The Load-Deflection Behavior of Pier Specimen Cast with PPC Panels as Form Works

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1. Introduction

Research for the analysis of reinforced concrete structures are being carried out by using different complicated methods (e.g. 2-D or 3-D Analysis) with distributed crack, smeared crack approach, etc. These approaches generally consume lot of computational time. In this paper, simple method for analysis using sectional properties of moment-curvature relationship is used to developed a program for the analysis of reinforced concrete members. Displacement control algorithm with inelastic material properties are implemented. The unloading of both concrete and reinforcement is assumed such that no residual stress exist when strain becomes zero. This assumption is adopted as the first step of the implementation for simplicity. In future, more realistic unloading criteria will be implemented. This program is applied to compare with experimental results of a column[1,2] under cyclic loading, that had PPC panels used as form work. The PPC panel was made of high strength concrete (500kgf/cm^2) whereas the core was made of normal strength concrete(240kgf/cm^2). Analysis of both monotonic and cyclic cases are presented.

2. The Material Properties

For concrete, three different material properties were used as shown in Fig. 2 and 3. Perfect bond between PPC panel and core concrete has been assumed. For the PPC panel and the cover concrete, compression stress-strain diagram with less ductility is assumed and for the core concrete, compression stress-strain diagram with higher ductility is assumed. For reinforcement in tension lower apparent yield strength (f'_y = 90% of the actual yield stress) and higher slope ($0.01E_s$) has been assumed[3] to take care of tension stiffening effect. In compression, nominal slope ($0.001E_s$) is assumed after actual yield stress for stability in the analysis.

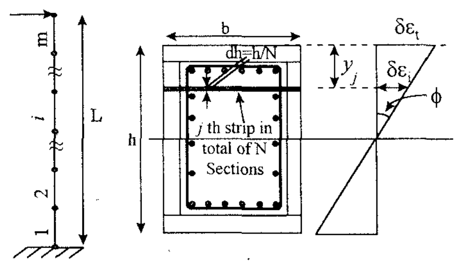


Fig. 1: Element and its Section Details

3. Analysis using Sectional Properties

Column, as shown in Fig. 1, of height L is divided into m elements. Element cross-section has been divided into N number of strips. On each ith node, deflection y_i, slope θ_i, curvature ϕ_i and bending moment M_i are to be calculated. For ϕ_i, M_i can be calculated from M-ϕ property of the cross-section. It is assumed that plain section remains plain before and after bending. Displacement control program is implemented, with displacement applied at the top of the column.

3.1 Stiffness Matrix Formation:

The strain in j_{th} strip of a section is

$$\delta\varepsilon_j = \delta\varepsilon_{t_i} - y_j \delta\phi_i \quad (1)$$

Applied axial force P_{ax} is assumed to be constant. Hence

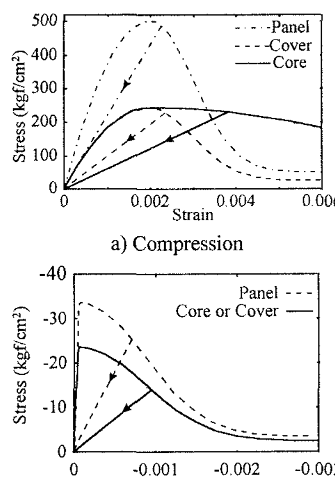
$$P_{ax} = \sum_{j=1}^{N} \sigma_j A_j \Rightarrow \delta P_{ax} = \sum_{j=1}^{N} \delta\sigma_j A_j = 0$$

$$\Rightarrow \sum_{j=1}^{N} \frac{\partial\sigma}{\partial\varepsilon_j} \delta\varepsilon_j A_j = 0 \quad (2)$$

Rearranging the terms,

$$\delta\varepsilon_{t_i} = \left(\sum_{j=1}^{m} \frac{\partial\sigma_j}{\partial\varepsilon_j} A_j \delta y_j \Big/ \sum_{j=1}^{m} \frac{\partial\sigma_j}{\partial\varepsilon_j} A_j \right) \delta\phi_i \quad (3)$$

From equilibrium of bending moment,



a) Compression

b) Tension

Fig. 2: Stress-Strain of Concrete

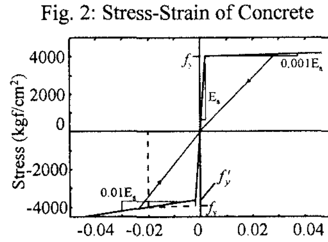


Fig. 3: Stress-Strain of Reinforcement

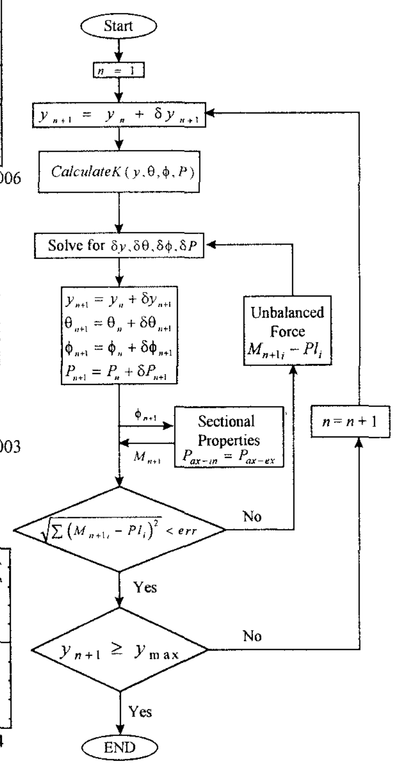


Fig.4: Program Flowchart

$$\delta M_i = \sum_{j=1}^{m} \delta\sigma_i A_j(\frac{h}{2} - y_j) = \sum_{j=1}^{m} \frac{\partial\sigma_j}{\partial\varepsilon_j} A_j(\frac{h}{2} - y_j)\delta\varepsilon_i \quad (4)$$

by substituting, Eq 1 and Eq 3 into Eq 4, we can simplify to $\delta M_i = B_i\delta\phi_i$. The balance of external and internal moments at the section($M_i = Pl_i$) in incremental form can be written as

$$\delta M_i - \delta Pl_i = B\delta\phi_i - \delta Pl_i = 0 \quad (5)$$

The deflection y, rotation θ and curvature φ in incremental form can be related as

$$\delta y_{i+1} - \delta y_i - \frac{\Delta l}{2}(\delta\theta_{i+1} + \delta\theta_i) = 0 \quad (6)$$

$$\delta\theta_{i+1} - \delta\theta_i - \frac{\Delta l}{2}(\phi_{i+1} + \phi_i) = 0 \quad (7)$$

Eq 5,6 and 7 are used to formulate the stiffness matrix. Fig. 4 shows the flow chart of the program. y_i, θ_i, ϕ_i are the nodal variables at each node and applied P is a global variable. Internal bending moments M_i can be calculated at each node for the calculated curvature ϕ_i based in internal equilibrium of axial force(Eq 2). Since this internal moment is calculated based on non-linear material properties with loading and unloading characteristics, unbalanced moment($M_i - Pl_i$) is taken as input for further iteration till the convergence between internal and external moment is achieved as shown in the flowchart(Fig. 4).

3.2 Comparison with Experimental results

Results of Specimen 4[1,2], as shown in Fig. 1, is taken for the analysis. A column had height 275cm and cross section of 45.0cm x 30.0cm with all longitudinal reinforcement of D13 and transverse reinforcement of D10. PPC panels with 3cm thickness were used as form work. Displacement controlled cyclic load was applied and results are shown in Fig. 5[1,2].

Fig. 6 and 7 shows the comparison with analytical results under monotonic and cyclic loading respectively with that of the envelop obtained from the experimental results. We can see that in both cases, the analytical results matches well with the experimental results. The difference between load-deflection diagram in the initial stage can be attributed due to the pullout effect of the reinforcement[3]. We can see that unloading paths in Fig. 7 passes zero load and deflection point. This is because of the assumption that no stress is retained when strain becomes zero for both concrete and reinforcement.

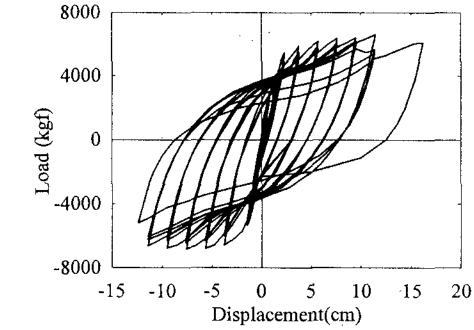


Fig. 5: Experimental Results for Cyclic Loading

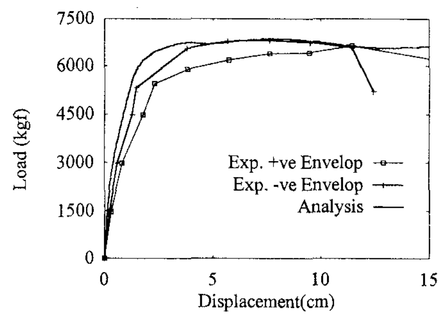


Fig. 6: Analytical results for Monotonic Loading

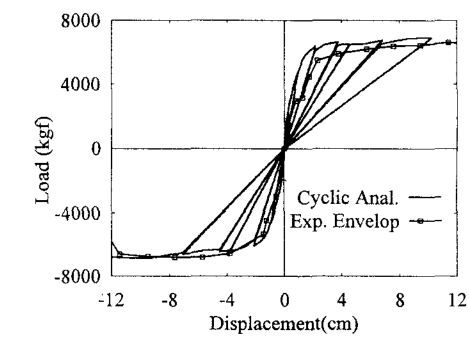


Fig. 7: Analytical results for Cyclic Loading

In the final stage, failure as observed in the experiment could not be obtained possibly because the buckling of reinforcement in compression is not properly simulated as observed in the experiment. Adoption of more realistic curve for reinforcement in tension after ultimate strain might also provide better results in the final stage.

4. Conclusions

From analytical results shown in Fig. 6 and 7, it can be concluded that simple method of analysis using sectional properties with proper implementation of confinement of concrete, tension-stiffening effect, unloading path can simulate good results for cases of bending failure. However, it could not simulate the failure in the final stage. The cyclic behavior as simulated in Fig. 7 is different from the experimental results shown in Fig. 5. At present work on implementation of advanced inelastic unloading paths to simulate more realistic cyclic behavior for reinforced concrete elements is in progress.

References

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