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STRUCTURAL BEHAVIOR OF MODIFIED REACTIVE POWDER AND REACTIVE POWDER CONCRETE WALL PANELS SUBJECTED TO HIGH TEMPERATURE

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ABSTRACT: - This research presents an experimental work to investigate the structural behavior of concrete wall panels subjected to high temperature (350^0c) and distributed load. Eight specimens of wall panels were tested in this study and divided equally in to four groups, each group consists of two wall panels first one with slenderness ratio (H/t=20) and the other one with slenderness ratio (H/t=14). The first group with normal strength concrete, the second with high strength concrete, the third with modified reactive powder concrete (MRPC) and the fourth with reactive powder concrete (RPC). The results shows that for wall panels with slenderness ratio H/t=20 the lateral deflection decrease by about 19% as concrete strength (fc') increase from 30.2Mpa to 69.5Mpa and about 58% as concrete strength (fc') increase to 128Mpa at failure load, while for H/t=14 the lateral deflection decrease by about 49% as concrete strength (fc') increase from 30.2Mpa to 69.5Mpa and about 28% as concrete strength (fc') increase to 128Mpa at failure load. For Wall panels with high strength concrete, MRPC, and RPC there is no big difference between lateral deflection at the linear part for panel with H/t=14and panel with H/t=20. This difference increases at the nonlinear part of the curves. The failure load increases by about 62% as the concrete strength (fc') increases from 30.2Mpa to 69.5Mpa and increases to 50% as the concrete strength (fc') increases from 69.5Mpa to 128Mpa for panels with H/t=14. The failure load increases by about 89% as concrete strength (fc') increases from 30.2Mpa to 69.5Mpa and the failure load increases by 56% as the concrete strength (fc') increases from 69.5Mpa to 128Mpa for panels with H/t=20. For all tested panels the failure mode was buckling failure and the cracks for wall panels with RPC and MRPC are close to center of the panel.
Keywords: High temperature, Modified reactive powder concrete, Wall panel and Concrete strength

1. INTRODUCTION

Reactive powder concrete (RPC) is one of the latest type of ultra-high performance concrete, is caricaturized by very dense matrix due to the improvement of the granular packing of the very fined powders and a firm microstructure. Depending on curing method, the compressive strength may reach to 800 MPa [(1)]. With this advent 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 wall[(1)]. The fire effect may be defined in terms of elevated temperatures that are considered as an indirect fire effect. The main effects of fire may be considered as loss in compressive strength, cracking and spelling of concrete, and destruction of the bond between the cement paste and the aggregates as well as the gradual deterioration of the hardened concrete[(2)]. Elevating the temperature results in decreasing the strength of concrete up to failure depending on the temperature and exposure time. The first effects of a slow temperature rise in concrete will occur between 100 and 200℃ when the free moisture, contained in the concrete mass, evaporates [(3)]. Direct exposure can result in spalling through generation of high internal steam pressures. As the temperature approaches 250 ℃, dehydration or loss of the non-evaporable water begins to take place. Sizable degradation in compressive strength is usually experienced between 200 and 250°C [2, 3]. At 300℃ the strength reduction is in the range of 15-40%. At 550℃ the reduction in compressive strength would typically range from 55-70%of its original value[4].The range between 400°C and 800°C is critical to the strength loss of normal strength concrete (NSC)[4]. At a temperature over 600°C, all tested concretes suffer deterioration and only a small value of the initial strength is left [5].

2- SIGNIFICANT OF RESEARCH

Few researches were deal with the effect of high temperatures on the structural behavior of concrete walls. In addition, there were fewer or no researches that studied the influences of elevated temperature degrees on the walls made with RPC. This research is an attempt to eliminate this leak investigations.

3- EXPERIMENTAL WORK

Eight specimens of wall panels were tested in this study and divided equally in to four groups, the first group with normal strength concrete (NSC), the second with high strength concrete (HSC), the third with modified reactive powder concrete (MRPC), and the fourth with reactive powder concrete (RPC). The dimensions for tested wall panels were (700*500mm) with thickness of 35mm and /or 50mm. All tested samples were reinforced with one layer of welded steel mesh of (4mm diameter bars@90mmc/c) and placed centrally through the panel thickness. The horizontal and vertical reinforcement ratio (ρh, ρv) are equal to 0.0032 for all wall panels samples according to the minimum requirements of American Concrete Institute (ACI318-08) [6]. Fig. (1) shows the dimensions for wall panels used in this study.

The materials used in concrete mixes are Ordinary Portland cement type (I)[7] , natural sand which has fineness modulus of (2.6) and Crushed gravel with maximum size of (14mm)[8]. For the concrete mixes used in RPC and MRPC, super plasticizer (high rang water reducing agent) based on poly carboxylic, Glenium 51 was used with normal dosage of (0.5-0.8) L/ 100 kg of cementation mass.

The mix proportion of each wall panels in group shown in Table (1).

The properties of steel fiber used in MRPC and RPC mix are shown in Table (2).

3.1-Test Rig Set-Up

For axially loaded wall, two main conditions must be achieved. Firstly, the supports must be allowed to rotate freely. Secondly, the axial load must be uniformly distributed across the length of the test panel. 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 were also 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. This operation was made very carefully and with high accuracy to ensure a straight lines and no gaps allowed to be within the support and welding. Details of the simply supported top and bottom hinged edge are shown in Fig. (2a), Fig. (2b) shows a photo for the tested panels with the simply supported edges.

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 applying the load to the failure of the panel.

Fig. (3) shows the universal testing machine and steel reinforcement used in tested wall panels

4- RESULT AND DISCUSSION

The average of three (150×150×150 mm) cubes were taken to express the compressive strength of concrete, as shown in Table (3).

All the samples of wall panels were subjected to high temperature of (350^0c) before testing. The results express the lateral deflection, compressive strength verse the failure load, cracks and failure type for all tested wall panels. Fig (4) , Fig(5), and Fig(6) and Fig (7), show the lateral deflection for tested wall panels with NSC, HSC, MRPC and RPC respectively.

From Fig (4) 0f wall panels with ($fc^'$=30.2Mpa), slenderness ratio (H/t=20), and (H/t=14), it can be seen that the wall panel exhibits a ductile failure.The lateral deflection for wall panel with slenderness ratio (H/t=14) is less than that for wall panel with slenderness ratio (H/t=20) by about 28% at the cracking load. Moreover, the failure load is more than by about 41%.

From Fig (5) for wall panels with ($fc^'$=69.5Mpa), slenderness ratio (H/t=20), and (H/t=14), it can be seen that the wall panel with slenderness ratio (H/t=20) exhibits a ductile behavior while the wall panel with (H/t=14) exhibits a brittle. From Fig (5), it can be seen also that there is no big difference in lateral deflection between the two wall panels (H/t=14 and H/t=20) until the applied load reaches to 170kN. After that it becomes clear and reaches 7% with the lateral deflection for wall panel with H/t=14 less than the lateral deflection for wall with H/t=20. The failure load for panel with H/t=14 is more than that for panel with H/t=20 by about 21%.

From Fig (6) for wall panels with ($fc^'$=100Mpa), slenderness ratio (H/t=20), and (H/t=14), it can be seen that the wall panel exhibits more brittle behavior than the panels with ($fc^'$=69.5Mpa) shown in Fig (5). The two curves of Fig (6) show a linear behavior and no big difference in lateral deflection between the two curves up to load 180kN. After that the two curves show a nonlinear behavior up to failure. In nonlinear part, the lateral deflection for wall panel with H/t=14 is less than the lateral deflection for wall panel with H/t=20 by about 30% at the failure load.

Besides, the failure load for wall panel with H/t=14 is more than the failure load for wall panel with H/t=20 by about 16%.

From Fig (7) for wall panels with ($fc^'$=128Mpa), slenderness ratio (H/t=20), and (H/t=14), it can be seen that the wall panels exhibits more flexural behavior than the wall panels with ($fc^'$=69.5Mpa and $fc^'$=100Mpa). The two samples shows that there is no big different in the lateral deflection between the curves up to load 230kN. After that the two samples shows anon linear behavior up to failure. In the nonlinear part, the lateral deflection for wall panels with H/t=14 is less than the lateral deflection for wall panel with H/t=20 by about 43% at the failure load.

Besides, the failure load for wall panel with H/t=14 is more than the failure load for wall panel with H/t=20 by about 16%.

Fig. (8) and Fig.(9) shows Lateral deflection for panels with slenderness ratio (H/t=20) and (H/t=14),it can be seen that lateral deflection decrease by about 19% as concrete strength ($fc^'$)increase from 30.2Mpa to 69.5Mpa and about 58% as concrete strength ($fc^'$) increase to 128Mpa at failure load, while for H/t=14 the lateral deflection decrease by about 49% as concrete strength ($fc^'$) increase from 30.2Mpa to 69.5Mpa and about 28% as concrete strength ($fc^'$) increase to 128Mpa at failure load.

Fig (10) shows the failure load verses the concrete strength ($fc^'$). It can be seen that the relation seems to be linear for panels with H/t=14 and H/t=20. It can also be seen that for H/t=14, the failure load increases by about 62% as the concrete compressive strength increases from 30.2Mpa to 69.5Mpa. The failure load increases to 50% as the concrete compressive strength increases from 69.5Mpa to 128Mpa for H/t=14.

For H/t=20 the failure load increases by about 89% as the concrete compressive strength increase from 30.2Mpa to 69.5Mpa. The failure load increases to 56% as the concrete compressive strength increases from 69.5Mpa to 128Mpa for H/t=20.

Also it can be seen that the failure load for panel with H/t=14 is more than that for panel with H/t=20 by about (41%, 21%, 13%, 16%) at concrete compressive strength (fc'=30.2Mpa, 69.5Mpa, 100Mpa and 128Mpa) respectively.

From above, for wall panels subjected to high temperature at about 350^0C and applied to load, the lateral deflection decrease as the slenderness ratio decrease and the concrete compressive strength increase. The large difference between the failure load for wall panel with RPC mix and other wall panels may be due only a fine materials used in RPC mix which give better homogeneity as only a very fine sand without coarse aggregate, which give more bond strength between the concrete mix. Moreover the ability of steel fiber to absorbed large energy before failure, makes the failure load for wall panels with MRPC and RPC more than the failure load for other panels with NSC and HSC. The failure load and the cracking loads (according to the first crack) are given in table (4).

Photos in Fig (11), Fig. (12), Fig.(13) and Fig.(14) show the cracks pattern for the wall panels after exposure to temperature of 350^0C and a test under load up to failure. It can be seen that the failure was buckling failure and the cracks for wall panels with RPC and MRPC are close to center of the panel.

5-CONCLUSIONS

As the slenderness ratio decreases the lateral deflection decreases under the same applied load and temperature. For Wall panels with high strength concrete, MRPC, and RPC, there is no big difference between lateral deflection at the linear part for panel with H/t=14and panel with H/t=20. This difference increases at the nonlinear part from the curves. The structural behavior of the wall panels seems to be flexural behavior as the concrete strength for panels increases. Panels with H/t=14 and H/t=20, the failure load increases by about (62%and 89%) respectively as the concrete strength (fc'') increases from 30.2Mpa to 69.5Mpa, and increases to (49% and 56%) respectively as the concrete strength (fc') increases from 69.5Mpa to 128Mpa. For all tested panels, the failure mode was buckling failure.

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Table (1) Mix proportion

Group	Mix designation	w/c Ratio	Mix Properties (kg/m^3)					Steel Fiber kg/m^3
			Water	Cement	Sand	Gravel	SP	
A	NSC	0.3	135	450	600	1150	6.75	
B	HSC	0.37	170	450	780	885	14.5	
C	MRPC	0.22	205	933	539	489	234	12.7
D	RPC	0.23	215	933	1030	--	234	12.7

Table (2) Properties of steel fiber *

Property	Density Kg/m^3	Ultimate Strength MPa	Modulus of Elasticity MPa	Average length mm	Normal Diameter mm	Aspect Ratio(L/d)
Specification	7860	1130	200*10^3	250	0.4	625

*Provided by the manufacturer

Table (3) Compressive Strength Results

Group	Cube strength (Mpa)
NSC	30.2
HSC	69.5
MRPC	100
RPC	128

Table (4) The Cracking and Failure Loads

panel	Panel with normal concrete		Panel with high strength concrete		Panel with MRPC		Panel with RPC	
	H/t=20	H/t=14	H/t=20	H/t=14	H/t=20	H/t=14	H/t=20	H/t=14
Failure load(KN)	97.5	137.5	184.5	222.5	228.5	265	287.5	332.5
Cracking load(KN)	57	62	146	200	170	230	240	265

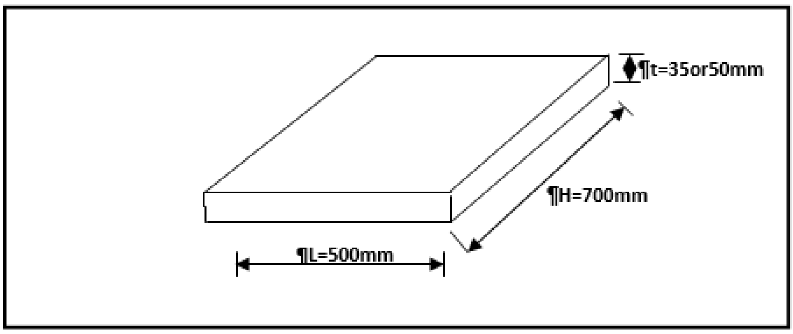


Fig. (1) Dimensions of wall panel

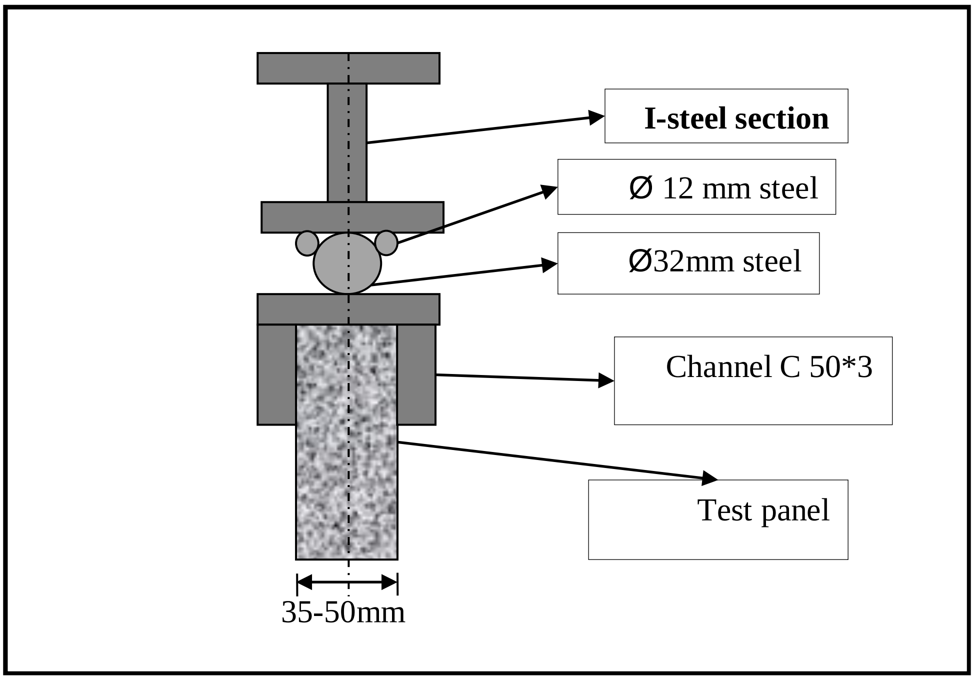


Fig. (2a) Detail of Supports used in this work

Fig. (2b) tested panel with the simply supported edges

a) Universal testing machine b) steel reinforcement

Fig. (3) Universal testing machine and steel reinforcement

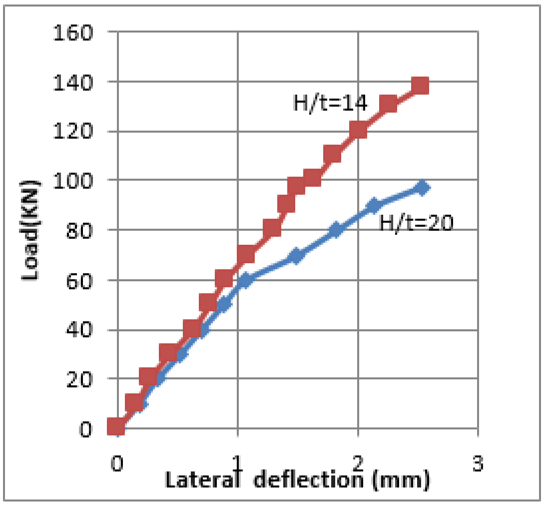


Fig.(4) Lateral deflection for panels with normal strength concrete (fc'=30.2Mpa)

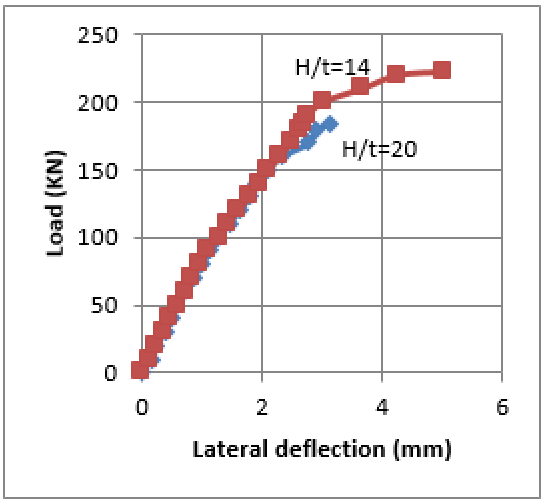


Fig.(5) Lateral deflection for panels with high strength concrete (fc'=69.5Mpa)

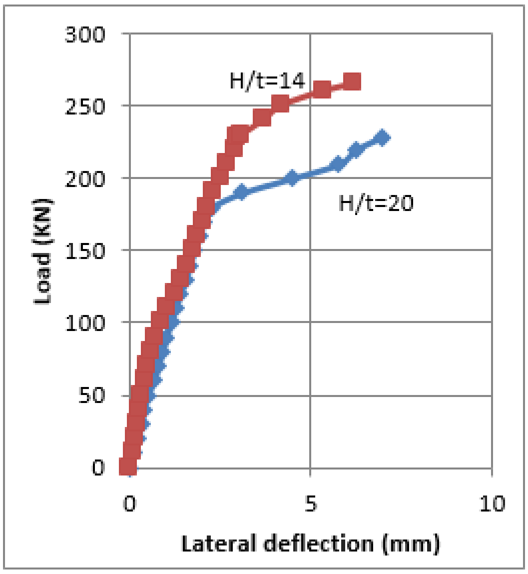


Fig. (6) Lateral deflection for panels with MRPC(fc'=100M)

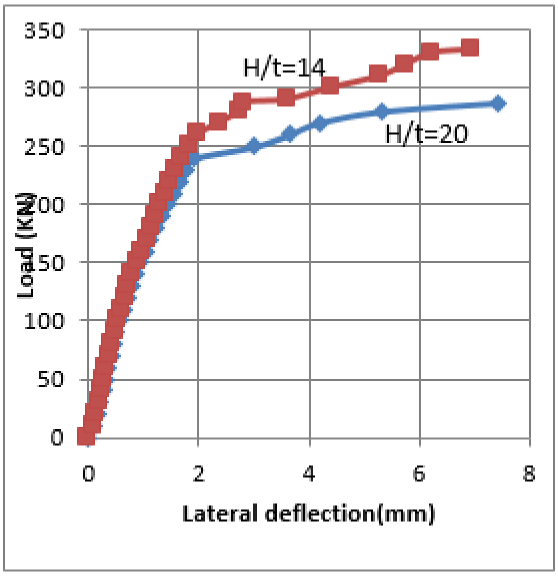


Fig. (7) Lateral deflection for panels with RPC(fc'=128Mpa)

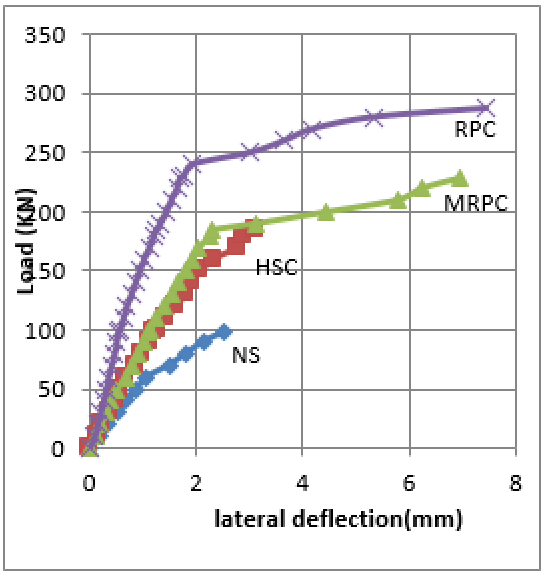


Fig.(8) Lateral deflection for panels with slenderness ratio

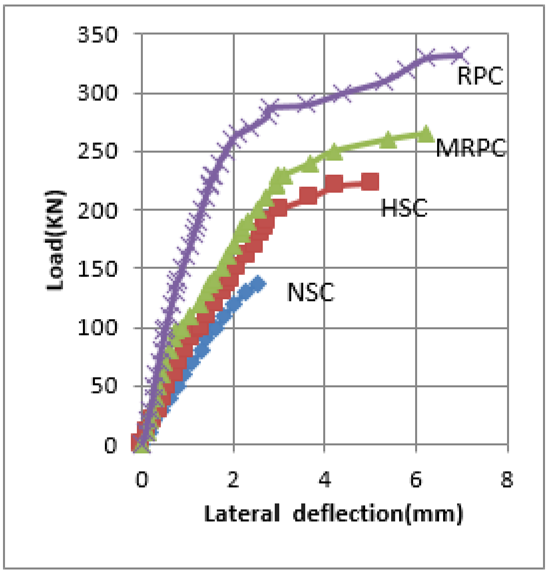


Fig.(9) Lateral deflection for panels with slenderness ratio (H/t=14), (t=50mm)

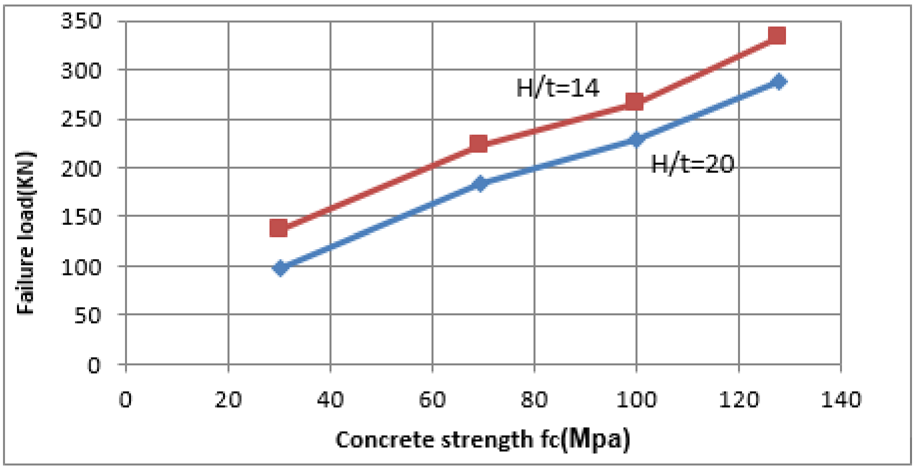
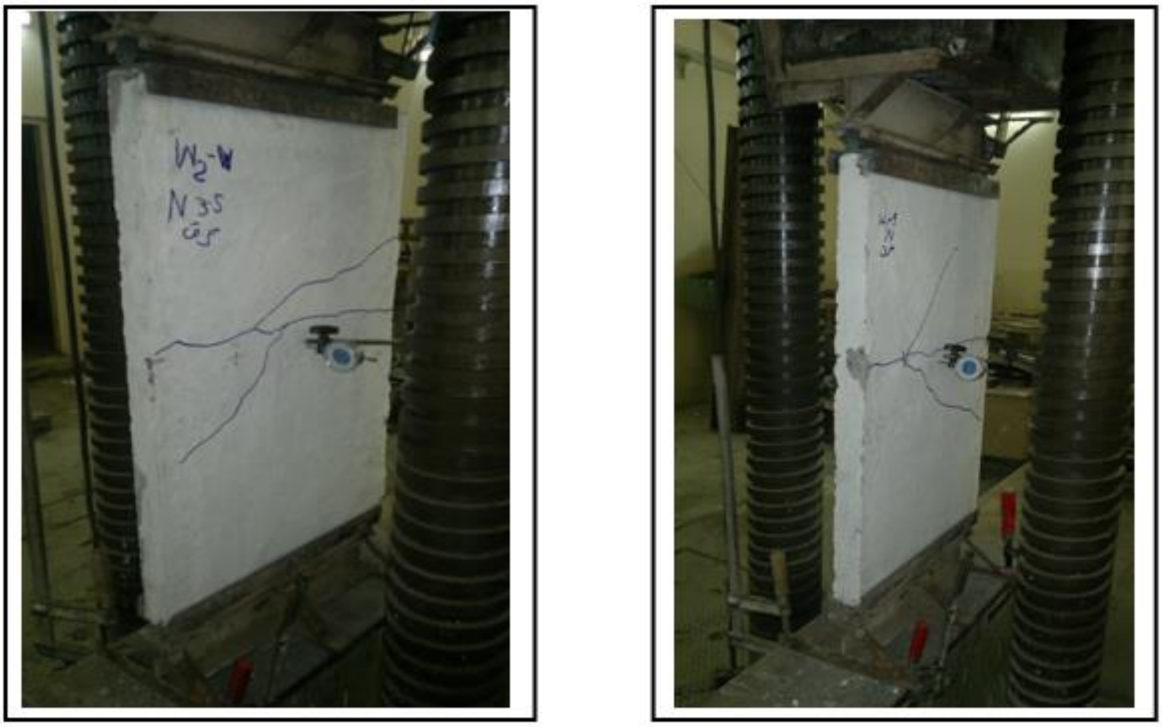


Fig. (10) Effect of concrete strength fc'on failure load for panels with normal, high strength, MRPC and RPC



((H/t=20)

(H/t=14)

Fig. (11) Tested wall panels with slenderness ratio (H/t=20) and (H/t=14) for normal strength concrete after exposure to temperature and axial load

(H/t=20)

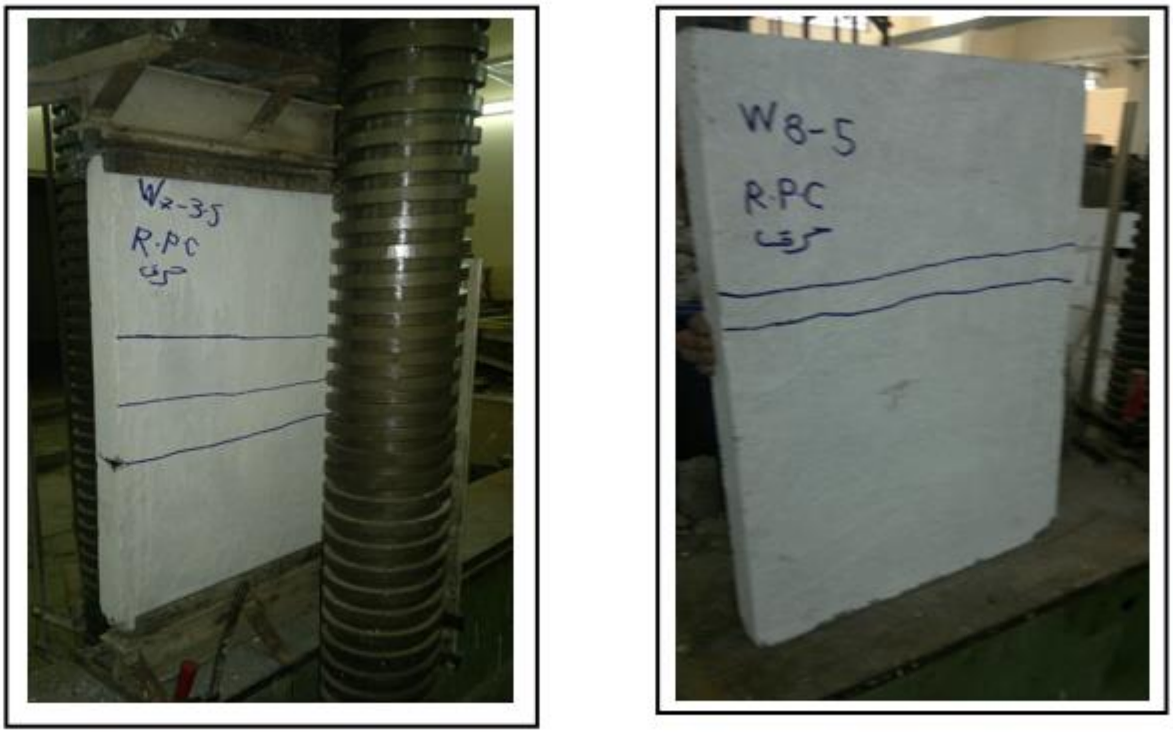
(H/t=14)

Fig. (12) Tested wall panels with slenderness ratio (H/t=20) and (H/t=14) for MRPC after exposure to temperature and axial load

(H /t=20)

(H /t=14)

Fig. (13) Tested wall panels with slenderness ratio (H/t=20) and (Hlt=20) for high strength concrete after exposure to temperature and axial load



(H/t=20) (H/t=14)

Fig. (14) Tested wall panels with slenderness ratio (H/t=20) and (H/t=14) RPC after exposure to temperature and axial load

الخلاصة

يتضمن هذا البحث دراسة عملية لتصرف الجدار اللوحي الكونكريتي المعرض لدرجة حرارة عالية (350^0C) و حمل منتشر . أظهرت النتائج أنه للنماذج ذات نسبة النحافة (H/t=20) فأن الأنحراف الجانبي يقل بنسبة 19% بزيادة مقاومة الانضغاط من 30.2Mpa الى 69.5Mpa و بنسبة 58% بزيادة مقاومة الانضغاط الى 128Mpa عند حمل الفشل. بينما للنماذج ذات نسبة النحافة (H/t=14) فأن الأنحراف الجانبي يقل بنسبة 49% بزيادة مقاومة الانضغاط من 30.2Mpa الى 69.5Mpa و بنسبة 28% بزيادة مقاومة الانضغاط الى 128Mpa عند حمل الفشل. تم دراسة ثمانية نماذج من الجدار اللوحي الكونكريتي وقسمت هذه النماذج الى اربعة مجاميع متساوية كل مجموعة تحتوي على نموذجين الاول يكون ذا نسبة نحافة (H/t=20) والثاني ذا نسبة نحافة (H/t=14) . المجموعة الاولى تتضمن نماذج تكون فيها الخرسانة ذات مقاومة انضغاط اعتيادية و الثانية ذات مقاومة انضغاط عالية و المجموعة الثالثة الخرسانة المستعملة هي خرسانة المساحيق الفعالة المعدلة (MRPC) و المجموعة الرابعة الخرسانة المستعملة هي خرسانة المساحيق الفعالة (RPC). النماذج ذات مقاومة الانضغاط العالية (MRPC,RPC) تظهر النتائج انه لا يوجد اختلاف كبير في الجزء الخطي من الانحراف الجانبي للوحي الكونكريت ذا نسبة نحافة (H/t=20 و H/t=14) , هذا الاختلاف في الانحراف الجانبي للوحين يزداد في الجزء اللاخطي. حمل الفشل يزداد بنسبة 62% بزيادة مقاومة الانضغاط للكونكريت من 30.2Mpa الى 69.5Mpa وتكون نسبة الزيادة في حمل الفشل بنسبة 50% بزيادة مقاومة الانضغاط من 69.5Mpa الى 128Mpa للجدار اللوحي ذا نسبة نحافة H/t=14 . كما ان حمل الفشل يزداد بنسبة 89% بزيادة مقاومة الانضغاط للكونكريت من 30.2Mpa الى 69.5Mpa وتكون نسبة الزيادة في حمل الفشل بنسبة 56% بزيادة مقاومة الانضغاط من 69.5Mpa الى 128Mpa للجدار اللوحي ذا نسبة نحافة H/t=20 . لجميع النماذج من الجدار اللوحي الكونكريتي يكون نوع الفشل هو فشل انبعاج وتكون الشقوق بالنسبة للالواح المصنوعة من (MRPC,RPC) بالقرب من المنتصف.