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BEHAVIOR OF SHEAR WALLS UNDER AXIAL LOAD AND CYCLIC LOAD

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Reinforced concrete walls are often introduced into multistory buildings to resist lateral forces which can be exists due to winds or earthquakes. In the present study, six models I–section shear wall models were tested under combined action of a constant axial load and reversal horizontal increased loading until failure. The study has investigated the effects of some parameters such as, the height – to – width ratio, the compressive strength of concrete, and the variation of main flexure reinforcement ratio on the behavior of high - strength concrete shear walls. The obtained results from the tests have helped to identify the causes of wall failure modes. The test results included the determination of ultimate load, deflection, mode of failure, crack pattern, ductility, stiffness, and energy absorption.

KEYWORD: *RC design, High strength concrete, Seismic analysis, Cyclic loading*

INTRODUCTION

In the early days of using reinforced concrete, structural walls were mainly designed and analyzed as wide columns. During the 1950's experimental studies were focused on shear behavior and axial load carrying capacity of the shear walls. All portions of a shear wall should be designed to resist the combined effects of axial load, bending, and shear determined from a rational analysis of the structural system.

The present study adds more information about the behavior of high-strength reinforced concrete shear walls under the effect of lateral and vertical loads for different steel ratios as well width to height ratios. A comparison between structural behavior of high strength and normal strength concrete shear walls is presented.

High - strength concrete (HSC) is considerably a new material, but in recent years it is widely used in deferent members of structural buildings, such as columns, platforms, beams, and walls, etc.

HSC may has properties deferent from the properties of normal - strength concrete, such as higher concrete compressive strength and the corresponding increase of tensile strength.

Shear failure of high strength concrete occur suddenly since it is to some extent not ductile material, hence, the traditional amounts of minimum shear reinforcement presented by codes may be not sufficient for high strength concrete walls. More studies are still needed to verify the use of structural elements constructed with high-strength concrete type.

1. TEST PROGRAM

1.1 Description of test specimens:-

Present study included six specimens, divided to two groups, the first group of specimens contains 3 specimens which are identified as HL–1, HL–2, and NL, and the second group contains 3 specimens which are identified as HS-1, HS-2, and NS. Each group has a different height and the cross sections for all specimens were **I** section as shown in Fig.(1).

The letter (H) indicates high-strength concrete (HSC), and letter (N) indicates normal strength concrete (NSC). For the first group, letter (L) indicates to the longest height of the two groups and is equal to 1100 mm. The second group, letter (S) indicates short height and is equal to 740 mm.

The normal concrete specimens NL, and NS were used as reference specimens to make a comparison between (NSC) and (HSC) shear walls.

Using numbers 1 and 2 in the specimen's names indicate the geometrical percentage of reinforcement, which are equal to 2.26% and 3.39%, respectively. Geometrical percentage can be defined as:

$$\rho_f = A_{sf} / A_c$$

Where:

A_{sf} is the area of vertical reinforcement of specimens flange, A_c is cross sectional area of the wall specimen, and b and t are breadth and thickness of flange, they are constant for all specimens, and are equal to 100 mm and 200 mm, respectively.

f_y is the yielding stress of steel, and equal to 420 MPa for high tensile steel and 240 MPa for normal mild steel.

f_{cu} is the cubic compressive strength of concrete, and it is approximately equal to 73 MPa for high-strength concrete and to 26 MPa for normal-strength concrete.

The height-to-width ratio of the specimens is identified as (H_w/ L_w), where L_w is the width of the specimen, and it is constant for all specimens, and equal to 600 mm. H_w is the height of the specimens, and is equal to 1100 and 740 mm for the first and second group, respectively.

1-2 Geometry of specimens:-

The dimensions of the tested long and short wall models are shown in Fig. (1) and Fig. (2), respectively. They have I-section, consisted of two flanged walls and a web wall. The web wall was 60 mm in thickness and 400 mm width and its height was variable as mentioned before.

Top slab and footing are made rigid enough to prevent lateral distortion (lateral rotation about vertical axis) of the cross-section under lateral load effect and to represent a building floor. The top slab dimensions were 120 mm in thickness, 200 mm in width, and 600 mm long. The dimensions of footing were 500 mm width, 1600 mm long, and 350 mm thickness. Shape of specimens is recommended by previous studies (1, 2, 3, 4).

1-3 Mixing and materials:-

Two types of concrete were used in the present study. The mix proportions of the concrete used are given in Table (1). Mechanical-mixed concrete with replacement of 10 percent (by weight) of the cement by silica fume was used to produce high – strength concrete (5). To produce high-strength concrete with water-cement ratio of about 0.30, high range water reducing admixture was added. Super plasticizer was added as admixture to reduce water content and to improve the workability of high-strength concrete during casting.

The silica fume has a specific gravity of 1.34 and surface area equal to 200,000 cm^2/gm, which is about 50 times finer than most Portland cement. Normal Portland cement product was used for all specimens. The fineness of used cement was 8% which is less than 10% according to the limits of Egyptian specification. The initial setting time is 90 min. and final is 5 hours. The slump test was made according to ASTM C143 and ranged from 60-68 mm for normal strength concrete, no segregation was observed.

Coarse aggregate with maximum nominal size of 10 mm was used in order to ensure good compaction of concrete. It had dry density of about 1050 kg/m^3. For each test specimen, six 150 x 150 x 150 mm cubes were casted. The cubes were used to determine the compressive strength of concrete.

Table (2) presents the average values of experimental results of compressive cube strength of the used two types of concrete for each specimen.

2- TEST INVESTIGATIONS AND RESULTS

The test results included the determination of ultimate load, deflection, mode of failure, crack pattern, ductility, stiffness, and absorption of energy. The test results are based on that the out – of – plane displacement and the base rotation values recorded during testing were negligible. Also, the results are based on that the walls were essentially subjected to the intended boundary conditions, in plane action at the top and nearly fully restrained displacement at the bottom of the specimens.

Deflection control system was used (6) where the starting four cyclic loads were $\pm$ 0.6, $\pm$ 1.2, $\pm$ 2.4, and $\pm$ 4 mm, and the range of cycling was changed for the following three cycles as $\pm$ 6 mm, $\pm$ 9 mm, and $\pm$ 12 mm. For the specimens which not failed up to that step, the loading continued using wide range cyclic with stroke of $\pm$ 16 mm and $\pm$ 22 mm until failure occurred. The cyclic load history is shown in Fig. (3). The main observed and measured test results are presented in the following:-

2.1 Effect of variation of main flexure reinforcement:-

Figures (4) through (9) present the applying cyclic loads for all specimens. Table (3) represents the horizontal load carrying capacity, and the side deflection for each wall. Specimen walls HL–1, and HL–2 which had a different main flexure steel ratio in the flange, it was observed that the ultimate lateral load of wall HL – 2, increased by 5 % of the ultimate lateral load of wall HL – 1, but the top lateral deflection was less by 10 % than of the deflection of wall HL – 1.

For walls HS –1, HS – 2 it was observed that the increasing of main flexure reinforcement caused the increasing in ultimate lateral load for HS – 2 by 8 % than that of HS – 1. The results show that increase steel area has an effect on the walls strength and this effect is higher in case of short wall than that of long wall. The ultimate lateral top deflection decreased by 33 % than that of wall HS –1.

Ductility index µd as listed in Table (3) is defined according to the ACI Code (7) as the ratio of the ultimate deflection at the ultimate load to the yield deflection at the load of the first yield of steel bars. The loads of the first yield of specimens were obtained from the recorded values of the steel strain. Table (3) shows that for long wall specimens HL - 1 and HL - 2 increasing steel ratio by 43% reduced the ductility by 15% and for short wall specimens HS – 1 and HS – 2 increasing steel ratio by 43% reduced the ductility by 40%. Thus the ductility was decreased as the steel ratio increased, and the effect of the steel ratio was more obvious in case of short wall than that of long wall due to the increase of overall stiffness.

Table (3) shows also that the absorption of energy was increased by increasing the main flexure reinforcement of wall HL–2 than that of wall HL– 1.

2.2 Effects of height –to- width ratio:-

The ultimate carrying lateral load was increased for short wall HS-1 than that of HL-1 by 7% and by 11 % for wall HS – 2 than that of wall HL-2, where the lateral top deflection decreased in case of HS – 1 by 37% than that of lateral top deflection of HL–1, and it was decreased for wall HS–2 by 53 % than that of wall HL – 2. The height to width ratio affects on the ductility of walls. The ductility of long specimens HL – 1 and HL – 2 was increasing than that of short specimens HS – 1 and HS – 2 by 11% and 60%, respectively as shown from Table (3). This result shows that the short walls sustained more loads while exhibited less value of horizontal deflections and this is clear due to the higher stiffness of short walls than long walls.

2.3 Effect of concrete compressive strength:-

For wall NL and HL – 2 which had the same height – to –width ratio equal to 1.83, it was observed that the carrying lateral load for wall HL – 2, was greater by approximately 50 % than that of wall NL The lateral load capacity of wall HS – 2 was greater by about 29.0 % than the lateral load capacity of wall NS. These results indicate that walls made of HSC can carry much higher loads than those made of NSC, which shows that HSC has great effect on the strength of shear walls.

Comparing the ductility of wall HL – 2 and NL, it was found that the ductility of NL was less than that of HL – 2 by about 14%, while the ductility of short wall NS was grater by 30% than that of wall HS – 2. This may due to that HSC short wall is very stiff and can not exhibit higher value of lateral deformation.

Table (3) presents that the energy absorption for wall HL – 2 was greater by 38 % than the energy absorption of NL wall. The long wall of (HSC) type exhibited more deformation than that of long wall of (NSC) type which gives bigger area under load – deformation curve and consequently higher energy absorption.

3. CRACKING PROPAGATION

3.1 Cracking process of high - strength concrete shear walls:-

The crack pattern was approximately similar for long wall specimen HL–1 and HL – 2 and for short wall specimens HS – 1 and HS – 2. The first inclined crack observed at approximately at load 0.3 to 0.4 of ultimate failure load of the walls. Shear cracks initially appeared in the web at approximately half the height of specimens from the base. With increasing the load, these cracks grow and propagate towards the base of the wall.

After yielding, a flexural plastic hinge region formed at the lower portion of the flange of the wall, the height of the web plastic zone was approximately at the lower third portion of the wall height. This type of cracks near to the base of web and propagated diagonally indicated shear failure in the wall web with some flexural failure near to the flange. No crushing in the flange was obsereved. The concrete cover at the compression toe spelled off, while more number of diagonal cracks developed in-word and up-word in the plastic hinge zone. Figs. (10) and (11) shows the crack patterns after failure of specimens HL-1 and HL=2, respectively as a sample of wall cracks.

3.2 Cracking process of normal - strength concrete shear walls:-

For specimen NL a crack process overall similar as the long specimens of high - strength concrete shear walls HL – 1 and HL – 2. The first crack was appeared in the cyclic load (+2.4 mm) as the load was 50 KN (0.4 P_{ult}), After the yielding occurred, a vertical crack in flange was appeared and a flexural plastic hinge was formed at 0.40 of the height of the wall. At the beginning of cyclic load (+ 9 mm) the width of diagonal cracks reached up to 4.0 mm, then shear failure was occurred in the web at 0.5 the height of the wall. Specimen NS the first inclined crack appeared was fine when the load reached to 70 KN (0.5 P_{ult}) the first crack was appeared approximately at 0.5 of the height of the wall. Before yielding had occurred a few horizontal cracks were appeared in the flange at the 0.4 of the height of the wall from the base. The failure can be classified in case of normal strength concrete as ductile shear failure where shear failure was occurred in the web with some failure in the flange more than that in the case of high strength concrete walls.

Table (4) presents the stiffness at pre-cracked case, cracked case, and pre-ultimate case.

Degradation which defined as the ratio of the secant stiffness at the pre-ultimate case and the secant stiffness at the pre-crack case [K_{ult} / K_i], and the reduction factor of stiffness which defined as the percent between the secant stiffness at pre-crack case and the secant stiffness at initial case [K_i / K_e], were calculated and presented.

The stiffness was increased by increasing the main flexure reinforcement for the same height to width ratio and approximately the same cylinder compressive strength. For wall HL – 2, it was noted that its stiffness increased by increasing of the main steel ratio by about 30% more than that of wall HL – 1. For wall HS – 2 the stiffness was increased by 34 % than stiffness of HS–1 with increasing of main flexure reinforcement by 43 %. These results were common as the steel ratios effect on the inertia of structural elements.

It was noted that the reduction factor (K_i / K_e) of stiffness of wall HL-2 decreased by 20 % than that of wall HL – 1. It was also observed that for wall HS – 1, the secant stiffness was greater by 21 % than the secant stiffness of wall HL – 1, and the secant stiffness of wall HS – 2 was greater by 25 % than the secant stiffness of wall HL – 2. These results indicated the clear effect of the aspect ratio where the short walls HS (with low height to width ratio) had an initial stiffness higher than that of long walls HL (with big height to width ratio).

Table (1) Summaries mix proportions of concrete

Material	Unites	Concrete type	
		High Strength Concrete (m^3)	Normal Strength Concrete (m^3)
Cement	kg	550	350
Silica fume	kg	55	_
Water-Cement ratio	**%**	0.903	0.22
Water	Liter	138.7	156.7
Superplasticizer	Liter	11	_
Sand	kg	676	720
Coarse aggregate	kg	1050	1200
Coarse aggregate Size	mm	5 to 19	5 to 20

Table (2) Compressive strength of concrete specimens

Specimen No.	f_{cu} MPa
HL-1	76
HS-1	70.4
HL-2	76
HS-2	70.4
NL	26.3
NS	26.3

Table (3) Summaries the experimental results

Specimen No.	1st Crack load (kN) P_o	Def. at 1st Crack load (mm) Δ_0	Yield load (kN) P_y	Def. at Yield load (mm) Δ_y	Ultimate load (kN) P_u	Def. at ultimate load (mm) Δ_u	Ductility index Δ_u / Δ_y μd	Absorption of Energy (kN.mm)
HL-1	53.93	1.316	80	3	168.49	10.238	3.41	1019.29
HS-1	65	1.249	90.65	2.1	180.56	6.42	3.05	681.00
HL-2	70.03	1.227	100.16	3.1	176.65	9.199	3.00	999.38
HS-2	75	0.819	110	2.4	195.26	4.29	1.78	420.47
NL	49.96	1.98	94.84	3.5	118.23	9.07	2.57	759.37
NS	69.7	2.27	98	3.5	139.7	8.23	2.35	733.66

Table (4) Determination of stiffness

Specimen No.	Pre-cracked Stiffness K_e	Cracked Stiffness K_i	Stiffness at Pre-ultimate Load K_{ult}	Degradation K_{ult} / K_i	Reduction Factor of Stiffness at Pre-crack Case K_i / K_e
HL-1	31.438	26.340	6.860	0.260	0.838
HS-1	107.830	56.190	8.880	0.158	0.521
HL-2	47.990	32.540	10.960	0.337	0.678
HS-2	139.970	69.150	29.260	0.423	0.494
NL	25.613	21.470	7.330	0.341	0.838
NS	30.220	26.100	11.450	0.439	0.864

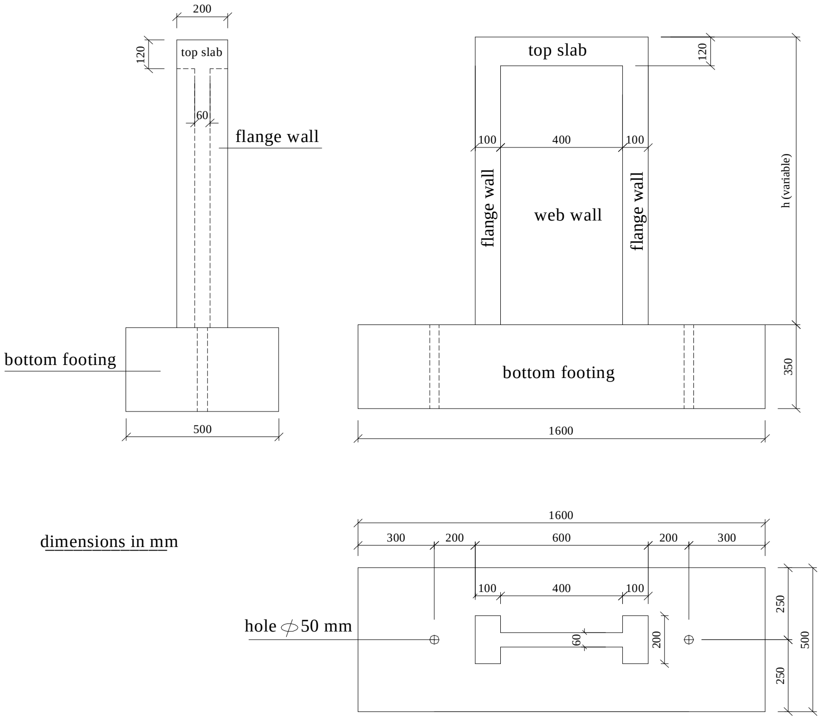

Fig. (1) Dimensions of the tested long walls

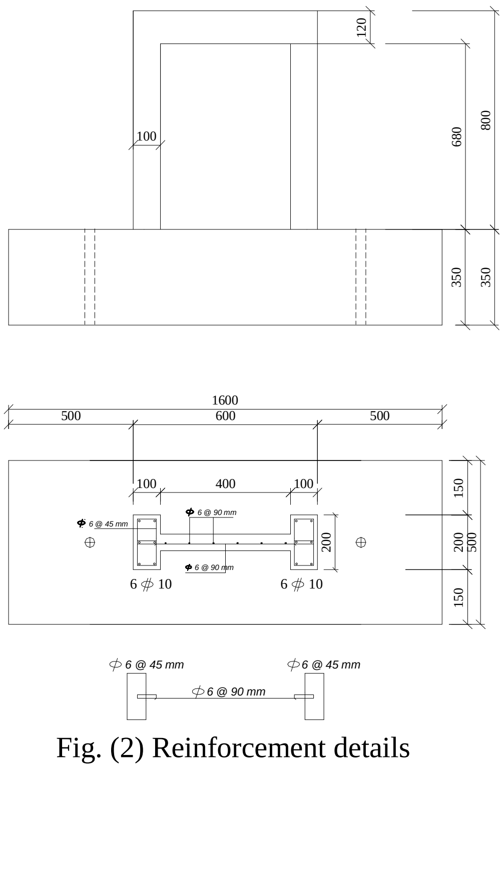

Fig. (2) Reinforcement details

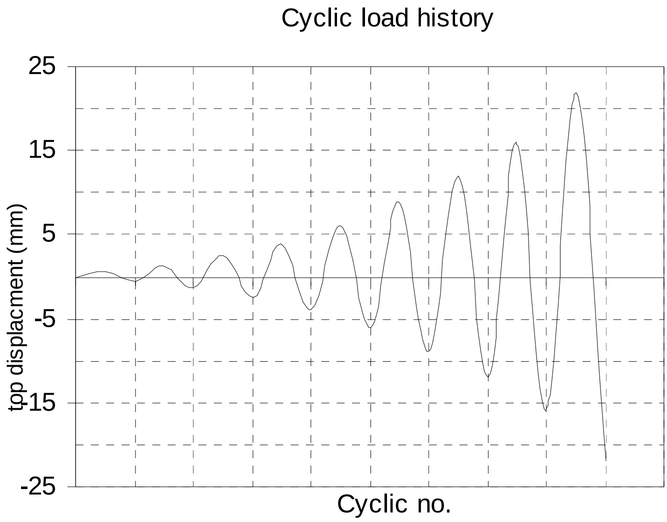

Fig. (3) Cyclic load history used for tests

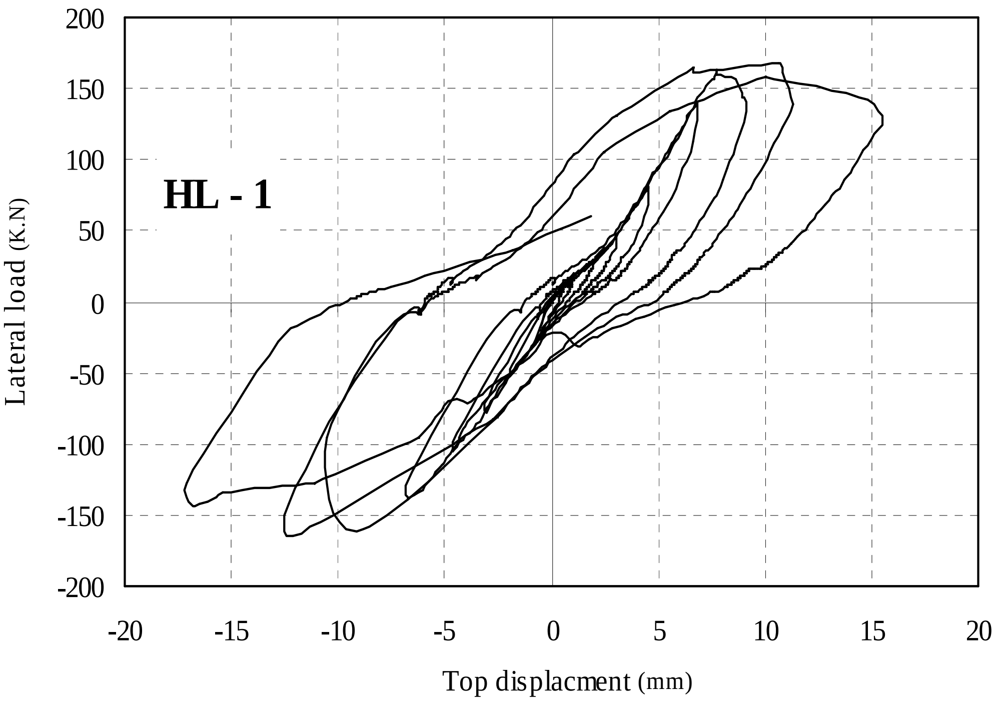

Fig. (4) Cyclic load and displacement of specimen HL – 1

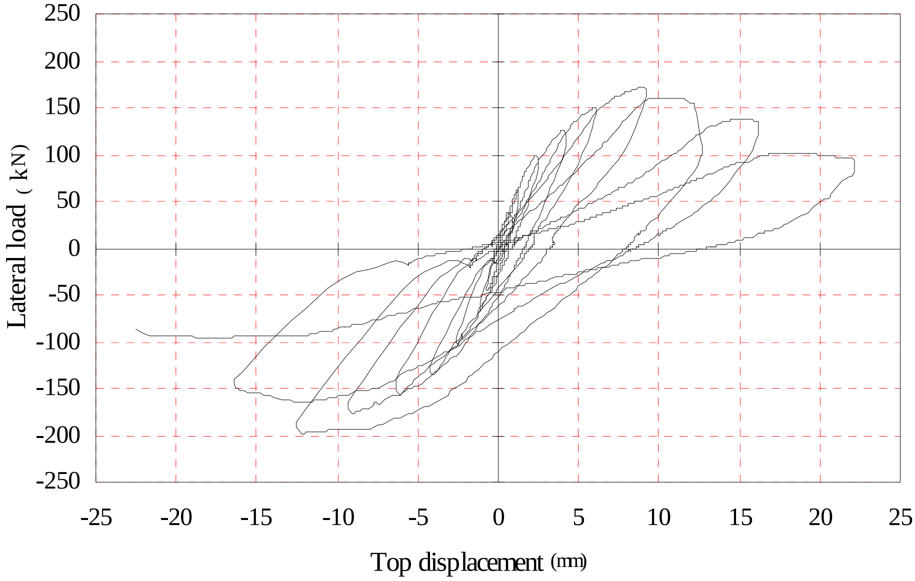

Fig. (5) Cyclic load and displacement of specimen HL – 2

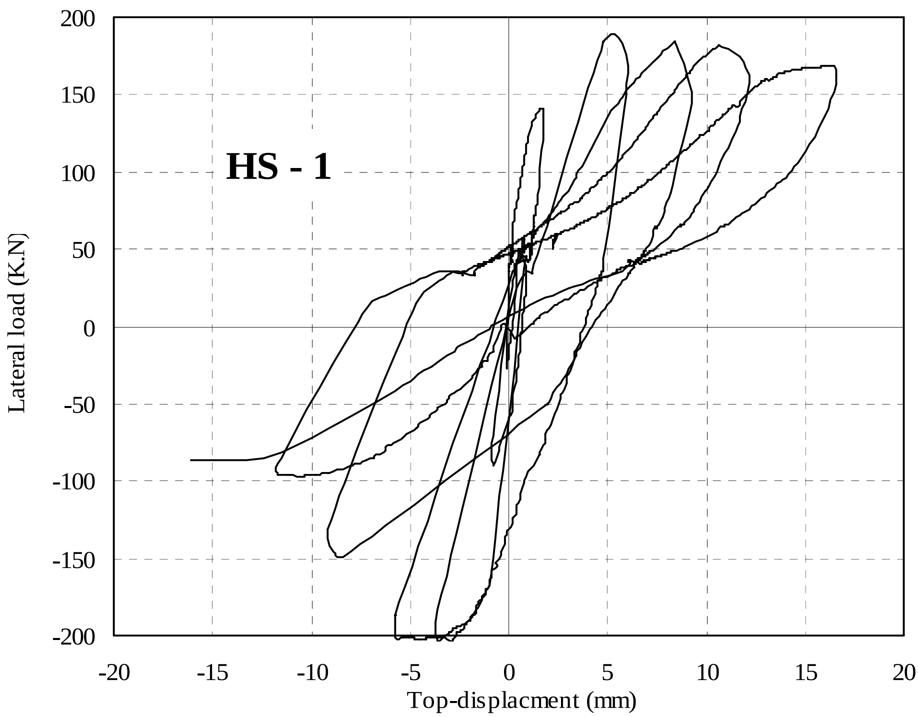

Fig. (6) Cyclic load and displacement of specimen HS – 1

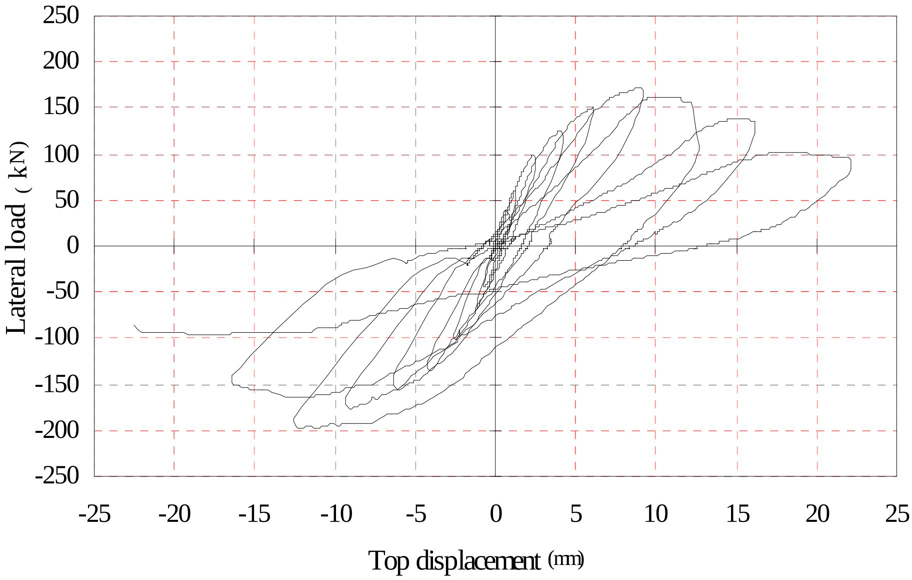

Fig. (7) Cyclic load and displacement of specimen HS – 2

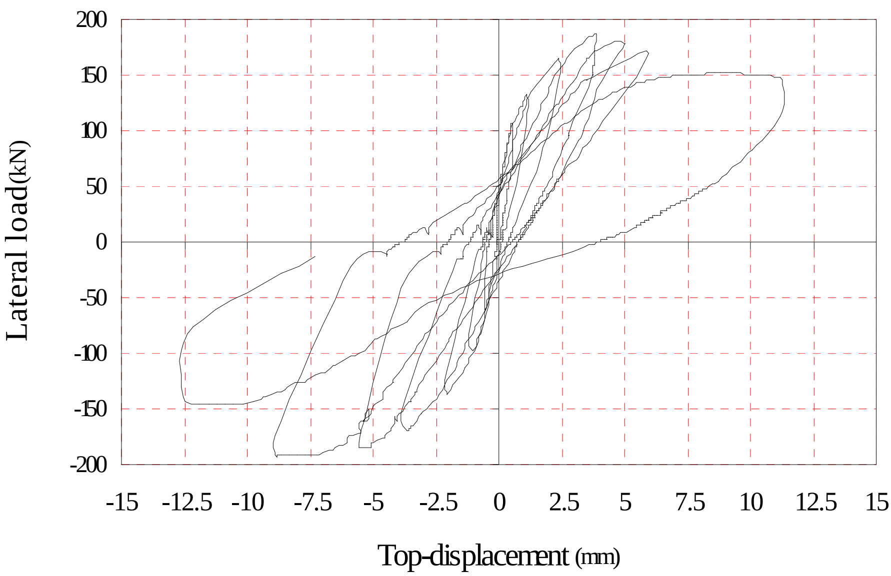

Fig. (8) Cyclic load and displacement of specimen NL

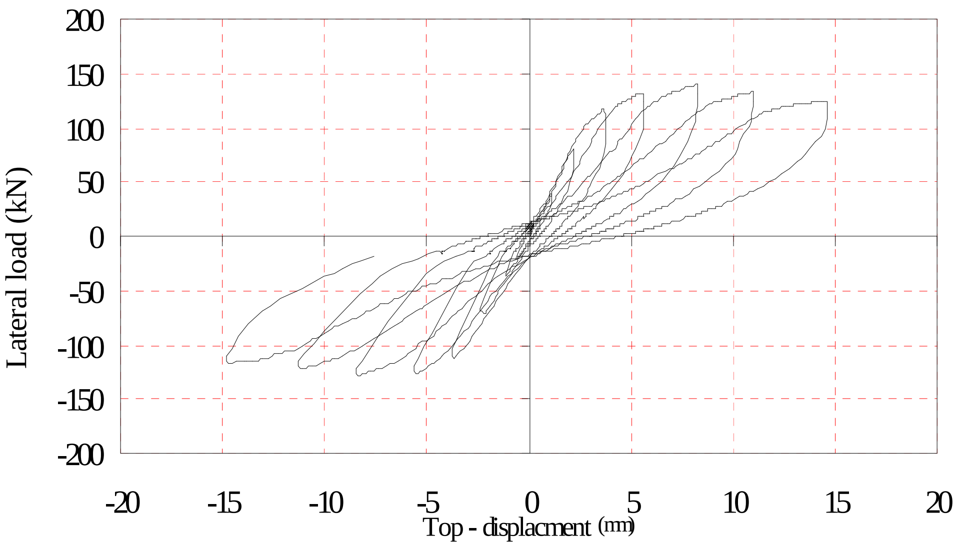

Fig. (9) Cyclic load and displacement of NS

Fig. (10) Crack pattern of specimen HL-1

Fig. (11) Crack pattern of specimen HL-2

CONCLUSION

Within the range of variables and obtained measurements, the following conclusions are obtained:

1. Increasing of compressive concrete strength (HSC) in long walls and in short walls exhibited higher values of the lateral load capacity for the section by 49 % and 40 %, respectively than that of normal strength concrete (NSC).
2. Increasing of compressive concrete strength (HSC) in long walls and in short walls exhibited higher values of the stiffness by 33 % and 60 %, respectively than that of normal strength concrete (NSC).
3. A significant increasing of the stiffness was observed for short (HSC) wall, which have height-to-width ratio of 1.23 by approximately 20 % more than that of the stiffness of long (HSC) wall, which have height–to–width ratio of 1.83.
4. Generally, increasing the steel ratio or reducing the height – to – width ratio of high – strength shear walls reducing the ductility index μ_d. The ductility of (NSC) short wall was greater than that of (HSC) short wall, while it was less in the case of (NSC) long wall that of (HSC) long wall.
5. For (HSC) long walls HL-1 and HL-2, the first crack occurs at 0.3 and 0.4 P_{ult}, respectively, and the yielding load was at 0.5 and 0.6 P_{ult}, respectively, and the failure was flexural- shear failure. But for (HSC) short walls HS-1 and HS-2 the first cracking load was at 0.4 P_{ult}, approximately, for the two walls and yielding loads were at 0.7and 0.8 P_{ult}, respectively, and the failure was shear failure.

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سلوك الحوائط الخرسانية في القص تحت تأثير الاحمال الرأسية والارتدادية

يحتوى هذا البحث على دراسة عملية ونظريه لسلوك الحوائط الخرسانية المقاومة للقص والمصنوعة من الخرسانة عالية المقاومة إثر تغير نسبة الحديد وكذلك نسبة الارتفاع إلى الطول ومقارنتها بسلوك الحوائط الخرسانية المقاومة للقص والمصنوعة من الخرسانة عادية المقاومة وعلى ذلك فقد تم تصنيع و صب عدد ستة حوائط خرسانية أربعة منها عالية المقاومة واثنين منها عادية المقاومة و لها قطاع خرساني ثابت على شكل حرف I بعرض ثابت 60سم وتم تقسيم العينات إلى مجموعتين من حيث الارتفاع:

المجموعة الأولي: العينات الطويلة HL- 1 , HL-2 من الخرسانة عالية المقاومة بنسبة ارتفاع إلى العرض 1.83للعينتين ونسبة حديد تسليح 2.36% للعينة HL- 1 ونسبة حديد 3.39% للعينة HL- 2 وكانت العينة المقارنة المصنوعة من الخرسانة عادية المقاومة هي NL بنفس نسبة الارتفاع إلى العرض ونسبة حديد 3.39%.

المجموعة الثانية: فهي العينات القصيرة وهى HS- 1, HS- 2 من الخرسانة عالية المقاومة بنسبة ارتفاع إلى العرض1.23 للعينتين ونسبة حديد تسليح 2.36% للعينة HS- 1 ، 3.39% للعينة HS- 2 وكانت العينة المقارنة المصنوعة من الخرسانة العادية المقاومة هي NS بنفس نسبة الارتفاع إلى العرض ونسبة حديد تسليح 3.39%.

وتم عمل تحليل للنتائج التي تم الحصول عليها معمليا وتحليل تأثير كل متغير على سلوك الحوائط الستة المختبرة.