

Research on Hysteretic Behavior of Steel Reinforced Concrete Frame with Angle-Steel Concrete Columns

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Abstract—This paper describes a theoretical research on the hysteretic behavior of steel reinforced concrete (SRC) frame structure with angle-steel concrete columns based on the Park tri-linear model. The theoretical results were compared with tests, and main parameters effecting on the mechanical performance of the SRC frame were observed. It is indicated that the program IDARC-2D could simulate the whole loading process, and could provide a suitable method for the hysteretic behavior simulation.

Keywords—hysteretic behavior; steel reinforced concrete beam; angle-steel concrete column; frame; numerical model

I. INTRODUCTION

In order to ensure the normal service of building in reconstruction for adding storeys, a new type of steel reinforced concrete (SRC) frame with angle-steel concrete (ASC) columns was proposed by Wang [1].

At present, the hysteretic behavior analysis of this kind of innovative SRC frame under low cyclic loading is relatively rare [2-4]. In this paper, the hysteretic behavior of the SRC frame with angle-steel reinforced concrete columns were conducted by soft IDARC-2D, and the calculated hysteretic curves were obtained and compared with testing results. In addition, the impacts of the main parameters on mechanical performance of SRC frame were studied. The results provide a suitable method for simulating hysteretic behavior of such SRC frames.

II. TESTING DESCRIPTION

An one-storey and one-span specimen for SRC frame with angle-steel concrete column is shown in Figure 1, in which column shear ratio is 5, and the axial pressure ratio is 0.1. The principle of strong-column-weak-beam is adopted in design. The beam and column connection was designed according to the Reference [5], in which two short I-shaped steel anchors were welded on both sides of the steel beam flanges to avoid bond-slip failure.

The concrete with grade C40 for the SRC frame specimen was constructed in two stages, first from the bottom to the height of 2.5m and second the rest. Hence, the concrete cube strength for the columns and beam (including connection) were 47.9 N/mm² and 69.8 N/mm² respectively. The tensile yielding strength for angle-steel, I-shaped steel and steel plate stirrup is 287.0 N/mm², 302.3 N/mm² and 312.0 N/mm², respectively. The tensile yielding strength for the reinforced bars with

diameter of 10mm and 12mm was 348.7 N/mm² and 338.6 N/mm², and tensile yielding strength for stirrups with diameter of 10mm is 303.0 N/mm².

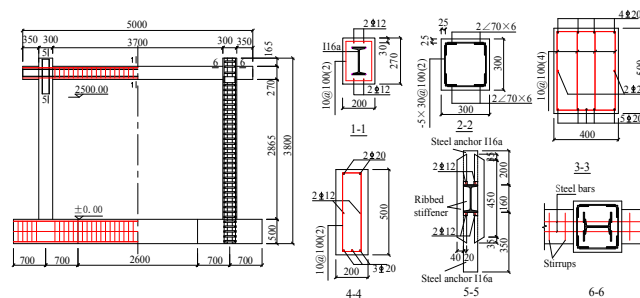


FIGURE 1. REINFORCEMENTS OF TWO SPECIMENS (UNITS: MM)

Fig.2 showed the test set-up. The specimen frame was fixed on the ground, and constant vertical concentrated load affected on each column is 300kN, and two concentrated loads with each value of 40 kN were applied on the beam. The horizontal low cyclic loading was applied, and load-displacement hybrid control rules were adopted.

III. HYSTERETIC BEHAVIOR ANALYSIS

Nonlinear soft IDARC-2D was adopted in this paper to simulate the hysteretic behavior of the SRC frame specimen under low cycle loading. In this analysis, some basic assumptions were given as follows: (1) the cross-section for beams and columns remains plan before and after bending; (2) reliable bond between steel bars as well as steel-shapes and concrete; (3) the bending damage was prior to shear damage for beam and columns; (4) the failure of the beam and column connection is ignored, and the shrinkage or creep of concrete were not taken into consideration.

A. Analytical Model

The frame structure could be discrete into beam and column components, and beam-column element model could be adopted for structural analysis based on the nonlinear program IDARC-2D. Therefore, the beam-column element with concentrated plastic hinges could be utilized for the element model.

Due to the specific loads applied on the frame structure, the position of the potential plastic hinges for beam and column could be identified, and the nodes for the analytical model

could be arranged at this position of the potential plastic hinges. The analytical model for the frame structure was shown in Fig. 3. Due to the fact that the steel anchors were arranged in top of the columns about 430mm, this part column could be regard as rigid zone for simplification. The frame column is divided into only one element along the whole length; otherwise, the beam could be divided into four elements according to the loads distribution and geometrical characteristics. The nodes were set in the vertical concentrated load point and the mid-span. In Fig.3, the "+" represents the node.



FIGURE II. TEST SET-UP

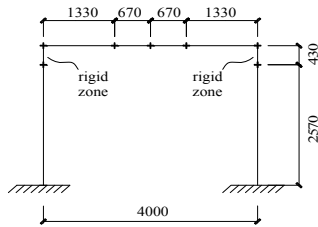


FIGURE III. ANALYTIC MODEL

B. Constitutive Relationship of Materials

The stress-strain relationship of concrete was adopted as Expression (1), as shown in Fig4. Where, E_c is the initial elastic model of concrete; ϵ_0 is the concrete stress corresponding to the peak stress; ϵ_{cu} is the ultimate compressive strain; f_c is the compressive strength of the concrete; f_t is the tensile strength of the concrete.

The ideal elastic-plastic model is adopted for steel bars, I-shaped steel and angle-steel, as shown in Figure 5 and the Expression (2). Where f_y and f'_y are the tensile and compressive strength, respectively.

C. Hysteretic Model for Beam and Column

Park tri-linear model provided by IDARC-2D was utilized for the frame beam and column cross-section, where the initial cross-section stiffness, the positive and negative crack moment and curvature, as well as yielding ones, and cross-section stiffness after yielding could be given. In addition, stiffness degradation parameter α , strength degradation parameters β_1, β_2 and slip degradation parameters γ should be given for hysteretic model. The corresponding parameters are shown in Table 1 according to previous analysis and calculation.

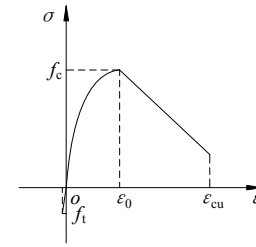


FIGURE IV. STRESS-STRAIN RELATIONSHIP OF CONCRETE

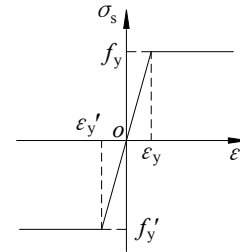


FIGURE V. STRESS-STRAIN RELATIONSHIP OF STEEL

$$\sigma_p = \begin{cases} E_c \epsilon_c & f_t / E_c < \epsilon_c < 0 \\ f_c \left[\frac{2\epsilon_c}{\epsilon_0} - \left(\frac{\epsilon_c}{\epsilon_0} \right)^2 \right] & 0 \leq \epsilon_c < \epsilon_0 \\ f_c \left[1 - 0.15 \left(\frac{\epsilon_c - \epsilon_0}{\epsilon_{cu} - \epsilon_0} \right) \right] & \epsilon_0 \leq \epsilon_c < \epsilon_{cu} \end{cases} \quad (1)$$

$$\sigma_s = \begin{cases} -f_y & \epsilon_s < \epsilon'_y \\ E_s \epsilon_s & \epsilon'_y \leq \epsilon_s \leq \epsilon_y \\ f_y & \epsilon_s > \epsilon_y \end{cases} \quad (2)$$

TABLE I. DEGRADATION PARAMETERS FOR CROSS SECTION

Degradation parameters	SRC beam	ASC column
α	4	10
β_1	0.15	0.35
β_2	0.15	0.15
γ	1.0	1.0

D. Comparison of Test and Calculated Results

The hysteretic curves for the SRC frame was calculated by IDARC-2D, and the calculated P-Δ curves were compared with the test results, as shown in Fig. 6. Comparison of the P-Δ skeleton curve were shown in Fig.7. It can be seen that calculated results were in good agreement with test results:

- During the whole loading process, the initial stiffness of the calculated curves is in accord with the test results, and difference between the test and calculation of peak load is less than 5%.
- The loading stiffness, unloading stiffness and residual deformation of the simulation are close to the tests, and it indicated that the selected degradation parameters are reasonable.

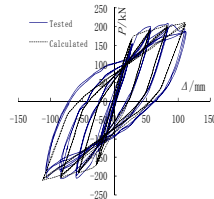


FIGURE VI. COMPARISON OF HYSTERETIC CURVES

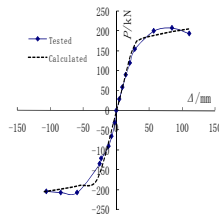


FIGURE VII. COMPARISON OF SKELETON CURVES

E. Plastic Hinges

The sequence of plastic hinges on the end of beams and columns under lateral loadings can be get by the program IDARC-2D, shown in Fig.8.

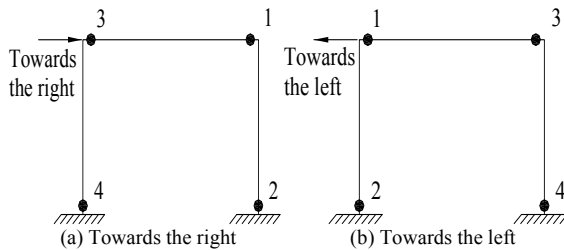


FIGURE VIII. DISTRIBUTION OF PLASTIC HINGES IN TWO FRAMES

IV. PARAMETER ANALYSIS

In order to study the seismic performance of this innovative SRCF frame, $P-\Delta$ skeleton curves were observed in some aspects, such as the column height to the beam span (H/L), axial compression ratio (n_0), the steel ratio of column (ρ_{cs}), the steel ratio of beam (ρ_{bs}) and the longitudinal reinforcement ratio of beam (ρ_{bs}).

Fig.9 gives the influence of H/L on $P-\Delta$ skeleton curves. It is indicated that when the span is constant, H/L increases, ductility increases, while the structural stiffness and peak load decreases. The influence of n_0 on $P-\Delta$ skeleton curves is shown

in figure 10. It illustrates that peak load increases with n_0 increases, while displacement corresponding to peak loads and ductility reduces. From Fig 11 to 12, as can be seen that initial stiffness on $P-\Delta$ skeleton curves are almost the same, peak loads increase and ductility have no obviously change obviously along with the ρ_{cs} , ρ_{ss} and ρ_{bs} increase.

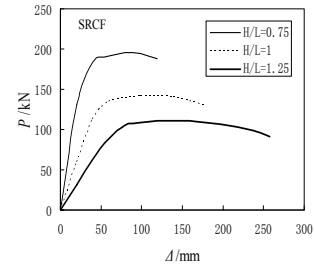


FIGURE IX. INFLUENCE OF H/L ON $P-\Delta$ CURVES

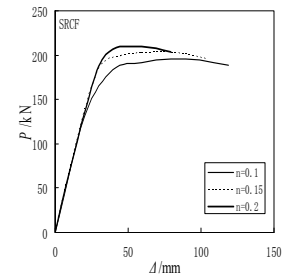


FIGURE X. INFLUENCE OF n_0 ON $P-\Delta$ CURVES

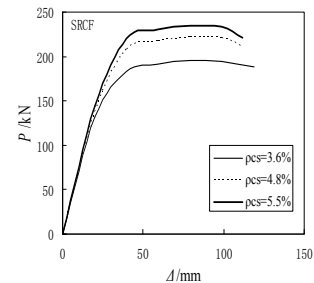


FIGURE XI. INFLUENCE OF PCS ON $P-\Delta$ CURVES

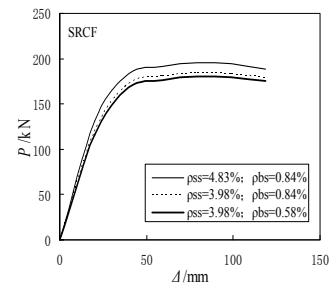


FIGURE XII. INFLUENCE OF PSS AND PBS ON $P-\Delta$ CURVES

V. CONCLUSIONS

A numerical investigation on the hysteretic behavior of SRC frame with angle-steel concrete column is conducted. The theoretical result is compared, and some conclusions are drawn:

- Numerical hysteretic analysis of SRC frame conducted by the program IDARC-2D is in accordance with test results. It is indicated that the numerical model built by IDARC-2D in this paper could be applied to analysis the mechanical performance of this innovative frame.
- Parameter analysis indicated that structure stiffness decreases, peak load reduces and ductility increases along with H/L increases. When the axial compression ratio n_0 increases, the peak loads increase, displacements corresponding to the peak loads reduces and ductility decrease. Peak loads increase and ductility have no obvious change when the steel ratio of column ρ_{cs} , the steel ratio of beam ρ_{bs} , and the longitudinal reinforcement ratio of beam ρ_{bs} increase.
- The analytical method utilized in this paper could provide reference to investigate hysteretic behavior for other similar frames.

VI. ACKNOWLEDGEMENTS

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