Structural Behavior of Normal and High Strength Concrete Wall Panels Subjected to Axial Eccentric Uniformly Distributed Loading

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ABSTRACT.
In most cases, the concrete wall panels are subjected to axial eccentric distributed loading; due to this type of loading, concrete wall panels behave and fail somehow. There are many parameters that affect the structural behavior of the concrete wall panels.

This study presents experimental investigation the structural behavior of concrete wall panels subjected to axial eccentric distributed loading; also evaluates the effect of the parameters, slenderness ratio (H/t), aspect ratio (H/L) and concrete strength on the behavior of concrete wall panels.

The experimental program includes testing fifteen concrete wall panels hinged at top and bottom with free sides, by applying the load axially with eccentricity equal to (t/6); these panels are divided into five groups, each group consists of three panels with slenderness ratio (H/t) equals to (20 , 25 , 30) for each panel, three groups of normal concrete strength with aspect ratio (H/L) equal to (1.0 , 1.5 . 2.0) for each group and the other two groups are of high strength concrete with aspect ratio (H/L) equal to 2.0 for both two groups.

The deflections of concrete wall panels depend on the slenderness ratio (H/t), aspect ratio (H/L) and concrete strength.

The failure mode of the concrete wall panels is greatly affected by the aspect ratio (H/L); the panels with low aspect ratio tend to fail by crushing, while panels with high aspect ratio tends to fail by buckling.

Keywords:Axial Load , concrete wall panels, slenderness ratio , concrete strength , aspect ratio.

1. INTRODUCTION.

Reinforced concrete walls are widely used as structural elements in locations where they are subjected to axial loads and end moments., and appear as integral components in box frames, folded plates, box girders, box culverts, tee beams, etc.[1]

In the past, concrete walls were designed in most structures for protection against the external environmental conditions with little consideration for the capability of the wall as a structural member. This approach was mainly due to the very low allowable design stresses for walls specified in early versions of published concrete codes.

Over the years, reinforced concrete walls have gained greater acceptance, by practicing engineers, as load-carrying structural members. This acceptance is due to the increased research undertaken on concrete walls and the subsequent increase in allowable design stresses incorporated in various current concrete codes.[2]

The study and development of new technologies in concrete and steel structures for construction have been increased in recent decades. With the advent of high-strength concrete and through the use of prefabrication it becomes possible to produce thin concrete elements, which has enabled significant cost reductions through the use of the most resistant and thinner

walls.[3][4]
This experimental study was made on 15 concrete wall panels with various aspect and slenderness ratios and various concrete strength; all these panels were hinged at top and bottom with free vertical edges.

2. PANEL DESIGNATIONS AND DIMENSIONS.

The plan of the experimental work consists of casting and testing 15 wall panels, divided into five groups, three groups of normal concrete strength and the other two groups of high strength concrete. The nominal slenderness ratio varied from 20 to 30, aspect ratio for normal concrete strength varied from 1 to 2.0, aspect ratio for high strength concrete was fixed at 2.0. All panel thickness was 30 mm.

Panels are designated as (W x_1 x_2), the number x_1 refers to the number of the group. Groups 1, 2 and 3 were of normal concrete strength, while groups 4 and 5 were of high strength concrete, the number x_2 refers to the number of the panel within the group. The dimensions and designations of the wall panels are summarized in **Table (1).**

3. STEEL REINFORCEMENT.

All the concrete wall panels were reinforced with one layer of a plain steel welded mesh, consisting of 4 mm diameter bars with spacing of 90 mm c/c, placed centrally through the panel thickness. The vertical and horizontal reinforcement ratios, ρ_v and ρ_h, were both 0.0032 for all panels, satisfying the minimum requirements of the American Concrete Institute Code (ACI 318-08)[5]. The yield strength was determined from tensile test at the Structural Lab. of the College of Engineering of AL-Mustansiryia University. The average yield stress was 390 MPa.

4. MIX PROPORTIONS.

All mix proportions were conducted by trail mixes according to the previous research, more than five mixes were made to achieve concrete strength targets, for normal strength (20 -25 MPa), for high strength two groups, (40 -50 MPa) and (60-70 MPa). The proportions that conducted were as below:

4.1. Normal Strength Concrete.

Groups 1, 2, 3 consist of normal strength concrete and material proportions were 1:2:3 w/c=0.5 by weight as shown in **table (2)**.

4.2. High Strength Concrete.

Groups 4, 5 consist of high strength concrete and material proportions were 1:1.5:2.5, w/c = 0.4 and 1:1.2:2, w/c = 0.3 respectively with the addition of super plasticizer, all proportions were by weight as shown in **Table (3)**.

5. FORMWORK FOR TEST PANEL.

The formwork for casting the concrete test panels was fabricated from rectangular timber planks with 20 mm thickness, made of form ply-wood. Three formwork with clear dimension of (900 ×900 mm),(750×750 mm) and (600×600 mm) ,these formwork were made of wooden bed (ply-wood) and four movable sides ,the sides were steel angles 30×30×3 mm and fixed to the bed by screws. To achieve other dimensions of panels, one of the sides is moved and fixed in another position to the accurate dimension. **Fig. (1)** shows this formwork.

6. TESTING.

6.1 Testing Machine.

The main testing machine is a universal testing machine (8551 M. F. L. system) available in the Structural Lab. in Civil Eng. Dept. College of Eng. of AL-Mustansiryia University as shown in **Fig.(2)**. The panels are tested by this machine after making some arrangement to simulate the support condition for the panels. Cubes and cylinders are also tested by this machine.

6.2 Test Rig Set-Up.

The test rig in the case of axially loaded walls (hinged at top and bottom) must satisfy two main conditions. Firstly, the supports of the wall panel to be tested must be allowed to rotate freely, while at the same time they should not move or deflect laterally. Secondly, the axial load must be uniformly distributed across the length of the test panel at a certain eccentricity [(6)]. Based on the previous researches used test rigs ,and in order to make a simple , economical and functional test rig (support simulation) ,it has been seen that the best one for our study was the test rig used by **Swartz et al (1974)[7]**.

With some amendments to the test rig used by **Swartz, et al (1974)[7],** each top and bottom hinged support conditions is simulated by attaching a 32 mm diameter high strength steel rod on a channel of size (C50 mm×3 kg/m) and welded very well for a length of rod and channel 1.0 m to ensure that the panels will be within the length of the channel. Two high strength steel rods of 12 mm then attached and welded very well to either flange of I-steel section to make a suitable guide for the steel rod of 32 mm that attached to the channel.

In order to satisfy the eccentricity when the loading is applied, the concrete panels restrained with a series of screws fixed on one side at the top and bottom channel. These screws could be adjusted for various eccentricities. Details of the simply supported top hinged edge are shown in **Figs.(3 and4)**.

The two I-sections fixed to the test machine by many clamps tightly, top and bottom taking care with the straightening of the two I-sections. After the test rig has been fixed, the panel fixed to the top and bottom hinge supports, leveling the panel to ensure the perpendicularity of the panel and then tightening the screws to satisfy eccentricity and also fixing the panel, and applying the load to the failure of the panel. **Fig. (5)** show these arrangements

7. DEFLECTION CHARACTERISTICS.

The load versus lateral deflection (out of plane) profiles for both the normal and high strength concrete walls are shown in this paragraph. During the test, the applied load and the corresponding deflections, at mid center of the panels were recorded using dial gauges (reading to 0.01mm) located on the compression side of the wall to prevent possible damage to the gauge. All the tests are carried out under the condition of load control of 5kN increments.

Fig.s (6 to 16) show the structural behavior for all panels.

It is important to know that most of the tested panels failed in a brittle mode and the sudden failure of these panels made it difficult to record deflection at failure. Thus, in these Figures, the absolute maximum failure loads and the corresponding maximum deflections are not shown, these failure loads are shown in the **Table (4)**.

In order to compare the structural behavior of the panels accurately, each Figure will represent three panels with two constant parameters and third parameter is varied. First five chart represents the panels of groups (1,2,3,4,5) which have constant concrete strength and constant aspect ratio, other three charts represent panels with constant slenderness ratio and normal concrete strength and varying aspect ratio and the last three charts represent panels with constant slenderness ratio , aspect ratio and with varying concrete strength.

The following observations are made from the test results plotted in the Figure 6 to Figure 16:

1) from **Fig.(6)** W12, W13 panels show linear curves up to failure load, while W11 shows linear curve for the initial loading and then followed by non-linear trend with lateral deflections increasing rapidly as failure was approached.

   For comparison, there is no obvious difference between W12 (H/t=25) and W13 (H/t=20), and the three panels have the same lateral deflection before failure.

2) The **Fig.(7)** show that the walls exhibited more ductile failure behavior. This was reflected in the continually increasing values of the deflections as the test loads approached failure.

   All panels show linear curves up to failure load. The lateral deflection before failure for panels W22 and W23 was more than lateral deflection before failure for panel W21 by about 34%, and this was due to crushing failure mode for panel W21.

   For comparison, with the behavior of group (1), it is obvious that the slope of curves for group (1) is more than the slope of curves for the group (2), and this can be explained due to the difference in aspect ratio, and it can be concluded that higher aspect ratio will give more ductile behavior.

3) In **Fig.(8)** the panels W32 and W33 showed approximately linear curves up to failure load, while panel W31 showed nonlinear curve, this can be explained by high slenderness ratio (30) and high aspect ratio for the panel W31.

   The panel W33 exhibited the lowest lateral deflection in this group due to buckling failure mode that occurs at the bottom of the panel, while panel W32 exhibited the higher value of lateral deflection in this group.

   For comparison, with the behavior of group (1) and (2), it is obvious that the slope of curves for group (1) and (2) are more than the slope of curves for the group (3), and this can be explained due to the difference in aspect ratio, and it can be concluded that higher aspect ratio will give more ductile behavior.

4) From **Fig.(9)**, it can be noticed that the behavior of group (4) is similar to that of group (1), although they differ in aspect ratio and concrete strength. A reason for this may be the effect of aspect ratio (which equal to 1.0 for group (1) and equal to (2.0) for group (4)) is similar to the effect of concrete strength for specific ratios.

5) The **Fig.(10)** show that all panels show linear curves up to failure load. The final lateral deflections were different for panels in group (5). The lowest value of lateral deflection was for the panel with highest slenderness ratio and the highest value was for the panel with the lowest slenderness ratio. A reason for this may be the highest failure load for panel with lowest slenderness ratio which results in continuous loading with continuous lateral deflection.

   In general, for all the above curves of all groups, the lateral deflection for panels with slenderness ratio (H/t= 30) was more than the lateral deflection for panels with slenderness ratio (H/t= 25 and 20) for the same axial load. This led to conclude that the panels with high slenderness ratio exhibited more lateral deflection.

6) Also from **Fig.(11)** it can be noticed that panels show nonlinear curves except panel W21, and it's lateral deflection was the smallest value among this group and the higher value of lateral deflection was for panel with the higher aspect ratio in the group (H/L=2.0 for panel W31). This led to conclude that higher aspect ratio will result in high lateral deflection.

7) In **Fig.(12)** all panels show linear curves. It is obvious to notice that the difference in lateral

deflection between the panels was large especially after the stage of (1 mm) deflection.

It is also obvious that the lateral deflection at any load stage was the higher value for the panel with high aspect ratio (H/L=2.0), and the smallest value correspond to the panel with the smallest aspect ratio (H/L=1.0).

8) All panels in the **Fig.(13)** are approximately show linear curves. It is obvious to notice that the difference in lateral deflection between the panels was small.

From **Figs. (11),(12) and (13)**, it can be said:

- The increase in aspect ratio will result in increase in lateral deflection.
- For panels with different aspect ratios, the decrease in slenderness ratio will decrease the difference in lateral deflection between the panels.

9) In order to compare the lateral deflection for panels with different concrete strength, the other parameter must be kept constant. It is obvious from **Fig.(14)** the large difference between the slopes of the curves, and that higher slope of the curves indicates the panel with higher strength concrete, and it is clearly noticed that the difference in lateral deflections for the same load was large. This means that the panels of high strength concrete will behave in brittle type of failure.

10) It is obvious from **Fig.(15)** the large difference between the slopes of the curves, and that higher slope of the curves indicates the panel with higher strength concrete, and it is clearly noticed that the difference in lateral deflections for the same load was large. . This means that the panels of high strength concrete will behave in brittle type of failure.

Further review of the deflection curves in **Figs (14) and (15)** shows that the difference in lateral deflection between normal and high concrete panels was larger for panels with slenderness ratio H/t=30 especially between panels (W31 and W41),(W32 and W42).

11) Also it was obvious from **Fig.(16)**, that the curves for panels W43 and W53 became close to each other, this means that the panels with high strength concrete exhibited small difference in lateral deflection when the slenderness ratio is decreased.

From **Figs (14), (15) and (16)** it can be said:

- Panels with high strength concrete behave in brittle mode failure.
- When the slenderness ratio is decreased, the difference in lateral deflection is increased between normal and high strength concrete, and it was decreased for panels with high strength concrete.

8. CRACKING CHARACTERISTICS.

8.1 Cracking Loads.

Cracking load is that load at which the first visible surface crack is seen by the naked eye on the surface of the wall. Although great care was taken in marking the first visible crack, the values of the cracking loads are still approximated and do not necessarily represent the load at which the actual cracking of concrete had started. This is because the crack at the beginning is tiny and cannot be seen until it grows up.

The cracking loads corresponding to the appearance of first crack and failure loads recorded are given in **Table (4)**.

From the **Table (4)**, the cracking loads are about (52-75) % of the ultimate loads for panels with normal concrete strength and about (79-85) % for panels with high strength concrete.

For panels with normal concrete strength, the cracking loads are about 63-72 % of the ultimate loads for panels with H/t=30 and about 55-71 % for panels with H/t=25 and about 52-75 % for panels with H/t=20.

For panels with normal concrete strength, the cracking loads are about 52-63 % of the ultimate loads for panels with H/L=1.0 and about 65-68 % for panels with H/l=1.5 and about 71-75 % for panels with H/L=2.0.

From all above, it can be concluded that:

- Panels with high strength concrete failed after a short time from the appearance of the first crack, and this may be the reason for the brittle failure that occurs for high strength concrete panels.
- The variation of slenderness ratio has no effect on the cracking loads.
- The aspect ratio has an obvious effect on the cracking loads. It is obvious that the increase in aspect ratio results in increase in the ratio of the cracking load. This means that the panels with high aspect ratio will fail after a short time from the appearance of the first crack.

8.2 Crack Patterns.

The crack patterns observed on the tension face of the wall panels are shown in **Figs (17-21)**.

All these photographs are taken for the panels after failure of these panels and marking the visible crack with black lines as can as possible.

From **Fig.(17)** which shows the panels of group (1), it is obvious that all panels of this group exhibited cracks near top or bottom edges of the panel. These cracks are horizontal and perpendicular to the loading direction with some small diagonal cracks occurred near the corners of the panels.

From **Fig.(18)** which shows the panels of group (2), it can be noticed that the panel W21 exhibited horizontal cracks near the top edge of the panel, while panels W22 and W23 exhibited horizontal cracks near the centre of the panels with some small diagonal cracks occurred near the corners of the panels.

From **Fig.(19)** which shows the panels of group (3), it can be noticed that the panel W33 exhibited horizontal cracks near the bottom edge of the panel, while panels W31 and W32 exhibited horizontal cracks near the centre of the panels, also one small diagonal crack occur at the corner of the panel W32.

From **Fig.(20)** which shows the panels of group (4), it can be noticed that the panels W41 and W42 exhibited horizontal cracks near the top and bottom edge of the panel, while panel W43 exhibited horizontal crack near the centre of the panel, and one small diagonal crack occur at the corner of the panel.

From **Fig.(21)** which shows the panels of group (5), it is obvious that all panels of this group exhibited horizontal cracks near the center of the panels with small tiny diagonal cracks occurred at the corner of the panels and one diagonal crack occurred at the center of the panel W52.

It can be said as a reason for the occurrence of the diagonal cracks that the load tries to pass through the shorter path or the weakest area.

9. FAILURE CHARACTERISTICS.

9.1 Failure Modes.

The modes of failure of concrete wall panels tested under axial eccentric distributed loading could be divided, in general, into two types:

9.1.1Crushing at Support Failure.

This type of failure occurred at the top and bottom of the panels, near the supports, at which the concrete was crushed before buckling, has occurred. This type of failure occurred for panels (W11, W12, W13, W21, W41, and W42). It is obvious that all panels with aspect ratio H/L=1.0 failed by crushing of concrete, which can be concluded that the panels with low aspect ratio tend to fail in crushing mode.

9.1.2 Bending or Buckling Failure.

In this type of failure, the panel deflected in a single curvature in the vertical direction and continue to deflect until the failure occurred by flexural mechanism. This type of failure occurred for panels (W22, W23, W31, W32, W33, W43, W51, W52, and W53). It is obvious that the line of failure lies near the center of the panels except for panel W33 which lies at the bottom part of the panel. It was noticed also that all panels with aspect ratio H/L=1.5 and 2.0 except panels (W21, W41, and W42) failed by buckling, which can be concluded that the panels with high aspect ratio tend to fail by buckling mode.

Table (5) shows the failure mode for each panel:

In order to study the results closely, a statistical **table (6)** is made for the panels and their failure mode as shown below:

From **Table (6)**, some observations are made:

- From the effect of slenderness ratio, it seems that panels with low slenderness ratio tend to fail by buckling, while panels with high slenderness ratio tend to fail by crushing, noticing that this conclusion was based on results of only 5 panels for each slenderness ratio, therefore more test results are required to give final or exact conclusion about the effect of slenderness ratio on the mode of failure.
- From the effect of aspect ratio, it is obvious that the panels with low aspect ratio tend to fail by crushing, while panels with high aspect ratio tend to fail by buckling. A reason for this may be that the increase in length of the panel (L) with constant height (H) will decrease the aspect ratio and the panel will be wider and it is very difficult to buckle and tends to crush.
- From the effect of concrete strength on the mode of failure, it seems that the concrete strength has no effect on the mode failure of the panel if it fails by buckling or by crushing. The effect of concrete strength was on the failure, whereas it was brittle or ductile failure as mentioned before in the section of deflection characteristic.

9.2 Observations of Failure.

Some of the observations, noticed during the test of wall panels and their failure, must be mentioned and gave some conclusions and remarks on these observations as shown below:

- At the beginning of applying the load on the top of the panel, all panels deflected in a single curvature in the vertical direction even those panels failed by crushing and this was due to nature of supporting (free vertical side of the panel and hinged at top and bottom) and the eccentric loading that was applied.
- The sign of warning before the occurrence of failure was the rapid movement of the dial gauge with no increase in the applied load.
- The failure of panels with high strength concrete was explosive failure and there was a yield in steel reinforcement for panels (W52 and W53), this indicates that the high strength concrete panels possessed a more brittle failure mode, with some yielding of reinforcement taking place before concrete failure. This suggests that the use of slender

and high strength concrete wall panels may be dangerous in practice, when only minimum reinforcement is provided, as abrupt failure may occur.

10. CONCLUSIONS.

Depending on the test results of the experimental program, the following conclusions are obtained:

1) The structural behavior and final lateral deflections of concrete wall panels depends on the slenderness ratio (H/t), aspect ratio (H/L) and concrete strength as follows:

   a) The increase in slenderness ratio (H/t) of the wall panels causes to increase the lateral deflection of the wall panels.
   b) The increase in aspect ratio (H/L) of the panels causes to increase the lateral deflection of the concrete wall panels.
   c) As concrete strength of the wall panel increases, the structural behavior of the wall panels tends towards the brittle failure. This suggests that the use of slender and high strength concrete wall panels may be dangerous in practice, when only minimum reinforcement is provided, as abrupt failure may occur.

2) The failure mode of the concrete wall panels is greatly affected by the aspect ratio (H/L), the panels with low aspect ratio tend to fail by crushing, while panels with high aspect ratio tends to fail by buckling.

The concrete strength has no effect on the failure mode (buckling or crushing) of the concrete wall panels.

More tests will be needed before a final conclusion can be made for the effect of the slenderness ratio on the failure mode.

11. REFERENCES.

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Table (1): Panels Designations and Dimensions.

Group No.	Wall panels	Dimensions (mm)			Aspect ratio H/L	Slenderness ratio H/t
		H	L	t		
Group 1	W11	900	900	30	1.0	30
	W12	750	750	30	1.0	25
	W13	600	600	30	1.0	20
Group 2	W21	900	600	30	1.5	30
	W22	750	500	30	1.5	25
	W23	600	400	30	1.5	20
Group 3	W31	900	450	30	2.0	30
	W32	750	375	30	2.0	25
	W33	600	300	30	2.0	20
Group 4	W41	900	450	30	2.0	30
	W42	750	375	30	2.0	25
	W43	600	300	30	2.0	20
Group 5	W51	900	450	30	2.0	30
	W52	750	375	30	2.0	25
	W53	600	300	30	2.0	20

Table (2): Mix proportions for Normal Concrete.

Groups	Cement Kg/m^3	Sand Kg/m^3	Gravel Kg/m^3	Water Liter/m^3	Super plasticizer Litter /100 kg cement
1, 2, 3	400	800	1200	200	-

Table (3): Mix proportions for High Strength Concrete.

Groups	Cement Kg/m^3	Sand Kg/m^3	Gravel Kg/m^3	Water Liter/m^3	Super plasticizer Litter /100 kg cement
4	480	720	1200	192	0.8
5	570	684	1140	171	1.0

Table (4): Cracking Loads.

Panel	W11	W12	W13	W21	W22	W23	W31	W32	W33	W41	W42	W43	W51	W52	W53
Cracking load(kN)	75	80	80	65	80	85	65	75	75	95	110	120	140	170	190
Failure load(kN)	120	145	155	95	120	130	90	105	100	120	140	150	175	200	235
$P_{cr} / P_u \times 100$ %	63	55	52	68	67	65	72	71	75	79	79	80	80	85	81

Table (5): Failure Modes for all Panels.

Group	1			2			3			4			5		
Panel	W 11	W 12	W 13	W 21	W 22	W 23	W 31	W 32	W 33	W 41	W 42	W 43	W 51	W 52	W 53
Failure Mode	Cr	Cr	Cr	Cr	Bu	Bu	Bu	Bu	Bu	Cr	Cr	Bu	Bu	Bu	Bu

Cr = crushing at support, Bu = buckling

Table (6): No. of Panels correspond to Failure Mode.

Failure Mode	**No. of Panels with**							
	Slenderness Ratio(H/t)			**Aspect Ratio (H/L)**			**Concrete Strength**	
	20	25	30	1.0	1.5	2.0	normal	high
Crushing	1	2	3	3	1	2	4	2
Buckling	4	3	2	0	2	7	5	4
Total	5	5	5	3	3	9	9	6

Figure (1): Formwork used in this study.

Figure (2): Universal testing machine (8551 M. F. L. system).

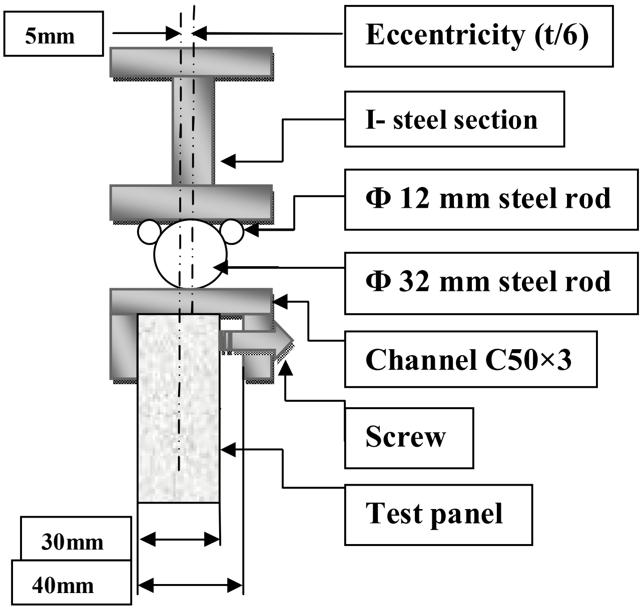

Figure (3): Detail of Supports used in this work.

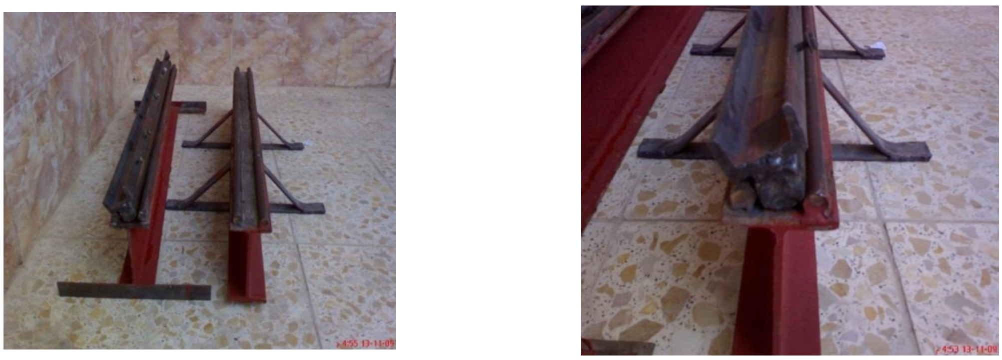

Figure (4): Photograph of Supporting Elements.

Figure (5): Arrangement before testing panels.

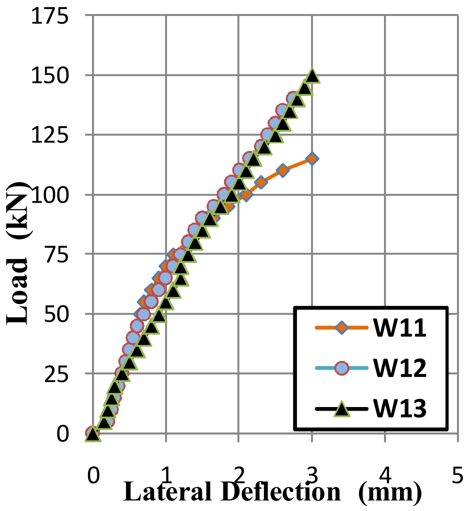

Figure (6): Load - Lateral deflection curves for Group (1) (H/L=1.0 , f_c' = 20.4 MPa).

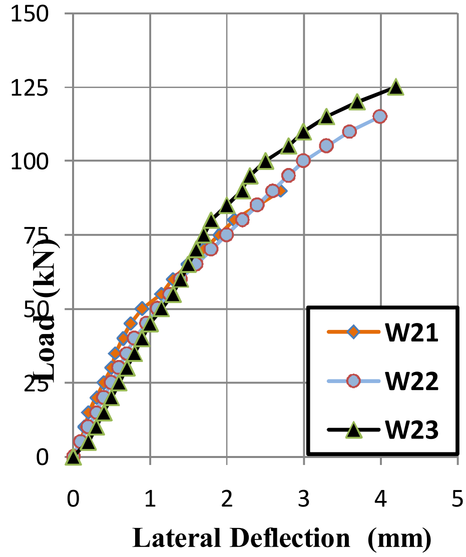

Figure (7): Load - Lateral deflection curves for Group (2) (H/L=1.5 , f_c' = 20.9 MPa).

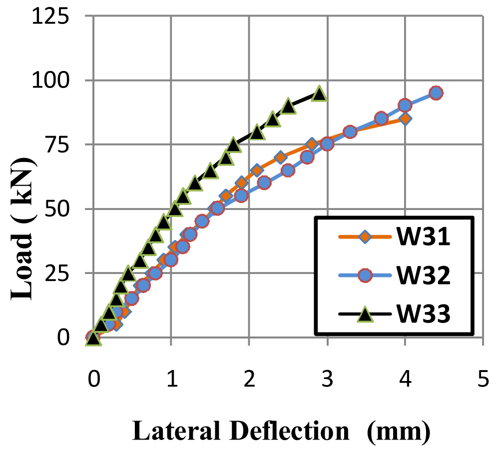

Figure (8): Load - Lateral deflection curves for Group (3) (H/L=2.0 , f_c' =20.3 MPa).

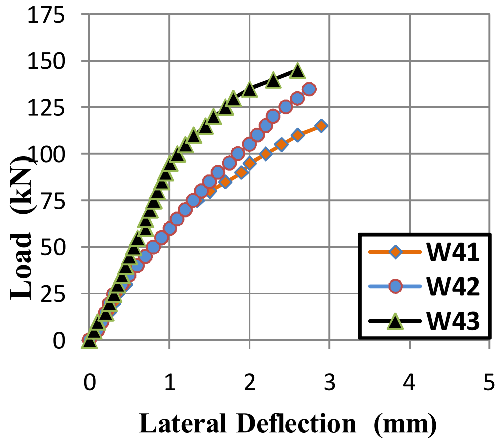

Figure (9): Load - Lateral deflection curves for Group (4) (H/L=2.0 , f_c' =43.6 MPa).

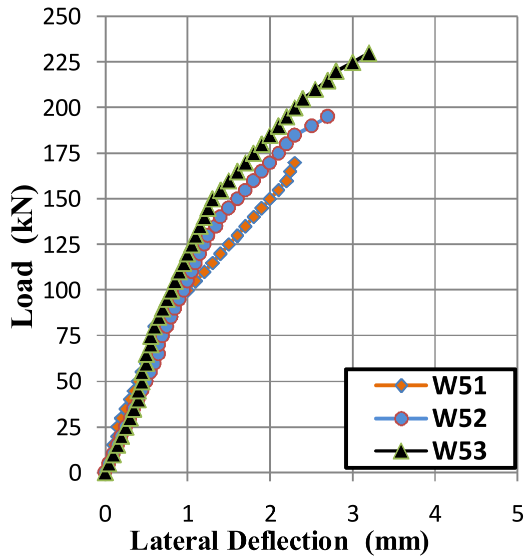

Figure (10): Load - Lateral deflection curves for Group (5) (H/L=2.0 , f_c' =56.7 MPa)

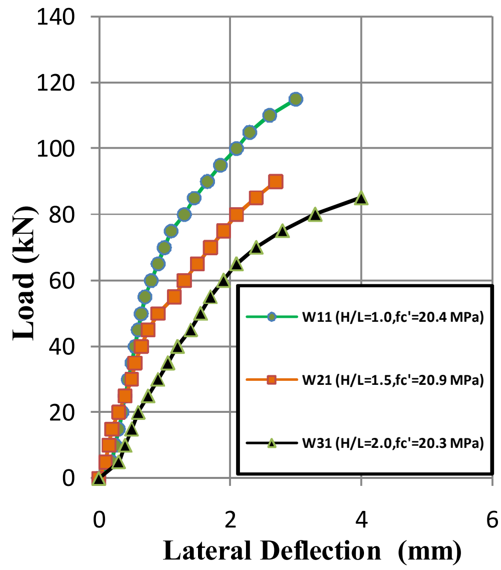

Figure (11): Load -Lateral deflection curves for panels with (H/t=30 , Normal Strength Concrete, Varying aspect ratio)

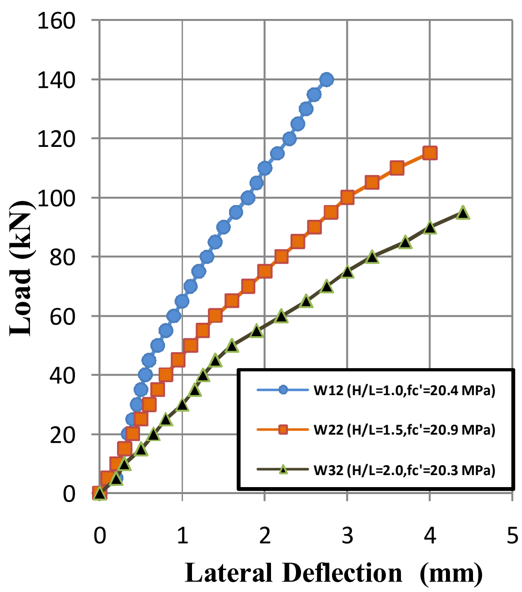

Figure (12): Load - Lateral deflection curves for panels with (H/t=25 , Normal Strength Concrete, Varying aspect ratio).

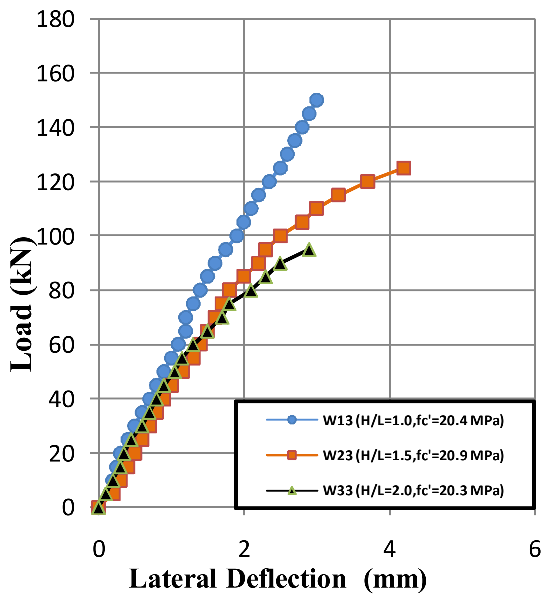

Figure (13): Load - Lateral deflection curves for panels with (H/t=20 , Normal Strength Concrete, Varying aspect ratio).

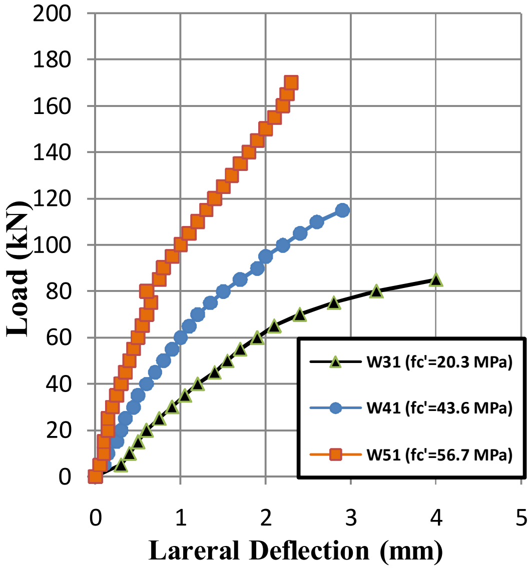

Figure (14): Load - deflection curves for panels with (H/t=30, H/L=2.0 and varying f_c').

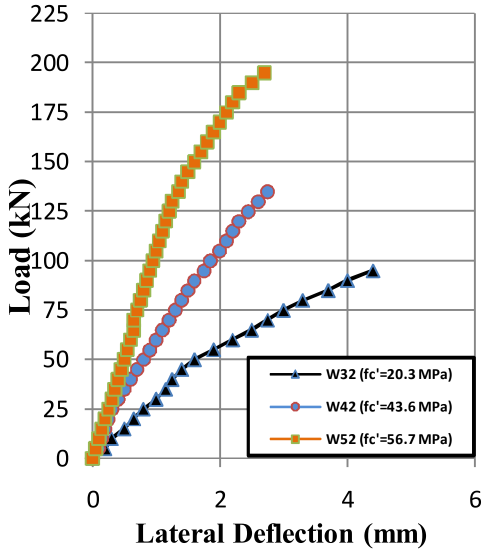

Figure (15): Load - deflection curves for panels with (H/t=25, H/L=2.0 and varying f_c').

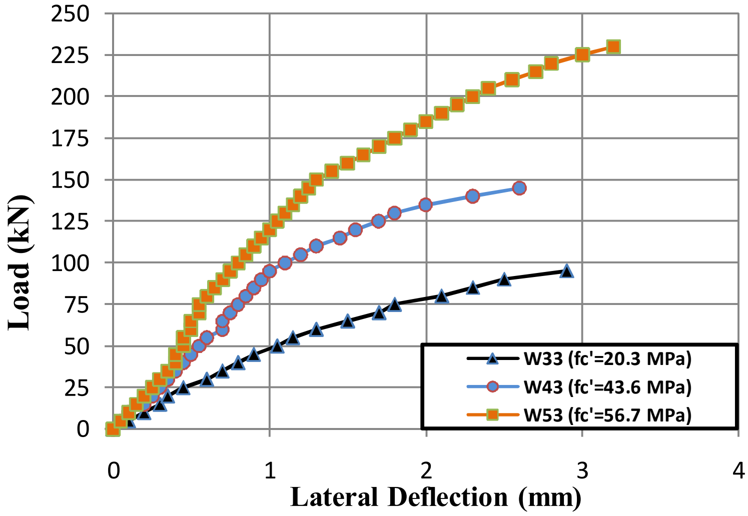

Figure (16): Load - deflection curves for panels with (H/t=20, H/L=2.0 and varying f_c').

Figure (17): Crack Pattern for Wall Panels of Group (1).

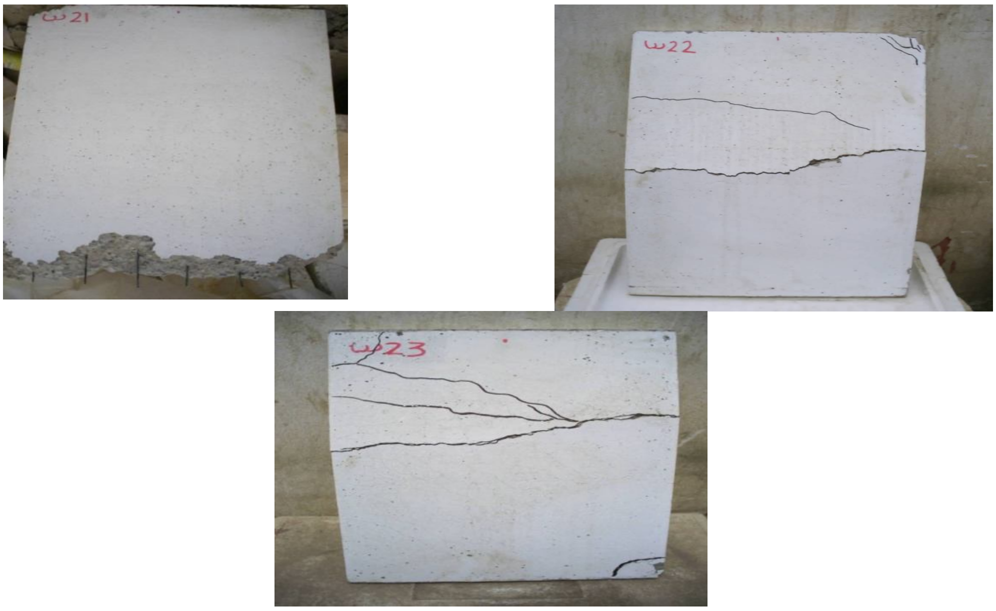

Figure (18): Crack Pattern for wall panels of Group (2).

Figure (19): Crack Pattern for Wall Panels of Group (3).

Figure (20): Crack Pattern for wall panels of Group (4).

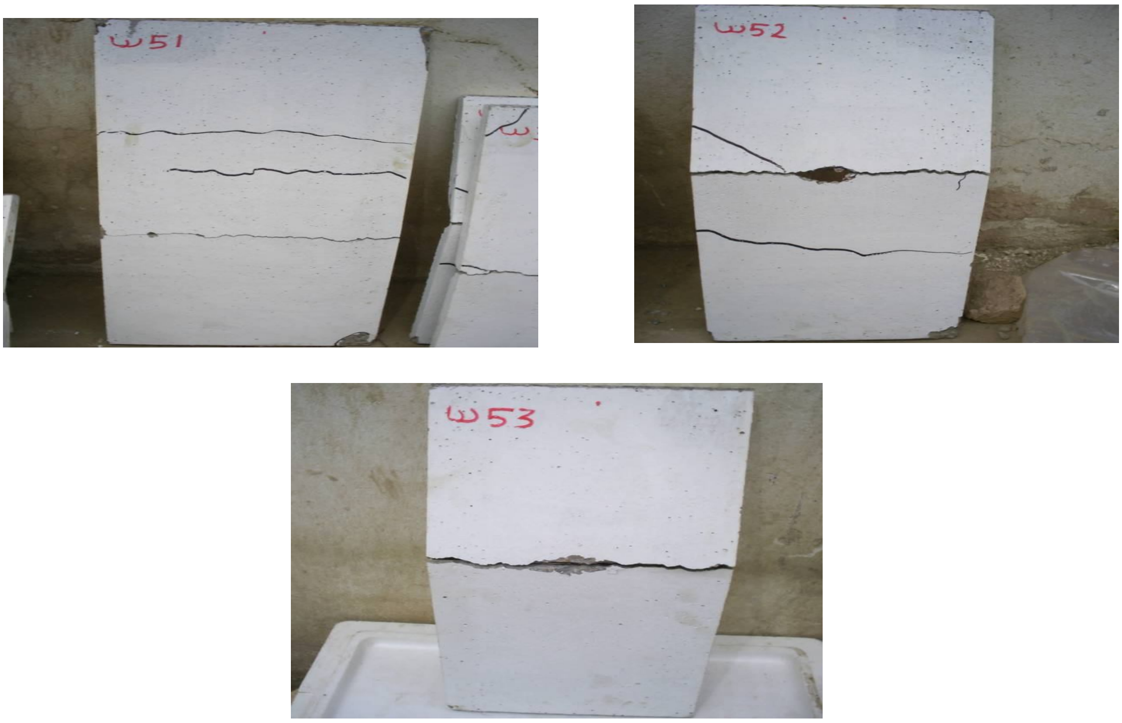

Figure (21): Crack Pattern for Wall Panels of Group (5).

السلوك الإنشائي لألواح جدران من الخرسانة الاعتيادية والعالية المقاومة المعرضة لأحمال محورية لامركزية منتشرة بانتظام

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الخلاصــة.

في اغلب الحالات تتعرض ألواح الجدران الخرسانية إلى أحمال محورية لامركزية منتشرة بانتظام ، وبسبب هذه الأحمال فان ألواح الجدران الخرسانية تنحرف وتفشل بطريقة ما، وان هناك العديد من العوامل المتغيرة التي تؤثر على السلوك الإنشائي لألواح الجدران الخرسانية.

تقدم هذه الدراسة بحثا عمليا للتحري عن السلوك الإنشائي لألواح الجدران الخرسانية المعرضة إلى أحمال محورية لامركزية منتشرة بانتظام ، وكذلك تقيم تأثير العوامل المتغيرة ، نسبة النحافة (الارتفاع \ السمك) ، نسبة الانحدار (الارتفاع \ الطول) ، مقاومة الخرسانة على السلوك الإنشائي لألواح الجدران الخرسانية.

يتضمن البرنامج العملي فحص خمسة عشر لوحا جداريا خرسانيا مثبتا بمفصل في الأعلى والأسفل وحرةً من الجوانب ، بتسليط حمل محوري مع لامركزية تساوي (السمك \6) ، قسمت هذه الألواح إلى خمسة مجاميع ،كل مجموعة تتألف من ثلاث ألواح لكل لوح نسبة نحافة (الارتفاع\ السمك) كالآتي (20 ،25 ، 30)، ثلاثة مجاميع من الخرسانة الاعتيادية المقاومة مع نسبة انحدار (الارتفاع \ الطول) مساوية إلى (1.0 ، 1.5 ، 2.0) لكل مجموعة ، المجموعتان الباقيتان من الخرسانة عالية المقاومة مع نسبة انحدار (الارتفاع \ الطول) مساوية إلى 2.0 لكلا المجموعتين.

تشير النتائج المستحصلة من البرنامج العملي إن انحراف ألواح الجدران الخرسانية يعتمد على نسبة النحافة ونسبة الانحدار ومقاومة الانضغاط للخرسانة ، وان شكل الفشل لألواح الجدران الخرسانية يتأثر كثيرا بنسبة الانحدار (الارتفاع \ الطول) ، وان الألواح ذات نسبة الانحدار الواطئة تميل إلى الفشل بالتهشم ، بينما الألواح ذات نسبة الانحدار العالية تميل إلى الفشل بواسطة الانبعاج.

الكلمات الرئيسية: أحمال محورية، ألواح الجدران الخرسانية، نسبة النحافة، مقاومة الخرسانة، نسبة الانحدار.