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SIMPLE MODEL FOR SHEAR RESPONSE OF R.C. PANELS

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SUMMARY

Presented in this paper is a study to provide a practical method for estimating the shear stiffness of a cracked web and to submit monotonic and hysteritic models for shear response of reinforced concrete panels. The models, based on the yield criterion and calibrated to the test results, take into account the contribution and orientation of the reinforcement and are relatively simple. Due to its simplicity and ability to produced reasonably good results with a minimum amount of computation, the approach offers attractive alternative to the use of available complicated and lengthy analytical procedures.

INTRODUCTION

The contribution of shear deformation to the overall deformation of a deep or stubby reinforced concrete element/structure subjected to increasing monotonic and hysteritic loads has always been of great interest. Although the subject has been studied for quite some time (Ref. 1), it may be concluded that up to now "accurate" theoretical estimation of shear deformation of reinforced concrete elements is difficult to obtain. The limited available experimental data on the subject showed a relatively large scattered results. Since performance of a theoretical model should be measured by its ability to modeled the response of a real structure or its component by reasonably matching the related experimental results, the above conclusion suggest the use of the simplest approach in derivation of the shear response model. The fact that the use of complicated formulae such as the one introduced by Lenschow (Ref. 2) or Vecchio (Ref. 3) were still not able to accurately estimate the actual shear response of a reinforced concrete element strengthen the logic of the proposed approach.

MODEL FOR THE MONOTONIC SHEAR RESPONSE OF R.C. PANELS

Basically the model (Ref. 1) may be described in the following, see Fig. 1. The first stage of the shear response is the precracked state. The second stage is the cracked stage up to yielding of the reinforcement, and the last stage is the state beyond yield.

In the precracked state the shear stiffness of a reinforced concrete panel is assumed equal to the shear modulus of rigidity of concrete, **G**. The point of cracking is defined by

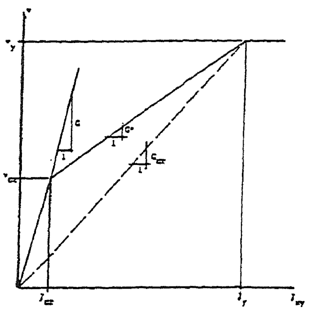


Fig. 1 Basic Model for Proposed Monotonic Shear Response

$$v_{cr} = 0.33\sqrt{f'_c} \qquad (1)$$

$$\gamma_{cr} = v_{cr}/G \qquad (2)$$

where

v_{cr} = cracking shear stress, in MPa,

γ_{cr} = cracking shear strain.

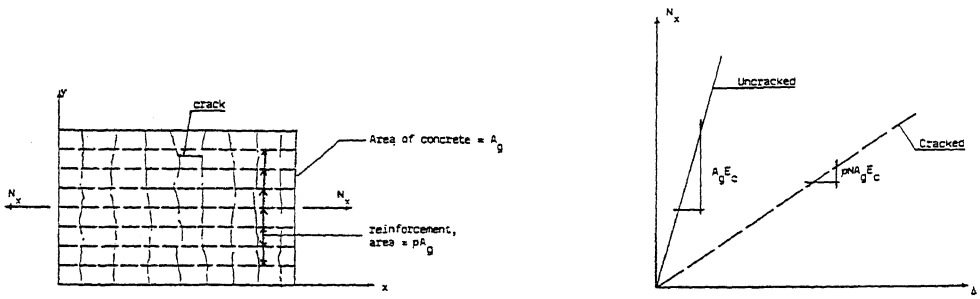
Beyond crack up to yielding the shear stiffness G^* is defined as

$$G^* = \frac{v_y - v_{cr}}{\gamma_y - \gamma_{cr}} \qquad (3)$$

where v_y and γ_y are the yielding shear stress and the yielding shear strain. The value of γ_y is computed from

$$\gamma_y = v_y/G_{cr} \qquad (4)$$

in which G_{cr} is the cracked shear modulus of rigidity of the reinforced concrete panel. A complete shear response could then be obtained if the value of the yielding shear stress, v_y, and the cracked shear modulus of rigidity, G_{cr}, are known.



(a) Reinforced Concrete Element (b) Tensile stiffness of the element

Fig. 2 Reinforced Concrete Element in Uniaxial Tension

Consider a cracked reinforced concrete plate element in tension shown in Fig. 2(a). The cracks are shown in full lines, while the reinforcement are shown with broken lines. Figure 2(b) illustrates the tensile stiffness of the element for the case of uncracked and cracked conditions. For uncracked condition the tensile stiffness is approximately equal to $A_g E_c$, where A_g is the gross area of the section perpendicular to the tensile force direction, and E_c is the modulus of elasticity of concrete. For the cracked condition the tensile stiffness could be approximated by the condition at the cracked region and governed only by the steel reinforcement. Here the tensile stiffness is ρ **n** $A_g E_c$, where ρ is the ratio of the area of reinforcement to the gross area of concrete and **n** is the ratio of the modulus of elasticity of the reinforcement to the modulus of elasticity of concrete.

In his study Lenschow (Ref. 2) found that the stiffness of a reinforced concrete plate element at yield is governed mainly by the steel reinforcement. Hence combining his conclusion with the above information concerning the tensile stiffness of a reinforced concrete element, a parallel approximation is sought for defining the cracked shear stiffness of a reinforced concrete element. It is the assumed that

$$G_{cr} = f(\rho\ n\ G) \qquad (5)$$

or that the cracked shear stiffness of a reinforced concrete element is a function of (ρ **n G**).

For the case of isotropically reinforced concrete element, it is obvious that the value of ρ is the same in both directions of reinforcement and hence does not present any problem. However in nonisotropically reinforced concrete element, an appropriate value of ρ should be sought. Logically, if the value of ρ is taken as $\rho = \rho_{max}$, then an upper bound condition is defined. Similarly if $\rho = \rho_{min}$, then a lower bound condition is defined. Due to the dearth of supporting data, it will be assumed that the value of ρ is taken as the average values of ρ in the two directions of reinforcement, or

$$\rho = \frac{\rho_x + \rho_y}{2} \qquad (6)$$

The yielding shear strength of the steel reinforcement clearly governed the yielding shear stress of the reinforced concrete element. Again as was the case in defining the cracked shear stiffness above, the same problem will have to be solved in the case of nonisotropically reinforced concrete element. An empirical formula to calculate the shear yielding stress is then proposed. This formula is derived as a simplification of the two extreme cases which are the case of υ_o = **0.0**, and υ_o = **1.0**, in which υ_o is

$$\upsilon_o = \frac{\rho_{min}\, f_{symin}}{\rho_{max}\, f_{symax}} \qquad (7)$$

where

ρ_{min} =the minimum reinforcement ratio,
f_{symin} =yield stress of steel reinforcement with minimum ratio,
ρ_{max} =the maximum reinforcement ratio,
f_{symax} =yield stress of steel reinforcement with maximum ratio.

The resistance function for both cases as defined by Lenschow (Ref. 2) are used as a guide. The proposed formulae are

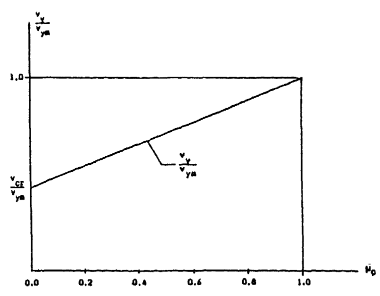


Fig. 3 Proposed Shear Yield Stress

$$v_y = v_{cr} + \upsilon_o(v_{ym} - v_{cr})\ v_{cr} \qquad (8)$$

$$v_{ym} = \rho_{max}\, f_{symax} \qquad (9)$$

It will be defined further that

$$v_{lim} \leq 0.83\sqrt{f_c'} \qquad (10)$$

where v_{lim} is the assumed theoretical shear stress limit. Equation 8 is illustrated in Fig. 3.

Vecchio's (Ref. 3) test results for the twenty two specimens which were loaded in "monotonically pure shear" stress are used to calibrate the model and to define the constant involved in Eq. **5**. Upon examining the comparison between the computed **G_{cr}** and the trend of the measured responses of the specimens, it is found that the best fit formula for **G_{cr}** is

$$G_{cr} = \omega \rho n G \tag{11}$$

where

$$\omega = 1.0 \quad \text{for } v_o \leq f_c'/8 \tag{12a}$$

$$= 0.7 \quad \text{for } v_o > f_c'/8 \tag{12b}$$

and

$$v_o = (\rho_x f_{sxl} + \rho_y f_{syl})/2 \tag{13}$$

$$f_{sxl} = f_{sxy} \leq 400 \text{ MPa} \tag{14a}$$

$$f_{syl} = f_{syy} \leq 400 \text{ MPa} \tag{14b}$$

Physically the complete proposed shear response as defined by Eq. 1 through 14b was represented in Fig. 1 described earlier.

MODEL FOR THE HYSTERITIC SHEAR RESPONSE OF R.C. PANELS

The above monotonic shear response model will be adopted as the basis of developing the hysteritic model for shear deformation. It is assumed that the envelope of the hysteritic curves are the same as the monotonic shear response curve. Vecchio (Ref. 3) showed that the unloading paths of his specimen PV-30, which was loaded in cyclic "pure shear", were straight lines merging at the vicinity of the origin, see Fig. 4. Based on this observed behavior, the hysteritic shear model as shown in Fig. 5 is proposed. Rules of the behavior of the proposed model are:

1. The envelope curves are define by Eq. 1 through 9, and Eq. 11 through 14b,
2. No shear strength limit is specified for the model,
3. Loading and unloading within the envelope curves follows a straight line connecting point O(0.0,0.0) and the maximum point in the envelope curve reached in the previous loading step,

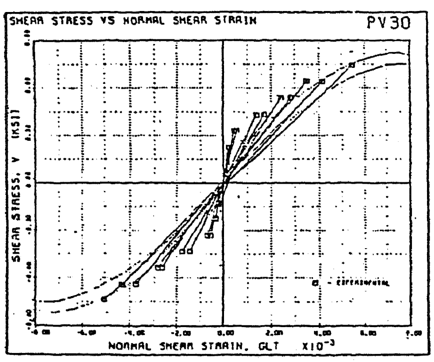


Fig. 4 Response of Specimen PV-30 Under Load Reversals

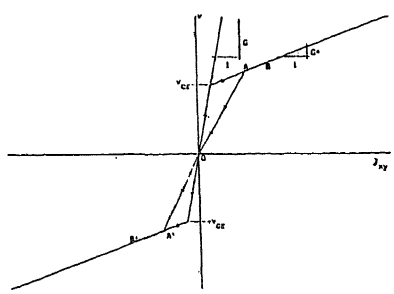


Fig. 5 Proposed Shear Response Model for Hysteritic Load

4. If the current maximum point in the envelope is reach and the loading continue, the rules defined in point 1 are followed.

In Fig. 5 point 3 of the above rules correspond to line OA and OA', while point 4 correspond to line AB and A'B'.

COMPARISON WITH TEST RESULTS

The test results of the twenty two Vecchio's specimens which were loaded in "pure monotonic shear" were used to calibrate the performance of the proposed monotonic shear response model. Figure 6 depicts comparisons of the measured and computed responses (Vecchio's and proposed model) of specimens PV-6 and PV-7.

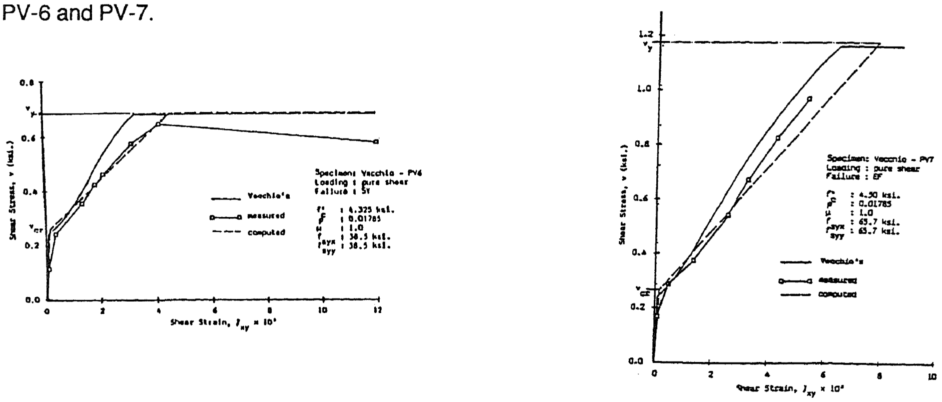


Fig. 6 Comparisons of Measured and Computed Shear Stress vs. Shear Strain Diagrams of Vecchio's Specimens

Figure 7 shows further comparisons between measured and computed shear stress vs. top deflection of Umemura's box-type and cylindrical-type specimens (Refs. 4,5). In these case the proposed shear response model were used together with the flexural response model where the effect of bar slip and nonlinear bending strain distribution were considered (Ref. 1).

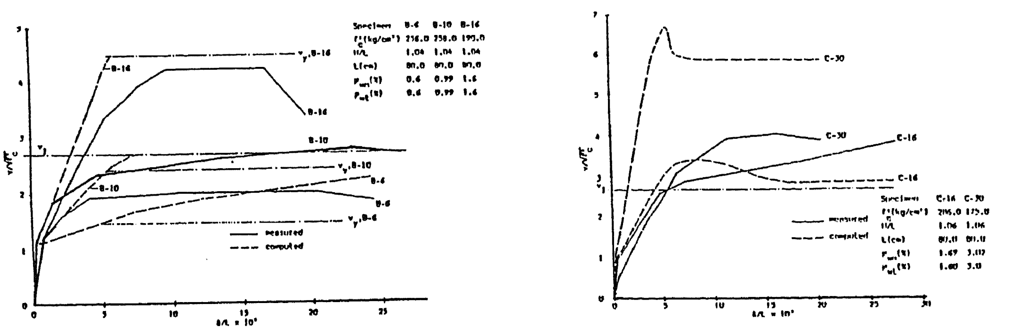


Fig. 7 Normalized Measured and Computed Shear Stress vs. Deflection Diagrams, Umemura's box and cylindrical-types specimens

Performance of the proposed hysteritic shear model, analyzed together with the hysteritic flexural model (Ref. 1), were examined by analyzing specimens tested by Wight (Ref. 6), Cervenka (Ref. 7), and Umemura et al (Refs. 4,5). Figure 8 shows result of the comparison of measured and computed Load-Deflection curves of Cervenka's shear panel specimen W-4.

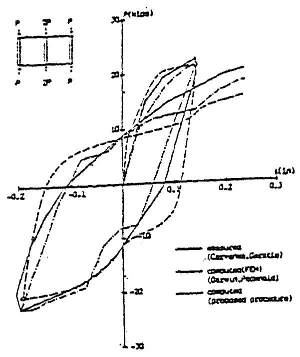


Fig. 8 Comparison of Measured and Computed Load-Deflection Curves od Cervenka's Shear Panel Specimen W-4.

Overall considering the simplicity of the approach used, the result of the comparisons between measured and computed responses of the specimens, for monotonic and hysteritic loading conditions, were good. In case of hysteritic loaded specimens, the returning slope of the shear model proposed however was conservative, suggesting that the observation reported by Vecchio from the test result of specimen PV-30 may not necessarily apply directly to condition in a reinforced concrete element subjected to bending as well as shear. Further improvement of the models may be obtained by refining the constants used in Eq. 1 through 14b to better fit experimental results, when available.

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