COMPARATIVE STUDY OF THE NORMAL AND HIGH STRENGTH POZZOLANIC CONCRETE ON THE STRENGTH AND DURABILITY AT ELEVATED TEMPERATURE

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ABSTRACT

Cement based materials play an important role in the design and performance of future deep radioactive waste disposals and used as a solidification matrix and / or as backfill and construction material for determine the aqueous chemistry of low – and intermediate level radioactive waste for very long periods of time. The strength and durability performance of normal- and high-strength pozzolanic cement concretes incorporating silica fume, fly ash, and blast furnace slag was compared at elevated temperatures up to 800°C. The strength was studied and determined by using different scenarios, including the case, an unstressed residual compressive strength test, while durability cement was investigated by rapid chloride diffusion test, and crack pattern observations. It was found that pozzolanic cement concretes containing fly ash and blast furnace slag give the best performance particularly at temperatures below 600°C as compared to ordinary Portland cement concretes. Explosive spalling occurred in most high-strength concretes (HSCs) containing silica fume. A distributed network of fine cracks was observed in all fly ash and blast furnace slag concretes, but no spalling or splitting occurred. The high-strength pozzolanic concretes showed a severe loss in permeability-related durability than the compressive strength loss. Thirty percent replacement of cement by fly ash in HSC and 40 wt % replacement of cement by blast furnace slag in normal strength concrete (NSC) was found to be optimal to retain maximum strength and durability after high temperatures

Key words: Concrete, pozzolanic materials, high strength concrete, durability

INTRODUCTION

Introductory Remarks

The decay of encapsulated radioactive nuclides may cause elevation of the temperature in the waste forms .Cement has been used for the immobilization of low and intermediate level radioactive wastes. Compared to the materials which are used to immobilization of radioactive wastes, cement is not a very expensive raw material. Heat resistance materials are usually used for structural purposes. The unstable component of concrete is the Portland cement pastes. Hydrated Portland cement pastes contain a large amount of Ca (OH)2, which dehydrated to CaO between 500 °C and 600°C . On cooling and exposure to moist or wetting, the CaO is rehydrated to Ca(OH)2 with a volume expansion of 97% and formation of crack [1]. For shielding from gamma and X-rays the density of the concrete is the most important parameter and increasing the density leads to reduce thickness of concrete required. Thermal stresses are generated either by external sources of heat as, for example, in the case of shields for nuclear reactors, or from heat arising from the attenuation or absorption of gamma and neutron radiation within the shield.

Pointed out that in most shielding structures stresses due to thermal effects or thermal loads could be of greater importance than stresses due to mechanical or non-thermal loads on the structure [2]. Pozzolanic concretes are extensively used throughout the world; the oil, gas, nuclear, and power industries are among the major users. The applications of such concretes are increasing day by day due to their superior structural performance, environmental

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friendliness, and energy-conserving implications. Apart from the usual risk of fire, these concretes are exposed to high temperatures and pressures for considerable periods of time in the above mentioned industries.

Although concrete is generally believed to be an excellent fireproofing material, many recent studies have shown extensive damage or even catastrophic failure at high temperatures, particularly in high strength concrete (HSC). During the last decade, there has been extensive research on the fire performance of normal-strength concrete (NSC) and HSC. However, most studies emphasized on types of aggregate, addition of fibers, heating rate and maximum temperature level, methods of testing, etc. There is very little research work available on the comparative performance of different pozzolanic concretes at elevated temperatures. Moreover, these studies considered only the residual strength and no attention was given on the durability loss, which can reduce the remaining service life of the structure very rapidly. In this research, a comprehensive experimental work has been done to consider the effect of pozzolan type and content on the fire performance of NSCs and HSCs at macro- and micro levels. The following sections present a brief review of the previous research conducted on the fire performance of pozzolanic concretes [3,4].

Fire performance of silica fume concrete

An earlier research [5] indicated that silica fume concrete is highly prone to spalling and cracking at elevated temperatures. A special 170-MPa concrete containing 15 – 20wt % silica fumes was prepared. Five of the fifteen 100X200-mm cylinders exploded at 650°C. The results indicated that on average, the residual compressive strength of silica fume concrete increased with temperature up to 350°C and then decreased sharply. In a later study, [6] another series of specimens with granite aggregate and silica fume contents of 0%, 5wt %, 10wt %, and 15wt % of cement, and moisture content in equilibrium with air conditions were prepared. None of these specimens exploded when heated at rates of 1°C/min and 5°C/min to 600°C. It was concluded that concretes densified by means of silica fume at high moisture contents are more likely to explode and suggested an upper limit of 10wt % silica fume to avoid spalling. The same conclusions were also verified [7] after conducting a fire test on a monolithic beam-slab (T-beam) specimen containing 8% silica fume. It was found that replacement of ordinary Portland cement (OPC) by 10% silica fume did not offer any additional benefits at elevated temperatures. The residual strength of silica fume concretes appeared to be worse than the concretes containing 100% OPC, [8]. Five-series were prepared of specimens containing 0–5% wt silica fume with normal-weight crushed gravel or lightweight aggregates and were subjected to a maximum temperature of 600°C at a heating rate of 2°C/min. Concrete without silica fume showed slightly better performance in terms of lower strength loss and reduced spalling, [9]. Concretes with 6.7% and 9.7% silica fume, flint aggregates, and Type V Portland cement, also obtained the similar results. In short, it can be concluded that the addition of silica fume highly densifies the pore structure of concrete, which results in explosive spalling due to the buildup of pore pressure by water vapors. Since the evaporation of physically absorbed water starts at 80°C, which induces thermal cracks, such concretes show inferior performance as compared to pure OPC concretes at elevated temperatures.

Fire performance of fly ash concrete

The research on performance of pulverized fly ash (PFA) concretes at elevated temperatures dates back to 1969; however, due to limited applications of PFA concrete before the last decade, only few studies were carried out. A series of research on 25-MPa mass concrete containing 25 wt % lignite fly ash were conducted [10]. The specimens were exposed to high temperatures of 21–232°C for periods of over 6 months. They found an increase in strength of concrete in the temperature range of 121–149°C of about 152% of the original strength. However, the strength of PFA concrete was reduced to about 27% of the original value at 232ºC. It was suggested that the increase in strength for fly ash concrete is due to the formation of tobermorite (a product of lime and PFA at high pressure and temperature), which was reported to be two to three times stronger than the CSH gel. A similar increase in strength was reported [11] between 200°C and 250°C in a 90-MPa concrete incorporating Class F fly ash. However, the residual strength was below the original value at all temperatures for OPC–PFA paste containing 30 wt % PFA by weight under a series of temperatures up to 650°C. The relative residual compressive strength was 88wt % at 450°C and 73% at 600°C, which was almost double than the residual strength of pure OPC pastes. Other research to investigate the effects of high temperature and pressure on the strength and elasticity of HSC incorporating silica fume and high-calcium lignite fly ash together was studied. The fly ash content was 20wt% or 60wt%of binder, while silica fume was studied with a constant dosage of 10wt % of the total binder. The exposure temperature was varied from 21°C to 232°C, while pressure was varied from 5.2 to 13.8 MPa. A gradual deterioration of strength and static modulus of elasticity was observed with a rise in temperature at all pressures. The 20% PFA replacement showed more loss; however, in any case, the residual compressive strength was more than 60% of the original strength. In a recent research, the effects of PFA replacement level, water/binder ratio (W/B), and curing conditions on the residual properties of concrete at elevated temperatures were studied. An increase in strength was observed at 250°C. All PFA concrete specimens showed better performance till 650°C than pure OPC concrete; however, after that, there was no significant difference in the residual strength of all specimens. It was found that a high dosage of PFA enhanced the residual properties of concrete at elevated temperatures. The results were also verified by porosity analysis done by mercury intrusion porosimetry (MIP) technique. Conclusively, it was found that the PFA improved the performance of concrete at elevated temperatures as compared to silica fume or pure OPC concretes. However, this improvement was more significant at temperatures below 600°C. Moreover, it was discovered that the PFA also reduces the surface cracking of concrete both at elevated temperatures and after post cooling in air or water [13].

Fire performance of blast furnace slag concrete

An earlier work on the performance of slag cement at elevated temperatures has been done by Grainger[14]. He tested four cement pastes containing 0%, 50wt %, 70wt %, and 90wt % replacement of slag by weight with OPC. The maximum temperature was 500°C with an interval of 100°C. All slag–cement paste specimens experienced an increase in strength between 100°C and 250°C. The 70wt % slag replacement showed the best results with a residual compressive strength of 190wt % of the original strength at 110°C. Moreover, the residual strength of this paste was higher at all temperatures as compared to the original strength. The other two slag–cement paste specimens also showed better residual strengths as compared to the pure cement paste specimen. Three HSCs mixes incorporating condensed silica fume (CSF), PFA, and granulated blast furnace slag (GGBS) were independently prepared. Then subjected to a maximum temperature of 900°C. The GGBS concrete showed the best performance followed by PFA and CSF concretes. Cement paste and concrete specimens incorporating 65wt % slag by weight of cement and firebrick aggregates [9]. The results were compared with pure OPC cement paste/concrete and 30% PFA cement paste. The maximum temperature was 700°C, while the residual properties were measured at every 100°C interval. They found that the slag–cement paste and concrete gave the best results among all the specimens. The residual compressive strengths of slag concrete were 102% and 80% of the initial cold strength at 450°C and 600°C.The properties of silica fume and blast furnace slag concretes prepared with two ‘‘thermally stable’’ aggregates (viz., Lytag and crushed firebrick)were studied. The results indicated that concretes with the lightweight aggregate had lower residual strength at temperature above 150°C than the corresponding concrete prepared with firebrick aggregate. The use of firebrick with blast furnace slag in concrete also gave superior elevated temperature performance [15] .

RESEARCH OBJECTIVES

The preceding review reveals that most of the studies used only one or maximum two pozzolans at different dosages in concrete to evaluate its fire performance. The types of aggregate, curing and testing conditions, heating rate, and strengths were different in some studies, which make it difficult to generalize the results for a given condition. Moreover, these analyses were limited to residual mechanical properties only, and no durability studies were conducted. To compare the strength and durability performance of different pozzolanic concretes at elevated temperatures, it is necessary to prepare and test them under the same set of material and environmental conditions. In this study, an attempt is made to achieve this objective by preparing nine HSC and five NSC mixes incorporating CSF, PFA, and ground GGBS, then subjected to heating levels of 200°C, 400°C, 600°C, and 800°C. In addition to

GGBS, then subjected to heating levels of 200°C, 400°C, 600°C, and 800°C. In addition to measuring residual compressive strength, durability analysis was also carried out by using leaching procedure of Cs from mixtures.

EXPERIMENTAL DETAILS

Materials

A locally manufactured OPC complying with ASTM Type I, a low-calcium fly ash equivalent to ASTM Class F, CSF, and GGBS were utilized as binders. All the binders are commercially available in Egypt. The chemical composition and physical properties of these materials are shown in Table 1.

Aggregates

The fine and coarse aggregates were local natural river sand and crushed granite with maximum nominal sizes of 10 and 20 mm, which were mixed in a ratio of 1:2, sieve size and percentage are shown in Table 2.

Table 1 : Chemical analysis of GGBS used in the study

Property	PC	SF	FA	GGBS
Oxide composition (%)				
SiO2	19.61	90.26	56.79	38.14
AL2O3	7.33	0.63	28.21	6.53
Fe2O3	3.32	0.33	5.31	0.40
CaO	63.15	3.18	< 3.00	35.77
MgO	2.54	0.33	5.21	13.65
SO3	2.13	0.4	0.68	-
NaO2	-	-	-	0.4
TiO2	-	1.0	-	-
C	-	3.0	-	-
LOI	2.97	4.84	3.90	0.76
Mineral Composition				
C3S	61.7			
C2S	11.4			
C3A	7.6			
C4AF	6.8			
Physical properties				
Specific gravity	3.16	2.22	2.31	2.53
Specific surface	360	22500	450	470
(m2/kg)	1225	250	1000	560

Table 2 : Grading of fine and coarse aggregates

Coarse aggregate		Fine aggregate	
Sieve size {mm}	**Percentage retained**	**Sieve size**	**Percentage retained**
10	1.4	5mm	10.7
5	98.5	2.36mm	28.3
2.36	99.6	1.18mm	36.2
		600µm	45.1
		300µm	84.6
		150µm	99.3

Superplasticizer
A sulphonated, naphthalene–formaldehyde condensate was used as a superplasticizer (SP) in HSC mixes. This SP was a dark brown liquid containing 38.6% solids.

Mix proportions
The mix proportions of nine HSC and five NSC mixes are shown in Tables 3 and 4. All pozzolans were introduced as cement replacement materials. One control mix incorporating pure OPC was also prepared for each type of concrete for comparison

Table 3 : Mix proportion of HSC mixtures (in Kg/m^3)

Mix	SF%	FA%	GGBS%	W/B	Water	Cement	Fine agg.	Coarse agg.	SPa
HS-CCb	-	-	-	0.30	150	500	758	927	0.5
HS-SF5	5	-	-	0.30	150	475	710	1066	0.6
HS-F10	10	-	-	0.30	150	450	620	1151	0.8
HS-A20	-	20	-	0.30	150	400	618	1147	0.8
HS-A30	-	30	-	0.30	150	350	615	1143	0.7
HS-A40	-	40	-	0.30	150	300	613	1139	0.7
HS-SF+FA	10	20	-	0.30	150	350	615	1142	0.8
HS-GGBS30	-	-	30	0.30	150	350	616	1145	0.7
HS-GGBS40	-	-	40	0.30	150	300	615	1142	0.7

Table 4 : Mix proportion of NSC mixtures (in Kg/m^3)

Mix	SF%	FA%	GGBS%	W/B	Water	Cement	Fine agg.	Coarse agg.	SPa
HS-CCb	-	-	-	0.50	195	390	768	917	-
HS-SF5	-	30	-	0.50	195	273	626	1133	-
HS-SF10	-	40	-	0.50	195	234	625	1129	-
HS-FA20	-	-	30	0.50	195	273	626	1135	-
HS-FA30	-	-	40	0.50	195	234	625	1132	-

Curing conditions
The specimens were demolded after 24 hr of casting and placed in a water tank at 24°C. After 28 days of water curing, they were transferred to an environmental chamber maintained at ≈25°C and 65% relative humidity, which are the average climatic conditions in Egypt. The specimens were kept there for 2 month until heating.

Heating and cooling regimes
At an age of 60 days, the specimens were heated in an electric furnace up to 200°C, 400°C, 600°C, and 800°C. Each temperature was maintained for 1 h to achieve the thermal steady state [16]. The heating rate was set at 2.5°C/min. The specimens were allowed to cool naturally to room temperature.

Specimen dimensions and testing details
Unstressed residual compressive strength test was performed on 100mm concrete cubes .Three specimens were tested at each stage and average values are reported.

Leaching test

Leaching test was done according to the ANSI / ANS-16.1- 2203. The leachant was dematerialized water with conductivity < 5µ Ohm / cm in which the specimens , with a volume of leachant to external geometric surface area of the specimen (196.36 m2) ratio of 10 cm, was immersed in individual plastic containers . The leachant analyses were carried out on quadruplicate specimens. The cement in each cylinder was thoroughly mixed for 3 min to ensure the Cs sufficiently distributed throughout the cement. The cylinders were then sealed and the cement was allowed to cure for 28 days at 25°C. Moist curing was not used to more closely simulate the curing process of waste that would occur in the field. Upon completion of the curing process, the cement specimens were demolded and leached in accordance to the specifications and procedures outlined in ANSI/ANS 16.1. This standard was specifically designed to measure the release of radionuclides from solidified low-level radioactive waste forms. It permits the accumulation of leaching data in a relatively short period of time using test specimens of simple shapes and finite dimensions. The procedure involves sampling and replacing leachants at designated time intervals of 2 h, 7 h, 1, 2, 3, 4, 5, 19, 47 and 90 days. It also requires a rinse of the test specimen prior to the leaching regimen to determine the total activity of the Cs at the beginning of the first leaching interval. This value was designated as A0 per ANSI/ANS 16.1. [17]

RESULTS AND DISCUSSION

Residual compressive strength

There are three test methods available for finding the residual compressive strength of concrete at elevated temperatures: stressed test, unstressed test, and unstressed residual strength test. The first two types of test are suitable for accessing the strength of concrete during high temperatures, while the later is excellent for finding the residual strength after high temperature treatment. It was found that the last method gives the lowest strength and is therefore more suitable for treatment getting the limiting values and hence selected for this research The residual compressive strength are shown in Figs. (1–4) . The results indicate that each temperature range showed a distinct pattern of strength loss or gain. From 25°C to 200°C, the PFA and GGBS concretes showed increases in compressive strength with a greater increase in PFA concretes. The maximum strength gain was shown by the HSC containing 40% PFA, which was 122% of the original strength. A slight strength loss was observed in OPC and CSF concretes. The strength gain was probably due to the formation of tobermorite, which was formed by the reaction between unhydrated PFA or GGBS particles and lime at high temperatures. However, this increase in strength was more pronounced in HSC as compared to NSC. The possible reason is the greater percentage of unhydrated PFA or GGBS particles in HSC due to its dense structure. No visible cracking or spalling was observed in this temperature range. From 200°C to 400°C, most HSCs maintained their original strength, while a significant decrease was observed in NSCs, which was 19–26% of the original strength. This reduction is due to the pore structure coarsening, which was found to be more in NSC [18]. Again, the pozzolanic concretes performed better and showed higher residual strength. Hairline cracks were observed in CSF concretes, while no spalling occurred in any specimen. A severe loss in strength was observed in the 400–600°C temperature range. The average loss was 44% in HSC and 60% in NSC. The CSF concretes experienced extensive cracking and spalling and their residual compressive strength was less than the pure OPC concrete. This was probably due to the very dense structure of CSF concrete, which results in a buildup of vapor pressure formed by the evaporation of physical and chemically bound water. The PFA and GGBS concrete performed better and showed no spalling or cracking except hairline cracks. The maximum strength was retained by the HSC containing 30% PFA replacement, which was 67% of the original value. The better performance of PFA and GGBS concretes at this temperature range is due to the reduced amount of Ca (OH)2, which otherwise results in strength loss and disintegration. At 800°C, all the concretes showed severe deterioration due to the decomposition of CSH gel [19]. The average residual strength was 26% for HSC and 17% for NSC. In each temperature range, HSC maintained a higher percentage value of residual compressive strength than NSC. The reason is the coarsening of the pore structure and increase in pore diameter, which was found to be more in NSC at elevated temperatures than in the HSC [19]. Due to this effect, NSC showed a gradual decrease in strength, while sharp decrease was observed in HSC between 400°C and 800°C. This topic will be further discussed in the following sections. The beneficial effect of pozzolans was more pronounced at temperatures below 600°C for both types of concrete. In HSC, PFA concretes gave the best performance followed by GGBS, OPC, and CSF concretes, while in NSC, this sequence was GGBS, PFA, and OPC. The optimum replacement levels were 30% PFA for HSC and 40% GGBS for NSC. Spalling was observed in 28% of CSF concrete specimens and 5% of OPC concrete specimens, while PFA and GGBS concretes only showed visible network of fine surface cracks.

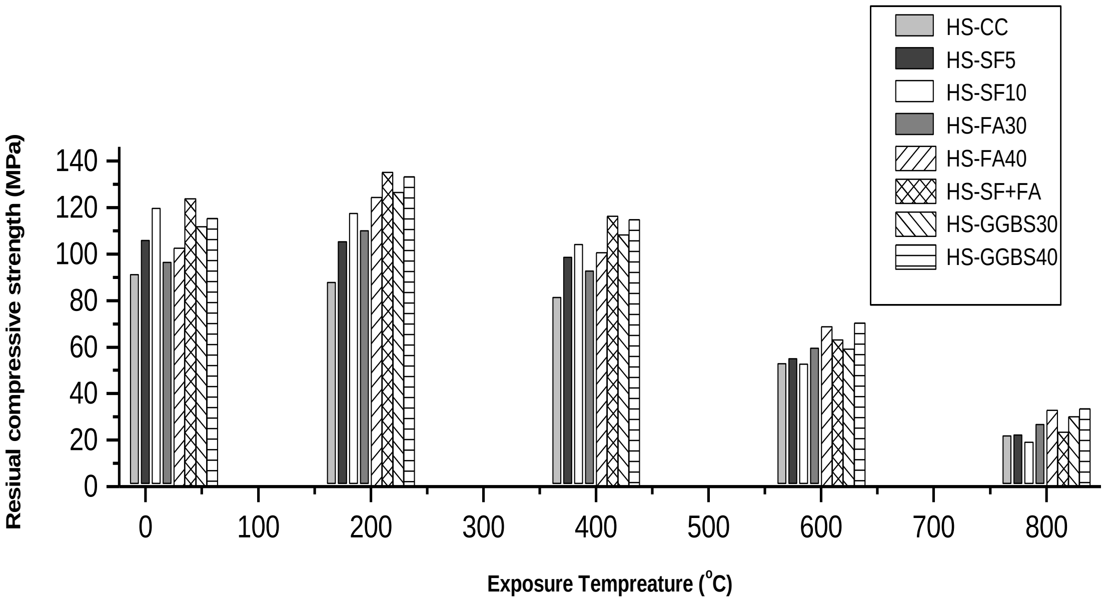

Fig 1 : Residual compressive strength of HSCs

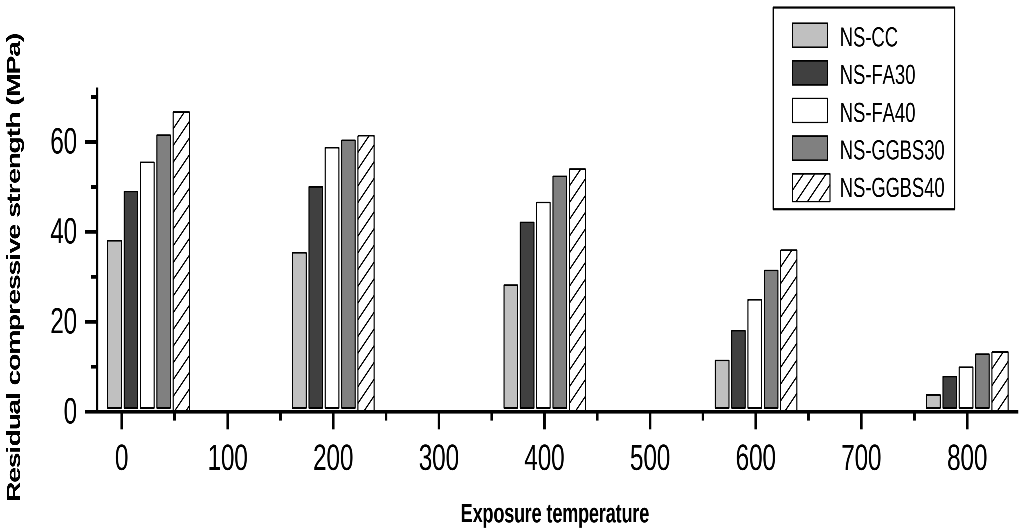

Fig 2 : Relative residual compressive strength of NSCs

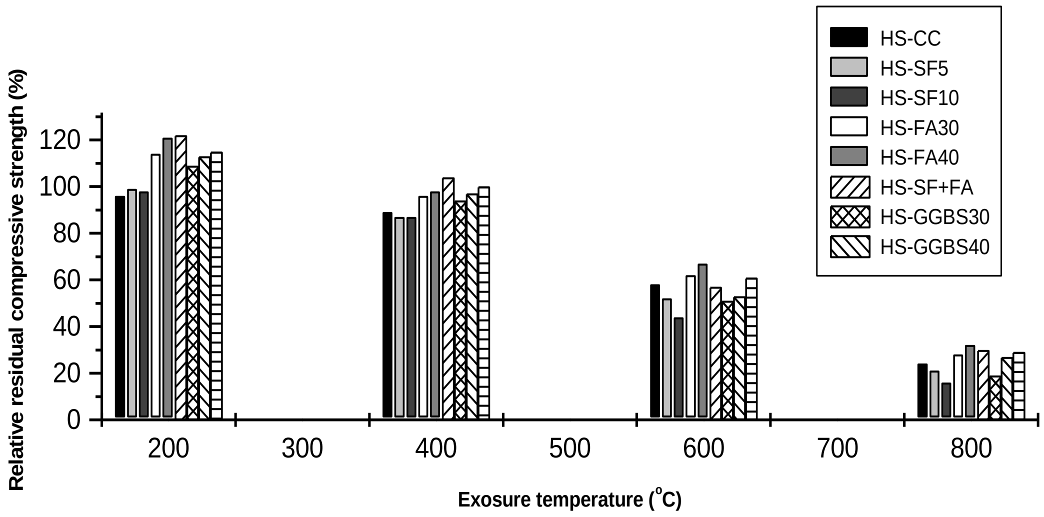

Fig 3 : Relative residual compressive strength of HSCs

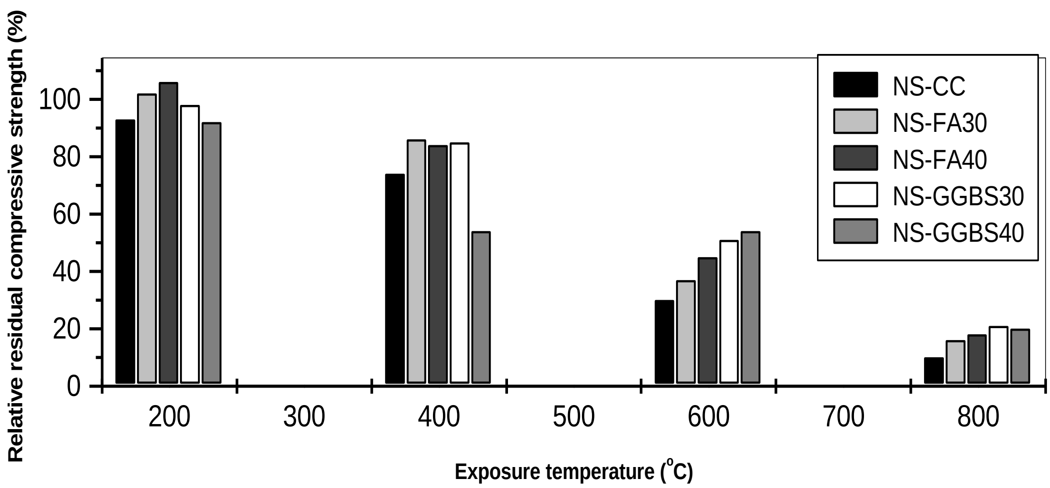

Fig 4 : Relative residual compressive strength of NSCs

leachability

Leaching in nuclear wastes is generally studied as a diffusion-controlled mass transport process .It has been found that micro-fractures, rather than pores in the paste, control the permeability of concrete. In addition to cracking caused by thermal and mechanical loads, cracking in concrete may be generated by shrinkage stresses due to drying or carbonation. Shrinkage cracks are commonly observed and can be minimized by reinforcing with short randomly distributed fibers, by addition of shrinkage-reducing admixtures, or by maintaining a low water content .Crack width can be further reduced by adequate fiber reinforcement, even under severe mechanical loading. Water penetration in concrete has been shown to be strongly dependent on the crack width of the medium consequently, permeability of cementitious materials can be reduced very significantly by minimizing the crack width or inhibiting crack formation altogether. Non-radioactive CsC1 was used in this study. Cs was selected due to its known high leachability from unmodified cement. All ingredients were mixed in a small mixer, then cast into cylindrical moulds of diameter 3.8 cm (1.5 inch) and height 3.8 cm (1.5 inch). The specimens were demolded after 24 h curing at room temperature and 90% relative humidity. Subsequently, the specimens were placed in 11 polyethylene containers filled with 500ml deionized water. The water volume to specimen surface area ratio is 10. At fixed time intervals (every 2 days), the concentration of Cs in the leachant was measured by using atomic absorption spectroscopy.

After sampling for analysis, the leachant was totally replaced with fresh deionized water. The test was performed at 25 °C from 1 day up to 90 days. Three specimens were prepared for each material, with the average value reported here. The leach rate was calculated as follows volume to specimen surface area ratio is 10. At fixed time intervals (every 2 days), the concentration of Cs in the leachant was measured by using atomic absorption spectroscopy. After sampling for analysis, the leachant was totally replaced with fresh deionized water. Three specimens were prepared for each material, with the average value reported here. Cumulative Fractional Release (CFR) [18]

Cumulative quantity leached (cm) = $\sum^{i} \frac{A_i}{A_o} \frac{v}{s}$

Where
A_i = weight of element in leachant
A_o =weight of element in original solid sample
v = volume of solid sample
s = surface area of solid sample
The quantity of Cs the (HSC) at worsening case at 800 °C are plotted in Fig 5. The total amount of Cs leached at 35 days is greatest in HSCs, followed by the HS-FA20, HS-SF10 and HS-SF+FA are shown in fig 5. The reductions in total quantity of Cs leached from HSCs, followed by the HS-FA20 , HS-SF10 and HS-SF+FA, are34% 46%, 69% and 71%, respectively

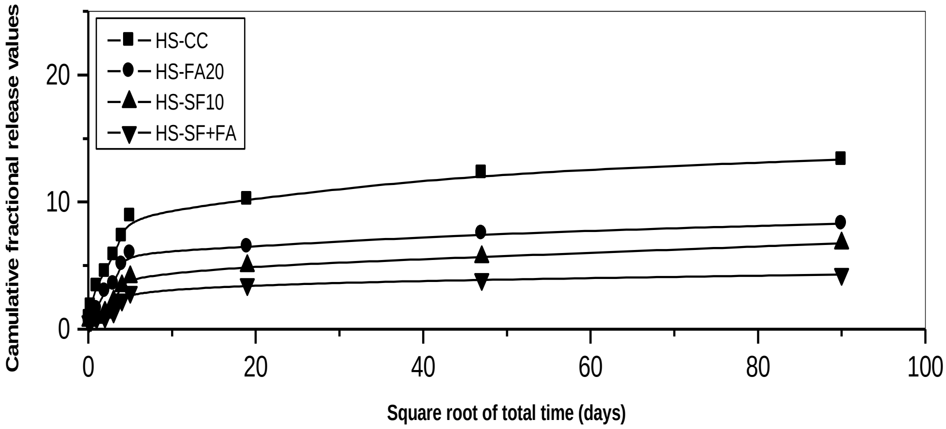

Fig 5 : Cumulative fraction leached of Cs ion versus the square root of time of the HSCs under worst case (800°C)

Integrative analysis of compressive strength and durability
To compare the performance of different pozzolanic concretes interactively, two worsening indices, d1 and d2, as defined are used. The index d1 is the worsening index of mechanical strength and d2 is the worsening index of the permeability-related durability. These indices are defined below in Eqs. (1) and (2): [19]

$$d_1 = 1 - \frac{\text{mean residual compressive strength}}{\text{mean original compressive strength}} \qquad (1)$$

$$d_2 = 1 - \frac{\text{cumulative volume of pores larger than } 1.3\ \mu\text{m before fire}}{\text{cumulative volume of pores larger than } 1.3\ \mu\text{m after fire}} \qquad (2)$$

The indices can have values ranging from 0 (no worsening) to 1 (full worsening). The calculated values of d1 and d2 are given in Table 10. The results clearly support the findings pointed in the earlier sections as:

1. As compared to NSC, HSC suffered a smaller worsening of its mechanical strength but a greater worsening of the permeability-related durability.
2. The CSF concrete showed greater deterioration of strength and impermeability than all other concretes due to its very dense microstructure.
3. In HSC, 30% PFA replacement, while in NSC, 40% GGBS replacement gives the best strength and durability performance at elevated temperatures.

These results also show a severe loss of durability as compared to the compressive strength, and indicate that even after the fire, if concrete retains a high proportion of its original compressive strength, an examination of durability should be made. This is because the loss in durability can result in deterioration of concrete and rusting of rebars, which may reduce the overall life of the structure.

Table 5 : Calculated values of worsening indices d_1 and d_2

Mix	Exposure Temperature			
	600°C		800°C	
	d_1	d_2	d_1	d_2
HSCs				
HS-CCb	0.419	0.620	0.760	0.829
HS-SF5	0.480	0.660	0.790	0.887
HS-SF10	0.560	0.681	0.840	0.913
HS-FA20	0.381	0.605	0.720	0.748
HS-FA30	0.330	0.543	0.680	0.753
HS-FA40	0.430	0.586	0.700	0.764
HS-SF+FA	0.490	0.716	0.810	0.947
HS-GGBS30	0.470	0.595	0.730	0.792
HS-GGBS40	0.390	0.587	0.709	0.787
NSCs				
HS-CCb	0.699	0.558	0.901	0.731
HS-SF5	0.629	0.500	0.839	0.666
HS-SF10	0.550	0.524	0.820	0.693
HS-FA20	0.480	0.541	0.791	0.651
HS-FA30	0.460	0.520	0.799	0.633

CONCLUSIONS

1. The pozzolanic concretes showed better performance at elevated temperatures than the pure OPC concretes except the mixes containing 10% CSF. This better performance was due to the reaction of these pozzolans with free lime, which enhances the strength and durability both at normal and high temperatures by reducing the free lime content
2. High temperatures can be divided into distinct ranges in terms of effect on concrete strength. In the range of 20–200°C, an increase in strength was observed in PFA and GGBS concretes. At 400°C, most HSCs maintained their original strength, while an average loss of 20% strength was observed in NSCs. After 400°C, both types of concrete lost their strength rapidly and the rate of strength loss was more in HSC.
3. In HSC, the PFA concretes showed the best performance at elevated temperatures followed by GGBS, OPC, and CSF concretes. The mix containing 30% PFA replacement gave the maximum relative residual strength. In NSC, the GGBS concretes gave the best performance followed by PFA and OPC concretes. The 40% replacement level was found to be the optimum.
4. The mechanical strength of HSC decreased in a similar manner to that of NSC when subjected to high temperatures up to 800°C. However, HSC maintained a greater proportion of its relative residual compressive strength than the NSC.

5. The HSC suffered a marginally smaller loss of mechanical strength but a greater worsening of the permeability-related durability than the NSC. Among HSCs, the PFA concretes suffered the least damage in impermeability followed by GGBS, OPC, and CSF concretes. In NSC, the sequence was GGBS, PFA, and OPC concretes.
6. The surface crack pattern gave a good indication about the internal pore structure of the concrete as shown by major cracks in CSF concretes and fine distributed cracks in PFA or GGBS concretes. More surface cracking was observed in HSC than NSC.
7. Severe deterioration and spalling was observed in most CSF concretes and some HS-OPC concretes. Most of the spalling occurred between 400°C and 600°C. No spalling was observed in PFA or GGBS concretes. Conclusively, the PFA and GGBS concretes were found to be able to retain their properties better at elevated temperatures and can be used in those places where there is a high risk of fire. The CFS concrete with more than 5% replacement should be avoided at such places due to the high risk of explosive palling.

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EFFECT OF ELEVATED TEMPERATURE ON THE BEHAVIOR OF RC BEAMS STRENGTHENED WITH FRP

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ABSTRACT

Fiber reinforced polymer, (FRP), have been used over the last decade in the form of laminates for strengthening reinforced concrete, (RC), structures. The non-corrosive characteristics and good fatigue properties of FRP significantly increase the service life of structures. Its high strength and low weight makes it feasible compared to conventional materials used for retrofit. Few test data is available on FRP resistance exposed to long term elevated temperature. This research is being carried out to study the behavior of reinforced concrete beams strengthened with FRP under different environmental conditions such as temperature degree, relative humidity, and thermal durations. Also, presence of protection layers on FRP laminates was studied. Different FRP types using carbon fiber reinforced polymer (CFRP) and glass fiber reinforced polymer (GFRP) laminates are used. The main experimental parameters include different temperature degrees (40, 50, and 60 °C), relative humidity 40 % and thermal durations (3, 6 and 9 months). The measured deflections, failure modes, ultimate carrying capacities, and strain of steel reinforcement and concrete and FRP laminates at critical sections of the beams exposed to different independent environmental conditions are reported. To determine the design strength of CFRP, and GFRP strengthened beams exposed to long-term environmental conditions as well as to evaluate the strength reduction factors associated with various independent conditions, 15 beams strengthened by CFRP and GFRP are tested. Results of the experiments show that the failure modes of all specimens are due to yielding of the reinforcing steel in tension followed by rupture of the FRP laminate. The results also show that beams strengthened with GFRP are more susceptible to aggressive environmental conditions than beams strengthened with CFRP.

Keywords: Beam, durability, FRP, humidity, temperature.

INTRODUCTION

Durability is the ability of the material and structure to resist cracking, oxidation, chemical degradation, delaminating, wearing or the damage effect of the strange object for specific period of time, under the appropriate load conditions and under specified environmental conditions[1]. Considering the aggressive civil infrastructure environment, the deterioration mechanisms of FRP composites can be different from the conventional building materials such as steel and concrete. The degradation mechanisms of FRP composites are related to the failure of different phases i. e. the fibers (fibers dominated mechanism), the matrix (matrix dominated mechanism) and the inter phase (inter phase dominated mechanism) or their combinations[2]. Laboratory investigations have proved the efficiency and durability of CFRP laminates for such applications and these results reported the following conclusions: Commonly used CFRP strengthening systems tend to be more sensitive to elevated temperatures and humidity than steel or concrete[3]. Tg is frequently used in defining the limiting thermal conditions for synthetic components. However, this parameter is limited relevance as the mechanical properties of the epoxy adhesive actually start to change at a lower temperature[4]. Another investigation studied

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the durability of reinforced concrete (RC) beams externally strengthened with (CFRP) plates and fabrics under adverse environmental conditions. It is concluded that the beams strengthened with CFRP plates and exposed to 10,000 hours of 100% humidity (at 38 ± 2 °C) experienced an average of 33% reduction in their strength[5]. Many investigations reported the effect of FRP strengthening on the ductility of concrete sections [5]. Although the short-term characteristics are known, it is difficult to predict the long-term behavior of combined systems and define its limiting conditions In addition, many studies clarifies that the high percentage of relative humidity; RH, environment had a bad significant effect on the bond strength between FRP sheet and the concrete interface. Therefore, protection precautions should be considered to control that effect and failure type [6].Several researchers investigate the effect of protection layers such as perlite applied on FRP laminates to protect it from aggressive environmental conditions and they reported its efficiency [7] .Ductility of RC beams strengthened by FRP laminates and exposed to aggressive environmental conditions is an important properties of the efficiency of the system, so many investigations reported the effect of strengthening by FRP laminates on the ductility of the section [8] .The individual components of a strengthened structure (concrete body. steel bars. FRP laminates, adhesive) react differently to fluctuations in temperature and humidity. Although these characteristics are known, it is difficult to predict the behavior of combined systems and define the limiting conditions. To investigate these issues, a long-term experiment was carried out in a climate chamber. Also investigate the temperature effect on FRP strengthened RC beams.

RESEARCH PROGRAM

Experimental program

The presented work is a part of more comprehensive investigation, carried out by authors, that covers the effect of different environmental conditions such as different temperature degrees and different relative humidity percentages on the behavior of RC beams strengthened with FRP. The experimental program was carried out to study behavior of reinforced concrete beams strengthened with FRP under different environmental conditions such as temperature degree, relative humidity and thermal duration. Also, presence of protection layers on FRP laminates is studied. Different FRP types using carbon FRP and glass FRP laminates are used. Table (1) shows the details of experimental program. The experimental work consisted of testing a total of (15) beams tested under flexural loading.

Table 1 : Experimental program

No	Temperature °C	R.H. %	Strengthening type	Protection	Thermal duration
1	Temperature °C Lab conditions (23 °C)	R.H. % Lab conditions (50%)	Baseline beam*	-------	-------
2			GFRP sheet		
3			CFRP sheet		
4	40	40	GFRP sheet	-------	3 months
5			CFRP sheet		
6	60	40	GFRP sheet	-------	
7			CFRP sheet		
8	50	40	GFRP sheet	-------	3 months
9			CFRP sheet		
10			GFRP sheet	-------	6 months
11			CFRP sheet		
12			GFRP sheet	-------	9 months
13			CFRP sheet		
14			GFRP sheet	With protection	3 months
15			CFRP sheet		

* Baseline beam is a beam without strengthening and tested under lab conditions

In this current study, the relative humidity was chosen due to the average relative humidity observed on the Upper Egypt during the year.

All beams have the same dimensions, that is, 150 mm wide, 200mm high, and 2200mm long. Two steel bars of 12 mm diameter are used as the main flexural reinforcement for each beam; two steel bars of 6mm diameter are placed for all beams as top reinforcement for the purpose of supporting stirrups. The closed-type stirrups of 8 mm-diameter bars are spaced at 142 mm along the beam length for all specimens. Bottom bars have yield strengths of = 418 N/mm^2 while both top bars and stirrups bars have yield strengths of = 280 N/mm^2. The average 28-days strengths of concrete cubes of = 28 N/mm^2. Extra cubes were cured in the same conditions as the beam specimens, and no significant change in the compressive strength was recorded.
CFRP and GFRP sheets were used in this study. The stress-strain relationship of CFRP and GFRP laminates are linearly elastic up to failure as reported by manufacturing company. The used CFRP laminates had 0.13 mm thickness.
The tensile strength and modules of elasticity of the laminates are 3500 MPa and 230 GPa, respectively as reported by the manufacturing company. The used GFRP laminates had 0.17-mm thickness. The tensile strength and modules of elasticity of the laminates are 2250 MPa and 70 GPa, respectively as reported by the manufacturing company. All irregularities found on the concrete surface of the beam were removed using a hand grinder and a masonry-grinding wheel.
The surface was sand blasted to ensure proper bonding of the FRP fabrics. The minimum compressive strength, flexural strength, tensile strength and bond strength to steel after ten days of the epoxy paste, which was applied on the beams surface, as reported by the manufacturing company are 75, 25, 10 and 10 MPa respectively.
Epoxy resin supplied by the same company, was used to bond the FRP laminates to the concrete. The flexure modulus, tensile strength and adhesive strength on concrete of the resin are 3800, 30 and 4 MPa, respectively as reported by the manufacturing company.
One layer of laminate with 100 mm width, and 1900 mm length were used for the strengthening of each beam.
The layer of the FRP fabric was bonded to the prepared surface of the concrete. Hand rollers were used to properly bond the fabric and to remove any trapped air between FRP fabric and concrete beams. Concrete dimensions and detailing of reinforcement and applied FRP fabrics are shown in Figure (1), and Figure (2).

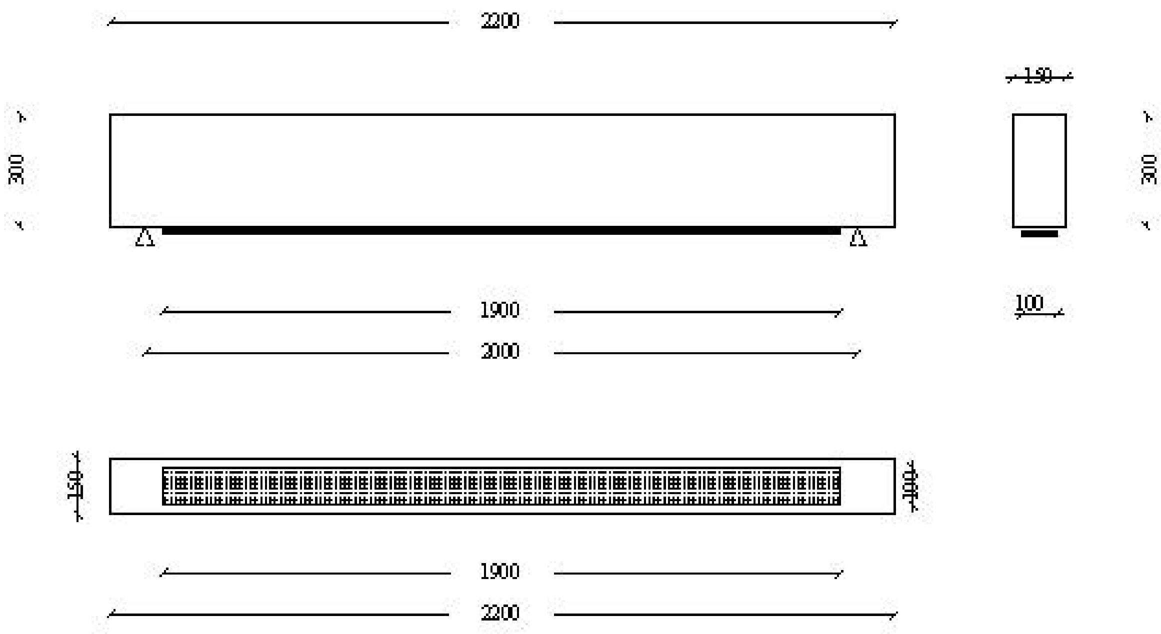

Fig. 1 : Concrete dimensions of tested beams

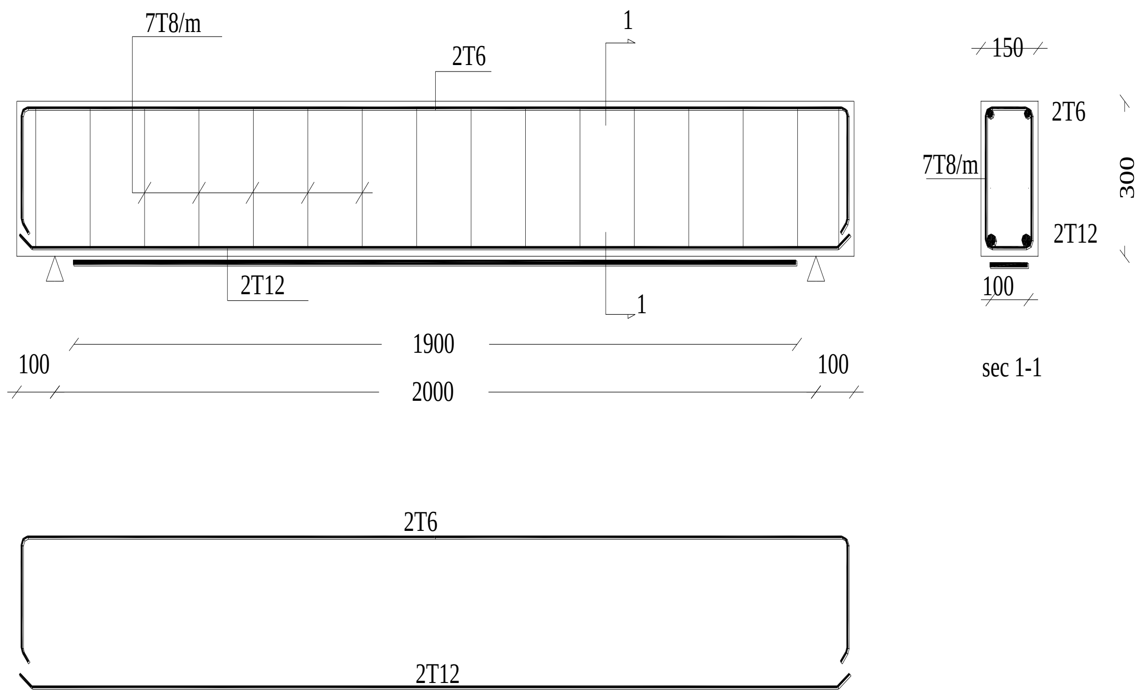

Fig. 2 : Reinforcement detail of tested beams

Environmental control

Temperature and humidity chamber was designed and constructed to control the environmental conditions such as temperature and relative humidity. The chamber consists of four separated rooms, two in the front and two in the back. Each room was designed to accommodate more than 9 beams arranged in three rows of 3 beams each. Each room was 2.5 m long, 1.25 m wide, and 2.25 m height. Each specimen was placed in the environmental chamber for the specified time and then testing in 4-point loading as discussed later. To produce the humidity inside each room, the electric boiler located above the climate chamber was used to produce steam, this boiler is connected to 4- control valves and each valve is connected to rubber pipe carrying the steam to the rooms. Each room is equipped with electric heater to control the temperature. Also each room has a control panel connected to 2 –sensors inside the room, one for controlling and monitoring the relative humidity and the other for controlling and monitoring the temperature. When the relative humidity reaches to the specified values, automatically steam valve is close. Also, when temperature degrees reach to the specified values, the electrical heater is shut off. The additional benefit of the steam is make an air circulation inside the room to distribute the temperature inside the room. Outside walls, inside partitions, floor, and roof of chamber were designed and constructed to isolate each room to keep the heat and humidity conditions inside each constant. So, chamber body consists of steel sandwich panels with isolation material of thickness not less than 50 mm in the middel sheets with isolating material with thickness not less than 50 mm between them. Figures (3, 4, 5, and 6) show the chamber details.

Test setup

A hydraulic machine of 5000 kN was used for loading the specimen with displacement control method. The machine head was attached to an electrical load cell of 2000 kN capacity. The beams were tested using two concentrated loads. The point of application of each load was offset 250 mm from the mid-span of the beam. The test setup and specimens instrumentations are given in Figure (7).

Fig. 3 : The chamber

Fig. 4 : Separated room

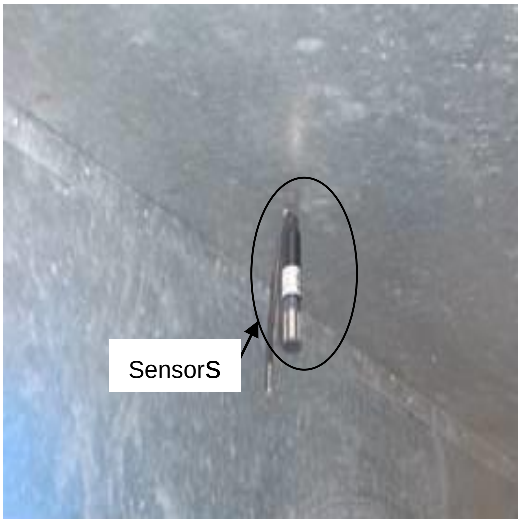

Fig. 5 : Sensors inside the room

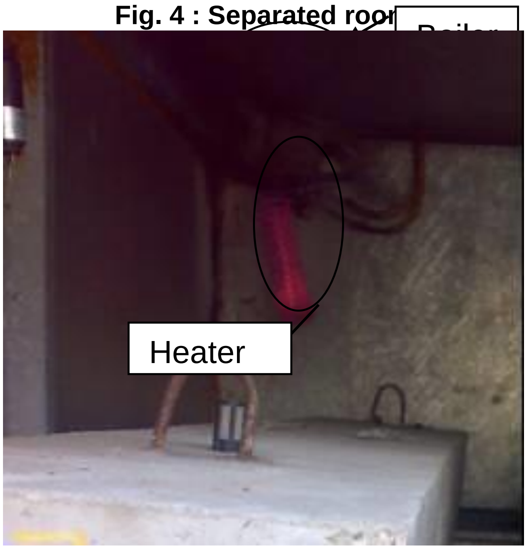

Fig. 6 : Electrical heater inside the room

INSTRUMENTATIONS

Data acquisition measurement and control systems were used to perform the tests. The specimens were instrumented to record the strain of the concrete at tensile and compression zones, strain of both compressive and tensile steel reinforcement as well as strain of FRP laminates. This is in addition to the load and deflection at mid-span of specimen. These strains were measured by electrical strain gages.. Two electrical strain gages were installed on the compressive and tensile steel reinforcement at mid-span section. Another strain gage was installed on FRP laminates to measure its strain in the fiber direction. The strains of the concrete at tensile and compressive zones were measured Using Linear Variable Displacement Transducers, (LVDT,s) with different lengths of 20 and 50 mm. Three LVDT,s were attached on the compressive and tensile concrete surfaces and mid-span sections as shown in Figure (7). One LVDT with length of 50 mm was attached to the bottom surface at the mid-span section to record the deflection of the beams.

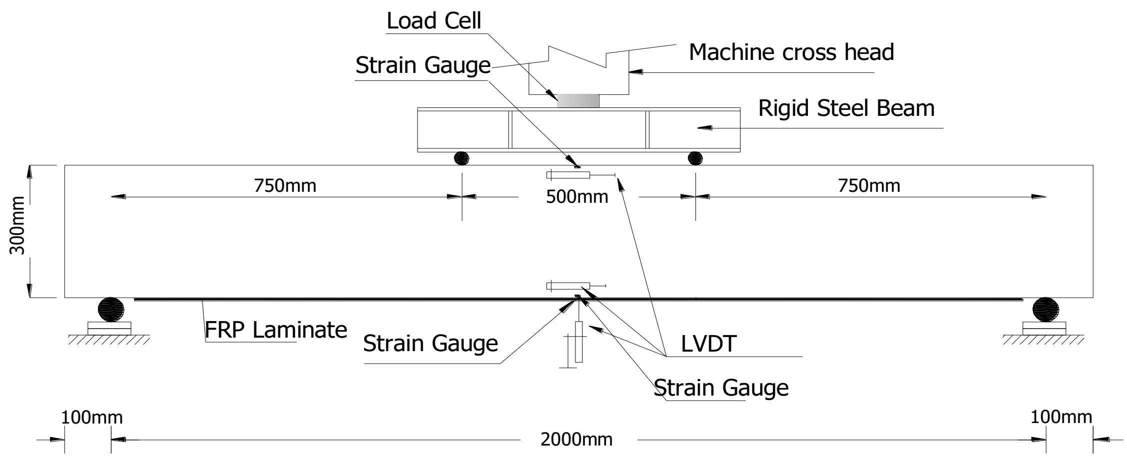

Fig. 7: Test setup and instrumentation

DISCUSSION OF TEST RESULTS

Ultimate load capacities, corresponding deflections, beam ductility, and failure modes of the beams exposed to different environmental conditions are presented in Table 2. At lab conditions, it is observed that external strengthening of RC beams using GFRP and CFRP sheets increased the strength of the beam by approximately 25 and 27%, respectively. It is also observed that the beams strengthened with GFRP sheets and exposed to different elevated temperatures for 3 month did not have any increase in the ultimate load capacity compared with the original control beam (without strengthening).

Table 2: Test results

No.	Beam Discription	Duration	Failure load (kN)	Deflection (mm)	Ductility Index (Ue)	Failure mode
1	Baseline beam	-------	88.6	25.1	16.243	*
2	GFRP control beam	-------	109.1	8.2	3.20	**
3	CFRP control beam	-------	112.3	18.2	7.40	**
4	GFRP, 40° C, 40%RH	3 months	92.6	8.95	5.21	**
5	CFRP, 40° C, 40%RH	3 months	117.8	17.6	7.56	**
6	GFRP, 50° C, 40%RH	3 months	91	6.27	3.04	***
7	CFRP, 50° C, 40%RH	3 months	102.8	13.97	5.12	**
8	GFRP, 60° C, 40%RH	3 months	89.9	5.84	3.04	***
9	CFRP, 60° C, 40%RH	3 months	100.3	12.57	5.00	**
10	GFRP, 50° C, 40%RH	6 months	89.7	6.9	3.04	***
11	CFRP, 50° C, 40%RH	6 months	95.3	14.2	4.10	**
12	GFRP, 50° C, 40%RH	9 months	89.4	6.6	3.05	***
13	CFRP, 50° C, 40%RH	9 months	92.8	10.6	3.80	**
14	GFRP, 50° C, 40%RH with protection	3 months	93	17.2	3.08	**
15	CFRP, 50° C, 40%RH with protection	3 months	107.9	18.1	6.50	**

* Yielding of tensile steel reinforcement followed by crushing of concrete at compression zone.

** Yielding of tensile steel reinforcement followed by rupture of FRP laminates with separation of concrete cover.

*** Yielding of tensile steel reinforcement followed by rupture of FRP laminates without separation of concrete cover

Unlike Figures (8, and 9) show load-deflection curves for strengthened beams with GFRP, and CFRP sheets respectively, at different temperatures (40, 50, and 60 ° C) with relative humidity 40 % for 3 months. It is shown that the load-carrying capacity of the beams strengthened with GFRP and exposed to (40, 50, and 60 ° C) with relative humidity 40 % along 3 months are significantly reduced (15, 16.6 , and 17.6 % respectively),), in comparison to that of corresponding control beam B2. On the other hand, the beams strengthened with CFRP and exposed to (50, and 60 ° C) with relative humidity 40 % for 3 months are significantly reduced (8.5, and 10.7 respectively), in comparison to that of corresponding control beam B3, while the beam exposed to 40 °C and 40% relative humidity resulted in increasing in failure load by about 4.9 % compared to control beam B3, the slightly increased failure load of this beams may be attributed to that the dry-heat conditioning temperature was close to that of the glass transition temperature Tg of saturating epoxy used for bonding the CFRP sheets with the concrete surface, which in turn led to the development of improved bond strength between fabrics and concrete surface, this result was meet with the results observed by Nabil F Grace and S. B. Singh 2005[5] .
As shown in Figures (8, and 9), the beams strengthened with GFRP, or CFRP) exposed to temperature conditions show a sudden drop of the load followed by a large deflection at a constant load before their complete collapse. This attributed to the failure of beams by onset of delamination of FRP sheets at a load close to the observed ultimate failure of beams.

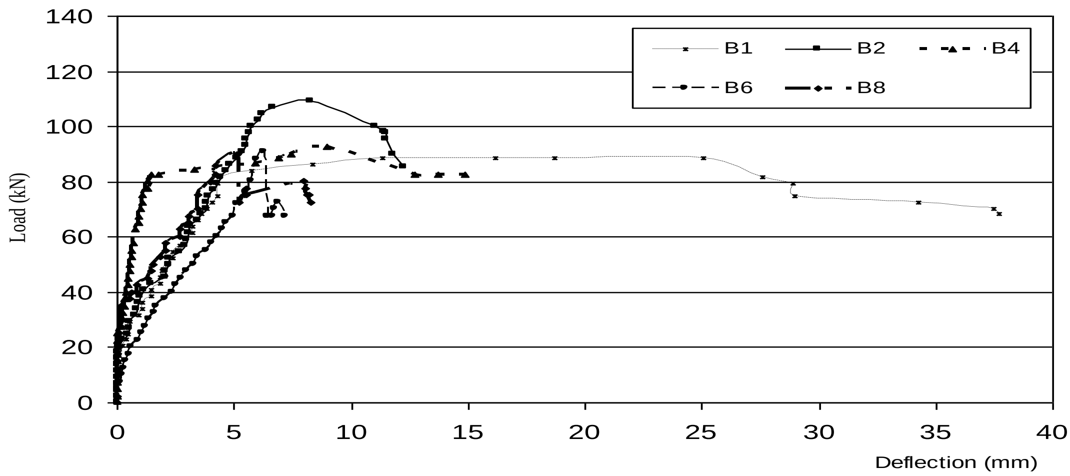

Fig. 8 : Load-deflection relationship for beams strengthened with GFRP

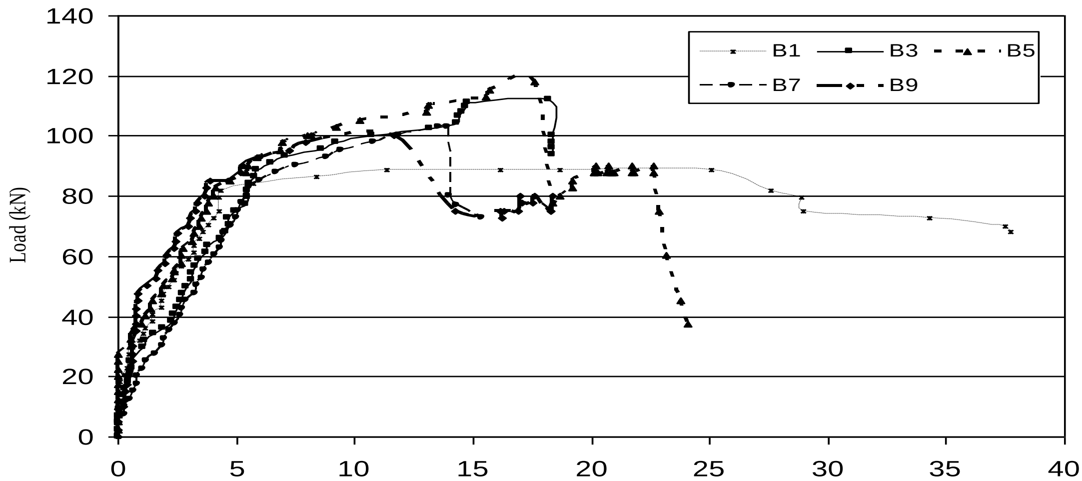

Fig. 9 : Load -deflection relationship for beams strengthened with CFRP

Figures (10, and 11) show load-deflection curves for strengthened beams with GFRP, and CFRP sheets respectively, exposed to 50 °C with relative humidity 40 % for (3, 6 , 9 months). It is shown that the load-carrying capacity of the beams strengthened with GFRP and exposed to 50 °C with relative humidity 40 % for (3, 6, and 9 months) are significantly reduced (16.6 , 17.8, and 18 % respectively), in comparison to that of corresponding control beam B2. This means that the GFRP sheets are not applicable for strengthened concrete beams when these beams are exposed to continuous elevated temperature. On the other hand, the beams strengthened with CFRP and exposed to 50°C with relative humidity 40 % during (3, 6, and 9 months) are significantly reduced (8.5, 15, and 17.4 % respectively), in comparison to that of corresponding control beam B3. It is clear that the beams strengthened by CFRP and exposed to continued elevated temperature up to 6 month miss its strengthening efficiency. As shown in Figures (10, and 11), the beams strengthened with GFRP, or CFRP) exposed to temperature conditions show a sudden drop of the load followed by a large deflection at a constant load before their complete collapse. This is attributed to the failure of beams by onset of delamination of FRP sheets at a load close to the observed ultimate failure of beams.

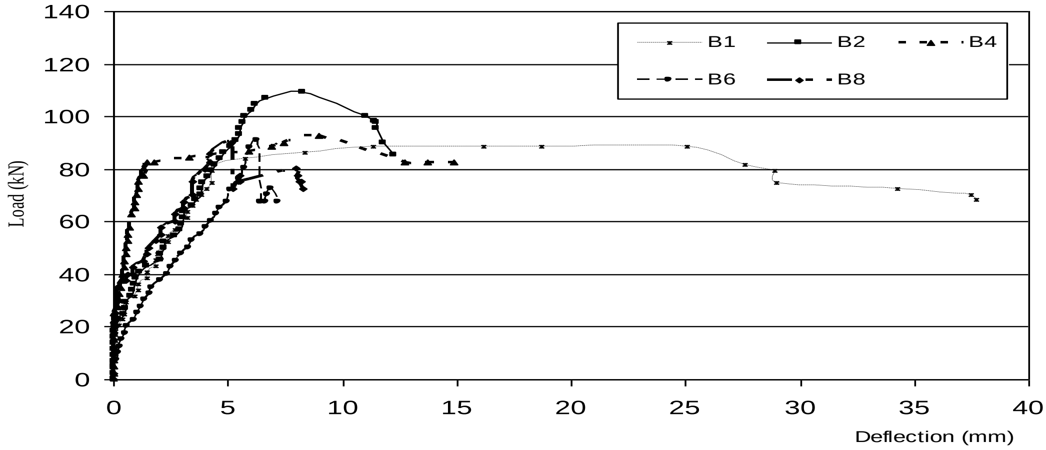

Fig. 10 : Load-deflection relationship for beams strengthened with GFRP exposed to temperature (50° c) and r.h. (40%) for (3, 6and 9 months)

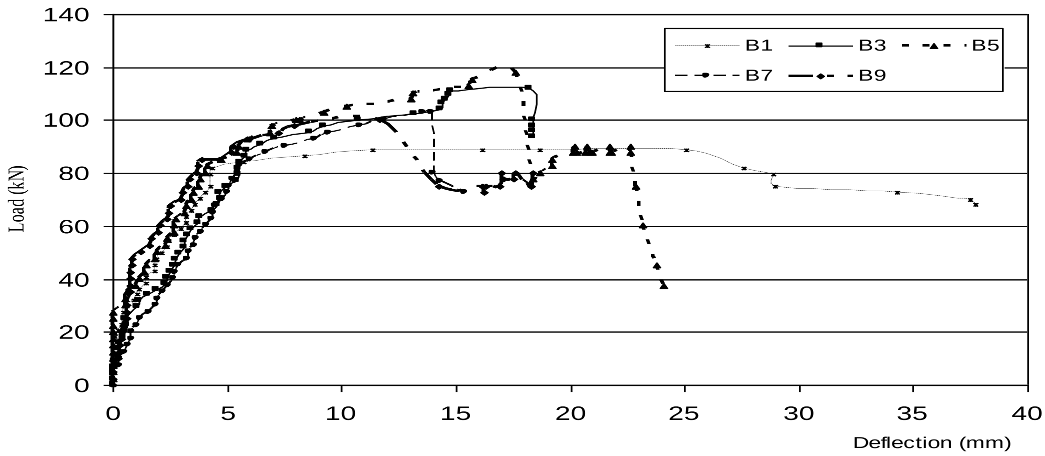

Fig. 11 : Load-deflection relationship for beams strengthened with CFRP exposed to temperature (50° c) and r.h. (40%) for (3, 6and 9 months)

Figures (12, and 13) show load-deflection curves for strengthened beams with GFRP, and CFRP sheets respectively, exposed to and 50 °C with relative humidity 40 % for 3 months with or without protection layer.

It is shown that the load-carrying capacity of the beams strengthened with GFRP and exposed to 50 °C with relative humidity 40 % for 3 months without protection layer is significantly reduced by 16.6 % in comparison to that of corresponding control beam B2, while the load-carrying capacity of B14 which was protected using layer of perlite mortar is reduced by 14.7 % when compared with corresponding control beam B2. It means that the protection has no significant effect with beams strengthened with GFRP. On the other hand, the beams strengthened with CFRP and exposed to 50 °C with relative humidity 40 % for 3 months without protection layer is significantly reduced by 8.5 % in comparison to that of corresponding control beam B3, while the load-carrying capacity of B15 which was protected using a layer of perlite mortar is reduced by 4 % when compared with corresponding control beam B3. It is meaning that the protection has a significant effect with beams strengthened with CFRP. As shown in Figures (12, and 13), the beams strengthened with GFRP, or CFRP) exposed to temperature conditions show a sudden drop of the load followed by a large deflection at a constant load before their complete collapse. This is attributed to the failure of beams by onset of delamination of FRP sheets at a load close to the observed ultimate failure of beams, and it is clear that the protection layer has no effect on the mode of failure of the strengthened beams.

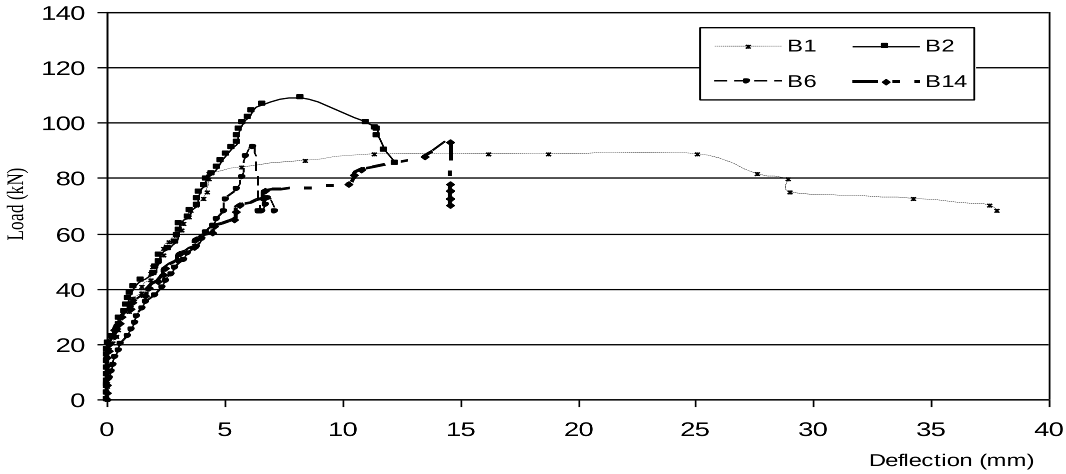

Fig. 12 : Load-deflection relationship for beams strengthened with GFRP exposed to temperature (50° c) and r.h. 40% for 3 months with or without protection layer

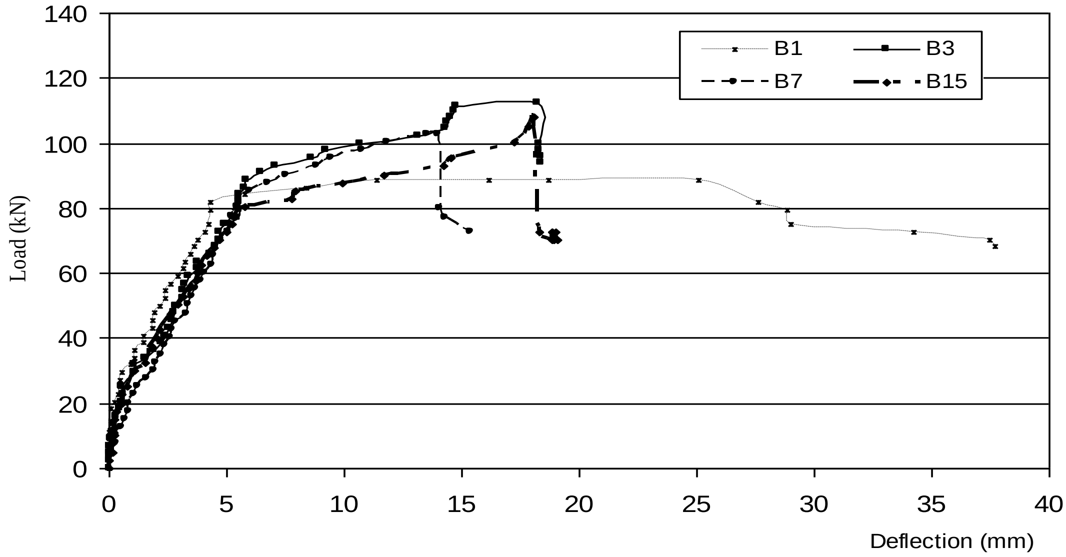

Fig. 13 : Load-deflection relationship for beams strengthened with CFRP exposed to temperature (50 ° c) and r.h. 40% for 3 months with or without protection layer

Mode of Failure

Figures (14 and 15) show the failure mode of the beam strengthened with GFRP, and CFRP respectively, after being exposed to temperatures conditions.

Fig. 14 : Failure mode of beams strengthened with GFRP

Fig. 15 : Failure mode of beams strengthened with CFRP

The onset of delamination (Fig. 14 and 15) was immediately followed by concrete crushing. It should be noted that all of the strengthened beams with or without exposure to environmental conditions and with or without protection layer failed due to the onset of delamination of the GFRP and CFRP sheets but it was noted that the failure mode of beams strengthened by CFRP and exposed to any temperature degrees up to 9 months and the beam strengthened by GFRP and exposed to 40 ° C and R.H. 40% for 3 months failed due to the separation of concrete cover (mode of failure**), while the mode of failure of beams strengthened by GFRP and exposed to 50 or 60° C and R.H. 40% did not experience separation of concrete cover (mode of failure***). Thus, onset of delamination is the actual mode of failure of FRP strengthened beams and governs the load- carrying capacity of these beams. The onset of delamination of FRP sheets is primarily dependent on the bond between the concrete and FRP sheets through structural/saturating epoxy. As shown in Figures (14, and 15), the decrease in the shearing strength of structural epoxy due to continuous exposure to temperature conditions led to significantly reduced load-carrying capacity caused due to the onset of delamination.

Ductility of the Tested Beams

The area under the load–deflection curve represents the ductility of reinforced concrete beams. This area provides information about the elastic and inelastic energy accumulated in the element under the applied load. In another expression, ductility may be expressed by the ratio of two areas under the load –deflection curve of reinforced concrete beam.

Energy ductility $U_e = E_u/E_y$

Where, E_u is area under the load-deflection diagram up to failure, where failure case is defined as the load decreased up to 80% of the maximum load, and E_y is area under the load-deflection diagram up to first yielding of tension steel (elastic energy). Based on the definition given in Equation (1), the energy ductility indices of the tested beams are shown in Table (2).

The results indicated that strengthening with externally bonded GFRP (B2) and CFRP (B3) laminates reduced the structural ductility of the beams by 80 and 54 % respectively compared with control beam without strengthening (B1). The reduction of beams ductility is due to lack of ductility of FRP laminates. FRP is characterized by linear stress-strain relationship up to failure. Also, the results indicated that the exposition to different temperatures (40, 50, and 60 °C) for different durations 3, 6, 9 months had no significant effect on the ductility of beams strengthened with externally bonded GFRP; in comparison to that of corresponding control beam B2. The increasing of temperature up to 60 °C decreased the ductility of the beams strengthened with CFRP laminates up to 32 %, on the other hand, the increasing of duration time of exposition decreased the ductility of the beams strengthened with CFRP laminates up to 49 %.

Using protection layer of perlite mortar improved the ductility of the beam strengthened by CFRP and exposed to 50 °C with relative humidity 40 % for 3 months by about 26% in comparison to that of beam without protection. On the other hand, the protection layer had no significant effect on the beam strengthened by GFRP. However, the results of the tested beams indicated sufficient ductility before failure and acceptable safety margins measured for the beams strengthened by CFRP.

CONCLUSIONS

Based on the experimental results reported in this study, the following conclusions can be made:

1. RC beams strengthened with GFRP laminates are more susceptible to aggressive environmental conditions than beams strengthened with CFRP laminates.
2. No failure due to end debonding of FRP laminates was observed. Use of sufficient development length of FRP laminates prevented end debonding of FRP.
3. The reduction values of load-carrying capacity for beams strengthened with CFRP laminates after 9 months exposure to 40% humidity, and different temperature values were smaller than that of beams strengthened with GFRP.
4. Exposure to environmental conditions up to 3 months and temperature of 40 °C had not negative effect on the load carrying capacity of strengthened beam with CFRP laminates but when the temperature is increased to 60 °C, the load carrying capacity of strengthened beam with CFRP laminates reduced by 10.7 % compared with similar beam with CFRP laminates tested under lab conditions. When the exposure time to environmental conditions reached 9 months, the load carrying capacity of strengthened beam with CFRP laminates reduced by 17.4 % when compared with similar beam tested at lab conditions, and almost equal to that of beam without strengthening and tested under lab conditions.
5. Using GFRP laminates to strengthen RC beams is not applicable when the beams are exposed to environmental conditions with temperature higher than 40 ° C and relative humidity 40 % for periods more than 3 months.
6. Using perlite mortar to protect the CFRP strengthening system improved the resistance of CFRP laminates to high temperature environmental conditions. On the other hand this protection system had no significant effect on the GFRP strengthening system.
7. The results indicated that strengthening with externally bonded GFRP and CFRP laminates reduced the structural ductility of the beams significantly. More reduction in the beam ductility was observed when exposed to elevated temperature.

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EFFECTS OF MARBLE RUBBLE IN INTERLOCKING PAVING UNITS

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ABSTRACT

This study examines the effect of marble rubble, in three forms; powder form, as aggregates ((3-8mm) and (20mm)) on the properties of manufactured interlocking paving units: Experiments were carried out to determine product compressive strength, water absorption, and abrasion resistance according to both Egyptian Standard Specifications (ESS) and American Society for Testing and Materials (ASTM) at ages 28 days and 6 months.

In addition, water absorption and indirect tensile splitting tests were carried out according to European Standard (EN) at ages 28 days and 6 months. Results proved that marble powder enhanced compressive strength and improved abrasion resistance significantly when compared to the control mix.

Keywords: interlocking paving units, marble rubble, compressive strength, water absorption, abrasion resistance, indirect tensile splitting test.

INTRODUCTION

Interlocking Paving Units have been recently included in the Egyptian Standard Specifications. The main constituent materials of Paving Units are cement, sand, crushed stone, and water. This study, however, investigates the effects of adding marble rubble to the constituent materials.

Grupta, et al, conducted experiments to check the effect of utilization of marble powder in concrete and the role of water content. Changes in properties of concrete like workability, segregation index and strength with the replacement of fly ash with marble powder were studied. In this study it was found that the value of Segregation Index increases with an increase in the amount of marble powder as a replacement of fly ash. [1]

Binicil; et al studied some mechanical properties of concrete containing marble dusts (MD) and limestone dusts (LD). Seven concrete mixtures were produced in three series with control mixes having 400Kg cement content.

The control mixes were modified to 5, 10, 15% MD and LD in place of fine sand aggregate. Also, abrasion resistance was investigated. Results indicated that MD and LD fine aggregate gave good workability and abrasion resistance was comparable to that of conventional concrete. Abrasion resistance was increased as the rate of fine MD and LD was increased. [2]

A powder obtained as a by-product of marble sawed and shaped was used as a mineral addition to mortars and concretes, especially for self-compacting concrete.

This marble powder showed a very high Blaine fineness value of about 1.5m^2/g with 90% of particles passing 50µm-sieve and 50% under 7µm.

Mixtures were evaluated based upon cement to sand substitution by the marble powder. Results obtained show that 10% substitution of sand by the marble powder provided maximum comp. strengths at about the same workability. [3]

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MATERIALS AND METHODS

cement

The cement used was ordinary Portland cement in accordance to ESS 4756-1/2007.

Fine aggregates (sand, and marble rubble)

Siliceous sand was used in this research program. The sieve analysis is shown in figure 1. Marble rubble of size 3-8mm was used as a replacement to sand and figure 2 shows its sieve analysis. Table 1 gives the physical properties of fine aggregates.

Table 1: Fine aggregates physical properties

Property	Sand	Marble rubble (3-8 mm)
Specific weight	2.56	2.63
Volumetric weight (tons/m^3)	1.52	1.6

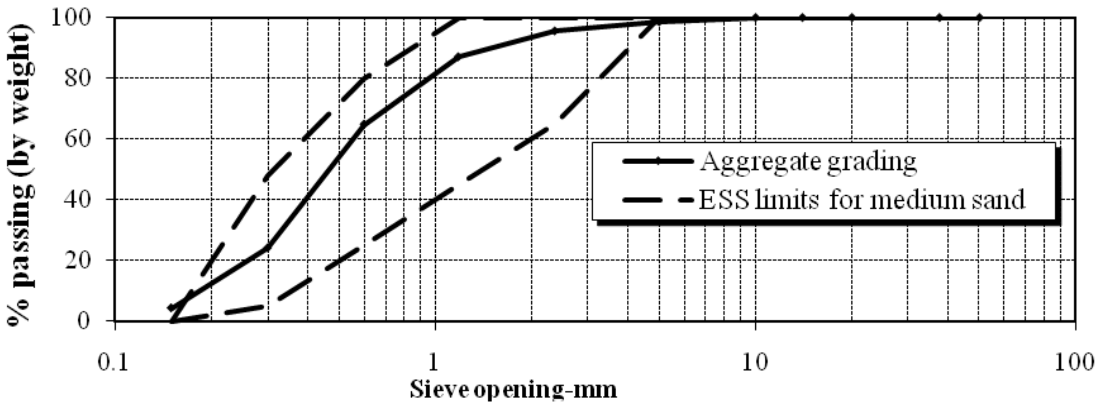

Fig. 1: Sieve analysis of sand

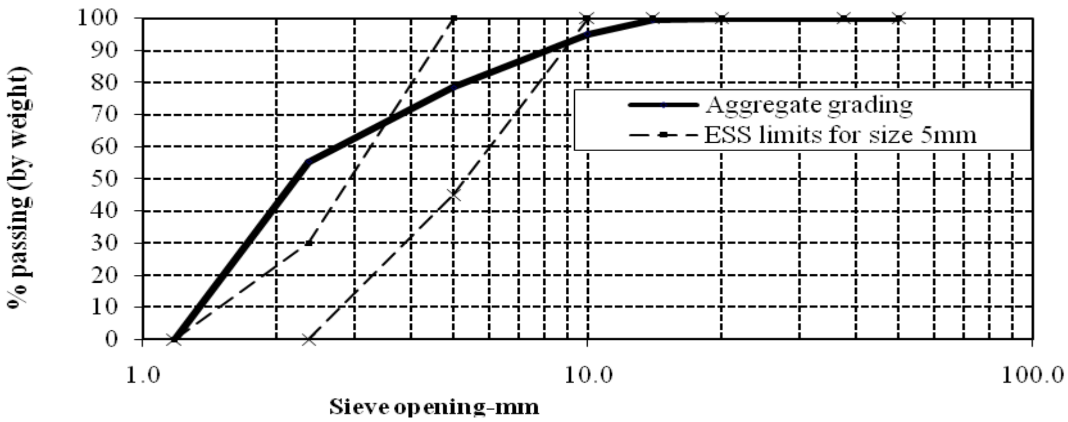

Fig. 2: Sieve analysis of marble rubble (size 3-8 mm)

Coarse aggregates (crushed stone, and marble rubble)

Crushed limestone was used in this research for control samples. Marble rubble of size 20mm were used as recycled aggregate.

Figures 3 and 4 show the sieve analysis of both types of aggregates, respectively, as supplied. Table 2 shows the physical properties of both types of coarse aggregates. The chemical analysis of marble is given in table 3

Table 2: Coarse aggregate physical properties

Property	Crushed stone	Marble rubble	Acceptance limits
Specific gravity	2.76	2.56	–
Volumetric weight (tons/m^3)	1.62	1.46	–
Absorption Percentage	0.55%	2.95%	Not more than 2.5% (1)
Clay and other fine materials (%)	0.2%	0.58%	Not more than 3% by weight (2)
Impact value (%)	25%	18.8%	Not more than 30% (2)

(1) According to the Egyptian Code of Practice issued 2001
(2) According to the Egyptian Standard Specifications 1109/2002

Table 3: Chemical analysis of marble

SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	SO_3	TiO_2	P_2O_5	L.O.I.	Total
1.31	0.19	0.19	54.18	0.21	0.04	0.09	1.36	0.02	0.12	41.99	99.7

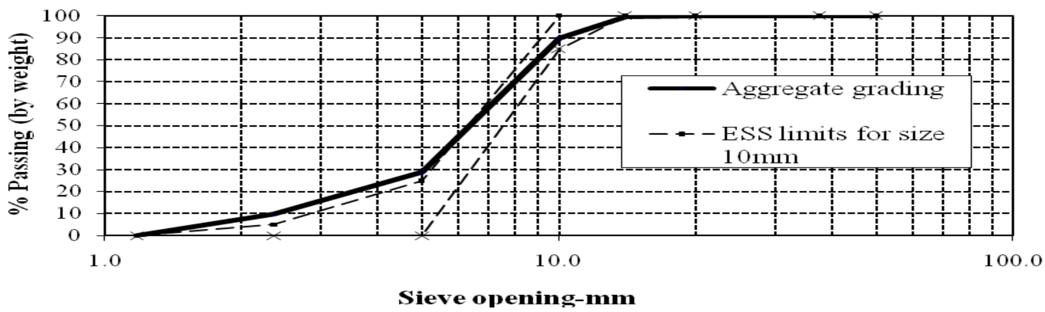

Fig. 3: Sieve analysis of crushed limestone

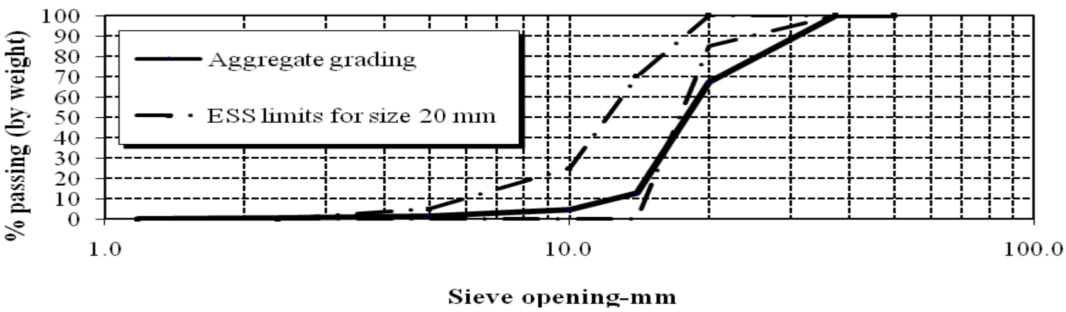

Fig. 4: Sieve analysis of marble rubble (coarse aggregate)

Interlocking mixture proportions

The control mix design for the manufactured product was that used at a producer's factory. Concrete mixture proportions are given in table 4 and mix (1) is the control mix. Some producers, however, claim that marble rubble in powder form improve the properties of interlocking paving units. Therefore, marble rubble in powder form was added in the other five mixes. The product consisted of two layers. The top layer (facing layer) was approximately 8mm→10mm thick while the bottom layer (backing layer) was about 70mm thick. The demoulding ability is an essential criterion for manufacturing paving units. The water contents of the paving units were adjusted based on this criterion. The (w/c) ratio was adjusted for each mix to maintain an almost zero slump. Both types of coarse aggregates (crushed stone and marble rubble) were not washed prior to mixing. The water content for the top layer was in the range of 45-60 l/m^3 while for the bottom layer was 165-180 l/m^3 while (according to the producers' recommendation). The testing plan is shown in table 5.

Table 4: Interlocking paving units mixture proportions

Constituent Materials (kg/m^3)	M_1	M_2	M_3	M_4	M_5	M_6
Facing layer						
cement	150	150	150	150	150	150
Sand	450	450	360	450	450	450
Marble rubble (3-8mm thick)	-	-	90	-	-	-
Water	55	60	60	55	55	55
Backing layer						
cement	600	600	600	600	600	600
Sand	1125	1125	1125	1125	1125	1125
Crushed stone	675	675	675	675	337.5	–
Marble rubble in powder form	–	225	–	450	450	450
Marble rubble (as coarse agg.)	–	–	–	–	337.5	675
Water	165	180	180	180	180	180

Table 5: Interlocking Paving Units testing plan

Tests carried out on Products	Testing method	Number of blocks (at each testing age)	Testing age
Compressive strength	ESS 4382	5	28 days & 180 days
Water absorption percentage	ESS 4382 EN 1338	5 5	28 days & 180 days
Abrasion resistance	ESS 4382 ASTMC 418	3	28 days & 180 days
Tensile splitting strength and failure load	EN 1338	8	28 days & 180 days

RESULTS AND DISCUSSIONS

Compressive strength and tensile splitting strength

The compressive strength and tensile splitting strength for the six mixes was determined at 28 and 180 days. For 28 days after casting, the specimens were sprayed twice daily.

According to ESS, the specimens should be tested at age 28 days. However, the specimens were tested again at age 180 days for durability purpose but were not sprayed. They were kept at room temperature and then tested. It is worth mentioning that the method of testing in the ESS is the same as that in the ASTMC140. However, the ASTM states that only three specimens should be tested and did not mention the method of curing nor at what age should the specimens be tested [4].

As noted earlier, mix (1) is the control. The difference between mix (1) and mix (2) is that mix (2) had marble powder added to it in the bottom layer (20% by weight of sand). Compressive strength of mix (2) increased by 9% at age 28 days and there was no difference in compressive strength at age 180 days.

According to the ESS, there are three categories for paving units based on compressive strength, and water absorption percentage. The three categories are; heavy duty, medium duty, and normal duty.
When comparing mix (3) with mix (1), the difference between them is that sand in the top layer was replaced by 20% marble of size (3-8mm) by weight in an attempt to improve the quality of the top layer concerning abrasion resistance. As shown from the compressive strength results, the average compressive strength of mix (3) decreased by 10% when compared to mix (1). Also, the average compressive strength of mix (3) was slightly less than that required by the ESS for normal duty. Mix (4) had marble powder added to it (40% by weight of sand) when compared to mix (1). The average 28 days compressive strength increased by almost 11% when compared to the control mix (mix 1). Both mixes 5 and 6 contained 50% and 100% marble rubble as coarse aggregates respectively, as well as, marble powder (40% added by weight of sand) in the backing layer. The 28 days compressive strengths results of mixes 5 and 6 were almost the same as that of mix (1). On the other hand, when comparing the results of both mixes with mix 4 (which contained 40% added by weight of sand marble powder and 100% crushed stone as coarse aggregates), the results show that the compressive strength of mixes 5 and 6 are less than that of mix 4 by 10%.
Based on the 28 days compressive strength test results, mixes 1, 2, 4, 5 and 6 fall in the normal duty category. Mix 3, on the other hand, gave slightly lower compressive strength.
As expected, the compressive strength at age 180 days was greater than the 28 days compressive strength for all six mixes. The increase in strength was in the range 17% to 27%. The test results are given in table 6. The mixes at 180 days compressive strength can be still categorized as normal duty except for mix (4) which gave 35N/mm^2 compressive strength. According to ESS, the paving units can be categorized as normal duty if product average compressive strength is 35N/mm^2 with no individual unit less than 30N/mm^2. Figure 5 shows the effect of marble rubble on compressive strength.
The conformity criteria in BSEN1338 [5] for tensile splitting strength and failure load per length are as follows: No block shall have a tensile strength less than 3.6MPa nor a failure load per length less than 250N/mm.
The results in table 6 show that the requirement for failure load was satisfied for the six mixes at both ages (28 and 180 days). However, all six mixes did not meet the requirement of minimum tensile strength at age 28 days. Mixes 1, 2, and 3 at age 180 days still did not satisfy the limit for minimum tensile strength. On the other hand, mixes 4, 5, and 6 gave tensile strength that was greater than that required by the European Standard.
The results indicate that utilization of marble powder (40% by weight of cement) solely as the case in mix 4, in addition to the use of marble as coarse aggregate (mixes 5 and 6) improve tensile strength at age 180 days. The effect of marble rubble on tensile strength and failure load are shown in figures 6 and 7 respectively.

Table 6: Test results of compression strength and tensile splitting strength

Mix no.	Average product compressive strength (N/mm^2)		Minimum Tensile Splitting Strength (N/mm^2)		Minimum Failure load/length (N/mm)	
	28 days	180 days	28 days	180 days	28 days	180 days
1	27.1	34.4	2.06	2.8	263	347
2	29.5	34.0	1.94	3.26	265	415
3	24.3	28.4	2.01	2.82	255	359
4	30.0	35.0	2.98	4.8	396	638
5	27.0	31.9	2.45	3.84	329	493
6	27.0	31.3	2.45	3.9	329	524

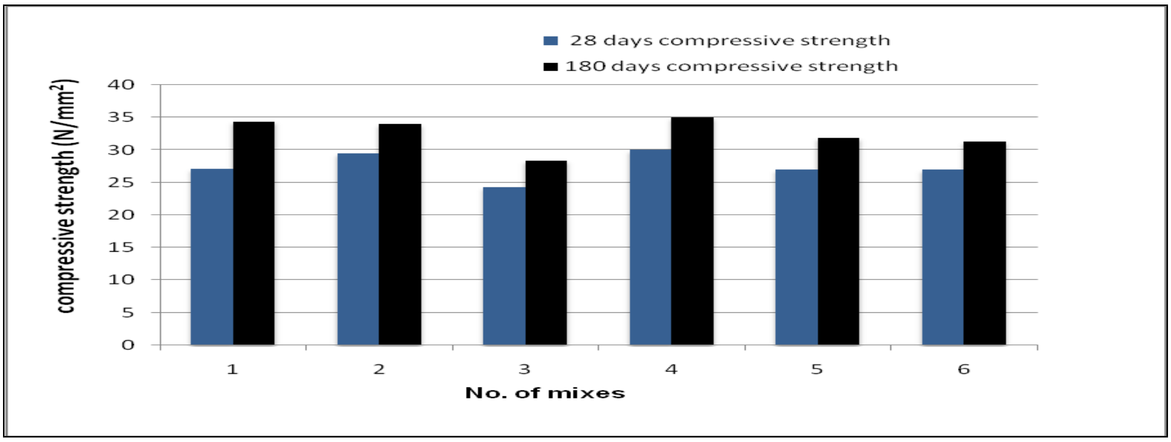

Fig. 5: Effects of marble rubble on compresive strength

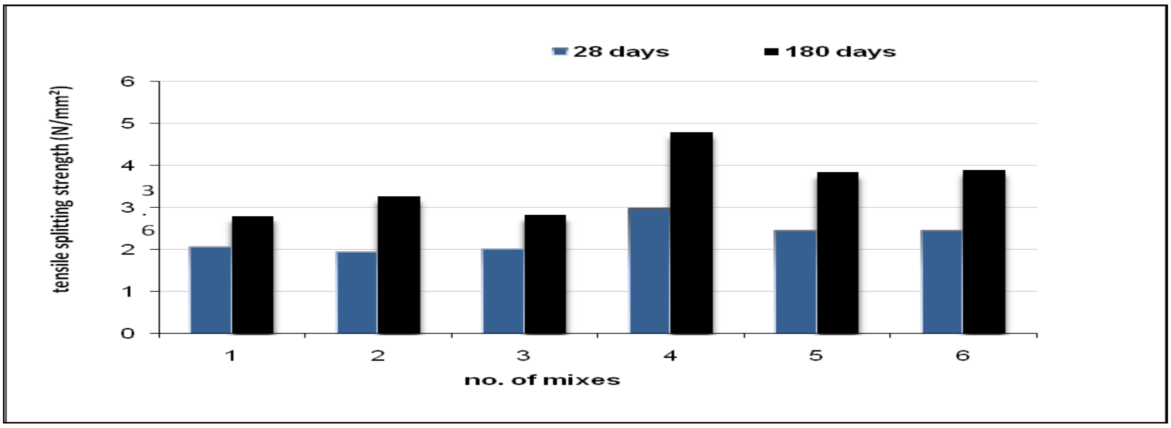

Fig 6: Effects of marble rubble on tensile splitting strength

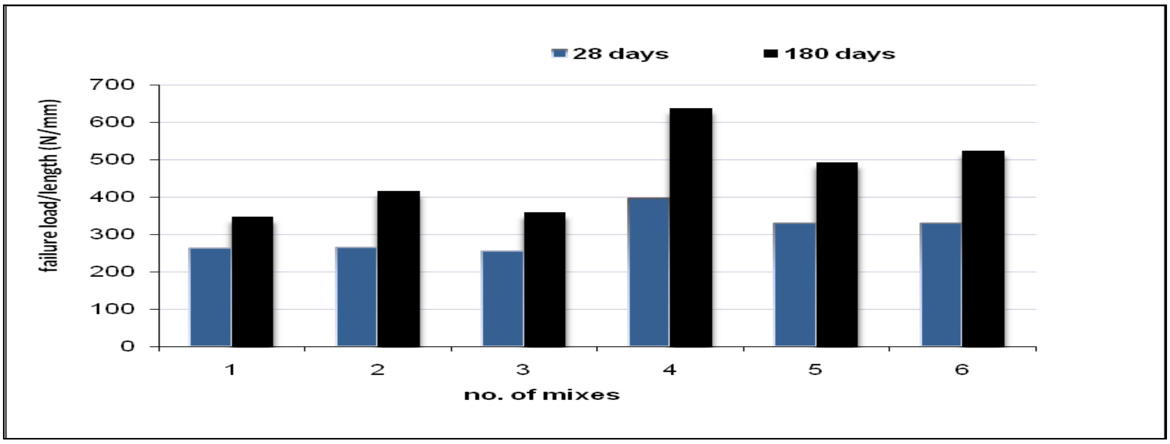

Fig 7: Effects of marble rubble on failure load / length

Water absorption Percentage

According to ESS, average water absorption for normal duty paving units should not be greater than 8% with no individual block greater than 10%.ASTM states that the average absorption of test samples shall not be greater than 5% with no individual unit greater than 7 % [6]. It should

be noted that ASTM does not categorize paving units as does the ESS. EN states that no block shall have a water absorption greater than 6% by mass.
The ESS and ASTM [7] require that the specimens be immersed in water at temperature 15.6 to 26.7 °C for 24h after which the specimens are weighed to obtain the saturated weight. Subsequent to saturation, the specimens are dried in a ventilated oven at 100 to 115°C for 24 h. The specimens are then weighed to obtain the oven-dry weight.
The same testing procedure is applied for the specimens tested according to EN specification except that the specimens are immersed for a minimum period of three days. The minimum period for drying the specimens in an oven at a temperature (105±5)°C is also three days.
The six mixes met the requirements dictated by the ESS and ASTM for both ages (28 and 180 days). As expected, mixes 4, 5, and 6 gave water absorption percentage less than that of the control mix. It is to be noted that these mixes contain marble rubble as 40% by weight of sand (mix 4) and also as coarse aggregates (mixes 5 and 6). The results are shown in table 7.
As expected, water absorption percentage was lower at age 180 days than that at 28 days for the 6 mixes when tested according to EN. Nevertheless, all six mixes did not satisfy the requirement of EN at both ages (the result of mix 6 is the average absorption percentage at age 180 days and not the individual unit). Figure 8 illustrates the effects of marble rubble on absorption percentage

Table 7: Test results of water absorption percentage

Mix no.	According to ESS		According to EN	
	28 days	180 days	28 days	180 days
1	5.4	4.6	7.3	6.7
2	6.4	5.3	8.45	7.6
3	5.7	4.78	9.0	7.7
4	4.1	4.0	7.15	6.9
5	4.2	4.0	7.2	6.9
6	4.37	4.24	6.3	6.0

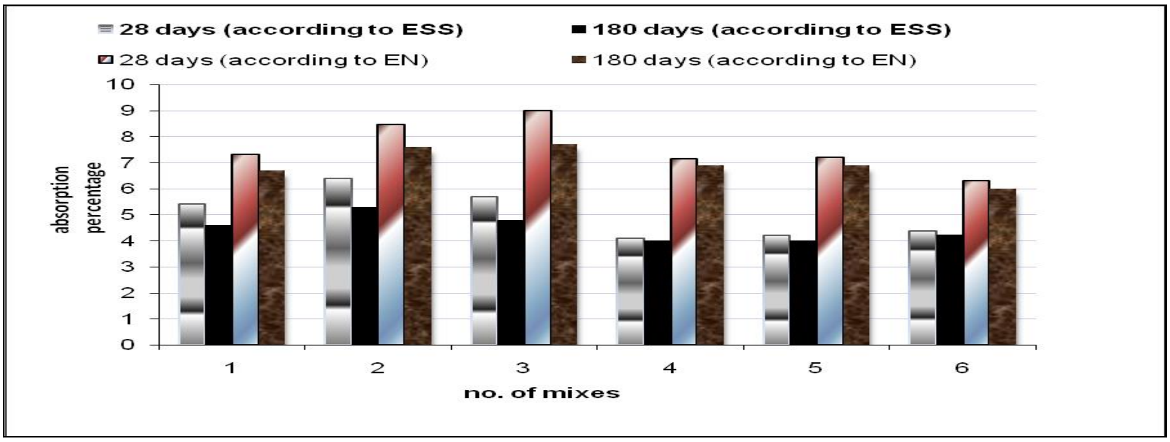

Fig. 8: Effects of marble rubble on absorption percentage

Abrasion Resistance

The abrasion resistance test was carried out by sandblasting. This procedure simulates the action of waterborne abrasives and abrasives under traffic on concrete surfaces. It is worth mentioning that the test procedure dictated by the ESS [8] is the same as that of the ASTM C418 [9]. Also the limits of the Egyptian Standard is the same as that of the ASTM C936 which covers the requirements for interlocking concrete paves manufactured for the construction of paved surfaces. In other words, ASTM C418 covers the testing procedure while ASTM C936 states the limits. The ESS, on the other hand, comprises two parts: part one covers the requirements for the three parameters: compressive strength, water absorption, and abrasion resistance; while part two covers the testing procedure for the three parameters.

The limits are as follows: the specimens shall not have a greater volume loss than $15cm^3/50cm^2$. The average thickness loss shall not exceed 3mm. It should be noted that the previous limits apply to the three categories of paving units in the ESS. The results are given in table 8 and shown in figures 9 and 10. Mixes 1 and 2 did not satisfy both conditions at the tested ages (28 days and 180 days). Mixes 3, 4, 5, and 6 met the condition concerning volume loss at 180 days testing age. Mix 3 contained rubble at the facing layer. However, the second condition which is the thickness loss was not satisfied by the six mixes at both testing ages. Nevertheless, it can be noticed that abrasion resistance improved with time for the six mixes and that the addition of marble rubble (40% by weight of sand) reduced the volume loss significantly for mixes 4, 5, and 6 (by almost 50%) at both testing age. Figures 9 and 10 illustrate the effects of marble rubble on volume loss and thickness loss respectively.

Table 8: Abrasion resistance results

Mix no.	Volume loss ($cm^3/50cm^2$)		Avg. thickness loss (mm)	
	28 days	180 days	28 days	180 days
1	43.3	27.5	10.0	8.3
2	49.86	29.6	12.25	11.76
3	34.53	13.41	10.15	9.5
4	21.0	12.23	10.12	9.9
5	19.31	8.21	8.45	6.9
6	24.75	13.22	11.64	7.3

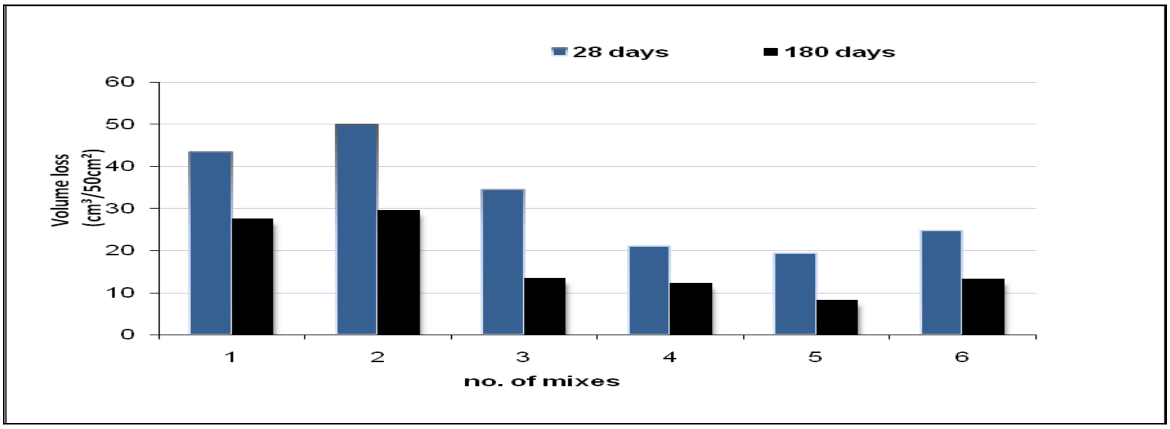

Fig. 9: Effects of marble rubble on volume loss

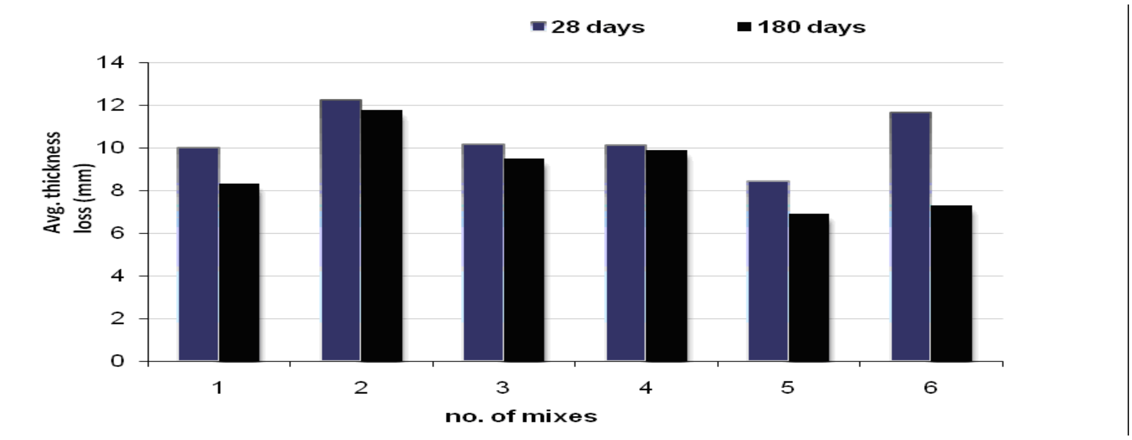

Fig. 10: Effects of marble rubble on thickness loss

CONCLUSIONS

Based on the experimental results obtained from this study, the following conclusions can be drawn.

1. In this study, marble rubble (in powder form) enhanced compressive strength at 20% and 40% by weight of sand.
2. The addition of marble as coarse aggregate decreased compressive strength.
3. All six mixes satisfied the requirement for failure load according to BSEN specification.
4. Mixes which contained marble powder at a dosage of 40% by weight of sand (Mix 4), as well as, mixes 5 and 6 (which contained 50% and 100% as coarse aggregates, respectively) gave greater tensile strength than that required by the European Standard at age 180 days.
5. Marble rubble in its various forms reduced water absorption.
6. Abrasion resistance improved significantly upon the addition of marble rubble, however, it still did not satisfy the requirements of both ESS and ASTM.

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ROLLER COMPACTED CONCRETE PROPERTIES CONTAINING HIGH PERCENTAGE FLY ASH AND HIGH SILICA FUME

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ABSTRACT

Concete pavement has widely been used around the world .Recently, among different types of concrete pavement roller compacted concrete(RCC) has been used as an alternative to conventional concrete pavements. Roller Compacted Concrete (RCC) is based on the concept that a no-slump concrete mixture transported, placed, and compacted with the same construction equipment that is used for earth and rockfill dam it can meet the design specifications for conventional mass concrete. RCC must be effectively consolidated,
And dry enough to support the weight of a vibratory roller, yet enough to permit adequate distribution of the paste throughout the mass during the mixing and compaction operation.
In this study, different percentages(10,20,30,40,50,60)% of both fly ash and silica fume were used as cement replacement, concrete mix design and the construction technique used for specimen preparation are relative to roller compacted concrete technology. The tests of mechanical properties of RCCP as compressive strength, and permeability were carried out. The test results showed that the addition of silica fume and flyash can obviously modify the mentioned characteristics and satisfy the required conditions for Roller Compacted Concrete Pavements (RCCP) based on ACI 325-1R.

Keywords: Concrete; Silica Fume;Fly ash , Roller; Compacted; Concrete; Pavement RCCP

INTRODUCTION

Rcc is stiff and hard concrete with no slump the maximum size of gravel usually not maximum than 19 mm and its component are compacted by vibratory rollers.The required minimum strength is about 28 Mpa.It placed in layers not exeed than 250 mm .It has many application in roads, airports,dams,nuclear constructions and irrigation works.It reduces the cost for no need for steel reinforcement,and forms so this increase construction speed opposite to the other types of different concretes Rcc is stiff enough to be compacted by vibratory rollers, so it is required high density and low absorption(Portland Cement Association,2006) then ,the previous properties lead toexcellent durability and eleminate seepage through pavement (Portland Cement Association,2006). [4]
.Rutting caused by heavy loads and truck in flexible pavement.Instead of curing by new asphalt RCC can be usedas a more durable option(Portland Cement Association 2005) because the asphalt adher to RCC.
Many roller compacted concrete pavement projects have been constructed in many parts of the world. Using RCC in North America started in the early 1940s. RCC has proven to be a reliable, economical, durable material for low speed, heavy-duty pavements. . Other successful projects documented by the Portland Cement Association(PCA) include ports and intermodal terminals, storage areas, and roads (Piggott 1999).(RCC pavements have been used for: Highway weigh stations, Airport aprons, Docks and container ports, Multimodal facilities, and Heavy industrial facilities such as logging and automobile manufacturing.
The first project using RCC was built in North America in 1942 by the US ArmyCorps of

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Engineers in Yakima, WA. The following list shows how RCC has developed over the years and suggests different types of applications. These were provided by the PCA (Piggott 1999). Some form of RCC pavement was built in Sweden in the 1930s. The U.S. Army Corps of Engineers built a pavement in 1942. It was an RCC runway in Yakima, WA Cement-treated base (CTB) was used in the 1960's in the Oregon logging industry. CTB was used in Vancouver for a log sorting yard in 1976. The CTB was improved in this case by increasing the cement content of the soil-cement mixture from 6% to 12%.
Cementitious materials used in RCC pavement mixtures include Portland cement or blended hydraulic cement and may include pozzolan [5]. The selection of any pozzolan for use in RCC should be based on its conformance with applicable standard or specifications, its performance in concrete and its availability at the project location. Many of the RCC pavements constructed to date have been constructed using Type I or II Portland cement and Class F or Class C fly ash [2] or lignite ash [6]. Silica fume as a partial replacement of cement has been used in some RCC pavement projects and the durability aspects like permeability coefficient of this material has been investigated [7]. Moreover, it is shown that the addition of silica fume to RCC mixtures increases the working time [8].
Fly ash, is a by-product from thermal power plants, can be substituted for large portions of Portland cement, significantly improving concrete's environmental characteristics. Fly ash, consisting mostly of silica, alumina, and iron, forms a compound similar to Portland cement when mixed with lime and water. [9].
Fly ash is a non-combusted by-product of coal-fired power plants. It has been successfully used in cement based materials like concrete and controlled low-strength materials besides still being land-filled due to its large volume generation. However, when high volumes (more than 35% of cement replacement) are used in concrete and CLSM, it is called High Volume Fly Ash Concrete. It creates a stronger, more durable product and reduces concrete's environmental impact considerably. Due to its strength and lower water content, cracking is reduced. Two types of fly ash are available: Class C fly ash, which is typically light or tan colored and is produced from burning lignite or sub-bituminous coal, and Class F fly ash, which is dark grey and is produced from burning anthracite or bituminous coal. By displacing a large percentage of the cement in concrete, fly ash significantly reduces the associated environmental impacts of CO2 production and air pollution [9-12]. .

Advantages of utilization HVFA concrete

- Less energy intensive manufacture
- Higher ultimate strength
- More durable
- Requires less water
- Uses a waste by-product
- Creates fewer global warming gases

In this paper, the effects of both silica fume and fly ash separately by incorporation to improve the mechanical properties of RCC pavement have been studied. Firstly, laboratory test on the selection of mixture proportions was conducted and then the mechanical tests have been carried out. The main objectives of the tests were as following:
To determine the mix proportion by evaluation of consistency test
To determine the relationship of the silica fume and fly ash dosage, and their effect on the relation between the compressive strength and the permeability for different concrete mixes**.**

MATERIALS

Cement

Egypt ordinary Portland cement from Iron steel factory was used. The whole quantity of cement is brought to the laboratory and stored in a dry place. The fineness, setting time, and compressive strength was within accept at the limits according to the Egyptian standard specifications .Table (1) shows the properties of cement.

SilicaFume

It is collected as a byproduct in the production of silicon and ferro silicon alloys such as ferrochromium, ferromanganese, ferromangesium and calcium silicon. Silica fume consists of very fine particles with a surface area of the order of 20000m2/kg and with particles approximately 100 times smaller than the average cement particles. Silica is a highly effective pozzolonic material because of its extreme fineness and high silica content. It is used in concrete to improve its various properties. adding silica fume at 10% with cement replacement release improvements. Table(2) illustrates its properties:

Fly ash

The flyash used in this experimentation was secured and imported from the India at Power Plant of Rourkela Steel Plant, Orissa (India). Physical and chemical properties of flyash are shown in Table (3)

Aggregates

1- Siliceous aggreates

The coarse aggregates used in this work are from Yahmom quarry near Helwan. It is siliceous gravel free from deleterious material with 3/4'' nominal maximum size. The gradation of the gravel is in Table (4) The physical properties of the gravel are shown in Table (5).

2- Sand

The fine aggregate used in this investigation is siliceous sand from Cairo quarries, Table . (5) shows the gradation of the used sand and Table (6) shows its properties.

3-Water

Normal tap water was used for mixing for the present investigation Water cement ratio is used to measure consistency is investigated from tests in vebe test. In addition its measure as percentage of the total dry weight of concrete.

4-Admixture

Water reducing plasticizer having trade name sicament was used during the mixing to increase workability under reduced water cement ratio.

Table 1: Physical properties and chemical composition of portland cement(grade 43)

Silicon Dioxide, SiO_2	75.57
Aluminum Oxide, Al_2O_3	16.10
Ferric Oxide, Fe_2O_3	5.0
$SiO_2 + Al_2O_3 + Fe_2O_3$	77
Calcium Oxide, CaO	3.8
Magnesium Oxide, MgO	2.3
Titanium Oxide, TiO_2	1.3
Potassium Oxide, K_2O	0.5
Sodium Oxide, Na_2O	0.4

Table 2 : Chemical composition of silica fume

Physical test	Results obtained
Fineness (retained on 90 μm sieve	7.1
Fineness: specific surface (air permeability test) (m^2/kg)	310
Normal consistency	32%
Initial	115
Final	205
3-day	24.2
7 -day 28-day	36.1 46.8
Chemical composition	
CaO	62.5
SiO2	21.4
Al2O3	5.2
Fe2O3	3.1
MgO	2.7
SO3	1.8
K2O	0.65
Na2O	0.25
LOI	1.8

Table 3 : Chemical composition of fly ash

Chemical analysis	ASTM requirement C- 618	Class F fly ash (%)
Aluminum Oxide, Al_2O_3		26.10
Calcium Oxide, CaO		3.8
Ferric Oxide, Fe_2O_3		5.0
LOI (1000°C),	6.0 max	1.9
Magnesium Oxide, MgO	5.0 max	2.3
Moisture	3.0 max	0.3
Potassium Oxide, K_2O		0.6
Silicon Dioxide, SiO_2		95.57
$SiO_2 + Al_2O_3 + Fe_2O_3$	70.0 min	87.9
Sodium Oxide, Na_2O	1.5 max	0.4
Sulfur trioxide, SO_3	5.0 max	1.6
Titanium Oxide, TiO_2		1.4

Table 4 : Properties of Fayoum gravel

Property	Test Result
1-Apparent specific gravity	2.56
2-Volume weight (kg/m3)	1700
3-Clay silt and fine dust dry wt.	1.00
4-Organic impurities	zero

Table 5 :Sieve analysis of sand

Sieve Size	No.4	No.8	No.16	No.30	No.50	No.100	No.200
% Passing by Weight	90	50	45	25	15	11	9

Table 6 : Properties of sand

Property	Test Result
1-Apparent specific gravity	2.5
2-Volume weight (kg/m3)	1730
3-Clay silt and fine dust dry wt.	0.9
4-Organic impurities	zero

MIXTURE PROPORTIONING

The methods for mix design concepts can be placed into two broad categories: 1) proportioning based on concrete consistency tests (concrete approach) 2) proportioning based on soil compaction tests (soil approach). For this experiment, the determination of RCCP mix proportion was mainly based on concrete approach**.**

The mixture proportions are given in Table 7.and 8. Two series of tests have been carried out in using 450 kg/m^3 cementations material. For the compositions, the fly ash and the silica fume content has changed in order to study its influence on mechanical behaviour of RCCP.

FABRICATION AND TESTING OF SPECIMEN

1) Proportioning based on concrete consistency tests

Conventional concrete specimen fabrication procedures cannot be used to fabricate RCC test specimens due to the stiff consistency of the concrete. In concrete approach, the procedures used involve vibrating the fresh RCC sample on a vibrating table under a definite surcharge load. The method of proportioning is by evaluation of concrete consistency for optimum workability using a modified Vebe apparatus (ASTM C1176-92) [9]. It consists of a vibrating table of fixed frequency (60 ± 1.67Hz) and a amplitude of vibration (0.43 ± 0.08 mm).

A representative sample of RCC is loosely placed in the container, in three lifts. Three layers are vibrated, until paste forms around the edge of the surcharge, having a mass of 9.1 kg. The Vebe time is obtained based on ASTM C1170-91 [10] and optimum water content in the mix design is selected as the obtained Vebe time is located between 30 and 45 seconds. For a constant rate of 0.2 MPa/s.

2) Proportioning based on soil compaction tests (soil approach)

The samples are in the shape of 150x150x150 mmcubes. Modified proctor standard hammer was used to compact the concrete(ASTM D 1557) . This hammer weighs 4.5 kg and falls 457mm. Concrete was cast in three levels.

Laboratory Tests

Samples were tested for permeability and compressieve strength according to(B.S 1881-116,1983).The results are shown in Tables 9 and 10 for compressieve strength at 7,28,91,365 days for both fly ash and silica fumes concrete . In addition Tables 11,12 showed the relationship between the compressive strength and permeability at 28 days for fly ash and silica fume concretes.

Table 7: Silica fume Roller Compacted Concrete Mixture proportions

Mixture No.	M-1	M-2	M-3	M-4	M-5	M-6	M-7
Cement (kg/m^3)	450	405	360	315	270	225	180
Silica fume(%)	0	10	20	30	40	50	60
Silica fume (kg/m^3)	0	45	90	135	180	225	270
Water (kg/m^3)	162	162	162	162	162	162	162
W/(Cement + Silica fume)	0.47	0.0.47	0.47	0.47	0.47	0.47	0.47
Sand SSD (kg/m^3)	535	535	535	535	535	535	535
Coarse aggregate (kg/m^3)	1225	1225	1225	1225	1225	1225	1225
Superplasticizer (l/m^3)	3.8	4.3	4.4	4.6	4.7	4.6	4.5
Density (kg/m^3)	2375	2374	2374	2375	2376	2370	2366

Table 8: High Fly ash Roller Compacted Concrete Mixture proportions

Mixture No.	M-1	M-2	M-3	M-4	M-5	M-6	M-7
Cement (kg/m3)	450	405	360	315	270	225	180
Fly ash (%)	0	10	20	30	40	50	60
Fly ash (kg/m3)	0	45	90	135	180	225	270
Water (kg/m3)	162	162	162	162	162	162	162
W/(Cement + Fly ash)	0.36	0.36	0.36	0.36	0.36	0.36	0.36
Sand SSD (kg/m3)	535	535	535	535	535	535	535
Coarse aggregate (kg/m3)	1225	1225	1225	1225	1225	1225	1225
Superplasticizer (l/m3)	3.8	4.3	4.4	4.6	4.7	4.5	4.0
Density (kg/m3)	2375	2374	2374	2375	2370	2370	2363

ANALYSIS OF THE TEST RESULTS

Compressieve Strength

Fly Ash Roller Compated Concrete

Compressive strength of concrete mixtures made with and without fly ash were determined at the ages of 7, 28, 91 and 365 days, and results are shown in Fig. 1 and Table (9) . They showed the variation of compressive strength with cement replacements at various ages. Form the test results, it can be seen that :

The compressive strength of high-volume fly ash concrete mixtures with 30, 40, 50, and 60% cement replacement was lower than the Control Mixture (M-1) at all ages. At 7 days, compressive strength of Control Mixture M-1 was 27 MPa whereas it was 17.0 MPa for Mixture M-4 (30% fly ash), 15 MPa for Mixture M-5 (40% fly ash), 12.0 MPa for Mixture M-6 (50% fly ash), and 10.0 MPa for Mixture M-7 (60% fly ash).

At 28 days, compressive strength of Control Mixture M-1 was 44 MPa, whereas it was 30.0 MPa for Mixture M-4 (30% fly ash), 27.0 MPa for Mixture M-5 (40% fly ash), 25.0 MPa for Mixture M-6 (50% fly ash), and 20.0 MPa for Mixture M-5 (60% fly ash).

It must be taken into consideration that at 10,20% (mixes M-2,M-3) of fly ash replacing cement the results of compressive strength are less than the others control one ,in small percentage so, good results released with at 10,20% with fly ash

Table 9: Compressive Strength for High Fly ash Roller Compacted Concrete Mixes

Mix No.	Fly ash Content %	Compressieve Strength Mpa			
		7 Days	28 Days	91 Days	365 Days
M-1	0	27	44	45	50
M-2	10	25	40	43	48
M-3	20	20	35	40	45
M-4	30	17	30	37	44
M-5	40	15	27	33	39
M-6	50	12	25	32	36
M-7	60	10	20	25	27

Table 10: Compressive Strength for High Silica fume Roller Compacted Concrete Mixes

Mix No.	Silica Fume Content %	Compressieve Strength Mpa			
		7 Days	28 Days	91 Days	365 Days
M-1	0	27	44	45	50
M-2	10	33	47	49	54
M-3	20	35	45	50	55
M-4	30	37	39	46	51
M-5	40	30	32	39	46
M-6	50	22	30	36	38
M-7	60	19	26	34	37

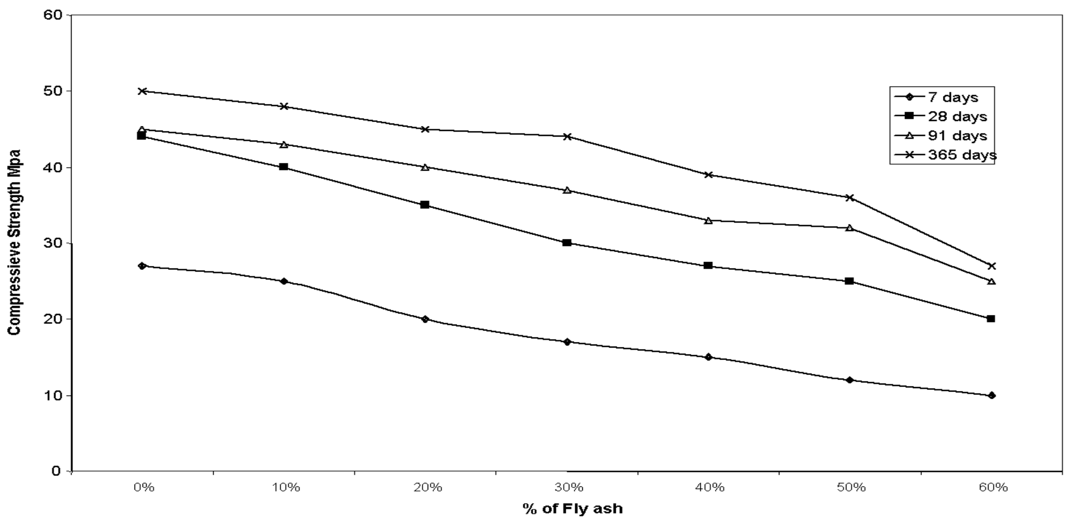

Fig 1 : Compressieve strength of high fly ash concrete

Silica Fume Roller Compated Concrete
Compressive strength of concrete mixtures made with and without silica fume were determined at the ages of 7, 28, 91 and 365 days, and results are shown in Table (10) and Fig.2. They

showed the variation in compressive strength with cement replacements at various ages. Form the test results, it can be seen that :
the compressive strength of high-volume silica fume concrete mixtures with 10,20,30and 40, % cement replacement was higher than the Control Mixture (M-1) at 7 days, compressive strength of Control Mixture M-1 was 27 MPa whereas it was 33,35,37,30 for M-2,M-3,M-4,M-5.For 28-days the compressieve was 44Mpa in the control mix and it was 45 Mpa at 20% silia fume addition . In addition the compressieve was higher than the control mix especially at 20% of silica fume addition .At 40,50and 60 % the compressieve was decreased than the control one.

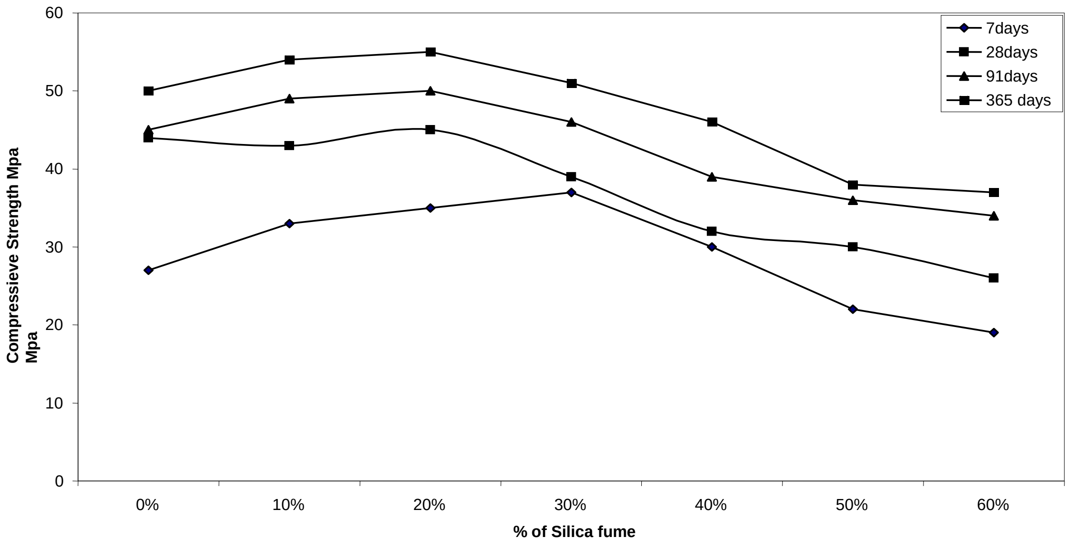

Fig 2 : Compressive strength of high silica fume roller compacted concrete

Permeability
Fly Ash Roller Compated Concrete
As the % of flyash increase the permeability decrease .Table (11) and Fig.(3) showed the permeability for the different flyash concrete mixes but at 50 and 60 % fly ash replacing cement the permeability of concrete samples increased than the control one.

Silica Fume Roller Compated Concrete
As the % of silica fume increase the permeability decrease , but at 60% of silica fume replacing cement the Table (12) and Fig.(4) showed the permeability for the concrete samples increased than the control on

Table 11: Test Results for Compressive Strength And Permeability For Fly ash Concrete Samples

Mix No.	Fly ash Content %	Compressive Strength Mpa	Permeability(mm) at 28 Days
M-1	0%	44	30
M-2	10%	40	27
M-3	20%	35	25
M-4	30%	30	22
M-5	40%	27	29
M-6	50%	25	35
M-7	60%	20	44

Table 12: Test Results for Compressive Strength And Permeability For Silicafume Concrete Samples

Mix No.	Silica Fume Content %	Compressive Strength Mpa at 28 days	Permeability(mm) at 28 Days
M-1	0	44	30
M-2	10	43	20
M-3	20	45	15
M-4	30	39	18
M-5	40	32	20
M-6	50	30	25
M-7	60	26	39

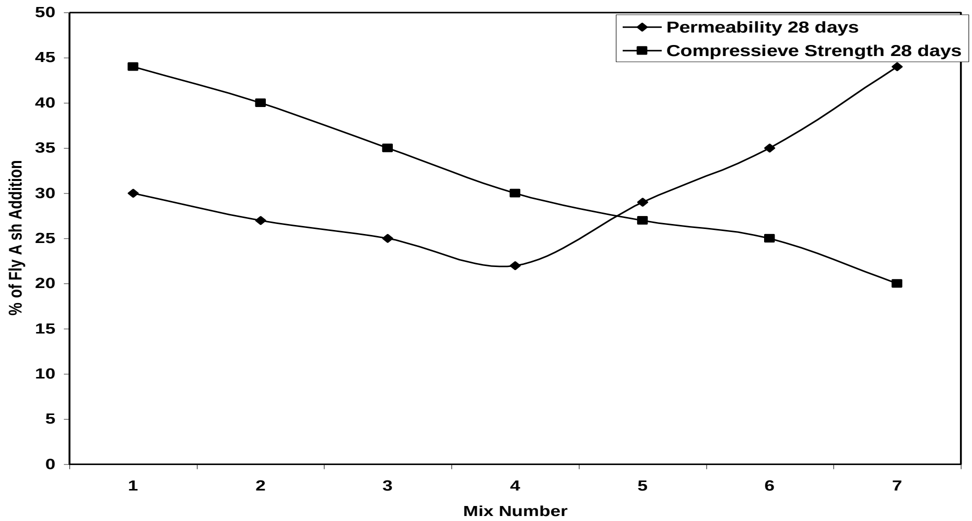

Fig 3 : Relation between compressive strength and permeability of high fly ash roller compacted concrete mixes at 28 –days

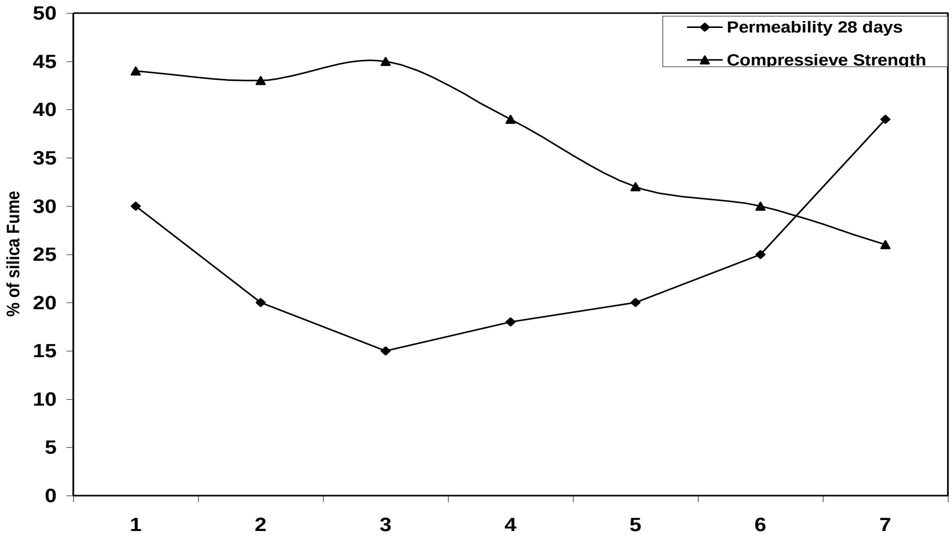

Fig 4 :Relation between compressive strength and permeability of silica fume roller compacted concrete mixes at 28 -days

CONCLUSIONS

1. The compressive strength of high-volume fly ash concrete mixtures with 30, 40, 50, and 60% cement replacement was lower than the Control e At 7 days,
2. It must be taken into consideration that at 10,20% (mixes M-2,M-3) of fly ash replacing cement the results of compressive strength are less than the others control one ,in small percentage so, good results released with at 10,20% with fly ash.
3. As the % of flyash increase the permeability decrease .
   The permeability at 50 and 60 % fly ash replacing cement increased than the control.
4. The compressive strength of high-volume silica fume concrete mixtures with 10,20,30, 40, 50, and 60% cement replacement was higher than the Control Mixture (M-1) . At 7 day
5. As the % of silicafume increase the permeability decrease ,
   at 60% of silica fume replacing cement ,the permeability for the concrete samples increased than the control one.
6. At 60% of silica fume replacing cement the permeability for the concrete samples increased than the control one.

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SHEAR BEHAVIOR OF SLURRY INFILTRATED FIBER CONCRETE BEAMS

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ABSTRACT

An experimental study has been carried out to investigate the shear behavior of slurry infiltrated fiber concrete (SIFCON) beams. Thirteen specimens have been tested with different steel fiber volume of fractions (V_f), different shear span-depth ratios, different fiber shapes, and different sizes of beams. Beams have been tested under four-point loading. The results demonstrated that the nominal shear stress at diagonal cracking and the ultimate shear strength increased with the increase of V_f or the decrease in shear span-depth ratio. Compared to plain and corrugated fibers, hooked-end fiber was proved to give the highest shear strength of SIFCON beams. Less size effect was observed for SIFCON beams than conventional ordinary concrete (RC) beams. Using a simplified analytical model, a design equation was deriven to calculate the shear strength of SIFCON as well as fiber reinforced concrete (FRC) beams. The comparison between the proposed equation and the experimental results was proven to give good agreement.

Keywords: SIFCON; shear strength; diagonal; cracking; steel fibers.

INTRODUCTION

Shear failure in reinforced concrete beam takes place when the principal tensile stress within shear span exceeds the tensile strength of concrete and a diagonal crack propagates through the beam web. This failure has catastrophic nature due to the brittle behavior of concrete in tension. The addition of discontinuous fibers to such matrices leads to drastic improvement in their toughness [1]. The addition of small discrete steel fibers in concrete helps improving the post-cracking tensile stress of concrete and, hence, leads to enhancing the shear performance. However, the practical volume fraction of fibers in conventional FRC is within 2%, because premixing fibers more than 2% by volume may lead to fiber segregation, fiber balling and excessive air entrainment [2,3,4]. This limited volume of fibers will not improve the strength in a significant way; though it will increase the toughness. In order to obtain an increase in strength, rather than only an increase in toughness, a higher volume of fibers should be used. Slurry infiltrated fiber concrete (SIFCON) is a high-performance fiber reinforced cementious composite (HPFRCC), in which a high volume fraction of fibers is used. It is made by pre-placing a large volume fraction of fibers into the formwork, followed by infiltration of cement-based slurry. SIFCON was first used in 1968 by Haynes [3]. Later in 1984, Lankard and Newell [4] showed the concept of using of SIFCON in the construction of pavement overlays. Late in 1980s and 1990s, Naaman and his group extensively studied the basic SIFCON behavior [5-14]. The practical range of fiber volume fractions in SIFCON is 4 to 12%. However, volume fractions up to 27% have been reported in the technical literature [7]. Due to its high volume of fibers, the SIFCON properties are superior in tension, compression, shear, bond with reinforcing bars, and

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impact. As illustrated in Fig. 1 [14], for a member under tension, the higher volume of fibers (Fig. 1b) would lead to multiple cracks that follow the first crack, in an opposite manner to low volume of fibers (Fig. 1a), which is only accompanied by single crack.

Consequently, an increase in strength takes place in the concrete with higher volume of fibers and this takes place parallel with cracking process in a way similar to strain hardening. Finally, failure is localized in one single crack and concrete experiences softening.

Though it is possible to use only cement slurry, the use of very fine sand in slurry enhances the properties of SIFCON. This helps reducing shrinkage cracking and enhances the bond between the fibers and the paste. The benefits of using fine sand might be significant in terms of strength, stiffness and cost [7]. Four main variables affect SIFCON properties; slurry strength, fiber volume (V_f), fiber alignment, and fiber type. The cement slurry is the backbone of the specimen. It affects the elastic modulus, tensile strength, and compression strength of SIFCON matrix. Fiber pullout strength also depends upon the slurry compressive strength. The cracking procedure is directly affected by the fiber volume as well as fibers alignment (random, normal to, or parallel to loading). Therefore, both the fiber volume and alignment greatly affects the ultimate strength, residual strength, ductility, and energy absorption. The fiber type was also found to affect the cracking procedure and the elastic modulus of SIFCON [7].

The purpose of the present study is to investigate the shear behavior of slurry-infiltrated fiber reinforced concrete beams.

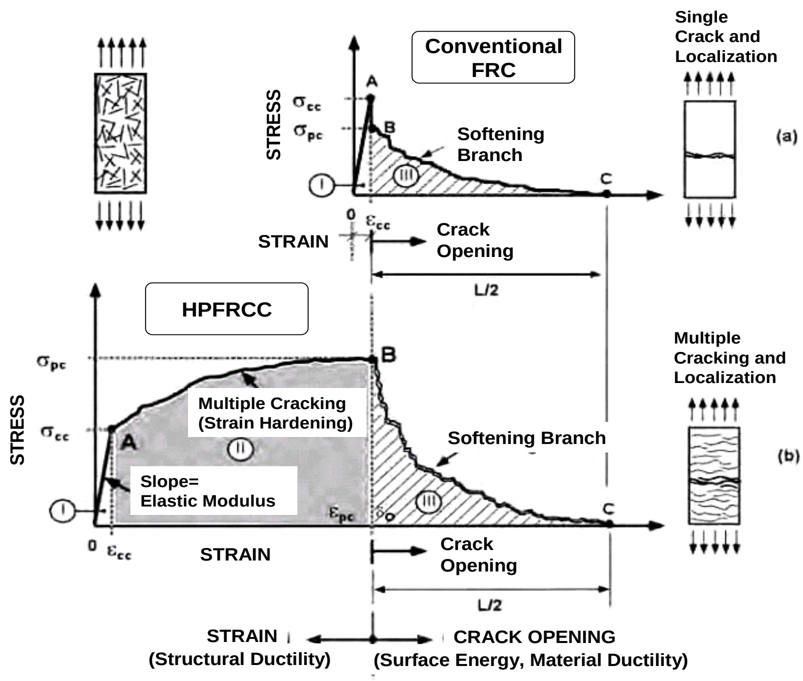


Fig. 1: Behavior of HPFRCC versus conventional FRC [14].

RESEARCH SIGNIFICANCE

Shear failure in reinforced concrete structures is highly brittle when compared to flexural failure. The addition of steel fibers in the concrete matrix is effective in mitigating the brittle failure of reinforced concrete structures. The addition of steel fibers in concrete helps improving the post-cracking tensile strength of concrete and, hence, leads to enhancing the shear strength of reinforced concrete beams.

In conventional fiber reinforced concrete, the volume of fraction of fibers is limited to avoid fiber segregation, balling and excessive air entrainment. Higher fiber volume of fraction is possible in Slurry Infiltrated Fiber Concrete. In this study, the shear behavior of SIFCON beams is investigated. The significance of the outcome of this study arises from the possible huge increase in both shear capacity and ductility which is favorable in reinforced concrete beams.

EXPERIMENTAL PROGRAM

The experimental program consists of 13 specimens constituting five groups as shown in Table (1). The objective of the different groups is as follows:
(a) To study the mechanical behavior of SIFCON under shear.
(b) To investigate the effect of different types of fibers (hooked-end, corrugated, and straight type) (Group B).
(c) To investigate the effect of different fiber volume of fraction (V_f = 1, 4, 5.5, 8, and 10%) (Group C).
(d) To estimate the effect of shear span to effective depth ratio (*a/d* = 2, 3, and 4) (Group D).
(e) To investigate the size effect of beam (*d*= 170, 270, and 370) (Group E).

Test Specimens

Specimens have identical rectangular cross-section (150 mm x 200 mm) except for group E, where size of the beam is variable (200mm x 300mm and 300mm x 400mm). The length of all beams was 1850 mm except for group E (2850 mm and 3850 mm). All beams have no shear reinforcement in shear span. Figure 2 shows typical test specimen.

Table 1: Details of test beams and test variables.

Group	Specimen	Cross Section (bmmxtmm)	Type of Mix	Fiber volum (V_f) %	Fiber type	*a/d*	Longitudinal Reinforcement (mm)	
							Top	Bottom
A	B1	150 x 200	SIFCON	4	Hooked-end	2	2Y12	3Y18
	B2	150 x 200	RC	-	-	2	2Y12	3Y18
	B3	150 x 200	Mortar	-	-	2	2Y12	3Y18
B	B4	150 x 200	SIFCON	4	Straight	2	2Y12	3Y18
	B5	150 x 200	SIFCON	4	Corrugated	2	2Y12	3Y18
C	B6	150 x 200	SIFCON	4	Hooked-end	3	3Y16	6Y18
	B7	150 x 200	SIFCON	4	Hooked-end	4	3Y16	6Y18
D	B8	150 x 200	FRC	1	Hooked-end	2	2Y12	3Y18
	B9	150 x 200	SIFCON	5.5	Hooked-end	2	2Y12	3Y18
	B10	150 x 200	SIFCON	8	Hooked-end	2	3Y16	6Y18
	B11	150 x 200	SIFCON	10	Hooked-end	2	3Y16	6Y18
E	B12	200 x 300	SIFCON	4	Hooked-end	2	3Y18	7Y18
	B13	300 x 400	SIFCON	4	Hooked-end	2	4Y22	8Y25

MATERIAL PROPERTIES

Table 2 shows the properties of the steel bars and fibers used in this study.

Preparation of Test Specimens

The mix proportions of the SIFCON, FRC, and RC are given in Table 3. The mixes were designed for a compressive strength of 40 MPa at 28 days. For SIFCON beams, the fibers were placed in four layers in the beam where the slurry was cast layer by layer as shown in Fig. 3.
For RC and FRC, conventional mixing and casting procedures were adopted. After casting, specimens were covered with wet clothes.
Table 4 shows the 28 day material properties obtained from compression cylinder tests, splitting tests, and flexure tests. The concrete cylinder specimens were (100 mm diameter, 200mm long) for the compression and spilt-tensile tests, while the flexure tests were adopted for (100 mm x 100 mm x 500 mm) specimens.

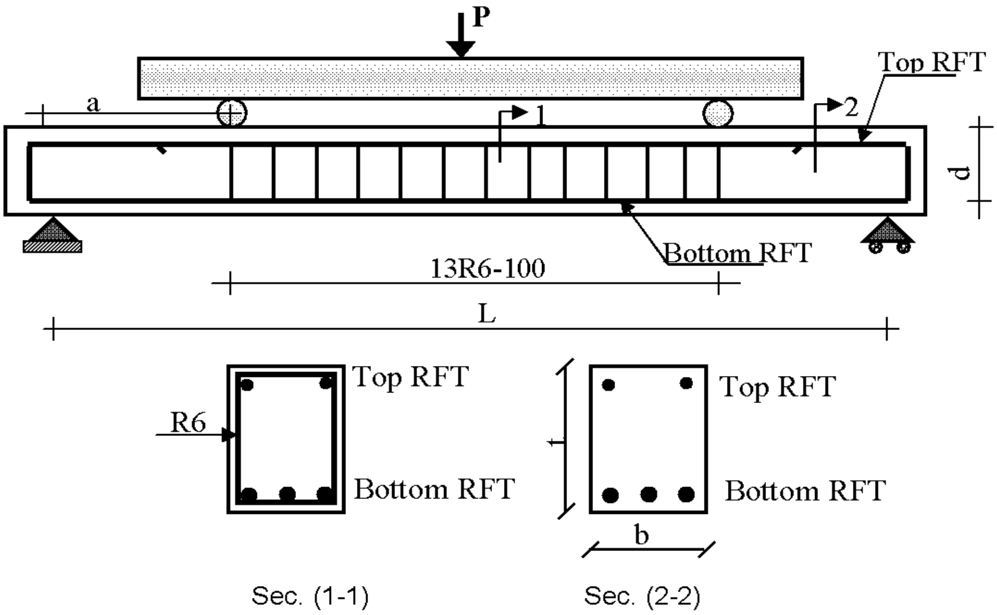


Fig. 2: Beam reinforced details for all groups

Table 2: Properties of the steel bars and steel fibers.

Type	Diameter (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)	Elongation %
Mild steel	6	361	511	19.15
High strength steel	12	404	650	17.6
	16	431	712	14.13
	18	533	779	13.45
	20	548	875	19.4
	22	444	708	15.56
	25	410	676	13.3
Steel fiber	1.0	-	1298.0	-

Table 3. Proportions of slurry and concrete mix (kg/ m^3)

Material	Concrete & FRC	SIFCON [12]
Fine aggregate (Sand)	690	901.83
Coarse aggregate (dolomite)	1032	-
Cement	400	901.83
Water	200	360.73
Super-plasticiser	0.8	11.3

Table 4. Measured properties of slurry and concrete

Mix	Average Compressive Strength (MPa)	Average Tensile Strength (MPa)	Average Flexural Strength (MPa)
	After 28 days		
RC	27.9	4.4	4.52
FRC, V_f = 1%	28.5	4.5	4.7
SIFCON, V_f = 0%	22.7	3.3	3.9
SIFCON, V_f = 4%	42.3	9.9	11.5
SIFCON, V_f = 5.5%	42.4	10.4	11.6
SIFCON, V_f = 8%	46	10.6	12
SIFCON, V_f = 10%	53.6	11.7	13.5
SIFCON, V_f = 4% (straight type)	36	6.9	7.2
SIFCON, V_f = 4% (corrugated type)	38.4	8.7	9.9

Test Set Up and Instrumentation

The beams were tested in a four-point loading condition with the load applied as two equal concentrated loads by means of a steel spread beam as shown in Fig. 4. The load was applied as a displacement control. A calibrated load cell was placed between the jack and spread beam for measuring applied load. LVDTs were used to measure the deflections at the mid-span of the beam. Steel strain gauges were attached on longitudinal steel, and two LVDTs for measuring diagonal concrete strains were attached in the shear spans.
All readings were connected to a data aqusition system for data recording. The loading continued till failure, which was achieved when shear failure took place or when excessive deformations are reached.

Fig. 3: SIFCON casting procedure.

EXPERIMENTAL RESULTS AND DISCUSSION

Effect of mix type, group (A)
Figure 5 shows the cracking pattern for specimens B1 (SIFCON, V_f=4%), B2 (RC) and B3 (mortar). For SIFCON beam (B1), at early loading levels, about 70 kN, flexural cracks formed in the mid-span. Upon increasing the load, diagonal shear cracks appeared at one of the beam ends at a load level equal to 100 kN. The steel fibers in the slurry matrix reduced the rate of cracks widening. At higher levels of loading, the diagonal shear cracks propagated along the line connecting the support and the point of load application constituting a major shear crack at a load of 150 kN. Finally, the beam failed in compression at the tip of the shear crack (shear-compression failure) at a load of 257.5 kN. However the presence of fibers preserved the integrity of the beam and made the failure to occur in a less sudden fashion.

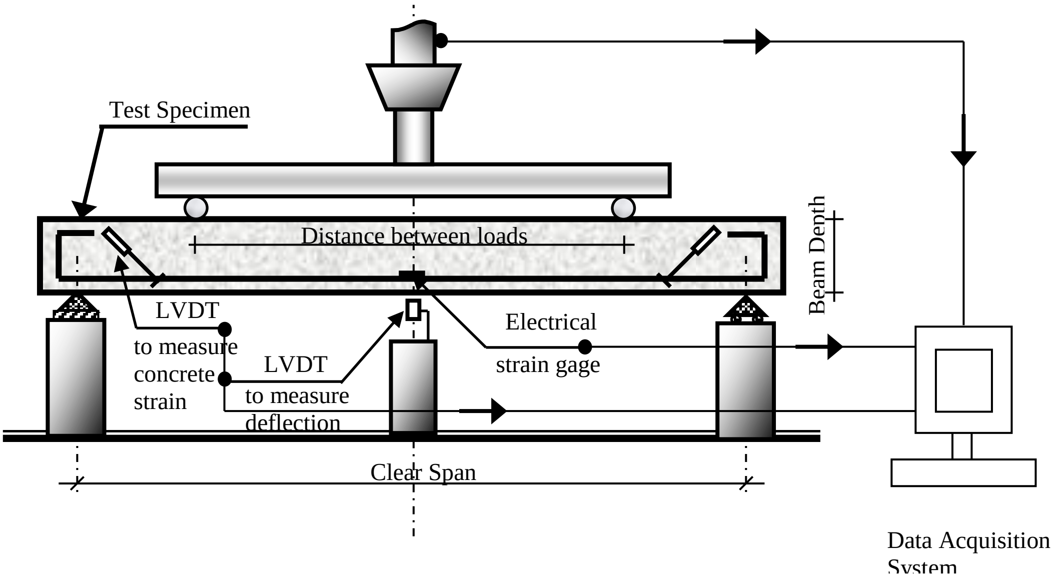


Fig. 4: Test set up and the location of the LVDT.

Generally speaking, the mortar specimen showed similar behavior to the normal concrete specimen. For both concrete and mortar a main diagonal crack was observed with few hair flexural cracks at the mid-span. The failure was sudden and brittle. The mortar specimen showed more brittle behavior where cover spalling took place as shown in Fig. 5.
The load-deflection curves for specimens B1, B2, and B3 are shown in Fig.6. The load deflection curves showed generally three main stages; linear one up to the first crack, nonlinear one and eventually a descending post-peak stage.
Figure 7 shows an increase in the shear diagonal cracking load of 18% and 42% for the SIFCON specimen over the RC and mortar ones. An increase in ultimate strength of 69% and 78% for the SIFCON specimen over the mortar and RC specimens, respectively, was observed as shown in Fig. 7. SIFCON beam failed at a load of 257 % of the diagonal cracking load, while concrete beam failed at only 170%. This implies the role of the fibers in bridging the diagonal crack and retarding failure.
Comparison of the peak deflections at ultimate load reveals that there was great improvement in the ductility due to high fiber volume in B1 than others as shown in Fig.8.
Shear failure in general does not show plastic deformations. Therefore, and in order to quantitatively measure the ductility of the test specimens, an arbitrary ductility index is used in which the ductility is calculated as the area under the load-deflection curve up to the ultimate load, i.e the strain energy of failure. As shown in Fig. 8, SIFCON beam showed a ductility of about three times the mortar and RC beams.

(B1) SIFCON

(B2) RC

(B3) Mortar

Fig. 5: Cracking pattern at failure.

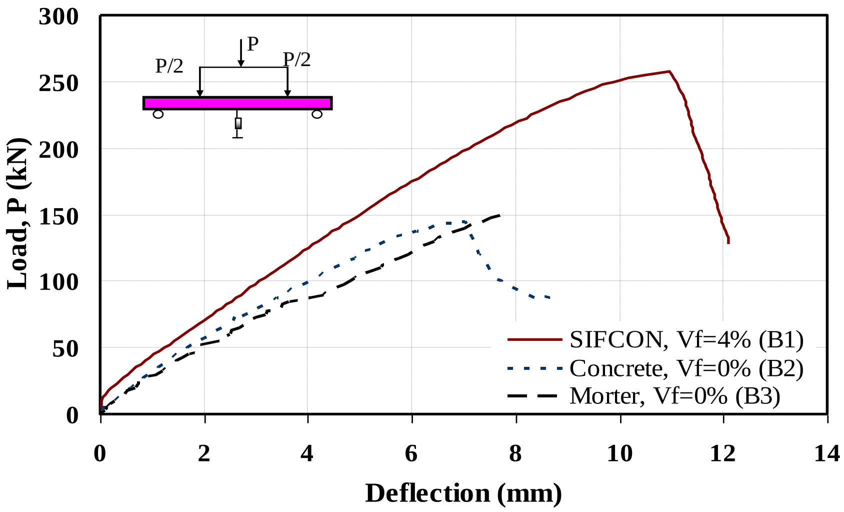


Fig. 6 : Load-deflection curve for group (A).

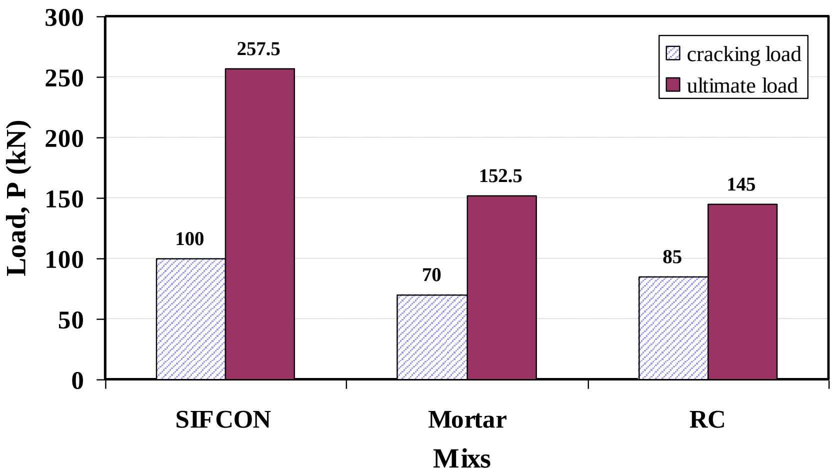


Fig. 7: Comparison between diagonal cracking and ultimate loads for group (A).

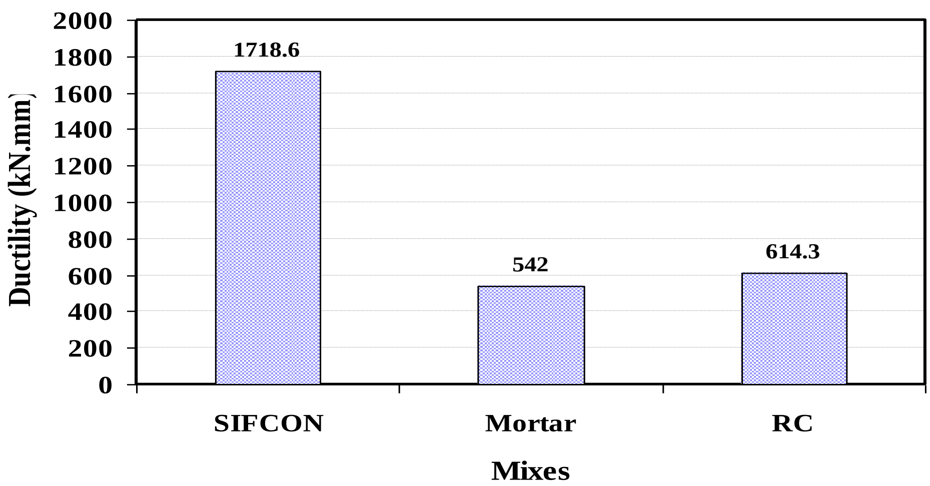


Fig. 8: Comparison between ductility for group (A)

Effect of Fiber Shape, group (B)

For all types of fibers the beam cracking pattern was almost the same. The main diagonal crack starts from the support to the loading point. Hair cracks adjacent to the main one varied from one beam to the other based on the fiber type. For hooked-end shape, multi diagonal hair cracks were observed, and eventually merged together to one major crack at failure as shown in Fig. 9. All beams failed in a shear-compression mode. Beam reinforced with corrugated fiber had the same behavior of beam reinforced with hooked-end type but with less ultimate load. For straight type fewer hair cracks occurred compared to the deformed types. The load-deflection curves for these specimens are shown in Fig.10. It can be seen that all beams had linear behavior from initial loading up to the occurrence of the first diagonal crack. After formation of the cracks, all the beams showed non-linear load-deflection characteristics that are eventually followed by a descending branch. Figure 11 shows that the hooked-end and corrugated types have higher cracking load than straight type by 33%, 20% respectively. As for ultimate load, the corrugated and straight types almost have the same load while the hooked-end is higher by 17% and 14%, respectively. The behavior shows also that the beam reinforced with hooked-end type is more ductile than other two types. Using hooked-end fibers increases the beam ductility by 15%, 3% compared to corrugated and straight types, respectively as shown in Fig.12.

(B1) Hooked-end (B5) corrugated (B4) straight

Fig. 9: Cracking pattern at failure.

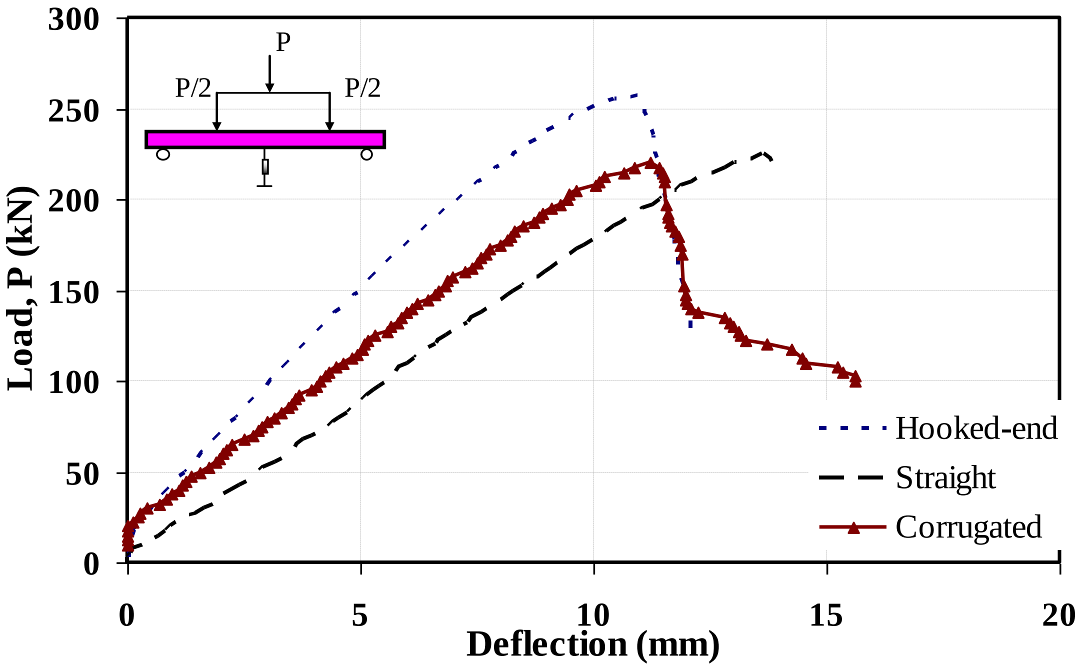


Fig. 10: Load-deflection curve for group (B).

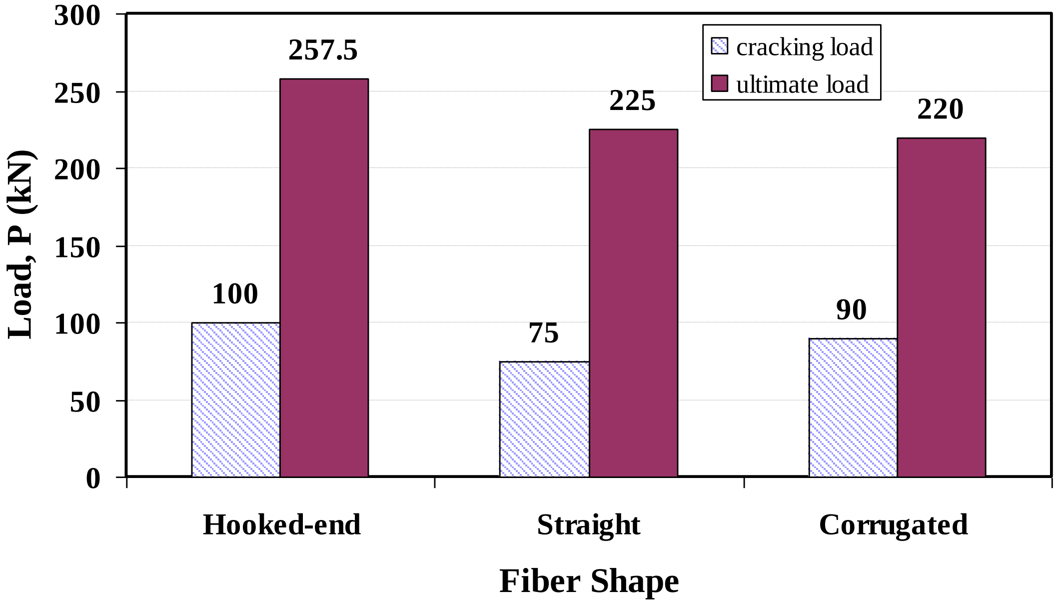


Fig. 11: Comparison between diagonal cracking and ultimate loads for group (B).

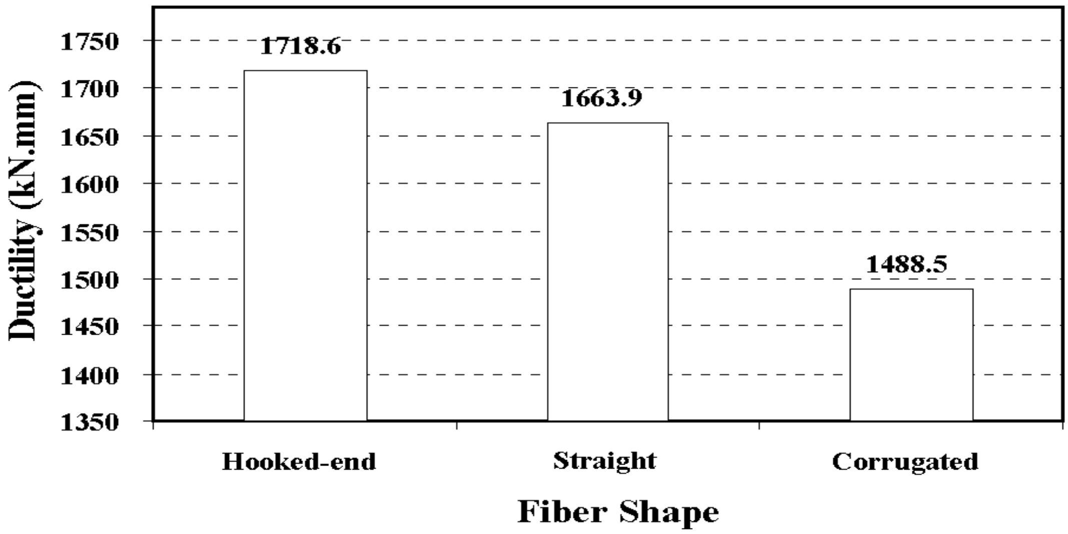


Fig. 12: Comparison between ductility for group (B).

Effect of Shear Span-Effective Depth Ratio, *a/d*, group (C)

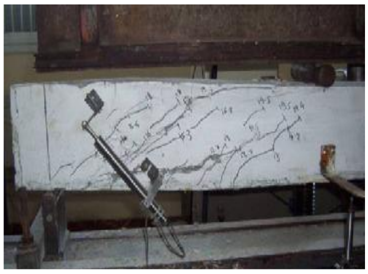
For *a/d* = 3, and 4 diagonal cracks had generally initiated in the shear span at a depth of approximately two-thirds of the total depth of the beam. Fibers bridged those cracks which propagated upon cracks widening. This was evidenced by the propagation of cracks, parallel to the beam axis along the main tensile reinforcement as shown in Fig. 13. These cracks continued to extend until splitting failure occurred (shear-tension failure). On the other hand, in specimen with *a/d* = 2, the inclined cracks were observed in the shear zone. These cracks extended diagonally from the support directly towards the point of applied load (shear-compression failure).

As shown in Figure 14, the ultimate deflection increased by 29%, 40% with increasing shear span to effective depth ratio from 2 to 3 and 4, respectively. As shown in Figure 15, both diagonal cracking load and ultimate load decreased with the increase of *a/d*. The decrease in diagonal cracking load was 10% and 30% for *a/d* = 3 and 4 compared to *a/d* = 2, respectively. The decrease in ultimate load was 10% and 24% for *a/d* = 3.0 and 4.0 compared to *a/d* = 2.0.

It was observed that, the ductility increased by increasing *a/d* as shown in Fig. 16. For beam with *a/d* = 2, ductility is less by 23.59%, 31.15% from beams with *a/d* = 3.0 and 4.0 respectively.

(B1) *a/d* = 2

(B6) *a/d* = 3



(B7) *a/d* = 4

Fig. 13: Cracking pattern at failure

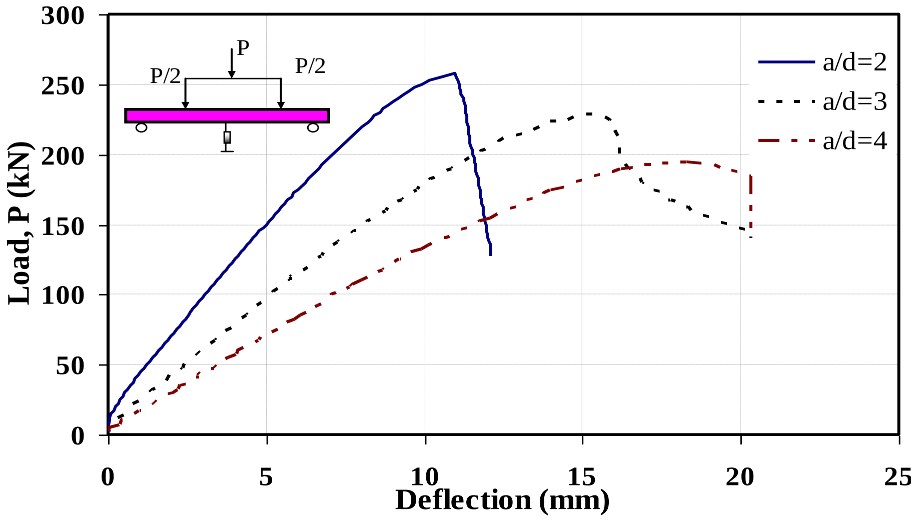


Fig. 14: Load-deflection curve for group (C)

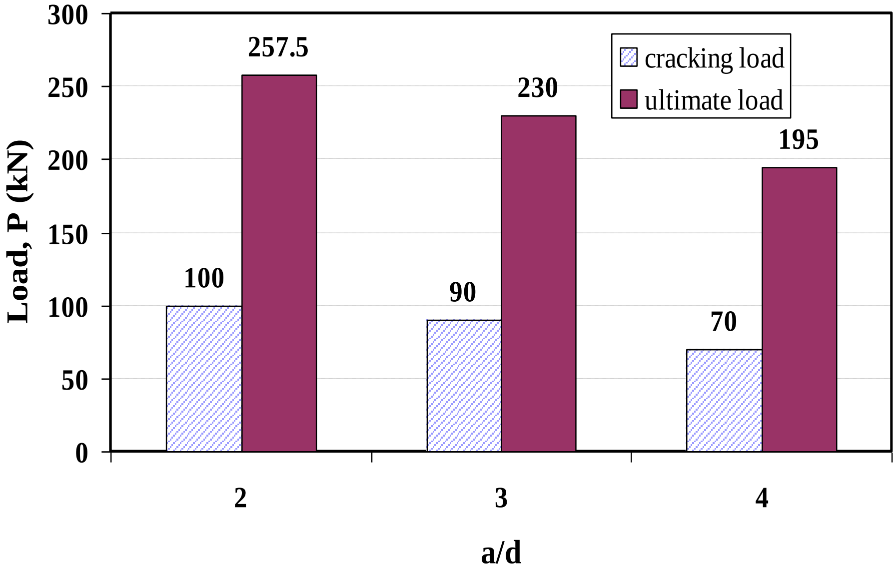


Fig. 15: Comparison between diagonal cracking and ultimate loads for group (C)

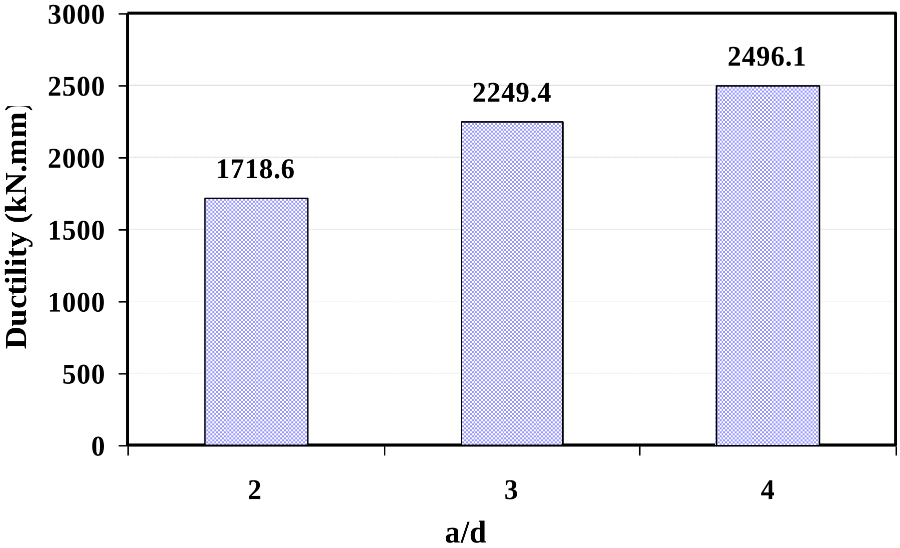


Fig. 16: Comparison between ductility for group (C)

Effect of Fiber Volume, V_f, group (D)

In SIFCON beams, an increase in fiber volume to 5.5, 8, and 10% lead to change the failure mode to a combination of shear-tension and shear-compression. In such failure pattern, multiple diagonal shear cracks and cracks parallel to the longitudinal steel formed, prior to failure. The shear cracks were spaced more closely as the volume of fibers increased as shown in Fig. 17.

Figure 18 shows the effect of fiber volume on the load-deflection.

In general, both ultimate strength and ductility increased with increase in fiber volume.

Figure 19 shows the effect of V_f on the cracking load, and the ultimate load, where it can be seen that both the cracking load and the ultimate load increased proportionally by increasing fiber volume. For V_f=10% the ultimate load almost reached three time that of the RC specimen.

Figure 20 shows the the effect of V_f on the ductility, where it can be seen that the beam ductility greatly increased by increasing fiber volume.

The ductility enhancement when fiber volume increased from 1% to 4% was 90.5%. For fiber volume increase from 4% to 5.5%, 8%, and 10% the ductility increased by 66%, 96%, and 201%, respectively. SIFCON with V_f=10% showed a ductility of almost eight times the RC specimen.

(B1) (V_f = 4%) (B10) (V_f = 8%) (B11) (V_f = 10%)

Fig. 17: Cracking pattern at failure.

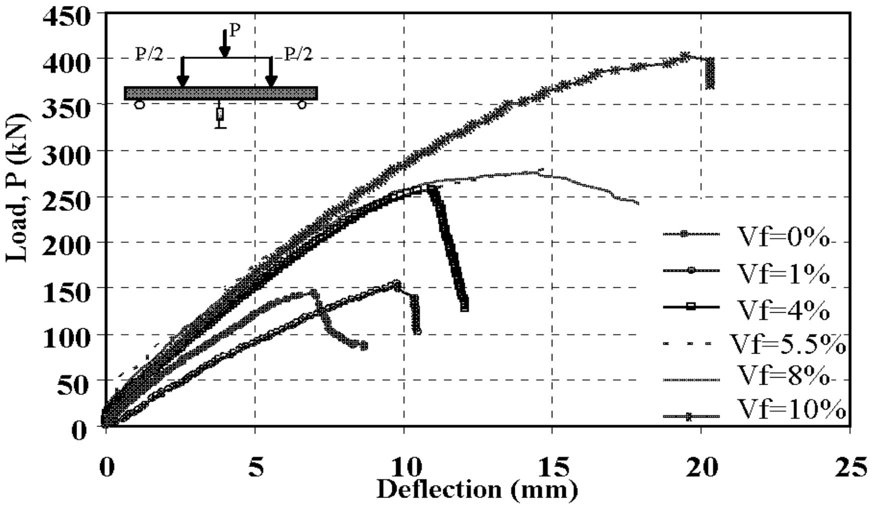


Fig. 18: Load-deflection curve for group (D).

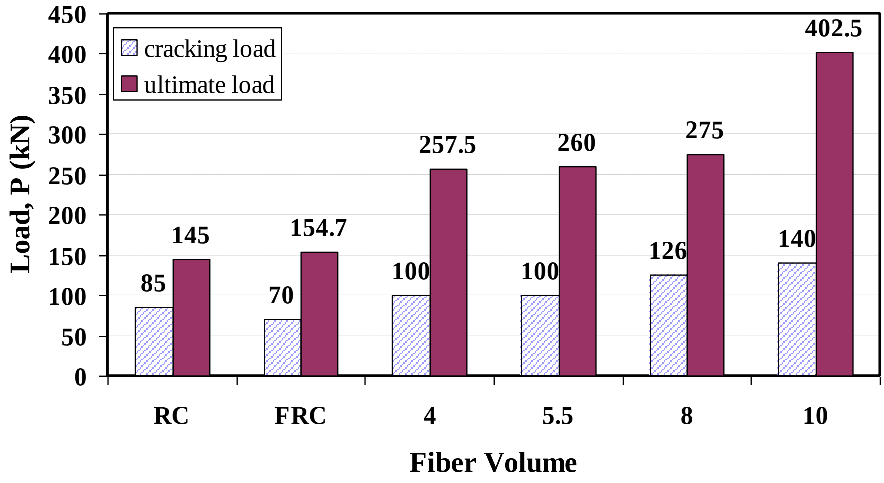


Fig. 19: Comparison between diagonal cracking and ultimate loads for group (D).

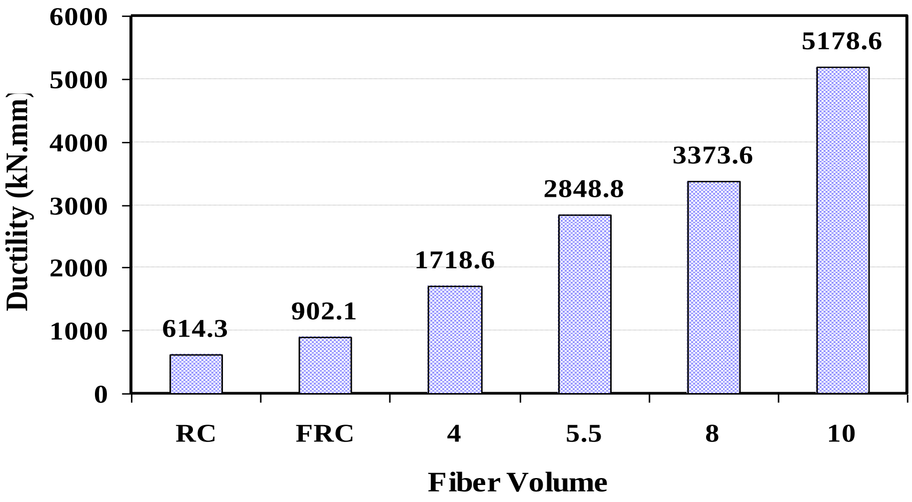


Fig. 20: Comparison between ductility for group (D)

Size Effect, group (E)

In this group the size effect was studied where, three beams with different size but with the same beam length to depth ratio were used. All beams had similar crack pattern and failure (shear-compression) as shown in Fig. 21.

Figure 22 shows that the diagonal cracking shear stress is not much affected by beam size. Excluding specimen with *d*= 270 mm, the same observation for the ultimate shear strength was recorded as shown in Fig. 22. Beam with *d*= 270 mm was excluded as it experienced a premature failure.

Figure 23 shows that the beam size did not have large effect on SIFCON ultimate shear strength compared to ordinary concrete. As shown in Fig. 23, and after eliminating specimen with 270 mm depth due to its relatively odd results, the reduction in shear strength associated with the increase in size has a tendency less than that of normal concrete [15].

The size effect model for conventional concrete is adopted from [15] as follows;

$$v = \frac{A_2}{B_2 + 0.9d}\sqrt{f_c^{'}} \qquad (1)$$

where

$$\frac{A_2}{\rho^{0.33}} = 6.9agg + 125 \qquad (2)$$

$$B_2 = 560 + 27agg \qquad (3)$$

v is shear strength, f_c' is the compressive strength, *agg* is the maximum specified aggregate size of the concrete, and ρ is the reinforcement ratio.

(B1) d = 170mm

(B12) d = 270mm

(B13) d = 370mm

Fig. 21: Cracking pattern.

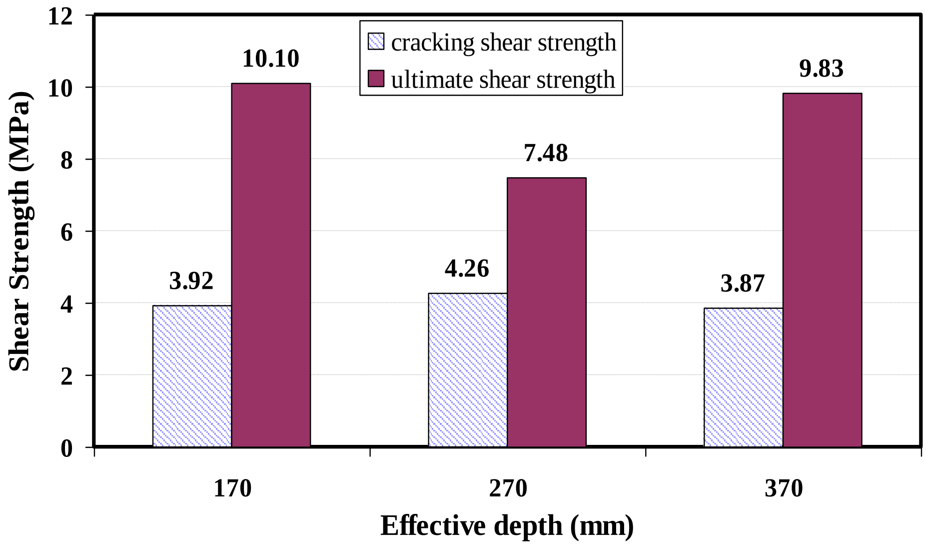


Fig.22: Comparison between diagonal cracking and ultimate shear strength for group (E).

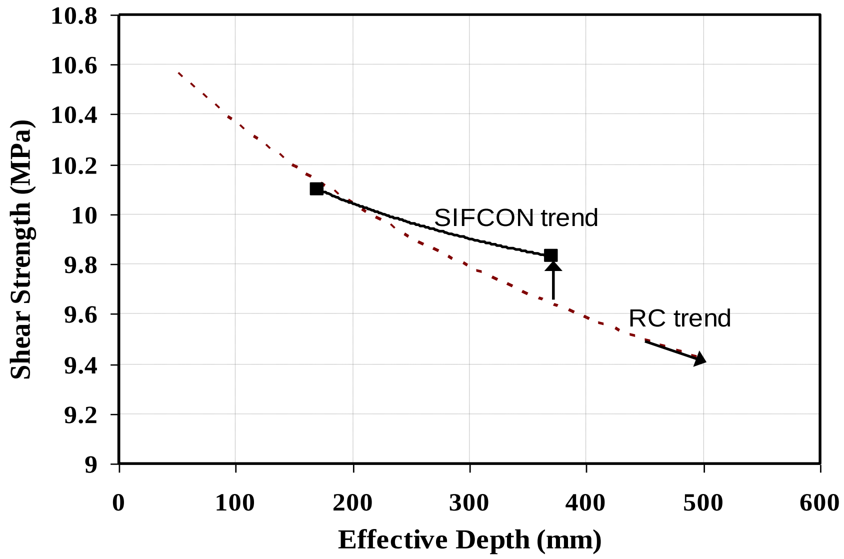


Fig. 23: Shear size effect for ordinary concrete and SIFCON beams

A SIMPLIFIED MECHANICAL MODEL FOR SHEAR STRENGTH OF SIFCON

A free body diagram of a part of the shear span of a simply supported fiber reinforced concrete beam is shown in Fig.24. The total shear force Q_u can be written as $Q_u = Q_c + Q_{ay} + Q_d + Q_f$ where Q_c is the shear force across the compression zone resisted by concrete, Q_{ay} is the aggregate interlocking force (vertical component), which is not valid in SIFCON case, Q_d is the dowel action force, and Q_f is the vertical component of the fiber pullout force along the inclined crack.

Considering the shear resistance of concrete without web reinforcement Q_c, the ultimate shear strength can be written as [16]

$$Q_c = \left(0.157\sqrt{f_c^{'}} + 17.2\rho\frac{Qd}{M}\right)bd \qquad (4)$$

where ρ is the reinforcement ratio, f'_c is the compressive strength of the concrete cylinder (MPa), d is the effective depth (mm), b is the width of the beam (mm), Q and M are the shear force and bending moment at section.

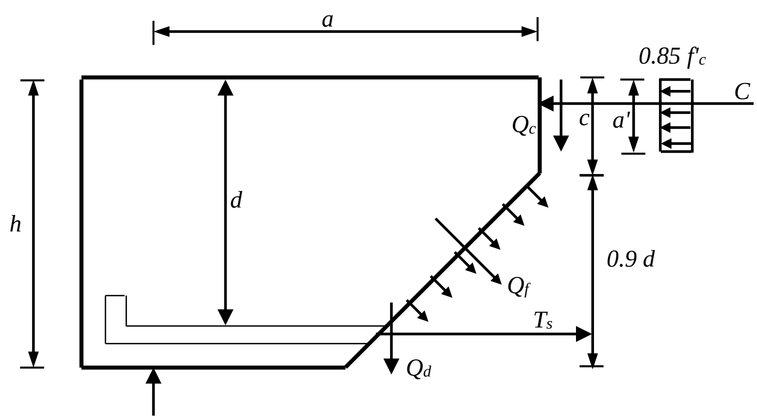


Fig. 24: Free body diagram for shear strength calculations.

The inclined angle of shear crack, β, is assumed equal to 45^0. The diagonal crack length L can be taken equal to $0.9d \tan 45^0$, where the vertical component of the diagonal crack assumed by $0.9d$. Thus the ultimate shear strength can be written as

$$Q_u = \left(0.157\sqrt{f_c^{'}} + 17.2\rho\frac{Qd}{M}\right)bd + 0.9 f_t^{*} bd \quad (5)$$

where f^*_t is the tensile strength of SIFCON.
For this study to get the fiber contribution first get the tensile strength of SIFCON. It is believed that state of stresses along shear crack is different from direct tension test. Therefore, the tensile strength of SIFCON is obtained from the results of the structural beams experiments as follows. First Eq. (5) is used to get (f_t^*) according to the experimental results. The ratio between the tensile strength at failure (f_t^*) and compressive strength (f_c) is obtained. The average ratio between the tensile strength and compressive strength of the mixes can be obtained as a function of volume fraction of fiber. Figure (25) shows the trend lines for the ratio and different volume fraction of fiber. The ratio can be estimated by equations as follows:

$$\frac{f_t^*}{f_c} = 0.1687 - 0.0003V_f + 0.0008V_f^2 \quad (6)$$

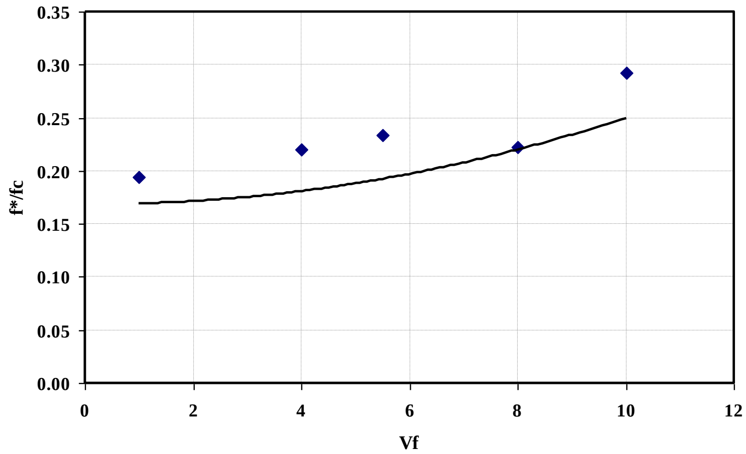


Fig. 25: Prediction of *f*/f_c*.

COMPARISON OF THE PREDICTED AND EXPERIMENTAL SHEAR STRENGTH

The observed ultimate shear strength Q_u, and the predicted ultimate shear strength obtained in this work are compared in Fig. 26 where it is believed that the predicted values are acceptable.

CONCLUSIONS

Based on the experimental results, the following conclusions could be obtained;

1. SIFCON beams showed superior behavior compared to RC and FRC beams where it becomes stronger and more ductile with multiple hair cracks produced in the shear zone
2. Increasing V_f leads to increasing both ductility and shear strength, where, an increase of up to 77.5% and 180 % of strength and ductility, respectively, over the conventional RC could be obtained.
3. Increasing V_f leads to a change in failure mode from shear-compression to shear-tension.
4. Hooked-end steel fiber was proven to be the most effective fiber type
5. Increasing shear span-to-depth ratios change the mode of failure to be combined between shear-compression and shear-tension failure, increases the beam ductility but decrease its shear strength.
6. The use of SIFCON in shear zone was proved to reduce the size effect on the shear strength of beams.
7. a simplified equation was deriven for predicting shear strength of SIFCON and proved to be accurate.

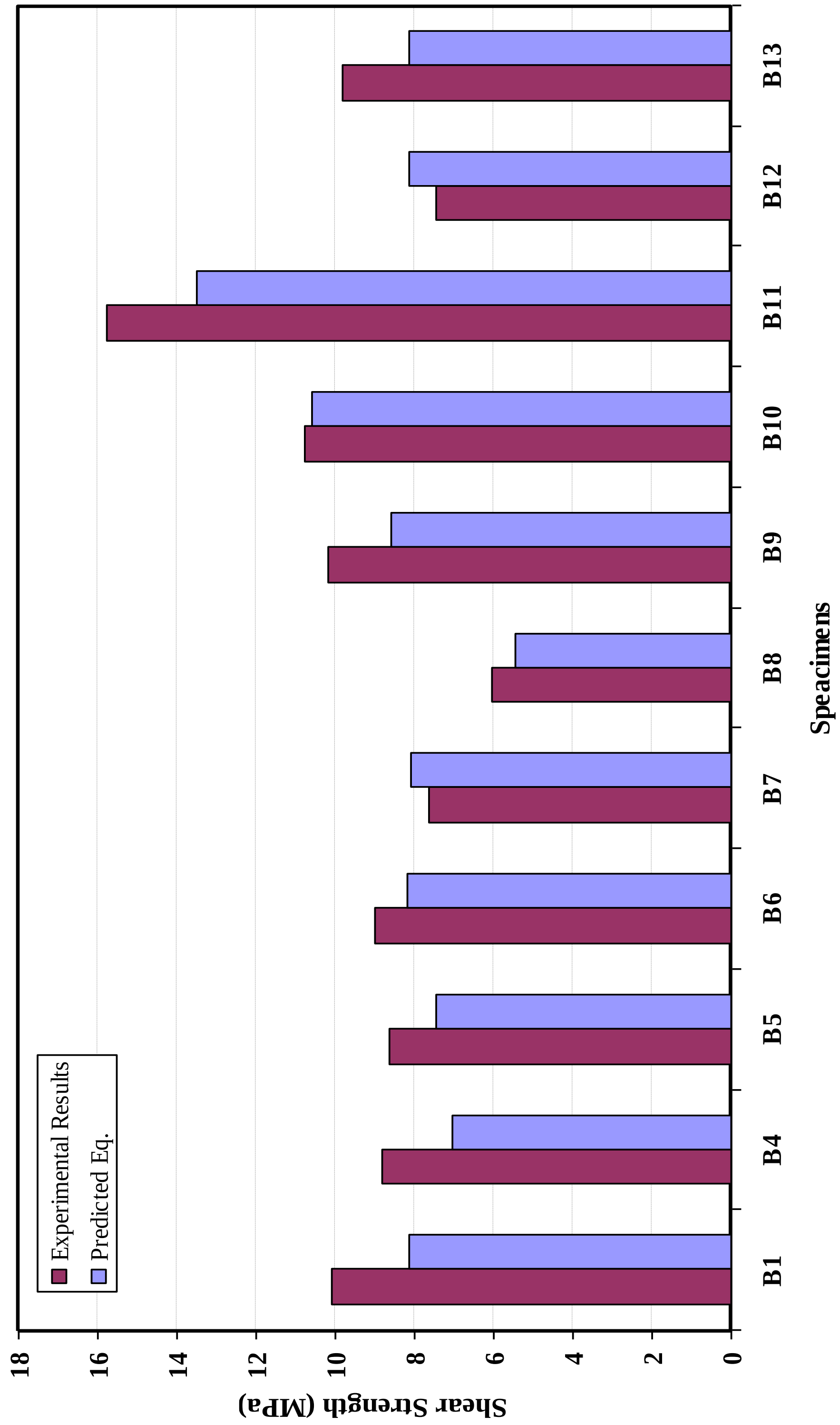


Fig.26: Comparison of experimental and predicted shear strength.

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URBAN DEVELOPMENT IN EAST PREFECTURE OF ATTICA, GREECE, SOCIAL AND ECONOMIC CHANGES DURING THE SUMMER OF THE YEAR 2004

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ABSTRACT

The aim of this study is to examine how urban development has affected the social and economic life in four specific semi rural regions, which are located in the east prefecture of Attica in Greece. These semi rural regions were affected significantly by social and economic changes, because of substantial new infrastructure development that took place related to a major urban plan development for Attica during the year of 2004. The effect of urban development on the characteristics of residents and local entrepreneurship along with land use changes was examined. The results showed that that there has been a substantial population increase and residential land has increased at the expense of farmland. In addition, relevant results were deducted from the logistic regression models.

INTRODUCTION

Central themes in structural analysis of a region include differentiation, centrality, and solidarity (Young, 1971). Differentiation of regions is the pattern of institutions in a town like schools, universities, hospitals, hotels, recreation facilities, etc. Young and others have found that there is a connection between the presence of various institutions and services in a region and they do not appear randomly. For instance, the presence of medical services in a region is linked to the presence of pool halls and hotels. Therefore, the differentiation of a region is the result of its makeup and not the result of individual actions. Also, centrality, describes the degree of connection of a region to the external world. The centrality of a region can range from the lowest level, as that of a settlement, to the highest level of centrality, as that of a metropolitan centre. While generally centrality increases as differentiation increases, these variables are separate dimensions of region structure and they can vary independently. Furthermore, solidarity is a system of values, ethics, standards, and guidelines that keep social life together. Solidarity can also range from low to high, and a minimum amount of solidarity is needed to keep together a region with a given amount of differentiation. Therefore, social structure and human agency play important roles in urban and rural regions development. Humans, like other beings, must compete for a spatial niche in the larger habitat. This competition gives rise to a struggle in which each type of land user ends up in the location to which the user is best adapted (Munroe et all, 2005; Vias et all, 2002; Theodoropoulos, 1999; MacCannell, 1978). In structuralism, social changes in urban regions are a naturally occurring process, but sustainable urban development is depended on several socio-economical policies such as compensation, health, environmental, infrastructure, housing, tourism services etc. For example, family planning is believed to be an important social policy that can determine development (Deng and Huang, 2004; Smith, 2003; Shiffman, 2002; Shiffman et all, 2002; Pobutsky, 2001). Also, economical measures for employment, income or production can affect sustainable development over time (Midgley et all, 2004; Vias et all, 2002; Simon, 1998). Furthermore, city planning measures which could be related to the region's habitants' demographic characteristics such as age,

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gender, education, culture etc. can affect the sustainable urban development (Azari and Pick 2005; Berdeguè et all 2001; May et all 2000; Blassingame, 1998; AtKisson, 1996, Burnette, 1996). During the last five years, substantial new infrastructure development took place in the east prefecture of Attica in Greece, which is accountable for changing the structure of the communities in this region from semi rural to semi urban. Differentiation, centrality and solidarity of this region are changing causing the adjustment of local life under the new circumstances. The most important structures that were developed and changed the character of the study region were a new high-speed road (Attiki Odos) that joined the city of Athens with this region, the new Athens International Airport, a health centre with urban character, modern athletic, stallion and gun shooting centres, new schools, renovation of existing parks and beaches and finally many social programs. In view of the fact that this region is now more accessible from the city of Athens, many people, who work in the city of Athens, choose and commute and live in this region, searching for better living conditions. The result is that there is an increasing demand for housing and the character of the land is changing from that of farms to residential regions. The increasing population and real estate without a specific plan for growth can have a negative effect on sustainable development. The aim of the present study is to investigate these changes and propose appropriate solutions in order to have sustainable development for a good quality urban life.

MATERIALS AND METHODS

A region with four municipalities located in the east prefecture of Attica in Greece was examined (Figure 1). The characteristics of this region based on the 2001 census (www.ypes.gr) were as follows: The municipality of Vari is a 2,500 years old city with a population of 10,998 people and an area of 22,189 sq.m., which is mostly semi rural. The municipality of Kalivia is a semi-mountainous area at an altitude of 110m above sea with a population of 12,202 people and an area of 70,415 sq.m. The municipality of Koropi has a population of 25,325 people and covers a rural area of 120,000 sq.m. Finally, the municipality of Markopoulo has a population of 15,605 people and covers 81,844 sq.m. Since ancient times the main production of this region was grapes and wine.

The statistical frame of the study region was based on a sample of 192 residents randomly selected in the four regions of east Attica. More specifically, 33 questionnaires were selected from Vari, 37 from Kalivia, 75 from Koropi and 47 from Markopoulo, which represents $3^{0}/_{00}$ of the population of each region. Data on the demographic, social and economic characteristics of residents along with their entrepreneurship preferences were collected through a questionnaire survey during the spring of 2005. The data collected were analysed by using descriptive statistics for calculating the means and standard deviations of continuous variables and the frequencies and percentages of discrete variables. Data elaboration and statistical analysis was performed using SPSS10, logistic regression models ($ln(p/1-p)=a+bX$) were used to explain the variation of local development, as well as resident's belief for sustainable development.

RESULTS

Descriptive and cross-tabulation statistical methods

Based on the data analysis of the 192 questionnaires the majority of responders were men (54%) and their age ranged from 29 to 38 years old (33%). Most of the responders were married (65%) and the average number of children per married responder was two. Most of the individuals were self employed (38%), or employed in the private sector (12%) or they were housekeepers and retirees (20%). Their education level was mostly high school (31%), while 28% had finished University. Also, according to the analysis of the data on infrastructure most of the responders (89%) were living in houses in legal residential regions. Most of the individuals (54%) had connection to Internet. Only 16% of the total study region had a sewage system but there were water supply plumbing facilities everywhere. Also, 46% of the responders replied that there were playgrounds close to their houses, 51% that there was a health centre in less than 1 klm from their houses, and 22,9% that there were at least 3 pharmacies close to their house. Most of the responders (99%) replied that there was a centre for elders in their region, (77%) that there was a program through the Municipality for help at home, but 60% of the responders replied that there is some or no access to public places for people with special needs. Most (91%) of the residents owned a house or apartment and 66% leave close to their work. Real estate construction has increased in the study region (44%) and mainly new construction is happening where farms used to be before the 2004.

Binary logistic regressions

Initially, a binary logistic regression was analyzed to investigate the direct effects of local characteristics variables on the local development of the study region. The dependent variable was measured based on the sample's responses to a 2-point scale: yes, no to the following statement: "If local development have been accoplished". The independent variables included private investment, and local characteristics.

The equation for the effects of local characteristics variables on the local development is the following:

*ln Local development t= 3,606*** + 1,516****** Private investment – 1,108****** Population density – 0,418 Environment deterioration + 0,817*** City planning measures + 0,258 Internet access – 0,087 Hospital vicinity.*

Where:

*(*** P-value<0.01* and * *P-value<0.10)*

Table 1 : List of variables used in the local development logistic regression model

Variable	Type	Description
Local development	Nominal	1 if respondent argues that local development have been accoplished; 0 otherwise
Private investment	Nominal	1 1 if there is private investment in the region; 0 otherwise
Population density	Ordinal	1=Very crowded, 2=less crowded, 3=not at all
Environment deterioration	Ordinal	1=Very, 2=less, 3=enough, 4=not at all
City planning measures	Nominal	1 for regions where there is an urban planning; 0 otherwise
Internet access	Nominal	1 if there is Internet access; 0 otherwise
Hospital vicinity	Scale	How close is a hospital

The analysis of the first logistic regression model showed that 81% of the variance of *Local development* was significantly explained by the local characteristics variables. Specifically, *Local development* was significantly associated with *Private investment*, *Population density* (p<0,01) and *City planning measures* (p<0,10). *Private investment* and *City planning measures* were associated with 1,516 and 0,817 increase of *Local development* respectively. These results suggest that as the *Private investment* and *City planning measures* increase the *Local development* increases. On the other hand, *Population density* was associated with 1,108 decrease of *Local development*, which suggests that as the *Population density* increases the *Local development* decreases.

A second binary logistic regression was analyzed to investigate the direct effects of local characteristics variables on the sustainable development of the studying region. It is believed that a city is more sustainable if there are social programs, chances for employment and adequate number of public services. The dependent variable was measured based on the sample's responses to a 2-point scale: yes, no to the following statement: "If your region is a

sustainable region". The independent variables included amount of homeless, social programs and public services.
The equation for the effects of local characteristics variables on the sustainable development is the following:

ln Sustainable development = - 5,308** – 2,558*** Homeless + 1,759** Help at home + 1,277 Playgrounds + 3,119** Shelter for elder+ 1,586** Chances for employment + 0,686 Public transportation+ 1,674 Plumbing facilities.
Where:
(*** P-value<0.01 and ** P-value<0.05)

Table 2: List of variables used in the sustainable development logistic regression model

Variable	Type	Description
Sustainable development	Nominal	1 for sustainable city; 0 otherwise
Homeless	Nominal	1 if there are homeless in the region; 0 otherwise
Help at home	Nominal	1 if there is the social programme for help at home; 0 otherwise
Playgrounds	Nominal	1 if there are many/enough playgrounds; 0 otherwise
Shelter for elder	Nominal	1 if there is a shelter for elder; 0 otherwise
Chances for employment	Nominal	1 if there are lot of/ adequate chances for employment; 0 otherwise
Public Transportation	Ordinal	1=there is public transportation accessibility and adequate infrastructure, 2=some, 3=none
Plumbing facilities	Nominal	1=if the plumbing facilities are adequate; 0 otherwise

The analysis of the second logistic regression model showed that 80% of the variance of Sustainable development was significantly explained by the local characteristics variables. Specifically, Sustainable development was significantly associated with Homeless (p<0,01) and Help at home, Shelter for elder and Chances for employment (p<0,05). Help at home, Shelter for elder and Chances for employment were associated with 1,759, 3,119 and 1,586 increase of Sustainable development respectively. These results suggest that as the Help at home, Shelter for elder and Chances for employment increase the Sustainable development increases. On the other hand, Homeless was associated with 2,558 decrease of Sustainable development, which suggests that as the number of Homeless increases the Sustainable development decreases.

Comparison Of The Demographic, Residential And Development Data In The Four Municipalities In The Study Region

Table 3 represents percentages of the responses to questions related to demographic and residential characteristics of the four municipalities in the study region on a three score scale

(3=high, 2=medium, 1=low). The table shows that in the study region there is the belief from the residents that there is a high percent of communal recreational facilities, building new residents and monthly income, while there is a medium percent of number of rooms per person and distance from a hospital and a low percent of distance from work and level of unemployment.

Table 3: Percentages of the responses to questions related to demographic And residential characteristics

Variables	High %	Med. %	Low %	n	M	s.d.
Facilities in the region: garden, pool, sport grounds (High=many facilities, Medium=enough facilities, Low=very few of no facilities)	38	33.9	28.1	192	2.10	0.81
Distance from work (High=the job is in the community Medium=the job is close to the residence community Low=the job is far from the community)	21.3	12.9	65.8	155	1.55	0.82
Distance from hospital (High=<2km Medium=2-5km High=>5km)	22.9	66.1	10.9	192	2.12	0.57
Construction of new residences (High=there are a lot of new constructions Medium=there are few new constructions Low=there some or no new constructions)	44.3	40.3	14.8	192	2.30	0.71
Satisfaction from income (High=very satisfied Medium=satisfied enough Low=not satisfied)	44.9	40.3	14.8	176	2.30	0.71
Years of unemployment High=more than two years Medium=six months up to two years Low=up to six months)	10.4	17.2	72.4	192	1.37	0.66

n: number of responses
M: Sum of scores/n ($1 \leq M \leq 3$; $M \geq 1.5$: Medium and high level)
s.d.: standard deviation

Table 4 shows the percentages of the responses to questions related to residential characteristics of four municipalities in the study region on a three score scale (1=disagree, 2=fairly agree, 3=strongly agree). It is observed that there is a strong percent of agreement for a sustainable city, local development and that there is illegal construction. There is a fairly percent of agreement for satisfaction from infrastructure, green regions and overpopulated regions. Finally, there is a low percent of agreement from job satisfaction.

Table 4: Percentages of the responses to questions related to residential characteristics of the four municipalities in the study region

Variables	Strongly agree %	Fairly agree %	Disagree %	N	M	s.d.
Job satisfaction	32.7	27.2	40.1	162	2.07	0.85
Satisfaction from roads	22.4	49.5	28.1	192	2.06	0.71
Adequacy of mass transportation	33.3	33.3	33.3	192	2.00	0.82
Satisfaction from green regions	34.9	63.5	1.6	192	2.33	0.50
Sustainable city	89.1	9.9	1	192	2.88	0.36
Illegal construction	46.4	40.1	13.5	192	2.33	0.70
Local development	81.8	17.1	0.5	192	2.81	0.40
Over-populated region	13.5	67.5	19.3	192	1.94	0.57

n: number of responses
M: Sum of scores/n ($1 \leq M \leq 3$; $M \geq 1.5$: Medium and high level)
s.d.: standard deviation

Fig 1: Map of Attica, Greece (Study areas in square).

CONCLUSIONS

Various factors have been identified in previous studies as important for sustainable urban development. Such factors are several socio-economical policies such as compensation, health, environmental, infrastructure, housing, tourism services etc., as well as, the city's habitants' demographic characteristics such as age, gender, education, culture can affect sustainable development.

In the present study, factors that were found to be affecting urban sustainable development were the population density, private investment and city planning measures. Furthermore, it was found that the number of homeless people, the access to social services and chances for employment affects urban sustainable development.

The results showed that differentiation, centrality and solidarity of the studying region are changing causing an adjustment of life under some new circumstances in order to have sustainable development for a good quality urban life. More specifically, the population boost in the municipalities of Vari, Kalivia, Koropi and Markopoulo has increased social and cultural diversity. We found that increasing private investment led to the founding of new stores and enterprises, which in turn led to the expansion of public and private services like banks, health centres, coffee shops, restaurants, infrastructure and transportation. All the above and in addition, the new high-speed road that joined the city of Athens with these regions caused more people to seek employment or just to reside in the region, where living condition are better even if they must continue to work in the city. On the other hand this progress in the region attracted the homeless people since it is easier than before to find food and possible shelters. Therefore, socio-economic and demographic changes have increased the social and cultural diversity of these regions. Since the demand for housing has increased, local people sell their farms as valuable residential land and earn money that can be invested or consumed, which causes a new lifestyle and brings new values to the local population. As population and service sectors grow significantly at the expense of agriculture there is environmental pollution since the infrastructure for sewer, plumbing, and litter recycling is not adequate to lever this extended increase. These policies must suggest specific constraints for population increase in order to avoid environmental deterioration, to specify the residential region and to promote vocational programs in order to increase employment, to encourage environmentally friendly activities, to introduce social programs that will help homeless and special needs persons and the most important to create the necessary infrastructure in the region.

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LOW-COST HOUSING IN MEGACITIES OF THE DEVELOPING WORLD: A CHALLENGE FOR THE 21st CENTURY

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ABSTRACT

This paper discusses low-cost housing in the context of megacities, taking into account the increasing population, scarcity of land, financing, and technological aspects. It takes a global view of the housing problem focusing on the developing countries, examines case histories based on available data and offers recommendations about how low-cost housing can be financed and built economically. Low-cost housing must be given priority by the developing countries to offer their citizens self-sufficiency, pride and well-being. Finding a successful strategy to address this festering issue is a challenge for policy-makers, planners, architects, engineers, sociologists and economists. Only through a multi-disciplinary and collaborative approach by all concerned, an optimum solution to low-cost housing problem can be achieved that will make large cities sustainable and livable for people of all economic classes. Recommendations for the successful low-cost housing are presented in terms of initial building cost, sustainable design aspects, affordable methods for construction, and choice of materials, as well as over the building's life-cycle.

Keywords: low-cost housing, megacities, financing, technology.

INTRODUCTION

In 1975, when the United Nations Population Division (UNPD) created the term "megacity," with a population of 10 million or more, there were only five in the entire world. With the emergence of several megacities in the past three decades they have become the homes of a large percentage of the world population. Megacities are present in both developed and developing countries: Tokyo, Mexico City, Seoul, New York, Sao Paolo, Cairo, Mumbai, Delhi, Calcutta, Dhaka, Jakarta, and London, to name a few.

Pollution, housing, crime, infrastructure and energy consumption are universal challenges in large cities. According to the UNPD, by the year 2050, Asia will account for 61 percent of the world's population. Most of Asia, aside from Japan, belongs to the developing world; similar is the case with Cairo and Lagos in Africa, for example. There is a trend in population growth in the developing world that is severely affecting the megacities. People move from rural areas and smaller towns to the megacities seeking employment opportunities, aggravating the existing problems, particularly in the housing sector. It must be recognized that in developing countries technology and infrastructure will not be able to outpace the expansions in urban centers and population growth. The recent movement for sustainable development has set a new trend. If the needs of future generations are not considered now, megacities are almost certainly heading for failure.

The uncontrolled migration to megacities creates an acute housing shortage for the middle class with limited fixed income and leads to undesirable slum-dwellings for the poor and the underclass. Because of the scarcity of land, and to minimize urban sprawl, the solution to this problem often leads to the construction of low-cost, affordable high-rise buildings. Singapore,

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although not a megacity, but with megacity-like characteristics by virtue of being the most crowded country of the world with a population density of 4,400 people per square meter, has succeeded in resolving the problem with creative strategies of financing and placing most citizens in high-rise apartments. Singaporeans for the most part own their homes and apartments. A similar concept encouraging “ownership” can be extended to other cities and even to their slum-dwellers by providing them with incentives and subsidies by the government. Similarly, innovations in technology and materials can make houses in which the world’s poorest live cheaper and better.

CONCEPTS AND DEFINITIONS OF LOW-COST HOUSING

In the U.S., the federal government regulates the availability of affordable housing through Department of Housing and Urban Development (HUD), which defines affordable housing as that on which the occupant pays no more than 30 percent of gross income for housing costs, including utilities [1]. This definition is widely used by state and local governments, as well as non-profit organizations, developers, architects, lenders, etc. This working definition is solely based upon occupants’ social characteristics or income, and architectural, planning, and other aspects are not considered in this formula. In that sense, the concept of affordability directly relates to the financial attributes of the residents, and the initial building construction cost and operational costs are not considered. The architectural aspects, which greatly affect the affordability, are rather secondary. From the architectural perspective, this definition is missing some integral points related to the design, environmental, construction and other issues.

Sam Davis offers a different perspective on affordable housing, and he argues that the term is the most recent incarnation of terms such as low-income housing, social housing and subsidized housing [2]. Affordable housing receives direct or indirect financial assistance and is developed outside the purely market-rate private system through a combination of private and public mechanisms. In that sense, affordable housing is mainly governed by housing policies and the government. Davis also argues that the regulatory mechanisms and community involvement in all stages of the development is crucial for such projects. The designers must know about the behavior of the people who use the structures and spaces they create. Davis declares that in the design of affordable housing “the architect must imbue the work with a spirit that elevates the structure from a building to a home” [2].

Peter Calthrope does not offer a specific working definition of affordable housing, but specifies the design guidelines for the design of healthy, transit-oriented developments [3]. “Transit-Oriented Development (TOD) is a mixed-use community within an average 610-meter (2,000-foot) walking distance of a transit stop and core commercial area. TODs mix residential, retail, office, open space, and public uses in a walkable environment, making it convenient for residents and employees to travel by transit, bicycle, foot, or car” [3]. The main reasoning behind this concept is the fact that by integrating commercial and residential spaces it is possible to lower the transportation costs, thus creating affordable housing. In that sense, the outside factors are considered in this model. A mix of housing densities, ownership patterns, price, and building types is preferable in TOD.

U.S. Department of Energy offers more technical definition of affordable housing, mainly concentrating on the operational and energy costs of housing rather than the initial building costs, “Over the years, we in the home field have learned that housing for low- and moderate residential projects can be made significantly more affordable by improving its energy efficiency” [4]. The U.S. Department of Energy is in the process of developing models for zero-energy housing. “The first attempt at low first-cost, near zero-energy houses (ZEH) was in June 2002 in Tennessee as part of the U.S. Department of Energy's Building Technologies Program. The goal is to create technologies for low-cost net-zero energy residences by 2020” [4]. The various energy-efficient features of these sustainable residences include structural insulated panel (SIP) structure, photovoltaic solar power, geothermal pumps, and super-insulated buildings.

Jon Lang argues that the housing projects designed in the 1940s, 1950s and 1960s followed design ideas that had not primarily focused on the users and their needs, many being total-design on large-scale sites. “We have learned what, with the wisdom of hindsight, is common sense: the design of housing has to be congruent with the lifestyles of the inhabitants of the housing, recognizing the problems that they face” [5]. In that sense, the prime concern in design must be the dignity of people, and Lang offers few strategies in articulating such designs:

1. The goal of the design is to strive for ambiguity in built form
2. The designs must be open-ended
3. The requirements of population with the more basic needs should be met first
4. The new environments must be designed to be barrier-free—socially and physically; existing environments should be retooled to be so.

All of these perspectives offer different views towards the definition and the meaning of affordable housing. This paper is not praising one over the others, but is acknowledging different standpoints and understandings. It is necessary to present these varying definitions in order to fully understand the scope of the term “affordability” or for that matter “low cost” as it relates to housing today, especially in the context of megacities in the developing world.

In the case of developing countries, the direct correlation between the definition of low-cost housing and the household income applies even less. First of all, it is necessary to understand the difference between the concepts of “low-income” housing and “affordable” housing concepts. It is not clear when the term “affordable” substituted “low-income”, but in the last two decades the “affordability” concept became widely used by the design professionals, planners, and policy makers. It is closely related to the rising cost of housing, not proportional to the inflation rates. Pyatok states, “When architects, planners and housing designers read these numbers (rising cost of housing), we cannot help but dream again about some technical solutions to this worsening problem like higher densities to lower soft costs and land costs per unit or prefabricated elements to speed the process of erection while relying more on cheaper factory labor, or smaller more efficient dwellings to save materials, or better insulation and orientation to conserve energy, etc. But in a booming economy with severe labor shortages and high inflation in the construction industry along with runaway land speculation, even major construction breakthroughs and technical design innovations have limited ability in lowering costs” [6]. The differences between low-cost housing and quality housing should not be so much in their performance level as they are in different economic models catering to the financial needs of users of different economic classes.

Low-cost housing in the developing world should take into account many different aspects, such as the initial building cost, available materials and construction methods, efficiency, and operating costs. Also, megacities present further considerations, such as the urban density, availability of land, and rising population that need to be addressed for the design of low-cost housing.

MEGACITIES AND LOW-COST HOUSING

The challenge in designing the new cities of today and the future, as well as developing the old cities, is accommodation of large population while creating a higher quality of urban life. Planning for the increase in population is an intricate task, having in mind that problems such as pollution, urban sprawl and exploitation of natural resources are non-reversible outcomes of poor planning. It has been proven many times over that a city will grow whether or not it is possible. Growth is not something that can be stopped; only it can be contained and influenced. Despite the global and local effects, the cities will always thrive and continue to exist.

There is a trend in population growth in developing countries that is severely affecting the world population. Due to social, cultural and economic factors growth in developing countries is much more rapid than in developed countries. The UNPD has predicted that the urban population will grow by 75 percent by the year 2050. This means that not only is the world population growing but also that the percentage of people living in the city is increasing even faster. Population issues are not characteristic of only developing countries—they are also present in some of the developed countries, although to a lower degree.

Politics will play a major role in the success or failure of cities of the future. It is governments today, unlike in the past, which bring economic possibilities to a city. In the past cities were centers of economy because of their location. Though location still serves as a factor in industrial economies it is not as important as it has been in the past and for technological economies location is hardly a factor. It is impossible for a megacity of the future to become a utopian example without the proper resources to support the many necessary components.

The megacities of the developing world suffer from high-population density, poverty and limited resources [7]. The problems of megacities include:

- Explosive population growth

- Increase in poverty
- Massive infrastructure deficits, in particular transportation
- Pressure on land and housing
- Environmental concerns, such as contaminated water and air pollution
- Disease and high death-rates
- Capital scarcity

A crucial component of megacities is housing, and the issue of low-cost housing in the megacities of the developing world poses serious problems for the governments and the population. Significant considerations must be given to this problem, in terms of planning, financing, and technological aspects, which are unquestionably intertwined (Fig. 1). Also, there are limits to the rate at which the human population can draw from planetary sources and limits to the rate at which planetary sinks can absorb the wastes and pollution created by people. Sustainability involves staying within these limits on a local, regional and global scale. Megacities, as cities of the futures, should be designed as environmentally conscious to preserve the natural resources and provide for smart, sustainable growth. In that sense, low-cost housing should be carefully designed to address the affordability in terms of initial costs, as well as over the life-cycle of the buildings.

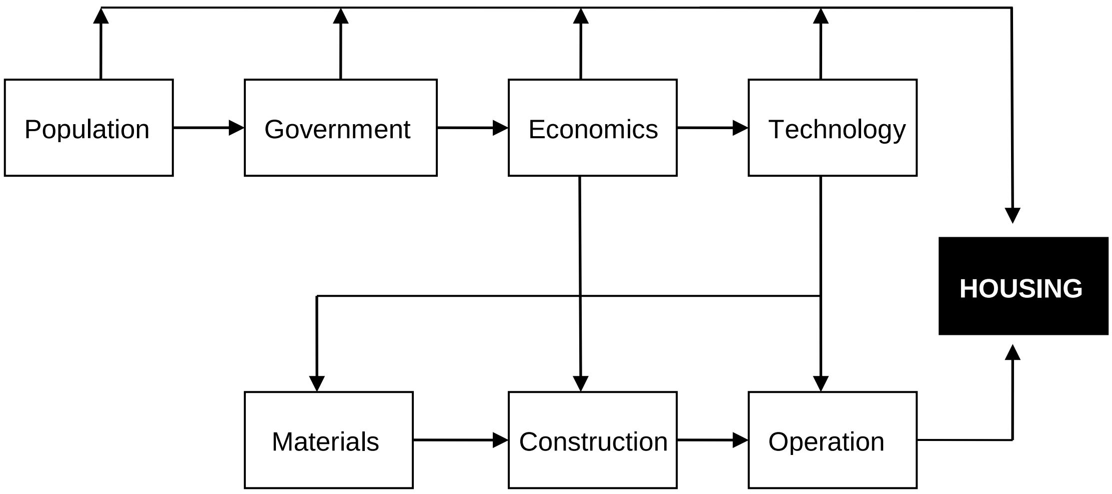

Fig. 1: Considerations for the design of low-cost housing

The focus of this paper is the housing problem. It must be stated that all of the problems listed above are inter-related and pose serious issues for the megacities of the developing world. Several examples will be presented, outlining the major difficulties and strategies for development of low-cost housing.

Singapore

Singapore is an island city-state with a population of 4.5 million. The housing problems started at the beginning of the last century, with drastic population increase due to immigration and extremely high birth-rates [8]. The first efforts of public housing begun as early as 1927 with the creation of the Singapore Improvement Trust (SIT), however, this organization had limited financial and legal powers. During the 1950s, widespread substandard conditions were present, which led to the formation of Housing and Development Board (HDB) in 1959, marking the beginning of large-scale and integrated developments. For the first 15 years, HDB built 155,000 units, housing 800,000 people or 41 percent of the total population of Singapore [8]. The successful housing developments in Singapore are the result of HDB’s strong activities, commitment, research conducted regarding resident satisfaction as the basis for policy-making, and the emphasis on the notion of home ownership.

The majority of the large-scale developments, most in the form of combination of high-rise and low-rise buildings, were situated on the outskirts due to the availability of land and the cost. Singapore’s plan was to create 23 high-rise, high-density New Towns. Also, public

transportation to the city center was a major consideration for the developments, since mass transportation is an effective, efficient way for commuting. The towns are organized around the island's mass rapid transit system. By actually following the Master Plan, the maximum amount of open space in between settlements was preserved [9]. Also, HDB set the socio-economic guidelines, where each household could be charged 15 percent of the income, and the public housing was perceived as one of the strategies for nation-building. Therefore, the planning process for low-cost housing included social and economic considerations, such as transportation, construction cost, household income and social image.
The size of Singapore's larger New Towns grew from a modest size to a population of 200,000 and more [9]. This planning has created a rich context of regional cities and smaller towns connected by the state's modern public transportation system. Along with the population growth, the housing changed shape too, from low-rise regularly spaced blocks to 50-story high developments. These high-rise buildings have been mixed into the low-rise housing. Today, 84 percent of Singapore's population lives in public housing high-rises, while 6 percent lives in private housing [10]. The concept of home ownership has been a principal driving factor for the successful resolution of Singapore's housing problems for lower and middle income groups.

Jakarta

Jakarta, Indonesia, is one of the world's fastest-growing cities. United Nations estimates put the city's 1995 population at 11.5 million. The present population is estimated at between 20 and 25 million. The daily parade of combustion engines clogs the city streets and thickens the air, making Jakarta one of the world's most polluted cities. Rapid growth has become one of the city's greatest challenges, as migrants continue to arrive in Jakarta from the surrounding countryside in search of higher-paying jobs. In the face of such growth, the city has been unable to provide adequate housing, despite repeated attempts to launch urban improvement programs.
The Kampung Improvement Program (KIP), established in the 1980s, was initially highly successful in boosting living conditions for more than 3.5 million established migrants, but it has been unable to accommodate the continued migrant influx.
The cost of land greatly affects the overall building cost—increasing prices lead to lack of low-cost housing for the most vulnerable population.
The analysis of the cost of land in Jakarta shows the importance of infrastructure provision and tenure for land prices [11]. Also, the recent price increases have been constantly greater in the suburban areas and in informal-sector plots, rising from the demand for from low-income household for affordable housing. In that sense, it is essential that the land prices be monitored by the government.
Studies show that Jakarta has experienced drastic decrease of population density in the city center since 1970 [12]. The peripheral zones have been pushed by urbanization, causing uncontrolled rural-to-urban conversion of large areas of land surrounding the city. Low-cost housing needs for all still remains to be met in Jakarta.

Dhaka

This capital city of Bangladesh had only 250,000 people in 1947 growing to 7 million in 1994; it has more than 14 million people today. The city has been gaining population at a rate of nearly 7 percent a year since 1975 and the population is still increasing at an alarming rate. The city is beset with socio-environmental problems, including traffic congestion, housing shortage and growth of slums, flooding, solid waste disposal, vehicular and industrial emissions, air and noise pollution, and pollution of water bodies by industrial discharge.
Nearly 45 percent of all Bangladeshis live below the poverty line. Urbanization and the pressures of poverty are severely stressing the country's once-abundant natural resources. Some other problems include poor management of aquatic and terrestrial resources; overuse of resources; unplanned building projects; and expansion of agriculture onto less-productive lands, creating erosion and runoff, among others.
Government of Bangladesh has been actively involved in the construction and management of public housing projects for its employees through Public Works Department. The funding mainly came from the internal and external sources, particularly taxation, borrowing and foreign aid, but the problem of financing had remained a great issue [13]. The maintenance cost was determined one the basis of original capital cost, and the Directorate for Accommodation

decided the standard rent to be charged from the tenants, which is generally 7.5 percent of the household income. However, the rest of the population, especially low-income majority, does not have access to public housing. Muhammad Kaysar Hussain states, “Among the low-income poor people there may be gradation of poverty.

There are people who are so poor that, not to speak of a home or shelter of their own, they do not even possess minimum amount of food and clothing. The reason of not possessing a house and other necessary things is poverty…” [14]. Also, the lowest-income households spend as much as 50 to 75 percent of their income for substandard accommodation [14]. Because of the population growth and prosperity of many citizens these numbers may have changed now, but on the whole the picture remains the same, As much of 57 percent of the households have no land in the city, while only 4 percent of the large ownership group holds much as 27 percent of the land [15].

Growth of slums in Dhaka in the last ten years is an enormous challenge in this megacity. In 1996, the slum population was 1.5 million in 3,000 clusters. In 2005, it has grown to 3.4 million residing in 50,000 clusters [16]. Until the early 1990s, majority of the slums were located on public lands, however, this has drastically changed in the last decade mainly because government had started to evict slum dwellers. Today, 77 percent of slums are located on private land in the inner city [16]. It is evident that without tenure and ownership the slum dwellers will continue to move from one area to another.

TECHNOLOGICAL ASPECTS

From the architectural standpoint, technologies are available for the construction and operation of low-cost housing. Affordable materials, modular housing, prefabricated units—these are some of the means for the design and creation of affordable housing, however, the methods must vary according to the context, location and needs.

Low-Tech Temporary Housing

Shigeru Ban developed an innovative technology using cardboard tubes as structural pieces to design and build temporary shelters. Log house was designed after an earthquake disaster in Kobe, Japan in 1995, and the concept was used to build 21 houses for earthquake victims. This low-technology approach uses paper tubes as structure, beer crates as foundation system to raise the structure off the ground, and canvas as roofing. Easily assembled, it takes about six hours to construct one house.

Waterproofing of the tubes is achieved by laminating the tubes. The size of the house is 16 sq.m. (172 sq.ft.) and the total cost is around $2,000 per house. The residents are expected to use log houses only for about six months.

Since the structure is made from paper, it can be easily recycled after the residents have moved. The innovative technology used in the log houses proved to be very efficient and cost effective for temporary shelters.

The Portable House intended for those who have disposable income but not enough capital, has expandable and contractible spaces. It has a natural ventilation scheme, plastic foam insulation and floors of plyboo, a laminated bamboo product.

Modular and Prefabricated Housing

One of the early examples of modular prefabricated housing was Habitat ’67, designed by Moshie Safdie as a thesis project and later built in Montreal in 1967.

The initial design incorporated 1000 dwelling units, constructed of prefabricated concrete cubes stacked in a pyramid form.

The initial idea was to create an example of affordable, high-quality housing, but the overall construction cost proved to be more than initiated, and the final design had 354 units. However, this project was very successful in terms of providing pleasant, varying living spaces from modular units, creating 15 different floor types and variation in apartment sizes. The importance of this complex is that modular design in residential architecture, especially in affordable housing, is not a novel idea.

Some factors that must be taken into account in modular design include site attributes, construction methods, construction costs and choice of materials.

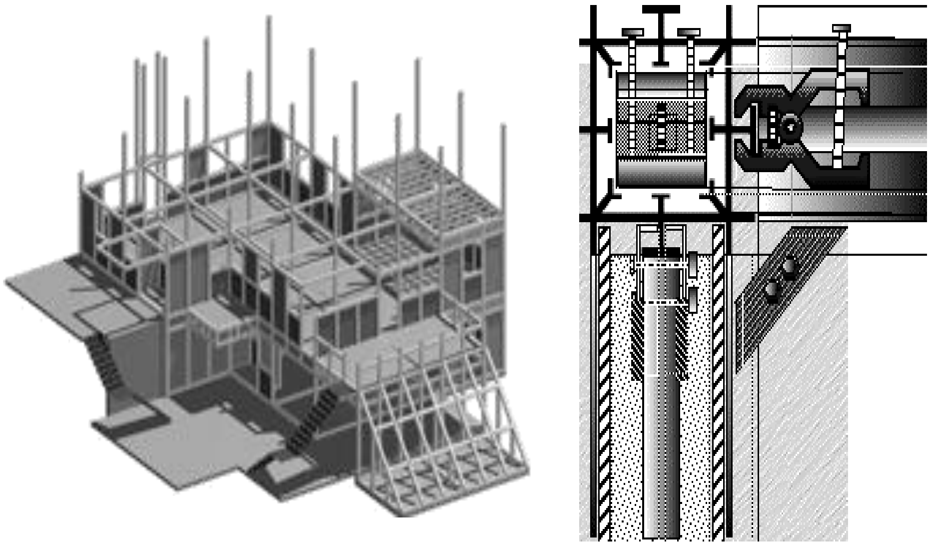

Fig. 2 : Modular housing system (MHS) and component detail

The Modular Housing System (MHS) consists of prefabricated modular aluminum structural extrusions fastened together by a patented quick-connect technology (Fig. 2).
Unlike most residential construction designs, MHS employs a post-and-beam method that uses simple rectilinear space frame geometry and a bolted assembly method .
The cladding system uses either structural insulated panels (SIP) or any of a number of other panel materials, strawboard panels, gypsum board, or light concrete block, which slide into the grooves of the framing profiles.
The profiles can accommodate modular window panels, or composite panel walls can be framed to accommodate more conventional windows of any shape.
Modular aluminum structural design allows one to freely and quickly disassemble, repair, or modify and reassemble structures without causing any damage to the components and materials (Fig. 3).

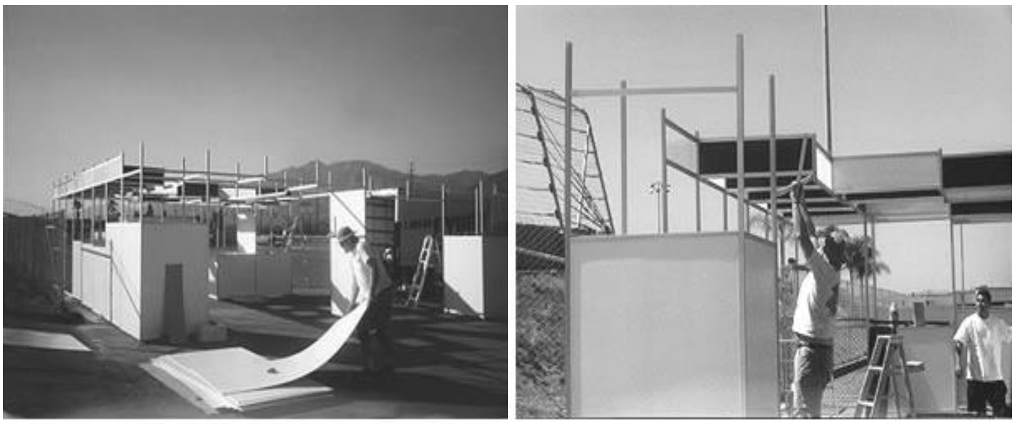

Fig. 3 : Ease of construction of structural system and panels

Covintec panels are fabricated by spray-on plaster on mesh surrounding foam insulation.
Such insulated panels are extremely lightweight and easy to handle and install [17].
Economy in modular construction is derived from the large-scale fabrication of the panels and the ease and speed of material handling and construction, as well as their durability (Fig 4).

Fig. 4 : Covintec panels

Sustainability and Transfer of Technology

Initial building cost, as well as land cost, is important attributes of affordable housing. However, the building operation cost is also a key aspect in the overall expenditure over the building life-cycle, and should be addressed. In the case of developing countries, it becomes even more crucial, since low-income tenants often have limited resources for housing and utilities.

Simple strategies for green design, such as building orientation, natural ventilation, daylighting, use of local materials and construction practices, improved insulation, sun shading and response to climatic conditions, are few examples that can be incorporated to lower the operational costs of housing.

For example, wood construction has been the primary building method for the construction of low-rise residential architecture in the United States and Canada, primarily because of the availability and ease of construction. However, during the last decade increasing cost of lumber has led to innovations in residential building technology. In particular, wood structural insulated panels, insulated concrete forms and light-gauge steel framing have seen wider application than ever before [18]. SIP consists of insulating material, usually polystyrene or polyurethane, sandwiched between oriented strand board, waferboard or plywood. SIPs have better performance than the conventional wood framing because of the enhanced structural properties, thermal properties, prefabrication and the cost of construction. SIP can be prefabricated with insulation, framing and sheathing already included. They are lighter and can be delivered to the site, and ready for construction. Electrical and plumbing can also be included within the panels. SIP can be used for exterior and interior wall and roof construction. The thermal properties account for better performance in terms of energy efficiency where it is needed, and the insulation used can be made of recycled materials. The advantages of SIP panels are:

- Energy efficiency: Typical R-values for nominal 100mm (4 in.) thick wall panels range from R - 13 to R-22, depending on the type of insulation, where thicker wall panels have corresponding higher thermal resistance.
- Speed of construction: Use of SIP reduces the time required to assemble and construct a building. Continuous structural sheathing simplifies the attachment of the exterior and interior finishes.
- Superior structural performance: Since SIP act as stressed skin panels, bracing is incorporated into the panels, and under wind and seismic loads they have superior
- performance over conventional wood framed walls, which have structural sheathing only on one side.
- Minimized constructional waste on site: SIP panels are prefabricated and shipped to the building site, minimizing the amount of waste produced during the construction (Fig 2).

Comparative study for U.S. conditions was conducted at the University of Illinois at Urbana-Champaign, evaluating the cost of conventional wood-framed housing and SIP, examining the overall initial price and the operating costs [19]. The initial cost of SIP house would not be necessarily more affordable than the conventional house; however, the savings are evident in the operational cost. Fig. 5 shows the floor plan and construction sequence. Although SIP may not be immediately applicable to the much of the developing world, similar concepts using local materials and appropriate technology can be developed there.

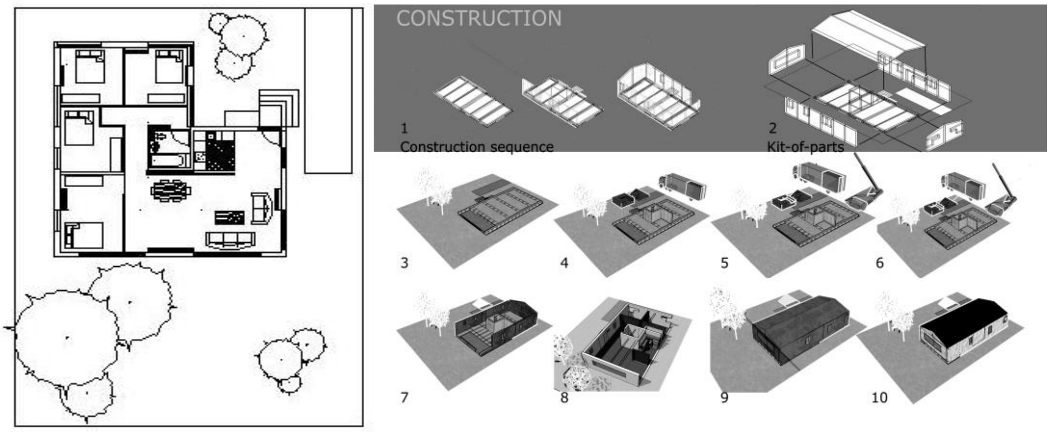

Fig. 5 : Typical Floor Plan Used for the Comparative Study and the Construction Sequence of SIP

Table 1: Detailed Cost Analysis of Conventional Wood Framing and SIP Housing

CONVENTIONAL WOOD FRAMING		SIP HOUSE	
Permits	$3,864	Permits	$3,864
Site work	$668	Site work	$668
Sewer system	$1,044	Sewer system	$1,044
Water system	$510	Water system	$510
Sidewalks and driveway	$742	Sidewalks and driveway	$742
Foundation	$7,435	Foundation	$7,435
Framing	$17,488	SIP	$23,561
Exterior	$5,148	Exterior	$5,148
Insulation	$2,989	Insulation	$0
Drywall and trim	$4,758	Drywall and trim	$4,758
Roofing	$3,736	Roofing	$3,736
Windows	$7,937	Windows	$4,409
Doors	$8,186	Doors	$5,885
Floor coverings	$3,293	Floor coverings	$3,293
Kitchen	$5,107	Kitchen	$5,107
Plumbing	$5,465	Plumbing	$5,465
HVAC	$7,017	HVAC	$4,000
Electrical	$6,607	Electrical	$4,937
Landscaping	$1,867	Landscaping	$1,867
Total	$93,861	Total	$86,429

Table 2 : Operating Costs for Conventional Wood Framing and SIP Housing

		SIP House	Conventional House	Tenant's Savings
Estimated operating costs at 4% annual cost increase	5 years	$2,264	$5,647	$3,383
	10 years	$5,263	$11,552	$6,289
	15 years	$8,911	$17,742	$8,831
	20 years	$13,349	$24,246	$10,897

Tables 1 and 2 show, respectively, a cost analysis of the two systems and their estimated operating costs. Similarly, the newly emerging low-cost composite structural insulated panels (CSIP) have a great potential for application for modular construction. Transfer of technology from developed countries applying emerging technologies to developing countries can pave the way for innovation in those countries. Clearly, pre-engineered and prefabricate building components using paneling concepts that do not require skilled labor for installation must be seriously considered for low-cost housing in developing countries. The panels may be made thin, water-resistant, and of high strength for wind resistance.

RECOMMENDATIONS

Following considerations should be addressed for the development of low-cost housing in the megacities of developing world:

- Balance in the scale of developments within cities
- Balanced development of all cities in a country
- Local needs and resources
- Land cost and distribution
- Local, low-cost materials and constructions methods
- Transfer of technology
- Energy efficiency and sustainable practices
- Low-tech aspects for green design
- Availability of transportation and infrastructure
- Policy and financial planning
- Balance between public and private interfaces
- Ownership and governmental subsidy
- Community building focusing on environment and socio-economics

Strong balance between these different aspects is crucial for successful developments of affordable housing. Low-cost housing in not only an architectural or technological issue, it compromises many more aspects, such as politics, economy and social issues, and serious deliberation for all of these factors is necessary.

In terms of design and construction, special attention is required on durability and cost of materials, intersection of dissimilar materials, detailing of openings and low-tech rainscreen principles to keep rain out. Social engineering and integration are other important issues for low-cost housing.

In low-cost high-rise housing for low-income groups in the public housing sector social problems arise in the developed world as in the case of Pruitt Igoe Housing Project in St. Louis, Missouri in the United States [20, 21, 22]. However, in the developing world, such major projects to house the low-income people must be initiated and controlled by the government in collaboration with the private sector.

Mixing middle and lower income groups in the same housing complex could be beneficial because those in the lower income group can look up to the other group as role models. However, the lower income group should constitute no more than 30 to 40 percent of the population in mixed housing systems so that the middle income group is not otherwise discouraged to live in such housing complexes. Building housing units in

the outskirts causes urban sprawl, whereas consolidating them in high-rises within the city may result in concentration of poverty, unless done in the context of a mixed-housing system with proper planning and building design. The Singapore example of mixing high-rise building into low-rise housing could be a model to emulate for some cities.

CONCLUSIONS

This paper has discussed the issues of low-cost housing for megacities of the developing world. The problems include population increase, land shortage, financial and economical constraints, and the conditions of the lowest income population. Examples of Singapore, Jakarta and Dhaka have been presented in order to show the differences in needs and approaches taken by local governments to address these issues. It is evident that the standards and practices of the developed countries do not necessarily apply in the solutions for the housing crisis in the developing world. It must be emphasized that within the developing countries there are different needs and resources that should be taken into account. The technological aspects of low-cost housing are addressed, in particular, temporary shelter, modular and prefabricated housing, and sustainability. The emphasis is also placed on the building operation cost as one of the strategies for creation and design of affordable housing, where sustainable building practices would result in overall cost reduction for inhabitants. Technology transfer fro the developed to the developing world is suggested in the paper.

Examples of megacities of developing world presented in this paper show that housing issue is in fact not only an architectural problem, but rather a combination of different aspects—economy, availability of land, construction costs, availability of materials, and socio-political issues. The keys to solutions are complex, encompassing adoptions of efficiency policies, development of capital, community participation, and focus on technology [7].

Policies designated at using resources more efficiently include creation of financial reforms to enhance private-public interfaces, implementation of municipal services and subsides for most vulnerable population. Also, the balance between local and regional development, such as central city development and suburban, is crucial since transportation and infrastructure systems need to be provided for developments that are located on the outskirts. Combination in scales of developments is also necessary, where high-rises are optimal solutions for inner cities, with medium and low-rises moving away from the center.

Underprivileged people are unable to get housing in megacities because landlords often charge more rent since the housing market is generally in their favor. This leaves many people in developed countries in urban settings without a decent place to live. Something must be done for them. Government must subsidize some of the building costs as well as rent payments. Both private and public businesses should be given grants and incentives to build low-cost housing.

Political corruption and bureaucratic inefficiency also hurt the effectiveness of financing solutions. Microfinance pioneered by Grameen Bank in Bangladesh is one method that can be employed to meet the needs of many people. It allows the private sector to take a more active role in financing home buying. In collaboration with governmental and international organizations, community-based enterprises can better serve the financial needs of the people.

Finally, the success of architects' and urban planners' work depends on the ability to predict human behavior and the way the built work will accommodate the changing actions and functions of the users, as well as the limitations posed on the design. The architectural solution for the concept of low-cost housing is limited, in terms of accommodating land speculation, trends, politics and other social and economical factors. However, there are certain strategies which designers can take to address these issues, such as material selection, construction method, and careful analysis of requirements to find the best suitable solution. After all, the need for shelter is one of the essential necessities of every human being, and it is a collective concern that this is realized for all.

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REVITALIZING ARCHITECTURAL EDUCATION IN CAIRO

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ABSTRACT

This paper is a reflection on the annual students' competition at Cairo University. This student competition aimed beyond being an occasional activity, rather, a way to enhance the architectural educational process. Moreover it was thought of as a tool to bypass the existing hurdles in the Egyptian educational system. In short it was thought of as leading a parallel reform to the existing system that can be seen as stagnant and in a dire need for positive actions.
This paper focuses on the first two cycles of the competition where issues of technology and history were highlighted. How these subjects were understood and dealt with by the students of architecture in a developing country like Egypt was the issue of the competition. This paper consists of mainly four parts; the first deals with how a certain issue is articulated through the competition brief. The second part deals with the jury process and the debate that took place. The third deals with the students' perception of the addressed issues. A fourth part reflects on the benefits and lessons of the whole process. Finally the conclusion sums up the main finding of the experiment.

Keywords: Architectural Education, Students' Competitions, Architectural Design.

INTRODUCTION

The architectural education in Egypt faces many challenges in order to be able to perform its role. The design studio which is the core of this education is one that needs to be given serious attention. Many would argue that the architectural design studio at the Egyptian universities needs to undergo serious reform [1]. The challenges facing the design studio are many, for example; the number of students in the design studio which some times exceeds one hundred students. The process of education itself which minimizes the student/ instructor interface, the quality and the relevance of the content in relation to the community aspirations are also some of the serious challenges [2].
In this paper, the ideas presented were thought of to bypass the formal educational system in an attempt to respond to rising challenges through introducing other parallel activities that might address those challenges more effectively. The initial idea of this students' competition that can be considered as an educational experiment started back in August 2003. At that time, as a new head of the architectural department at Cairo University assumed the responsibility, new activities were launched. An annual conference on architecture was initiated as one of the newly proposed events that aim at instigating vitality into the still environment of architectural education. As one of the accompanying events, the students' competition was also proposed by the author.

OUTLINING THE COMPETITION AS AN EDUCATIONAL TOOL

The students' competition was thought of as an attempt to restore the missing environment that focuses on innovative qualities in design in addition to addressing the real issues that matter to

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the Egyptian community. To do that, the competition focused on the national level to encourage the involvement of nation's wide Egyptian students of architecture. On the other hand, international jurors were invited to participate in the jury process in order to expose students' work to a wider scope of critical views and ideas.

One of the assumptions that underlined the competition efforts was that architectural education and especially architectural design lacks seriousness and clarity in dealing with real issues confronting the Egyptian community [3]. One of the competition objectives was to challenge and confront the rules and (mis)conceptions that are locally prevalent. The prevalent emphasis on the architect as an individual was challenged by requiring the students to work in teams. The absence of emphasis on the importance of real site visits and analysis in the design studio was also challenged by requiring drawings and other data that cannot be delivered without immersing in the site. Moreover, the size of the competition project was small enough to allow for more time for thinking deeply and bringing about ideas and visions that inform the architectural design process.

The competition needed to seriously and critically address issues that relate to society. Furthermore, it sets out to explore major societal issues and their impact and meaning in architectural design terms. Therefore, participants would be ideally students who got a sufficient grip on what is architecture and those who articulated (to a certain extent their intellectual ability). Accordingly, the suitable category of targeted students was the senior students. This issue was not clear in the brief. In practice, all students whether junior or senior were welcomed to participate, may be out of concern for wider participation.

To encourage a nation wide participation a competition poster was prepared and sent to all architectural departments in Egypt (at that time around 17 departments in public and private universities). Few schools included the competition as a design project. Among those few schools was the architectural department at Cairo University, where second grade students were assigned the competition as their first design project. One of the private universities, MSA architectural department, allowed the students to choose alternatively between the main design project and the competition. 36 entries were submitted representing 15 architectural departments in Egypt.

To extend the benefits of such an event, each one of the international jurors was asked to deliver a public lecture relevant to the subject of the competition. These lectures targeted the entire architectural community in Egypt and not only the students. In doing so the students' competition was intended to become a critical forum for discussions and learning on essential issues of architecture and community in Egypt.

THE COMPETITION' ISSUES AND THE COMMUNITY

Where community meets… offers the Egyptian students an opportunity to consider new territories in questioning how certain building types interact with the street and social life in a particular community? One can ask how a successful intervention could take place. How do people use these spaces; and many other questions? The competition encouraged and required an intense and broad approach where by the scope of architecture on the life of the community critically debated.

While the idea of issues that faces the developing community was clear early on the formulation of the competition, the specific issues were not identified. After the first competition process was completed and the documentation of it started to materialize, the commanding issues were thought of and laid out. In addition to technology, four other issues where identified; history, nature, art, and knowledge. These issues were considered by the competition initiators to form the main intellectual and theoretical hurdles facing the Egyptian community. These five issues were thought of as constituting both philosophical and conceptual challenges with essential impact on the creation of places and the quality of the environment in Egypt. The underlying argument of selecting these particular issues was that Egyptian society needs to come to terms with them if it is to advance into more developed environment both physically and intellectually.

The First Cycle of the Competition

The first cycle of the student competition dealt with technology as a central issue and how it can be understood in a way that would help in creating a better place for the local community. Technology cannot only be seen as inseparable of society but it" has a huge potential to shape the urban and its cultures.” Technology can be seen as a major influence in shaping places and spaces of our cities where” patterns of social divisions and even ways of being” are continuously taking place [4]. Since technology aims at changing the reality surrounding us, one can approach it as a basic characteristic of the human progress and development [5]. It can neither be separated form a real concern for the community development, nor from shaping the place where it is being dealt with. As the developing countries are striving to reach the status of a welfare society, the very deep history of civilization comes in confrontation with modern technology. Light rail cuts through greater Cairo city and passes through different local communities. The light rail station building becomes a place where traditional community uses a very advanced technology. In this context the competition aimed at addressing this situation and reaching creative proposals.

The previous ideas were the basic assumptions of the brief of the competition. The objectives of the competition brief were to encourage primarily three main aspects in any of the schemes presented by the participating students. First, critical understanding of the main issue as it relates to the form generation process was considered as a characteristic in any proposal. Second, the relevance to community concerns and interests needed to be manifested. Third, innovation is not only the goal of the competition process but also a product of serious investigation.

The site of the competition is Al-Malek Al-Saleh station, located near the first foundation site of Cairo, the old Coptic churches complex and the remains of the roman fortress. It is surrounded by a hospital on the east side, a printing press for one of the publishing houses, shops, a residential area then the Nile on the west side. The area was renewed after the completion of the Metro first line in the late 80s. However, the station image at present suffers from numerous deficiencies and can not adequately respond to the needs and aspirations of the everyday users as well as the surrounding community.

The main component of the competition is to redesign the station and its immediate context, keeping the location of the two platforms as is while changing the location and dimensions of the station entrances. It is expected that the existing pedestrian bridges will be removed and replaced by tunnels (beyond the scope of the competition). The competition consists of three integrated parts; First: A proposal for the immediate urban context, Second: Designing the station building and Third: Designing the shed

In addition to detailing the site, the program, the design issues and the requirements, the competition brief incorporated another interesting statement. The introduction of the brief highlighted a tradition that is badly needed nowadays. It used a historical statement which is "Imagination is a way of knowledge" in order to stress the point of view that the challenges the community is facing cannot be resolved without the extensive use of imagination. At the same time imagination can be seen as the core of architecture.

Debating the Competition Main Issues

The jury represented different intellectual backgrounds therefore formulating a well rounded discussion on historical issues, modernity, development, theoretical and global notions. The jurors were; Peter Davey, Howaida ElHarithy, Renato Rizzi, Abdelhalim Ibrahim Abdelhalim and Nabeel Elhady. Peter Davey was the editor of the Architectural Review Magazine, and based in London. Howaida ElHarithy was the Chairperson of the department of architecture at the American University in Beirut. Renato Rizzi, is an architect and professor at the School of Architecture at Venice. Abdelhalim Ibrahim Abdelhalim is an Architect, and professor at Cairo University. Nabeel Elhady is an Architect and professor at Cairo University. The jury worked for one day and was assisted by a technical committee of researchers to help evaluate the entries.

At the beginning of the jury deliberations prof, Rizzi pointed out to the basic contradiction that exists between community or society and technology. He noted that the ancient Egyptian production of architecture is an evidence of how to get a highly aesthetical building while being at the same time based on a sophisticated building technology. On the other hand, modernism is based on nihilism and great abstraction and is very destructive to objects of value. His view was that technology represents science and its nihilistic basis, while society and especially the Egyptian one has its metaphysical basis. The first (i.e nihilism) denies, neutralizes and destroys form while the second (i.e metaphysics) has a strong vision of form. He sees that architecture has turned its back to art as a source of inspiration to science. This might have taken place because it is easier to deal with a formula that leads through step by a step approach to safe results, while it is much more difficult to do that with art [6]. Other jurors saw that the seemingly contradiction between community and technology has to do with the associations both concepts have. Technology for example is mostly associated with the west while community is known for its historical reference.

The term "community" as mentioned in the brief was seen by number of jurors as an ambiguous issue as it lacks a clear place based connections. For this and other reasons, one of the jurors questioned the necessity to include the term community in the main topic of the competition. Yet other jury members argued about more pragmatic aspects of the issue.

They focused on what is meant by community and whether it is restricted to the community surrounding the station or the community of commuters and other users of the station. One of the jurors noted that, in relation to the station; the surrounding community can be seen as a static one. The other is dynamic which contains the users of the train and the station. While some of the jurors were in favor of the dynamic community others thought that the station should celebrate both. The two communities meet at the station.

This discussion highlighted the importance to readdress and critically discuss the issue of what is meant by community which is taken for granted in most of the design studios. In some sense this debate relates to the critique that points to the misuse of the term community under the banner of new urbanism.

As Paul Clarke wrote community has to be approached as “an arena of participation that evolves through discourse, a constituent missing within the philosophy and constructed developments of New Urbanism” [7].

Understanding the station as a technological edifice, the debate among the jury members also dealt with the extent to which technology contributes to dividing the existing community or whether it can be seen as a link that ties what the railways have done in separating the communities on both sides of the tracks. Redefining technology from the community's point of view was the major task of the student. The concept of mobility was also raised in the discussion. Mobility is seen as an important issue of the contemporary city where the train and its stations is nothing but a manifestation. Mobility is certainly a “constituent of modernity as is urbanity” [8].

Perceiving the Competition Issues

Regarding how the project addressed the notion of community one can hardly find an apparent definition. Most of entries reflected, in some way or another, a vague understanding of the community. This can be understood not only through reviewing the submitted text but also through the lack of articulation of proposals that dealt with the surroundings and the station as a meeting place for different sub-cultural groups of the community. Some of the projects suggested integrating some commercial and cultural activities to the station. Some of the projects even proposed providing local markets in the area adjacent to the station. Others provided green areas and parking spaces around the station. Yet, most of the projects did not concern themselves with a marked intervention to the immediate context.

Considerable number of entries dealt with the station itself as a bridge connecting eastern and western platforms.

Some of the entries proposed a superstructure that covers the whole stations while others limited themselves to two separate sheds. Emphasizing the form was very clear in most of the projects. Some entries used a direct analogy with historical shapes such as the crescent, the ancient Egyptian sacred Scarab (the dung beetle), and Islamic shapes, particularly the Iyobid symbol. Others used more abstract language such as contrasting materials (glass, metal) and other used forms such as cube, to indicate certain approach to technology or community. Few projects were able to overcome the immediate obsession with form.

Reviewing the winning schemes will further our understanding of how some of the noted entries perceived the issues addressed by the competition. It will also help in reviewing how the debate of the jury informed the winner's selection.

First Prize Winner

The concept of this project uses the metaphor of the heart to denote the station as the centre of movement and energy; where a parallel was drawn between the train movement and heartbeat. Patterns of movements from one node to the next were denoted by the heart valves as a metaphor of routes to different places symbolizing rhythm and continuous flow.

Fig. 1: Shows one of the entries that used the Egyptian sacred Scarab as a metaphor for form generation, the author.

Fig. 2: Shows one of the entries, the author

Across the station there was an introduction of a pedestrian overpass where people could move from one platform to another. What can be seen clearly in this project is the clear ability to deal with the urban, architectural and details level with a high degree of consistency .
The approach of how to deal with technology seemed to be thought out well. The architecture of the station reflected the thoughts and ideas in a sophisticated manner. More emphasis is put in this project on the station itself. While the context can be seen in some of the views yet the prevailing feeling is that the station is strongly present in its context.

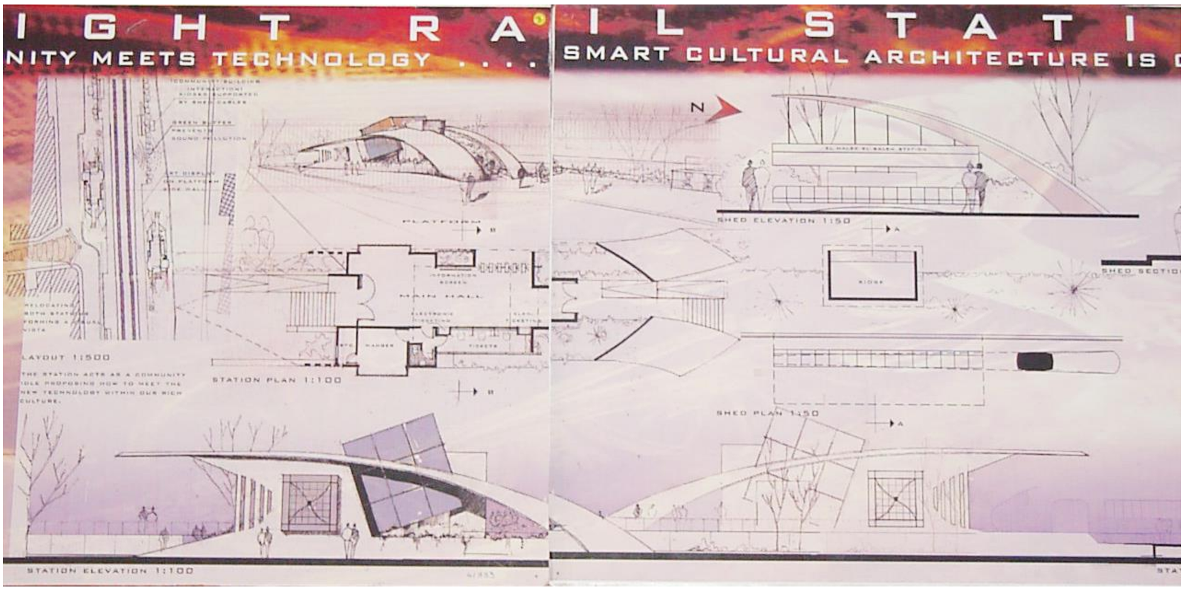

Fig. 3: Shows one of the entries, the author.

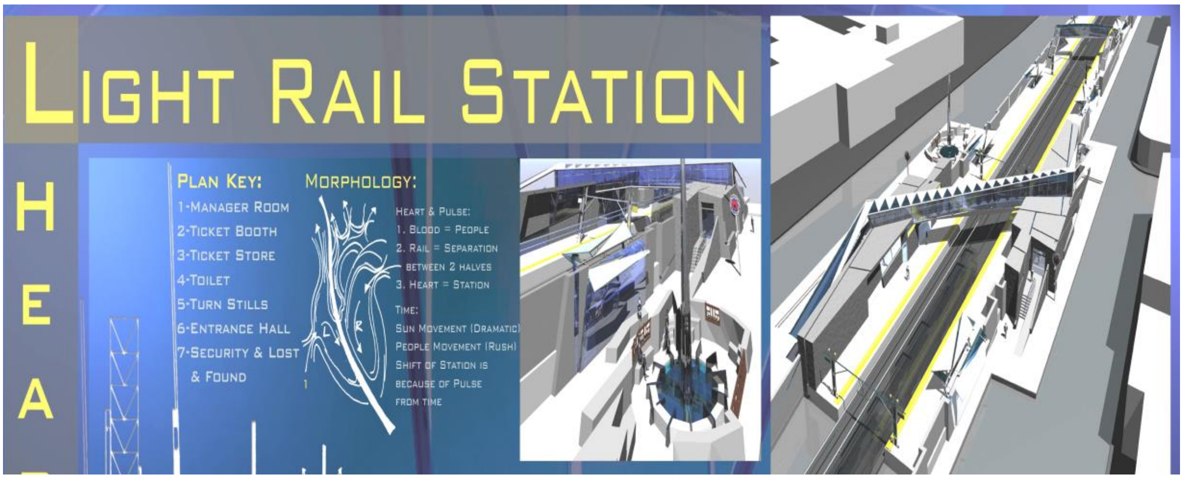

Fig. 4: Shows an image of first prize winner entry, the author.

Second Prize winner

The concept behind the design was to introduce current technological trends into a community where technology was once present and advanced but became recently absent or slow developing . The purpose and driving force behind the design is to attempt to merge the technology with the community and to create a relationship at the point of interaction between spaces.

The idea developed by exploring the contradictions found between the serene and the busy nature of spaces, movement and stillness, the congestion and emptiness all of which would be expressed in the design of the station.

This contradiction in nature was expressed through the functional advancement of the design and the use of the traditional elements in the building material. The project is seen as a symbolic attempt to show the adaptation of technology in a historical setting.

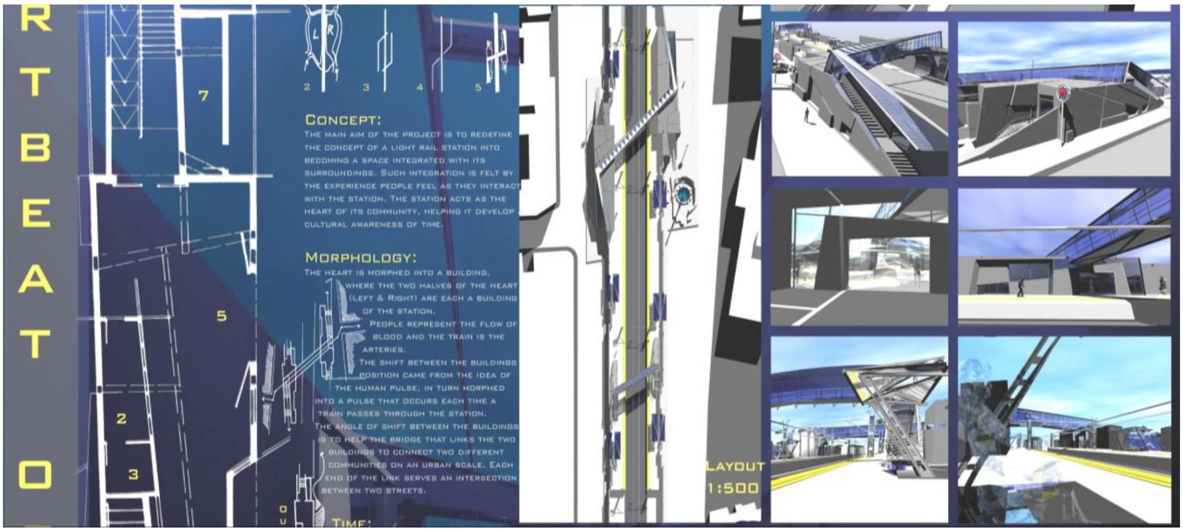

Fig. 5: Shows an image of first prize winner entry, the author.

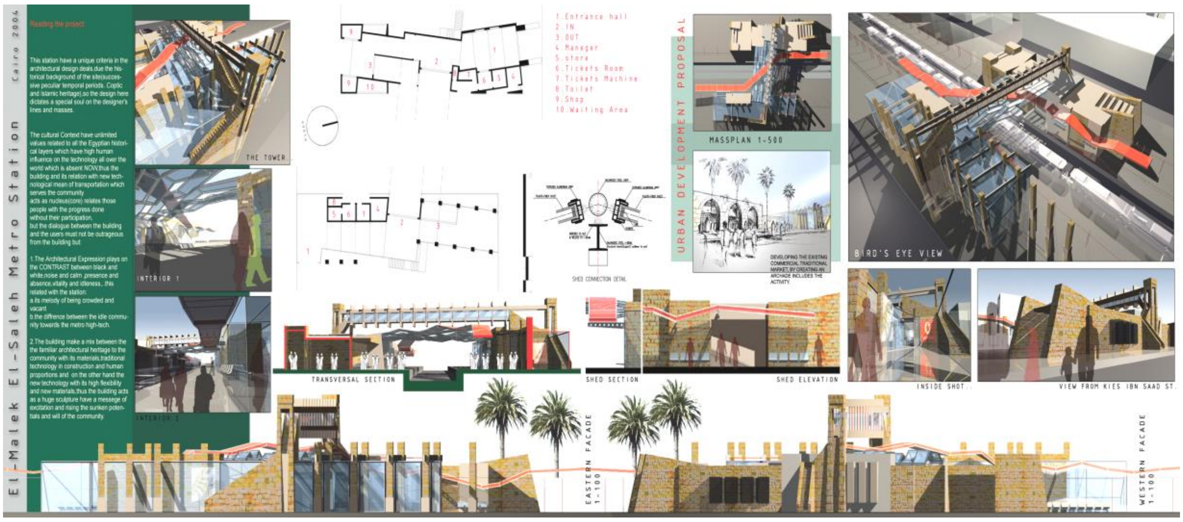

Fig. 6: Shows an image of second prize winner project, the author.

A further representation and design maturity was absent in some drawings and sections, also a further simplicity in the main building was added due to the inclusion of smaller and secondary structures which disrupted the main design at times. This approach might appeal to some critics as "disneyfication" meaning an exaggerated collection of shapes similar to that of architecture in theatre or theme park.

Third Prize Winner

The introduction of the climatic and cultural elements was the base of the concept for this design. With the socio-climatic elements as a main feature of the design there was room for the exploration of the effect this would have on the design of the station as a whole. The attempt to create a landmark was a key approach, also and interestingly the introduction of garden nodes in a barren setting was a new feature for train stations. Much research of the environmental aspects was addressed and incorporated in the scheme with the study of air movement, cooling and reduction of energy consumption. Islamic geometric patterns were used to define both the aesthetic and structural features of the architecture of the station, yet they were interpreted to fit the functionality of the space and house technology.

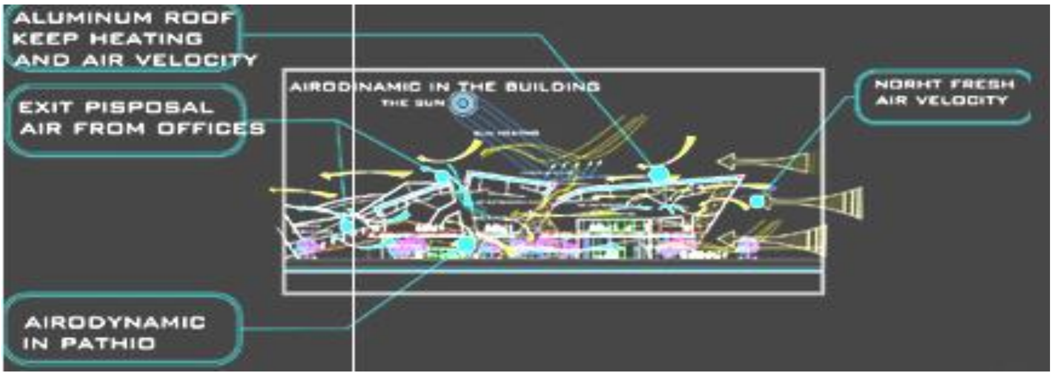

Fig. 7: Shows an image of second prize winner project, the author.

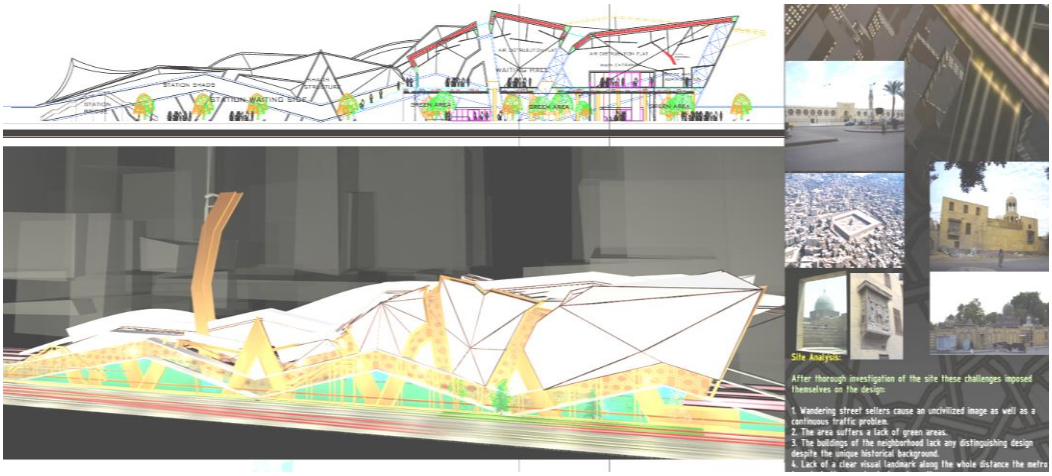

Fig. 8: Shows an image of second prize winner project, the author.

REVITALIZING ARCHITECTURAL EDUCATION IN CAIRO

The organizer of the competition used the possible means to promote the competition and make it accessible for the students of architecture all over Egypt. Posters were sent to most of the architectural departments. Messages to the head of departments were also sent to ask them to encourage their students to participate. Contacts with design professor, chairs of departments as well as faultiness's deans were made to make sure of the wider participation in the competition. Design professors were urged to include the competition in their class agenda. The design studio of the second year at the architectural department at Cairo University decided to make the competition as one of the students' term projects. At the Modern Science and Literature private university the head of architecture department allowed the students to choose the competition as an alternative to the already given project. No other similar commitments that one knew of were made in other departments of architecture.
One can also note that the issue of the competition in the first cycle, i.e. technology, at least as was stated in the competition brief, was not adequately explained. While, this might be seen as purposefully done to motivate the students to explore and investigate by themselves the subject at stake. Yet, as a student competition, where education is the core, the issue could have been laid before the students in a more conspicuous way that leads them to the beginning of the research trail.
The jury process of the competition was blind and closed. This closed and exclusive process was based on the argument that the participant teams should be able to articulate their ideas without a need to be physically present to discuss or defend their ideas. Whether this concept is more beneficial from an educational point of view or whether the open jury where participants would be allowed to present and defend their ideas is more relevant, was not discussed and

maybe is in need of further investigation. Indeed, it can be argued that the open jury, especially when other students and staff are allowed in, will be more valuable in educational terms.
The national nature of the competition, where the native language of students is Arabic poses also another issue that it was not clear how it affected the jury's deliberation process. The international nature of the jury, where English is prevalent, required that the participants submit their entries in English.
One of the requirements of the competition entries was a statement that clarifies the participants' ideas and intentions. The reading and understanding of the statements proves to be difficult to most of entries which is believed to undermine the communicational ability of such statements. Therefore , the language issue needs to be carefully considered and addressed to find better and more effective ways to clarify and communicate ideas.
The discussion amongst the jurors was intense and revealing about the situation of the design education in the Egyptian universities. As Peter Davey noted, this was the first time for him to see this number of formalistic designs [6]. This comment is, I believe, explained and supported by what Khaled Asfor had observed regarding most of the entries of Egyptian architects who participated in the Grand Egyptian Museum international architecture competition. As he noted that most of them were obsessed with forms not with meanings and deep concepts and understandings [9].
As the process of the competition shows, the creation of meaningful places in the Egyptian community highlights the need for discussing technology. Also , as innovation is becoming more and more a survival condition in the practice of architecture [10]. Technology can be seen as closely related to the need for innovation. Any innovative technology needs to be well integrated into our daily life and activities and as Mark Weiser noticed “The most profound technologies are those that disappear. They weave themselves into the fabric of every day life until they are indistinguishable from it” [10].
It became clear through the competition that stations as places of technology in the city are becoming more and more important focal points.
Stations can no longer be seen as congested, dirty, and unwelcoming places. There is a real potential for the stations to become places that are clean, pleasant and urban. They provide vital, lively, and energized places for people interaction. They cannot only raise revenue, but they can be central points in the city life [11]. The importance of stations gets beyond architecture into having a major role in town planning as they demonstrate a key role in the surrounding community and the city at large. In this way one can see how getting deep into the subject would further our understanding of architecture to other scales especially in how it relates and impacts the city and its life.
The theme of the competition which related community and technology readdresses the issue of the business of architecture and whether it should be concerned with solving the community problems.
In a developing county like Egypt one can not agree with Joeseph Rykwert assertion that it is not the business of architecture to solve societies' problems. According to Rykwert, “the duty of the architect is to give physical form to social establishment, to provide the screens which the passer-by outside and the user participant within recognize as the demarcation lines of a social situation” [12].
Considering the objectives of the competition as well as the whole process, the question of how to disseminate the product of the competition, both intellectually and educationally, becomes an important one. This dissemination should be done in order to instigate and encourage a critical discussion on the architectural design education in Egypt that can be seen as the prime goal of the whole event.

CONCLUSIONS

In conclusion, the competition could be looked at as an attempt yet , it should be critically reviewed and assessed by others. The assumptions, objectives and questions that the competition raised while intended to be critical seems in need of more elaboration. For example the assumption of the ability of such competition to by pass the formal educational system (with its enormous inertial) needs to be reviewed.
Through the jury process and the entries review, it was apparent that technology has to be dealt with more seriousness in architecture in the Egyptian universities. It has to be explored and

thought of with an eye focusing on the people that existed in the context. Technology has to be dealt with as one of the elements that can instigate social change that work in harmony with and not dependant of other elements [5]. Yet, what is in stake here is architecture itself. How a student or a professor of architecture address the core subject of architecture and raise (or may be to start to) the most essential critical debate about it in a city and community that is full of unexplored ideas and understanding of this human art and necessity.
The ideas underlying the competition, the ambitions adopted and the challenges that were set to face, are in need for an objective assessment together with the competition process as a whole. This assessment needs to study the goals and objectives and measure the degrees by which they are met and if not how this can take place. This review process can lead to further development of the competition and may help transform it into a real force of change amidst the complicated situation of architectural education in Egypt.

Acknowledgements
I would like to thank prof. Dr. Abdelhalim who collaborated with me through the inception of the first idea and its developments, also who was a key in the whole process. I would also like to thank Omar Nagaty for his review and comments on the early stages of this paper.

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NATURAL VENTILATION INSIDE A ROOM MODEL THROUGH OPENINGS IN TWO ADJACENT WALLS

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ABSTRACT

Natural ventilation is a ventilation strategy that uses wind, heat, solar gain and pressure differential to move fresh outside air through an interior space. Using operable windows as well as other façade elements to provide ventilation air is an intuitive manner of improving the atmosphere within built environment. There are several ways to provide natural ventilation and minimize the use of mechanical force, thereby conserving electrical energy. Some of these methods utilize the stack effect, thermo-syphon, wind scoops and double skin building facades. The stack effect induces natural air flow by using the basic tenet that hot air rises. As heated air rises into a central tower or architecturally formed opening such as an atrium, it is gradually replaced by cooler air drawn in from the openings at the perimeter of the building. The fresh air is drawn through the building and exhausted at the roof level. Thermo-syphon ventilation relies on solar gain to increase temperatures in a specific section of a building to create buoyant air which then escapes near openings at the top the building. This can be accomplished with a south-facing double skinned façade, a solar chimney or other sunspaces near thermal mass. Wind scooping is a technology used to assist natural ventilation by harnessing the pressure of wind, basically pulling air through the upper part of the building. Wind scoops rotate based on wind direction to maximize natural ventilation. Cross ventilation in the building can also be accomplished by providing operable windows on opposite sides to facilitate air currents or induce pressure differential. In this work, the natural ventilation inside a room model was studied. For this work a test room model was built with dimension .02 x .02 x .02 m with a scale 1:15 to simulate the actual case. This model has opening at two adjacent walls to simulate the perpendicular air flow. The test model was provided with electric heater at the ceiling and connected with a power source to simulate the heat gain from solar radiation. Twenty five thermo-couples were fixed at different point in the heater surface and connected to a data-logger to measure the heater temperature. The room model was tested in a wind tunnel which was designed and built in the housing and building research center [9] to study the effect of air flow inside the room thought the openings on the average temperature of the heated ceiling. An experimental work was agreed to another scale model using a smoke tunnel which was designed and built in the housing and building research center [10] to give a clear picture of the streams of air flow inside the model. A CFD computer program was used to obtain the air flow velocity distribution inside the model in form of velocity contours and vectors. The results show the best model orientation which gives low average temperature, i.e. high air flow inside the room.

Keywords: natural ventilation, wind tunnel, CFD computer software program,

INTRODUCTION

Ventilation [1] is the process by which fresh air is introduced and ventilated air is removed from an occupied space. The primary aim of ventilation is to preserve the qualities of air. Sometimes, ventilation may also be used to lower reduce the temperature inside an occupied area .
Natural ventilation is the process of supplying and removing air by means of purpose-provided

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aperture (such as opened windows, ventilators and shafts) and the natural forces of wind and temperature-difference pressures.

Natural ventilation may be divided into two categories:

Controlled natural ventilation which is intentional displacement of air through specified openings such as windows, doors, and ventilations by using natural forces (usually by pressures from wind and/or indoor-outdoor temperature differences). It is usually controlled to some extent by the occupant. Infiltration is the uncontrolled random flow of air through unintentional openings driven by wind, temperature-difference pressures and/or appliance-induced pressures across the building envelope. In contrast to controlled natural ventilation, infiltration cannot be so controlled and is less desirable than other ventilation strategies, but it is a main source of ventilation in envelope-dominated buildings. Windows [2] should be designed to admit natural ventilation. The most important design issue is whether the windows provide single-sided or cross-ventilation to interior spaces. The amount of air that flows through a window (ventilation capacity) depends on the area and vertical distribution of openings. These depend on the way of the window opens.

Horizontal pivot windows offer the highest ventilation capacity:

For single-sided ventilation, place them as high as possible to exhaust warm air at ceiling level.

Air entering through the upper opening is directed toward ceiling, making night-time cooling more effective. Center vertical pivot windows have less ventilation capacity than horizontal pivot windows, but can act as wind scoops when wind direction is parallel to the building face

One can create natural cross-ventilation [3] by opening windows and doors, and adjusting the size and location of the openings to ventilate different parts of the building.

Inlets and outlets located directly opposite each other cool only those areas in between, in the direct path of the airflow. We can cool more of the home or business if we force the air to take a longer path between the inlet and outlet. Experiment with different patterns of window venting to move fresh outside air through all of the rooms in the home or business. This may involve leaving some windows closed if they interfere with air moving along a longer path.

Andy Walker [4] studied types of natural ventilation effects, He concluded the Design Recommendations for natural ventilation as follows:Maximize wind-induced ventilation by siting the ridge of a building perpendicular to the summer winds , naturally ventilated buildings should be narrow and each room should have two separate supply and exhaust openings.He concluded that Natural ventilation in most climates will not move interior conditions into the comfort zone 100% of the time. He recommended to use the mechanical ventilation also, especially in hot and humid climate. A. Roderick, et al [5] studied the natural ventilation for the prevention of air borne contagion. The study was carried out in eight hospitals in Lima, Peru; five hospitals were of “old-fashioned” design built pre-1950, and three of “modern” design, built 1970–1990. In these hospitals 70 naturally ventilated clinical rooms where infectious patients are likely to be encountered were studied. They concluded that, opening windows and doors maximizes natural ventilation so that the risk of airborne contagion is much lower than with costly, maintenance-requiring mechanical ventilation systems. Old-fashioned clinical areas with high ceilings and large windows provide greatest protection. Natural ventilation costs little and is maintenance free, and is particularly suited to limited-resource settings and tropical climates, where the burden of TB and institutional TB transmission is highest. In settings where respiratory isolation is difficult and climate permits, windows and doors should be opened to reduce the risk of airborne contagion.

Dana O. Porter, P.E. [6], studied the Ventilation in Poultry and Livestock Shelters. They concluded that the Design of ventilation systems is based upon four major considerations, hot weather conditions, cold weather conditions, humidity and Odor control, They also concluded that the Failure of a ventilation system can result in death by asphyxiation (from lack of oxygen and increased carbon dioxide), by heat prostration, by poisoning from harmful gases, or from a combination of these. J Axley, et al , [7] presents an approach to design the natural and hybrid ventilation systems that accounts for specific climatic and operational conditions and is organized to serve the building designer at each distinct phase of design. The approach makes use of: a) a climate suitability analysis method, presented in a companion paper, that establishes design ventilation rates needed for preliminary design calculations; b) the loop equation design method to estimate preliminary sizes of system components and control and operational strategies; and c) detailed multizone coupled thermal-airflow analysis, using CONTAM97R, for design development and, ultimately, system performance evaluation.

They concluded that, the design of natural and hybrid ventilation systems for cooling buildings is inherently different from the design of more conventional mechanical systems directed to the same purpose. This is due to that a part of this difference is related to the need to account for the natural driving forces provided by wind and buoyancy effects that these systems seek to take advantage of. F.CRON, et al [8] , studied experimental and numerical analysis of a hybrid-ventilated room. An experimental cell, called HYBCELL, was designed at the ENTPE-LASH Laboratory to provide both mechanical and natural ventilation. Various sensors were installed in the cell for the indoor environment, in the hall, and outside for the boundary conditions. Modeling was performed by ENTPE-LASH and LEPTAB. They find that, For both simulated seasons, hybrid ventilation showed better performances in terms of temperatures, CO2 concentration exposure and energy consumption than traditional ventilation systems. It can be a good solution for indoor air quality at lower costs, but it requires control strategy on both temperature and CO2. Some more case studies need to be performed, with another exposition to the sun and with a higher thermal capacity (possibilities of night cooling). Hassan et al [11] presented results of an investigation of natural ventilation criteria as affected by various factors are presented. In particular, the effects of opening location and size (window-to-wall ratio (WWR)) and building orientation relative to prevailing wind direction are considered. Flow visualization and finite element techniques were adapted to gain better understanding of these effects. Abed el Aziz et al [12] presented an experimental and computational investigation to study the flow characteristics and thermal behavior of air in a ventilated room model with different opening locations. The flow and thermal results were obtained for five opening locations (25 test runs) of both the front façade and rear wall. Wind tunnel tests were carried out to evaluate the thermal performance of a heated ceiling of models with different openings. At constant air flow, the temperature distributions on the heated ceiling were measured and heat transfer coefficient were calculated and averaged. A CFD program (ANSYS code 2001) was used to calculate the velocity distributions (contours and vectors) inside the ventilated model.
This work studies the effect of indoor natural ventilation through openings (in two adjacent walls) for reducing the ceiling temperature.

EXPERIMENTAL WORK

Since it is difficult to study the natural ventilation on an actual building with different window shapes and different air flow directions. An experimental study was agreed on a room model with a scale 1:15 of the actual case. This test model was fabricated using Plexi-glass with dimension 20x20x20 cm with two openings in two adjacent walls as shown in Fig. 1. The test model was provided with an electric heater supported at the ceiling to simulate the heat gain through the building roof by the solar radiation. A twenty five thermocouples were fixed on the heater plat to measure the temperature on different plat points surface. The study was agreed for different air flow directions in a wind tunnel and for different window to wall ratio (WWR) and the normalized average temperature (Tnorm) for each case was calculated. These experimental studies were agreed in the Housing & Building National Research Centre- Cairo – Egypt.
The normalized average temperature was calculated using the equation:
Tnorm = (Tsurface-Tair) /Tair where, Tsurface is the surface temperature of the heated ceiling, Tair is the air flow temperature.
The window to wall ratio was calculated from the relation:
WWR = (Window area / Wall area) /100 %

Computer Work

Since it is difficult to measure the air flow velocity distribution around and inside the room model, a software computer package (ANSYS CFD Flotran) was used to investigate the air velocity distribution inside the room model in the form of velocity contours and vectors. , The program is a three dimensional one, that utilize the finite element approach which uses the k-ε turbulence model and solves the Reynolds equations, the energy equation and the equations for turbulence energy and its dissipation. In the present work, the boundary conditions stipulated that the flow velocity at all the solids surfaces is zero (satisfying the real viscous fluid configuration). As shown in Fig. 3. Also the approaching velocity profiles were prescribed by logarithmic law.

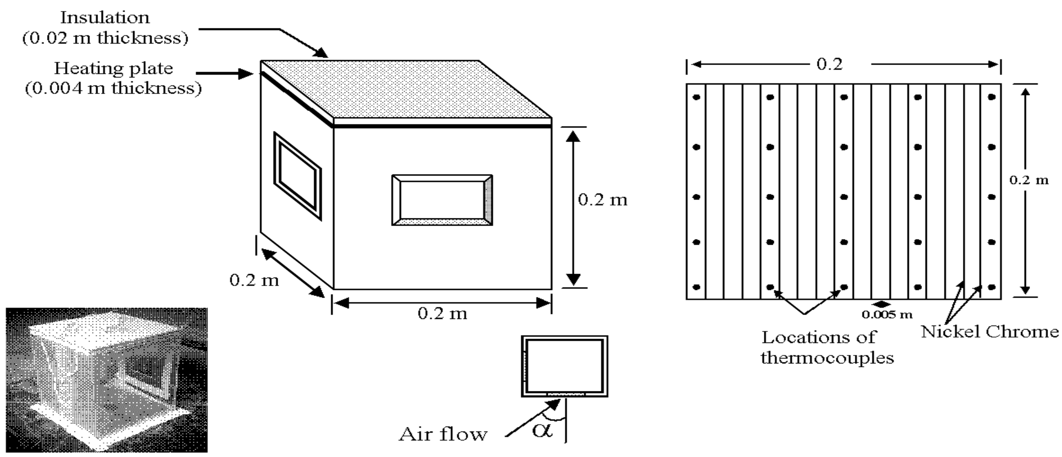
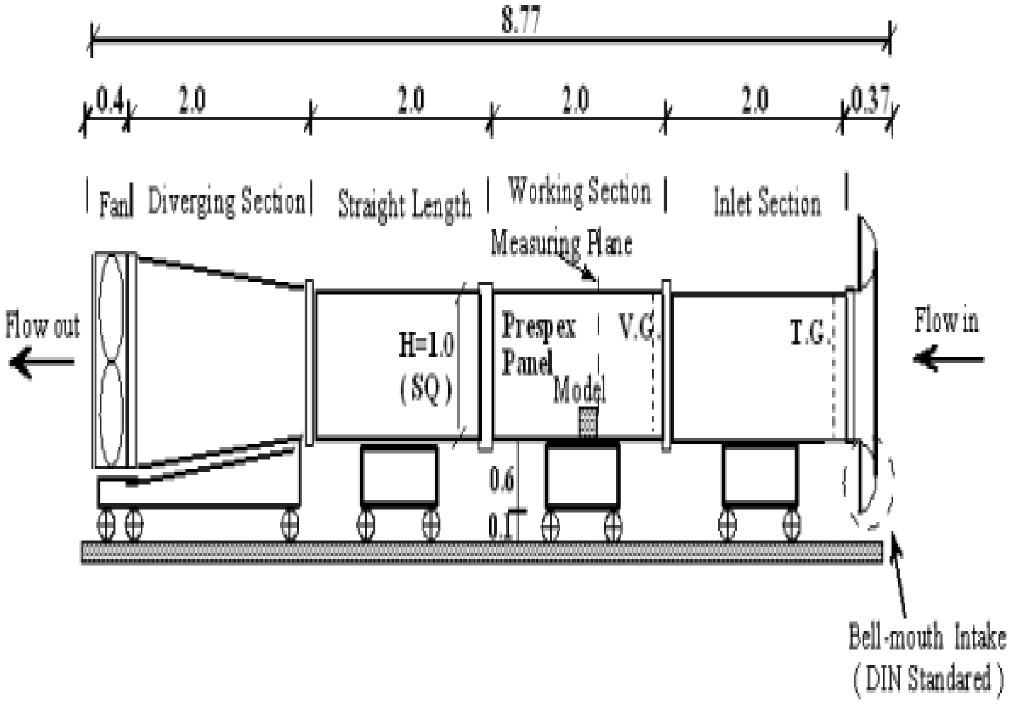


Fig. 1: The test model and the electric heater



CFD

Fig. 2: Sketch diagram of Wind Tunnel.

Three general assumption are assumed , the first that the fluid is Newtonian, the flow is a single-phase one and the solution domain is of constant geometry, in addition, the flow is steady incompressible and the body forces are neglected. In this paper an experimental study on a single opening for different air flow direction. And also experimental study on a test room with two windows in two adjacent walls for different window to wall ratio. The computational study is only for the two adjacent window model of WWR= 10 %. The experimental and computed program for WWR=10% is shown in Table 1.

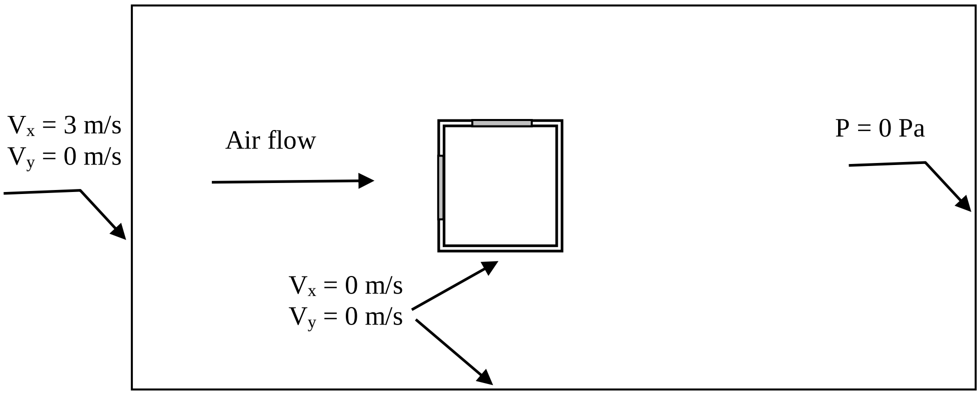
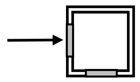
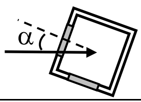
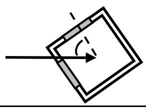
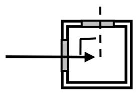
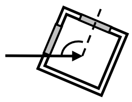
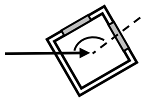
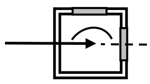
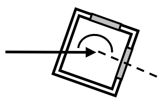
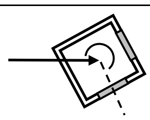


Fig. 3: Boundary condition on the test model and wind tunnel.

Table 1: Experimental and computed study of the test model with two openings of WWR=10% for different air flow angles.

study / Case No.	Air flow angle (□) degree	Experimental Wind Tunnel	Computational ANSYS (CFD)
Case 1	0º (90º) air flow (top view) 	done	done
Case 2	30º(60º) air flow 	done	done
Case 3	60º(30º) air flow 	done	done
Case 4	90º(0º) air flow 	done	done
Case 5	120º air flow 	done	done
Case 6	150º air flow 	done	done
Case 7	180º air flow 	done	done
Case 8	210º air flow 	done	done
Case 9	240º air flow 	done	done

RESULTS

Figure 4 shows the Experimental results of the normalized average temperature of the model ceiling with different air flow angles for one sided opening, in case of WWR=10%. It is noted that the temperature decrease until it reach its minimum value at air flow direction angle 30o with the man facade then increases at air flow angle 90o since the air flow passes parallel to the window so less air flow enter the room.

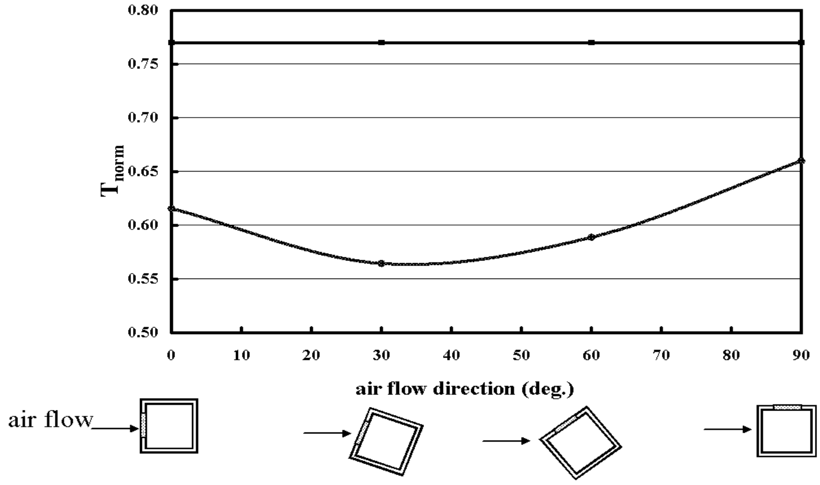
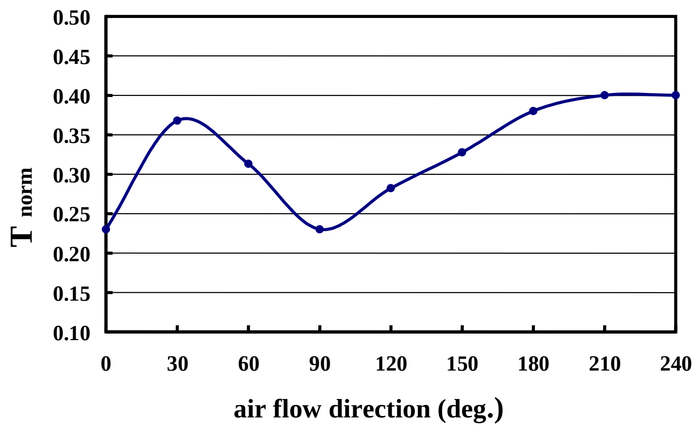


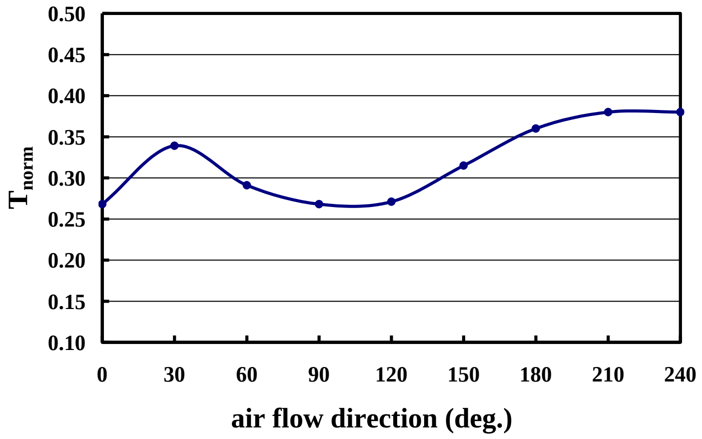
Fig. 4: Experimental results of the normalized average temperature with different air flow angles for one sided opening, (WWR=10%).

Figures 5-a,b,c,d represent experimental results of the normalized average temperature with different air flow angles for different window to wall ratio (WWR) of two adjacent sided windows. The figures shows that the temperature reaches its lowest values at air flow angles 0o.90o and 120o due maximum air flow enter the room at these locations, and the temperature reaches its maximum values at air flow angles 180o, 110o, and 180o since the two openings lies at the back ward of air flow and less air flow enters the room than the other cases. It is noted that the temperature values fluctuation decreases as the WWR increases so that the effect of the air flow direction with the model decreases as the WWR increase.

Figure (6) gives the experimental ceiling temperature contours (using Surfer program) and the corresponding computed velocity contours and vectors for case 1 (α = 0o, see Table 1), Top view plan at the centre of the model, (WWR=10%). In comparison between the experimental and computed results it is clear that the temperature contour has lower values (light contour color) through the air flow pass from the front window to the side one due to the maximum air flow showed from the computed air flow contour and vectors. The contour temperature increases at the back area of the room (dark contour color) due to the air flow has its minimum values in this area.

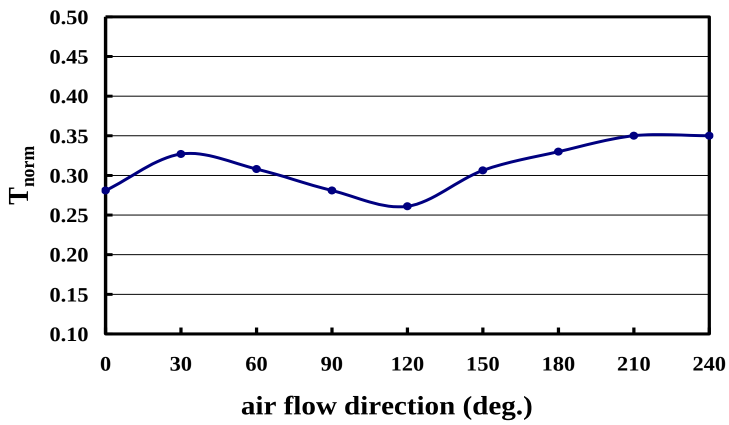


WWR 10 %

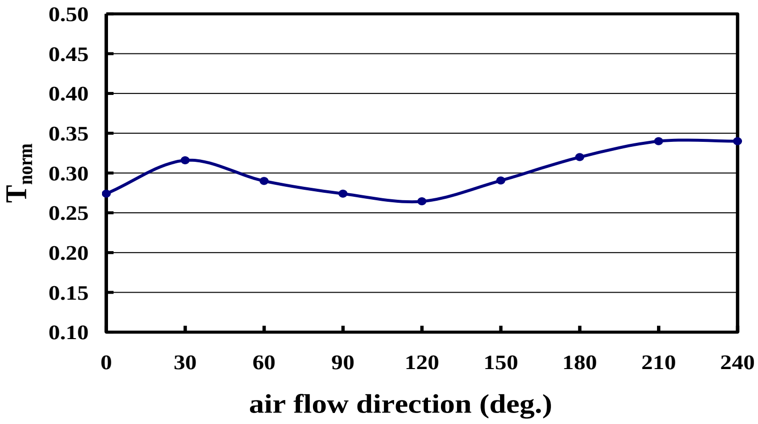


WWR 15 %

Fig. 5 (a, b): Experimental results of the normalized average temperature with different air flow angles for different window to wall ratio, (WWR) of two sided windows.



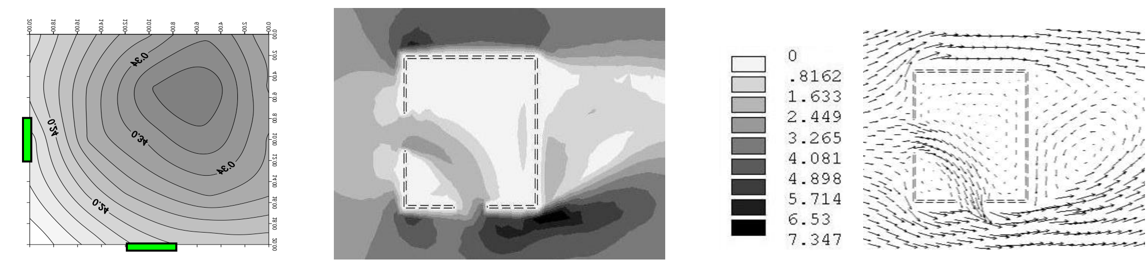
WWR 20 %



WWR 25 %

Fig. 5 (c, d): Experimental results of the normalized average temperature with different air flow angles for different window to wall ratio, (WWR) of two sided windows.

Air flow →



Experimental T_{norm} Computed velocity contours Computed velocity vectors

Fig. 6: Computed Velocity contours and vectors for case 1 (α = 0o), Top view plan at the centre of the model, (WWR=10%).

Figures From 7 to 14 give the experimental ceiling temperature contours and the corresponding computed velocity contours and vectors for different air flow angles , Top view plan at the centre of the model, (WWR=10%). It is clear that the surface temperature is affected with the air flow angles and gives a reduction in temperature through angles from 30o to 150o and the bad results of high temperature contours due low air flow through the openings at air flow angles 180o, 210o and 240o due to that the openings lies at the leeward of the building model.

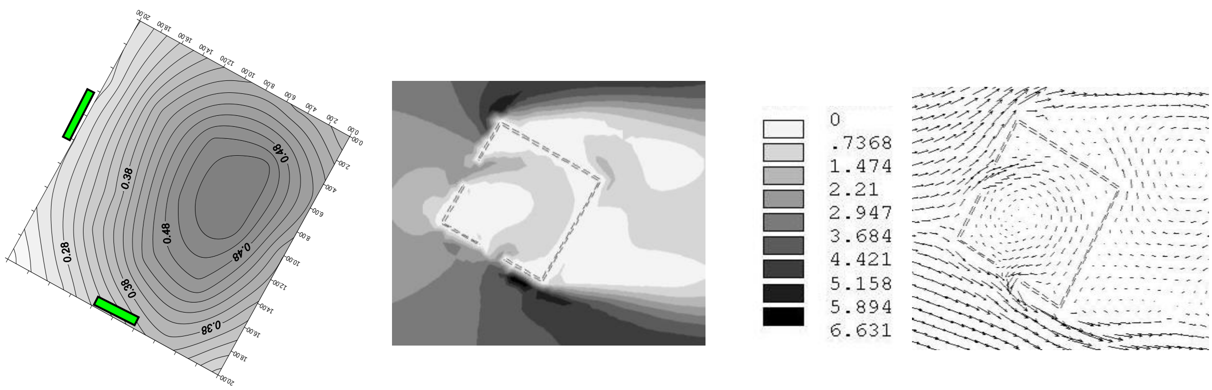


Fig. 7: Computed Velocity contours and vectors for case 2 (α = 30o), Top view plan at the centre of the model, (WWR=10%).

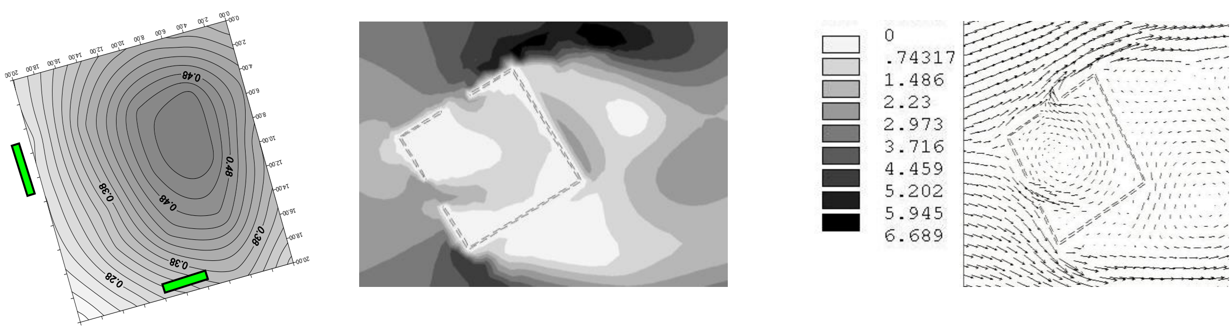


Fig. 8: Computed velocity contours and vectors for case 3 (α = 60o), Top view plan at the centre of the model, (WWR=10%).

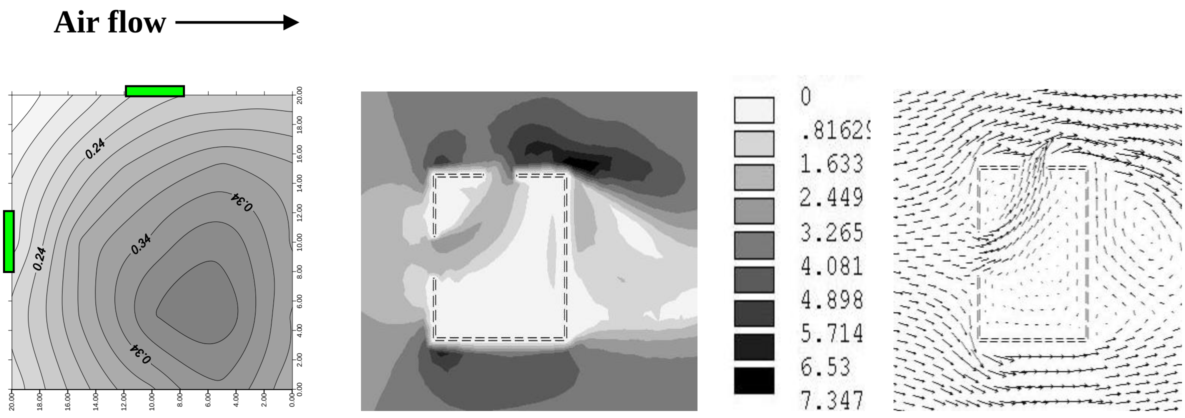


Fig. 9: Computed velocity contours and vectors for case 4 (α = 90o), Top view plan at the centre of the model, (WWR=10%).

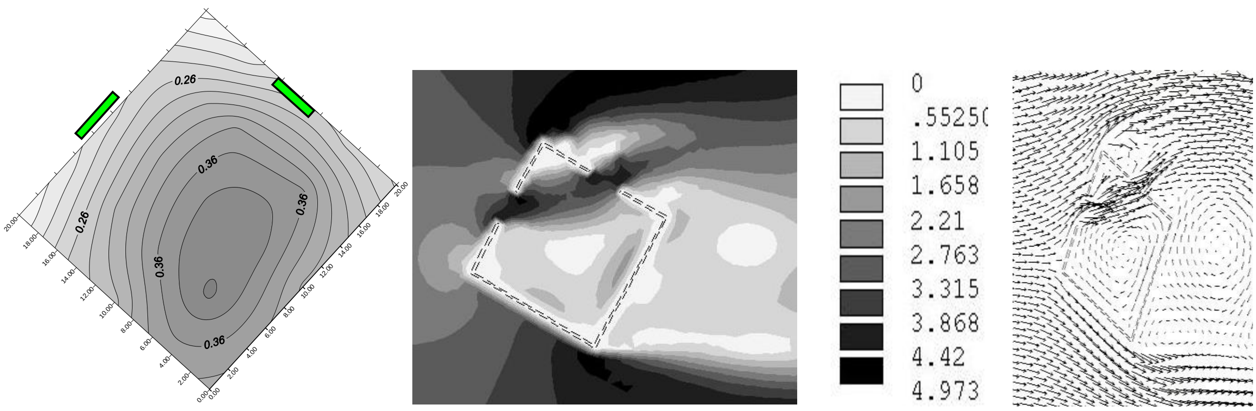


Fig. 10: Computed velocity contours and vectors for case 5 (α = 120o), Top view plan at the centre of the model, (WWR=10%).

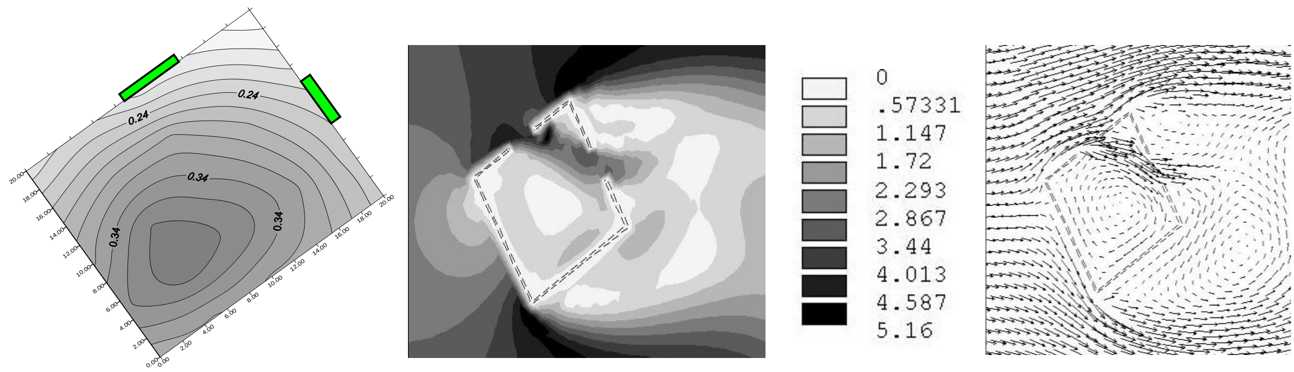


Fig. 11: Computed velocity contours and vectors for case 6 (α = 150o), Top view plan at the centre of the model, (WWR=10%).

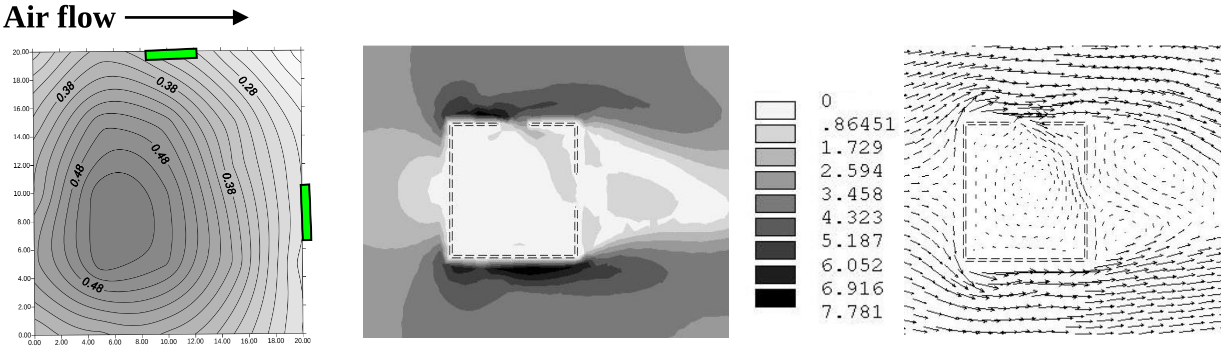


Fig. 12: Computed velocity contours and vectors for case 7 (α = 180o), Top view plan at the centre of the model, (WWR=10%).

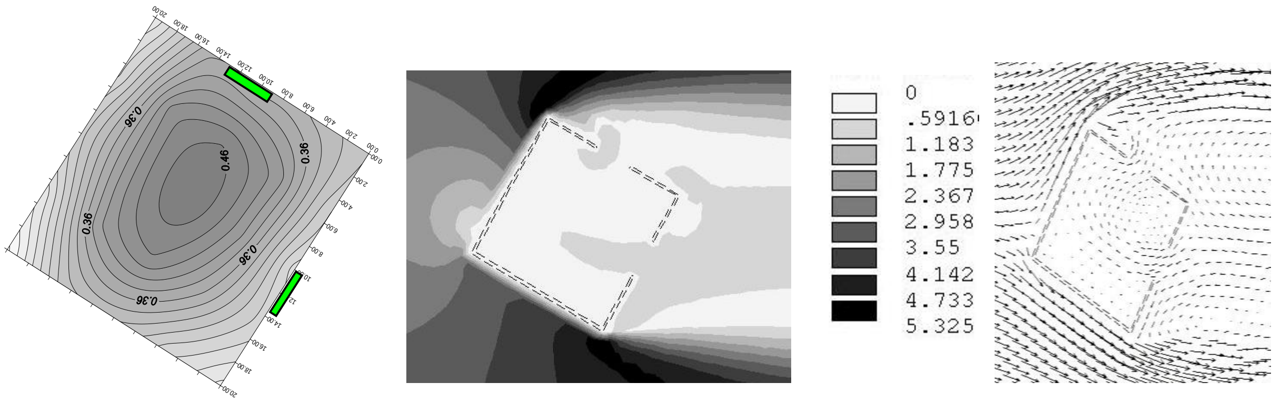


Fig. 13: Computed velocity contours and vectors for case 8 (α = 210o), Top view plan at the centre of the model, (WWR=10%).

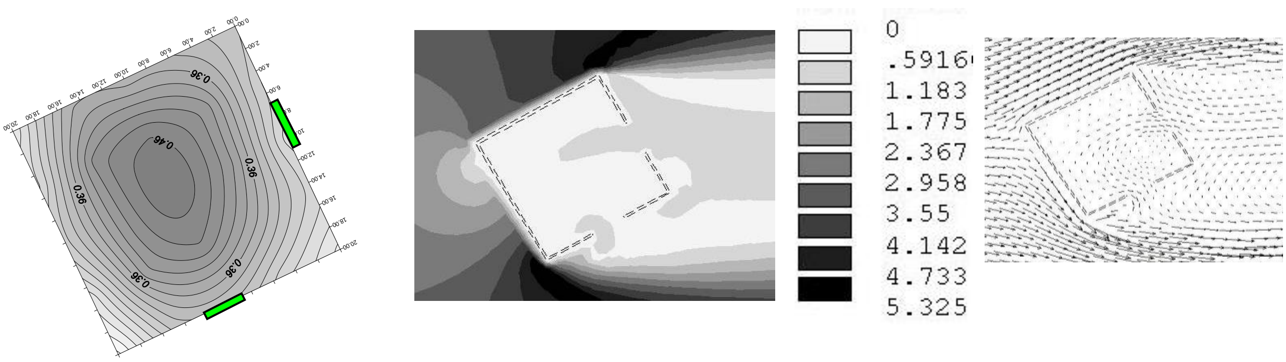


Fig. 14: Computed velocity contours and vectors for case 9 (α = 240o), Top view plan at the centre of the model, (WWR=10%).

Figure 15 represents the computed average velocity inside the test room of two openings for different air flow directions, (WWR=10%). It is clear that The velocity decreases through air flow angles 30o and 60o since the two openings are lies at the front of the air flow (i.e. has low pressure difference) and increases to its maximum vales at angle 120o Since one openings lies at the windward and the other lies at the backward (i.e. high pressure difference). The average velocity decreases gradually as the air flow angles increases from 150o up to 240o as the two windows lies completely at the wake region of the test model (low pressure differences). These results are compatible with the experimental results shown in Figure 5-a since as the average velocity inside the room increases the indoor or ceiling temperature decreases.

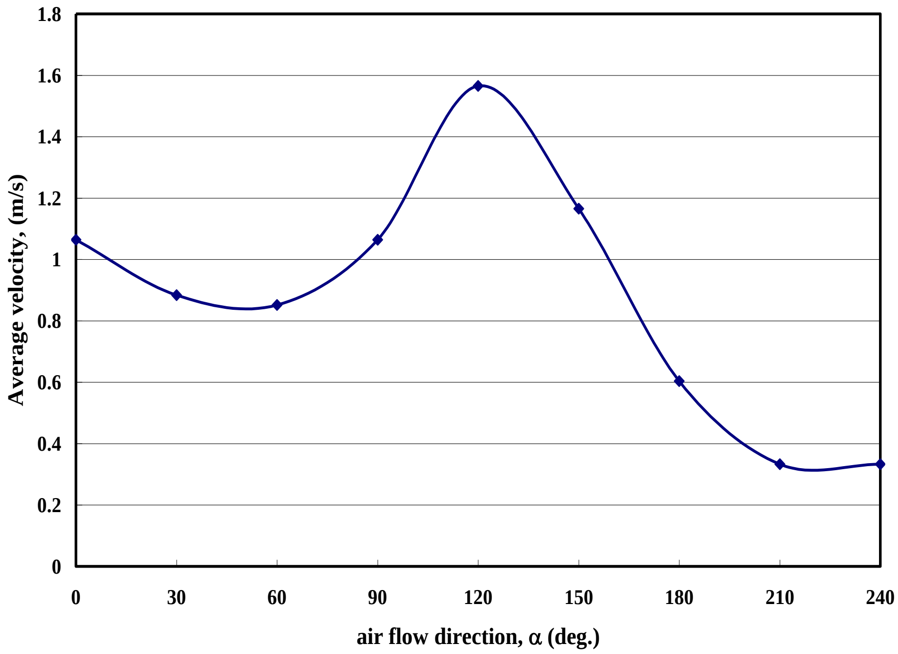


Fig. 15: Computed average velocity inside the test room for different air flow directions, (WWR=10%)

CONCLUSIONS

From the experimental and computed results it can concluded that :
The natural ventilation is an important factor in reduction of indoor and wall temperature.
The one side openings yields to bad ventilation (high temperature) that needs to another openings in the same wall or increasing the opening width.
For two openings as the WWR increases the variation of the ceiling temperature with air flow direction decreases, i.e. the effect of orientation on room ventilation decreases as the WWR increases.
For two openings at two adjacent walls the openings at the windward gives low ventilation effect as well as as the two openings lies at the backward of the wind flow direction.
As one of the openings lies in the windward and the other lies in the backward of the air flow direction this give maximum air flow inside the room and in accordance low room temperature.
There is a good agreement between experimental and computed results.

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Economic Study of Thermal Insulation in Egypt

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ABSTRACT

This paper studies the possibility of improving the thermal performance of buildings through using different types of thermal insulating materials and construction as insulating systems to conserve energy from envelope of residential and commercial building sectors in Egypt. Different types of insulating materials have been investigated in the laboratories of Housing and Building Research Center to determine the important physical and mechanical properties of building materials, insulating materials and insulating systems. In addition, a feasibility study has been carried out in the Egyptian markets to study the payback period of different types of thermal insulating materials. The results of this investigation demonstrated that most of the local building materials in Egypt have a thermal resistance lower than that required for thermal resistance determined by the Egyptian code of energy. These local building materials need to work with thermal insulating materials to achieve human comfort and conserve energy. The main function of thermal insulation is keeping the mean radiant temperature of indoor surfaces constant. This function reduces the energy consumption for heating and cooling loads in buildings. Insulating materials such as refractive, cellar, flag and lightweight concrete are the best materials used in insulating systems through Egypt from north to south.

Keywords : Insulation materials, Thermal Conductivity, Heat transfer, Building materials, Energy comfort.

INTRODUCTION

Before the oil crisis of 1973, the energy efficiency of the envelope components was not regarded as important factor in the design of buildings. However, since 1973 several standards and regulations have been developed and implemented to improve the energy efficiency of various components of building envelopes. The energy efficiency of building envelope has appeared in different energy codes and in many researches (1). The insulating materials are the common factor that appeared in these codes and researches to improve the thermal behavior of envelope beside other factors such as orientation_ etc (2).

At present, insulating materials are the best solution to conserve energy in the building after the air conditioning devices have been used on a large scale in Egypt. As the thermal resistance of the envelope of buildings is increased, the energy dissipated for heating and cooling is decreased. Many studies were carried out in recent years, the results of which show that insulating materials must be added to local and new building materials to improve the thermal characteristics of construction elements under the effect of external climatic conditions of hot regions (3). Also, the technology of thin film, and glass technology introduced a new glazing which has better thermal insulation characteristics compared with glazing normally used in buildings. One of the new studies concludes that insulating materials pay itself quickly in the field of insulation of building envelopes (4)

Energy efficiency improvements of building envelope systems are generally expensive and are not cost-effective especially for large commercial buildings. However, increasing the energy performance of building shell can be justified for low-rise and small buildings based not only on

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energy cost saving but also on improving thermal comfort and integrity of the building structure indoors. For residential buildings, use of weather –stripping system to reduce infiltration losses is always economically justifiable (5 – 10).

Climate and Comfort Zone in Egypt

Climate strongly influences human health. Human thermal comfort zone is a narrow one that lies between lower and upper limits of effective temperature. In this zone, man can work without any strong influence on health and longevity and hence, can contribute, with others for development. The most important parameters, which determine the state of thermal comfort, are air temperature, mean radiant temperature, air humidity, relative air velocity, clothing and activity of human. Fig. (1) Shows the variation of the human comfort zone with mean radiant temperature and air temperatures. This figure also shows that the important function of thermal insulation in the building is to control the mean radiant temperature of the internal surfaces. Many experiments were carried out to determine the effect of radiant energy from surfaces such as walls and ceilings. The results of this study shows that thermal insulating materials play a significant role in controlling the mean radiant temperature indoor surface and this helps to reach the comfort feeling and also to save energy (11, 13).

Building Materials in Egypt

There are many types of building materials in Egypt. The first type is the natural building material such as limestone, sandstone, marble and other decorating stone called pharoahnic stone. The second type of building material is dependent in its manufactory on the shell clay. The third type is dependent on cement such as cement bricks and cement blocks that appeared in the last few decades and are used as insulating materials with cement blocks, commonly available in the Egyptian markets and cold isoblocks. The study of the thermal properties of this material demonstrated that all the building materials in Egypt have a low thermal resistance. Table (1) (14,15)demonstrated the thermal properties and characteristics of building materials in Egypt.

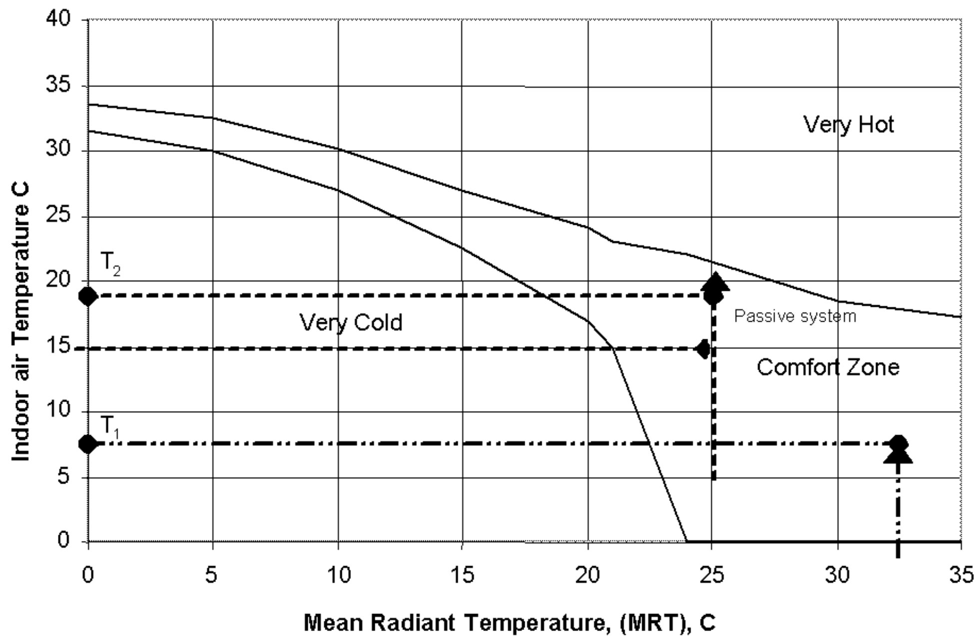


Fig 1 : The variation of comfort zone with respect to the mean radiant temperature and indoor air temperature

Table 1: Thermo-Physical Properties and Characteristics of Building Materials

Including Some Insulating Materials

Building Materials	Thermo-Physical Properties			Thermo-Physical Characteristics (Wall Thickness 30º mm)	
	Density, ρ (kg/m^3)	Thermal Conductivity, K (W/m °C)	Specific Heat, C_p (J/kg °C)	Thermal Resistance, R (m^2 °C/W)	U-value (W/m^2 °C)
Natural Building Materials					
Limestone	1600-2100	0.73-1.10	840	0.46-0.60	1.70-2.20
Sandstone	1800-2200	0.97-1.10	840	0.38-0.50	2.00-2.60
Marble	2600	2.60	880	0.31	3.20
Granite	2800	3.50	900	0.28	3.60
Sand	1520	0.43	800	0.70	1.40
Gypsum	1200	0.93	1080	0.51	1.96
(b)Cementing Materials					
Cement Bricks	1600-2000	1.30-1.50	880	0.39-0.42	2.40-2.60
Hollow Cement Bricks	1200-1500	0.80-1.00	880	0.49-0.57	1.80-2.04
(c) Clay Bricks					
Clay Bricks	1850-2000	0.55-0.65	830	0.65-0.74	1.35-1.54
Hollow Clay Bricks	1450-2500	0.45-0.65	830	0.65-0.85	1.18-1.54
Lecka Bricks	1000-1300	0.35-0.45	830	0.67-1.10	0.90-1.49
(d) Sand Bricks					
Heavy Sand Bricks	1800	1.60	840	0.38	2.60
Light Sand Bricks	600	0.30	840	1.19	0.84
(e) Insulating materials					
Foam Concrete	450-515	0.18-0.21	1000	1.86	0.53
Light Concrete	800	0.275	1000	1.28	0.78
Celton	350-450	0.09-0.12	550	3.50	0.29
Expanded polystyrene	14-40	0.037-0.032	1200	8.10-9.40	0.10-0.12
Extruded Polystyrene	25	0.03	1200	10.19	0.09
Polystyrene Sheet	30	0.027	1100	11.30	0.09
Roockwool	140	0.040	660	7.69	0.13
Glasswool	52	0.038	660	8.10	0.12
Perlite	34-192	0.04-0.05	1200	7.07	0.13
Vermiculite	100-240	0.06-0.08	1200	4.5	0.22

Mechanism of Heat Transfer

Heat flow must have a path or means of transfer or transmission, which are conduction, convection and radiation.

The Mechanism of Heat Transfer through Building Materials

Building materials are massive materials. The solar energy is absorbed in the external surfaces of building elements and also heat transfer from surrounding environments takes place. The propagation of heat by conduction through the building construction elements is dependent on the diffusivity ($k/\rho c$) of the materials as well as the heat capacity. Time lag, decrement factor and damping factor of the propagating waves are the important factors affecting the transfer of heat from hot surface to cold surface of the building element. The thermal properties of building elements such as thermal conductivity, specific heat and density are considered to be the basic thermo-physical parameters used in calculation of the other characteristics.

The Mechanism of Heat Transfer through Insulating Materials

(a) Mass Materials

Understanding the mechanism of thermal insulation (or how it works) is essential for proper selection and use. With the exception of the refractive type, thermal insulation depends upon totally enclosed, very small pocket bubbles in the materials containing air or gas. These contained pockets of air or gas retard the flow of heat. These small pockets are formed in the material itself. Each pocket must be sufficiently small to cause considerable resistance to air or gas flow so that little heat is transferred by convection from one side. The path which heat transfer, through the solid matter, must be long enough to limit the amount of heat by conduction through solid portion of the mass. Also, the solid materials must be opaque to reduce heat transmitted by radiation. See Fig. (2a).

The mechanism of heat transfer through the mass insulation is explained according to the different paths of heat through the mass insulating materials as follows:

1. The path of heat through the solid material by conduction (Q_{cond}).
2. The microconvection through the air or gas, which is found in the small Pockets and it dependant on the degree of freedom of air or gas ($Q_{mic\ conv}$).
3. The macroconvection through the air or gas, which found in the small pockets ($Q_{mac\ conv}$).
4. The heat transfer by radiation between the walls of pockets (Q_{rad}).

The amount of heat path through the insulating materials is the summation of the amount of heat path by conduction, micrconvection, macroconvection and radiation and they are written in the mathematical form as follows:

$$Q_{tot} = Q_{cond} + Q_{mic\ conv} + Q_{mac\ conv} + Q_{rad} \qquad (1)$$

The thermal conductivity of insulating materials is the apparent conductivity of different kinds of heat transfer path through the mass insulation.

$$K_{ap} = k_{cond} + k_{mic\ conv} + k_{mac\ conv} + k_{rad} \qquad (2)$$

It is clear from the above that the mechanism of heat transfer through mass insulation is dependant on the area and type of solid materials, the volume of the air or gas inside the poures system of materials and obviously, the type of cell (closed or opened) and also on the optical properties such as emittance of the wall of cells.

To improve the thermal properties of any mass insulation there are 4 ways to do so:

1. Reducing the mass of the solid part, this means that lower density materials means low thermal conductivity.
2. Increasing the ratio between closed cell relative to the open cell. This means that if the closed cell increases, the mean free path of the air or gas decreases, i.e., limited free convection.
3. Increasing the emmisavity of the internal walls of cells. This phenomenon depends on the optical properties of the internal walls of the cells. As emmisavity decreases, the thermal conductivity decreases.

(b) Refractive insulation

Refractive insulation is that which obtains its major resistance to heat transfer by sheet material of low emittance and absorbtance. The amount of heat transfer by radiation can be calculated as follows:

$$Q = 4\ \sigma\ f_{12}\ F_{12}\ \Delta\ T^3_{avg}\ \Delta T \qquad (3)$$

Where:

σ : Stafen Boltzman`s constant

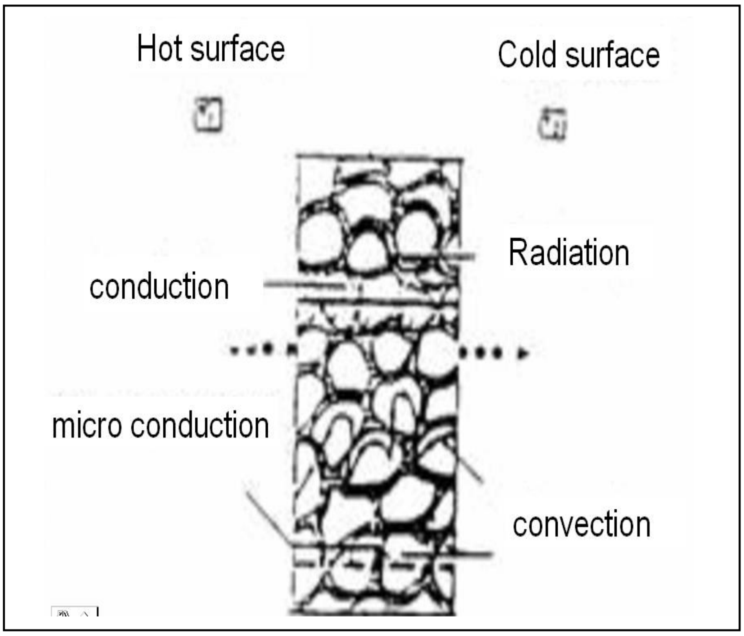
f_{12}: emissivity factor

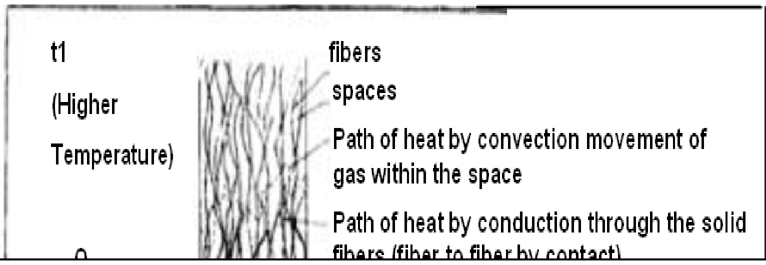
F_{12}: view factor

ΔT_{avg} :

To raise (increase) the efficiency of this type of insulation materials, only the emmisavity factor is considered.

The use of thermal refractive insulation in the field of glass improves the thermal properties of glass by about 200 – 300 % relative to single glass. The super-insulating glass has often three glazing layers, one of which is of selective low- emissivity long-wave radiation. This coating reduces the radiant heat loss from glazing, which is in the long-wave part of the radiation spectrum (see Fig. 2b).





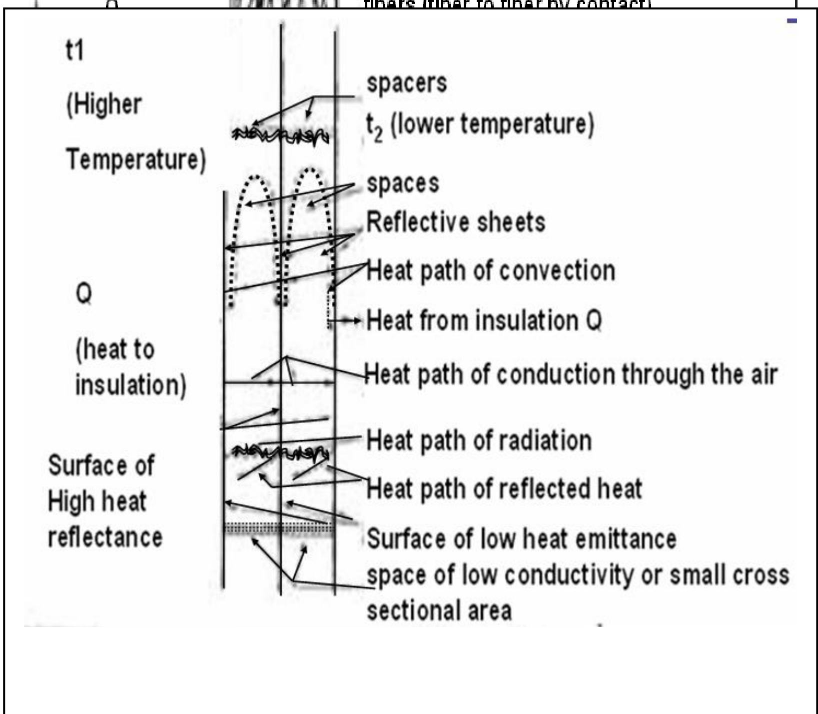


Fig. 2: Mechanism of heat transfer through different types of thermal insulating materials

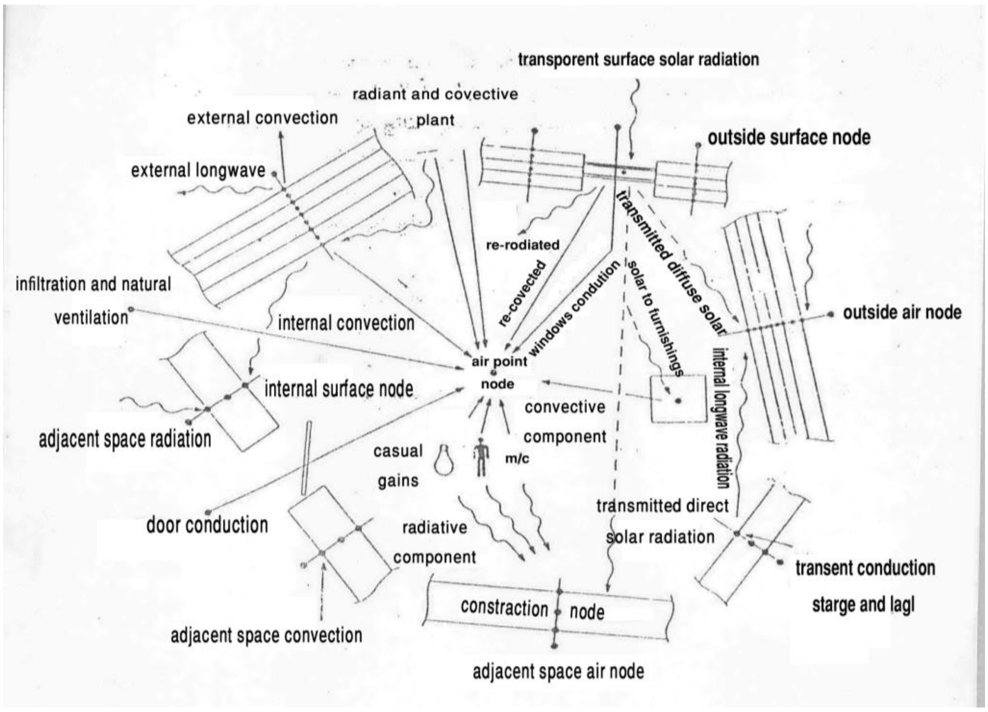
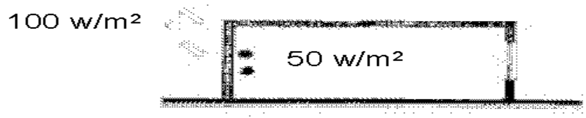
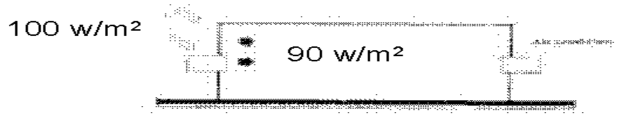


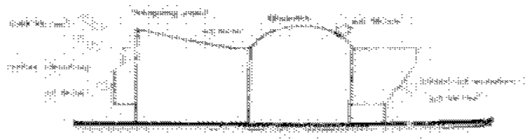
Fig. 3: Different paths of heat through building



First entrance: Thermal Insulation



Second Entrance: Air Conditions



Third Entrance: Passive Systems

Fig. 4 : Different considerations for human comfort in building

(c) Vacuum insulation

In this type, the transfer of heat is dependent on the reduction of transfer of heat by microconvection and macroconvection. The distance between the thermal surface of the vacum insulation is more important to be considered. This type of insulation can be used to reduce the heat transfer through windows of buildings. New technology of glazing reduces the heat transmattance from 6.4 to 0.9 W/m^2c° .

(d) Paths of Heat through Buildings

The thermal insulating materials play a significant role in solid and transparent parts. In the solid part, mass insulation is used to reduce the flow of heat through this part. Refractive insulation technology is used in the transparent part (see Fig. 3). The new technology of glass demonstrated that the U value of the glass is reduced from 5.6 $W/m^{2\circ}C$ to 0.9 $W/m^{2\circ}C$ and this is equivalent to a 400 mm thick clay brick wall.

RESULTS AND DISCUSSION

Different thermal studies of buildings aimed at finding harmony with human comfort zone. There are three strategies to make the indoor climate nearly or within the human thermal comfort under the external climatic conditions of Egypt (see Fig. 4). The first strategy is using thermal insulation materials. In this case, economic study of thermal insulating materials is extremity important. The second strategy is using the air condition and also thermal insulation is more necessarily to save energy. The third one is using the passive system and also thermal insulation must be used in this method. It is clear that the addition of thermal insulation materials is the major factor for all the above mentioned strategies.

Figure (5) illustrates different types of thermal insulating materials according to their composition and nature. In Egypt, the classification of the insulating materials is considered according to their composition and classification. Tables (1&2) demonstrate the location and mass production of different types of insulating materials in Egypt. It is clear that most materials are produced in northern of Egypt. Expanded polystyrene is considered the thermal insulating materials commonly used in Egypt. Up to 20 factories produce the expanded polystyrene with different densities and different shapes. Expanded polystyrene is used in external roofs to increase the thermal resistance of this roof. The Egyptian Code of Energy demonstrated that all the exposed external roofs must be insulated by about 5 cm using insulting materials having a thermal conductivity of about 0.032 $W/m^{\circ}C$ and this value is considered with the value of thermal conductivity of expanded polystyrene and extruded polystyrene The second material, used in insulating walls and roofs, is the extruded polystyrene.

Part of the extruded polystyrene is imported while the remaining part is produced locally on Egypt. The tables also demonstrate that the thermal properties of the two materials expanded polystyrene and extruded polystyrene are nearly the same at the density of 20 - 35 kg/m^3. While the capital cost of 1 m^3 of expanded polystyrene reaches about 240 L.E. During the period of this study (2004), the capital cost of extruded polystyrene reached to about 720 L.E. The best property, which may lead to the selection extruded polystyrene, rather than expanded polystyrene is the water absorption. The experimental results carried out in different laboratories show that the water absorption of extruded polystyrene reaches about 1 % from the volume while it reaches 2.5 % in the case of expanded polystyrene relative to the volume after 28 days. Bassive and Daw Companies's carried studies on expanded polystyrene in Europe. The results of this studies show that the expanded polystyrene can work to about 30 years without any change in its thermal properties, shape and dimensions.

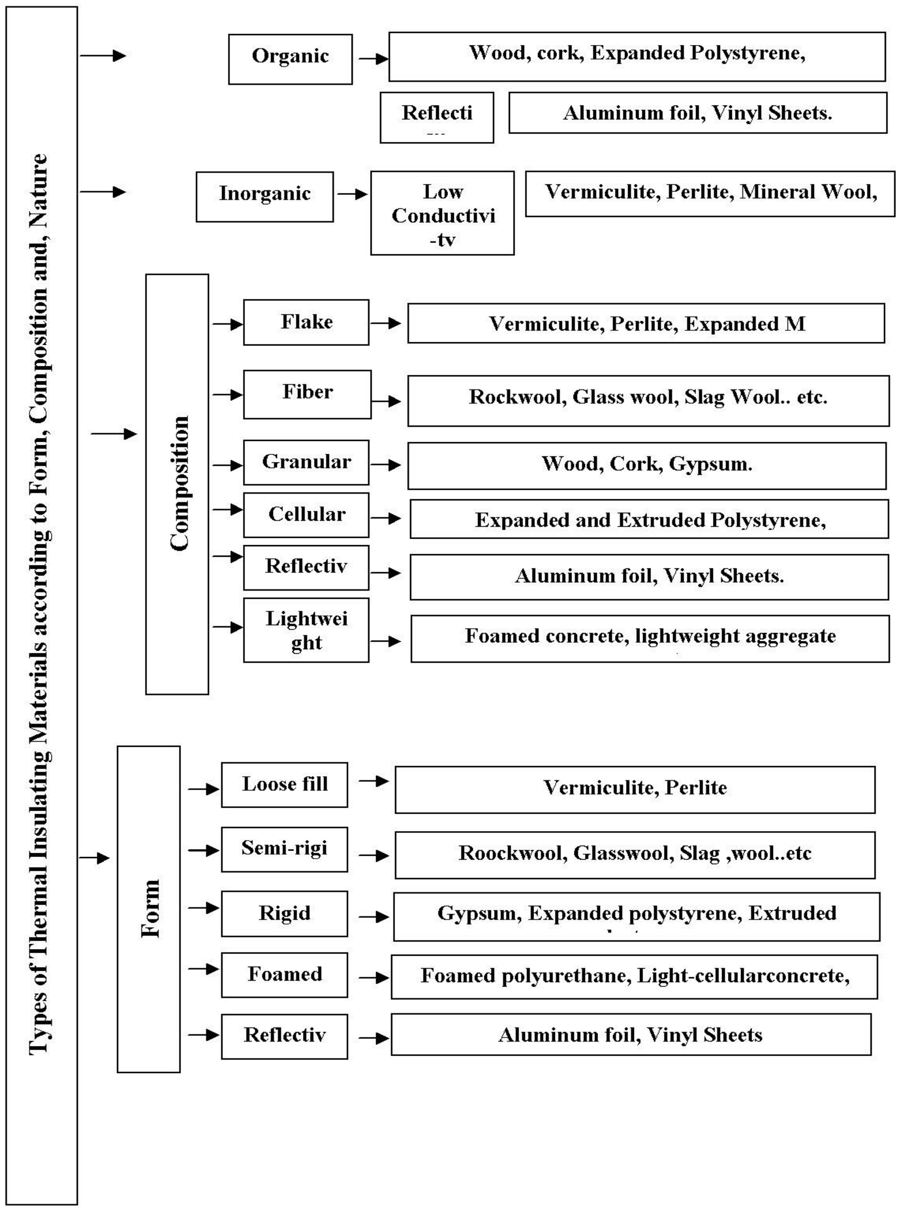


Fig.5: Different types of thermal insulating materials according to their form, composition and nature. in Egypt

Moreover, perlite, vermiculite and celton are insulating materials, which are mixed with cement to produce insulating concrete. The thickness of this mix reaches about 10 cm to cover R-value required by Egyptian code of Energy. Also, perlite, vermiculite are used perfectly in the air gab and are considered the best insulating materials because they increase the R-value of walls and reduce the U-value to about 50 % and in this case, the cost is considered economic. In addition, in hot regions, it is easy to use the perlite and vermoclite without any risks.

Table 2: Location and Mass Production of Different Types of Thermal Insulating Materials and their Prices during 2003 in Egypt.

Raw Materials		Location of Raw materials	Manufacturing in Egypt	Productions (ton/day)	Selling Prices (L.E)
Loose fill	Vermiculite	Egypt	Cairo		200 L.E/m^3
	Perlite	-	Borg-Elarab	10	300 L.E/m^3
Semi-rigid	Roockwool	Egypt	Helwan	5-7	3000 L.E/ Ton
	Glass wool	Egypt	Helwan 10-Ramadan	5-7	5-15 L.E/m^2
	SLAG Wool	Egypt	Helwan	2	-
Rigid	Gypsum board	Egypt	Alexandria	-	-
	Expanded polystyrene	-	10 – Ramadan	3	90-240 L.E/m^3
	Extruded polystyrene	-	Different places in Cairo and new cities	5-20	550-600 L.E/m^3
Foamed	Foamed polyurethane	-	Building site	According to needs	500-600 L.E/m^3
	Light-cellular concrete	Egypt	Biulding site	According to needs	120-150 L.E/m^3
	Foamed glass	-	-	-	-
Reflective	Aluminum foil,	Egypt	10th of Ramadan 6th of October	According to needs	2-3 L.E/m^3
	Vinyl Sheets	-	-	-	-

*The prices of thermal insulating materials depend on the exchange rate against the dollar.

CONCLUSIONS

From the above study, it can be concluded that:

1.Insulating materials pay for itself in the building sector.

Insulating materials play a significant role in reducing air conditioning instruments capacity and help the building to become comfortable during few months of the year.

2.The results of this investigation show that expanded polystyrene, extruded polystyrene, perlite and vermiculite are the best insulating materials used in the building sector in Egypt.

3.The demands for thermal insulating materials increase as one heads from north to south of Egypt due to the increase in solar intensity and air temperature.

4.The results of this investigation demonstrate that reflective insulation is more suitable for roofing in the south of Egypt.

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