens with a predefined crack in unreinforced concrete restrained by external rods were used in a number of experimental programs.[6,7,8] Initial crack widths up to 0.030 in. were used. These tests demonstrated that aggregate interlock is an effective means of transferring shear stress across cracked concrete surfaces. The testing by White and Holley[6] is the basis for the current ASME-ACI Code Provision[9] allowing a conservative nominal tangential shear stress of up to 160 psi to be resisted by orthogonal reinforcement across open cracks in containments. However, test specimens with external rods did not accurately model the coupled effects between aggregate interlock and restraint from reinforcement embedded in the concrete across the crack.

Other researchers[10,11,12,13] have used specimens with embedded reinforcement crossing predefined cracks. Testing programs included specimens subjected to reversing load.

Mattock[12] determined that reversing load decreases the strength of the interface shear transfer mechanism to approximately 80% of the monotonic strength. Also, increase in initial crack width decreased shear transfer strength. A specimen with an initial crack width of 0.025 in. showed a strength reduction of approximately 15% compared with a specimen with an initial crack width of 0.015 in. However, the strength of the specimen with an initial crack width of 0.025 in. still exceeded ACI Building Code[5] allowable shear friction strength of 1.4 ρf_y Also, the strength of this specimen was 660 psi. This is a very high stress for interface shear transfer and not likely to be encountered in real structures.

Test programs in References 2 through 13 were conducted to evaluate shear transfer in the plane of one predefined crack with an initial opening or with tension applied perpendicular to the crack. Although these tests contributed greatly to knowledge of the detailed behavior of interface shear transfer, the test specimens modeled a relatively artificial situation. Containment walls contain orthogonally cracked elements resulting from membrane tension caused by pressurization. The capacity of shear transfer mechanism in the concrete under a state of biaxial tension had not yet been verified experimentally. Therefore, current ASME ACI Code provisions[9] still require all but a nominal amount (up to 160 psi) of tangential shear to be resisted by inclined reinforcement. The inclined reinforcement is difficult to fabricate. It also adds significant congestion and inhibits concrete placement.

To provide experimental verification of the behavior of concrete containment walls subjected to biaxial tension and tangential shear forces, test programs have been conducted by the Construction Technology Laboratories (CTL) of the Portland Cement Association[14,15], Cornell University[16,17], and the University of Toronto.[18] Specimens were concrete panels containing internal reinforcement in two or four directions. Cracking was induced at random locations by tensioning the elements. A membrane shear stress in orthogonal directions was simulated in these specimens rather than applying a direct shear stress across one localized plane.

Results from these concrete panel tests and other testing and analysis, conducted primarily in Japan,[19-27] indicate that the current ASME-ACI[9] Code provisions for tangential shear strength are very conservative. Significantly higher shear stresses can be allowed without inclined reinforcement. The purpose of this report is to present recommended design criteria for tangential shear based on available test data.

NOMENCLATURE

The common approach to design for shear in reinforced concrete is to allocate some strength to the "concrete contribution," V_c. The remaining required strength is provided by reinforcement, V_s. The "concrete contribution" consists of shear through a compression zone, aggregate interlock, and dowel action. Although the steel used for V_s has some indirect influence on V_c, because of dowel action, there is no reinforcement directly provided for V_c.

Nomenclature used in the current ASME ACI Code[9] is inconsistent with this approach and also inconsistent within itself. For tangential shear, V_c is defined in the current code as shear force carried by concrete. However, V_c is calculated as the strength provided by orthogonal (meridional and hoop) reinforcement. The "steel contribution," V_s, is provided by inclined reinforcement and no "concrete contribution" is considered. However, for radial and peripheral shear, V_c is a "concrete contribution" in that no reinforcement is required for this portion of shear strength.

Because of inconsistency in terminology, there is some confusion as to what V_c means. It is recommended that nomenclature be redefined to be consistent with other codes and within the ASME-ACI Code(9). Toward this goal, it was recommended by the Task Group on Shear that the following definitions and relationships be used.

V_c = Tangential shear strength provided by concrete

V_{so} = Tangential shear strength provided by orthogonal (meridional and hoop) reinforcement

V_{si} = Tangential shear strength provided by inclined reinforcement

$$V_u = V_s + V_c$$

$$V_s = V_{so} + V_{si}$$

These changes were included in Subgroup on Design Action Item D83-1, Joint Committee Item JC 83-16. This item was passed by the ASME B&PV Committee in November 1983 and included in the Summer 1984 Addenda to the Code.

STRENGTH PROVISIONS

As stated under Review of Experimental Work in this report, it is not likely that a localized plane subjected to only shear stress in one direction and of the magnitude measured in some of the shear test specimens (600 to 1000 psi) would be encountered in a real structure.

Results of panel tests(14-17) demonstrated that the interface shear transfer across open cracks is adequate to resist loads up to a level of stress where diagonal cracking occurs.

After diagonal cracking, a truss[15] mode of shear transfer takes over and orthogonal cracks are closed by the resulting diagonal compression. Testing of reinforced concrete panels indicates the interface shear transfer strength is adequate in specimens subjected to a biaxial tension steel stress up to 90% of specified yield. These specimens were subjected to cyclic shear with initial orthogonal crack widths up to 0.040 in. After diagonal cracking, shear strength is limited by either yield of the reinforcement in a diagonal tension mode or crushing of the concrete from diagonal compression.

Diagonal Tension

All specimens tested by CTL and Cornell[14-17] lost load capacity by yielding of reinforcement across a diagonal crack. Figure 1 shows potential yield planes across the specimens tested by CTL. Using the free-body diagram shown in Fig. 2 the following equilibrium equations for yielding of reinforcement in the weaker of the horizontal or vertical directions are derived.

$$A_s f_y = N + V_{max} \qquad [1]$$

$$\frac{A_s}{bt} f_y = \frac{\frac{N}{A_s}}{\frac{bt}{A_s}} + \frac{V_{max}}{bt}, \qquad [1a]$$

$$v_{max} = \frac{V_{max}}{bt}, \quad \rho = \frac{A_s}{bt}, \quad f_s = \frac{N}{A_s}$$

$$\rho f_y = \rho f_s + v_{max} \qquad [2]$$

$$v_{max} = \rho f_y \left(1 - \frac{f_s}{f_y}\right) \qquad [2a]$$

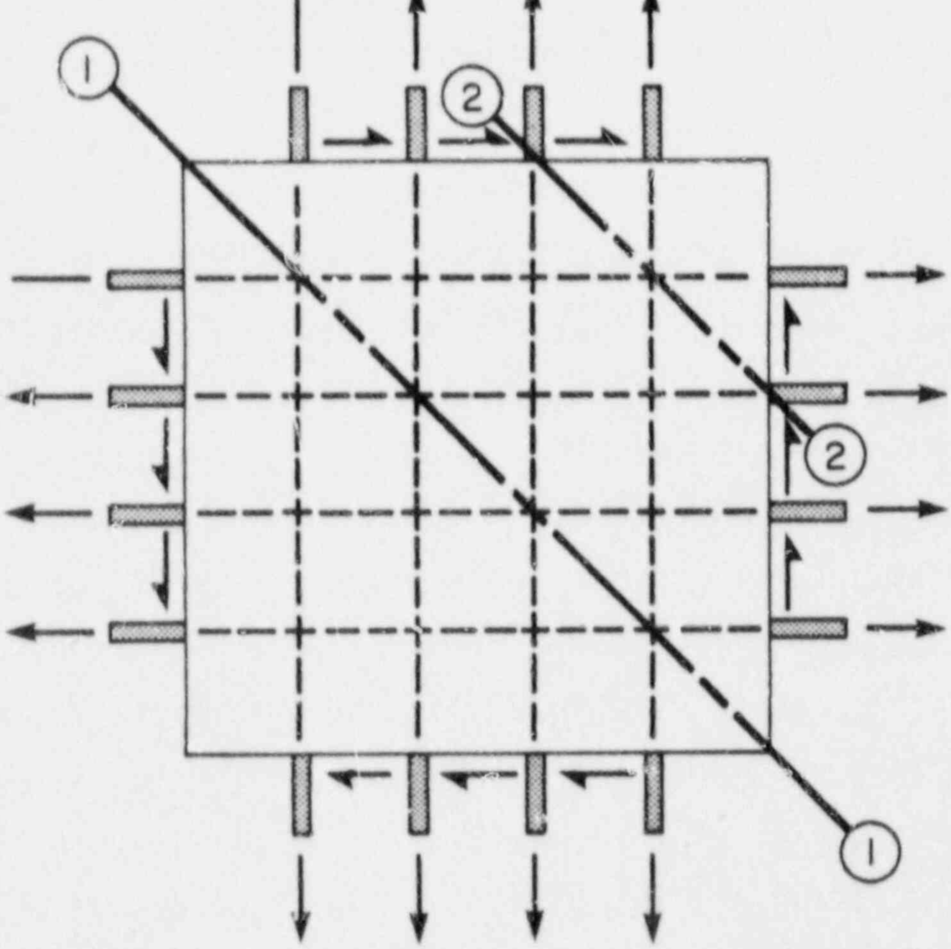


Fig. 1 Planes Through Specimen for Diagonal Tension Strength

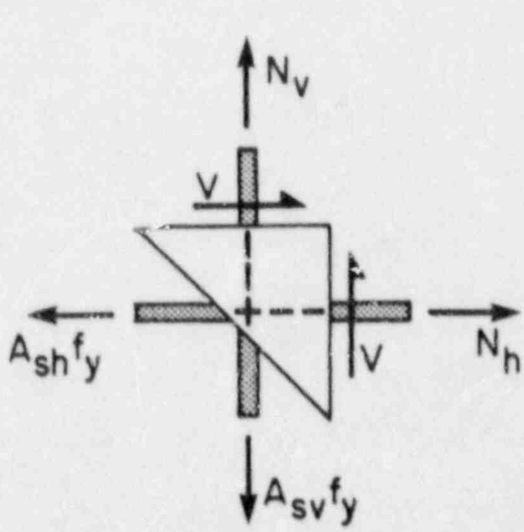


Fig. 2 Free-Body Diagram for Diagonal Tension Equilibrium

The design equations for tangential shear in the current ASME-ACI Code[9] are based on equilibrium and are expressed in a form similar to Eq. [1] with a strength reduction factor of 0.9 for reinforcement yield stress. Figure 3 shows maximum observed shear stress, v_{max}, for specimens tested by CTL and Cornell, versus calculated effective diagonal tension strength. The dashed line represents the simple diagonal tension equilibrium equation in the form of Eq. [2a] with the strength reduction factor of 0.9. Figure 3 shows that there is significant shear strength under biaxial tension. No specimens failed due to sliding shear or dowel splitting. Shear transfer across the orthogonal cracks was adequate for the shear stresses sustained by the specimens up to a diagonal tension failure.

The equilibrium equation with a reduction factor of 0.9 encompasses all but one data point. The first reversing load specimen tested in the large-scale program lost shear capacity at a load lower than that predicted by simple equilibrium. This failure was attributed to stress concentrations in the loading system and should not be taken as indicative of the specimen diagonal tension strength.

Figure 4 indicates a summary of Japanese test results.[28] The diagonal line represents simple diagonal tension equilibrium. As shown in this figure, the equilibrium equation is confirmed by Japanese testing up to a shear stress level of approximately $20\sqrt{f'_c}$. The upper limit for shear stress is discussed under Maximum Strength in this report.

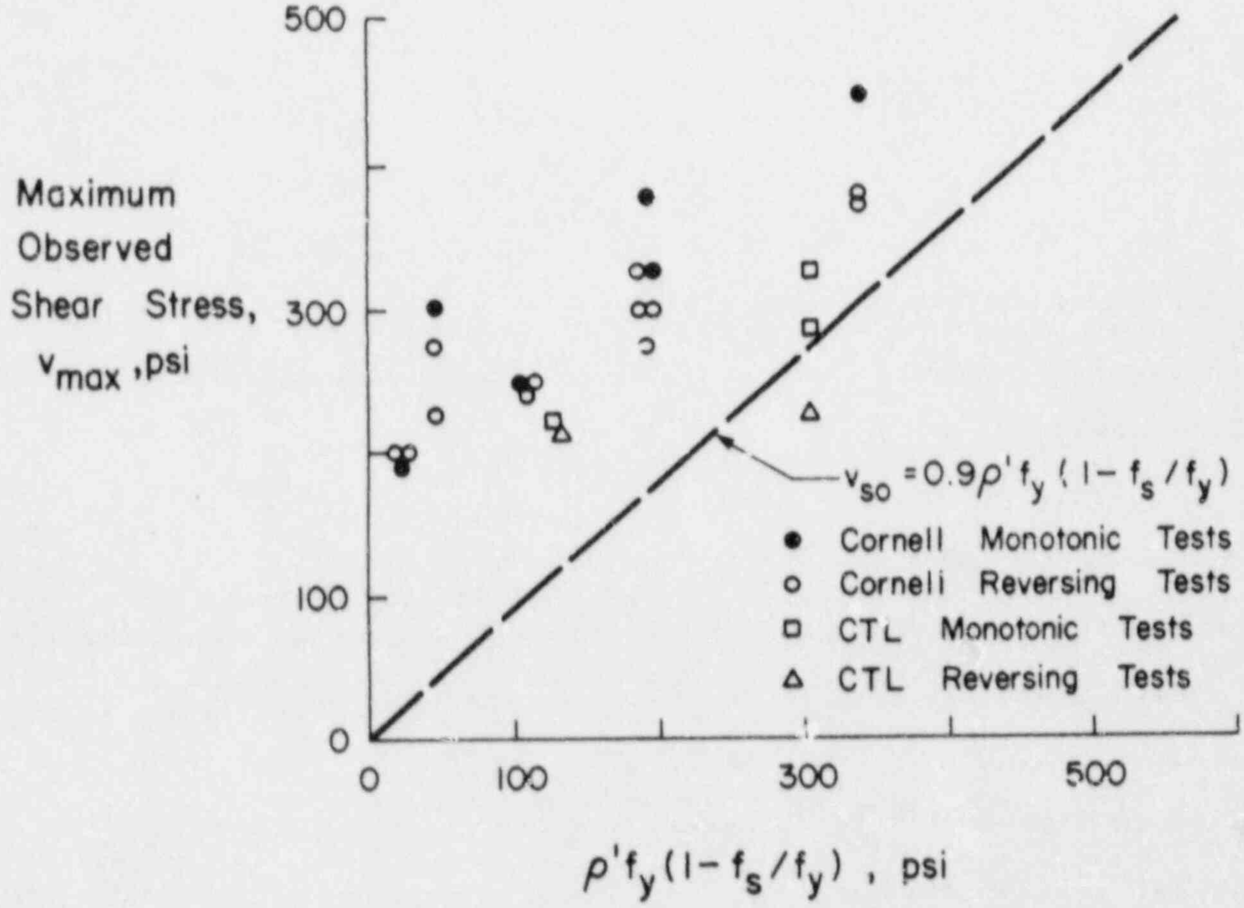


Fig. 3 Diagonal Tension Strength

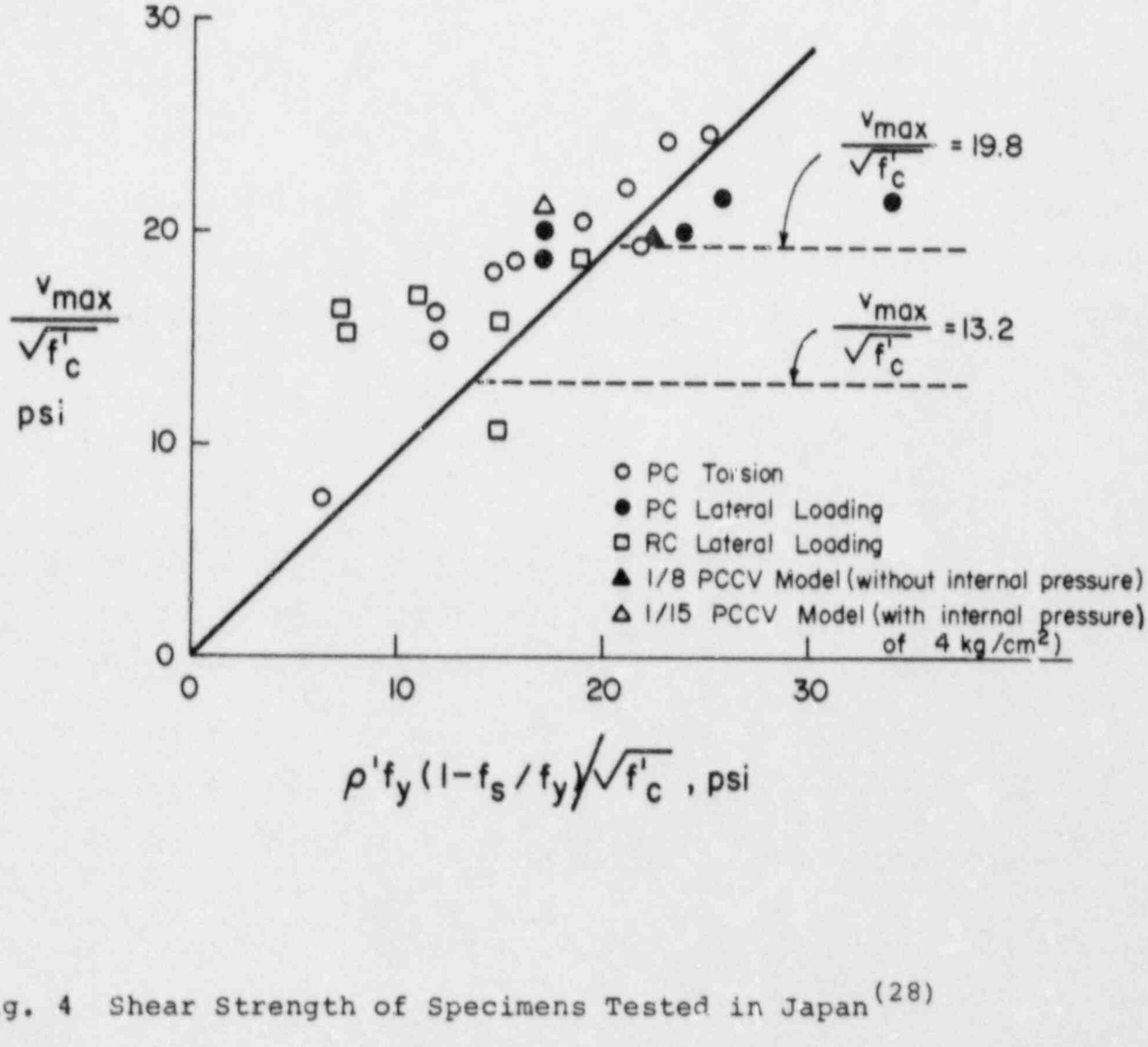


Fig. 4 Shear Strength of Specimens Tested in Japan(28)

It is the recommendation of the Task Group on Shear that the equilibrium equations similar to those currently in the code[9] continue to be used for design of tangential shear reinforcement with the following exception.

1. The term V_{so} replaces the term V_c as discussed in the section on Nomenclature in this report. (Note: this change is included in the Summer 1984 Addenda as previously discussed)

2. The normal and shear forces resulting from earthquake loading be combined with a Square Root of the Sum of Squares (SRSS) approach similar to that in the current code case.[29] This SRSS approach is based on calculations for maximum combination of N and V that can occur anywhere along the circumference of the containment as described in Appendix A of this report.

3. Required area of orthogonal (hoop and meridional) reinforcement, with or without inclined reinforcement, provided for combined membrane and tangential shear strength shall be computed by:

$$A_{sh} + A_{si} = \frac{N_h + \left[N_{hl}^2 + V_u^2\right]^{1/2}}{0.9\ f_y} \qquad [3]$$

$$A_{sm} + A_{si} = \frac{N_m + \left[N_{ml}^2 + V_u^2\right]^{1/2}}{0.9\ f_y} \qquad [4]$$

4. Any combination of orthogonal and inclined reinforcement as required for strength according to Eq. [3] and [4], and as required to control shear deformations, may be used. However, limits must be placed on maximum

shear strength provided by the orthogonal reinforcement V_{so}, and maximum total shear V_u, so that reinforcement will yield before crushing of the concrete in compression can take place. These limits are discussed under maximum strength in this report.

Concrete Contribution

Reinforced Concrete - Currently, there is no "concrete contribution" allowed when designing for tangential shear in reinforced concrete containments under combined membrane tension and shear. The V_c in the current ASME-ACI Code is actually a V_{so} as discussed under Nomenclature in this report. Also, it is noted that recommended Eq. [3] and [4] do not include any "concrete contribution" term.

The difference between observed strength and calculated diagonal tension strength shown in Figs. 3 and 4 represent additional strength due to a "concrete contribution" and strain hardening of reinforcement. Figure 3 suggests that at low levels of $\rho' f_y(1-f_s/f_y)$ (high levels of biaxial tension), there is significant "concrete contribution," V_c. However, it is apparently reduced by reversing loads and by increasing $\rho' f_y(1-f_s/f_y)$ (decreasing biaxial tension). The apparent loss of V_c with decreasing biaxial tension is probably due to the influence of boundary conditions and methods of loading the test specimens. As biaxial tension is decreased, the diagonal tension shear strength increases. Therefore, the level of shear stresses carried by the specimens increases. The boundary conditions and loading methods would have a larger influence on measured strength at the higher levels of shear stress indicating an unrealistic loss of V_c. However, to stay within the limits of experimental data, it is conservatively recommended at this time that $V_c = 0$ for load cases that include membrane tension in reinforced concrete containments.

<u>Prestressed Concrete</u> - In reinforced concrete containments, orthogonal cracks generally occur during the structural integrity test. Therefore, reinforced concrete elements will behave as cracked sections for any further loading. Since V_c has traditionally been associated with the shear force causing diagonal cracking, the fact that a containment is precracked has always been a reason for questioning the "concrete contribution" V_c in reinforced concrete containments. As stated above, although there is some experimental evidence that a significant V_c exists, it is conservatively recommended that $V_c = 0$ for reinforced concrete containment.

A prestressed containment, however, should not crack significantly during the structural integrity test. The structure will behave initially as uncracked for further load. Therefore, it is reasonable to consider a "concrete contribution" V_c for prestressed containments.

Figure 5 [30] demonstrates the difference in shear strength between initially uncracked and initially cracked interface shear test specimens[31]. An additional strength of approximately 250 psi or $4.0 \sqrt{f'_c}$ is observed in the uncracked

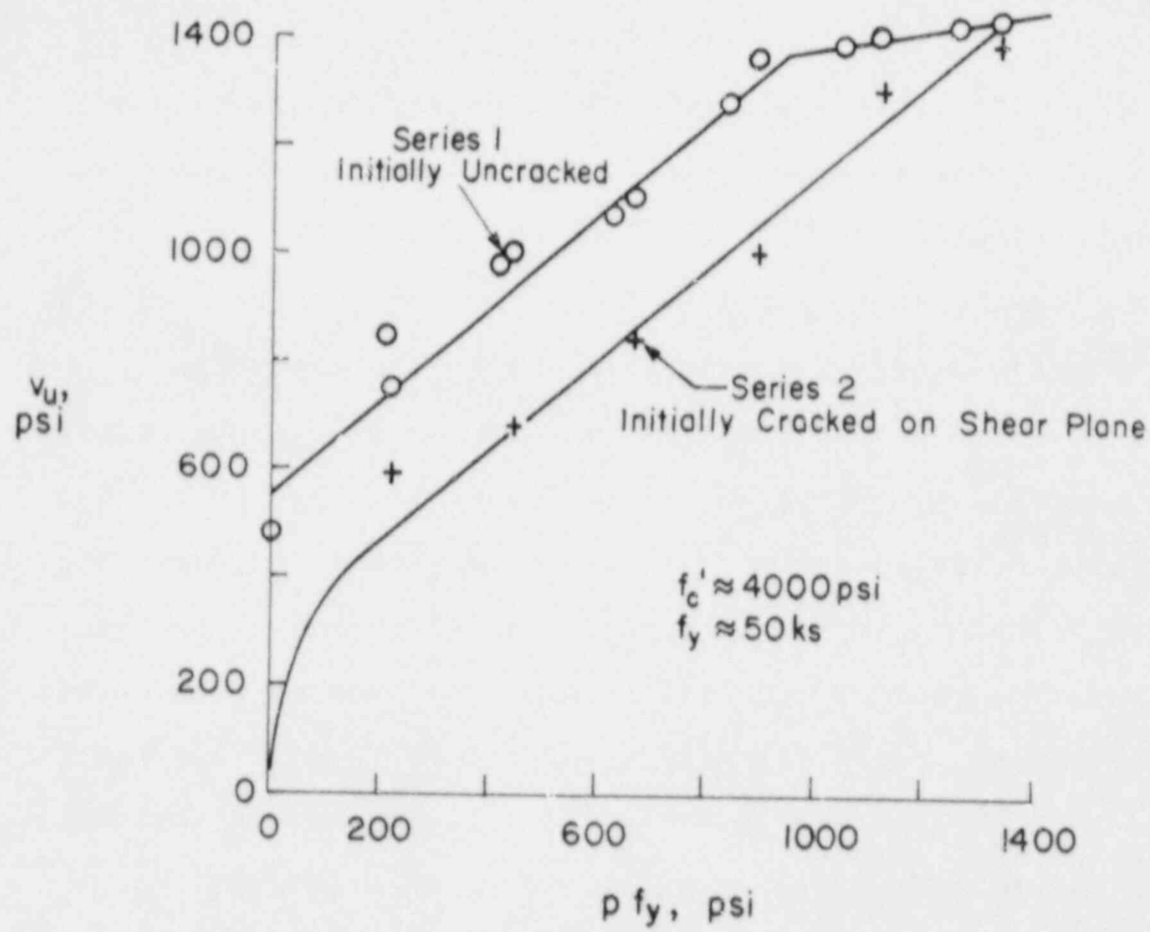


Fig. 5 Variation of Shear Strength with Reinforcement Parameteter ρf_y with and without a Crack along the Shear Plane

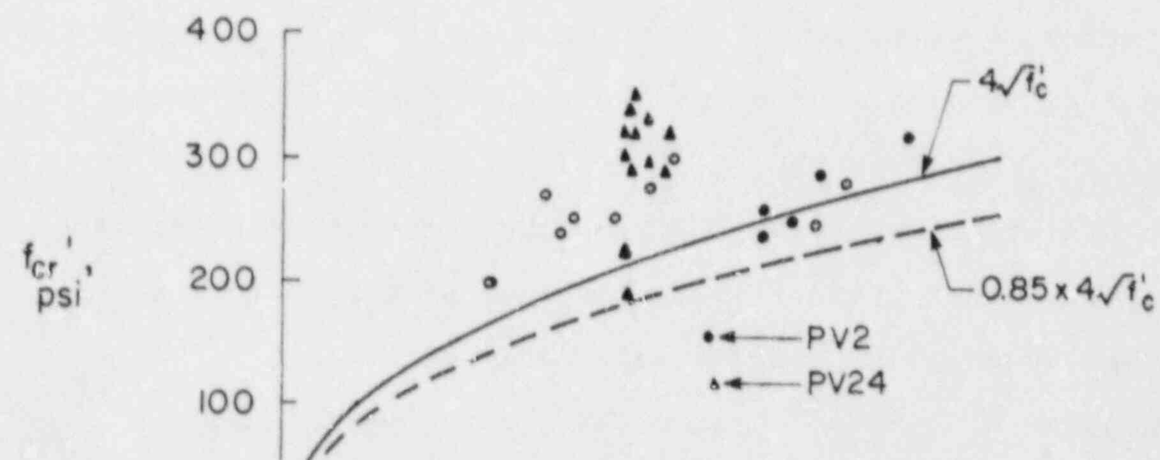


Fig. 6 Principal Tensile Cracking Stress versus Cylinder Strength

specimens. However, as discussed in the introduction of this report, interface shear test specimens model a relatively artificial situation.

The wall panel specimens tested under combined biaxial and shear stresses model the behavior more realistically. Of the panel test programs cited in the Introduction, the initially uncracked specimens tested at the University of Toronto[18] are applicable to behavior of a prestressed containment.

Figure 6 shows the principal tensile stress at cracking, f'_{cr}, in the Toronto specimens. Except for two specimens, PV2 and PV24, all cracking stresses are close to or above the line indicating $4\sqrt{f'_c}$. Specimen PV2 was precracked and specimen PV24 had inadequately consolidated concrete. These test results confirm the diagonal cracking criteria in the ACI Building Code[5].

The criteria for shear in prestressed concrete members contained in Section 11.4.2.2 of the ACI Building Code[5] allows a principal tensile stress of $4\sqrt{f'_c}$ in the web of the members. Therefore, it is recommended that initially a principal tensile stress of $4\sqrt{f'_c}$ be carried by the concrete in prestressed containments. This corresponds to following "concrete contribution" derived from Mohr's Circle:

$$V_c = 4\sqrt{f'_c}bt\sqrt{1 + \frac{f_m + f_h}{4\sqrt{f'_c}} + \frac{f_m f_h}{(4\sqrt{f'_c})^2}} \qquad (5)$$

where f_m and f_h are positive for compression.

No additional reinforcement for shear reinforcement is required if V_u is less than 0.85 V_c. The 0.85 factor is a strength reduction normally associated with shear. If the shear load V_u is greater than 0.85 V_c in Eq. [5] then the concrete should be considered cracked with no "concrete contribution." The entire shear should be resisted by reinforcement according to Equations [3] and [4].

Maximum Strength

For reinforced concrete containments with orthogonal steel providing part of the shear strength, the current code(9) limits shear strength V_u, to $8\sqrt{f'_c}$ bt for Factored Loads.

Limits on maximum shear strength are stated in building codes(5,32) for three reasons.

(a) Prevent a diagonal crushing failure in the truss mechanism of shear transfer.

(b) Prevent a sliding shear failure (a local combined shear-crushing failure along a horizontal plane) in the shear friction mechanism of shear transfer.

(c) Prevent large, unsightly shear cracks at the service load level.

The ACI 318 Building Code(5), limits V_s to $8\sqrt{f'_c}$ bt, which then limits V_u to about 10 to $12\sqrt{f'_c}$ bt. These limits appear for crack control at sustained service load levels for non-prestressed beams with Grade 60 reinforcement(33). Since tangential shear for sustained service loads is negligible, crack control at service loads should not be a governing factor for containments. Strength and deformations at factored loads should govern. In general, without longitudinal or transverse steel yielding, the diagonal compression strength or sliding shear strength in reinforced containments should be significantly higher than $8 \sqrt{f'_c}\ bt$.

For diagonal compression crushing, the CEB-FIP Model Code[22] allows $V_u = 0.3\ f'_c\ bt$ for orthogonal steel arrangements and up to $V_u = 0.45\ f'_c\ bt$ if diagonal shear reinforcement at 45° is used.

For sliding shear, Mattock[34] recommended a limit of $V_u = 0.3\ f'_c\ bt$ based on monotonically loaded monolithic push-off specimens and composite specimens with good bond between castings. This limit was reduced to $V_u = 0.24\ f'_c\ bt$ for reversing loads. Using large scale specimens similar to those tested by Mattock, Aoyagi[2[illegible]] derived a "balanced" reinforcement ratio corresponding to a sliding shear strength limit of $0.27\ f'_c\ bt$.

The Japanese had proposed a shear strength limit of $0.18\ f'_c\ bt$[19]. However, this limit is based on test results of specimens with yielding horizontal reinforcement and significant shear distortions occurring prior to a shear failure.[20] A more recent proposed Japanese design criteria[28] is based on testing of full cylindrical models varying up in size up to a 1/8 scale model.[23,24,27] These models exhibited maximum shear stress ranging from $19.6 \sqrt{f'_c}$ to $22 \sqrt{f'_c}$. Using a factor of safety of 1.5, new maximum shear strength limit of $13.2 \sqrt{f'_c}\ bt$ is recommended by the Japanese, as shown in Fig. 4. For $f'_c = 4000$ psi, this limit is equal to $0.21\ f'_c\ bt$.

Panel specimens tested in the experimental programs conducted by CTL[14,15] and Cornell[16,17] all lost load capacity by yielding of reinforcement across diagonal cracks. Therefore, these data cannot be used to establish a limit on maximum strength. However, specimens tested by Vecchio and Collins[18] contained relatively high reinforcement ratios. Therefore, concrete crushing or sliding shear failures were the observed failure modes in most of the specimens.

Results indicated that concrete shear strength decreases as transverse and longitudinal tensile strains increased. With both transverse and longitudinal strains at zero, shear strength was 0.47 f'_c bt. However with both transverse and longitudinal strain at 0.002, (typical yield strain for reinforcement) shear strength was 0.30 f'_c bt. The presence of biaxial tension and reversing shear load reduced shear strength to 0.25 f'_c bt. With a strength reduction factor of 0.85 to account for uncertainty normally associated with shear[5], shear strength would be 0.21 f'_c bt.

Based on review of the available test data, it is recommended by the Task Group on Shear that maximum shear strength for factored loads V_u be limited to 0.2 f'_c bt when orthogonal reinforcement is used to resist shear loads without inclined reinforcement present, i.e.

$$V_{so} \leq 0.2 f'_c bt \tag{6}$$

where $V_{so} = V_u - 0.9 f_y A_{si}$

It should be noted that 0.2 f'_c is near the maximum tangential shear stress that might ever be expected in a containment. However, it additional strength is needed, inclined reinforcement can be used to increase the maximum shear strength.

When only orthogonal reinforcement is present, all the diagonal compressive stresses of the truss mechanism of shear transfer are resisted by concrete. Use of inclined reinforcement provides steel to resist a significant portion of the diagonal compression. When a symmetrical pattern of diagonal reinforcement is present, the strength of the inclined steel in compression balances the strength of inclined steel in tension.

Because of this balance of strength, it might be argued that the shear strength of a containment reinforced with inclined steel should only by limited by the amount of reinforcement that can be placed practically in the walls. However, compatibility must also be considered. The strain associated with yield of reinforcement of 60,000 psi is approximately 0.002. This is a very high compressive strain for concrete that is in tension in the orthogonal direction.

In order to evaluate the relationship between maximum concrete compressive stress and the amounts of orthogonal and diagonal reinforcement, a series of analyses of membrane elements were carried out. These analyses were made to evaluate parameters affecting the maximum shear stress and compressive stress corresponding to full yield of reinforcement in the element. The variables included orthogonal and inclined reinforcement ratios, ρ_m, ρ_h, and ρ_i, concrete strength, f'_c, and concrete strain at peak stress, ϵ_o.

Analyses were made using equations of equilibrium and compatibility formulated by Duchon.[35] The equations were modified to account for yielding of the orthogonal reinforcement. The following nonlinear concrete stress-strain relationship based on panel tests at the University of Toronto[36] was used:

$$f_d = \frac{f'_c}{\beta} \left[2 \frac{\epsilon_c}{\epsilon_o} - \left(\frac{\epsilon_c}{\epsilon_o} \right)^2 \right] \tag{7}$$

$$\text{where } \beta = 0.8 + 0.34\ \epsilon_I/\epsilon_o \tag{8}$$

Equation 7 is a relationship for the effective strength of the concrete in diagonal compression, f_d, as a function of the principal tensile strain ϵ_I. Effective concrete strength f_d, of the compression struts decreases as the tensile strain in the reinforcement running perpendicular through the strut increases. Solution was obtained using an iterative technique with an effective secant modulus for the concrete.

In the series of analyses, the orthogonal reinforcement was varied to represent designs with V_{so} ranging from 0 to 0.2 f'_c bt. With V_{so} = 0, the orthogonal reinforcement is designed to resist only membrane forces from pressurization. With V_{so} = 0.2 f'_c bt, the orthogonal reinforcement was designed to resist normal forces from pressurization plus shear forces. V_{so} = 0.2 f'_cbt corresponds to the recommended maximum allowable design shear discussed in the preceding section for a containment with only orthogonal reinforcement.

At a particular level of orthogonal reinforcement, the amount of inclined reinforcement was increased in the analyses until a concrete crushing failure was calculated to occur prior to general yield of the reinforcement, i.e., $\sigma_{II} = f_d$.

Analyses were carried out with two different peak concrete strains, $\epsilon_o = 0.0015$ and $\epsilon_o = 0.002$. Varying ϵ_o affects the stiffness of the concrete and thereby affects the relative amounts of stress carried by the concrete and steel in compression.

Results of the analyses are shown in Figures 7 through 9. Figure 7 shows the relationship between principal compressive stress σ_{Π} and maximum shear for analyses made with $\epsilon_o = 0.0015$. A diagonal compression crushing was calculated to occur at $\sigma_{\Pi}/f'_c = 0.48$. The maximum shear force at which crushing occurred is dependent on the amount of orthogonal reinforcement allocated to resist shear. Maximum shear strength decreases as V_{so} increases.

Figure 8 presents data similar to Fig. 7 but with the principal stress normalized by f_d. In these plots $\sigma_{\Pi}/f_d = 1$ represents a calculated diagonal compression crushing. The dashed line in Fig. 8 represents a conservative design limit for the concrete compressive stresses. It is drawn at $\sigma_{\Pi}/f_d = 0.72$ which includes a strength reduction factor, $\varphi = 0.85$, normally associated with shear, multiplied by another reduction of 0.85 to account for effects of load reversals.

Maximum shear strength as a function of V_{so} determined from the dashed line in Fig. 8 is shown by a dashed line in Fig. 9 along with a similar line determined for analyses with $\epsilon_o = 0.002$. These dashed lines in Fig. 9 demonstrate the relationship between maximum shear strength and concrete stiffness. When inclined reinforcement is present, maximum strength decreases as concrete stiffness increases.

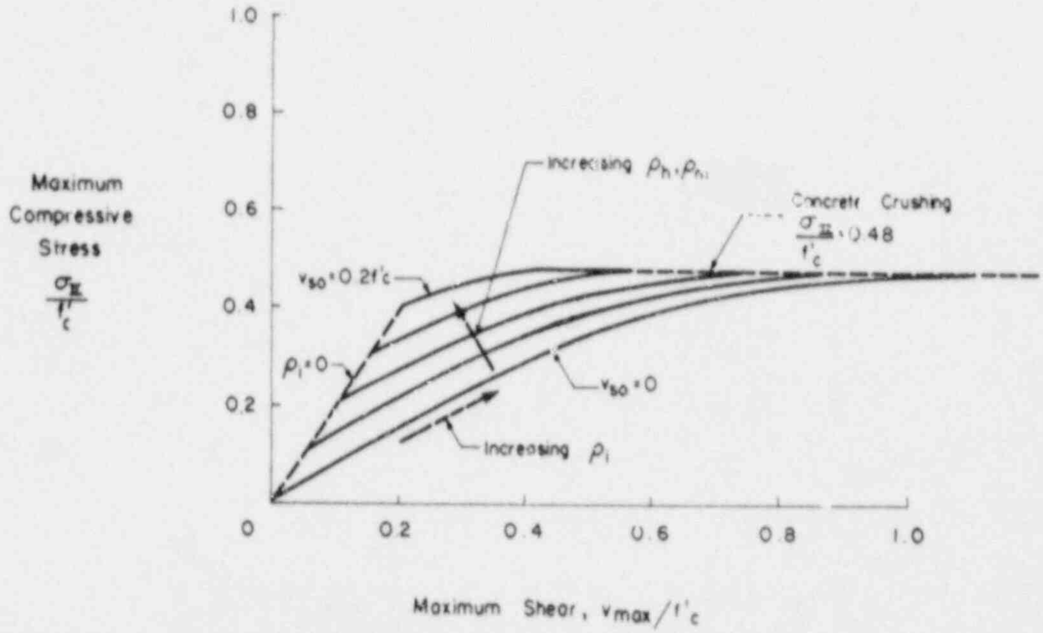


Fig. 7 Maximum Compressive Stress (Normalized by f'_c) Versus Maximum Shear with Varying Amounts of Orthogonal and Diagonal Reinforcement Using $\varepsilon_o = 0.0015$

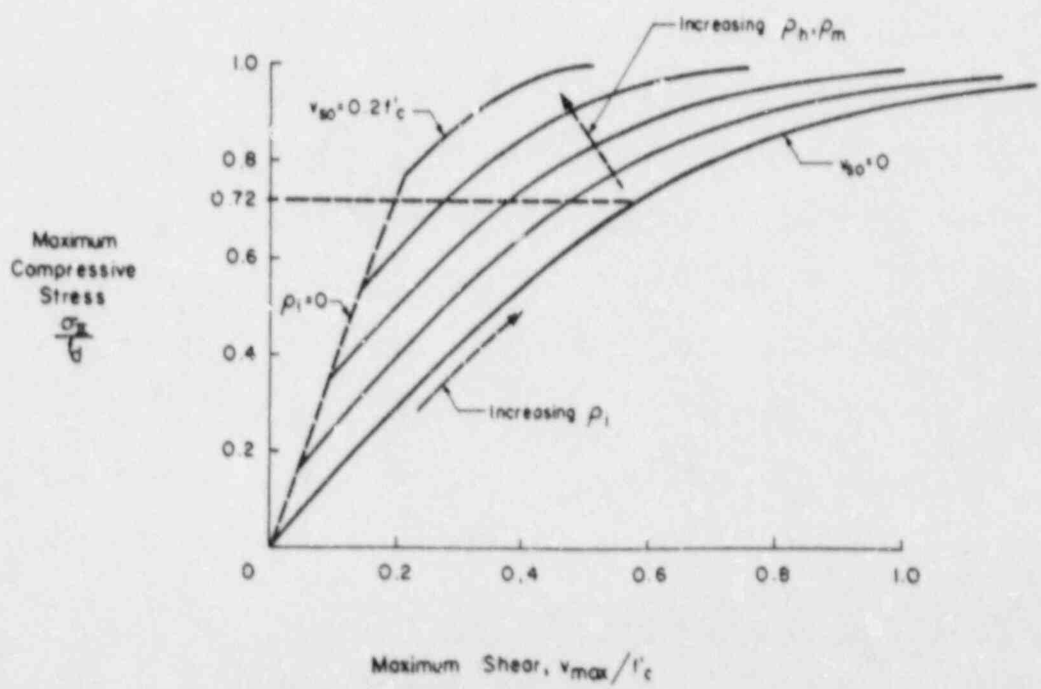


Fig. 8 Maximum Compressive Stress (Normalized by f_d) Versus Maximum Shear with Varying Amounts of Orthogonal and Diagonal Reinforcement Using $\varepsilon_o = 0.0015$

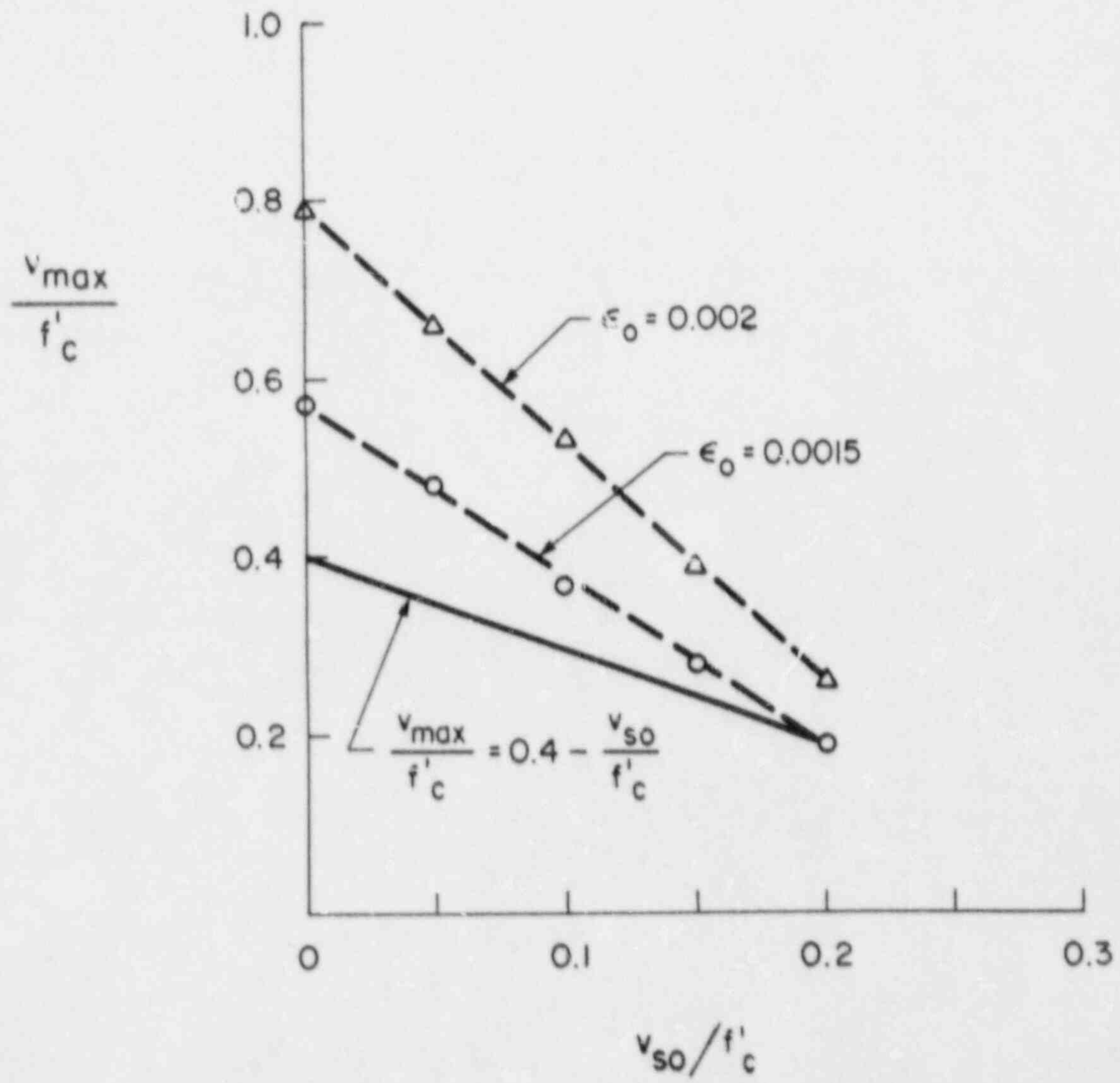


Fig. 9 Maximum Shear Stress Versus Shear Stress Resisted by Orthogonal Reinforcement

The solid line in Fig. 9 corresponds to the following recommended design limit for shear with or without diagonal reinforcement:

$$V_u \leq 0.4\ f_c'\ bt - V_{so} \tag{9}$$

This limit is conservative even for very stiff concrete but will allow containment wall design for all practical situations.

Equation [9] implies that if $V_{so} = 0.2\ f_c'\ bt$, then $V_u = 0.2\ f_c'\ bt = V_{so}$. No inclined reinforcement can be added to increase strength because the orthogonal reinforcement has used up all available concrete strength in diagonal compression. If $V_{so} = 0$, then $V_u = V_{si} = 0.4\ f_c'\ bt$. In other words, a maximum shear corresponding to $0.4\ f_c'$ can be obtained if shear forces are only carried by inclined reinforcement.

A combination of orthogonal and inclined reinforcement can be used to obtain intermediate strengths if $V_{so} < 0.2\ f_c'\ bt$. As an example, say $V_{so} = 0.1\ f_c'\ bt$. Then $V_u = 0.3\ f_c'\ bt$ with $V_{si} = 0.2\ f_c'\ bt$.

DEFORMATION PROVISIONS

The strength provisions presented previously are intended to insure against loss of shear load capacity. However, because of the importance of leak-tightness integrity of the liner and interaction with attached equipment and piping, deformations should also be considered in design.

Results of testing concrete specimens with plane shear across a predefined crack,[1-4,6-8,10-13,22] panel specimens subjected to membrane shear,[14-18,21] and full cylindrical models[23-25,27] demonstrate that shear stiffness reduces significantly after cracking. As an example, in full cylindrical models tested with monotonic shear load by Bader and Krawinkler,[25] shear stiffness after cracking was 7.5% of the uncracked stiffness. Open cracks due to pressurization and abrasion from cyclic load will further reduce the shear stiffness.

Although this behavior of shear stiffness reduction has been known for some time, it is the recommendation of the Task Group on Shear that a statement regarding shear distortions be included in Section CC-3310 General (considerations), Containment Design Analysis Procedures of the ASME-ACI Code.[9]

It is the opinion of the Task Group on Shear that the code should not prescribe how shear distortions should be considered other than to retain the current limit of 2 ϵ_y for maximum strain in the reinforcement.

There are finite element methods available in current literature[26,37,38,39] for modeling cracked concrete. Finite element models have been used successfully to model deformation in full cylindrical models of containments[24,26,27,40].

Shear distortion is expected to reach the sum of the strains in the meridional and hoop reinforcement.[41] The shear distortion can be limited approximately to the strain in the inclined reinforcement by providing inclined reinforcement to carry the entire tangential shear force, i.e., $V_{so} = 0$. The shear strain can also be limited by providing an excess amount of orthogonal reinforcement to carry tangential shear. Design considerations using this approach are suggested by Oesterle[42] based on observed shear deformations in panel tests. The criteria for acceptable deformations should be established from requirements of attached equipment and piping.

It should be noted that when a reducing shear stiffness model under reversing load is considered in dynamic analyses, the maximum shear stresses induced by seismic loading will probably be lower than the stresses normally calculated. Research is needed to develop simplified analytical procedures to efficiently incorporate shear deformations into design criteria.

SUMMARY

The purpose of this report is to present recommended design criteria for tangential shear based on available test data. The following is a summary of the recommendations by the Task Group on Shear:

1. The term V_c in the design equation of the current Code[9] should be replaced by V_{so}.
2. For reinforced concrete containments the "concrete contribution" V_c should be taken as zero.
3. Required area of orthogonal (hoop and meridional) reinforcement, with or without inclined reinforcement, provided for combined membrane and tangential shear strength shall be computed by:

$$A_{sh} + A_{si} = \frac{N_h + \left[N_{hl}^2 + V_u^2\right]^{1/2}}{0.9\ f_y} \qquad [3]$$

$$A_{sm} + A_{si} = \frac{N_m + \left[N_{ml}^2 + V_u^2\right]^{1/2}}{0.9\ f_y} \qquad [4]$$

4. For prestressed concrete containments V_c should be based on a principal tensile stress of $4\sqrt{f'_c}$ carried by the concrete. If V_u exceed 0.85 V_c, the entire shear should be resisted by reinforcement designed according to Equations [3] and [4].

5. Any combination of orthogonal and inclined reinforcement as required for strength according to Eq. [3] and [4], and as required to control shear deformations may be used with the following limits on maximum shear force:

$$V_{so} \leq 0.2\ f'_c\ bt \qquad (6)$$

where $V_{so} = V_u - 0.9\ f_y\ A_{si}$

$$V_u \leq 0.4\ f'_c\ bt - V_{so} \qquad (9)$$

6. A statement regarding consideration of shear distortions should be included in the Codes[9] statements on containment design and analysis procedures in Section CC-3310.

7. Further research should be conducted to develop simplified analytical procedures to efficiently incorporate shear deformations into design criteria.

Example calculations to determine required areas of reinforcement and to check maximum reinforcement strains with and without the use of inclined reinforcement are presented in Appendix B of this report.

ACKNOWLEDGMENTS

A significant amount of current research referenced in this report, including Ref. 14-17, 38 and 40, was supported through contract research funding by the U.S. Nuclear Regulatory Commission (USNRC). This research was accomplished within a USNRC program directed specifically to investigate safety related issues regarding tangential shear in nuclear containments. Dr. B. S. Browzin was the technical representative for the Nuclear Regulatory Commission.

As disucssed in the forward, this report represents the results of the work of the Task Group on Shear within the ASME/ACI Joint Committee on Concrete Pressure Components - Subgroup on Design. Members of the Task Group included J. A. Curtin of Stone and Webster Engineering Corporation, T. E. Johnson and P. Shunmugavel of Bechtel Power Corporation, A. Walser of Sargent and Lundy Engineers, and R. N. White of Cornell University. R. G. Oesterle of Construction Technology Laboratories served as Chairman of the Task Group.

NOTATIONS

A_{sh} = Area of bonded reinforcement in the hoop direction (in^2/ft)

A_{sm} = Area of bonded reinforcement in the meridional direction (in^2/ft)

A_{si} = Area of bonded reinforcement in one direction of inclined bars at 45° to horizontal ($in.^2/ft$ along a line perpendicular to the direction of the bars)

N_h and N_m = Membrane force in the hoop and meridional direction respectively due to pressure, prestress and dead load. N_h and N_m are positive when tension and negative when compression. The prestress force shall be the effective value.

$N_{h\ell}$ and $N_{m\ell}$ = Membrane force in the hoop and meridional direction respectively from lateral load such as earthquake, wind, or tornado loading. When considering earthquake loading, this force is based on the square root of the sum of the squares of the components of the two horizontal and vertical earthquakes. The force is always considered as positive and the units are k/ft.

V_c = Tangential shear strength provided by concrete

V_s = tangential shear strength provided by reinforcement

= $V_{so} + V_{si}$

V_{so} = Tangential shear strength provided by orthogonal (hoop and meridional) reinforcement

V_{si} = Tangential shear strength provided by inclined reinforcement

V_u = The peak membrane tangential shear force resulting from lateral load such as earthquake, wind, or tornado loading. When considering earthquake loading, this force is based on the square root of the sum of the squares of the components of the two horizontal and vertical earthquakes. The shear force shall be considered as positive and the units are k/ft.

b = Unit length of section

f'_c = Compressive strength of standard 6x12-in. concrete cylinders

f_m = Concrete membrane stress in the meridional direction

f_h = Concrete membrane stress in the hoop direction

f_s = Maximum orthogonal reinforcement tensile stress from membrane forces N_h or N_m

f_y = Yield strength of reinforcement

t = Net wall thickness considering any reduction due to tendon ducts

v_{max} = Maximum observed shear stress in test specimens

v_{so} = V_{so}/bt = design shear stress for orthogonal reinforcement

v_u = V_u/bt = total design shear stress

ϵ_c = Strain in concrete

ϵ_o = Strain in concrete at peak stress

ϵ_y = Yield strain of reinforcement

ϵ_Π = Principal tensile strain in the membrane element

ρ = Lesser of ρ_h or ρ_m

ρ' = Effective reinforcement ratio for diagonal tension equilibrium of test specimens

ρ_h = Horizontal reinforcement ratio A_{sh}/bt

ρ_m = Vertical reinforcement ratio A_{sm}/bt

ρ_i = Inclined or diagonal reinforcement ratio A_{si}/bt

σ_Π = Principal compressive stress in the concrete

REFERENCES

1. Colley, B. E. and Humphrey, H. A., "Aggregate Interlock at Joints in Concrete Pavements," Research and Development Bulletin D124, Portland Cement Association, 1967, 18 pp.

2. Fenwick, R. C., and Paulay, T. O., "Mechanism of Shear Resistance in Concrete Beams," Journal of the Structural Division, ASCE, Vol. 94, No. ST 10, October, 1968.

3. Paulay, T., and Loeber, P. J., "Shear Transfer by Aggregate Interlock," ACI Special Publication SP42, Shear in Reinforced Concrete, Vol. 1, American Concrete Institute, 1974 pp. 1-16.

4. Houde, J. and Mirza, M. S., "Investigation of Shear Transfer Across Cracks by Aggregate Interlock," Research Report No. 71-06, Ecole Polytechnique de Montreal, Dept. of Gene Civil, Division de Structures, 1972.

5. ACI Committee 318, Building Code Requirements for Reinforced Concrete, ACI Standard 318-83, American Concrete Institute, Detroit, 1983, 111 pp.

6. White, R. N. and Holley, M. J., "Experimental Studies of Membrane Shear Transfer," Journal of the Structural Division, American Society of Civil Engineers, No. ST3, August 1972.

7. Laible, J. P., White, R. N., and Gergely, P., "Experimental Investigation of Seismic Shear Transfer Across Cracks in Nuclear Containment Vessels," ACI Special Publication SP53, Reinforced Concrete Structures in Seismic Zones, American Concrete Institute, 1977, pp. 203-226.

8. White, R. N., and Gergely, P., "Shear Transfer in Thick Walled Reinforced Concrete Structures Under Seismic Loading," Department of Structural Engineering, Cornell University, Report No. 75-10.

9. "ASME Boiler and Pressure Vessel Code," Section III, Division 2, Code for Concrete Reactor Vessels and Containments, American Society of Mechanical Engineers, New York, 1983 Edition.

10. Jimenez, R., Gergeley, P., and White, R. N., "Shear Transfer Across Cracks in Reinforced Concrete," Report No. 78-4, Department of Structural Engineering, Cornell University, Ithaca, New York, August 1979.

11. Mattock, A. H., "Effect of Aggregate Type on Single Direction Shear Transfer Strength in Monolithic Concrete," Report SM74-2, University of Washington, August 1974, 34 pp.

12. Mattock, A. H., "The Shear Transfer Behavior of Cracked Monolithic Concrete Subjected to Cyclically Reversing Shear," Report SM74-4, University of Washington, November 1974, 31 pp.

13. Walraven, J. C., Vos., E., and Reinhardt, H. W., "Experiments on Shear Transfer in Cracks in Concrete, Part I: Description of Results," Report 5-79-3, Delft University of Technology 1979, 87 pp.

14. Oesterle, R. G. and Russell, H. G., "Shear Transfer in Large Scale Reinforced Concrete Containment Elements-Report No. 1," Report NUREG/CR-1374, U.S. Nuclear Regulatory Commission, Portland Cement Association, April 1980, 67 pp.

15. Oesterle, R. G. and Russell, H. G., "Shear Transfer in Large Scale Reinforced Concrete Containment Elements-Report No. 2," Report to U.S. Nuclear Regulatory Commission, Portland Cement Association, July 1981, 85 pp.

16. Perdikaris, P. C., White, R. N., and Gergely, P., "Strength and Stiffness of Tensioned Reinforced Concrete Panels Subjected to Membrane Shear: Two-Way Reinforcing," Report to NUREG/CR-1602, U.S. Nuclear Regulatory Commission, Cornell University, July 1980, 394 pp.

17. Conley, C. H., White, R. N., and Gergely, P., "Strength and Stiffness of Reinforced Concrete Panels Subjected to Membrane Shear: Two-Way and Four-Way Reinforcing," Report to NUREG/CR-2049, U.S. Nuclear Regulatory Commission, Cornell University, April 1981, 165 pp.

18. Vecchio, F. and Collins, M. P. "The Response of Reinforced Concrete to In-Plane Shear and Normal Stresses" Publication No. 82-03, University of Toronto, March 1982, 332 pp.

19. Ichikawa, K., Adyagi, Y., and Watanabe, Y., "Design Concepts of Concrete Containment Vessels for Shear and Thermal Stresses," Paper J4/1, Transactions of the 5th Conference on Structural Mechanics in Reactor Technology, Berlin, 1979.

20. Uchida, T., Ohmori, N., Takahashi, T., Watanabe, S., Abe, H., and Adyagi, Y., :Behavior of Reinforced Concrete Containment Models Under the Combined Action of Internal Pressure and Lateral Force," Paper J4/4, Transactions of the 5th Conference of Structural Mechanics in Reactor Technology, Berlin, 1979.

21. Aoyagi, Y., Ohmori, S. and Yamada, K., "Strength and Deformation Characteristics of Orthogonally Reinforced Concrete Containment Models Subjected to Lateral Forces," Paper J4/5, Transactions of the 6th Conference on Structural Mechanics in Reactor Technology, Paris, 1981.

22. Aoyagi, Y., Sakamoto, S., Takao, K. and Yamasaki, A., "Full Scale Model Push-Off Test of Reinforced Concrete Block With 51 mm Dia. Deformed Steel Bars," Paper J4/7, Transactions of the 6th Conference on Structural Mechanics in Reactor Technology, Paris, 1981.

23. Osaki, Y, Kobayashi, M., Takega, T., Yamaguchi, T., Yoshizaki, and Sugano, S., "Shear Strength Tests of Prestressed Concrete Containment Vessels," Paper J4/3, Transaction of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

24. Ogaki, Y., Kobayash, M., Takeda, T., Yamaguchi, T., and Yoshioka, K., "Horizontal Loading Tests on Large-Scale Moore if Prestressed Concrete Containment Vessel," Paper J4/2, Transactions of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

25. Bader, M., and Krawinkler, "Shear Transfer in Thick-Walls Reinforced Concrete Cylinders," Paper J3/7, Transaction of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

26. Miyashita, T., and Sozen, M. A., "Nonlinear Analysis of Reinforced-Concrete Containment Vessel Using Shear Transfer Stiffness of Cracked Elements," Paper J3/12, Transaction of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

27. Adyagi, A., Ohnuma, H., Ichikawa, K., and Isobata, O., "Test of a PCCV Under Load Combination of LOCA and Earthquake," Paper J4/9, Transaction of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

28. Ohsaki, Y., IBE, Y., and Adyagi, Y., "Drafted Japanese Design Criteria for Concrete Containment," Paper J1/2, Transaction of the 6th Conference on Structural Mechanics in Reactor Technology, Paris 1981.

29. ACI-ASME Technical Committee on Concrete Pressure Components for Nuclear Service, Item No. JC 7-5, Code Case N-250, Alternate Rules for Design of Tangential Shear Forces for Section III, Division 2, Class CC, 1980.

30. Joint ASCE-ACI Task Committee 426, "The Shear Strength of Reinforced Concrete Members," ASCE Journal, Structural Division, ST6, June 1973, pp. 1091-1186.

31. Hofbeck, J.A., Ibrahim, I.A. and Mattock, H.H., "Shear Transfer in Reinforced Concrete," ACI Journal, Vol. 66, No. 2, February 1969, pp. 119-128.

32. Comite Euro-International Du Beton, "Shear and Torsion," CEB-FIP Model Code for Concrete Structures, Bulletin D'Information No. 126, Paris, June 1978.

33. Collin, M. P. and Mitchell, D., "Shear and Torsion Design of Prestressed and Non-Prestressed Concrete Beams," Journal, Prestressed Concrete Institute, Vol. 25, No. 5, Sept./Oct, 1980, pp. 32-100.

34. Mattock, A. H., "Shear Transfer Under Cyclically Reversing Loading Across an Interface Between Concrete Cast at Different Times," Report SM 77-1, University of Washington, Seattle, June 1977, 26 pp.

35. Duchon, N. B., "Analysis of Reinforced Concrete Membrane Subject to Tension and Shear," ACI Journal, Proceeding Vol. 69, No. 9, September 1972, pp. 578-583.

36. Private Communications Between M. Collins and R. Oesterle. Information is included in a paper that has been accepted for publication in ACI Journal.

37. ASCE Task Committee on Finite Element Analysis of Reinforced Concrete Structures, Finite Element Analysis of Reinforced Concrete Structures, American Society of Civil Engineers, 1982, 545 pp.

38. Calvo, J., Buyukozturk, O., and Connor, J. J., "Design of Reinforced Concrete Containment Wall Elements Under Combined Action of Shear and Tension," Report to U.S. Nuclear Regulatory Commission, NUREG/CR 3157, Massachusetts Institute of Technology, February 1983 211 pp.

39. Gupta, A. K., "Design of Membrane Reinforcement for Concrete Nuclear Containments" North Carolina State University, January 1982, 25 pp.

40. Conley, C. H., White, R. N. and Gergely, P., "Analysis of Reinforced Concrete Containment Vessels with Nonlinear Shearing Stiffness," Report NUREG/CR-3255, U.S. Nuclear Regulatory Commission, Cornell University, April 1983, 98 pp.

41. P. Shummugavel, "Analysis and Design of Concrete Containments for Tangential Shear Loads," Paper No. J5/2, 7th International Conference on Structural Mechanics in Reactor Technology, August 1983.

42. Oesterle, R. G., "Tangential Shear Design in Reinforced Concrete Containments: Research Results and Applications," Transactions of the 7th International Conference on Structural Mechanics in Reactor Technology, Panel Session JK-P, August 1983.

APPENDIX A

FORCE DISTRIBUTION

A concrete containment shell is generally a vertical thin-walled cantilever structure with a circular cross section. Tangential shear forces are in the plane of the containment shell resulting from lateral loading such as wind or seismic loads.

Wind Load

The lateral wind load causes an overall moment M and shear V, both of which are varying along the height of the containment (Figure A1). The resulting stresses at an elevation of the containment are shown in Figure A2 corresponding to an uncracked elastic condition. The maximum meridional force N_{vw} occurs at the outermost fiber of the cross section while the maximum tangential shear force V_{uw} occurs at the centerline of the cross section. The maximum forces are given by:

$$N_{vw} = \frac{M}{\pi r^2}$$

$$V_{uw} = \frac{V}{\pi r}$$

where

r = mean radius of the containment cross-section

The forces at any location along the circumference of the containment are expressed as:

$$N_\theta = N_{vw} \cos\theta$$

$$V_\theta = V_{uw} \sin\theta$$

where

θ = angle from the direction of wind (Figure A2).

The maximum value of $(N_\theta + V_\theta)$ occurs at θ_1 which can be derived as:

$$(N_\theta + V_\theta)_{max.} = \sqrt{N_{vw}^2 + V_{uw}^2}$$

$$\theta_1 = \text{Tan}^{-1} \left(\frac{V_{uw}}{N_{vw}}\right)$$

Earthquake Load

An earthquake has three orthogonal (two horizontal and one vertical) components which cause the following distributions of stresses corresponding to an uncracked elastic condition as shown in Figure A3. The internal forces at a location defined by angle θ are expressed as:

$$N_\theta = \pm E_h \cos\theta; \quad \pm E_h \sin\theta; \quad \pm E_v$$

$$V_\theta = \pm T \sin\theta; \quad \pm T \cos\theta; \quad 0$$

where

$E_h = \frac{M}{\pi r^2}$ = maximum meridional force from a horizontal component of the earthquake

$E_v = \frac{N}{2\pi r}$ = meridional force from the vertical component

$T = \frac{V}{\pi r}$ = maximum tangential shear from a horizontal component

N = overall meridional force from the vertical component

M, V = overall moment and shear from a horizontal component

It should be noted from Figure A3 that at all locations along the circumference, either the meridional force N_θ or the shear V_θ from a horizontal earthquake component has an opposite sign compared to those from the other horizontal component.

The sum of meridional and shear forces at a location is expressed as:

$$(N_\theta + V_\theta) = \pm (E_h \cos\theta + T \sin\theta);$$

$$\pm (E_h \sin\theta - T \cos\theta); \quad \pm E_v$$

Combining the responses from the three components by the square-root-of-the-sum-of-squares (SRSS) method,

$$(N_\theta + V_\theta) = \pm [E_h^2 (\cos^2 \theta + \sin^2 \theta)$$

$$+ T^2 (\sin^2 \theta + \cos^2 \theta) + E_v^2 + 2E_hT \cos\theta \sin\theta$$

$$- 2E_hT \sin\theta \cos\theta]^{1/2}$$

$$= \pm [E_h^2 + E_v^2 + T^2]^{1/2}$$

$$(N_\theta + V_\theta) = \pm\sqrt{N_{ve}^2 + V_{ue}^2}$$

where

$$N_{ve}^2 = E_h^2 + E_v^2$$

$$V_{ue} = T$$

Thus, the total response $(N_\theta + V_\theta)$ from the three earthquake components are the same at all locations, i.e., independent of the angle θ.

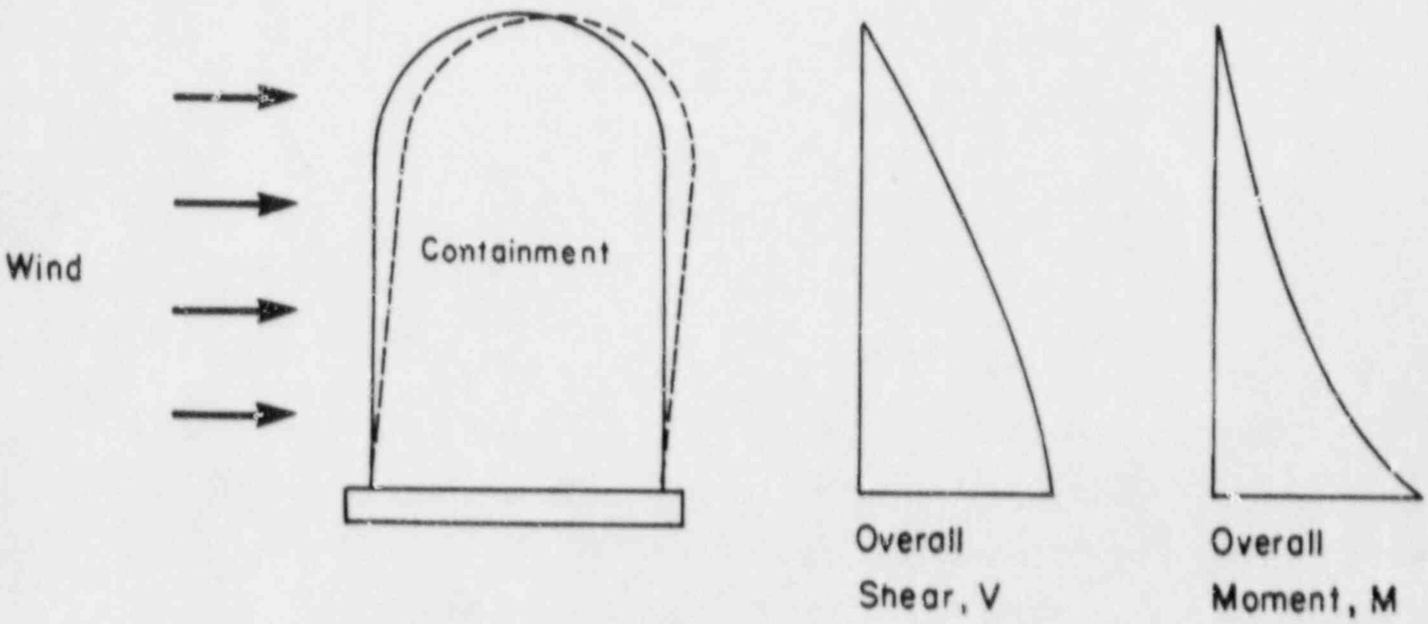


Fig. A1 Shear and Moment on Containment
from Wind Loadin

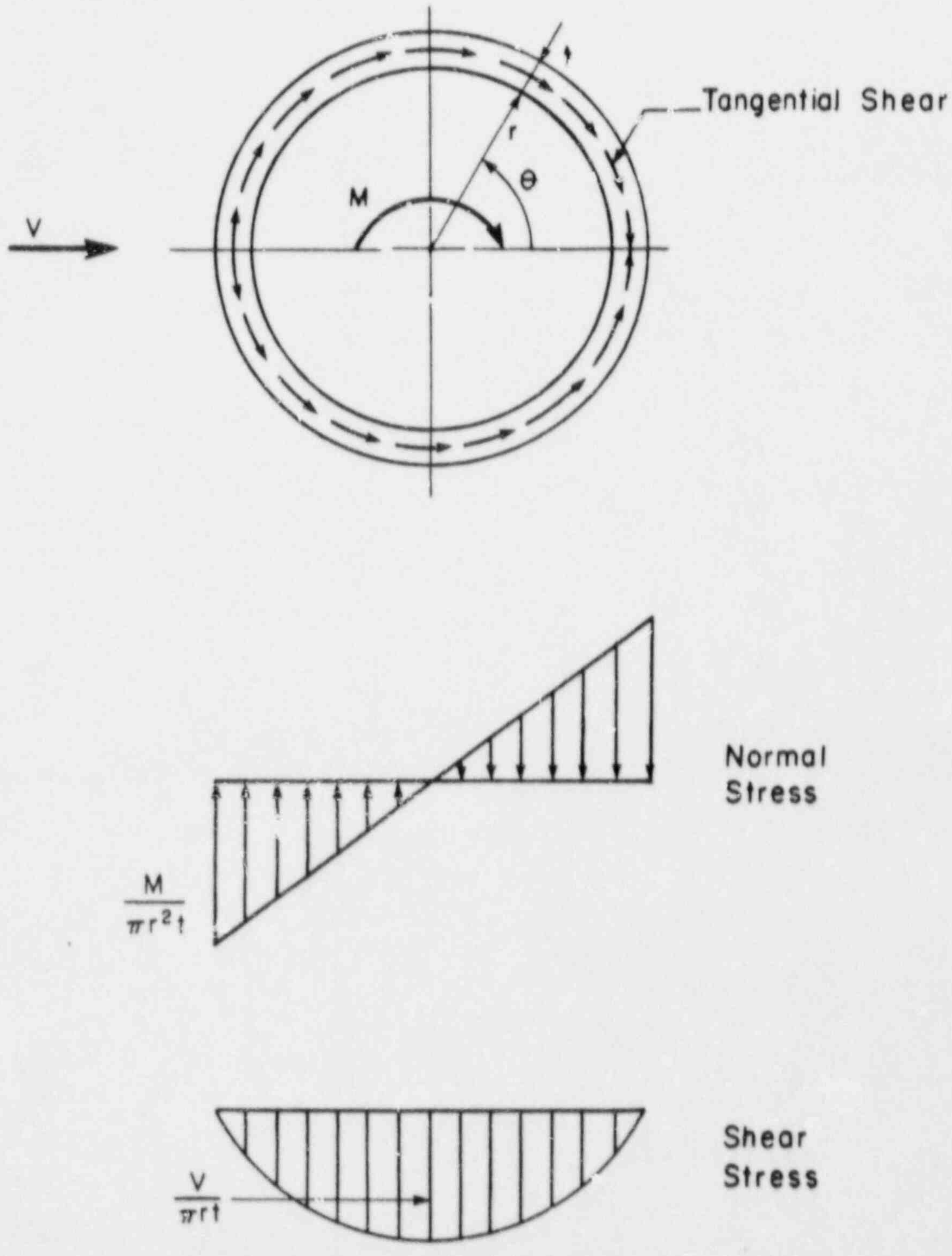
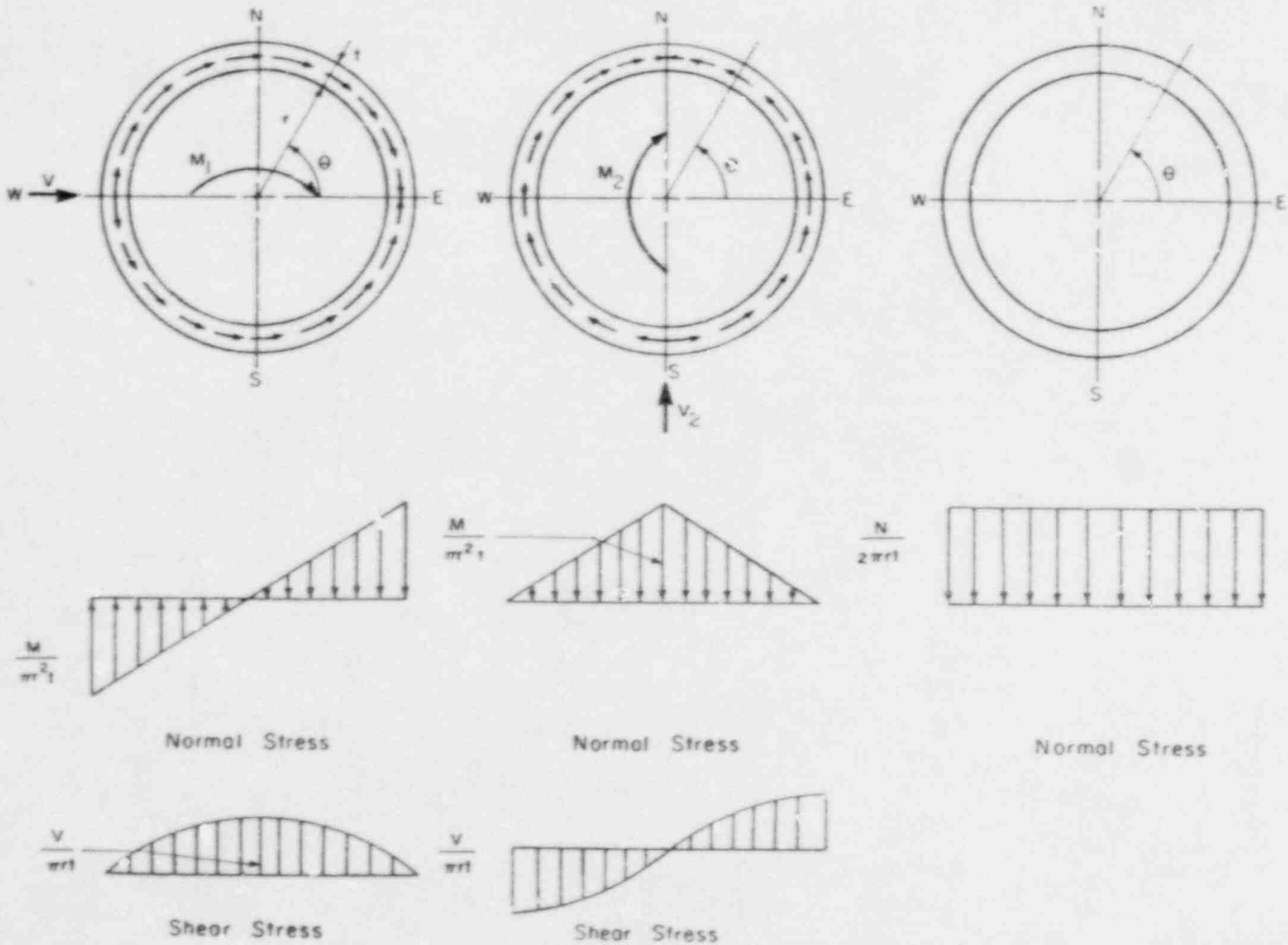


Fig. A2 Distribution of Stresses from Wind Load



a) Stress Distribution in East-West Direction Due to East-West Component

b) Stress Distribution in East-West Direction Due to North-South Component

c) Stress Distribution in East-West Direction Due to Vertical Component

Fig. A3 Distribution of Stresses from Earthquake Loads

APPENDIX B

EXAMPLE CALCULATIONS FOR

DESIGN FOR TANGENTIAL SHEAR - MEMBRANE REGION

Design Parameters : 0.6g SSE, P_a = 52 psig

f'_c = 3 ksi f_y = 60 ksi   b = 12 in.

E_c = 3150 ksi E_s = 29,000 ksi   t = 53.625

Load Combination : D + P_a + E_{ss}

N_m = 116 k/ft N_h = 480 k/ft

N_{ml} = 504 k/ft N_{hl} = 17 k/ft   V = 324 k/ft

$$A_{sh} + A_{si} = \frac{N_h + (N_{hl}^2 + V_u^2)^{1/2}}{0.9\ f_y} \qquad \text{[Eq. 3]}$$

$$= \frac{480 + (17^2 + 324^2)^{1/2}}{0.9 \times 60} = \underline{15.0\ \text{in.}^2/\text{ft}}$$

$$A_{sm} + A_{si} = \frac{N_m + (N_{ml}^2 + V_u^2)^{1/2}}{0.9\ f_y} \qquad \text{[Eq. 4]}$$

$$= \frac{116 + (504^2 + 324^2)^{1/2}}{0.9 \times 60} = \underline{13.25\ \text{in.}^2/\text{ft}}$$

$0.2\ f'_c\ bt = 0.2 \times 3 \times 12 \times 53.625 = 386$ k/ft

V_{so} = Tangential shear provided by orthogonal rebars

$= (V_u - 0.9\ A_{si}) \leq 0.2\ f'_c\ bt$ [Eq. 6]

V_u = 324 k/ft < $0.2\ f'_c$ bt = 386 k/ft

No inclined shear rebar is required

Load Combination : D + 1.25 P_a + 1.25 E_o

N_m = 179 k/ft $\qquad$ N_h = 598 k/ft

N_{ml} = 400 k/ft $\qquad$ N_{hl} = 13 k/ft

$$A_{sh} + A_{si} = \frac{N_h + (N_{hl}^2 + V_u^2)^{1/2}}{0.9\ f_y} \qquad \text{[Eq. 3]}$$

$$= \frac{598 + (13^2 + 255^2)^{1/2}}{0.9 \times 60} = \underline{15.8\ \text{in.}^2/\text{ft}}$$

$$A_{sm} + A_{si} = \frac{N_m + (N_{ml}^2 + V_u^2)^{1/2}}{0.9\ f_y} \qquad \text{[Eq. 4]}$$

$$= \frac{179 + (400^2 + 255^2)^{1/2}}{0.9 \times 60} = \underline{12.0\ \text{in.}^2/\text{ft}}$$

Considering both load combinations, rebars required are as follows:

$$A_{sh} + A_{si} = 15.8\ \text{in.}^2/\text{ft}$$

$$A_{sm} + A_{si} = 13.25\ \text{in.}^2/\text{ft}$$

Provide the following rebars:

Example a) <u>Without Inclined Seismic Rebars</u>:

A_{sh} = 16.25 in.^{2}/ft $\qquad$ A_{sm} = 13.50 in.^{2} /ft $\qquad$ A_{si} = 0

Example a) <u>Without Inclined Seismic Rebars</u>:

A_{sh} = 16.25-3.2 $\qquad$ A_{sm} = 13.5-3.2

A_{sh} = 13.05 in.^{2}/ft $\qquad$ A_{sm} = 10.3 in.^{2}/ft

$A_{si} = A_{s3} = A_{s4}$ = 3.2 in.^{2}/ft

Required reinforcement areas are determined based on a SRSS of normal and shear forces resulting from earthquake loading as discussed in Appendix A. To determine membrane forces to use in calculations for strain compatibility to check maximum reinforcement strain, make the following adjustments to the forces:

a. <u>Load Combination</u> : <u>$D + P_a + E_{ss}$</u>

$$N_h' = N_h + (N_{h1}^2 + V_u^2)^{1/2} - V_u =$$

$$480 + (17^2 + 324^2)^{1/2} - 324 = 481 \text{ k/ft}$$

$$N_m' = N_m + (N_{m1}^2 + V_u^2)^{1/2} - V_u =$$

$$116 + (504^2 + 324^2)^{1/2} - 324 = 391 \text{ k/ft}$$

$$V_u = 324 \text{ k/ft}$$

b. <u>Load Combination</u> : <u>$D + 1.25\ P_a + 1.25\ E_o$</u>

$$N_h' = 598 + (13^2 + 255^2)^{1/2} - 255 = 598 \text{ k/ft}$$

$$N_m' = 179 + (400^2 + 255^2)^{1/2} - 255 = 398 \text{ k/ft}$$

$$V_u = 255 \text{ k/ft}$$

Table B-1 Design for Tangential Shear - Membrane Region : Summary of Initial Results*

Rebar Patterns	Load Comb	Force System (k/ft)	Adjusted Forces for Compatibility	Rebars Reqd. (in.^{2}/ft)		Rebars Provided (in.^{2}/ft)					Computed Rebar Stresses (ksi)				f_c (ksi)	β**	$V_s = f_c \times \sin\beta \times \cos\beta$ (ksi)	Remarks
				$A_{sh} + A_{sl}$	$A_{sm} + A_{sl}$	A_{sh}	A_{sm}	A_{sl}	$A_{sh} + A_{sl}$	$A_{sm} + A_{sl}$	Merd. f_m	Hoop f_h	Seismic f_{s3}	Seismic f_{s4}				
Orthogonal Rebars Only	$D + P_a + E_{ss}$	$N_m = 116$ $N_{ml} = 504$ $N_h = 480$ $N_{hm} = 17$ $V_u = 324$	$N_m' = 391$ $N_h' = 481$ $V_u = 324$	15.0	13.25	16.25	13.5	0	16.25	13.5	52.47	49.96	0	0	-1.0	44.4°	0.50	$V_s = 0.50$ ksi $< 0.2 f_c'$
	$D + 1.25 P_a + 1.25 E_o$	$N_m = 179$ $N_{ml} = 400$ $N_h = 598$ $N_{hm} = 13$ $V_u = 255$	$N_m' = 398$ $N_h' = 598$ $V_u = 255$	15.8	12.00	16.25	13.5	0	16.25	13.5	48.90	52.10	0	0	-0.793	45.79°	0.397	$V_s = 0.397$ ksi $< 0.2 f_c'$
Orthogonal and Diagonal Rebars	$D + P_a + E_{ss}$	$N_m = 116$ $N_{ml} = 504$ $N_h = 480$ $N_{hm} = 17$ $V_u = 324$	$N_m' = 391$ $N_h' = 481$ $V_u = 324$	15.0	13.25	13.05	10.3	3.20	16.25	13.5	41.96	40.54	87.49	-5.07	-0.547	44.54°	0.273	$f_{s3} = 87.49$ ksi > 54 ksi Limit f_{s3} to 54 ksi and readjust stresses
	$D + 1.25 P_a + 1.25 E_o$	$N_m = 179$ $N_{ml} = 400$ $N_h = 598$ $N_{hm} = 13$ $V_u = 255$	$N_m' = 398$ $N_h' = 598$ $V_u = 255$	15.8	12.00	13.05	10.3	3.20	16.25	13.5	37.81	43.95	84.95	-3.19	-0.355	46.99°	0.177	$f_{s3} = 84.95$ ksi > 54 ksi Limit f_{s3} to 54 ksi and readjust stresses

*Calculations made using computer program to solve equations of equilibrium and compatibility from Reference 35.

**Angle between meridional and principal major axis.

Initial results indicate incline tension reinforcement will yield. Therefore, the calculations must be repeated with adjustments for effects of yielding of inclined seismic reinforcement:

Stresses in tensile diagonal rebars are as follows:

f_{s3} $(D + P_a + E_{ss})$ =

87.49 ksi > 0.9 f_y (54 ksi), $\underline{\epsilon_{s3} > \epsilon_y}$

f_{s3} $(D + 1.25\ P_a + 1.25\ E\ )$ =

84.95 ksi > 0.9 f_y (54 ksi), $\underline{\epsilon_{s3} > \epsilon_y}$

Restrict f_{s3} to 54 ksi and neglect it for any further load-carrying purpose. Readjust membrane axial forces and tangential shear neglecting tensile inclined rebar (A_{s3}) and forces associated with it.

Adjusted forces:

a. <u>Load Combination</u> : <u>$D + P_a + E_{ss}$</u>

N_m" = 391 - 54 x 3.2 x 0.5* = 304.6 k/ft

N_h" = 481 - 54 x 3.2 x 0.5* = 394.6 k/ft

V_u" = 324 - 54 x 3.2 x 0.5* = 237.6 k/ft

b. <u>Load Combination</u> : <u>$D + 1.25\ P_a + 1.25\ E_o$</u>

N_m" = 398 - 54 x 3.2 x 0.5* = 311.6 k/ft

N_h" = 598 - 54 x 3.2 x 0.5* = 511.6 k/ft

V_u" = 255 - 54 x 3.2 x 0.5* = 168.6 k/ft

---

*$\sin^2 45$ = 0.5

Final strains in tensile diagonal rebars:

a. Load Combination : $D + P_a + E_{ss}$

$$\epsilon_{s3} = \frac{f_m + f_h}{2\ E_s} + \frac{\gamma}{2} = \frac{54.12 + 48.99}{2 \times 29{,}000} + \frac{.00394}{2} = 0.00375$$

$$= \frac{108.68}{E_s} < 2\ \epsilon_y \quad \frac{120}{E_s}$$

b. Load Combination : $D + 1.25\ P_a + 1.25\ E_o$

$$\epsilon_{s3} = \frac{f_m + f_h}{2\ E_s} + \frac{\gamma}{2} = \frac{47.34 + 51.70}{2 \times 29{,}000} + \frac{.00373}{2} = 0.00357$$

$$= \frac{103.61}{E_s} < 2\ \epsilon_y \quad \frac{120}{E_s}$$

Equilibrium Checks: $A_g = 643.5$ in.^{2}/ft, $A_{sm} = 10.3$ in.^{2}/ft,

$A_{sh} = 13.05$ in. /ft, $A_{si} = A_{si} = 3.2$ in.^{2}/ft

a. Load Combination : $D + P_a + E_{ss}(\beta = 44.21°,\ f_c = -\ 0.707\ \text{ksi})$

$$N_m' = 10.3 \times 52.12 + \frac{3.2}{2}(54-6.55) - 643.5 \times .707 \times \sin^2\beta = 391.5 \qquad \underline{(391\ k/ft)}$$

$$N_h' = 13.05 \times 48.99 + \frac{3.2}{2}(54-6.55) - 643.5 \times .707 \times \cos^2\beta = 481.5 \qquad \underline{(481\ k/ft)}$$

$$V_u' = \frac{3.2}{2} \times (54+6.55) + 643.5 \times .707 \times \sin\beta\ \cos\beta = 324 \qquad \underline{(324\ k/ft)}$$

b. Load Combination : $D + 1.25\ P_a + 1.25\ E_o$

$(\beta = 46.15°,\ f_c = -0.503\ ksi)$

$$N_m' = 10.3 \times 47.34 + \frac{3.2}{2}(54-4.63) - 643.5 \times$$

$$.503 \times \sin^2\beta = 398.3 \qquad \underline{(398\ k/ft)}$$

$$N_h' = 13.05 \times 51.7 + \frac{3.2}{2}(54-4.63) - 643.5 \times$$

$$.503 \times \cos^2\beta = 598.3 \qquad \underline{(598\ k/ft)}$$

$$V_u' = \frac{3.2}{2} \times (54+4.63) + 643.5 \times$$

$$.503 \times \sin\beta\ \cos\beta = 255.6 \qquad \underline{(255\ k/ft)}$$

Table B.2 Design for Tangential Shear - Membrane Region : Summary of Final Results*

Rebar Patterns	Load Comb.	Force System (k/ft)	Adjusted Forces for Compatibility	Rebars Reqd. (in.^{2}/ft)		Rebars Provided (in.^{2}/ft)					Computed Rebar Stresses (ksi)				f_c (ksi)	β**	$v_s = f_c \times \sin\beta \times \cos\beta$ (ksi)	Remarks
				$A_{sh} + A_{sl}$	$A_{sm} + A_{sl}$	A_{sh}	A_{sm}	A_{sl}	$A_{sh} + A_{sl}$	$A_{sm} + A_{sl}$	Merd. f_m	Hoop f_h	Seismic f_{s3}	Seismic f_{s4}				
Orthogonal Rebars Only	$D + P_a + E_{SS}$	$N_m = 116$ $N_{ml} = 504$ $N_h = 400$ $N_{hm} = 17$ $V_u = 324$	$N_m' = 391$ $N_h' = 481$ $V_u = 324$	15.0	13.25	16.25	13.5	0	16.25	13.5	52.47	49.96	0	0	-1.0	44.4°	0.50	$v_s = 0.50$ ksi $< 0.2 f_c'$ $\gamma = 0.00418$
	$D + 1.25 P_a + 1.25 E_o$	$N_m = 179$ $N_{ml} = 400$ $N_h = 590$ $N_{hm} = 17$ $V_u = 255$	$N_m' = 390$ $N_h' = 590$ $V_u = 255$	15.8	12.00	16.25	13.5	0	16.25	13.5	48.90	52.10	0	0	-0.793	45.79°	0.397	$v_s = 0.397$ ksi $< 0.2 f_c'$ $\gamma = 0.00338$
Orthogonal and Diagonal Rebars	$D + P_a + E_{SS}$	$N_m = 116$ $N_{ml} = 504$ $N_h = 400$ $N_{hm} = 17$ $V_u = 324$	$N_m' = 391$ $N_h' = 481$ $V_u = 324$	15.0	13.25	13.05	10.3	3.2	16.25	13.5	52.12	48.99	54.0	-6.55	-0.707	44.21°	0.353	$\gamma = 0.00394$ $f_{s3} = 54$ ksi $\epsilon_{s3} = \frac{108.68}{E_s}$
	$D + 1.25 P_a + 1.25 E_o$	$N_m = 179$ $N_{ml} = 400$ $N_h = 546$ $N_{hm} = 17$ $V_u = 255$	$N_m' = 390$ $N_h' = 590$ $V_u = 255$	15.8	12.00	13.05	10.3	3.2	16.25	13.5	47.34	51.70	54.0	-4.63	-0.503	46.15°	0.252	$\gamma = 0.00373$ $f_{s3} = 54$ ksi $\epsilon_{s3} = \frac{103.61}{E_s}$

*Calculations made using computer program to solve equations of equilibrium and compatibility from Reference 35.

**Angle between meridional and principal major axis.

NRC FORM 335 (2-84) NRCM 1102, 3201, 3202 U.S. NUCLEAR REGULATORY COMMISSION

BIBLIOGRAPHIC DATA SHEET

SEE INSTRUCTIONS ON THE REVERSE

1. REPORT NUMBER (Assigned by TIDC, add Vol No., if any): NUREG/CR-5209

2. TITLE AND SUBTITLE: Design Provisions for Tangential Shear in Containment Walls

3. LEAVE BLANK

4. DATE REPORT COMPLETED: MONTH April, YEAR 1988

5. AUTHOR(S): R. G. Oesterle

6. DATE REPORT ISSUED: MONTH August, YEAR 1988

7. PERFORMING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code):
Construction Technology Laboratories, Inc.
5420 Old Orchard Road
Skokie, Illinois 60077-1030

8. PROJECT/TASK/WORK UNIT NUMBER

9. FIN OR GRANT NUMBER: FIN No. D2077

10. SPONSORING ORGANIZATION NAME AND MAILING ADDRESS (Include Zip Code):
Division of Engineering
Office of Nuclear Regulatory Research
U.S. Nuclear Regulatory Commission
Washington, DC 20555

11a. TYPE OF REPORT: Technical

b. PERIOD COVERED (Inclusive dates)

12. SUPPLEMENTARY NOTES: Prepared in cooperation with the ASME/ACI Joint Committee on Concrete Pressure Components - Subgroup on Design, Task Group on Shear

13. ABSTRACT (200 words or less)

The purpose of the work accomplished in preparing this report was to synthesize results of available research concerning the capacity of cracked reinforced containment walls to transfer tangential shear stresses while in a state of biaxial tension from internal pressurization. A review of experimental work is presented. Results of experimental work indicate that the current ASME-ACI Code Provisions for Tangential Shear Stresses are very conservative.

Recommendations for redefinition and revised use of the terms V_c "concrete contribution", and V_s "steel contribution" are provided. Results of testing programs are used to formulate revised design provisions for diagonal tensile strength. Significantly higher shear stresses can be allowed without inclined reinforcement. Also, an analytical study based on recent testing programs is used to define a conservative maximum limit for tangential shear stress. The maximum limit is dependent on the relative amounts of orthogonal reinforcement and inclined reinforcement used to provide the tangential shear strength in the containment walls.

While testing programs indicate that significant shear strength is available in cracked reinforced concrete, the testing also demonstrates that shear stiffness reduces significantly after cracking. The need to consider the reduced shear stiffness is discussed. Recommendations for revised design provisions are summarized and design examples are provided.

14. DOCUMENT ANALYSIS - a. KEYWORDS/DESCRIPTORS: Containment structures, shear tests, seismic design, biaxial tension, shear strength, shear stiffness, shear deformation, yield strength, cyclic loads, design procedures.

b. IDENTIFIERS/OPEN-ENDED TERMS: Structural walls, concrete (reinforced)

15. AVAILABILITY STATEMENT: Unlimited

16. SECURITY CLASSIFICATION: (This page) Unclassified; (This report) Unclassified

17. NUMBER OF PAGES

18. PRICE

☆U.S. GOVERNMENT PRINTING OFFICE:1988-202-292:80148

UNITED STATES
NUCLEAR REGULATORY COMMISSION
WASHINGTON, D.C. 20555

OFFICIAL BUSINESS
PENALTY FOR PRIVATE USE, $300

SPECIAL FOURTH-CLASS RATE
POSTAGE & FEES PAID
USNRC
PERMIT No. G-67

120555137330 1 1AN1RD
US NRC-OARM-ADM
DIV FOIA & PUBLICATIONS SVCS
SECTION CHIEF
REGULATORY PUBLICATION SECT
P-210
WASHINGTON DC 20555