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ARABIC SECTION

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NONLINEAR SEISMIC ANALYSIS OF HIGH STRENGTH CONCRETE STRUCTURES

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ABSTRACT

This research deals with studying the response of High Strength Concrete (HSC) structural elements subjected to transverse forces simulating seismic actions. The analysis of such elements necessitated further development of the Inelastic Damage Analysis Program Code "IDARC3" in order to predict the behavior of HSC elements. The authors developed a modified version of the program, titled, "IDARC-M" in this investigation. The program was validated by comparing the obtained hysteretic load deformation curves with the experimental results of different structural elements published in the literature.

INTRODUCTION

Although the use of HSC with compressive strength higher than 500 kg/cm^2 has become more common in reinforced concrete members, especially in columns, there is a difference of opinion on the behavior of HSC structures in seismic risk areas due to the limited number of researches [1]. During the last few years, extensive experimental and analytical work has provided a better understanding of the behavior of HSC [2 and 3]. The use of HSC in building frames construction is particularly attractive due to many advantages such as: increased stiffness and strength of columns, reduced column size, more durable material and enhanced construction economy [4].

Since the occurrence of the 1992 earthquake in Egypt, most of structural engineers focused on the effect of earthquakes on multistory R.C. building frames [5]. Present codes recommended a "strong column-weak beam" design philosophy to minimize the probability of structural collapse or loss of structural serviceability [6]. Design guidelines for earthquake resistant R.C. buildings based on ultimate strength concept for normal strength materials include proportioning equations with empirical constants. However, for HSC elements, it is not so rational to establish the guidelines for the members without analytical understanding of the shear and flexural resistance mechanism for such members [7]. Several computer programs for seismic analysis of R.C. structures are available in the market. Among those, the enhanced computer program IDARC3 is capable of predicting the response of building frames during earthquakes [8].

The program idealizes the building as a series of plane frames linked together by floor slab and transverse beams using 2-dimensional stiffness method in order to clarify the flexural and shear mechanisms of the studied members [8].

The objective of this paper is to further develop the computer program IDARC3 in order to be capable of predicting the seismic response of both normal strength and HSC elements such as columns, beams and shear walls. The modified version "IDARC-M" has been verified through simulation of experimentally recorded behavior of three different HSC structural elements.

ANALYTICAL MODELING

IDARC3 Computer Program

IDARC3 is a computer program for two-dimensional analysis of 3D building systems in which a set of frames parallel to the loading direction are inter-connected by transverse elements to permit flexural-torsional coupling [8]. The library of the program includes different element types such as beam-column elements, shear walls, inelastic axial elements, transverse

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beams and discrete spring elements. Details of the element types that currently exist in the IDARC library can be found in the program user manual [8].

Moment-Curvature Envelopes

The moment-curvature analysis used in the program is adopted from Mander [9]. The analysis is carried out on the cross-section by dividing the concrete area into a number of layers or fibers. Steel areas and their respective locations are identified separately. Figure (1) shows a typical rectangular section subjected to a combination of an axial load and a moment. The strain at any section is given by:

$$\varepsilon(z) = d\,\varepsilon_o + z\,d\varphi \quad (1)$$

where $d\varepsilon_o$ is the centroidal strain, z is the distance from the reference axis and $d\varphi$ is the curvature of the cross-section. The complete procedure for developing the moment-curvature envelope is detailed in the program user manual [8]. It is worth mentioning that the effect of hoop spacing on column capacity of circular sections is included in the program. It is assumed that the capacity of the column remains unchanged after the concrete cover has spalled. Hence,

$$0.85\, f_c'\, A_g = f'_{cc}\, A_{cc} \quad (2)$$

where f_{cc} is the confined compressive strength, A_{cc} is the area of concrete core and A_g is the gross concrete area. Park and Paulay, [10] gave an expression relating confined to unconfined strength of concrete, which was based on the confining stress relation of Richart et al, [11]:

$$f'_{cc} = f_c' + 2.05\,\rho_s\, f_y \quad (3)$$

where ρ_s is the volumetric ratio of confinement steel to core concrete, given by:

$$p_c = \frac{A_h JId_c}{SA_{cc}} \quad (4)$$

where A_h is the cross-sectional area of the hoop steel, d_c is the diameter of the concrete core, and S is the spacing of hoops. The modified compressive stress of concrete is finally obtained from substitution of Equation (3) into Equation (2):

$$f'_{cm} = \frac{(f_c' + 2.05 p_s f_y)A_{CC}}{0.85A_g} \quad (5)$$

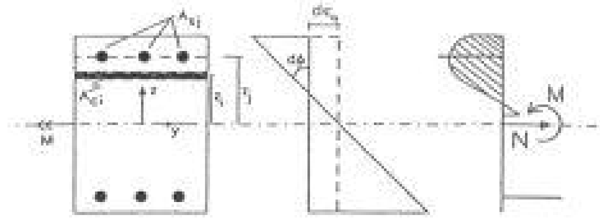


Fig. 1- Section detail for fiber model analysis [8].

Material Modeling

Concrete

Program IDARC3 was originally developed for normal strength concrete. The stress-strain relationship for unconfined concrete is shown in Figure (2). The relationship for concrete in tension is linear till cracking. After cracking, tension cut-off was assumed, and the stiffness normal to a crack direction was set to be zero. The only factor that considered influencing the ultimate deformation capacity of the section is the degree of confinement. Since confinement does not signify-cantly affect the maximum compressive stress, the original program only considers the effect of confinement on the downward slope of the concrete stress-strain curve (see Figure 2). The effect was expressed by Kent and Park [12] as the factor Zf to define the descending branch and was included in the program:

$$Zf = \frac{0.5}{\varepsilon_{50\mu} + \varepsilon_{50h} - \varepsilon_o} \quad (6)$$

where:

$$\varepsilon_{50\mu} = \frac{3.0 + \varepsilon_o f_c'}{f_c' - 1000.0}$$

$$\varepsilon_{50h} = 0.75 p_s \sqrt{\frac{\bar{b}}{s_h}}$$

in which $\bar{b}$ is the width of the confined core, and s_h is the spacing of hoops. The effect of introducing this parameter to define the descending branch of the concrete stress-strain curve is to provide additional ductility to well confined columns. Improved formulations for stress-strain behavior of confined concrete can be found elsewhere [13].

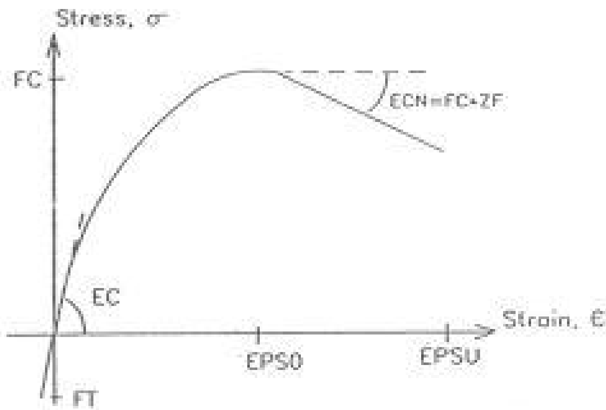


Fig. 2- Stress-Strain curve for unconfined concrete [8].

Reinforcement

The longitudinal and lateral reinforcement in columns, beams and walls were assumed to be a linear element. The stress-strain relationships of the longitudinal and lateral reinforcement were assumed to be bilinear and trilinear, respectively (see Figure 3).

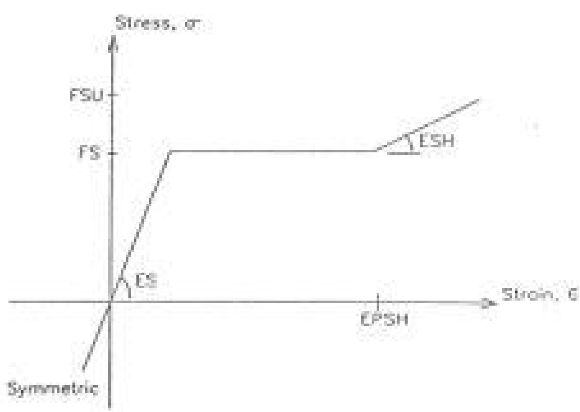
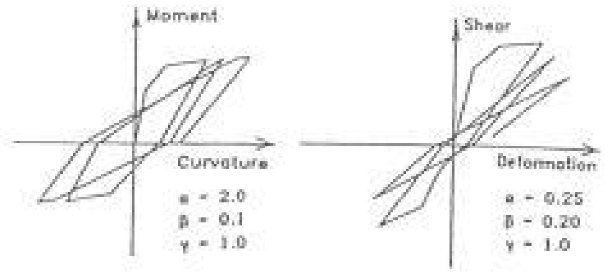


Fig. 3- Stress-Strain curve for reinforcing bars [8]

Hysteretic Response Modeling

The hysteretic model for the analysis includes three parameters in conjunction with non-symmetric trilinear curve to establish values under which inelastic loading reversals take place [14]. The three main parameters represented in the model are stiffness degradation α, strength deterio-ration β and bond-slip or pinching γ. These para-meters can be combined in various ways to achieve a wide range of hysteretic behavior patterns (Figure 4). Details of hysteretic para-meters formulation are outlined elsewhere [8].



(a) Flexural spring (b) Shear spring

Fig. 4- Three parameter hysteretic modeling of beam-columns and walls [14]

Further Development of Material Modeling "IDARC-M"

The authors modified the program in order to be capable for the analysis of HSC structures. A model developed by Collins et. al. [15] was used for the ascending part of the compressive stress-strain relationships as follows:

$$\sigma_C = \sigma_B \cdot \frac{\varepsilon_C}{\varepsilon_B} \cdot \frac{n}{n-1+(\varepsilon_C/\varepsilon_B)^{nk}} \quad (7)$$

where;

σ_C = stress in concrete (kgf/cm^2)

σ_B = compressive strength of concrete (kgf/cm^2)

ε_C = strain in concrete

ε_B = strain in concrete at the compressive strength

k equals 1 when $\varepsilon_C / \varepsilon_B$ is less than 1 and when $\varepsilon_C / \varepsilon_B$ exceeds 1,

Collins and Michell [16] suggested that:

$$k = 0.67 + \sigma_B/620 \ \ (\text{kg/cm}^2) \quad (8)$$

and that

$$n = 0.8 + \sigma_B/170 \ \ (\text{kg/cm}^2) \quad (9)$$

If the initial stiffness of concrete, E_c of the stress-strain curve is known or can be estimated, the strain at peak stress, ε_B can be found from:

$$\varepsilon_B = \frac{\sigma_B}{E_c} \cdot \frac{n}{n-1} \quad (10)$$

While E_c depends strongly on the stiffness of the aggregates being used, Carrasquillo et al. [17] suggested that:

$$E_c = 10500\sqrt{\sigma_B} + 69000 \quad (\text{kg/cm}^2) \quad (11)$$

The ultimate strain, ε_u can be obtained as follows:

$$\varepsilon_u = \frac{0.8}{Zf} + \varepsilon_B \quad (12)$$

The stress-strain curves that result from the above equations for a range of concrete strengths are shown in Figure 5. Confining effect by the lateral reinforcement on the compressive descending part of the stress-strain relationships was represented by Kent-Park model as described previously [12 and 18].

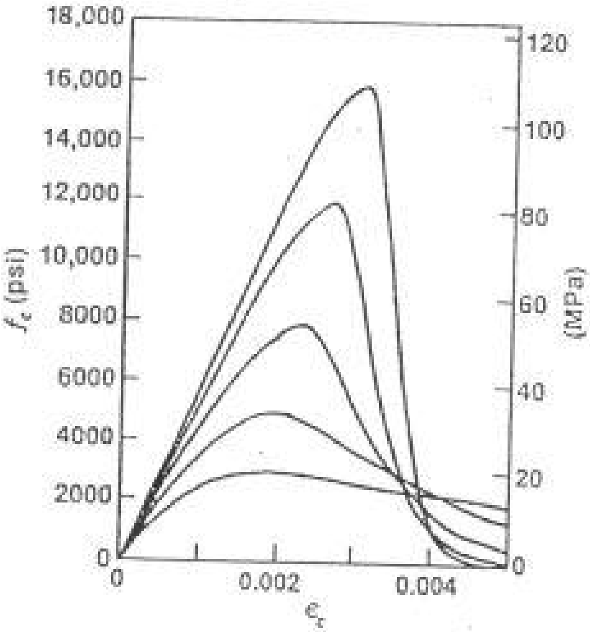


Fig. 5- Influence of concrete strength on shape of stress-strain relationship [15]

VALIDITY OF THE MODIFIED PROGRAM "IDARC-M"

In order to verify the modified model, three different structural elements experimentally tested in the literature were analyzed. This section presents the studied cases, emphasizing geometric and material descriptions, the input excitation and selected results.

Behavior of HSC Columns under Axial Loads and Simulated Seismic Loads

Eight square columns representing base story columns under combined constant compressive axial load and increasing lateral cyclic load were experimentally tested in order to investigate the behavior of HSC columns confined by rectilinear ties and subjected to different loading conditions [19]. The main variables were the concrete compressive strength, the amount of transverse reinforcement, the yield strength of transverse reinforcement, the tie configuration, the level of applied axial load and the history of the applied lateral load. Three column specimens were chosen for the analytical modeling by IDARC-M, namely, C1, C4 and C5. Details of the studied specimens are shown in Figure 6. All specimens had identical longitudinal steel with a ratio equal to 2.26%, nominal yield strength of 3600 kg/cm^2 and lateral ties 10 mm bar diameter every 5 cm, except C4 had lateral ties of 12 mm diameter every 5 cm. Two different configurations of ties were used; type 1 for columns C1 and C5 and type 2 for column C4 (see Figure 6). Specimen C5 was tested under lateral load only, while all other specimens were subjected to axial load equals 12% of the nominal axial strength in addition to a lateral load history as shown in Figure 7.

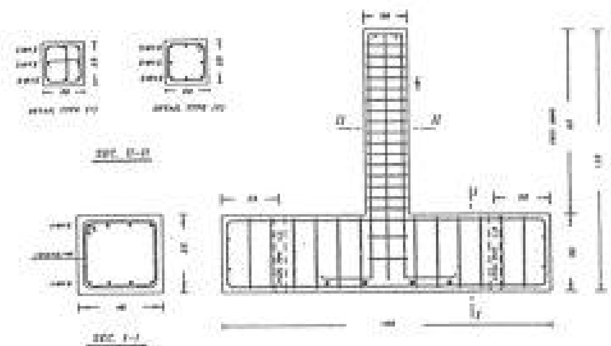


Fig. 6-Details of specimens reinforcement [19].

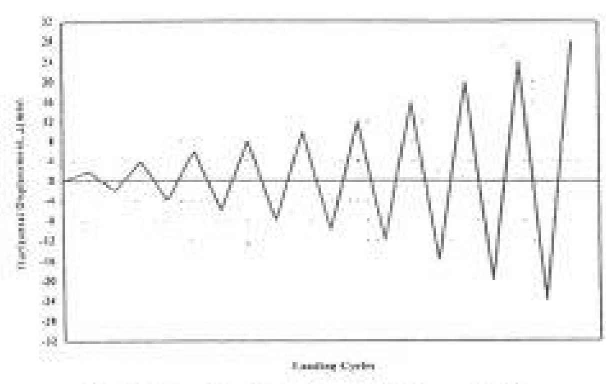
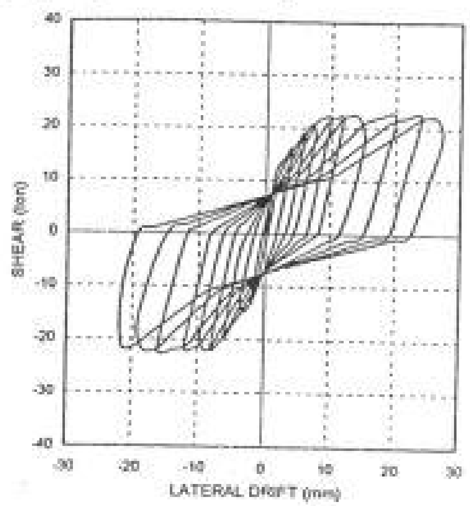


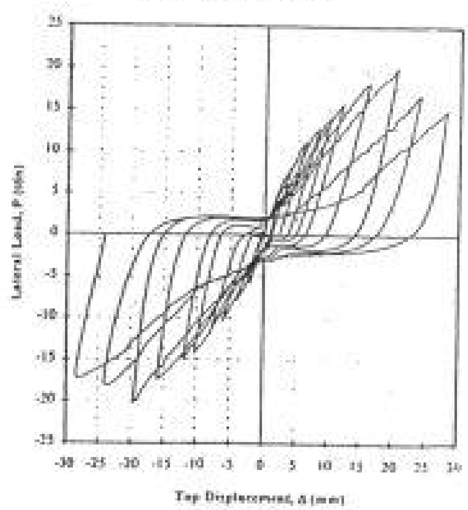
Fig. 7- Nominal lateral loading history [19].

Figures (8-a), (8-b) and (8-c) illustrate the load displacement hysteresis loops obtained from both test results and the computer analysis for the three chosen specimens. The figures show that the axial load ratio had a significant influence on the performance of high strength concrete columns under seismic loading. For example, the capacity of specimen C1, tested under axial stress ratio of 12%, was higher than that of specimen C5, which was tested under lateral load only, by 35%

approximately. In addition, it can be noticed from the results of specimens C1 and C4 that tie configuration has a negligible effect on the strength and deformability of these specimens which were subjected to moderate axial compressive forces. This is similar to the observations reported by Thomsen and Wallace [20]. However, Figures (8-a) and (8-b) show that specimen C4 displayed significantly higher strength decay rate than that of specimen C1. It can be argued that specimen C4 had no cross ties "Type 2 tie configuration". It can be seen from Figure 8 that, generally, the analytical results are in a good agreement with those obtained experimentally. However, the analytical results are slightly higher than the experimental ones.

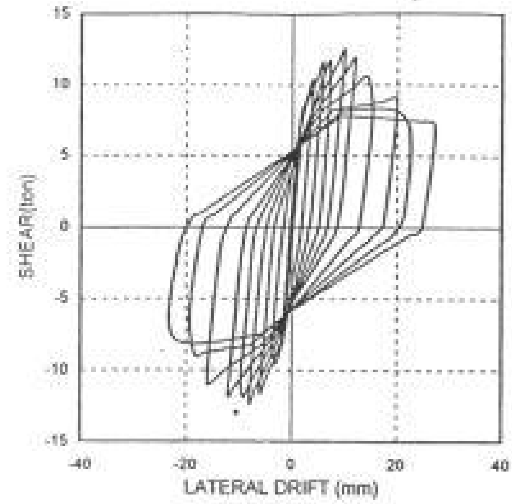


a) Analytical results

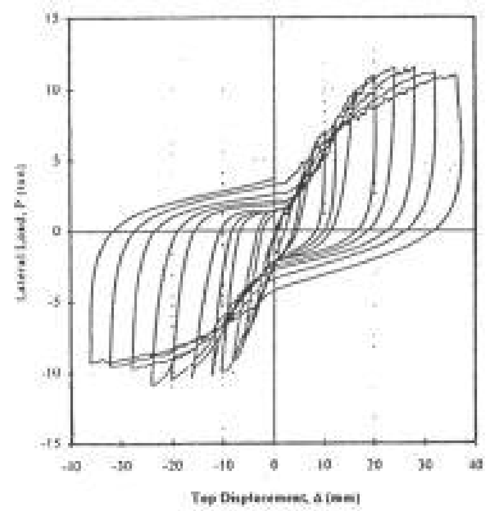


b) Test results

Fig. (8-a) - Load displacement loop for specimen C1.



a) Analytical results

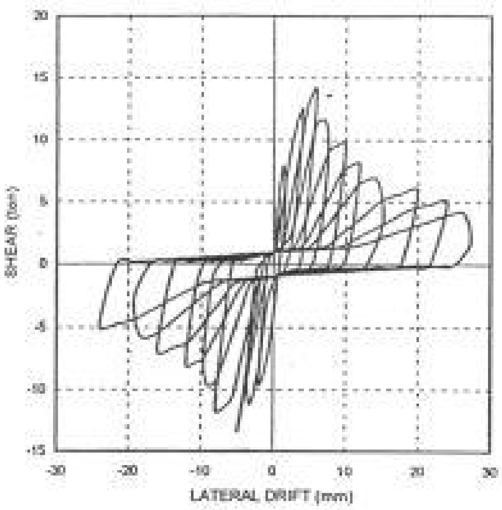


b) Test results

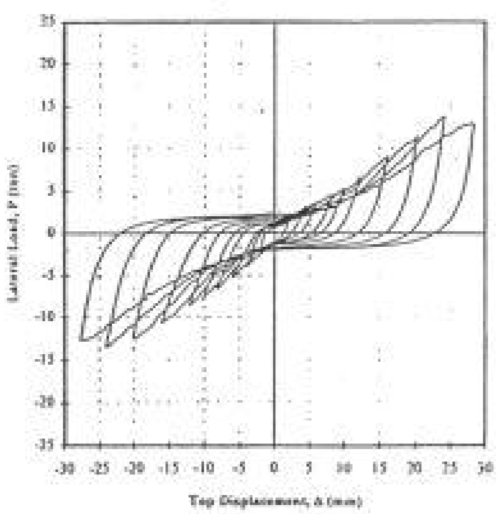
Fig. (8-b)- Load displacement loop for specimen C4.

Seismic Behavior of HSC Slender Wall under High Axial Load

An experimental investigation was carried out to examine the behavior of five wall specimens with barbell shaped cross section under reversed cyclic loading [21]. Sizes of boundary elements and wall thickness are illustrated in Figure 9. The studied variables were concrete strength (40 and 80 MPa), axial load (1800, 1300 and 800 KN), yield strength of longitudinal bars in boundary elements (400 and 600 MPa) and tie spacing of boundary elements (50 mm and 35 mm). Loading history employed consisted of displacement angle of one cycle of ±0.002, 0.0033, 0.005 and 0.0075 followed by two cycles of ±0.01, 0.015, 0.02 and



a) Analytical results



b) Test results

Fig. (8-c)- Load displacement loop for specimen C5.

0.03. Two specimens (W8N13 and W8N08H) were analyzed by IDARC-M in order to check its accuracy and validity. The first specimen, W8N13, had an axial load of 1275 KN and tie spacing of 50 mm, and the second one, W8N08H, had an axial load of 785 KN and tie spacing of 50 mm also.

Figures (10-a) and (10-b) show the load displacement hysteresis loops obtained from both test results and the computer analysis for the chosen specimens. Kimura and Sugano [21] found experimentally that there is an increase of the displacement with the reduction of axial load for the same concrete strength. However, the second specimen "W8N08H" subjected to a lower axial load exhibited smaller displacement than that of

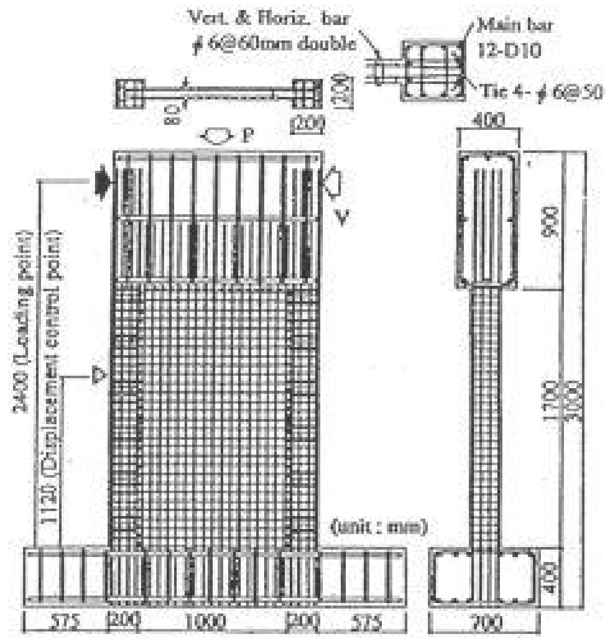


Fig. 9- Details of test specimen (W8N13) [21].

the first specimen "W8N13" subjected to a higher axial load (1300 kN). This may be attributed to the high strength of the main steel bars used in the boundary elements of specimen W8N08H which lead, in turn, to a load drop as a result of fracture of such steel. The experimental and analytical results are listed in Table (1). Figure 10 and Table (1) show that the results obtained from IDARC-M are in a good agreement with those obtained experimentally by Kimura and Sugano [21].

Table 1- Results of HSC Slender Wall under High Axial Load

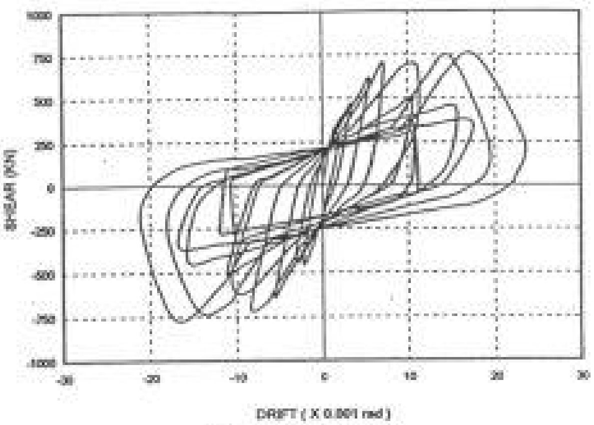
Specimen Number	Experimental Results		IDARC-M Results	
	Load (KN)	Displacement * (0.001 rad)	Load (KN)	Displacement * (0.001 rad)
Specimen (1) "W8N13"	763	15	763	11.12
Specimen (2) "8N08H"	689	15	703.9	12.5

*Displacement obtained at 80% of the maximum load

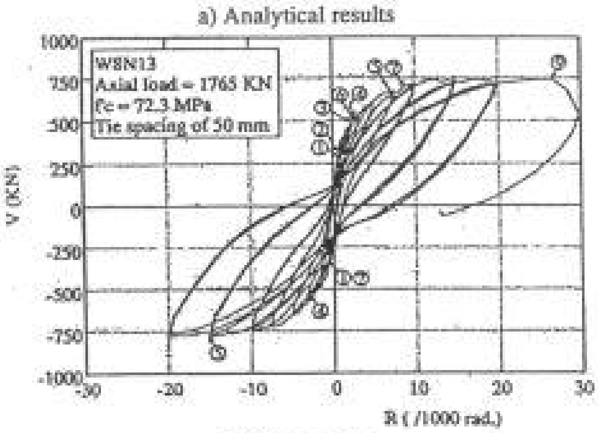
CONCLUSION

A summary of the major modification and enhancements to the computer model, IDARC 3, was presented. The modified version "IDARC-M" was developed in this study to analyze HSC structural elements under static and reversed cyclic loads. The proposed material model for HSC was easily linked to the original package.

The good agreement between the results obtained from the modified version and experimental results established the validity and

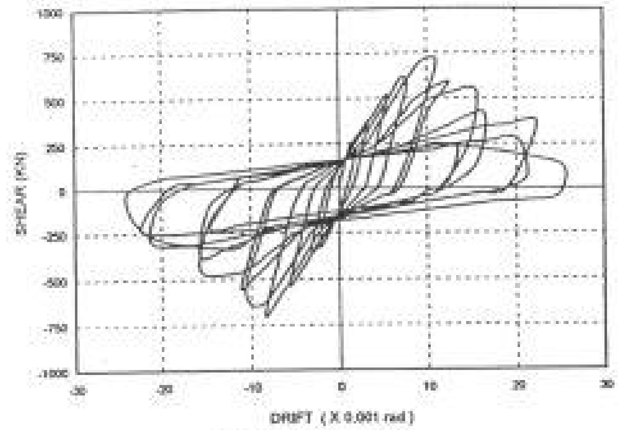


a) Analytical results

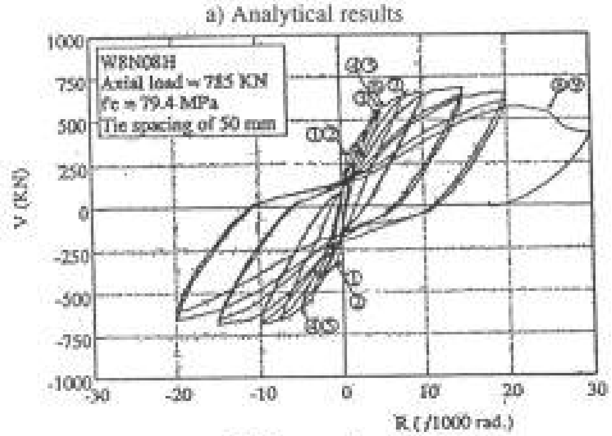


b) Test results

Fig. (10-a)- Load displacement loop for specimen No. 1.



a) Analytical results



b) Test results

Fig. (10-b)- Load displacement loop for specimen No. 2.

capability of "IDARC-M" for predicting the seismic behavior of HSC structural elements. Within the scope of the current investigation, the analytical initial stiffness of restoring force was higher than experimental one. This may reveal the need for further calibration of the current hysteretic model parameters, especially stiffness, strength and degradation of HSC.

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