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Flexural Behavior of RC Sandwich and Hollow Block Bearing Walls

Al-Tuhami AbuZeid Al-Tuhami AbdAllah and Ahmed Ismail Gabr

Abstract*— *The housing problem occupies a leading position in the list of social and economic priorities in many countries. Thus, there is an urgent need for alternative systems to fulfill the rapid expansion of the housing construction demand in terms of quality, strength properties and environmental aspects. Al-Tuhami [1] presents a method and technique for enhancing the mechanical properties of traditional concrete sandwich panel bearing walls. The suggested technique was based on presenting a fully composite action of sandwich panel bearing walls by adding longitudinal and transverse concrete ribs along with the existing two parallel concrete wythes. The aim of this paper is to examine the effect of the presence of ribs that connects the two concrete wythes for enhancing structural behavior of the wall panels that exposed to bending loads. Experimental work and 3D numerical analysis of sandwich panels as well as hollow block bearing wall panels subjected to flexural load is conducted. Parametric study is carried out in order to focus on the main sensitive parameters as span to depth ratio and wythe thickness that influence the flexural capacities of wall panels.

***Index Terms*— 3D numerical analysis, experimental work, flexural behavior, hollow blocks bearing walls, sandwich panels.**

I. INTRODUCTION

Structural concrete sandwich panel (SCSP) has been grown in North America for more than 50 years and the uses increased gradually because of the advantages they provide [2]. Light weight, energy savings, sound abatement, affordability and durability are the benefits of buildings constructed with SCSP. The panel consists of two concrete wythes and a polystyrene insulation layer in between the wythes. Shear connectors are provided to prevent individual wythe buckling. Salmon et al. [3] presents the results of a full scale test of prototype sandwich panels under transverse loading in a vertical position. Nighawan [4] measured, experimentally, the interface shear force and designed the shear connectors. Eiena et al. [5] used plastic composite diagonal elements in sandwich panels as shear connectors for increasing the thermal insulation of this system. One of the most drawbacks of sandwich panel walls is the buckling failure of shear connectors as a result of applying lateral loads (Kabir 2005 [6] and Fouad et. al, 2009 [7]). This phenomenon leads to a large decrease in stiffness and thus decreasing the maximum load carrying capacity of the Wall panels. Al-Tuhami [1] presents a method and technique for enhancing the mechanical properties of traditional sandwich panel bearing walls. The suggested technique was based on presenting a complete interaction wall panel in the two directions, by using fully interacting longitudinal and transverse concrete ribs along with the two parallel concrete layers.

The behavior of concrete sandwich panel and hollow block bearing walls are rather complicated due to non-linearity of their constituent materials and the wall configuration. Due to progressing knowledge and capabilities of computer technologies; the use of computer software to model these panels is much faster, and extremely cost-effective.

In this study the sandwich panels and hollow block bearing wall panels are modeled and analyzed using the commercial finite element software ANSYS [8]. In order to calibrate the FE model, the load-deflection response resulting from experimental work done by the authors is compared to the numerical results obtained by the present study. The effects of such factors as the span to depth ratio and layer thickness are studied.

II. EXPERIMENTAL WORK

This part presents an experimental study to investigate the structural behavior of precast hollow blocks bearing walls subjected to out-of-plane concentrated line load.

The steps of making the test hollow blocks bearing wall specimens were presented in Al-Tuhami [1] and E.G. patent number 26309.

Testes are carried out on two full-scale wall specimens having a concrete category of 25 N/mm^2 after 28 days. One specimen is taken as a reference wall representing sandwich panel and the other is made to represent the suggested hollow block bearing wall technique. The total length of each specimen is 280 cm and the width is 100 cm while the depth is 140-mm.

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The reference specimen panel SW1, as shown in Fig. 1, composed of the followings; a) a polystyrene core plate of 8 cm thickness with density of 15 kg/m^3, b) two parallel reinforced concrete wythes with of 3 cm thickness each. The second specimen SW2 Fig. 2 consists of light weight filling material blocks (polystyrene foam with density of 15 kg/m^3), two parallel reinforced concrete wythes and two longitudinal and four transverse reinforced concrete ribs. The longitudinal and transverse ribs reinforcement details are shown in Fig. 2. For the two test specimens, the reinforcement of each concrete wythe is electro-welded steel wire meshes formed by 2.7 mm diameter wire with horizontal and vertical spacing of about 70 mm. For the reference wall specimen SW1 only, the two parallel steel meshes are connected by ties from steel wires of 3 mm diameter.

In Specimen SW1 only, the two parallel steel meshes are connected by ties from steel wires of 3 mm diameter. The number of ties is 55 per square meter of the panel SW1.

It should be noted that, there are no any tie connectors used between the two parallel concrete layers in specimen SW2.

The design cube compressive strength of the concrete was 25 MPa after 28 days. The proportion of the concrete mix was ordinary Portland cement, siliceous sand and fine grained dolomite fragments are used in the concrete mix. The ultimate tensile strength of the reinforcing wythes meshes was 360 MPa. The used dolomite fragments are 90% passing sieve size 4.75mm, and the remaining 10% are passing sieve size 10mm. All the tested dolomite fragments are retained on sieve size 2.18 mm.

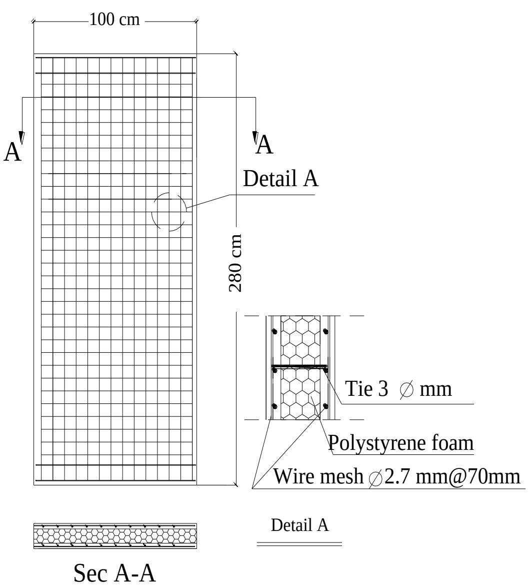

Fig.1: Dimensions and steel layout of sandwich panel specimen SW1.

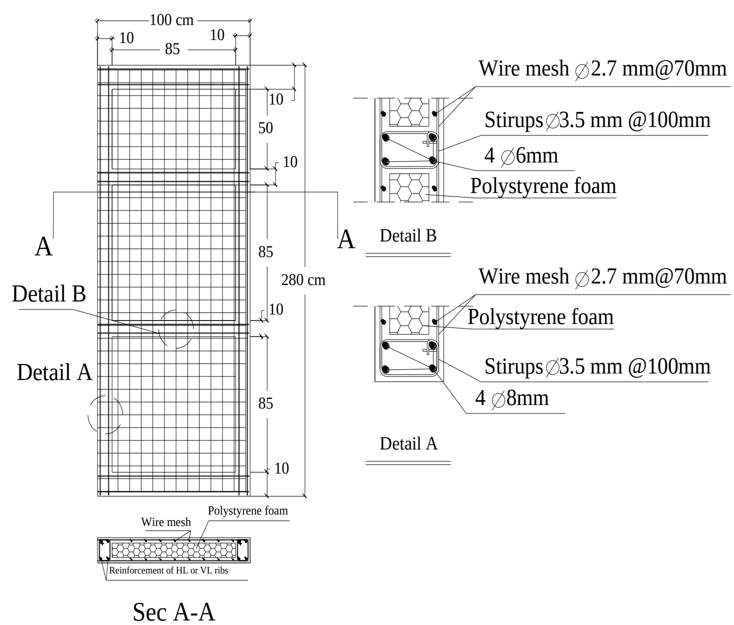

Fig. 2: Dimensions and steel detailing of suggested hollow block bearing wall specimen SW2.

Testing of the panel specimens are performed in a horizontal position using a test steel frame. The typical test setup and layout of the specimen is shown in Fig. 3. Load is applied to the specimens by oil jack with 230 kN capacities in a vertical position. The vertical displacement of the specimen is monitored using an LVDT (Linear Variant Differential Transformer) located at mid-span of specimen. The load was applied to the panels in increments. There was a pause after each load increment application to allow time to check for the development of any cracks in concrete, measure crack widths, and inspect for any structural distress that might have occurred. For each load increment the corresponding mid span displacement is recorded. After reaching a maximum mid span deflection and its corresponding load for each specimen, the load is slowly dropping until development of complete fracture in order to inspect the mode of failure and the connectors inside the sandwich panel specimen SW1. Panel specimens' configurations and experimental results are shown in Table 1.

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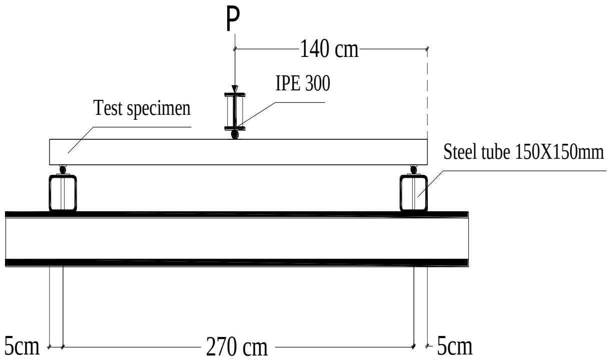

Fig. 3: Typical test setup (Dimensions in mm.)

Table 1: Panel specimens' configurations

Spec. No	Dimen. cm.	Ribs		f_{cu} N/mm^2	No of ties/m^2
		Hl. Ribs	Vl. Ribs		
SW1	100x270	4	2	25	-
SW2	100x270	-	-		55

A. Analysis of the Experimental Results

The load deflection curves of reference panel specimen SW1 as well as sandwich ribbed panel SW2 were shown in Fig. 4. The load-deflection behaviour of the two specemen like to be linear up to yield load folowed by nonlinear behaviour untile failure. Due to flexural stress, failure of shear connectors in panel SW1, in compression zones, occur as buckling. Thus the load carrying capacity is greatly reduced due to decreases the flexural stiffness of the sandwich panel SW1. These results assure the observation of Kabir 2005 and Fouad et. al, 2009 for sandwich panels subjected to bending load with ties or truss shear connector [5],[6]. This phenomenon may be attributed to shear connectors that pass through the insulating layer with a very weak density helps in the incidence of buckling to those connectors if exposed to compressive stresses. Whilst, presence of concrete ribs provides confinement action of stirrups and enhancement the resistance of buckling failure for these stirrups.

Fig. 4 shows the load deflection curves for wall SW1 and SW2. It could be seen that that the ultimate load for panel specimen SW1 was 6.1 kN at deflection of 33.61mm, while for panel SW2 the ultimate load was 36.95 kN at deflection equal to 19.67mm. Results of SW2 panel showed large increase in yield load along with a very noticeable increase in the overall panel stiffness compared to SW1 panel. The ultimate point load has increased using the present suggested technique by more than six hundred percent with increasing of panel concrete weight 40% only.

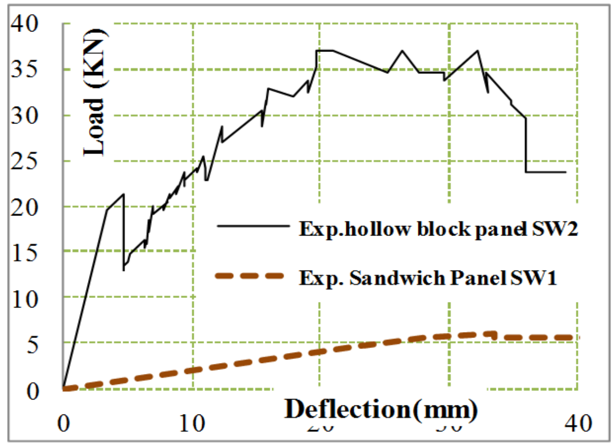

Fig.4: Experimental results of load deflection curves for suggested hollow block and sandwich panels

III. 3D NUMERICAL ANALYSIS OF HOLLOW BLOCK BEARING WALLS

The behavior of concrete sandwich panel and the suggested hollow block bearing walls are rather complicated due to non-linearity of their constituent materials and the wall configuration. The use of finite element method (FEM) provides a powerful means, which can be used to simulate the behavior of this type of panels under different load conditions. In this study the sandwich panels and hollow block bearing walls are modeled and analyzed using the commercial finite element software ANSYS 14.0.

A. Material Elastic-plastic Model

Willam and Warnke (1974) [9] developed a widely used model for the triaxial failure surface of unconfined plain concrete. The mathematical model of this failure surface considers a sextant of the principal stress space because the stress components are ordered according to major principal stresses σ1 ≥σ2 ≥σ3. The most significant nonzero principal stresses are in the x and y directions, represented by σ_{xp} and σ_{yp}, respectively. The failure surface in principal stress-space is shown in Fig. 5. The deviatoric trace is described by polar coordinates *r*, and θ where *r* is the position vector locating the failure surface with angle, θ. The failure surface is defined as:

$$\frac{1}{z}\frac{\sigma_a}{f_{cu}} + \frac{1}{r(\theta)}\frac{\tau_a}{f_{cu}} = 1 \qquad (1)$$

Where: σ_a and τ_a are the average stress components, z is the surface apex and f_{cu} is the uniaxial compressive

rebar are capable of tension and compression, but not shear.

The compressive uniaxial stress-strain relationship for the concrete model was obtained using the following equations to compute the multilinear isotropic stress-strain curve for

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strength.

The constitutive model based upon the Willam and Warnke [9] failure criterion is one of many models available in ANSYS program. The yield condition can be approximated by three or five parameter models distinguishing linear from non-linear and elastic from inelastic deformations using the failure envelope defined by a scalar function of stress $f(\sigma) = 0$ through a flow rule, while using incremental stress-strain relations.

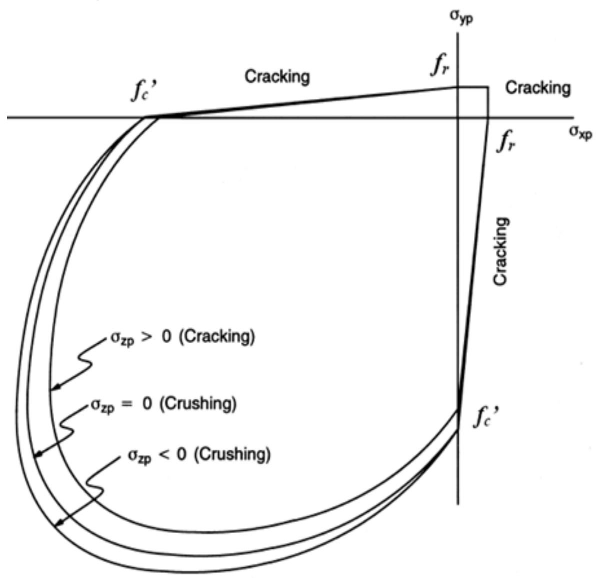

Fig. 5: William and Warnke failure surface in principal stress space with nearly biaxial stress.

The steel reinforcement material was modeled using elastic-perfectly plastic Von Mises criterion.

B. ANSYS Finite Element Model

The numerical model used in ANSYS program mainly consists of two types of element. The first is a solid element, SOLID65, used to model the concrete, while the second is a Link8 element that used to model the steel reinforcement. The solid element has eight nodes with three transitional degrees of freedom at each node. The most important aspect of this element is the treatment of nonlinear material properties, e.g. cracking in the three orthogonal directions, crushing in tension regime, plastic deformation and creep and crushing in compression. The the concrete (MacGregor, 1992) [10]:

$$f = \frac{E_c \varepsilon}{1 + \left(\frac{\varepsilon}{\varepsilon_0}\right)^2} \qquad (2)$$

Where: f is the stress at any strain ε in *psi* units; ε is the strain at stress f and ε_0 is the strain at ultimate compressive strength f_c', $\varepsilon_0 = \frac{2 f_c'}{E_c}$ and $E_c = \frac{f}{\varepsilon}$.

C. Material Properties

Compressive concrete strength, f_{cu}, according to laboratory results was 25 MPa while, tensile stress of concrete at failure f_t was taken 2.5 MPa, concrete modulus of elasticity. Parameters and material properties needed to define the material models can be found in Table 2.

The details of presenting 3-D finite element model includes: boundary conditions; applied loads; concrete and steel elements for both sandwich and hollow blocks panels are shown in Fig. 6. The insulation layer and hollow blocks are not represented in this model and it taken as free spaces.

D. Effect of the Presence of Ribs on the Flexural Behavior

In order to investigate the presence of ribs on the behavior of panels, three FE models were studied. The first and second model NP1 and NPH1 represents the sandwich and hollow blocks panels SW1 and SW2 that were tested and presented earlier in this study. The third model R1 represents a panel with ribs only without the two wythes. The load deflection behaviors of these three models along with experimental results of hollow block panel SW2 are plotted in Fig. 7. The load vs. deflection profiles as obtained experimentally for panel (SW2) and using FE for sandwich, hollow block panels and a panel with ribs only at different load stages, at mid-span of the panel are plotted in Fig. 7. At the initial stage of loading in hollow block panel, it is seen that the results correlate well till the yield load. At this stage, the FE deflection value was found to be higher by around 13.5% than that obtained experimentally. This difference may be due to casting quality and human reading errors.

Table(2):Parameter and material properties

Concrete material data used in ANSYS model Solid65				Steel material data used in ANSYS model Link8
Constant no.	meaning	symbol	value	

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1	elastic modulus	EX	18750(MPa)	elastic modulus EX	200000(MPa)
2	Poisson’s ratio	PRXY	0.3		
3	Open shear transfer coefficient B_t	ShrCf-Op	0.2		
4	Closed shear transfer coefficient B_t	ShrCf-Cl	0.8	Poisson’s ratio PRXY	0.3
5	ultimate uniaxial tensile cracking stress	UnTensSt	2.5(MPa)		
6	Uniaxial crushing stress (positive)	UnCompSt	25.0(MPa)	Yield stress	360 (MPa)
7	Biaxial crushing stress (positive)	BiCompSt	Default (0)		
8	Ambient hydrostatic stress state for use with constants 8 and 10	HydroPrs	Default (0)	Tang modulus	0
9	Biaxial crushing stress (positive) under the ambient hydrostatic stress state (constant 8)	BiCompSt	Default (0)		
10	Stiffness multiplier for cracked tensile condition, used if KEYOPT(7)	TenCrFac	0.6		

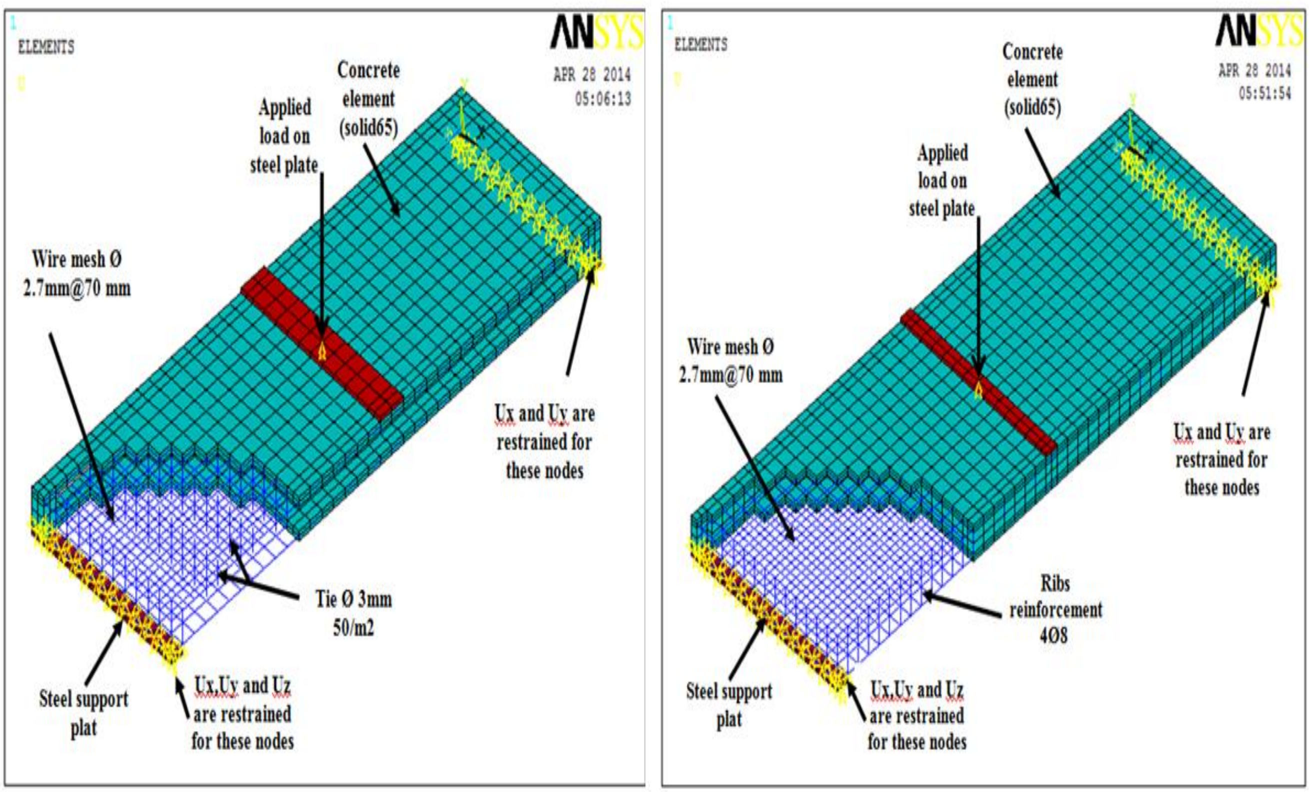

Fig. 6: FEM mesh, boundary conditions and applied bending load for a) sandwich panels and b) hollow block panels.

From Fig.7 it can be seen that, the ultimate loads for the panels NP1, R1, NPH1 are 8.7, 17.88 and 41.77 kN respectively which assure the important of presence of ribs for enhancing the behavior of the hollow block panel. Fig. 8 shows the deformed shape and measured deflection at failure for both sandwich and hollow block panels. The result illustrates that the deflection in sandwich panel has increased four times more that occurred in hollow block Panel. This improvement in the deformed shape and deflection value attributed to the presence of ribs.

Fig. 9 shows the deformed shape and measured deflection at failure for ribbed panel. It reflects that the ribs can carry considerable amount of strains. Fig. 10 illustrates mish and ties reinforcement stresses at failure of sandwich panel. From this figure it can be seen that, ties subjected to compression becomes inclined.

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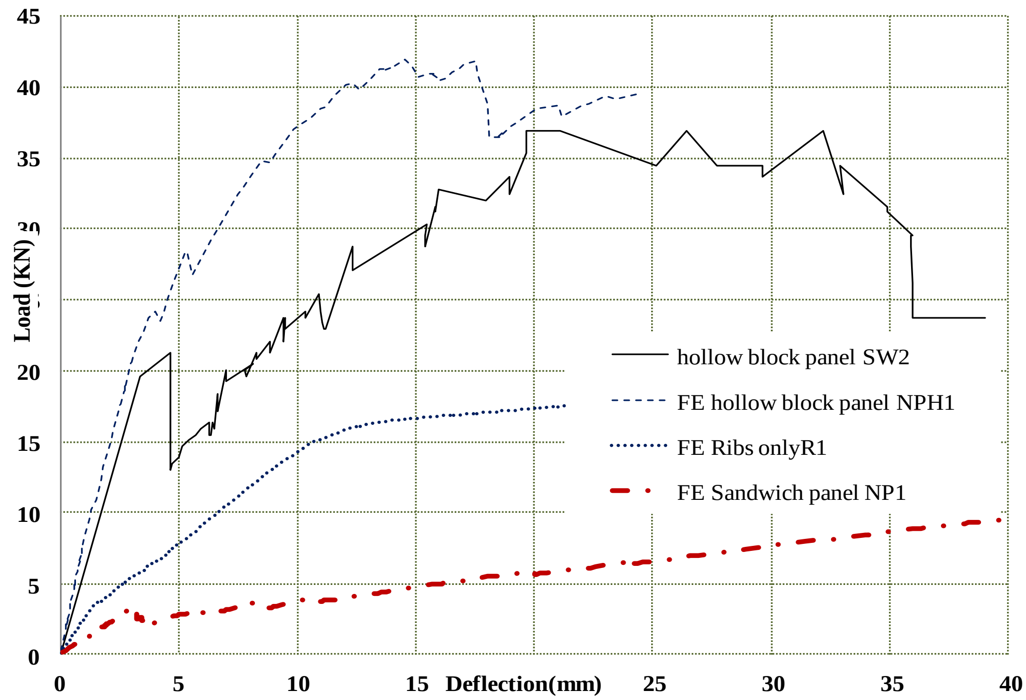

Fig 7: comparison of numerical and experimental results for sandwich, hollow blocks and ribbed

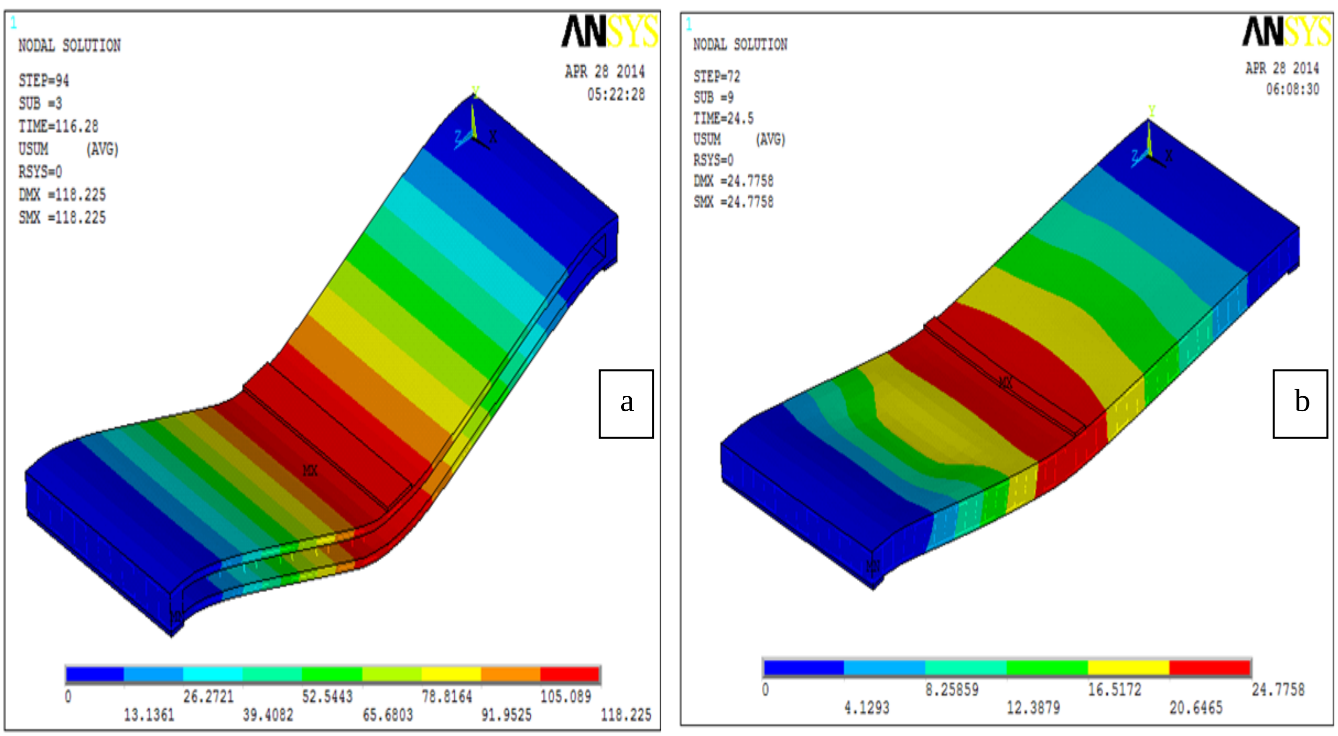

Fig. 8: Deformed shape and measured deflection at failure for both sandwich (a) and hollow block panel(b)

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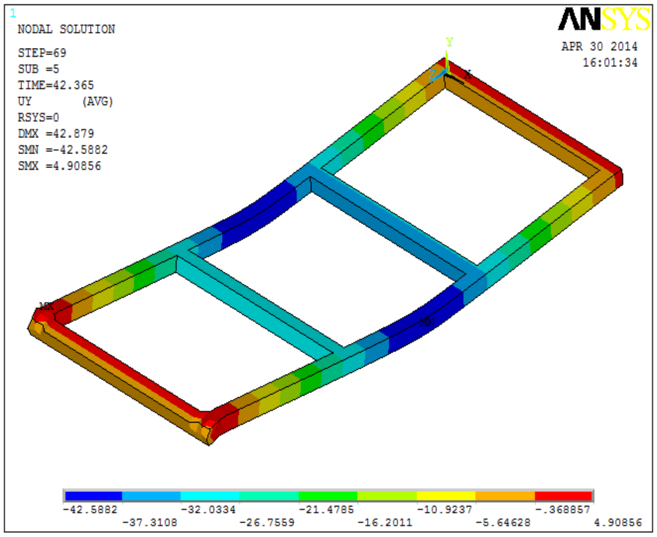

Fig. 9: Deformed shape and measured deflection at failure for ribbed panel R1.

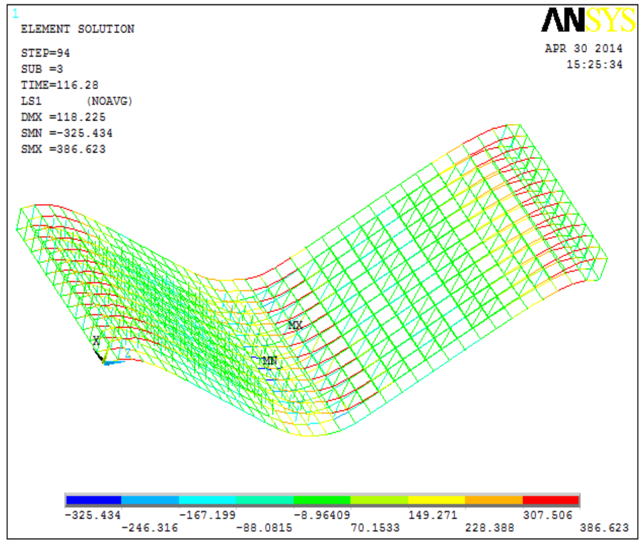

Fig. 10: Mish and ties reinforcement stresses at failure of sandwich panel.

E. Effect of Wythes Thickness on the Ultimate Load Capacity

Numerical models have been performed to study the effect of wythe thicknesses on the ultimate load capacities of both sandwich and hollow block panels. Table 3, shows dimensions, reinforcement ratio and material properties used for these numerical models.

It should be noted that reinforcement ratio is kept constant (0.35%) with increase wythes thickness as shown in Table 3. Fig. 11 illustrates the ultimate load versus wythe thicknesses with constant overall panels thickness obtained from numerical analysis of sandwich and hollow blocks panels. In this figure the effect of wythe thicknesses on the load carrying capacity are plotted. The wythe thicknesses varied from 30 mm till 70 mm while the overall panels thickness are constant and equal to 140 mm.

At wythe thickness equal to 70 mm the panel is solid without insulation layer. From this figure, in the hollow blocks panels, it could be seen that the ultimate load varies from 41.77 kN to 71.43 kN with increasing wythe thickness from 30 mm to 70 mm for each wythe. While for sandwich panels the ultimate load increases from 8.6 kN for wythes thickness equal to 30 mm to 29.03 kN for wythes thickness equal to 60mm. By increasing the thickness of the two layers of the hollow block panel to reach a solid panel, the weight increase 60% and the corresponding load carrying capacity increased by 70% only. The rate of increase in the maximum load began to recede at wythe thickness reach to 50 mm. From this result, it can be concluded that the optimum design is the thickness of the slab is equal to 50 mm.

Table 3: Details of numerical models for studying the wythes thickness on load carrying capacity.

Panel Types	Panel No.	Spec. Cross-sectional Dimension					f_{cu}(Mpa)	f_t(Mpa)	reinf. Fy (Mpa)	Wythe reinf. Ratio	Ult. load (kN)		
		Span (cm)	Width (cm)	Ribs (cm)	wythe thick. (mm)	Insulation thick.(mm)					Sand-wich panels	Hollow block panels	Ribbed panel
Hollow block	NPH2	270	100	10x14	30	80	25	2.5	360	0. 35%	-	39.44	
	NPH3				40	60					-	45.81	
	NPH4				50	40					-	56.9	
	NPH5				60	20					-	60.6	

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solid	NPH6			Solid slab 140 mm thick.							-	71.43	
Sandwich	NP1			-	30	80					41.77	-	
	NP2			-	40	60					53.42	-	
	NP3			-	50	40					63.02	-	
	NP4			-	60	20					66.94	-	
Ribbed	R1			10x8	-	-					-	-	17.88

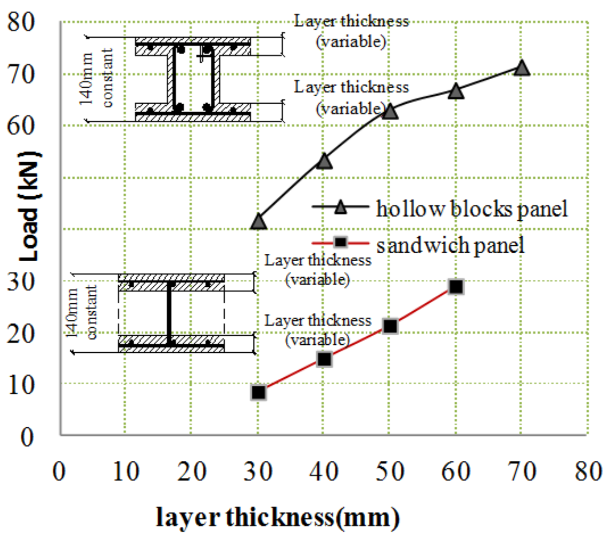

Fig. 11: Ultimate load versus Wythe thicknesses for both hollow block and sandwich panels.

F- Effect of Span to Depth Ratio on the Ultimate Load Capacity

Numerical models have been performed to study the effect of span to depth ratio on the ultimate load capacities of both sandwich and hollow block panels.

Table 4, shows dimensions, reinforcement ratios and material properties used for these numerical models. In these models, shear connector and rib reinforcement diameters increased with increasing the panels' thicknesses.

For a panel of 120 mm thickness, shear connectors are chosen to be of 3mm, however for panels of 200 mm thickness the shear connector diameter 6 mm. The number of ties is 55 per square meter of the sandwich panels from NP5 to NP9. Also, in hollow block panels, ribs reinforcement ratio ρ is kept constant at 0.35 % for all panel thicknesses. Fig. 12 illustrates the ultimate load versus span-to-depth ratio obtained from the numerical analysis of sandwich and hollow blocks panels.. From this Fig.12. It could be seen that: no significant increase in flexural strength of sandwich panels (NP5 to NP9). This is due to that the greater length of shear connector inside the insulating layer of weak density helped the occurrence of buckling to these connectors. On the contrary, the increasing of panel thickness in the hollow block panels followed by increased significantly flexural strength due to increase the stiffness of concrete ribs. These observation leads to distinct conclusion that, the ribs play significant role in flexural strength of the panels.

Table 4: Details of numerical models for studying span to depth ratio on panel ultimate load.

Panel Types	panel No.	Panel Cross-sectional Dimension (cm)					Wire mesh diameter (mm)	Shear connector diameter (mm)	reinf. f_y (Mpa)	Concrete f_{cu}(Mpa)	f_t(Mpa)	Ribs long. Reinf. Diam.	Span/depth ratio	Ultimate load (kN)	
		span	width	Panel Thick	Each wythe thick. mm	Insulation thick. mm								Sand-wich panels	Hollow block panels
Hollow block	NPH7	270	100	12	30	60	2.7	-	360	25	2.5	6	22.5	-	38.05
	NPH8			14		80		-				8	19.29	-	41.77
	NPH9			16		100		-				10	16.88	-	45.05
	NPH10			18		120		-				12	15	-	51.01
	NPH11			20		140		-				14	13.5	-	55.6
Sandw ich	NP5			12	30	60		3					22.5	7.87	-
	NP6			14		80		3.5					19.29	8.6	-

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	NP7			16		100		4					16.88	9.2	-
	NP8			18		120		5					15	9.6	-
	NP9			20		140		6					13.5	10.07	-

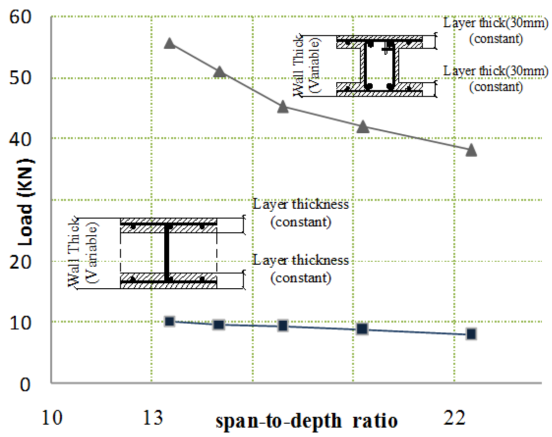

Fig. 12: Ultimate load versus Span-to-depth ratio for layer thickness 30mm.

IV. CONCLUSIONS

This paper mainly examine the effect of the presence of ribs that connects the two concrete wythes for enhancing structural behavior of the wall panels that exposed to bending loads. Experimental work and 3D numerical analysis of sandwich panels as well as hollow block bearing walls panels subjected to flexural load is conducted. Parametric studies are carried out in order to focus on span to depth ratio and wythe thickness that influence the flexural capacities of wall panels. Based on results of this study indicated that, the hollow block bearing walls subjected to flexural loads offers numerous advantages over sandwich panels with tie connectors which concluded on the following points:

- The presence of ribs enhances the behavior and failure mode of the specimens, delays the formation of cracks,
- Cracking pattern for both sandwich and hollow block panels has been shown that, the presence of ribs helped hollow block panel to withstand higher loads, while their absence leads to collapse of the sandwich panel prematurely.
- The connectors those connect the parallel concrete withes and pass through the insulating layer with a weak density helps in the incidence of buckling to those connectors if exposed to compressive stresses. Whilst, presence of concrete ribs provides confinement action of stirrups and make strapping action between the two concrete layers which highly increase the stiffness of the panel.
- Slenderness ratio has significant effect on the deflection profiles of hollow block bearing walls more than sandwich panels.
- With decreasing the slenderness ratio in the hollow block panels, flexural strength significantly increased due to increase the stiffness of concrete ribs and the complete interaction between ribs with concrete wythes

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First Author Al-Tuhami AbuZeid Al-Tuhami, PhD, P.Eng., is an associate professor at the Structural Engineering Department in the faculty of engineering at Zagzaig University in Zagzaig, Egypt, Director of Innovation and Applied Research Committee in Egyptian Engineering Syndicate.
AREA OF INTEREST
Innovative techniques in retrofitting the reinforced concrete structures, mechanical couplers for reinforcing bars and new building systems
PUBLICATIONS: Over thirty papers in Journals and Conferences
PATENTS:
Inventor Name: Al-Tuhami AbuZeid Al-Tuhami
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Second Author: Ahmed Ismail Gabr, MSC student at the Structural Engineering Department in the faculty of engineering at Zagzaig University in Zagzaig, Egypt,