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EFFECT OF SPECIFIC SURFACE AREA OF CEMENTS ON THE MECHANICAL PROPERTIES OF MORTAR

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ABSTRACT

This work is a contribution to the improvement of the properties of the mortars by mechanical activation of two types of cements (C.E.M II) manufactured in various cement factories (cements with various mineral additions : slag and tuff). The physical properties of cements (C.E.M II) activated mechanically at anhydrous state and the hydrated state (specific weight, consistency of the cement pastes and setting times), thus the characteristics of the mortars made at their bases, such as, the mechanical behavior (Flexural and compressive strengths for the mortar) were studied. According to the experimental results obtained, it comes that the increase of the specific surface and the chemical composition of cements with the mineral additions are the principal responsible to the improvement of the latent reactivity of mineral additions and increase the mechanical strengths (flexural and compressive) of the mortars.

Keywords: Specific surface, Mineral additions, Cement, Mortar, Mechanical strength.

INTRODUCTION

Over the recent period, mechanical activation and its positive influence on hydraulic properties of inorganic binders were the subjects of the numerous investigations [1,2]. It was found out that various characteristics of cement : behavior during solidification, compressive strength and bending strength can be improved by grinding. In fact, the hydration reactions of the clinker minerals are prompted, thus contributing to improved solidification and improved mechanical properties of cement. It is also well-known that various additives (slag, ash, tuff) are used in production of cements, both for their unkown hydraulic properties and for reduction of wastes. Having in mind that these additives can degrade the cement quality, particularly its mechanical characteristics, the amount of additive is usually restricted to only certain percentages [3].
The Portland cement (C.E.M II) with mineral addition presents a hardening slowed down at its initial period in comparison with an ordinary Portland cement (cement without secondary component : C.E.M I) [4]. This latent property of cement with mineral addition (C.E.M II), requires the use of a effective activation, chemical, mechanical or thermal [5].
The cements with mineral additions (C.E.M II) have a latent setting times than ordinary Portlands cements (C.E.M I), especially in the case of concreting in cold weather. It is known that setting times can be shortened :

* by high fineness (specific surface area) of cement,

* or by the use of accelerating admixtures (NaOH, KOH,......).

The objective of this study is to evaluate experimentally the influence of the fineness (specific surface area) of cements with mineral additions on the mechanical properties of the mortar. This work is a contribution to the improvement of the properties of the mortars by mechanical activation of two types of cements (C.E.M II) of various cement factories (cements made with various active and inert mineral additions : slag and tuff).

MATERIALS

Fine aggregates (natural sand)

The sand's equivalent measured by the NF P18 standard [6] shows that the dune sand used in this experimental study was clean, siliceous and contains very few fine dust or clayey elements. The fineness modulus calculated was M_f = 1.73.

The information on the physical properties of the natural sand used is given in Table 1.

Table 1: Characteristics of dune sand used in the tests

Materials	Absolute density	Apparent density (Kg/l)	Compactness (%)	Porosity (%)	Sand equivalent value (sight/test)
Dune sand (0/3)	2,56	1,64	64,06	35,94	76/77

Cements

Two types of Portlands cements (CEM II) of various cement factories (cements with various mineral additions : slag and tuff) were used in this experimental study who are :

* CEM II/A of Sour-El-Ghozlane (clinker : 85%, gypsum : 5% and tuff : 10%).
* CEM II/B of Hadjar-Soud (clinker : 65%, gypsum : 5% and slag : 30%).

These two types of cements studied are CEM II/A and CEM II/B, which presents different chemical compositions (different clinkers and mineral additions).
Each type of used cement was ground in a grinder in order to obtain various fineness (different specific surfaces area) while varying the time of grinding from 15 to 30 minutes :

1st fineness (F1) : t = 0 min (initial fineness of the cement factory) : S.S.A (CEMII/A) = 3206 cm^2/g and S.S.A (CEMII/B) = 3442 cm^2/g.

2nd fineness (F2) : t = 15 min (time of grinding is equal to 15 minutes) : S.S.A (CEMII/A) = 4452 cm^2/g and S.S.A (CEMII/B) = 4434 cm^2/g.

3rd fineness (F3) : t = 30 min (time of grinding is equal to 30 minutes) : S.S.A (CEMII/A) = 5038 cm^2/g and S.S.A (CEMII/B) = 5068 cm^2/g.

Table 2 present the type of various mineral additions used for the cements studied.

Table 2: Mineral admixtures of cements studied

Ciments used CEM II	Clinker (%)	Set regulator	Contents (%)	Admixtures	Contents (%)
CEM II/A	85	Gypsum	5%	Tuff	10
CEM II/B	65			Slag	30

The chemical composition of the two types of cements used in this research have been determined by the testing method "X-ray Fluorescence Spectrometry (XRF)". Table 3 gives the chemical composition of the two cements used in this experimental work.

Table 3: Chemical composition of studied cements

Types of cements used	SiO_2 (%)	Al_2O_3 (%)	Fe_2O_3 (%)	CaO (%)	MgO (%)	K_2O (%)	Na_2O (%)	SO_3 (%)
CEM II/A	23,48	5,63	3,28	60,60	1,38	0,97	0,19	1,18
CEM II/B	24,77	5,81	3,68	64,03	1,19	0,93	0,19	1,19

Two types of cements (CEM II) manufactured with different mineral additions (slag and tuff) were used to analyze the influence of the specific surface area of hydraulic cements at various mineral additions on the physical characteristics of hydraulic cements at anhydrous state and the hydrated state and also on the mechanical behavior (flexural and compressive strengths for the mortar). The S.S.A of hydraulic cement with mineral admixtures studied was determined by air permeability apparatus.

RESULTS AND DISCUSSION

Influence of the fineness on the specific weight of cement

Figure 1 presents the effect of fineness (specific surface) on the specific weight of cement. From the results obtained (Figure 1), the following conclusions may be drawn :

* a significant difference of the specific weight between the various studied cements.
* a reduction of the specific weight with the increasing of the fineness of cement.

The difference observed between the specific weights of studied cements, depends of the nature of the mineral addition incorporated in the cement (difference of the density of the addition mineral). The cement CEM II/A (made with the tuff) present the specific weights definitely higher compared to the cement CPJ-CEM II/B (made with the slag of blast furnace), it is mainly with the quantity of clinker present in each type of cement and with the porosity of the mineral addition used.

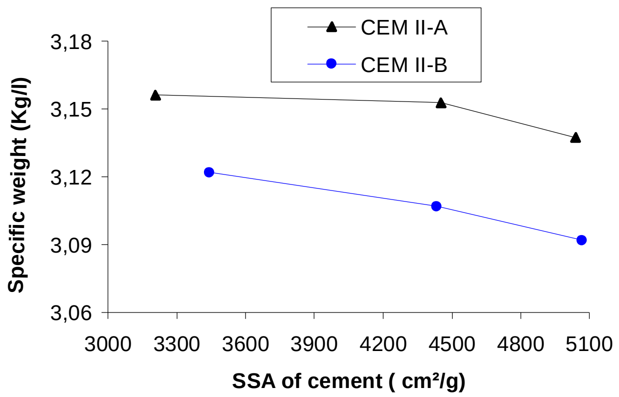

Fig. 1: Effect of S.S.A on the specific weight

Influence of the fineness on the cement paste studied

Figure 2 presents the effect of fineness (specific surface area) on the normal consistency of cement paste. The cements pastes are prepared with three different finenesses and the water demand needed to achieve the standard consistency is measured using the Vicat needle test (standard Vicat test). The influence of the fineness on the cement paste is expressed by the changes in normal consistency (water demand ratio).

One notices also that the granulometry of cement has a significant influence on the normal consistency of cement paste (water cement ratio), this is translated by increase of the total surface of the particles when the cement is ground more finely.

The initial and final set times of cement paste are shown in figures 3 and 4. When the fineness increased of cement, the initial and final setting times of cement paste are decreased. In general, the set time of cement paste is shortened with the increase of fineness. That is explained by the fact that the pozzolanic reactivity is accelerated in the short-term. The kinetics of hydration of the binder becomes increasingly fast according to the increase of the Blaine fineness (specific surface area) of cement.

Indeed, the very fine particles adhere the some to the others and activate the phenomenon of set time of cement paste. Thus the effect of the great Blaine specific surface on the acceleration of the pozzolanic activity reacts with the calcium hydoxide [$Ca(OH)_2$, Portlandite] to form C-S-H gel crystals. The pozzolanic reaction is : [$Ca(OH)_2 + SiO_2 + H_2O \rightarrow$ C-S-H].

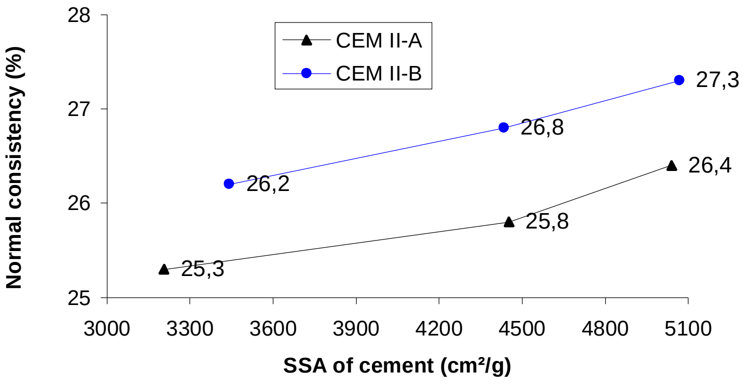

Fig. 2: Effect of S.S.A on the normal consistency of cement paste

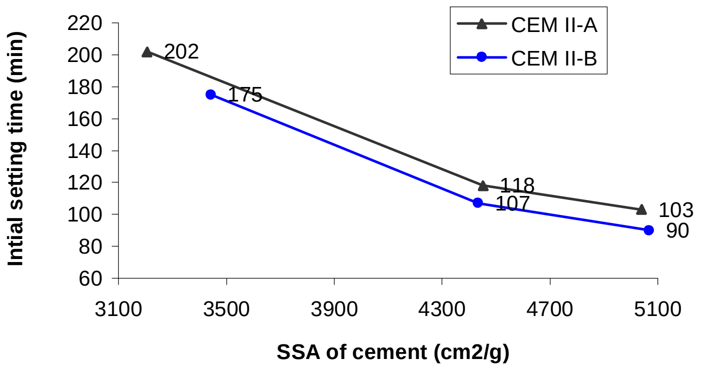

Fig. 3: Effect of S.S.A s on the initial setting time

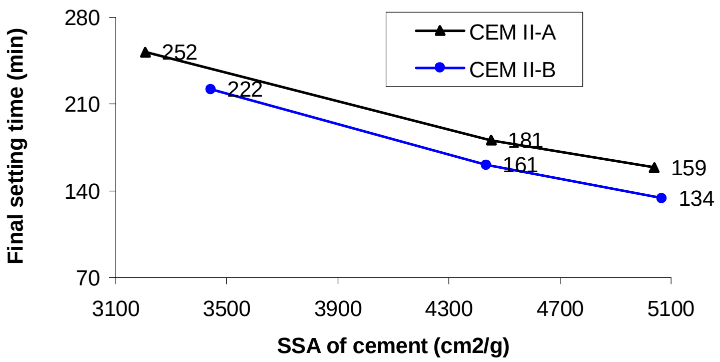

Fig. 4: Effect of S.S.A on the final setting time

Influence of the fineness on the mechanical strengths of mortar

The developments of flexural and compressive strengths of the test specimens are shown in Figures 5 and 6. The increase of the Blaine specific surface of cement gives an increase of the mechanical strengths. That is explained by the increase of the fast kinetics of hydration of the mineral C_3S (tricalcium silicate) and C_2S (dicalcium silicate). These latter are the two principal minerals which ensure the development of the resistances to short and medium-term.

The increase of the Blaine fineness of the cement clearly improves the mechanical strengths of the mortar. This confirms the role of the granulometry (mechanical activation or advanced grinding) in the fast and complete hydration of the cement (pozzolanic activity) by the formation of the $Ca(OH)_2$ released during the hydration of the cement. This pozzolanic reaction gives the second C-S-H supplementary, main responsible for the hardening of the mortar. Therefore the weakness of the strengths to the short-term can be compensated by mechanical activation of cement (increase of the fineness).

The increase of the mechanical responses as a function of the variation of the fineness (mechanical activation) believes a way different of a cement to another cement, this depends on the type and percentage of the mineral addition (reactivity of the admixture) incorporated in the cement.

Thus, it can be concluded that the fineness of cement is a significant characteristic : during the hydration of the mixture, more the particles are fine, more the cement surface in contact with water is large and more the hydration is fast and complete (shortening of set times).

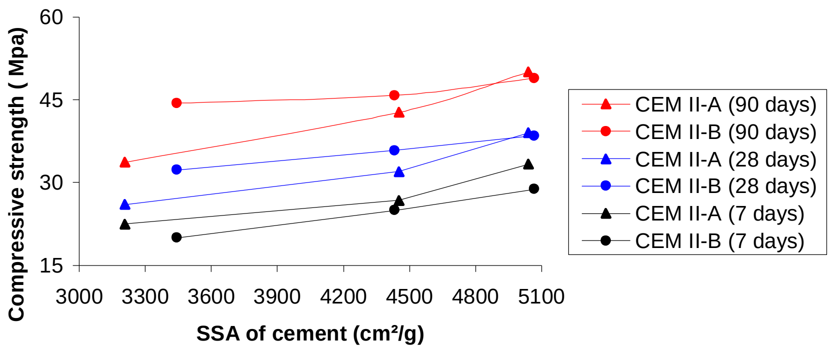

Fig. 5: Evolution of compressive strength of mortars as a function of S.S.A

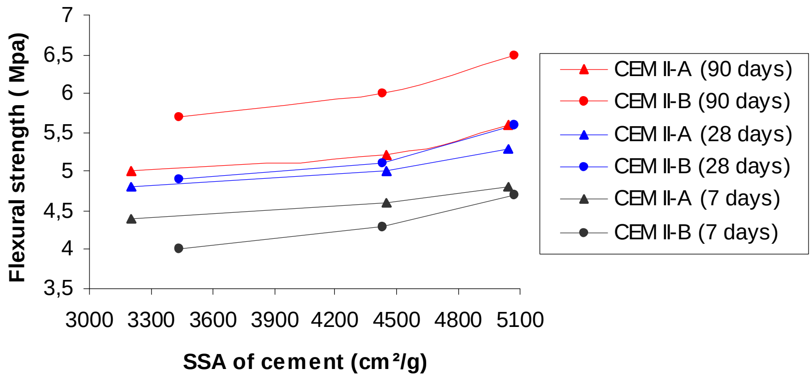

Fig. 6: Evolution of flexural strength of mortars as a function of S.S.A

CONCLUSION

The results obtained from this research, allow us to draw the following conclusions :

* the increase of the fineness (specific surface) of cements with mineral additives (composed cements) influence appreciably on the water demand necessary to have a normal consistency of cement paste.

* the setting times (initial and final) decrease proportionally with the increase of the fineness (specific surface) of cements with mineral admixtures.

* the mechanical activation (high fineness) of cements with mineral additives presents two essential advantages : high mechanical strengths (flexural and compressive) of the mortar as well as a kinetics of hydration reaction accelerated at the initial hardening (short-term).

Finally, cements with mineral additives must be finely to grind (S_p > 3500 cm^2/g) in order to accelerate the kinetics of hydration of short-term cements (improvement of the reactivity of the cementing mineral additions) and to ensure a high mechanical resistance of material.

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SULFATE RESISTANCE AND CARBONATION OF FLY ASH CONCRETE

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ABSTRACT

Durability is a major concern for concrete structures exposed to aggressive environments. Many environmental phenomena are known to significantly influence the durability of reinforced concrete structures. Carbonation and sulfate attack are considered as the major factors to cause concrete structures deterioration. Concrete gradually deteriorates with time and finally loses its strength when it is exposed to the sulfate-bearing water from the surroundings. Carbonation is the process by which carbon dioxide (CO_2) in the atmosphere reacts with water in concrete pores to form carbonic acid and then reacts with alkalis in the pores, neutralizing them. This can then lead to the corrosion of the reinforcing steel. It has been suggested that carbonation and sulfate resistance of concrete can be effectively improved by a partial replacement of Portland cement by fly ash. This paper presents some results of an experimental study on sulfate resistance and carbonation of concrete containing fly ash cement. The obtained results are compared with those of a control concrete made with ordinary Portland cement.

Keywords: Portland cement, Fly ash cement, , Curing, Sulfate, Carbonation

INTRODUCTION

For decades Portland cement has been the most used and best known cement type all over the world. After aluminum and steel, the manufacture of portland cement is the most energy intensive production process. Not only is the manufacture of portland cement highly energy intensive, it also is a significant contributor of the greenhouse effect and the global warming of the planet [1]. Therefore, as a result of the urge for saving energy and disposing of waste materials in many countries, other cement types and binders have been developed. Blended/multiblended cements based on industrial byproducts/pozzolanic materials like fly ash, calcined clay, microsilica, granulated blast furnace slag, etc., are the best examples of alternate cementitious materials. This class of cements is better known for their improved long-term strength and durability [2].

Current design methods of concrete structures have been organized with a focus on “safety”. Although “durability” has recently been incorporated in design as a function of time, it must be said that the environmental viewpoint is still extremely weak [3]. Promoting reduction in the environmental impact in the design of concrete structures, taking global warming and resource efficiency into account, can be considered basically the same as safety and durability design. [4]. For a variety of reasons, the concrete construction industry is not sustainable. First, it consumes huge quantities of virgin materials. Second, the principal binder in concrete is Portland cement, the production of which is a major contributor to greenhouse gas emissions that are implicated in global warming and climate change. Third, many concrete structures suffer from lack of durability which has an adverse effect on the resource productivity of the industry. Because the high-volume fly ash concrete system addresses all three sustainability issues, its adoption will enable the concrete construction industry to become more sustainable [5].

Generally speaking, currently in the concrete industry, the percentage of fly ash as part of the total cementing materials in concrete normally ranges from 15 to 25%, although it can go up to 30-35% in some applications. The use of fly ash in concrete will improve some aspects of the performance of the concrete provided the concrete is properly designed. The main aspects of the concrete performance that will be improved by the use of fly ash are increased long-term strength and reduced permeability of the concrete resulting in potentially better durability. The use of fly ash in concrete can also address some specific durability issues such as sulphate attack and alkali silica reaction. However, a few additional precautions have to be taken to insure that the fly ash concrete will meet all the performance criteria.

Concrete gradually deteriorates with time and finally loses its strength when, it is exposed to the sulfate-bearing water from the surroundings. It is believed that the sulfate-related deterioration of concrete is mainly due to the expansion brought about by the formation of gypsum and/or ettringite, which arises from the reaction of sulfate ions with calcium hydroxide and calcium aluminate hydrate in concrete. It has been suggested that the sulfate resistance of concrete can be effectively improved by a partial replacement of portland cement by fly ash. Fly ash induces three phenomena that improve sulfate resistance: 1) Fly ash consumes the free lime making it unavailable to react with sulfate. 2) The reduced permeability prevents sulfate penetration into the concrete. 3) Replacement of cement reduces the amount of reactive aluminates available. However, the effect of fly ash is dependent on the physical and chemical properties of both portland cement and fly ash used [6]. As a general rule, Class F fly ash can improve the sulfate resistance of concrete mixtures. On the other hand, the situation with Class C fly ash is somewhat less clear [7]. Recently, it has been shown that intermixtures of high and low calcium fly ashes results in an effective, environmental friendly and relatively cheap cementitious material to deal with mutual shortcomings associated with each type of ash [8]. However, the sulfate-related deterioration of concrete is a complicated phenomenon of physical and chemical process, and the mechanism of improvement of sulfate resistance by the addition of relatively large amounts of fly ashes is not still fully understood [9]. Moreover, DePuy [10] and Neville [11] concluded that, although sulfate attack has been extensively investigated, it is still not completely understood.

Carbonation is one of the major factors to cause concrete structures deterioration. Carbonation is the reaction of hydration products dissolved in the pore water with the carbon dioxide in the air, which reduces the pH of concrete pore solution from 12.6 to less than 9 and steel passive oxide film may be destroyed and accelerating uniform corrosion [12]. Carbonation reduces pH value and destroys the passive film around the steel, but it seems to densify concrete surface and reduce chloride ion permeability, reduce surface porosity and hence sorptivity in concrete [13]. Hence, carbonation could have both positive and negative effects on concrete durability [14].

EXPERIMENTAL WORK

The experimental work described herein is a part of an experimental program aiming at investigating the mechanical properties and durability aspects of concrete containing fly ash cement. The preliminary results with this new type of cement are promising. This paper gives an overview of the properties of FAC concrete, that is believed to be a very promising alternative for the industry seeking to meet the sustainable development objectives

Materials and Mix Proportions

Japanese fly ash cement type B (FAC) was used to produce fly ash concrete specimens investigated in the current research experimental work. While ordinary Portland Cement (OPC) was used to produce the control concrete mixtures for comparison. High-range water reducing admixture (HRWR), termed superplasticizer, was added to all concrete mixtures to enhance the concrete workability. It was added as 1.4% of the cement content by weight. Natural sand with

2.55 specific gravity, 2.49 fineness modulus and 1.62% water absorption was used as fine aggregate. While the coarse aggregate in the mixtures was crushed basalt of 2.61 specific gravity, 15 mm nominal maximum size and 6.18 fineness modulus. Three concrete mixtures were prepared with each type of cement using water/cement ratios of 0.4, 0.5 and 0.6. The cement content and sand/aggregate ratio were kept constant at 400 kg/m^3 and 0.5, respectively. The mix proportions of all concrete mixtures are listed in Table 1.

Mixing, Casting, and Curing

The following mixing procedure was used for all concrete mixtures. First, the total content of coarse aggregate, sand and cement were mixed all together (dry) in the mixer for 1.5 minutes. Afterwards, water and superplasticizer (mixed together) were added and the mixing was continued for further 1.5 minutes. After mixing, concrete was removed from the mixer and fresh concrete properties were measured. Then specimen moulds were filled with concrete and tempered with a standard steel rod. The concrete specimens were removed from the moulds at 24 hours after casting. Two different curing methods were applied in this study. In the first method, the specimens were totally immersed in water until the time of testing (curing A). While in the second method, specimens were subjected to 80% Relative Humidity (RH) and 22^{o}C and sprinkled with water twice a day for seven days (curing B).

Table 1: Mix proportions of concrete mixtures

Cement type		OPC	FAC
Cement content, kg/m^3		400	
Sand/aggregate		0.5	
Admixture (liter/m^3)		5.6	
w/c* = 0.4	Water, liter/m^3	160	
	Sand, kg/m^3	877	868
	Crushed basalt, kg/m^3	898	888
w/c* = 0.5	Water, liter/m^3	200	
	Sand, kg/m^3	826	817
	Crushed basalt, kg/m^3	845	836
w/c* = 0.6	Water, liter/m^3	240	
	Sand, kg/m^3	775	766
	Crushed basalt, kg/m^3	793	784

* w/c: water/cement ratio

Test Specimens and Procedure

The compressive strength and depth of carbonation tests were carried out for all specimens, at the age of 28 days. Resistance to sodium sulfate was measured for the two concrete mixtures with water/cement ratios of 0.4. In order to investigate the effect of sulfate solutions on concrete properties, 100 x 200-mm cylinders and 100 x 100 x 400-mm beams were cured for 28 days before immersion in a 10% sodium sulfate solution for the period of 360 days. Concrete properties were measured at 7, 28, 90, 180 and 360 days after complete immersion in sodium sulfate solution, in which the solution was periodically exchanged by a new one. The measured properties included compressive strength, flexural strength, pulse velocity, and dynamic elastic modulus. Carbonation depth was measured on 100 x 100 x 400-mm prisms that were cured for 28 days before exposure to carbonation environment (5% CO_2 concentration, RH of 60%, and temperature degree of 30^{o}C). Carbonation depths were measured at 1, 2, 4, and 8 weeks of exposure.

RESULTS AND DISCUSSION

Properties of Fresh Concrete

The measured properties of freshly mixed concrete are presented in Table 2. Slump test was carried out for concrete mixtures made with w/c = 0.4, while concrete flow was measured for the mixtures of w/c ratios of 0.5 and 0.6. Table 2 reveals that both slump and flow values of FAC concrete are higher than those of OPC concrete; this is valid for all w/c ratios. Similar findings were concluded by other researchers [15]. FAC favorably influences concrete workability because the spherical shape of its particles and high portions of particles finer than 10-μm [7].

Table 2: Properties of fresh concrete mixtures

Property of fresh concrete	w/c = 0.4		w/c = 0.5		w/c = 0.6	
	OPC	FAC	OPC	FAC	OPC	FAC
Slump (cm)	19.2	22.2	----	----	-----	----
Flow (cm)	----	---	69.4	72.6	71.9	75.2
Air Content (%)	1.8	2.3	1.2	1.5	0.9	1.4
Unit Weight (t/m^3)	2.384	2.322	2.361	2.289	2.342	2.261

Dixon [16] stated that, the physical shape of fly ash particles can be best described as fine glassy beads. When added to a concrete mix, these smooth round objects act as a sort of lubrication by attaching themselves to the cement particles and keeping them from globing together during hydration. This increases workability and responsiveness of the mix during placing and vibration. The small size of the particles also fills voids between cement particles that would normally be filled with excess water. This allows for lower water/cement ratio without sacrificing workability. The effect of using superplasticizer with high w/c ratios on the concrete properties was discussed in details in Ref. [17].

Compressive Strength of Concrete

The obtained results of compressive strength at 28 days are given in table 3. Generally, concrete mixtures made with OPC show higher compressive strength values than those of the corresponding mixtures made with FAC, for all investigated w/c ratios and curing methods. Some attempts were made to increase the early-age strength of FAC concrete by incorporating small percentages of silica fume to the system [15]. Compressive strength values of concrete specimens cured under water (curing A) are higher than those of the corresponding specimens subjected to curing B. This is true for both cement types and all adopted w/c ratios excluding the mix of OPC with w/c = 0.4, which shows a relatively comparable values of compressive strength. The effect of change of w/c ratios on the concrete properties was discussed in details in Ref. [17].

Table 3: Compressive strength of concrete at 28 days

w/c ratio	Curing A		Curing B	
	OPC	FAC	FAC	FAC
0.4	464	479	475	452
0.5	414	362	380	352
0.6	276	226	241	205

Concrete Resistance To Sulfate Attack

Bilodeau [15] reported, that the sulfate resistance of fly ash concrete is better than the reference concrete made with OPC. The primary reason appears to be the dilution effect, that is, the reduction in the C_3A and the $Ca(OH)_2$. Most of the available $Ca(OH)_2$ is consumed in pozzolanic reactions, thus inhibiting the sulfate reactions.

Effect of immersion period on compressive strength

The measured compressive strength values of OPC and FAC concrete mixtures before and after immersion in sodium sulfate are shown in Fig.1. For curing A, it can be concluded that compressive strength of both types of concrete increases with increasing the immersion period till 180 days, while the compressive strength at the period 360 days is lower that that of 180 days of immersion. This means that till 180 days of immersion, the strength gain, due to the hydration process, is higher than the strength loss due to immersion in 10% sodium sulfate solution. As for curing B, the corresponding immersion period till which the strength increases is 90 days only.

Effect of cement type on compressive strength

For curing A, the OPC concrete shows higher compressive strength values than those of FAC concrete till 28 days of immersion, while both types of concrete shows comparable values of compressive strength at 90 days of immersion. Moreover, the FAC concrete shows higher compressive strength values than those of OPC concrete at immersion periods more than 90 days. Additionally, at the end of the immersion period, FAC concrete has greater compressive strength value than the 28-day strength value. This tendency is in a good agreement with the results of Torii [6]. On the other hand and for curing B, the OPC concrete shows higher compressive strength values than those of FAC concrete till 180 days of immersion, while the opposite is true for later periods of immersion.

Effect of curing method on compressive strength

Compressive strength values of concrete specimens subjected to curing A are higher than those of the corresponding specimens subjected to curing B for both types of cement and at all immersion periods. This effect is more pronounced in the case of FAC concrete particularly at longer immersion periods. Thus, the sulfate resistance of concrete subjected to curing A is better than that of concrete subjected to curing B. Details of the effect of curing method on compressive strength were discussed in Ref [17].

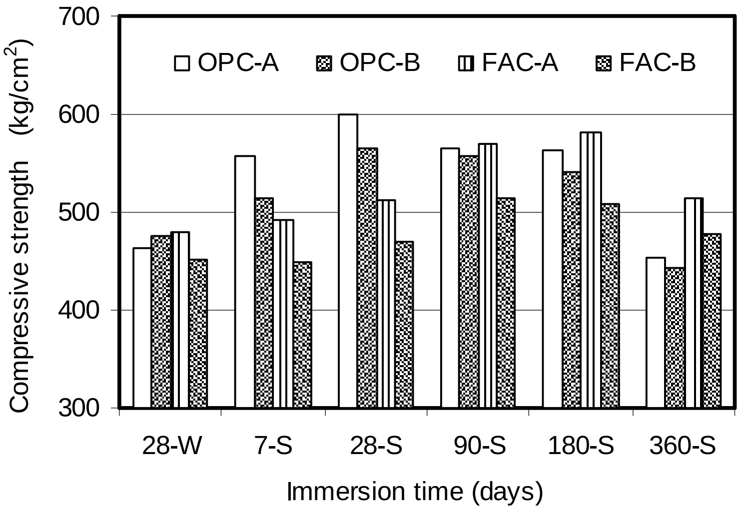

Fig. 1 Effect of sodium sulfate on the compressive strength of concrete

Effect of immersion period on flexural strength

Fig. 2 presents the effect of immersion period on the flexural strength of OPC and FAC concrete mixtures. Generally speaking, it is obvious that the flexural strength of both types of concrete under both curing methods increases with increasing the immersion period till 28 days and decreases at later periods of immersion.

Effect of cement type on flexural strength

The OPC concrete shows higher flexural strength values than those of FAC concrete at all immersion periods for curing A. While for Curing B, the OPC concrete shows higher flexural strength values than those of FAC concrete till 90 days of immersion, while the opposite is true for later periods of immersion.

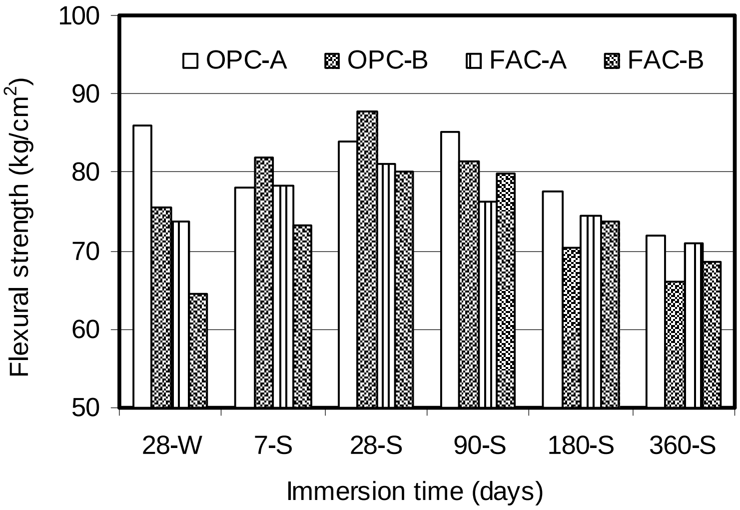

Fig. 2 Effect of sodium sulfate on the flexural strength of concrete

Effect of curing method on flexural strength

No specific conclusion can be drawn concerning the effect of curing period on flexural strength of concrete immersed in sulfate solution.

Effect of immersion period on pulse velocity

The effect of immersion period on pulse velocity through OPC and FAC concrete specimens is shown in Fig. 3. This figure reveals that the pulse velocity through OPC concrete specimens under curing A increases with increasing the immersion period till 90 days and decreases at later immersion periods. While for FAC concrete specimens, the corresponding immersion period till which the velocity increases is 28 days. For curing B, the pulse velocity through both types of concrete under both curing methods increases with increasing the immersion period till 180 days and decreases at later periods of immersion.

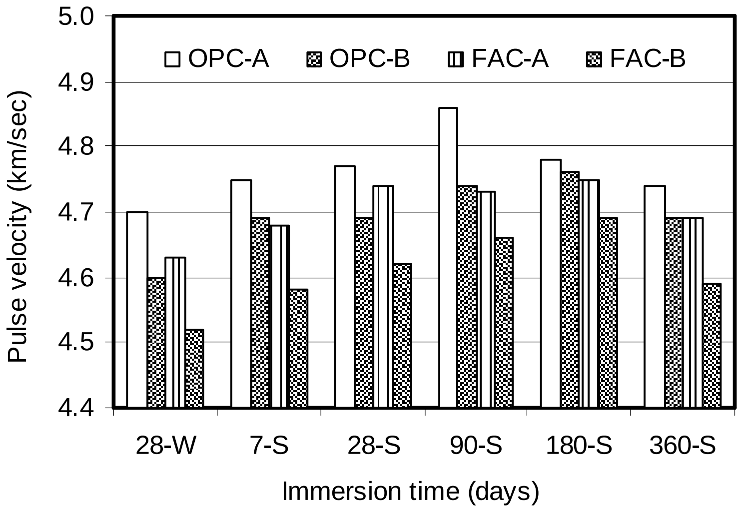

Fig. 3 Effect of sodium sulfate on the pulse velocity of concrete

Effect of cement type on pulse velocity

The pulse velocity values through OPC concrete are higher than its values through FAC concrete for both curing methods and at all immersion periods.

Effect of curing method on pulse velocity

The effect of curing method on pulse velocity values through concrete specimens is similar to its effect on compressive strength. This confirms that the sulfate resistance of concrete subjected to curing A is better than that of concrete subjected to curing B, especially in the case of FAC concrete.

Effect of immersion period on dynamic elastic modulus

Figure 4 depicts the effect of immersion period on the dynamic elastic modulus of both OPC and FAC concrete mixtures. Immersing concrete specimens in sodium sulfate for 7 days after 28 days of curing A increases the value of dynamic elastic modulus. Increasing the immersion period results in decreasing the value of dynamic elastic modulus. As for curing B, the corresponding immersion period till which the dynamic modulus increases is 28 days. These conclusions are valid for both types of concrete.

Effect of cement type on dynamic elastic modulus

The dynamic elastic modulus values of OPC concrete specimens are higher than the corresponding values of FAC concrete specimens for both curing methods and at all immersion periods. The only exception is the immersion period of 360 days under curing B where FAC concrete shows higher elastic modulus that that of OPC concrete.

Effect of curing method on dynamic elastic modulus

Dynamic elastic modulus values of concrete specimens subjected to curing A are higher than those of the corresponding specimens subjected to curing B for both types of cement and at all immersion periods.

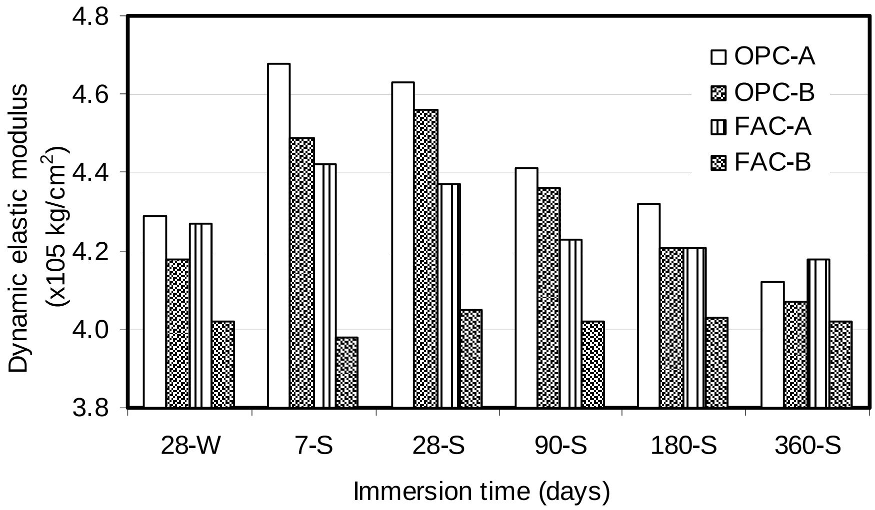

Fig. 4 Effect of sodium sulfate on the dynamic elastic modulus of concrete

Carbonation

Exposing the internal surface of concrete specimen and spraying on phenolphthalein indicator easily measures the carbonation depth. The phenolphthalein solution will remain clear, where concrete is carbonated and turn pink where concrete is still alkaline. The best indicator solution for maximum contrast of the pink coloration is a solution of phenolphthalein in alcohol or water, usually 1g indicator in 100ml of alcohol/water (50:50 mix) [18]. The measured carbonation depths (average of 10 measurements) are presented in Table 4 and shown in Fig. 5.

Effect of exposure period and w/c ratio on carbonation depth

The results show that, for both types of concrete and curing methods, the carbonation depth increases with increasing the exposure time. Similarly, the carbonation depth is directly proportional to the w/c ratio, i.e. the higher the w/c ratio the higher the carbonation depth, for both types of concrete and all exposure periods.

Effect of cement type on carbonation depth

The measured carbonation depths of FAC concrete specimens are greater than that of OPC concrete specimens for both curing methods and all w/c ratios and exposure periods.

Effect of curing method on carbonation depth

The obtained results show that the curing method affects the measured carbonation depths. The OPC concrete specimens, under curing A, indicate greater carbonation depths than those of the corresponding specimens subjected to curing B for all w/c ratios and all exposure periods. On the other hand, FAC concrete specimens under curing A show comparable carbonation depths to those of specimens subjected to curing B for all w/c ratios and exposure periods. Moreover, the effect of w/c on the carbonation is more clear than the effect of curing method as shown in Table 4 and Fig. 5.

Table 4: Carbonation depths of OPC and FAC concrete (mm)

Cement type	w/c ratio	Curing method	Exposure period (week)			
			1	2	4	8
OPC	0.4	A	0.1	0.2	0.8	0.3
		B	0.4	0.1	0.0	0.1
	0.5	A	0.9	1.6	5.4	6.9
		B	0.2	0.8	4.1	4.3
	0.6	A	2.9	8.2	11.3	14.6
		B	1.3	3.2	7.3	11.4
FAC	0.4	A	0.6	1.1	4.4	5.7
		B	0.5	1.9	2.0	6.5
	0.5	A	2.2	6.2	9.2	11.3
		B	2.7	6.3	8.7	12.8
	0.6	A	7.9	10.2	13.0	20.4
		B	5.9	9.1	13.6	19.6

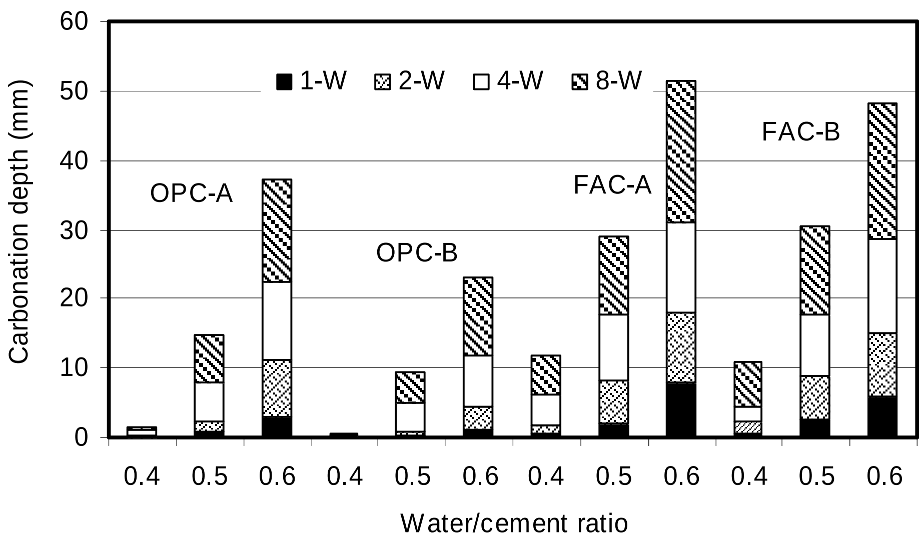

Fig. 5 Carbonation depths of OPC and FAC concretes with different w/c

CONCLUSIONS

1. It is possible to use FAC to obtain workable concrete with acceptable performance comparable to that of conventional OPC concrete.
2. Compressive strength of FAC concrete is higher than that of OPC concrete, after long periods of immersion (more than 180 days) in 10% sodium sulfate solution.
3. Sulfate resistance of concrete subjected to curing A is better than that of curing B, especially in the case of FAC concrete, because of the beneficial effects of fly ash on permeability and diffusivity tend to become more apparent with time (maturity with continues of curing).

4. Both pulse velocity and dynamic elastic modulus show approximately the same trend as the compressive strength and, consequently, it is possible to use both tests to compare or check the quality of concrete.
5. Carbonation depths of FAC concrete specimens are greater than that of OPC concrete specimens for both curing methods and all w/c ratios and exposure periods.

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THERMAL AND MECHANICAL EVALUATION OF THREE TYPES OF WOOD SUBJECTED TO DIFFERENT ENVIRONMENTAL CONDITIONS

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ABSTRACT

Wood is one of the most important natural materials for structural elements used in the building industry. This paper deals with the thermal and mechanical behavior of different types of wood subjected to different environmental conditions. Three types of wood were studied for their thermal and mechanical properties (Mahogany, Pitch Pine and Mosque). Mass loss with time at different temperatures, water absorption and density for each type of wood were studied. Also, bending and compressive strength in the parallel and perpendicular directions of the fibers were measured. Specimens from each type of wood were subjected to direct fire at nearly 600^oC for different exposure time. These samples were repaired by GFRP wrapping systems.
The results show that clear differences in both thermal and mechanical properties of these types of wood were observed. GFRP wrapping systems proved to be an effective method in repairing and strengthening wood structures subjected to fire.

Keyword: Thermal; mechanical; wood; GFRP; fire; repair; buckling.

1. INTRODUCTION

Worldwide wood is used more than metal or plastics. All types of wood are composed of cellulose, lignin, hemicelluloses, and minor amounts of extraneous materials contained in a cellular structure. Cellulose comprises about 50% of wood and is responsible for most of its mechanical properties. To use wood for its best advantage and most efficient in engineering applications, thermo-mechanical properties must be considered. Thermal, physical and mechanical properties of wood play an important role in heat and mass transfer [1-3]. Static tests procedure and test specimen sizes are mentioned in the standards [4-5]. Strengthening timber members with fiber reinforced polymers (FRP) has primarily focused on the use of FRP sheets. Triantafillou and Deskovic (1992) used prestressed FRP sheets as reinforcement for wood[6]. Their research showed that small amount of FRP reinforcement produced significant gains in strength and stiffness. Fiber reinforced polymer sheets, carbon fiber reinforced polymer (CFRP)strips, and GFRP strips have been used as external reinforcement for timber (Sonti et-al.1996) [7]. They reported an increase in the strength and stiffness of the beams. Bakoss et-al(1999) reported tensile failure in one of the bottom laminates, accompanied by partial or total

GFRP sheets to reinforce sawn timber sections [9]. Their work was conducted using small beams, tested on a simply supported span. They found that the increase in the moment of resistance of the reinforced beams was greater than that predicted by simple transformed section analysis [9]. Gentile et al. (2002) carried out a research using FRP bars as near-surface-mounted reinforcement [10]. They reported a 20% to 50% increase in the flexural capacity of the beams that depended primarily on the quality of timber. The strengthened beams failed in a ductile manner, and it was confirmed that the reinforcement bridged the existing defects in the beams. Triantafillou (1997) studied CFRP laminates and fabrics externally bonded to structural timber members in the critical shear zones to increase their capacity [11]. Twenty-one wood beams, designed to fail in shear, were reinforced with CFRP fabrics at various configurations and areas and then tested to failure in four-point bending. It was concluded that the effectiveness of the strengthening method depends on fiber orientation, with longitudinal placement showing to be the most effective. Svecova et al. (2004) carried out an experimental program to test timber strengthened with glass fiber reinforced polymer (GFRP) bars [12]. Various strengthening schemes were investigated as a means of increasing load carrying capacity of timber stringers in shear and flexure. They found that strengthening timber stringers with GFRP reinforcement increased the ultimate strength of the stringers and reduced its variability [12-13].

2 EXPERIMENTAL PROGRAM

An experimental program was designed to carry out thermal and mechanical tests of different types of wood.

2.1 Materials

2.1.1 Wood

Three types of wood were chosen (Mahogany, Pitch Pine and Mosque) and examined for their thermal and mechanical properties. The wood specimens were machined in dimensions suitable for every test according to ASTM [4-5]. Cubes 50x50x50 mm were used for density, loss of weight at different temperatures (50, 75 and 100^{o}C) water absorption and compression test parallel and perpendicular to fibers. Samples with dimensions 50x50x500 mm were prepared for bending test and columns 50x70x700 mm were prepared to be tested in axial compression. These columns specimens were tested before and after exposure to fire as well as after strengthening with fiber reinforced polymers wrapping systems.

2.1.2 Glass fiber reinforced polymer (GFRP)

Table (1) illustrates the typical mechanical properties of polyester resin and glass fiber wrap reported by the manufacturer.

Table (1): Typical mechanical properties of polyester resin (2504 APT-5) and glass fiber wrap as reported by the manufacturer

Material \ Property	Property	Value
polyester Resin (2504 APT-5)	Tensile strength, N/mm^2	60
	Flexural strength, N/mm^2	110
	Flexural modulus, N/mm^2	3.0x10^3
Glass Fiber Wrap	Tensile strength, N/mm^2	600
	Rupture strain, %	2.24
	Modulus of elasticity of FRP laminates, N/mm^2	26.13

2.2 Fire Processing

A fire furnace was especially constructed for this investigation. Three isolated thermocouples were used to record furnace temperature. The tested columns were subjected to fire not exceeding 600^{o}C.

2.3 Repairing Process

Columns specimens for each type of wood were subjected to fire at the same temperature for different time intervals (5, 10 and 15 minutes). To restore the original cross section of the wood specimens, a mixture of wood dust and polyester was used. The samples were repaired by GFRP wrapping systems (one, two and three layers). Figure (1) shows three different groups of wood samples: Mahogany (M), Pitch Pine (P) and Mosque (S), while figure (2) shows different methods of repairing process for different wood specimens by restoring the cross-section after exposure to fire.

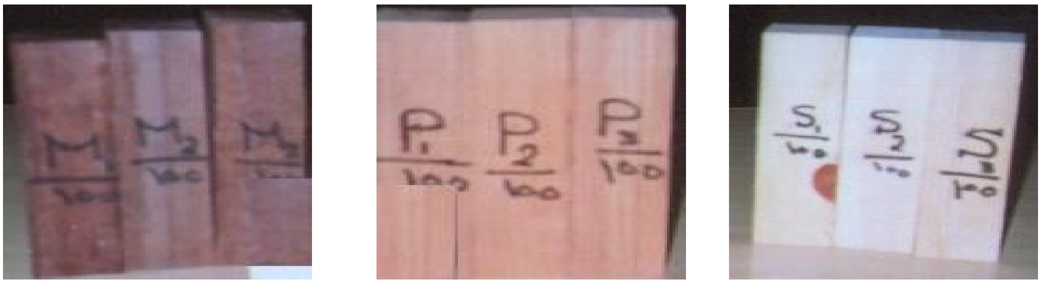

Fig. 1: Three different types of wood (Mahogany, Pitch pine and Mosque)

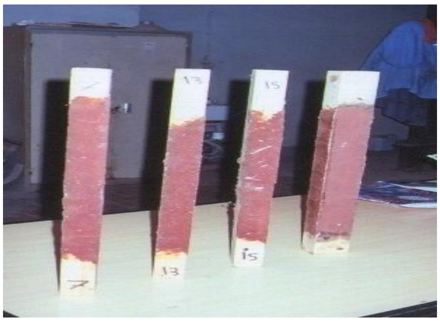

Fig. 2: Methods of repairing process for different wood specimens by restoring the cross-section after exposure to fire.

2.4 Thermo-physical Tests

Thermo-physical properties play a significant role in the heat and mass transfer in building materials especially wood. The density of wood vary widely and are affected significantly by moisture content which varies through its life starting from its initial cutting to its final use. The density, water absorption and loss of weight represent the most important functions governing the transition of heat and mass during the exposure to high temperature and the fire period. Density loss ratio after two hours at different temperatures (100 and 300^{o}C), loss of weight at different temperatures (50, 75 and 100^{o}C) and water absorption were measured.

2.5 Mechanical Tests

The compression test for specimens parallel and perpendicular to fibers was carried out on cubes of 50 x 50 x50 mm using a hydraulic testing machine of 2000 kN capacity. Flexure test was carried out on standard beam specimens of dimensions 50x50x500 mm [4-5].

3. Results and Discussion

3.1 Mechanical Tests Results

Figure (3) shows the compressive strength results of different types of wood loaded parallel and perpendicular to fibers. The compressive strength reached 57.3, 84.4 and 90.2 N/mm^2 in the parallel direction of fibers for the S, P and M wood types, respectively. The compressive strength of samples P and M increased by 47.29 and 57.41% respectively in comparison with that recorded for sample S, for parallel direction. Also, it is noticed that, the compressive strength perpendicular to fibers have the same trend as the parallel direction. Also, the compressive strength in the parallel direction increased by 6.51, 3.85 and 3.16 times that recorded in the perpendicular direction for the S, P and M samples, respectively. So, it can be concluded that, the wood sample M is the best in comparison with the other two samples.

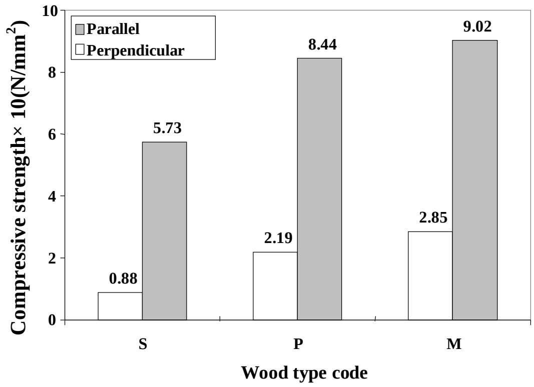

Fig. 3: The relationship between the compressive strength and different types of wood specimens at parallel and perpendicular direction of fiber

Figure (4) shows the bending test results of different types of wood S, P and M samples. The bending strength reached 67.2, 89.8 and 120.5 N/mm^2 for samples S, P and M, respectively. The bending strength of samples P and M increased by 33.63% and 79.31% in comparison with the sample S, respectively. Also, it can be concluded that, the wood sample M showed the highest flexure strength in comparison with the other two samples.

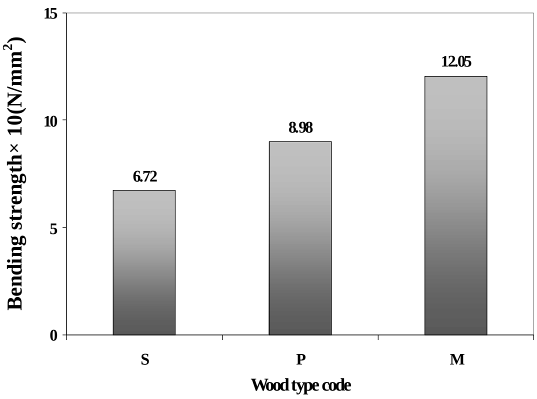

Fig.4: The relationship between the flexural strength and different types of wood specimens.

3.2 Thermal Test Results

Figure (5) illustrates the relationship between the density loss percentage and the temperature variation (100 and 300^{o}C) for two types of wood specimens (M and P). From the figure it can be noticed that, the density loss percentage increased with the increase of temperature. The density loss percentage reached 0.5 and 11.0% for M specimen while reached 1.58 and 20.27% for P specimen at 100^{o}C and 300^{o}C, respectively. The density loss percentage of the P sample increased 1.84 times than M sample at 300^{o}C. It can be concluded that, the wood sample M has the lowest density loss ratio compared to the other type.

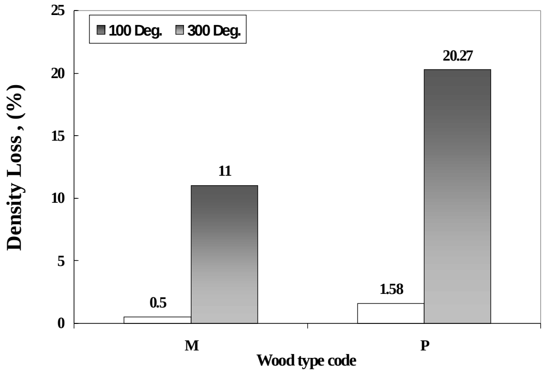

Fig. 5: The relationship between density loss ratio and temperature for different types of wood specimens.

Figure (6) shows the change of water absorption percentage with time at room temperature for different types of wood (S, P and M). The general trend was the increase in the water absorption percentage with time for all types of wood. The water absorption percentage reached 8.28, 13.55 and 22.29 % after 72 hours while reached 12.92, 24.18 and 32.9 % after 168 hours for M, P and S, respectively. The increase in water absorption percentage for samples P and S were 87.15% and 154.64% in comparison with sample M, respectively. So, it can be concluded that, the wood sample M showed the lowest water absorption percentage compared to the other types. Also, the S sample has the highest water absorption percentage than the other two types.

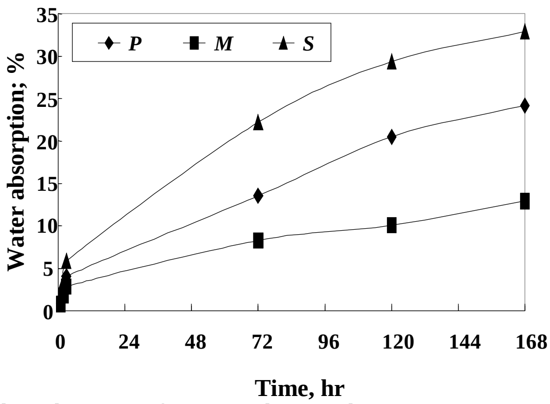

Fig. 6: The change of water absorption percentage with time for different types of wood specimens.

Figure (7) shows the weight loss with time for the different types of wood tested in this research at 75^{o}C. The general trend was that the weight loss increased with time for the three types of wood. During the first 20 hours, the weight loss at 75^{o}C for sample M reached 2.44, 4.01, 5.59 and 7.08% at 5, 10, 15 and 20 hours, respectively. For sample P the weight loss reached 2.91, 4.53, 6.58 and 8.54% while for sample S reached 4.30, 6.39, 8.48 and

11.01% at 5, 10, 15 and 20 hours, respectively. The weight loss for P and S samples increased by about 23.44 and 55.50% compared to M sample, respectively.

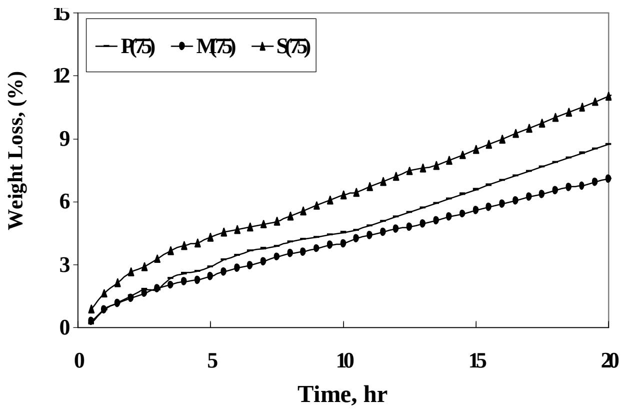

Fig. 7: The relationship between the weight loss and time for different type of wood at 75 °C.

Figure (8) illustrates the weight loss with time for sample P at different temperatures (50, 75 and 100°C). The general trend was increasing in the weight loss with time for different temperatures. The weight loss reached 1.61, 3.38 and 5.0% at 50°C after 5, 10 and 15 hours, respectively. The weight loss also reached 2.91, 4.53 and 6.58% at 75°C while reached 5.69, 11.62 and 16.77% at 100°C after 5, 10 and 15 hours, respectively. After 15 hours, the weight loss increased 1.31 and 3.35 times after exposure to 75°C and 100°C, respectively in comparison to that recorded for sample P when exposed to 50°C. So, it can be concluded that, the weight loss increased with the increase of the temperature of exposure and its duration for the types of wood (Mahogany, Pitch Pine and Mosque).

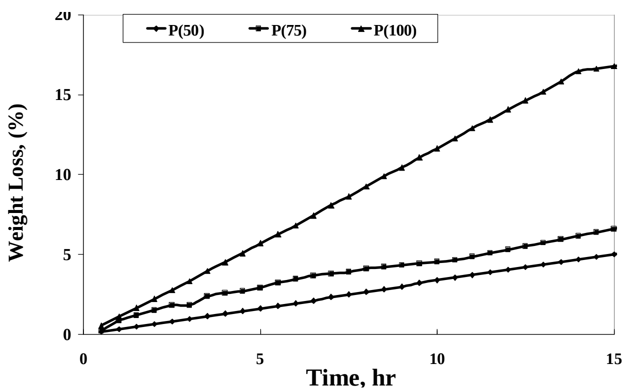

Fig. 8: The relationship between the weight loss and time for Pitch Pine at different temperatures.

3.3 Repaired Specimens Results

Different wood columns were repaired with GFRP layers after being exposed to fire at 600°C for 5, 10 and 15 minutes. Figures (9),(10), (11) illustrate the relationship between the ultimate load carrying capacity of the three different wood specimens studied and the GFRP repairing system. Figure (9) shows the effect of the repairing system on the ultimate load of mosque specimens subjected to fire for different time duration (5, 10 and 15 min.). By comparing the ultimate load values, it is clear that using one layer of GFRP wrapping system increased the load carrying capacity of the specimens 30%, when using two and three layers of GFRP systems, the enhancement reached 50 to 60%, respectively. From fig. (9) also, it is clear that, the samples subjected to fire for 10 and 15 mints have the same trend for loads and the enhancement for one, two and three layers ranged from 30, 50 and 60% compared to the control specimens, respectively. This can be attributed to, the repair process using GFRP wrapping system which makes a good confinement for the wood sample and consequently the GFRP layer resisted the load up to failure. It is clear also that, the process of restoring the original cross section of the specimens using a mixture of wood dust and polyester had good bond with the sample surface and also enhanced sample confinement through the loading process. The repair process is more effective in enhancing the ultimate loads carrying capacity no matter wood type.

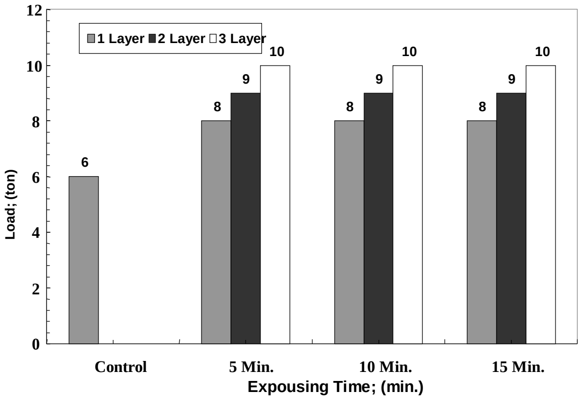

Fig. 9: The relationship between the ultimate load and the GFRP repairing system for mosque (S) specimen after exposure to fire for 5, 10, 15 min.

Figure (10) illustrates the ultimate load of pitch pine wood columns subjected to fire for 10 minutes and wrapped with GFRP. It is clear that, the repairing process of the columns specimens by one, two and three layers of GFRP wrapping system increased the load carrying capacity 15, 30 and 45% respectively compared to the control specimen. It was found that, the method used to restore the wooden column section by a mixture of wood dust and polyester is a very effective process in repairing the wood section.

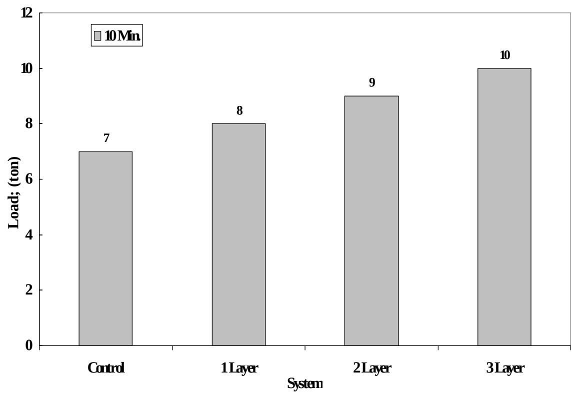

Fig. 10: The relationship between the ultimate load and the GFRP repairing system for Pitch pine (P) specimen after exposure to fire for 10min.

Figure (11) illustrates the relationship between the ultimate load carrying capacities and the method of repairing of the Mahogany wooden column specimen after being subjected to fire for 10 minutes. It was found that the repairing process by one, two and three layers of GFRP wrapping system increased the ultimate load carrying capacities 6, 12 and 20% compared to its control original values.

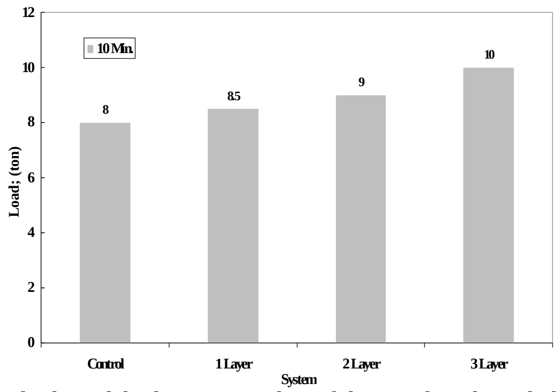

Fig. 11: The relationship between the ultimate load and the GFRP repairing system for Mahogany (M) specimen after exposure to fire for 10min.

4. Modes of Failure of the Tested Specimens

It was noticed that under compression loads the specimens were subjected to successive buckling and separation of the longitudinal fibers due to the weak bond between them in the lateral direction. For flexural test the mode of failure was tension failure at the midspan of the sample associated with warping of the cross section and successive deflection followed by breaking of the sample at the middle third of the bending specimen as shown in fig. (12 a, b and c) for mosque, pitch pine and mahogany. Complete separation at the different layers between the tension and compression zones of the mosque specimen were observed during flexural tests as shown in fig. (12-a) while clear deflection were remarkable for both pitch pin and mahogany wood specimen during flexural test followed by compete separation into two pieces as shown in fig. (12-b and c).

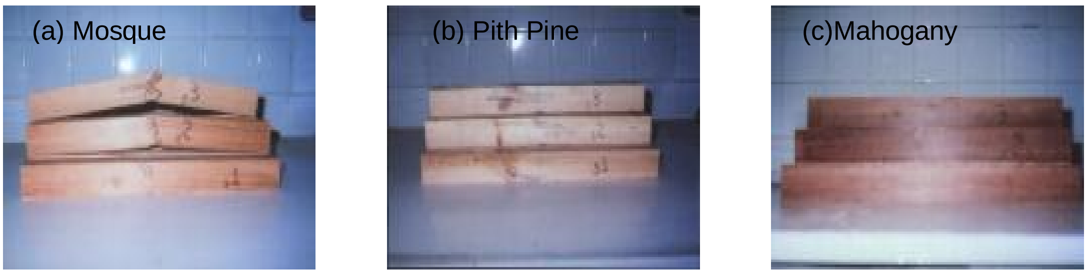

Fig. 12: Modes of failure for different types of wood

For the specimen subjected to fire, the mode of failure is shown in fig. (13 a and b). The mode of failure for the control specimens was buckling around its longitudinal axis followed by some cracks in the tension zone. For the repaired specimens by one, two or three layers of GFRP wrapping systems, the failure mode was buckling failure associated with some cracks in the matrix for the GFRP wrapping systems followed by tear of the fibers after complete failure as shown in fig. (13-a) for one layer of GFRP wrapping system and in fig. (13-b) for three layers of GFRP systems.

(a) One layer (b) Three layers

Fig. 13: Modes of failure of mosque wood columns after exposure to fire and repaired by different GFRP wrapping systems.

5. CONCLUSIONS

1. Mahogany wood has the highest compressive and flexure strength in comparison with Pitch Pine and Mosque.
2. Mahogany wood showed the lowest density loss percentage with temperature variations, water absorption and weight loss, when exposed to elevated temperature..
3. The process of restoring the original cross section of the specimens using a mixture of wood dust and polyester was very effective method which enhanced sample confinement.
4. For all types of wood, the load carrying capacity increased by increasing the number of GFRP wrapping layers at the same time of fire exposure.
5. The repairing process using GFRP wrapping system makes a good confinement for all wood types used in this investigation.
6. The failure mode was a shear failure associated with buckling for the specimens under compression. For flexural test the mode of failure was tension failure at the midspan of the sample associated with successive deflection and breaking of the sample at the middle third of the test specimen. Complete separation was observed for mosque specimen, clear deflection was observed for both pitch pin and mahogany specimens.
7. Wood specimen subjected to fire and repaired by one, two or three layers of GFRP wrapping layers have buckling failure associated with some cracks in the matrix for the GFRP wrapping layers followed by rupture of fibers.

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EVALUATE THE FLEXURAL CAPACITY OF ONE-WAY REINFORCED CONCRETE SLABS CAST BY USING COFFOR STRUCTURAL FORMWORK

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ABSTRACT

A newly structural stay in place metallic formwork system named "COFFOR" was used in construction of one way reinforced concrete slabs. Experimental and theoretical studies were conducted to evaluate the strength and behavior of such slabs. The experimental program included six one way concrete slabs. The tested slabs divided into three groups. The first group represents the reference slabs poured using traditional water proof formwork, while the second and third groups represents slabs poured using COFFOR formwork without and with reinforcement, respectively. The theoretical study evaluated the ultimate strength of the slabs based on the first principles. The results showed that the COFFOR formwork can be considered as an additional reinforcement for the slabs and the slabs main reinforcement could be reduced when using COFFOR formwork.

Keywords: Slabs, One way, COFFOR, Flexural, Capacity

INTRODUCTION

COFFOR is a light weight patented structural stay in place formwork system used for construction of reinforced concrete structures. The COFFOR system is an integrated formwork consists of two parallel faces connected to each other with a zigzag steel bar. Each face is composed of a steel screen mesh stiffened with cold formed steel channels. The panels are manufactured and assembled at the factory. COFFOR system can be used in construction of reinforced concrete slabs and walls with different shapes. When COFFOR panels used as a slab formwork only one face of the panel is used. Some research work were executed to evaluate the use of the COFFOR system as a structural formwork[1,2]. In these researches the contribution of the COFFOR in the ultimate strength of the reinforced concrete structural slabs was investigated. The behavior of the slabs poured using COFFOR formwork was studied and analyzed.

EXPERIMENTAL WORK

The research program was carried out to evaluate the flexural strength of one-way reinforced concrete slabs cast by using Coffor structural formwork. The experimental program consists of 6 reinforced concrete one-way slabs having the same dimensions.

Test Program

The tested slabs were classified into three groups. Group " I " consists of two slabs 1A and 1B. These slabs were cast in wooden formwork (using ply wood waterproof plates). Group " II " consists of two slabs 1C and 1D. They were cast in Coffor formwork without reinforcement. Group " III " : consists of two slabs 1E and 1F, which had the same reinforcement as group I and were cast in COFFOR formwork. Table 1 shows the details of the test program.

Table 1: The Experimental Program for the Tested Slabs.

Group	Specimen	dim.(mm)	RFT		Formwork
			Long	Short	
I	1A, 1B	1350x530x100	4ϕ8	9ϕ6	Water proof formwork
II	1C, 1D	1350x530x100	---	---	COFFOR formwork
III	1E, 1F	1350x530x100	4ϕ8	9ϕ6	COFFOR formwork

Details of the Tested Slabs and COFFOR Formwork

The dimensions of the tested slabs were 1350 x 530 mm in plane and 100 mm thickness. The slabs 1A, 1B, 1E and 1F were reinforced at bottom side with 4 ϕ 8 steel bars in the long direction and 9 ϕ 6 in the short direction as shown in Figure 1. The slabs 1C and 1D were cast without reinforcement. The compressive strength of concrete was 16.7 MPa. The COFFOR formwork consisted of one face of steel screen mesh stiffened with perforated cold formed steel channels at spacing of 230 mm as shown in Figure 2. The mechanical properties of the reinforcement and the COFFOR stiffening channels were shown in Table 2.

Table 2: The Mechanical Properties of Reinforcement and COFFOR.

Diameter (mm)	F_y (N/mm^2)	F_u (N/mm^2)	Elongation %
6	258	370	28.33
8	324	471	20
COFFOR	----	20 (KN)	1

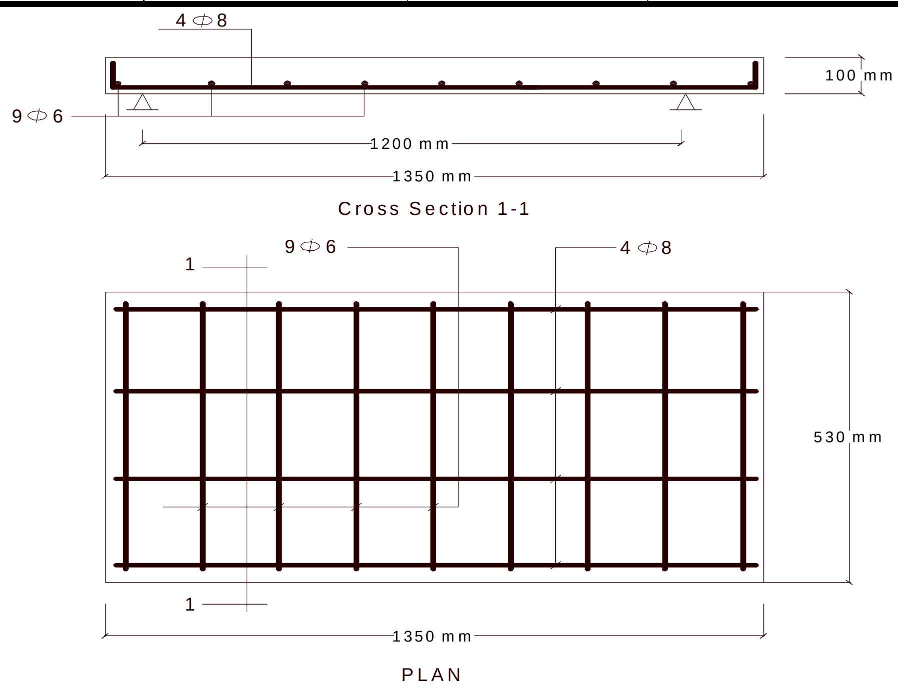
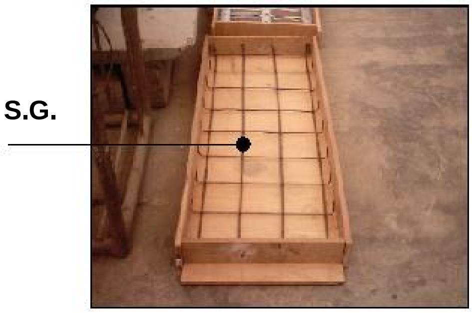
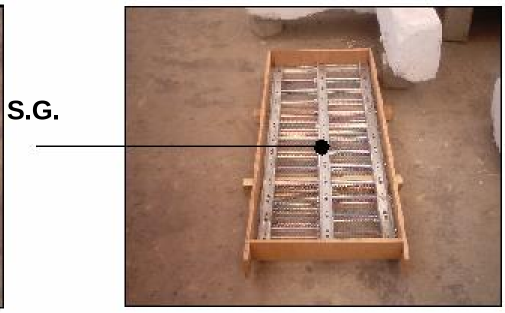


Fig. 1: Details of the Tested Slabs.



(a) Slab (1A, 1B) - RFT



(b) Slab (1C, 1D) - Coffor

(c) Slab (1E, 1F) - Coffor and RFT

Fig. 2: Details of Formwork and Position of Strain Gages.

Fig. 3: Test Setup.

Test Setup, Procedure and Measurements

The tested slab was simply supported over two rigid girders as shown in Figure 3. A line load was applied vertically at the mid span of the slab, and the slab was loaded gradually up to failure. The deflection was measured at the mid span by using $\pm$100 mm linear variable differential transducer (LVDT). The steel strain and COFFOR channel strain were recorded using electrical strain gages (S.G.) their positions were marked in Figure 2. The slabs were applied to a displacement central test performed by using data acquisition online computer system programmed using Lab View software. The central loop is shown in Figure 4.

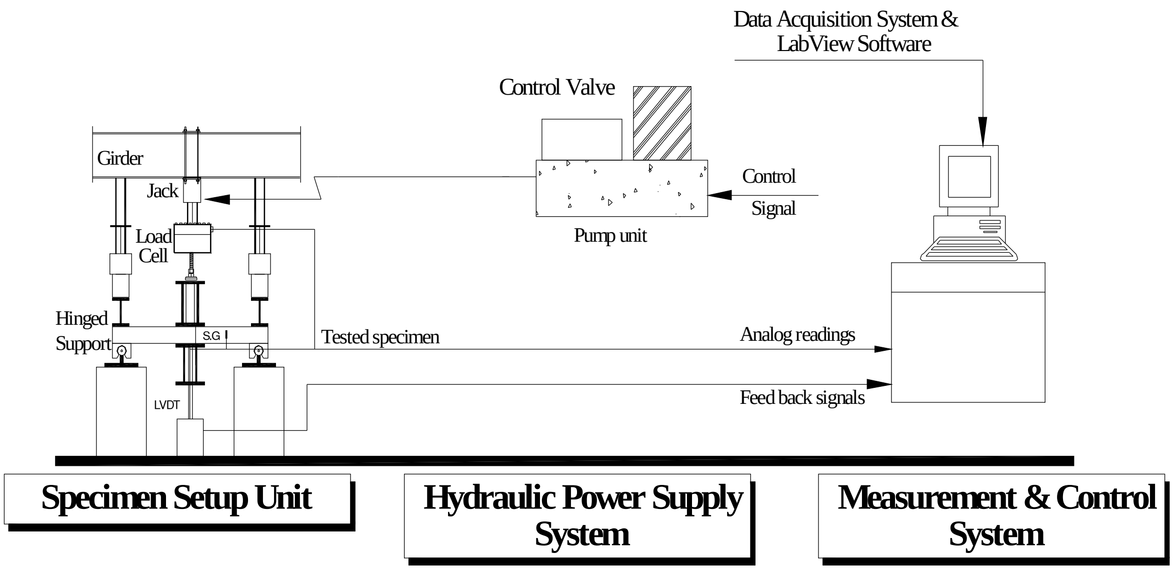


Fig. 4: Test Setup and Control System.

RESULTS AND DISCUSSION

Crack Pattern and Mode of Failure

All the slabs were failed in flexural mode of failure and the strain in the bottom reinforcement reached yielding. The crack appeared at the mid span and extended up to the top fiber at failure. In the slabs cast with COFFOR formwork the channels of COFFOR were cut at failure. Figure 5 shows the crack pattern for the tested slabs.

Load Deflection Relationships

The load deflection relationships of the tested slabs are shown in Figure 6. In Group I (RFT + wooden formwork): the ultimate load was reached at a deflection of 17.75 mm and the load was almost constant up to failure showing high ductility. In Group II (COFFOR without RFT) : the ultimate load was reached at a deflection of 12.0 mm and then the load dropped progressively up to failure. In Group III (COFFOR + RFT) : the ultimate load was reached at a deflection of 17.0 mm then the COFFOR channels were cut and the load dropped till it reduced to the strength of the reference group (Group I), and was unchanged up to failure.

(a) Group I

(b) Group II

(c) Group III

Fig. 5: Crack Pattern for the Tested Slabs.

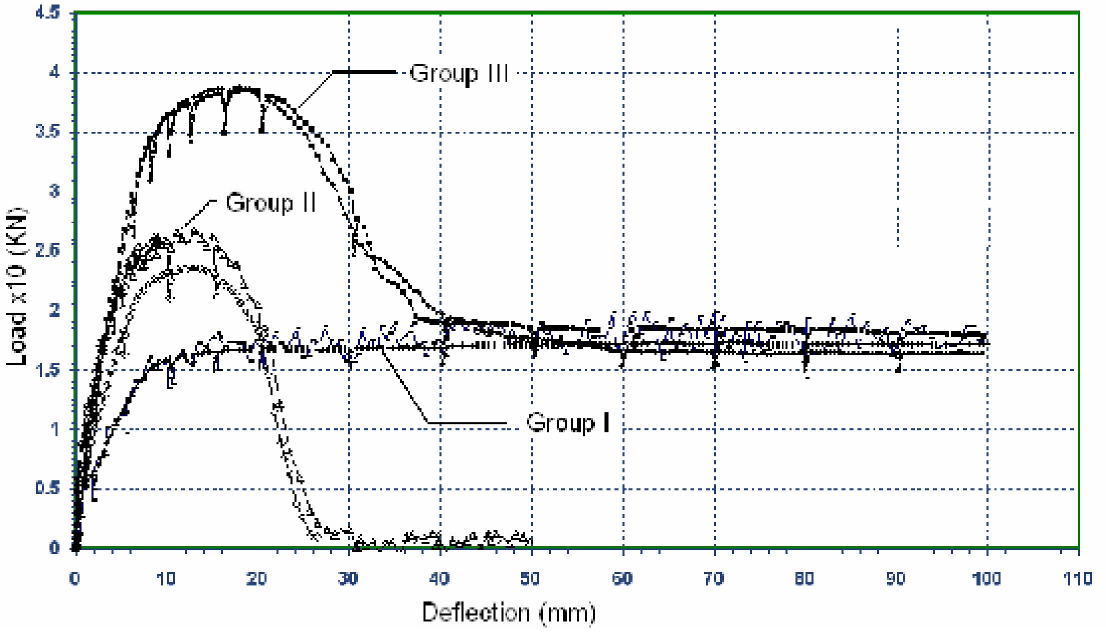


Fig. 6: Load Deflection Relation ships for the Tested Slabs.

Ultimate Strength

The ultimate load and the corresponding deflection of the tested slabs are shown in Table 3. The ultimate strength for Group II (COFFOR without RFT) and Group III (COFFOR + RFT) were found to be 1.43 and 2.2 times the ultimate load of the reference group, respectively.

Table 3: The Ultimate Load and Mid-Span Deflection.

Group	Specimen	Ultimate Load (KN)	Deflection (mm)	Ultimate Load / Ult. Load ref.
I reference	1A	18	15.5	
	1B	17	20.0	
	average	17.5	17.75	1
II	1C	26	11.0	
	1D	24	13.0	
	average	25	12	1.43
III	1E	38.5	16.0	
	1F	38.5	18.0	
	average	38.5	17	2.2

Stiffness

The stiffness for the slabs of group III (COFFOR + RFT) was almost twice the stiffness of the slabs in the reference group I, as illustrated in Table 4.

Table 4: The Linear Stiffness.

Group	Specimen	Stiffness	Ratio
I	1A	2.33	
	1B	2.50	
	average	2.40	1
III	1E	4.30	
	1F	4.90	
	average	4.60	1.91

Ductility
The ductility was defined as the ratio of the displacement at failure to the displacement at yield. The ductility of the tested slabs is shown in Table 5. It is clear that the ductility decreased considerably when using COFFOR formwork.

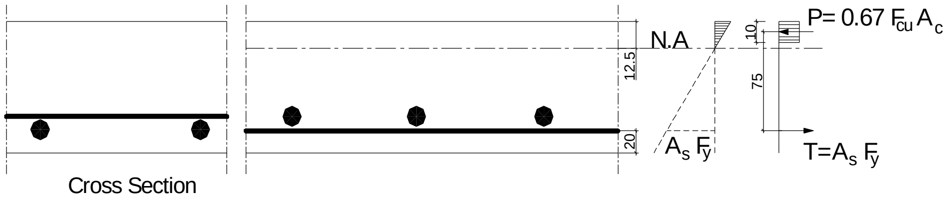
Table 5. The Ductility of Tested Slabs.

Group	Specimen	Ductility $\Delta f / \Delta y$
I reference	1A	>12.5
	1B	>15.4
	average	>14.0
II	1C	8.7
	1D	4.7
	average	6.7
III	1E	12.5
	1F	12.5
	average	12.5

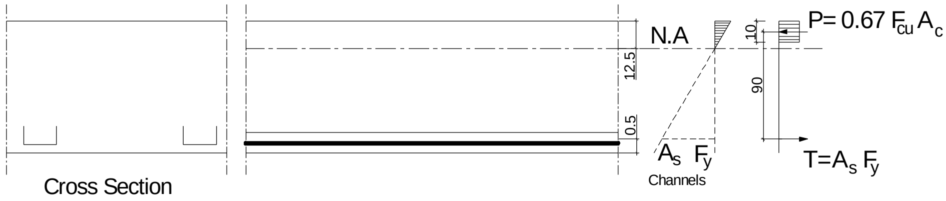
THEORETICAL ANALYSIS
The ultimate strength of the tested slabs was calculated theoretically based on the first principals and the following assumptions were considered.

- The COFFOR channels were considered as an additional reinforcement.
- Full bond between COFFOR channels and concrete was assumed (as observed in experimental test).
- The ultimate strength of the COFFOR channels was taken according to the material tests (refer to Table 2).
- The principals of ultimate theory for design of reinforced concrete were applied.
- All safety factors were considered equal to one.
- The contribution of the fabric steel mesh was ignored

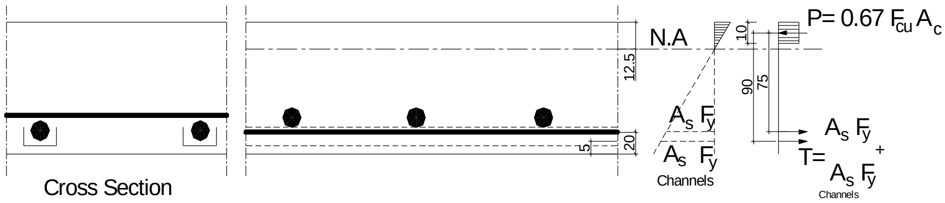
Table 6 shows the comparison between the theoretical and experimental results. The theoretical calculation of the ultimate strength of slabs cast by COFFOR formwork based on the ultimate theory was conservative. The increase of the experimental ultimate strength of slabs cast by COFFOR compared to the theoretical strength refers to the contribution of external fabric mesh .



Sectional Analasis For Slab Group I



Sectional Analasis For Slab Group II



Sectional Analasis For Slab Group III

Fig. 7: Sectional Analysis For the Tested Slabs.

Table 6: Comparison between Experimental and Theoretical Analysis.

Group	Specimen	M_u Exp. (KN.mm)x10^4	M_u Th. (KN.mm)x10^4	Ratio M_u Exp./ M_u Th	Notes
I	1A, 1B	0.54	0.4825	1.120	Water Proof Formwork + RFT
II	1C, 1D	0.75	0.5832	1.286	COFFOR Formwork
III	1E, 1F	1.15	1.065	1.085	COFFOR Formwork + RFT

CONCLUSION

Based on the experimental program and the theoretical study conducted in this research, the following conclusions may be drawn:

- COFFOR formwork can be considered as an additional reinforcement for slabs and it increases the ultimate strength by 120% compared to the slabs cast by traditional wooden formwork.
- The stiffness of the slabs cast by COFFOR formwork was about two times the stiffness of the reference slab cast in wooden formwork.
- The behavior of the slabs cast using COFFOR formwork showed poor ductility consequently, additional material safety factors should be considered in design to overcome the lack of ductility.
- The ultimate strength design theory can be applied to predict the strength of the slabs cast using COFFOR formwork.
- Full bond was observed between COFFOR channels and concrete.
- At failure the COFFOR channels were cut while the reinforcement steel was still at yield

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ASSESMENT OF A NEW CONSTRCUTION TECHNIQUE FOR REINFORCED CONCRETE STRUCTURES

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ABSTRACT

Last years have shown the development of new construction techniques in the purpose of saving time and cost. Stay-in-place metal formwork is one of those developed construction techniques that is faster to implement and does not need skilled labor. This research paper assesses the performance of a stay-in-place construction formwork namely "COFFOR" system. The "COFFOR" system is used for constructing various structural reinforced concrete members that includes foundation, walls, columns, beams and slabs. The "COFFOR" system is composed of three main items: expanded metal sheets, vertical galvanized stiffeners and horizontal bent tie bars.
An experimental program comprising eleven specimens was carried out. The specimens were divided into three groups, A, B, C, and were tested to investigate the homogeneity, axial behavior of concrete walls and axial behavior of concrete columns, respectively cast using "COFFOR" structural formwork. The test program included three homogeneity specimens, four wall specimens and four column specimens, respectively.
The results showed that it is better to use concrete with plastic consistency in members constructed using "COFFOR" structural formwork. The concrete members constructed using COFFOR formwork possess a good level of homogeneity. Regarding the structural behavior, it was found that the reinforced concrete "COFFOR" walls are more ductile compared to the conventional reinforced concrete walls while a reduction in their capacity was observed. Also, more work is needed to enhance the load carrying capacity of the reinforced concrete columns constructed using the COFFOR formwork system.

Key words: Construction technique, column, walls, Formwork, Stay-in-place

INTRODUCTION

The world over population forced the development of non-traditional construction techniques. Stay-in-place formwork is one of those techniques which has the ability to save both time and construction costs [1]. "COFFOR" construction formwork is a stay-in-place metal form commonly used in concrete constructions. "COFFOR" panels are produced in factory then fixed at the site to be ready to receive concrete. After pouring of concrete, the "COFFOR" formwork and the hardened concrete work together integrally.

Few research efforts have been directed to study the "COFFOR" construction technique and its development. Through the last years, Chinese researchers have spent grateful effort in the field of "COFFOR" application and behavior of structural elements cast using "COFFOR" system [2,3]. In addition, the "COFFOR" system was examined and approved by CSTB [4].

"COFFOR" is considered as structural stay-in-place formwork system for concrete construction. It is composed of two filtering grids made of rib lath, expanded metal with ribs, reinforced by vertical C channel stiffeners. The grids are connected by zigzag Patterned articulated rebar loops that fold. The rib lath and the C channel vertical stiffener are produced from cold rolled and hot galvanized steel, respectively. The articulated rebar loops are made of steel rebars of 5 mm diameter. The "COFFOR" panels can be reinforced by both vertical and horizontal conventional rebars as required by the structural design. Figure 1 shows the components of the COFFOR system.

1-Filtering grids:
Expanded metal sheet

2-Vertical stiffeners:
Galvanized C-channel sections.

3-Connection:
steel loop rebars in zigzag pattern

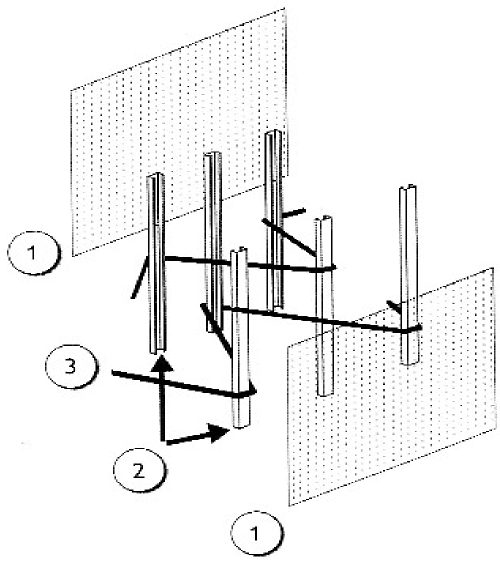


Fig. 1: Components "COFFOR" Structural Formwork

The construction process is very simple, after placement of the panel, they are aligned; braced, set the openings, insert the rebar, penetrate the service conduit, and finally pour concrete. "COFFOR" panels can be also delivered with integrated insulation on the exterior side. The insulation material is polystyrene or polyurethane of various thickness 40mm,60mm, or 80mm.

According to the manufacturer technical information guide [5] the "COFFOR" system is said to have many advantage over the traditional formwork systems:
- It is 2 to 3 times faster,
- they are not stripped as they stay in the wall
- bonding of finishing is excellent due the rough nature of the form surface.
- The problems of traditional reusable water proof heavy formwork are eliminated. "COFFOR" - does not require skilled labor or special equipment.
- The "COFFOR" system is environmental friendly, It protects depletion of the nature resources (mainly wood) and save the income of wood-poor countries.

This paper is a part of a research program conducted at Housing and Building National Research Center to assess the feasibility of utilizing "COFFOR" construction technique in Egypt. The paper presents an experimental program that includes eleven specimens designed to study the homogeneity and axial behavior of structural elements constructed using "COFFOR" system. The Concrete water/cement ratio and reinforcement ratio were the main parameters, respectively. The results were compared with reinforced concrete elements constructed using traditional wood formwork.

EXPERIMENTAL PROGRAM

Test Specimens

A total of eleven specimens, divided into three groups, A, B, C, were tested to investigate the homogeneity, axial behavior of concrete walls and axial behavior of concrete columns, respectively cast using "COFFOR" structural formwork. Table 1 shows the details of the tested specimens.

Table 1: Details of the Tested Specimens

Group	I.D.	Specimen Dimension H cm	W cm	B cm	Conc. Mix	Vertical Rft.	Horizontal Rft.	Remarks
(A) Homogeneity specimens	H1	130	27	20	1	-	-	"COFFOR"
	H2	130	27	20	2	-	-	"COFFOR"
	H3	130	27	20	3	-	-	"COFFOR"
(B) wall specimens	W1	122	76	15	1	-	-	"COFFOR" only
	W2	122	76	15	1	7Φ10/face	13 Φ8/ face	"COFFOR" + Rft.
	W3	122	76	15	1	5Φ8/ face	7 Φ8 / face	Code minimum Rft.
	W4	122	76	15	1	7Φ10/face	13 Φ8 / face	Rft.
(C) column specimens	C1	140	30	16	1	-	-	"COFFOR" only
	C2	140	30	16	1	6 Φ10	10 Φ 8 /m	"COFFOR" + Rft.
	C3	140	30	16	1	4 Φ 6	5 Φ 6 /m	Equivalent "COFFOR"
	C4	140	30	16	1	6 Φ10	10 Φ 8 /m	Rft.

The three specimens of Group A, H1, H2 & H3, were cast to investigate the homogeneity of the members. Three concrete mixes with different W/C ratios were used to produce three different consistency levels: plastic, wet and very wet. The homogeneity of the specimens was investigated by performing ultrasonic pulse velocity and concrete core tests. Three core samples were taken form each specimen, while the Ultrasonic Pulse Velocity was measured at seven positions along the height of the specimen. Figure 2 shows both the dimension configuration and position of the performed tests. In addition, photo 1 shows front and side view of specimen H1.

To investigate the axial behavior of concrete elements cast using the "COFFOR" structural formwork, four wall specimens, W1 to W4, representing group B were tested. Two specimens W1& W2 were cast using "COFFOR" formwork. Wall W1 was cast without conventional reinforcement while wall W2 was reinforced vertically by 7 # 10 high grade steel 36/52 in each face and was reinforced horizontally by 13Φ8 mild steel 24/37 per face. The other two specimens W3 and W4 were cast using traditional wood formwork. Wall W3 was reinforced by the minimum reinforcement required by the Egyptian Code for Design and Construction of Concrete Structure [6]. On the other hand, wall W4 was reinforced by the same reinforcement as wall W2. The upper and lower ends of the walls were additionally reinforced by Φ 6 U shape rebars spaced at 140 mm to strengthen the ends against any kind of premature failure. Full details of the tested walls are shown in Figure 4. Also, photo 2 shows the reinforcement cages for walls W2 and W3, respectively.

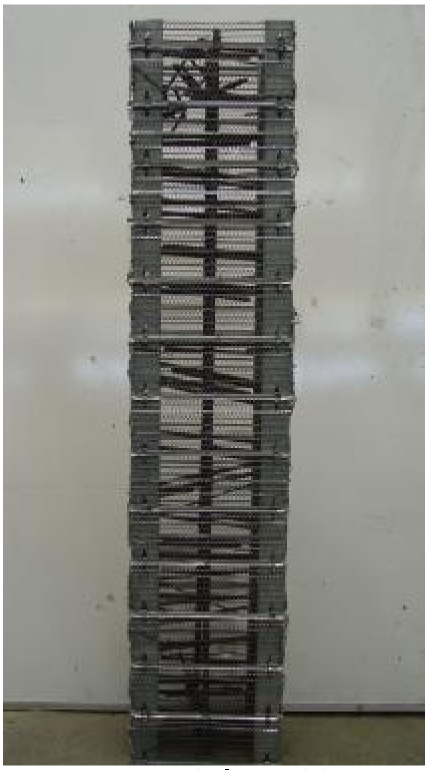
To investigate the possibility of using "COFFOR" structural formwork as a stay-in-place form for casting reinforced concrete columns, four column specimens, C1 to C4, were cast. Two columns C1 and C2 were cast using the "COFFOR" system. Column C1 was constructed without any conventional reinforcement while, column C2 was vertically reinforced by 6 Φ 10 high grade steel 36/52 and was horizontally reinforced by 10 Φ 8/m' mild steel 24/37. The reference columns C3 and C4 were cast using the traditional formwork. Column C3 was vertically reinforced by 4 Φ 6 and horizontally reinforced by 5 Φ 6 /m mild steel 24/37, which are equivalent to the vertical and horizontal stiffeners of the "COFFOR" formwork. On the other hand, column C4 was provided by the same reinforcement as column C2 to be a control specimen for it. Figure 3 shows full details of the tested columns while, photo 3 shows the reinforcement cages for column specimens C2 and C4.

Photo1: Preparation of Homogeneity Specimens H1

Wall W2

Wall W3

Photo 2: Reinforcement Cage for Wall Specimens



Column C2

Column C4

Photo 3: Reinforcement Cage for Column Specimens

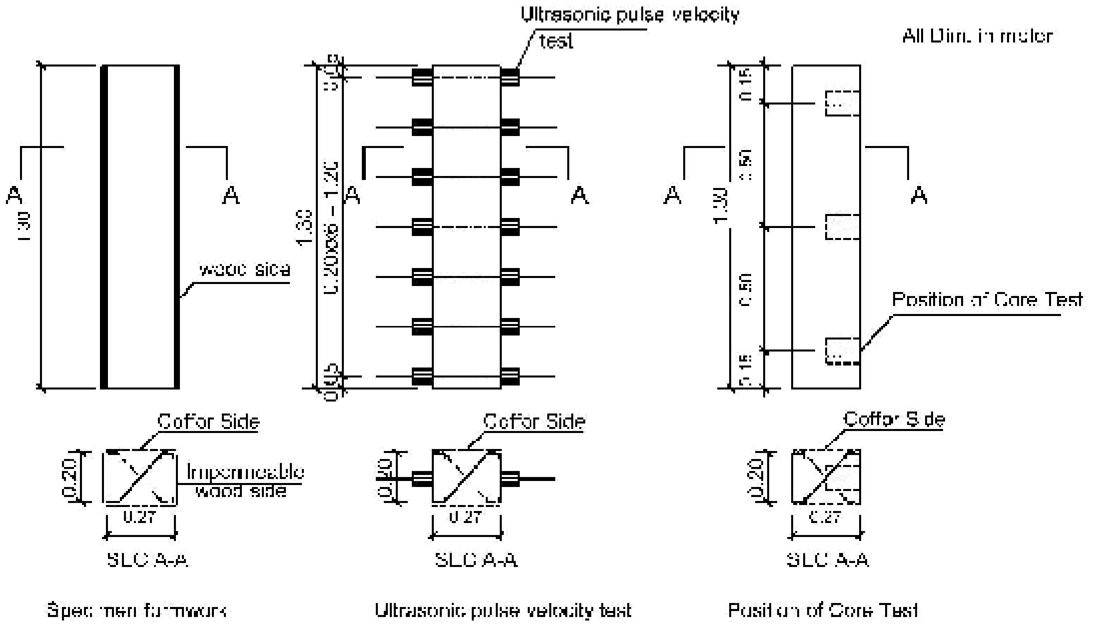


Fig. 2: Details and Tests of Typical Specimen of Group (A)

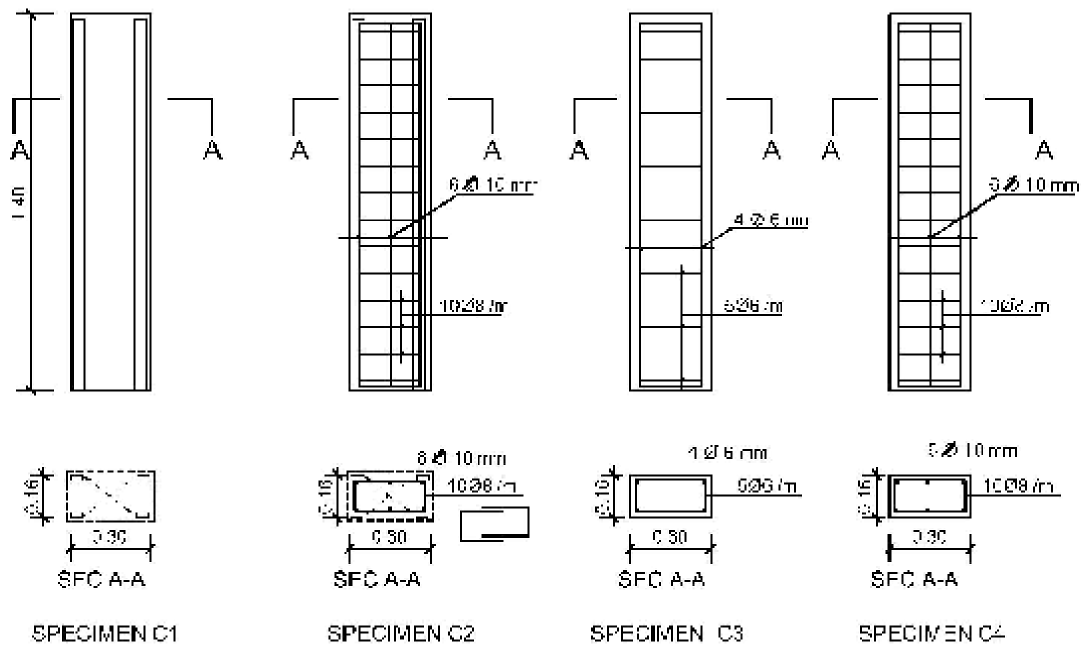


Fig. 3: Reinforcement Details of Group (C)

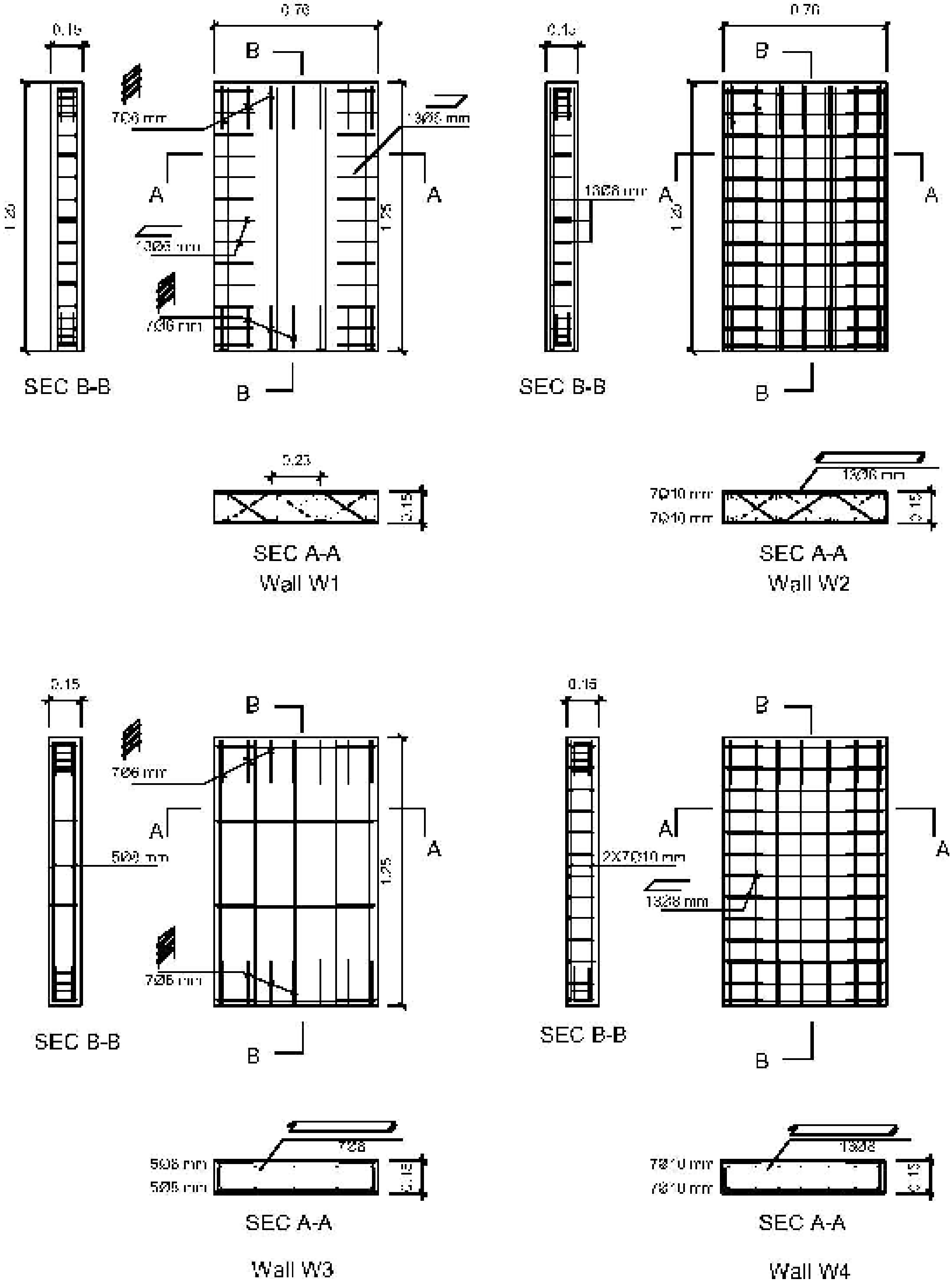


Fig. 4: Reinforcement Details of Group (B)

Materials

The concrete mix consisted of fine aggregate, coarse aggregate, cement, and water. The fine aggregate used was natural siliceous sand. The coarse aggregate used was crushed stones of nominal size equals 10 mm. Ordinary Portland cement from Suez Company was used. The materials were tested to assure their compliance with the Egyptian Standard Specifications.

Three concrete mixes 1,2 &3 were designed to possess three different levels of consistency: plastic, wet and very wet . As seen in Table 1, the three mixes were used to cast the homogeneity specimens H1, H2 and H3, respectively, of group A. On the other hand, mix1 was used to cast specimens of group B and group C. Table 2 gives the quantities of materials required for one cubic meter of fresh concrete for each concrete mix. Concrete test cubes of nominal size 15x15x15 cm were cast at the same time as the specimens to determine their concrete compressive strength. The compressive strengths of the concrete used to cast the specimens will be presented when describing each testing phase. Photo 4 shows the homogeneity specimen H2 during cast of concrete while, photo 5 shows the cast of column specimens.

High strength steel-deformed rebars of 10 mm diameter and mild steel-smooth rebars of 6 and 8 mm diameter were used in specimens' reinforcement. The steel reinforcements were obtained from El-Ezz steel Company. Tension tests were performed on the steel specimens using Shimadzu 500-KN universal testing machine according to the Egyptian Standard Specifications ESS 262-1999 [7]. Moreover, steel sheet specimens were extracted from the COFFOR formwork to perform tension test according to the American Standard Specification ASTM A370-97a[8]. Table 3 gives the physical and mechanical properties of the used reinforcements and the steel metal of COFFOR channel.

Table 2: Mix proportions of the concrete mixes

Mix No.	Cement (kg)	Dolomite Crushed Stone (kg)	Sand (kg)	Water (liter)	w/c	Slump (cm)
1	330	1209	604	237.6	0.72	8.0
2	330	1050	525	290	0.88	18.0
3	330	1026	480	303.6	1.04	22.5

Table 3: Properties of Reinforcing Steel and COFFOR Channels

Dimensions (mm)	Area (mm^2)	Yield strength (kg/cm^2)	Ultimate strength (kg/cm^2)	Elongation (%)
ϕ 6	28.56	2498	3709	23.9
ϕ 8	49.48	3223	4716	21.7
# 10	77.47	3962	5962	19.0
0.55x13.16*	7.48	-	5261	10.4

* Steel sheet from COFFOR Channels
- The specimen did not display yield value

Photo 4: Casting of homogeneity Specimen H2

Photo 5: Casting of column specimens

Testing and Instrumentation Procedure

Core test specimens were extracted from homogeneity specimens in accordance to Egyptian Standard ESS 1658-95 [9]. The core specimens were 100 mm in diameter and 150 mm length. The positions of the concrete cores are given in Figure 2. The specimens were cured for 2 days prior to the compressive testing. The ultrasonic pulse velocity test was conducted on the homogeneity specimens as a measure of the concrete quality. The test was performed along the height of the specimens at the positions illustrated in Figure 2.

For wall specimens, two linear variable displacement transducers (LVDT) were used to measure the vertical strains. The Two transducers were attached at the opposite sides of the wall specimens at a gauge distance of 70 cm. For column specimens, two vertical LVDTs were attached at a gauge distance of 50 cm. The data was recorded using a data acquisition system. All walls and columns were axially loaded using a compressive testing machine of 500 ton capacity. The specimens were loaded gradually till failure while continuous readings of the load and strains were recorded automatically.

DISCUSSION OF EXPERIMENTAL RESULTS

Homogeneity Specimens

Three different concrete mixes with different w/c ratios were designed to cast the homogeneity specimens. The purpose of this test group is to show the effect of w/c ratio on the drainage phenomena and homogeneity of concrete cast using the "COFFOR" structural formwork.

Examining the results given in Table 4 and 5 the following findings was found.
Increasing the water/cement ratio by 22%, from 0.72, for specimen H1, to 0.88, for specimen H2, resulted in a significant increase in the percentage of drainage of mortar by 86%. Further increase of the water/cement ratio to 0.92, specimen H3, resulted in dramatic increase in the percentage of drainage mortar by 216%. This indicates the harmful effect of increasing the w/c ratio on the percentage of drainage. It is better to cast the COFFOR system using concrete with plastic consistency in order to reduce the loss of the cement mortar.

Table 5 shows the results of the concrete compressive strength determined from core tests as well as from the standard cube tests. The concrete core strength of the column was considered as the average value of the results of the three core samples extracted through the height of each column of group (A). The average value was divided by 0.75 in order to obtain the equivalent cube strength according to the requirement of "Egyptian Code for Design and Construction of Concrete Structure". The equivalent cube strength values were 3%, 3.7% and 7.2% less than the standard cube strength of specimens H1, H2 and H3, respectively. This is due to the loss of cement content during drainage. It should be noted that the reduction in the strength of specimen H3 is not proportional to the high percentage of mortar drainage. This is because the angles of the filtering grid are designed to allow excess water to pass out while maintaining a major part of the solid material inside.

The results of ultrasonic pulse velocity are shown in Fig. 5. As can be seen, there is no significant variation of the pulse velocity along the specimens heights. The average value of the pulse velocity of the specimens H1, H2 and H3 were 3685, 3667 and 3458 m/sec, respectively. The coefficients of variations were 1.8, 1.5 and 1.6%, respectively. So it can be concluded that the concrete members constructed using COFFOR formwork possess a good level of homogeneity.

Table 4: Percentage of Drainage

Specimen I.D	Weight of conc. member (W) (Kg)	Weight of drained mortar (W_d) (Kg)	% drainage $\frac{(W_d)}{(W)}$ x 100	w/c ratio
H1	154	6.00	4.00	0.72
H2	154	11.00	7.10	0.88
H3	154	34.60	22.40	0.92

Table 5: Core Test Results

Specimen I.D	Average equivalent cube strength f_{ceq} (Kg/cm^2)	Standard Cube Strength f_{cu} (Kg/cm^2)	f_{ceq} / f_{cu} %
H1	138.67	143	96.5
H2	120.33	125	96.3
H3	86.33	93	93.5

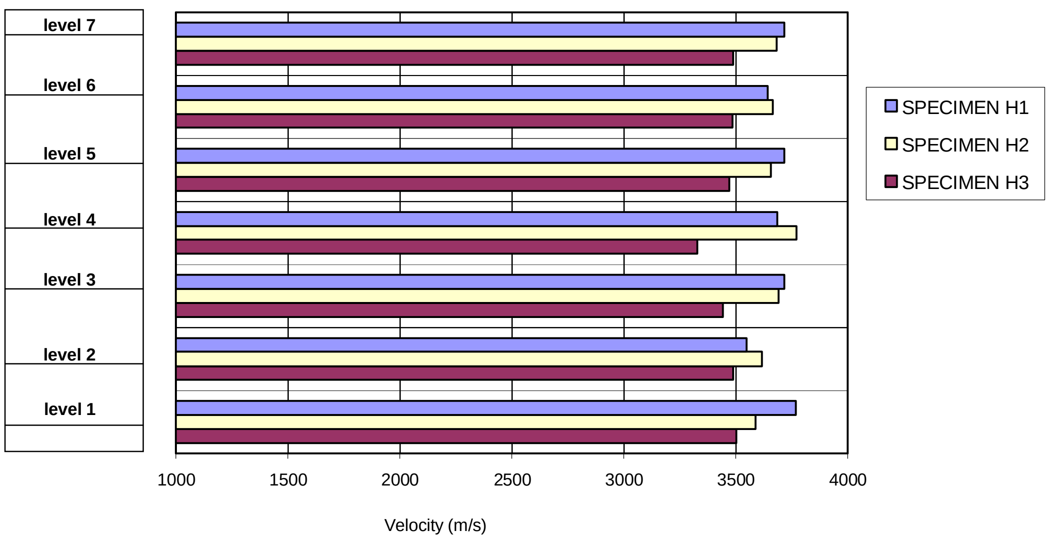


Fig. 5: Ultrasonic Pulse Velocity of Test Specimens H1,H2, and H3

Wall Specimens

Four "COFFOR" wall specimens, W1, W2, W3 and W4 were tested under axial load. Fig. 6 and Fig. 7 show the load vertical-strain relationships of the tested walls. Fig. 6 presents the comparison between walls W1 and W3 while Fig. 7 shows the comparison between walls W2 and W4. In addition, Table 6 presents the ultimate load, vertical strain at ultimate load and at 50% of ultimate load on the descending branch of the load-strain curve.

As seen in Fig. 6, both of the "COFFOR" wall W1 and the RC wall W3 have showed overall similar behavior regarding ultimate load and vertical stiffness. They recorded an ultimate load of 143.2 and 143.72 ton, respectively. This means that the COFFOR steel sections compensate the capacity of the minimum reinforcement required by the code. Moreover, the "COFFOR" wall W1 has showed more ductile behavior beyond its ultimate load. Referring to Fig. 7, the reinforced COFFOR wall W2 failed at an ultimate load 14% lower than that of the RC wall W4. This is because of the geometrical design of the COFFOR formwork that did not allow providing the vertical bars with adequate cross ties as in conventional walls. On the other hand, the behavior of both walls was nearly similar regarding the stiffness and post peak behavior.

The failure mechanism of the COFFOR walls W1 and W2 started with the premature buckling of the vertical stiffener C channel of the "COFFOR" panel followed by cracking of concrete. After that, wall W2 experienced buckling of longitudinal reinforcement bars. It is worthwhile mentioning that, the stiffness of the C channel and the confinement effect of the expanded metal sheet of the "COFFOR" panel decreased the rapid degradation of the capacity of the COFFOR wall beyond the ultimate load, i.e. better ductility. Moreover, the existence of expanded metal sheet preserved the integrity of the COFFOR walls after failure. The failure of the conventional reinforced concrete walls W3 and W4 was more brittle. It started with cracking and spalling of the concrete cover followed by the crushing of concrete. Finally, buckling of vertical bars occurred. Photo 6 shows the appearance of walls W1 and W4 after testing.

Table 6: Test Results of wall specimens

Wall No.	Ultimate Load (Ton)	Strain at Ultimate Load ε_{ult}	Strain at 50% of ultimate Load $\varepsilon_{50\%ult}$	Notes
W1	143.20	0.0088	0.0128	COFFOR
W2	140.51	0.0099	0.0140	Reinforced COFFOR
W3	143.72	0.0085	0.009*	Conventional RC (Min. Rft.)
W4	168.89	0.0088	0.0133	Conventional RC

* This value was obtained through extrapolation of the strain value at 63% of ultimate load

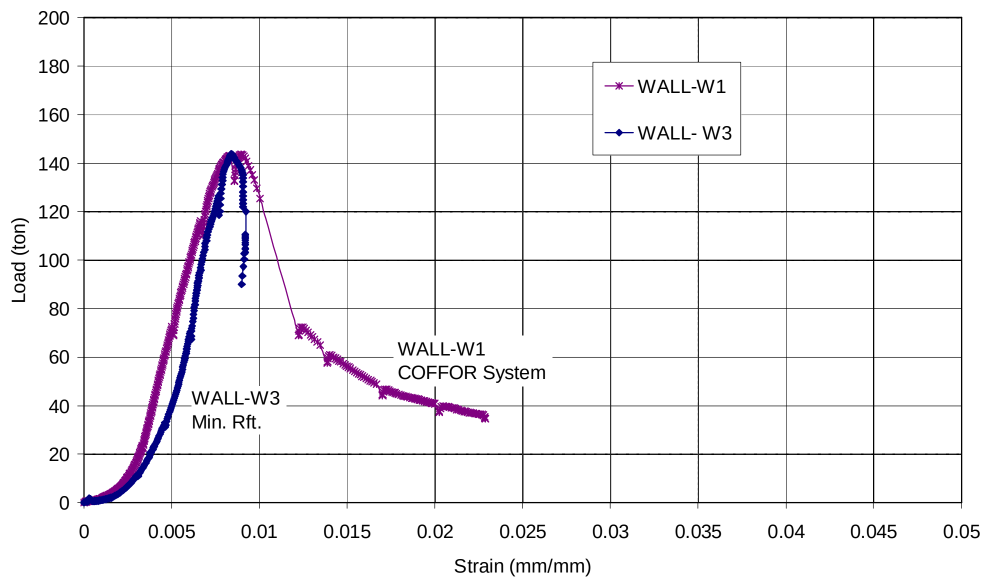


Fig. 6: Load-Vertical Strain Relationship of Walls W1 and W3

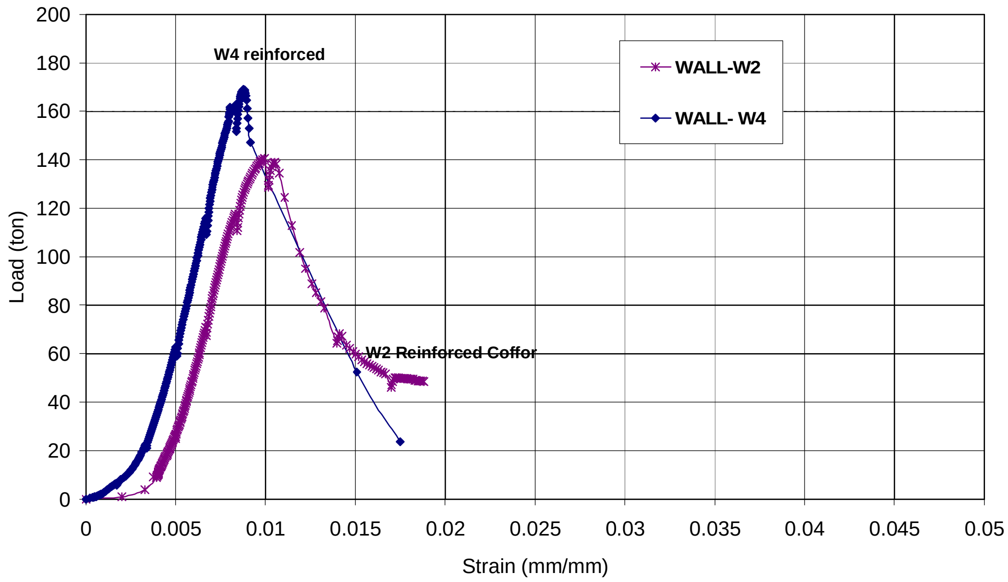


Fig. 7: Load-Vertical Strain Relationship of Walls W2 and W4

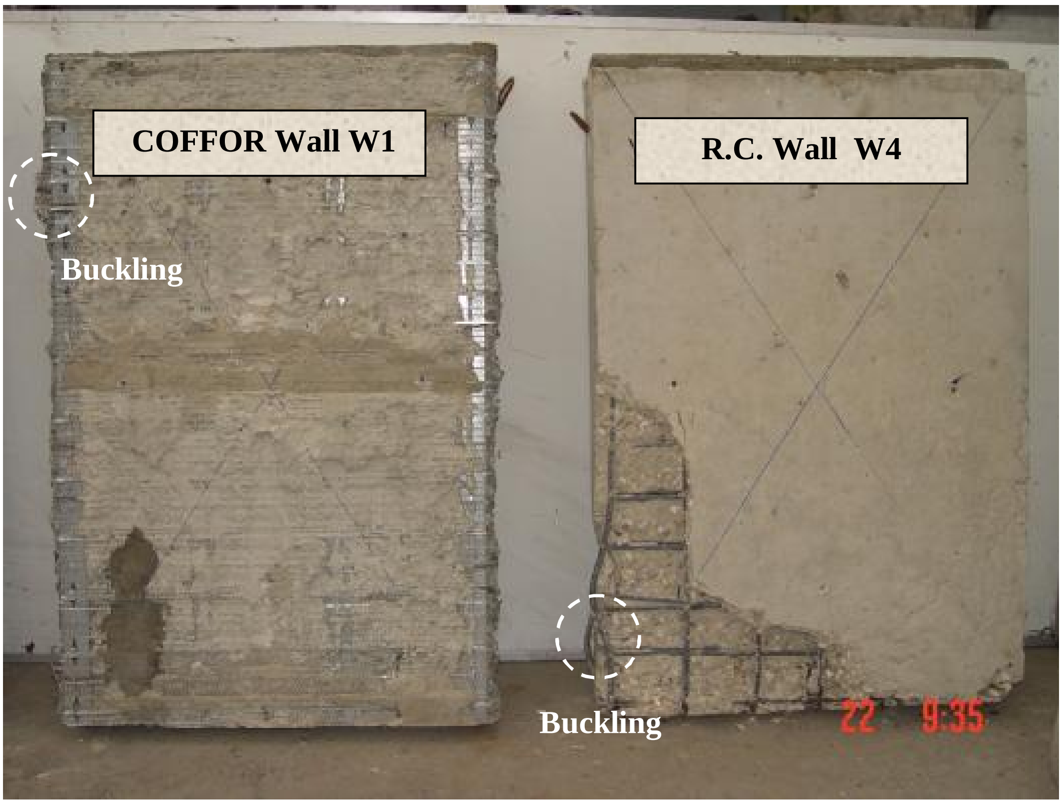


Photo 6: Appearance of Walls W1 and W4 after testing

Column Specimens

To study the feasibility of applying the "COFFOR" structural formwork as a stay-in-place form for reinforced concrete columns, two columns were cast using the "COFFOR" system and another two columns were cast using the traditional wood form. Table 7 shows the ultimate load, strain at ultimate load and strain at 50% of ultimate load on the descending branch of the load-strain curve of the tested columns. Fig. 8 compares the load- vertical strain relationship of the "COFFOR" column C1 and its control RC column C3. Fig. 9 shows the same relationship between the reinforced "COFFOR" column C2 and the traditionally reinforced concrete column C4.

Studying the figures 8 and 9 a general trend can be observed, the "COFFOR" columns had lower ultimate loads, lower stiffness and higher ductility when compared to the conventional reinforced concrete columns.

The ultimate load of the COFFOR column C1 is 24% lower than that of the conventional reinforced concrete column C3. However, column C1 has showed significantly higher ductility in comparison with column C3 as indicated by the obtained values of strain at 50% of the ultimate load on the descending branch. The higher ductility of the "COFFOR" column can be attributed mainly to the confinement of concrete core provided by the vertical stiffener and the metal sheet of the "COFFOR" panel.

Fig. 9 compares the behavior of reinforced "COFFOR" column C2 and the conventional reinforced concrete column C4. As similar to the un-reinforced "COFFOR" column, the reinforced "COFFOR" column C2 has showed lower ultimate load in comparison with the conventional reinforced concrete column C4. The ultimate load of column C2 was 48% lower than that of column C4.

Comparing the load-strain results of the un-reinforced "COFFOR" column C1 and the reinforced "COFFOR" column C2, it can be seen that, there is no significant gain from the reinforcing rebars. This unexpected behavior may be explained through their failure patterns presented in Photo 7. The premature buckling failure of the vertical stiffeners of the "COFFOR" structural form work is the governing factor that controls the failure of the "COFFOR" column. When the load acting on the C channel of "COFFOR" vertical stiffener reaches the buckling load, the C channel buckle outside the column face forcing the vertical reinforcement to buckle. Thus, no enhancement in the strength occurred due the addition of traditional reinforcement. So more research work is needed to enhance the load carrying capacity of reinforced concrete column constructed using the COFFOR system.

The traditionally reinforced concrete columns failed following the common failure mechanism of reinforced concrete column. The failure started with the propagation of vertical cracks followed by spalling of concrete cover and finally buckling of longitudinal reinforcement took place. The failure patterns for the tested columns C3 and C4 are shown in photo 8.

Table 7: Results of Column Specimens

Column identification	Ultimate Load (Ton)	Strain at Ultimate Load ε_{ult}	Strain at 50% of ultimate Load $\varepsilon_{50\%ult}$	Notes
C1	57.0	0.0020	0.00386	COFFOR
C2	50.0	0.00157	0.0048	Reinforced COFFOR
C3	75.0	0.00229	0.00240	Conventional RC (equivalent COFFOR)
C4	96.0	0.0020	0.0035	Conventional RC

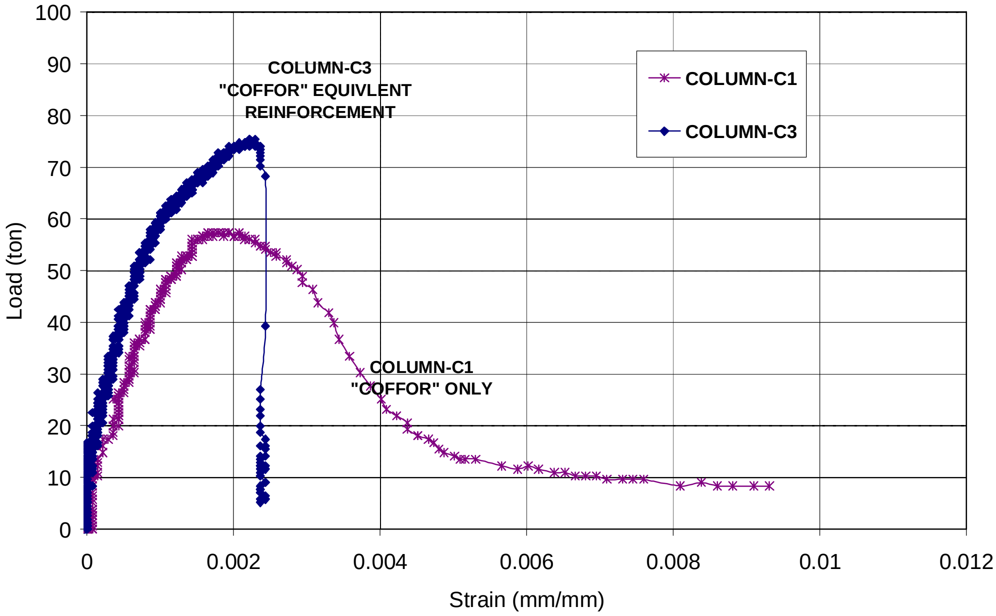


Fig. 8: Load-Vertical Strain Relationship of Columns C1 and C3.

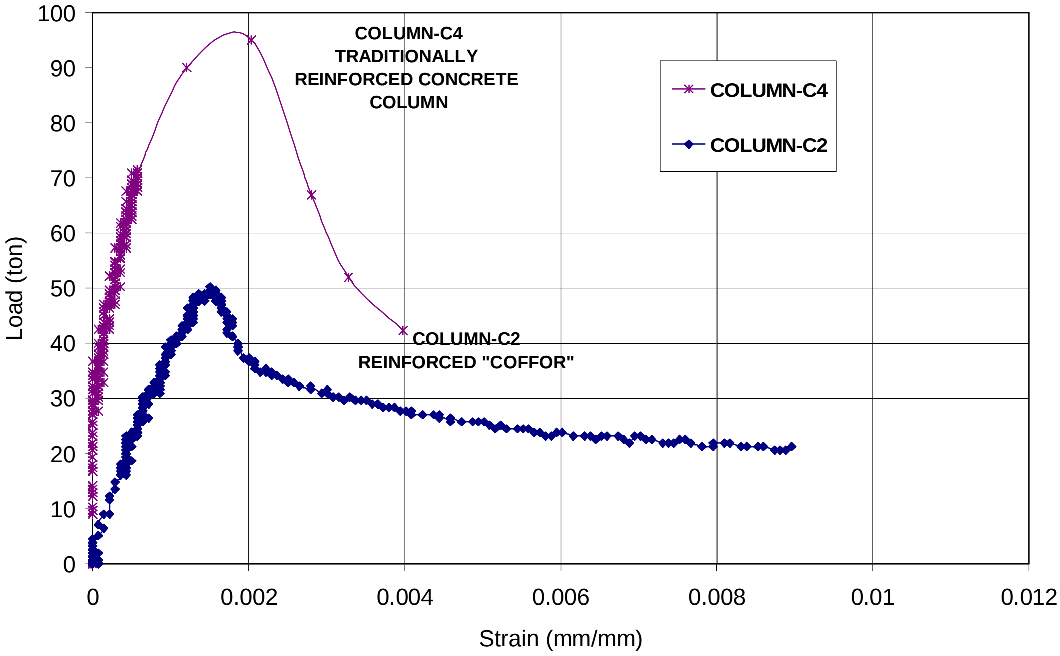


Fig. 9: Load-Vertical Strain Relationship of Columns C2 and C4.

Column C1

Column C2

Photo 7: Appearance of Column Specimens C1 and C2 after Testing

Column C3

Column C4

Photo 8: Appearance of Column C3 and C4 after Testing

CONCLUSIONS

Based on the experimental work that investigated the performance of "COFFOR specimens, the following findings can be drawn:

1- It is recommended to cast the COFFOR system using concrete with plastic consistency in order to reduce the loss of the cement mortar.
2- The concrete members constructed using COFFOR formwork possess a good level of homogeneity as indicated by the ultra sonic pulse velocity results.
3- For wall members, the "COFFOR" steel sections compensate the capacity of the minimum reinforcement required by the Egyptian code and result in a more ductile response.
4- The reinforced "COFFOR" walls showed more ductile behavior beyond its ultimate load compared to the conventional reinforced concrete walls.
5- The reduction in the capacity of the reinforced COFFOR walls indicated the need for enhancing the confinement method for the provided conventional rebars.
6- The existence of expanded metal sheet preserved the integrity of the COFFOR walls after failure.
7- The "COFFOR" columns had lower ultimate loads, lower stiffness and higher ductility when compared to the conventional reinforced concrete columns.
8- More work is needed to enhance the load carrying capacity of the reinforced concrete columns constructed using the COFFOR system

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EXPERIMENTAL BEHAVOIR OF REINFORCED CONCRETE COLUMNS STRENGTHENED BY STEEL JACKET

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ABSTRACT

This research presents experimental behavior of eight rectangular reinforcement concrete columns of low compressive strength concrete which strengthened by steel jacket. All the specimens were rectangular concrete column with aspect ratio equal two, 15x30 cm cross section, reinforced longitudinally by 6 bars 10mm diameter and transversely by ties 6 mm diameter each 15 cm. The column was provided at the two ends by two caps 15x50 cm cross section and 22.5cm height. The total length of the column was 165 cm. The concrete cube strength of the columns was 200 kg/cm2. The steel jacket consisted of vertical four angles in the corners and horizontal straps plates which were distributed along the length of column and welded between the corner angles. The experimental behavior of the strengthened concrete column was studied with variable size of corner angles and different spacing of the strap plates using either anchor in the middle on the long side of the columns or without anchors. The obtained results of the strengthened columns were plotted and the ultimate load was compared with the analytical analysis which was suggested by Wang [7]. The comparison between the results showed a good agreement for the ultimate load of the strengthened columns. Based on the obtained results recommendations were issued.

Keywords : Concrete, Column, Steel jacket, Strengthening **.**

INTRODUCTION

The steel jacket is one of the earliest technique of repair and strengthening of the reinforced concrete columns. The technique is widely used for seismic retrofit of non ductile reinforced concrete frame columns with inadequate shear strength. The most advantage of this technique is to increase the ductility by strengthening the columns without any significant modification regarding the concrete mass. The steel jacket acts as a passive confinement of the reinforced column to carry more axial loads with convenient ductile response. Jersa et. al. [1] carried out an experimental program to investigate the rehabilitation of shear critical concrete columns by using rectangular steel jackets. Several types of steel jackets were tested, including rectangular solid steel jackets and partial steel jackets. The test results indicated that the thin rectangular steel jacket can be highly effective to retrofit the reinforced concrete columns with inadequate shear resistance. Priestley [2] conducted a theoretical and experimental investigation to study the shear failure mode of reinforced concrete bridge columns to establish the effectiveness of full height steel jackets for enhancing the seismic shear strength. Priestly suggested a set of design equations and a design methodology for the calculation of the required thickness for circular and rectangular steel jacket in order to increase the shear strength. Al-Tuhami [3] suggested the application of pre-stressing force on the column before welding the lateral steel plate of the steel jacket. This technique provided additional external forces for the confinement of the reinforced column and increased the capacity of the axial load. The results of the experimental work by Al-Tuhami showed that the new technique increases the ultimate load of the reinforced concrete columns by 16% to 86% according to the level of pre-stressing regarding the strengthened columns. Sherif El-Zeiny [4] used the pre-stressing steel jacket technique to investigate the experimental and theoretical behavior of the strengthened concrete

columns method. He indicated that the strengthening using this technique increases the capacity of axial load of the concrete columns up to 146% by using suitable external pre-stressing force and lateral plates. Based on the experimental and the theoretical results, he suggested a new design formula to predict the ultimate axial load of the strengthened concrete columns with steel jackets. Mander, Priestly, and Park [5,6] proposed a theoretical model of the concrete column by evaluating the compressive stress-strain relationship for the confined and unconfined concrete column. This allow prediction of the ultimate load of the concrete column. Wang [7] modified Mander's model using an analytical method for evaluating the short – term axial load deformation behavior of rectangular and square reinforced compression member confined with glass hoops. He suggested a closed form equation that can be used to determine axial load versus longitudinal strain of the rectangular FRP jacked .
In fact, most building codes do not provide the required design guide lines for the strengthened concrete columns using steel jackets. The aim of this research is to produce a guide line to design the concrete columns using steel jackets.

TEST SPECIMENS

Eight pin-end concrete columns, whose details are given in table (1), were tested in this study. The cross section of the columns was 15x30 cm, the total length of the columns 165 cm. The longitudinal reinforcement was 6 bars 10 mm diameter of grade 60 located at the corners and the middle of the long side, providing a steel content of 1% of the cross section. All columns contained 9 ties 6 mm diameters grade 36 uniformly distributed along the length of the columns, as shown in figure (1). The specimen ended with two head 15x60x 22.5 cm as shown in figure (1). The test program was divided into 3 groups of specimens. The first group C1,C2 was 2 specimens used as control columns. The second group C3,C4,C5 consisted of 3 specimens strengthened by four corner angles 30x30x3, 40x40x4, and 60x60x6 respectively and strap plates 4x0.6 cm at intervals 15 cm. The third group C6,C7 as well as C4 was 3 specimens strengthened by four corner angles 40x40x4 and strap plates 4x0.6 cm at intervals 10 cm and 20 cm respectively.

Table 1. Details of test program and the results

Group	No.	Corner Angles	Strap Plates	fcu (Kg/cm2)	Po (ton)	Pmax (ton)
Group 1	C1	Control		198.8	78.78	73.12
	C2	Control		193.7	77.24	72.91
Group 2	C3	30x30x3	40x6 @ 150 with middle anchor D12	201.1	79.47	99.56
	C4	40x40x4	40x6 @ 150 with middle anchor D12	198.8	78.78	108.67
	C5	60x60x6	40x6 @ 150 with middle anchor D12	193.7	77.24	113.25
Group 3	C6	40x40x4	40x6 @ 100 with middle anchor D12	201.1	79.47	128.11
	C7	40x40x4	40x60 @ 200 with middle anchor D12	193.7	77.24	92.21
Group 4	C8	40x40x4	40x6 @ 150 mm without middle anchor	198.8	78.78	91.32

Po = 0.67 fcu . Ac + As . fy

fcu : The compressive cube strength at the time of test
Pmax : The ultimate load test of the column.

All the previous columns were strengthened using anchors 12 mm diameter in the middle long side of column at each strap plates for purpose of fixation. The fourth group contained one column strengthened by four corner angles 40x40x4 and strap plates 4x0.6 cm spaced 15 cm without anchors. The details of the test specimens are showed in Table (1). The reinforcement of the cage consisted of two parts, the first for the column and the second for the head caps as shown in figure (1). The column longitudinal bars extended through the caps. The ties were placed at 6 cm at the caps. Also, an additional steel cap as shown in figure (2) was used to provide extra confinement at the two ends of columns and to prevent failure at the ends. After erecting the steel caps, leveling grout was placed between the steel caps and the specimen to provide a uniform load distribution over the column cross section.

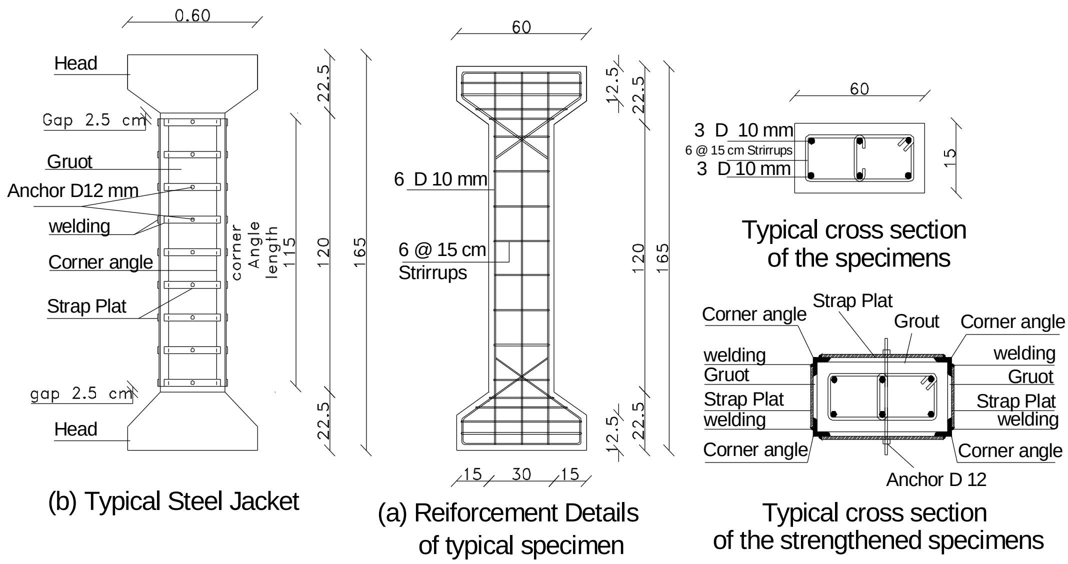

Figure (1) Details of the typical specimens.

Fig.2: Typical strengthened concrete column and details of the steel caps.

THE CONCRETE

The target compressive strength for the concrete was 200 kg/cm2 in all the specimens. 12 standard cubs were cast, 3 in each batch, and were tested parallel to the columns tests. Table (1) indicates the cube strength of each specimen.

TEST SET UP AND INSTRUMENTATION

All specimens were tested vertically under 500 tons axially loaded machine. At the upper end of the specimen, a hinge was attached that allows the specimen to rotate freely about the axis of the shaft. This mechanism allows a line of action of the axial load remaining unchanged during the test. The longitudinal and lateral deformations of the columns were measured by LVDTs installed at the critical region of the columns. Longitudinal strain was measured by LVDT at long and short sides of the columns. The lateral strain was measured by one LVDT at the long side at the upper third of the column. The strain of the reinforced steel was recorded with strain gauges on the longitudinal and tie steel at the upper third region of the column. Figure (3) shows details of the setup of all test specimens and the location of the LVDTs.

Fig.3: Typical set up of test specimen

TEST RESULTS

The specimens were adjusted and loaded gradually in small increments at stages of a deformation rate 1 mm per sec. Behavior of each specimen is presented graphically in the form of axially vertical load versus the longitudinal deformation readings measured by LVDTs, and by using a data acquisition program to read the measurements. The relationship between the vertical load versus the longitudinal concrete strain of the columns after the strengthening until the failure of columns is plotted, as shown in figures (4,5,6).

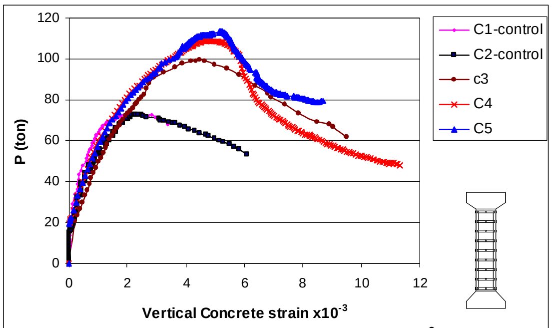

Figure (4) Load - Vertical concrete strain $x10^{-3}$ relationship for column group 2 (C3,C4,C5)

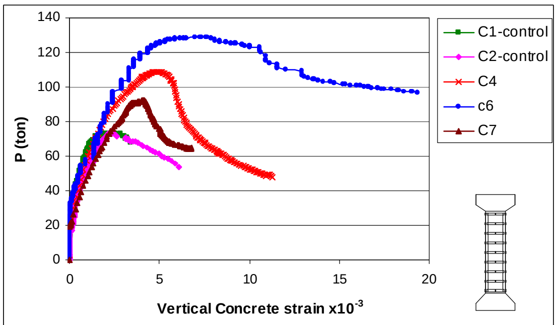

Figure (5) Load - Vertical concrete strain $x10^{-3}$ relationship for column group 3 (C4,C6,C7)

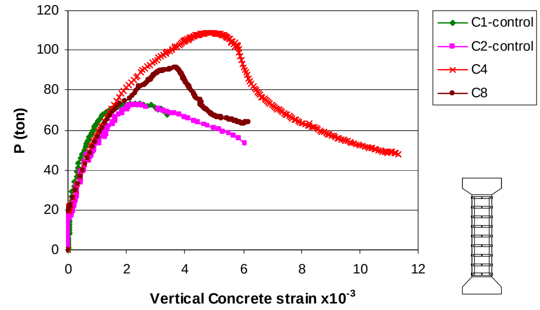

Figure (6) Load - Vertical concrete strain x10^{-3} relationship for column group 4 (C4,C8)

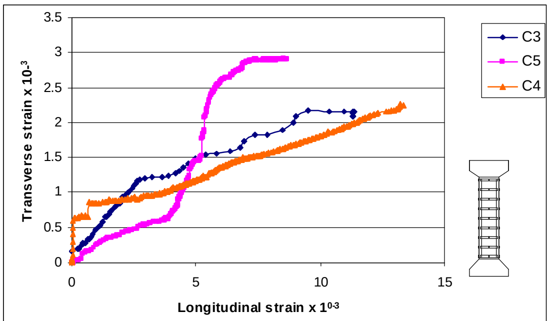

Figure (7) The relationship between the longitudinal and transversal strain for columns group 2

GENERAL OBSERVATIONS

The observation of the results indicates that:

- The relationships between the load and the longitudinal strain for all specimens are linear until 70% of the maximum load of the control columns, followed by a nonlinear behavior until failure.
- The slopes of the linear parts for all curves are the same for all the specimens with and without steel jackets.
- The failure of the strengthened columns occurred at 0.6 to 0.4 of the ultimate load after twice the longitudinal strain of the concrete. The load decreased rapidly after the ultimate load and the descending part of all curves do not represent the concrete stress – strain relationship.
- The relationship between longitudinal and lateral strains for the confined concrete columns is not constant and depends on the lateral strap plats as shown in figure (7).
- The longitudinal steel bars do not reach the yield stain before failure of the rectangular un-strengthened column, while it reaches the yield strain before failure for strengthened columns. This can be attributed to the effect of the confinement provided by the steel jacket.
- When increasing the area covered by the steel jackets this increases the ultimate load of the strengthened columns.

Fig.9: Failure Mode shape for strengthened columns.

Fig.8: Failure mode shape For un-strengthened columns.

FAILURE MODE

The failure mode of the original rectangular columns is brittle where the failure mode is inclined cracks as shown in figure (8). The strengthening of columns changes the failure mode, to be the crushing of the concrete at the upper third between the upper two strap plates. Local buckling of corner angles and longitudinal of reinforcement bars between the two upper strap plates were observed. Also, the longitudinal reinforcement of the columns buckled out of the column as shown in figure (9).

ANALYSIS OF THE RESULTS

Effect of Corner Angles

The effect of corner angles is shown in table (3) and figure (4). The study of this figure indicates that the use of corner angles 30x30x3, 40x40x4, 60x60x6 in the tested columns increase the ultimate load of concrete columns by 25 , 38 and 47 % respectively with strap plates 4x0.6 spaced 15 cm. These corner angles cover 26.67, 35.55 and 53.33 % of the surface area of the original concrete columns, while the strap plates cover 26.67%. The measured strength Δf increased by 41.5, 75 and 94.5 % where Δf = fcc -fco, fcc = Pmax/Acore, Acore is the area bounded by the centerline of the perimeter tie, and fco = 0.7 fcu. While the measured strength Ks increased by 100, 126 and 143%, where Ks= fcc/fco. The measured ductility μ85d were 1.52, 1.23, and 1.19 for using corner angles 30x30x3, 40x40x4 and 60x60x6 respectively, while for original column the measured ductility was 1.24. Where $\mu 85d = \varepsilon_{cc}/\varepsilon_{85d}$. Hence, This indicate that the use of longer leg angles increases the ultimate load of the concrete columns but the ductility decreases due to the increasing of the concrete confinement.

Table 2. The Results of Control Columns and the confinement measures Dfc and Ks

Group	No.	fcu (Kg/cm2)	Po (ton)	Pmax (ton)	Pmax/ Po	Pcmax	fco	fcc	Dfc	Ks
Group 1	C1	198.8	78.78	73.12	0.93	54.28	139.16	189.79	50.63	1.36
	C2	193.7	77.24	72.91	0.94	54.07	135.59	189.06	53.47	1.39

Table 3. The results of Strengthened Columns with Variable Size of Corner Angles With respect to the confinement measures Dfc and Ks

Group	No.	fcu (Kg/cm2)	Po (ton)	Pmax (ton)	Pmax/ Po	Pcmax	fco	fcc	Dfc	Ks
Group 2	C3	201.1	79.47	99.56	1.25	80.72	140.77	282.24	141.47	2.00
	C4	198.8	78.78	108.67	1.38	89.83	139.16	314.09	174.93	2.26
	C5	193.7	77.24	113.25	1.47	94.41	135.59	330.10	194.51	2.43

Table 4. The results of Strengthened Columns with variable Spacing of Strap Plates. With respect to the confinement measures Dfc and Ks

Group	No.	fcu (Kg/cm2)	Po (ton)	Pmax (ton)	Pmax/ Po	Pcmax	fco	fcc	Dfc	Ks
Group 3	C6	201.1	79.47	128.11	1.61	109.27	140.77	382.06	241.29	2.71
	C4	198.8	78.78	108.67	1.38	89.83	139.16	314.09	174.93	2.26
	C7	193.7	77.24	92.21	1.19	73.37	135.59	256.54	120.95	1.89

Table 5. The Results of Strengthened Columns with / without Middle Anchor Bolts of Strap Plates

With respect to the confinement measures Dfc and Ks

Group	No.	fcu (Kg/cm2)	Po (ton)	Pmax (ton)	Pmax/ Po	Pcmax	fco	fcc	Dfc	Ks
Group 4	C4	198.8	78.78	108.67	1.38	89.83	139.16	314.09	174.93	2.26
	C8	198.8	78.78	91.32	1.16	72.48	139.16	253.43	114.27	1.82

Po = 0.67 Fcu . Ac + As . Fy
Pcmax = Pmax - As t . Fy
Fco = 0.7 fcu

fcc = Pcmax / Acore
Δfc = fcc - fco
Ks = fcc / fco

Effect of strap plates spacing

The effect of strap plates spacing is shown in table 4 and figure (5). The figure indicates that the use of strap plates 4x0.6 cm at intervals 10, 15 and 20 cm increase the columns ultimate load 61, 38 and 19% respectively. The covered areas with respect to the original column with the steel jacket were 40, 26.67, 20%. The measured ductility μ85d using strap plates at intervals 10, 15 and 20 cm were 1.77, 1.23, and 1.19 respectively while the measured ductility for the original column without strengthening was 1.24. Hence, this indicates that by decreasing the spacing between strap plates increases the ultimate load and the ductility of the columns of low strength of concrete.

The effect of anchor

The effect of did not using the anchor in the middle of the strap plates of long side of the column is shown in table 5 and figure (6). The observation of the figure indicates that the use of strap plat, in case of columns of aspect ratio 2, without middle side increase the ultimate load by 19% only while the using of this middle anchor increase the ultimate load increase by 23% as in C4. The ductility measure of the strengthened decreased to 1.18 instead of 1.24 for C4. Then, the using of the anchor at spacing not beggar than 20 cm as a code of spacing of stirrups is essential in case of steel jacketing.

ANALYTICAL ANALYSIS

The analytical model which was suggested by Mander [5,6], and modified by Wang[7] was used to make the analytical analysis of the tested strengthened columns. The analytical model depends on the confinement of the concrete by both the jacket and the steel stirrups. The jacket represents an external lateral pressure and the steel hoops represent an internal lateral pressure on the concrete columns. The analytical method presented a closed form equations for the arching action of the confined concrete to evaluate the axial load - deformation behavior of rectangular concrete columns confined with jacket and it expressed as follows:

$$fc = \frac{fcc.x.r}{r-1+x^r} \qquad (1)$$

where,

$$x = (\varepsilon_c / \varepsilon_{cc}) \ , \ e_{cc} = e_{co}(1 + R(f_{cc}/f_{co}-1)) \qquad (2)$$

and,

$$r = \frac{Ec}{Ec - E\sec} \qquad (3)$$

Where ,

ε_{cc} : is the confined concrete stain (taken equal to 0.002)

ε_{co} : is the unconfined concrete stain (taken equal to 0.002), f_{cc} : is the compressive strength of the confined concrete and is calculated by Wang method [7}, f_{co} : is the compressive strength of the un-confined concrete, R : is a constant taken 5 in Mander's model, Ec = 5000√fco (Mpa), and E sec = f_{cc} / e_{cc} .

The analytical results were plotted and compared with the experimental obtained measurements as shown in figures (10,11) as examples for columns C4 and C6. Also, the predicted ultimate load using the analytical method and the experimental results are shown in table (6). Comparing the results a good correlation is observed for all the studied cases except that of the corner angles 60x60x6. this can be attributed to the fact that the angles were sufficient to participate a lot for both the vertical load and the lateral pressure. A modification of the Mander model [5,6] is suggested in order to adopt the theoretical equations and to minimize the difference with respect to the experimental results. As, the factor R in equation (2) depends on the characteristic properties of the unconfined and the confined concrete. The author suggests the following factor to adopt the analytical results with the experimental results:

$$x = (\varepsilon_c / \varepsilon_{cc})^k \quad , \quad \text{and} \quad R = 2 \tag{4}$$

Where,

$$k = 1 \quad \text{for } \varepsilon_c < \varepsilon_{cc} \quad \text{and} \quad k=3 \quad \text{for } \varepsilon_c > \varepsilon_{cc} \tag{5}$$

Table 6. Ductility Factor m85d

Group	No.	**Pmax (Exper)** (ton)	**Pmax (Analy)** (ton)	P_{Exp}/P_{analy}	e_{85d} (Exp.) x10-3	e_{cc} (Exp.) x10-3	m_{85d} (Exp.)
Group 1	C1	73.12	79.54	0.92	3.38	2.44	1.39
	C2	72.91	78.02	0.93	3.48	2.32	1.50
Group 2	C3	99.56	105.83	0.94	6.80	4.48	1.52
	C4	108.67	106.75	1.02	6.01	4.84	1.24
	C5	113.25	107.86	1.05	6.12	5.19	1.18
Group 3	C6	128.11	131.36	0.98	12.95	7.01	1.85
	C4	108.67	106.75	1.02	6.01	4.84	1.24
	C7	92.21	91.30	1.01	4.81	4.09	1.18
Group 4	C4	108.67	106.75	1.02	6.01	4.84	1.24
	C8	91.32	90.18	1.01	4.34	3.68	1.18

$\mu_{85d} = \varepsilon_{85d} / \varepsilon_{cc}$

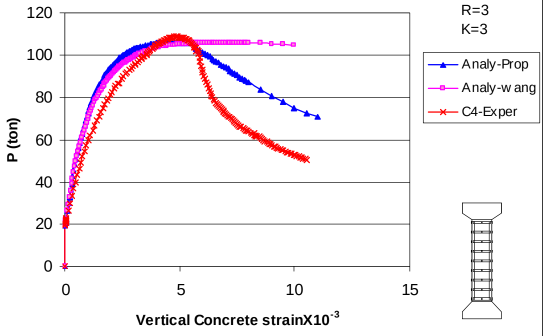

Figure (10) Load - Vertical concrete strain x10^{-3} relationship for column C4

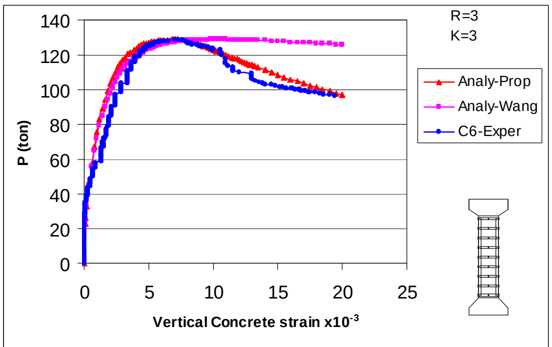

Figure (11) Load - Vertical concrete strain x10^{-3} relationship for column C6

CONCLUSION

From the previous results, it can be conclude that:

- The failure mode of the un-strengthened rectangular columns is brittle. The strengthening of column changes the failure mode.
- The use of the steel jacket can be successfully used for increasing the ultimate load of the concrete columns by 27.3, 32.1 and 74.8 % by using steel angle corners 30x30x4, 40x40,4 and 60x60x6 mm and strap plates 4x0.6 cm at intervals 15 cm. The steel jacket covers 46.22, 52.74 and 65.78 % of the surface area of the original column respectively.
- The decrease of the spacing between the steel strap plates improves the behavior of the strengthened columns and increases the ultimate load by 37.0 , 32.1 and 13.2 % by using steel corner angles 40x40,4 mm and strap plates 4x0.6 cm at intervals 10, 15 and 20 cm respectively. The steel jacket covers 61.33, 52.74 and 48.44 % of the surface area of the original column.
- The measured ductility was 1.52 by using corner angles 30x30x3 mm but it decreased to 1.24 for corner angles 40x40x4 mm and 1.18 for corner angles 60x60x6 mm, while this measure equals to 1.45 for original column without strengthening. The measured ductility increased to 1.85 by using strap plates at intervals 10 cm, but it decreased to 1.24 by using strap plates at intervals 15 cm and 1.18 by using strap plates at intervals 20 cm for the same corner angles 40x40x4 mm. Then, it should be mentioned that in same case of strengthening the ductility of the column may be decreased.
- The strengthening of columns by steel jacket without using anchors in the middle of the long side of the columns improved the ultimate load of the strengthened column by 16%, but it decreased the ductility of the strengthened column to 1.18 instead of 1.45 for the original column.
- In case of low compressive strength of concrete the steel jacket increased the ultimate load of the column while the corresponding ductility may increase or decrease for low compressive concrete strength. The steel jacket should be made with enough size corner angles, reasonable spacing between the strap plates and adequate anchors to fix the strap plates in the long side of the rectangular columns.
- The increased ultimate load capacity of the strengthened columns should be related to the ratio between the covered area of the jacket and the surface area of the concrete columns. The steel corner angles and the strap plates being welded together, the thicknesses should allow the process of welding. The size and the weld length should be proportional based on the lateral pressure using the analytical process.
- The analytical model proposed by Mander [5,6], showed a good agreement for the determination of the ultimate load of the strengthened columns and the evaluating of the stress – strain curves, if the values of R and K are well adopted. Therefore, the author recommends making serious experimental works to determine the factors R and K for different types of concrete and different types of the strengthening techniques.

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ASSESSMENT OF PUNCHING LOAD OF HSC : A FUZZY LOGIC APPROACH

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ABSTRACT

Fuzzy logic approach is applied to the problem of High Strength Concrete(HSC) punching failure. A comprehensive survey of the published literature concerning punching shear strength of high-strength slabs is carried out, to obtain an experimental data base of various tests and parameters affecting punching of High Strength Concrete. Collecting the experimental data base, the fuzzy logic is applied to obtain the punching capacity.
The results are compared to Egyptian Code (ECC-2001) and BS8110 code methods for the prediction of punching load.
The comparison showed that the used fuzzy logic can stand alone as a competitive method for solving the punching problem of HSC.

KEYWORDS
Punching shear; high-strength concrete; design Code; Fuzzy Logic

INTRODUCTION

The use of high strength concrete has steadily increased over the past years regardless of its brittleness. The great development of concrete technology allows the production of high strengths, with its advantages of producing smaller sections, reducing the dead weights, allowing longer spans and more durable structures.
Punching is a sudden brittle failure , thus special consideration should be given to HSC punching failure.
Several theories attempt to analyze the punching failure, however lack of simple theory to describe the punching failure is due to the complexities of the basic 3-D behavior and the uncertain shear transfer that exists before failure
All design codes of concrete structures comprise methods based on empirical formulae to account for the different parameters affecting the punching load, however, experimental results indicate that an interactive effect may exist between the different parameters .
Fuzzy logic, in a broad sense, refers to all of the theories that employ fuzzy sets, which are classes with unsharp boundaries. In simple words, a conventional black-and-white concept is generalized to a matter of degree. In such a way, two goals are accomplished: (1) ease of describing human knowledge involving vague concepts, and (2) enhanced ability to establish a cost-effective solution to real-world problems. Traditionally, fuzzy logic has been viewed as a theory for handling uncertainty about complex systems and an approach for approximation theory (Yen and Langari 1999; Ross 1995). Fuzzy logic is being applied in a wide variety of fields; economics, psychology, natural sciences, and engineering. The primary reason for the successful application of fuzzy logic in industry and engineering fields, to date, is its ability to provide practical and low cost solutions.

Fuzzy logic concept has been successfully utilized in many structural engineering applications. Juang et al. (1991) applied the concept to estimate the ultimate capacity of single piles driven in sand formations. The fuzzy logic concept was also adopted by Samieh (2003) to predict the friction capacity of driven piles in clay formations. Juang et al. (1992) reported a low-cost, qualitative evaluation scheme using the fuzzy logic concept for mapping slope failure potential. In addition, the fuzzy logic concept has been extended to investigate the shear strength of soils (Chung 1995), liquefaction potential (Elton et al. 1995; Chen and Chen 1997), relative density of sand formations from cone penetration test data (Juang et al. 1996), soil identification using piezocone data (Pradhan 1998).

This study investigates the possibility of establishing a fuzzy logic system that shall be capable of predicting the punching load of High Strength Concrete slabs using a database of experimental slabs tested with different dimensions, and reinforcement ratios. In addition, the study compares the prediction of the established fuzzy logic system with the prediction of some empirical methods currently in use for estimating the punching load. Also to check the adequacy of the established fuzzy logic system and to answer the question which of the fuzzy logic system or empirical method would be more reliable to predict the slab punching load.

THE PROPOSED FUZZY LOGIC SYSTEM

Estimation of the slab punching load is expected to depend on many factors. The major factors are the slab dimensions, concrete characteristic strength, reinforcement ratio, and steel type.

In the current study, the slab dimensions, thickness, characteristic strength of the concrete, yield stress of the steel and the reinforcement ratio are adopted parameters to establish the fuzzy logic system.

The influence of each of these parameters on the slab punching load is difficult to measure/quantify by testing . However, these parameters are expected to have different impacts on the final punching load value. In other words, there is a hidden/undefined relationship between these parameters and the punching load value. This undefined and vague nature of the problem gives rise to the potential success of artificial intelligence systems to solve the current problem. This is refered to the flexibility of the fuzzy logic systems and their ability to account for various uncertainties and hidden nonlinear relationships between variables.

A database compiled in the current study . This was obtained from a comprehensive survey of the published literature concerning punching shear strength of high-strength concrete slabs. The database of the current study consists of 40 slab loading records. The records are chosen to cover a wide range of the different parameters such as , the slab dimensions, thickness, column dimensions, column shape, characteristic strength of the concrete, yield stress of the steel and the reinforcement ratio . It should be mentioned that only specimens which failed by punching are included from the pervious research tests. Table 1 presents a summary of the slab punching load database parameters.

Table 1: Test Details

Notation	Specimen shape	Width Or Diameter	Fcu (MPa)	Col. shape	Column Side length (mm)	.Slab thickness (mm)	Reinforcement ratio %	FY	Punch Load (KN)
Marzouk and Hussien (1991)									
HS1	Square	1500	78.8	Sq.	150	120	0.491	490	178
HS2	Square	1500	82.6	Sq.	150	120	0.84	490	249
HS3	Square	1500	81.3	Sq.	150	120	1.47	490	356
HS4	Square	1500	77.4	Sq.	150	120	2.37	490	418

HS6	Square	1500	82.3	Sq.	150	150	0.944	490	489
HS7	Square	1500	86.8	Sq.	150	120	1.19	490	356
HS8	Square	1500	81.1	Sq.	150	150	1.11	490	436
HS9	Square	1500	87	Sq.	150	150	1.611	490	543
HS10	Square	1500	94.1	Sq.	150	150	2.33	490	645
HS11	Square	1500	82.3	Sq.	150	90	0.952	490	196
HS12	Square	1500	88.2	Sq.	150	90	1.52	490	258
HS13	Square	1500	80	Sq.	150	90	2	490	267
HS14	Square	1500	84.7	Sq.	220	120	1.473	490	498
HS15	Square	1500	83.5	Sq.	300	120	1.473	490	560
Morshed (2003)									
HS	Square	1600	78.2	Sq.	200	120	1.7	400	501
Melo(2001) B-1	Square	1800	71.6	Sq.	120	130	1.5	695	270
B-2	Square	1800	70.82	Sq.	120	130	1.4	695	335
Abd- Elrazek (1995)									
H.h.z.s. 1.0	Square.	2200	67.16	Sq.		150	1	480	511
Ramdane (1996)									
slab2	Circle.	1372	66.07	Circle.	150	125	0.58	550	212
slab12	Circle	1372	71.06	Circle.	150	125	1.28	550	319
slab14	Cirle	1372	71.53	Circle.	150	125	1.28	550	314
slab16	Circle	1372	116.71	Circle.	150	125	1.28	550	362
slab22	Circle	1372	99.06	Circle	150	125	1.28	650	405
slab5	Circle	1372	66.35	Circle	150	125	1.28	650	341
Hallgern and Kinnunen(1996)									
HSC0	Circ	2400	106.24	Circ.	250	240	0.8	600	965
HSC1	Circ.	2400	99.76	Circ.	250	240	0.8	600	1021
HSC2	Circ.	2400	100.35	Circ.	250	240	0.82	600	889
HSC4	Circ.	2400	107.76	Circ.	250	240	1.19	600	1041
HSC6	Circ.	2400	128.00	Circ.	250	240	0.6	600	869
HSC8	Circ.	2400	94.12	Circ..	250	240	0.8	600	944
Tomaszewicz(1993)									
ND65-1-1	Square	2500	75.29	Square	200	320	1.49	500	2050
ND65-2-1	Square	2200	81.88	Square	150	240	1.75	500	1200

ND95-1-1	Square	2500	98.82	Square	200	320	1.49	500	2250
ND95-1-3	Square	2500	105.41	Square	200	320	2.55	500	2400
ND95-2-1	Square	2290	103.53	Square	150	240	1.75	500	1100
ND95-2-3	Square	2200	105.41	Square	150	240	2.62	500	1450
ND95-3-1	Square	1100	100.05	Square	100	120	1.84	500	330
ND115-1-1	Square	2500	131.76	Square	200	320	1.49	500	2450
ND115-2-1	Square	2200	140.05	Square	150	240	1.75	500	1400
ND115-2-3	Square	2200	127.06	Square	150	240	2.62	500	1550

In the current study, the slab dimensions, thickness, reinforcement ratio, and the yield strength, are used in establishing the fuzzy logic system. These parameters are conventionally available/assessed. They can be easily determined and are expected to impose a direct impact on the punching load capacity.

THE FUZZY LOGIC SYSTEM: MODELING SCENARIO AND THE MODEL RESULTS

The fuzzy logic system established in this study utilized the adaptive learning technique. This technique provides a method for the fuzzy system to learn the characteristics of a given data set. Finding the characteristics of the data set leads the system to determine the parameters of the membership functions that allow the fuzzy inference system to capture the variations and the unseen properties of the given input/output data, as well as the training data. In brief, using a given input and output data set, a fuzzy inference system is constructed with membership function parameters adjusted using a back propagation approach in combination with a least squares technique to track the characteristics of the problem that is being modeled. In such a way, the fuzzy system learns from the data that is being modeled (MathWorks, 2000).

The software Matlab (MathWorks, 2000) has been used in the current study to establish the fuzzy logic system. To train the fuzzy system, part of the friction capacity database was used in the training process, around 75% of the database. The rest of the database, around 25%, which is referred to as testing/verifying data in the following discussion, was used to see how well the fuzzy system predicts the output of data not used in training the model. Trapezoidal membership functions were utilized in the established fuzzy model for the input data. Weighted average defuzzification algorithm with constant output membership functions was utilized in the current model to assess the single output value of the system, the punching load.

Figure 1 shows a comparison between the measured punching load and the fuzzy system prediction for both the training and testing data. A good comparison is exhibited between the measured and predicted punching loads. The degrees of correlation of the predicted capacity for the training and testing data of the database shown in Fig. 2 are 0.997 and 0.981, respectively, with average values of Pprediction / Ptest of 1.0 and 0.93 and standard deviations of 0.092 and 0.122 for the training and testing data of the database, respectively

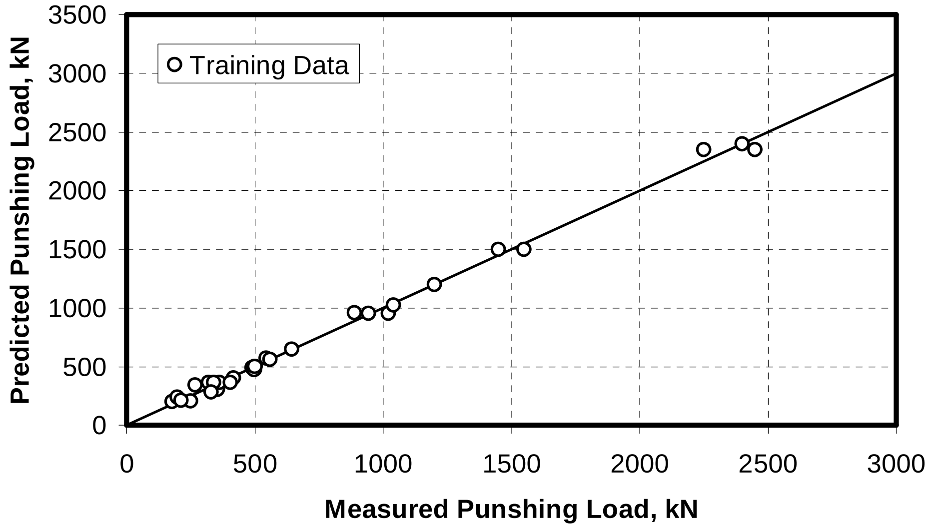

Fig. 1-a Comparison for Training Data

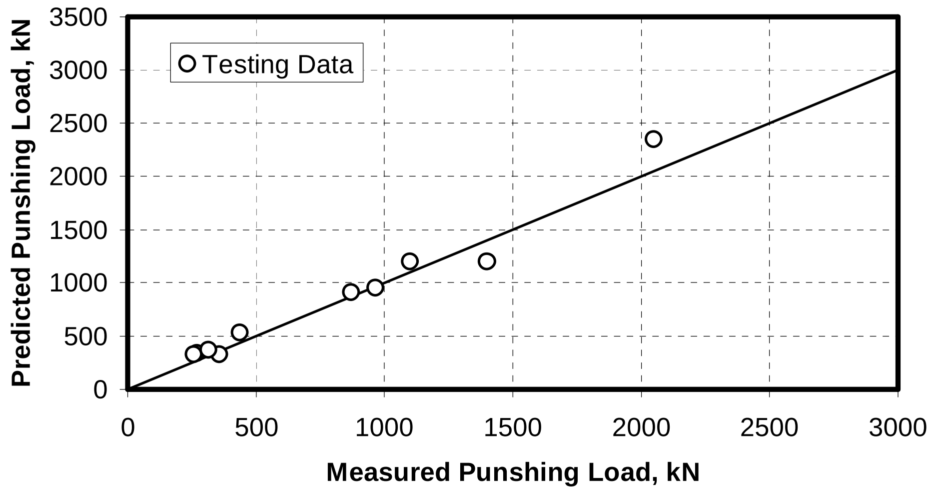

Fig. 1-b Comparison for Testing Data

Fig. 1 Comparison of Predicted (Fuzzy Logic) and Measured Punching Loads

PREDICTION OF SOME EMPIRICAL METHODS

(1) ECCS (EGYPTIAN CODE -2001)

The current code formula follows the critical perimeter method where the punching loads are calculated by multiplying a failure surface by critical nominal shear stress.

Critical section for checking punching load is assumed to be at (d/2) from the perimeter of the loaded area

Figure. (2) shows the the experimental ultimate loads (Ptest) of the slabs to the predicted (Pcode) values using ECCS .In the calculations , the limits with respect to the concrete strength have been ignored, and no allowance is made for material factor of safety regarding concrete strength ($_{c}$ =1).

The degree of correlation of the predicted capacity and the experimental data of the database shown in Fig. 3 are 0.975 , with average value of Pprediction / Ptest of 0.812 and standard deviation of 0.18 .

(2) BS8110

The BS8110 takes the effect of reinforcement ratio on the punching load , and the critical perimeter is assumed to be located at (1.5d) from the perimeter of the loaded area .

Figure (3) compares the experimental ultimate loads (Ptest) of the slabs to the predicted (Pcode) values using BS8110 regulations .In the calculation , the limits with respect to the concrete strength have been ignored, and no allowance for material factor of safety regarding concrete strength ($_{c}$ =1).

The degree of correlation of the predicted capacity for the predicted and experimental data of the database shown in Fig. 3 is 0.99 ,with average value of Pprediction / Ptest of 0.88 and standard deviation of 0.167.

However, it can be seen that punching shear strength values predicted by BS8110 are more consistent with the experimental data than does the ECCS 2001.

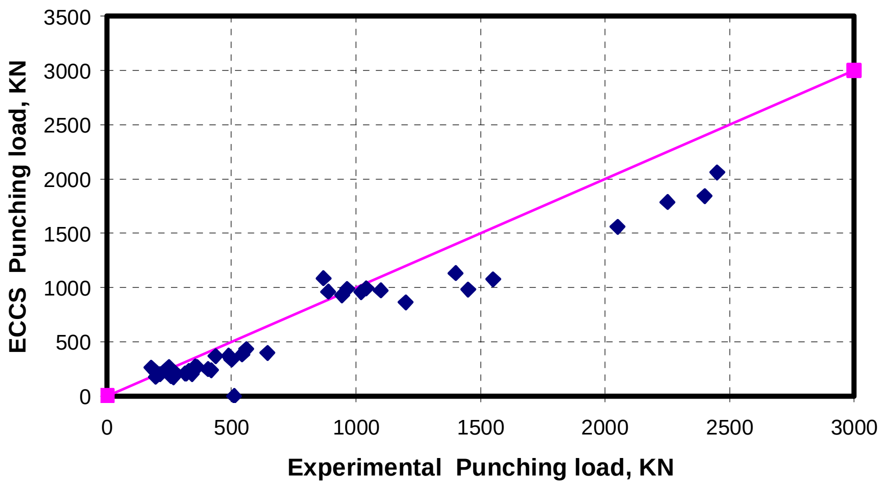

Fig. 2 Comparison of Predicted (Egyptian Code) and Measured Punching Load

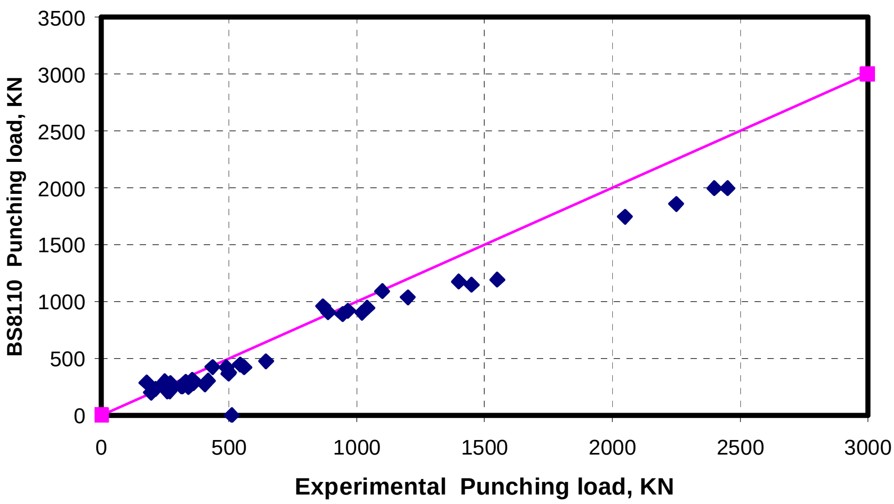

Fig. 3 Comparison of Predicted (BS8110) and Measured Punching Load

SUMMARY AND CONCLUSIONS

The principal target of this study is to demonstrate the ability of fuzzy logic to give a good estimate of the punching load of High Strength Concrete slabs. An adaptive fuzzy logic system was established and verified as a stand-alone system for the prediction of the punching load. The degree of correlation between fuzzy logic predictions and these from empirical formulae may consider that fuzzy logic approach can be viewed as a competitive method.

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SEISMIC ANALYSIS APPROACH FOR WELL - DESIGNED EQUAL RESISTANCE STRUCTURE

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ABSTRACT

The current Egyptian Code of Practice for Load Calculations, ECPLC, requires that the maximum dynamic base shear is equal to the values obtained from Multi Modal Response Spectrum method, MMRS. Further, the dynamic methods of analysis must be used in the principal directions of the structure. These requirements represent a new challenge to most structural engineers. The present code does not state how to define the principal directions for a three dimensional structure of arbitrary geometric shape. In addition, the design base shear can be different in each direction of the building, due to the difference in its structural systems. It can produce a different input motion for each direction, for both regular and irregular structures. Consequently, it can result in a structural design which is relatively weak in one direction. Therefore, the main objective of this research is to provide a dynamic analysis approach for a structural design that has equal resistance in all directions. This method is based on design response spectra defined in the code. A separate response spectrum analysis was conducted in each direction of the building, and special treatments of orthogonal loading effects were used in combining modal maxima.

Key words: Response spectra, Dynamic analysis, Equal resistance structure, Seismic analysis, Orthogonal loading, Structure design.

INTRODUCTION

In the new Egyptian Code of Practice for Load Calculations - ECPLC, Ref. (1), the basic magnitude of the seismic loads has changed significantly from previous code, Ref. (2). The major change is that the Multi Modal Response Spectrum method, MMRS, is the reference in calculating the design base shear. Therefore, the interpretation of the dynamic analysis requirement of the current code, ECPLC, represents a new challenge to most structural engineers. The structures are being analyzed in three dimensional. Despite the engineers have access to highly capable 3D analysis packages often including the ability to use dynamic analysis; they are faced with problems relating to the scaling of dynamic base shear and the treatment of the orthogonal loading effect in MMRS method.

The current Egyptian code follows the steps of the Euro code 8, Ref. (3). It should be noted that the MMRS method needs some significant requirements. First, a mathematical model of the physical structure shall represent the spatial distribution of the mass and stiffness of the structure to an extent that is adequate for the calculation of the significant features of its dynamic response. The number of considered modes shall satisfy the condition that at least 90 percent of the participating mass of the structure is included in the calculation for each principal horizontal direction. Last, it needs to special modal interaction effects when combining modal maxima to get the peak member forces, displacements and base reactions. In addition, there isn't a way to check the obtained results, if there is a mistake in the structural modeling and/or in modal combination. Therefore, The USA codes, Ref. (4, 5 & 6), stated that the maximum dynamic base shear is equal to the values obtained from Static Force Procedure, to avoid the mistakes that may happen of misusing the MMRS method.

In the following, a quick review for spectral analysis modal combination will be presented.

Spectral analysis modal combination

There are two types of modal combination. The first is valid for two dimensional structure analyses, i.e., modal maxima in each direction of the building. The latter is valid for three dimensional structure analyses, i.e., combination of two or three global dynamic motions.

One direction modal combination

The most conservative method that is used to estimate a peak value of displacement or force within a structure is to use the sum of the absolute of the modal response values, Ref. (7, 8 &12). This approach assumes that the maximum modal values, for all modes, occur at the same point in time, Eqn. (1).

$$U = \sum_{i=1}^{n} |u_i| \qquad (1)$$

Where:

U_i = i th modal generalized response.

Another very common approach is to use the Square Root of the Sum of the Squares, SRSS, on the maximum modal values in order to estimate the values of displacement or forces. The SRSS assumes that all of the maximum modal values are statistically independent, Eqn. (2).

$$U = \sqrt{\sum_{i=1}^{n} u_i^{\,2}} \qquad (2)$$

The relatively new method of modal combination is the Complete Quadratic Combination, CQC, method, Ref. (9 &10). The method is based on random vibration theory. It takes into account the effect of cross correlation between modal responses, Eqn. (3).

$$U = \sqrt{\sum_{i=1}^{n} \sum_{j=1}^{n} u_j \; u_i \; r_{ij}} \qquad (3)$$

Where

$$r_{ij} = \frac{8x^2 \left(1 + \frac{w_i}{w_j}\right)\left(\frac{w_i}{w_j}\right)^{3/2}}{\left(1 - \left(\frac{w_i}{w_j}\right)^2\right)^2 + 4x^2\left(1 + \frac{w_i}{w_j}\right)^2\left(\frac{w_i}{w_j}\right)}$$

$w_i = \text{The i}^{\text{th}} \text{ mode shape.}$

$x = \text{Critical damping ratio.}$

It should be noted that: 1) the cross correlation between modal responses is significant for modes with closely spaced frequencies. 2) the SRSS methods of modal combination can lead to gross errors in estimating the peak responses when the structure frequencies are closely spaced, due to neglecting this correlation.

GLOBAL DIRECTIONS MODAL COMBINATION

A well designed structure should be capable of equally resisting earthquake motions from all possible directions. In order to obtain the maximum stresses in a particular member, a large number of dynamic analyses at various angles of input should be done, Ref. (7 & 8). As shown in Figure (1), the basic input spectra S_1 and S_2 are applied at an arbitrary angle θ. The minor input spectrum is assumed a fraction of the major input spectrum, in order to simplify the analysis Eqn. (4).

$$S_2 = l\ S_1 \qquad (4)$$
$$l = \text{factor between } 0.0 \text{ to } 1.0$$

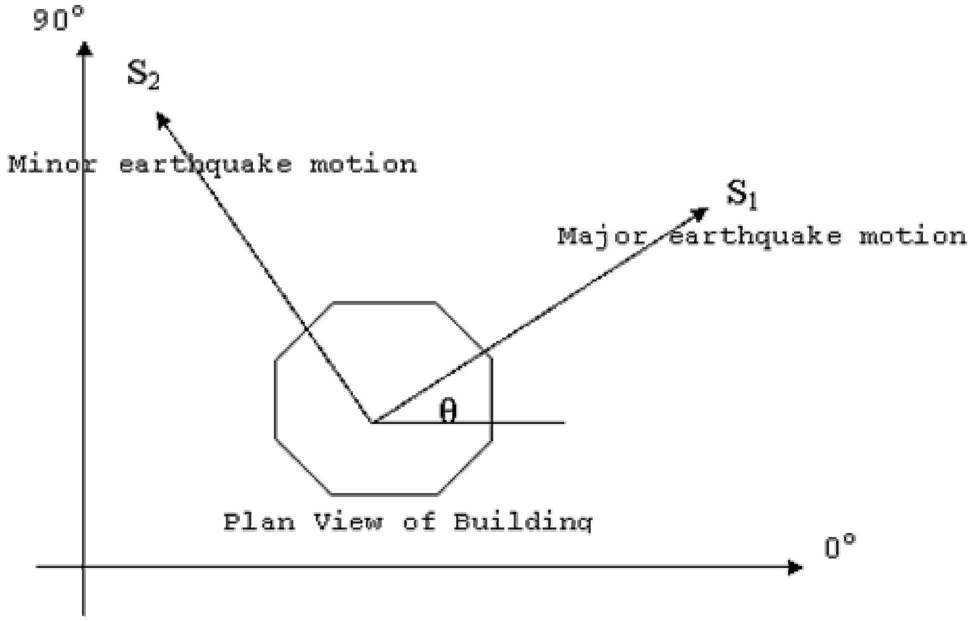


Fig.1: definition of earthquake Spectra input

Recently, the CQC3 method, Ref. (11), was conducted for the combination of the effects of orthogonal spectrum. The fundamental CQC3 equation for the estimation of a peak value is shown in Eqn. (5).

$$U = \begin{bmatrix} U_0^2 + l^2\, U_{90}^2 - (1 - l^2)(U_0^2 - U_{90}^2)\sin^2 q + \\ 2(1 - l^2)\, U_{0-90} \sin q \cos q + Uz^2 \end{bmatrix}^{\frac{1}{2}} \qquad (5)$$

Where

$$U_0^2 = \sum_n \sum_m u_{0n}\, u_{0m}\, r_{nm}$$

$$U_{90}^2 = \sum_n \sum_m u_{90n}\, u_{90m}\, r_{nm}$$

$$U_{0-90} = \sum_n \sum_m u_{0n}\, u_{90m}\, r_{nm}$$

$$Uz^2 = \sum_n \sum_m uz_n\, uz_m\, r_{nm}$$

The u_{0n} and u_{90n} are referred to the modal values which were produced by 100 percent of the lateral spectrum applied at 0.0 and 90 degrees respectively. The u_z is the modal response from the vertical spectrum.

It should be noted that the angle θ is not known. Therefore, in order to obtain the maximum response, Eqn (5) was differentiated with respect to variable θ and setting equal to zero. This procedure yielded the critical angle θ_{cr}., Eqn (6).

$$q_{cr} = \frac{1}{2}\tan^{-1}\left[\frac{2 \times U_{0\,-\,90}}{U_0^{\,2} \;\; - U_{90}^{\,2}}\right] \qquad (6)$$

Finally, it was concluded that, there is no guidelines for selecting the value of λ, to obtain the maximum response.

One option in existing design codes, Ref. (1, 3, 4, 5 & 6) for buildings and bridges requires that members be designed for 100 percent of the prescribed seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction. However, they give no indication on how the directions are to be determined for complex structures.

From all mentioned above, this paper is aimed to present an approach for well designed structure that should be capable of equally resisting earthquake motions from all possible directions. The main points of this study are:

1. A method to check the obtained dynamic base shear to avoid the mistakes in the structural modeling.
2. The accurate method that is used to estimate a peak value of displacement or force within a structure for each principal horizontal direction.
3. The accurate method that is used to estimate the peak responses within a structure for orthogonal directions in spectral analysis.

The above points will be discussed briefly through three examples. The design response spectra defined in Egyptian code, Ref. (1), will be used during this study. A separate response spectrum analysis will be conducted in each direction of the chosen buildings, and special treatments of orthogonal loading effects will be used in combining modal maxima.

DESIGN SPECTRA

The Egyptian code, Ref. (1), has defined specific equations for each range of the spectrum curve for four different soil types. The response spectrum type one curve, subsoil class "C", seismic zone 3, damping correction factor "1", and response modification factor "5" for moment resisting frame system, which is used during this study, is shown in Figure (2).

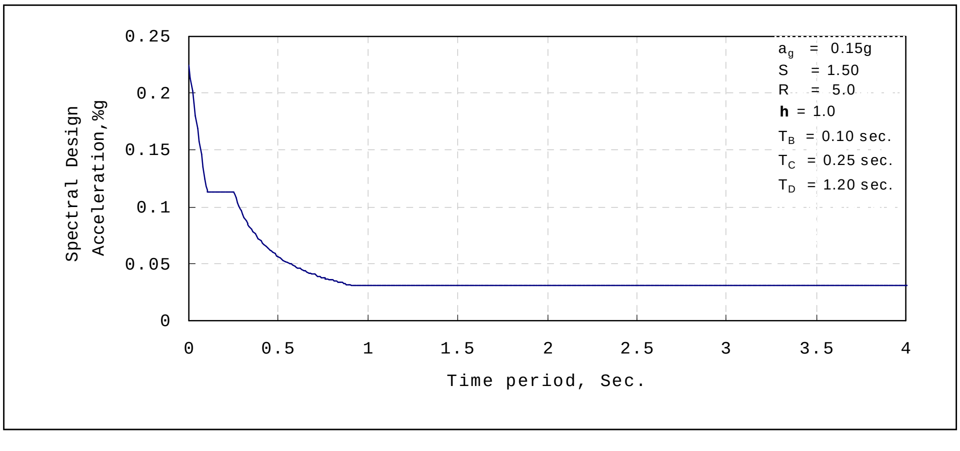


Fig.2: The Spectral design acceleration according to Egyptian code.

GROUND MOTION RECORDS

Three ground motion records are used in this study. Their peak accelerations are within the range of 0.21g to 0.348g. Characteristics of the chosen ground motion are presented in Table (1). The selected earthquake records are normalized to 0.15g. Their response spectrum acceleration curves were estimated for 5% damping ratio.

Table 1: Characteristics of earthquake ground motions

Earthquake record	Date and place of earthquake	Peak acceleration (%g)	Ground velocity (mm/sec)	Ground displacement (mm)	Duration time (sec.)
El-centro ELC-S 00 E	8/5/1940 USA	0.348	334.00	124.00	55.76
Hollywood HWD-N 90 E	9/2/1971 USA	0.211	211.00	147.00	81.00
Olympia Oly-S 86 W	13/4/1949 USA	0.28	170.94	92.96	91.06

APPLICATIONS

Three examples will be presented to explain briefly the principals for well designed structure. In example one, the problems associated with the unavoidable mistakes in getting the maximum dynamic base shear, which is based on the MMRS method will be discussed. Example two explains the problems associated with the use of the absolute sum and the SRSS of modal combination. The last one introduces the procedure of obtaining the members design forces that are not function of the chosen reference system, i.e., equally resisting seismic forces.

EXAMPLE 1:

The building is a three-story concrete building with flat slabs supported by columns, Figure (3). The columns are 0.50×0.50m. The floors and roof are 0.25 m thick flat slab. The building is symmetrical; however, the centre of mass, of all floors, is located 1.80m from the geometric centre of the building. The diaphragms are rigid in plane. The base shear is calculated for four cases of masses. Case one the masses of own weight, flooring, and walls is included. In case two, the mass of flooring is omitted and the mass of walls is omitted in case three. Both masses of floors and walls aren't included. The response spectrum analysis was conducted for the four cases using the response spectrum shown in Figure (2).

The obtained maximum shear forces for the four cases and the base shear calculated based on the static procedure is mentioned in Table (2).

Table 2: The maximum dynamic base shear according to MMRS method

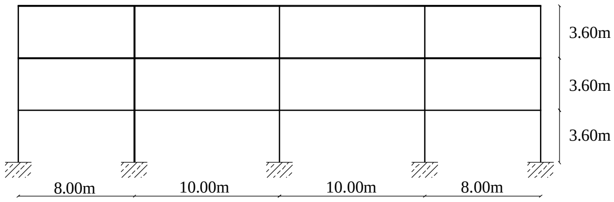
Base Shear (kN)	Base Shear (kN)	Base Shear (kN)	Base Shear (kN)	Base Shear (kN)
Case (1) $O.W^t$+ Flooring+Walls	Case (2) $O.W^t$.+ Walls	Case (3) $O.W^t$.+ Flooring	Case (4) $O.W^t$.	Static Force Procedure
1236.60	1073.11	967.94	804.42	2001

From Table (2), it is obvious that, the obtained values based on MMRS method hasn't any reference for the designer to check if the obtained value is correct or not. If the structural model is ill in simulating the masses of walls and/or flooring, the obtained base shear will be deviated on the correct value. Therefore, the maximum dynamic base shear based on a portion of the Static Force Procedure is essential to avoid the mistakes that may happen.

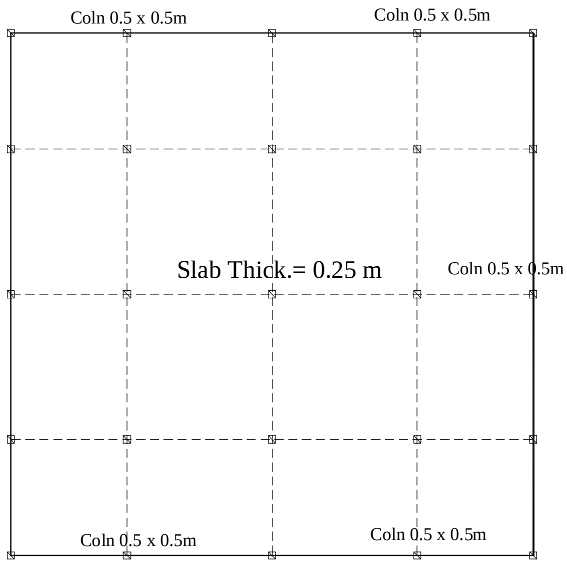
EXAMPLE 2:

The same building in example 1 is used. The building is solved three times. In each case, the building is subjected to one component of the selected earthquake (i.e., the dynamic forces applied in the X-direction only), as well as, to its response spectrum acceleration that was estimated for 5% damping ratio. In other meaning, the dynamic forces are applied in the X-direction only. In each case, the maximum modal base shears are obtained using different methods, i.e., Absolute Sum (ABS), SRSS, and CQC method. The obtained values are compared to the exact time history analysis.

The exact time history shear forces and the corresponding shear forces based on MMRS method using different methods of modal combination is illustrated in Table (3).



Elevation



Plan View

Fig.3: The Three Dimensional Building Layout, Examples 1& 2.

Table 3: Base Shears based on different Modal combination methods

	Base Shear (kN)		Base Shear (kN)		Base Shear (kN)		Base Shear (kN)	
Earthquake	Time History Analysis		Absolute Sum Method		SRSS Method		CQC Method	
	F_x	F_y	F_x	F_y	F_x	F_y	F_x	F_y
ELC	342.70	17.07	510.17	510.17	368.24	368.24	459.45	6.74
OLY	237.00	3.56	358.83	358.83	239.19	239.19	338.36	4.65
HWD	241.70	3.932	374.52	374.53	246.64	246.64	348.95	5.129

Table (30) shows that, the application of the CQC method allows the sum of the base shears in the direction of the external motion to be added directly. The obtained values by CQC method are matching with the time history analysis in both directions. The ABS and SRSS methods give equal shearing forces in the direction of motion and in the perpendicular direction too, despite the dynamic forces are applied in the X-direction only. The CQC method has the ability to eliminate the errors in the SRSS method in estimating the peak responses when the structure frequencies are closely spaced. In other meaning, the cross correlation between modal responses is significant for modes with closely spaced frequencies.

EXAMPLE 3:

The building is a very simple one-story concrete building with ordinary slab supported by columns, Figure (4). The column are pinned at base. The building is symmetrical. The diaphragm is rigid in plane. The response spectrum analysis was conducted using the response spectrum shown in Figure (2) in two perpendicular directions, i.e., 0.0^o and 90^o. A comparison between the results of the SRSS combination of two 100 percent seismic forces, and 100 percent of the seismic forces in one direction plus 30 percent of the prescribed forces is carried out. The moments at the top of columns 1,2,4 and 5 are summarized in Table (4).

Table 4: Moments at the top of symmetrical columns

Column N^o.	$M_{100/100}$	$M_{100/30}$	$M_{30/100}$	Error %
1	2.28	2.06	1.18	-9.6
2	4.40	3.89	2.45	-11.59
4	2.28	2.01	1.21	-11.84
5	4.40	3.82	2.49	-13.18

It should be high lighted that, members 1 and 4 and members 2 and 5 should be designed for the same moments due to symmetry. From Table (4) illustrated that the 100/30 combination rule produces moments which are not symmetric, whereas, the 100/100 combination rule produces logical and symmetric moments.

For this simple example, the 100/30 rule is failed in giving reasonable results. The SRSS method combination of two 100 percent spectra analyses, with respect to any user defined orthogonal axes, will produce design forces that are not a function of the reference system. In other meaning, the resulting structural design will has equal resistance to seismic motions from all directions.

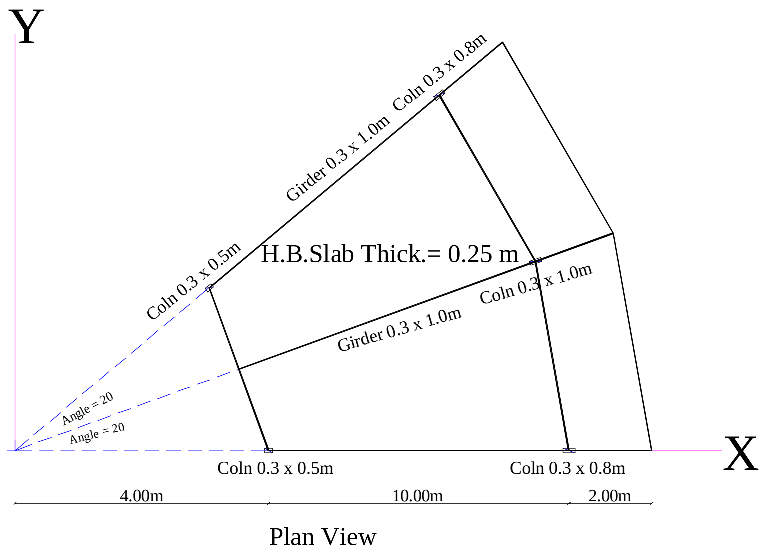


Fig.4: The Three Dimensional Building Layout, Example 3.

CONCLUSION

The well designed structure should have equally resisting seismic forces from all possible directions. From studying the lateral force requirements in Egyptian code, ECPLC, and in few foreign codes, the following issues could be concluded:

1. The maximum dynamic base shear is preferable to be equal to the values obtained from Static Force Procedure, to avoid the human mistakes that may happen of misusing the MMRS method.

2. The CQC method has the ability to eliminate the errors in the SRSS method in estimating the peak responses when the structure frequencies are closely spaced. i.e., the cross correlation between modal responses is significant for modes with closely spaced frequencies.

3. For three dimensional response spectra analysis, the design based on 100 percent of the seismic forces in one direction plus 30 percent of the prescribed forces in the perpendicular direction can:
   a) Underestimate the design forces in certain members.
   b) Produce a member design which is relatively weak in one direction.

4.The SRSS combination of two 100 percent spectra analyses, with respect to any user defined orthogonal axes, will produce design forces that are not function of the reference system.

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A NON-DESTRUCTIVE TEST FOR THE EVALUATION OF THE INTEGRITY OF PIERS DURING CONSTRUCION

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ABSTRACT

Concrete piers are vital structural members in most bridge systems. However, several problems accompany piers erection during construction. These problems vary from segregation of concrete to concrete caving and other problems resulting from piers frequent large sizes. This may consequently affect the safety of the whole bridge structure under actual service loads. As a result, clear needs for checking the integrity of piers exist during bridge construction. Despite that, the most dominant method for checking the integrity of structural members in the site is visual inspection though it may not be sufficient. This paper introduces a non-destructive technique for testing the integrity of piers after its construction and before loading. Global vibration-based system identification technique is used as a tool in the test. It is shown that by dividing the pier into vertical segment elements, the aforementioned non-destructive test gives stiffness parameter estimates for the corresponding elements. The results obtained indicate that on using the suggested test, good mean estimates of the stiffness coefficients are obtained for noisy signals and excellent estimates are obtained for clear signals. Moreover, the test is found to be capable of detecting a damage of 20% stiffness loss when it is incurred in one of the pier segments. The suggested test can be extended to include other production quality control applications in construction sites.

Keywords: Bridge, Piers, Integrity, Construction, Non-destructive, ERA, Stiffness.

INTRODUCTION

Piers Importance

Often, bridge piers are the most decisive factor in obtaining proper esthetics for a bridge structure. Proper location, selection of a suitable configuration that blends well with both the superstructure and the terrain, and careful proportioning of section dimensions have a significant influence on structure appearance and behavior [7]. Bridge piers serve the general purpose of transmitting vertical and horizontal loads to the foundation underneath. In addition to the requirement to carry loads to the foundation, bridge piers often are required to resist superstructure rotations, which are the result of moments induced by frame action when piers are cast monolithically with the superstructure. All bridge piers and compression members should be designed by strength design procedures with appropriate consideration given to serviceability at a working stress level. The primary serviceability criteria should be a crack control evaluation.

Piers Types

The types of piers can be divided as per their geometrical cross sections (configurations), or as per their modeling in the structural system as well as per the materials used in the detailing of such piers. Piers configurations may be single shafts or multiple column bents. Single shaft piers generally blend well with required esthetic treatments and offer minimal restriction to drift in a stream. It is essential that pier configurations be compatible with the type of superstructure that they support, and also with the type of connection made to the superstructure. Moreover, piers may be constructed to be integral with the superstructure or they might be connected by a pinned or an expansion-type of bearing. In addition, reinforced concrete is the material usually used in the bridge pier construction. Another type of pier utilizing concrete is a composite column that is a combination of structural steel and concrete.

Construction requirements and consequences

Due to the large sizes of piers, construction of such structures is not an easy task. A continuous flow of large concrete volumes during casting piers is required. This however may be subjected to any unexpected disturbance in the batch plant unit supplying concrete thereby causing a separating layer in the concrete body. In addition, special formwork that can withstand the lateral pressure on concreting is needed during piers erection. Moreover, piers are usually heavily reinforced due to expected lateral and vertical loads to be loaded on the pier structure. This heavy reinforcement together with large heights of piers makes it possible for concrete caving to take place. The large heights of concrete and the long duration of concreting together with slump requirements make occurrence of segregation of concrete a good probability. In addition, the problems in construction of piers are not restricted to those mentioned above. Many other different problems may occur and still affect the integrity of piers.

PROBLEM DEFINITION

Gathering all the factors accompanying the piers erection in site, some defects may occur during construction of piers. These defects, resulting from construction problems, vary from segregation of concrete to separated layers of cast concrete, as well as concrete caving and other defects. However, these defects though well understood yet may not be evident from visual inspection. In addition, knowing the importance of piers and the accompanying consequences of a malfunctioned pier in a bridge structural system, it can be easily figured out how crucial is detecting the integrity of piers. The question, thus, lies in the credibility of locating and identifying the damaging defect exactly. However, identifying the type of damaging defect is not a straightforward easy task. Moreover, the damage in a pier sometimes may not affect the overall global behavior of the pier itself. Thus, a quick decision of demolishing the pier, which had a problem during its construction history, is not an economical or a practical solution. Accordingly, knowing that a damage exists together with locating it, even without seeing it exactly, is considered an essential act. Using damage quantification and localization plus the history of the pier construction may eventually lead to the interpretation of more information about the incurred damage and how effective it may be to the life of the bridge structural system as a whole.

As erection of piers precedes the erection of the bridge deck system, it would be logical as well as economical to identify the integrity of piers before they are loaded. This is done especially when construction problems take place during erection of the tested piers. Doing that helps avoiding subsequent massive losses in material or human lives. This paper studies the identification of damage content (cast concrete defects) and damage location along the height of piers before piers are structurally loaded. The paper introduces an idealized structural model for the piers and uses the ERA (Eigensystem Realization Algorithm) technique [4-6, 9] in identifying the variations of the lateral stiffness in it. The model divides the pier vertically into horizontal strips and through vibration of their equivalent masses, the stiffness of these strips are evaluated. The change in stiffness is considered an existing damage.

IDEALIZATION AND MODELING

Within this paper, the single shaft pier is the case studied. The material of the pier body is assumed made of reinforced concrete, which is the most popular case. The cross section of the pier is assumed uniform along the height of the pier as shown in Fig. 1. The foundation base of the pier is considered totally fixed to the ground underneath (rigid foundation). In order to locate the damage, the pier is considered divided in the vertical direction into horizontal strips as shown in Fig. 1.

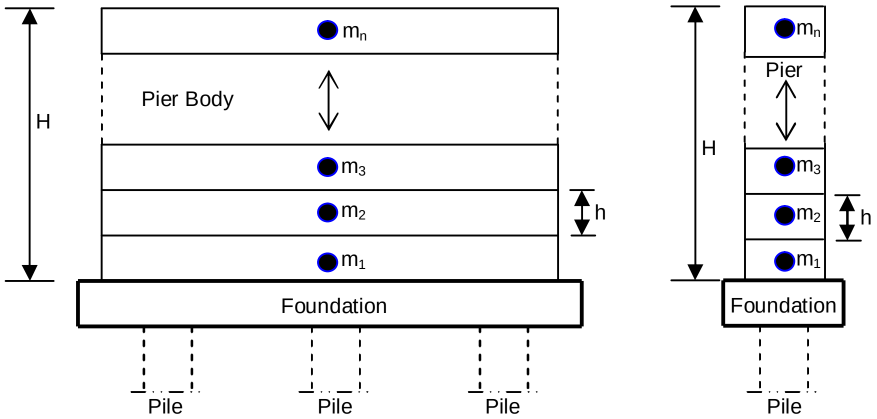

Fig. 1: Splitting single shaft piers into horizontal strips with lumped masses

Dividing the pier into strips, the mass of each strip is lumped as shown in Fig. 1 and the dynamic model of the whole pier can thus be dealt with as a shear-building model as shown in Fig. 2. The lumped mass of each strip is evaluated as per the volume of the suggested modeled strips. The strips in the proposed model are considered of equal heights, h, thus equal masses are imposed in the model. Similarly, the stiffness of each strip k_i, i=1 to n strips, is initially estimated as per the suggested cross section of the piers. Each mass is assumed to have a single degree of freedom so the number of degrees of freedom is relative to the number of masses. It is the role of the proposed non-destructive test to identify which strip includes damage. The damage in this paper is modeled in terms of a change or loss in the expected stiffness coefficient.

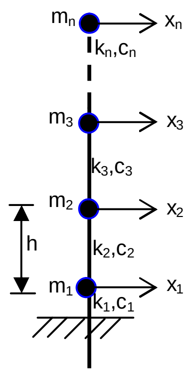

Fig. 2: Equivalent Shear building model of a single shaft pier

Numerical model

The numerical model of the pier here assumes actual values of the stiffness and damping coefficients of the pier that were used before in a lot of recent papers [3]. The values are obtained from actual pier sizes (cross sections) in bridges constructed in Japan. The actual stiffness coefficient of each strip in the model here is taken equivalent to 15791 kN/m in the short direction of the pier and the actual damping coefficient of each strip is taken as 125 kN.s/m. The stiffness of the different horizontal strips is considered the same due to uniformity of the shape of the pier. This is also applied on the damping coefficients of the strips. The tested pier is assumed of 4.5 m height. The pier is assumed divided into six strips in the vertical direction and the height of each strip is considered equivalent to 0.75 m. The pier is assumed made of reinforced concrete and each lumped mass has a single degree of freedom in the horizontal direction. The mass of each horizontal strip is assumed equivalent to 16.5 tons. In addition, the pulse responses of the different masses in the model are assumed measured a priori.

To check the potential of the test, it is assumed that the third strip from bottom has damage in terms of a loss in stiffness of 20%. This damage is considered a result of a construction problem that took place during the concreting of the pier. The reason of picking such a small loss in stiffness is to test the accuracy of the test to smaller damage. The test is applied assuming once clear response signals, then for practical reasons, the response signals are assumed noisy due to other influences such as the sensor noise.

PROPOSED TEST METHOD

The test proceeds with virtually dividing the tested pier vertically into horizontal strips, and considering the masses of the different strips lumped. A set of sensors (accelerometers) should be located in correspondence to the locations of the lumped masses as shown in Fig. 3. The sensors are fixed to the pier so that the horizontal accelerations corresponding to the different masses of the pier strips are measured in the weaker direction of the pier stiffness. This is done to make use

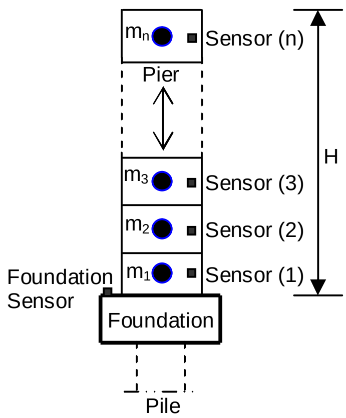

Fig. 3 Allocating accelerometer (sensor) locations to collect data

of larger magnitudes of vibration in the shorter direction of the pier cross section to achieve better test results. A sensor should be also located in the foundation level as well. The sensors are essential for measuring the pulse response of the different equivalent lumped masses, in case a hammer load is used to obtain a pulse response. Alternatively, ground ambient vibrations can also be used, and the NExT (Natural Excitation Technique) technique is thus used to obtain the autocorrelation functions between the highest strip response and the other lower strip responses [1-2]. These autocorrelation functions are utilized by the identification technique instead of the pulse response (Markov parameters). After collecting the sufficient data in terms of the vibrations of the different masses (pulse response) then the ERA (Eigensystem Realization Algorithm) is used to identify the stiffness coefficients.

Using the estimated stiffness coefficients resulting from identification together with documents summarizing the history of construction of the pier, a talented engineer can expect the not only the damage quantity and location but its type and classification as well.

Identification Technique (Eigen System Realization Algorithm)

The Eigensystem Realization Algorithm (ERA) is a least squares modal testing method [10-11]. The technique begins by forming a block data matrix which is obtained by deleting some rows and columns of the generalized Hankel matrix, but maintaining the first block matrix intact. The algorithm then uses the system Markov parameters (pulse response) to compute the eigen values and eigen vectors of a system. The later are used subsequently to find the natural frequencies, damping ratios, and mode shapes of the simulated structure. The system is assumed to be discrete-time, linear, and time invariant with n_u inputs and n_y outputs, of the form:

$$\mathbf{x}(k+1) = \mathbf{A}\mathbf{x}(k) + \mathbf{B}\mathbf{u}(k), \qquad \mathbf{y}(k) = \mathbf{C}\mathbf{x}(k) \tag{1}$$

The Markov (pulse response) parameters are given by:

$$\mathbf{Y}(k) = \mathbf{C}\mathbf{A}^{k-1}\mathbf{B} \tag{2}$$

where **A** is the discrete state matrix of dimension $2n \times 2n$, *n* is the number of independent co-ordinates. The matrix **B** is the input influence matrix of dimension $2n \times l$ where *l* is the number of inputs. The matrix **C** is the output influence matrix for the state vector **x**, its dimensions are $2n \times m$ where *m* is the number of outputs. In this paper, the Markov parameters are measured in the time domain by introducing impulses into system inputs. A generalized Hankel Matrix **H**(*k*) is thus:

$$\mathbf{H}(k) = \begin{bmatrix} \mathbf{Y}(k) & \mathbf{Y}(k+1) & \mathbf{L} & \mathbf{Y}(k+s) \\ \mathbf{Y}(k+1) & \mathbf{Y}(k+2) & \mathbf{L} & \mathbf{Y}(k+s+1) \\ \mathbf{L} & \mathbf{L} & \mathbf{O} & \mathbf{L} \\ \mathbf{Y}(k+r) & \mathbf{Y}(k+r+1) & \mathbf{L} & \mathbf{Y}(k+r+s) \end{bmatrix} \tag{3}$$

Where *r* and *s* are arbitrary integers. The generalized Hankel Matrix is then evaluated for *k*=0. This is followed by performing a singular value decomposition on the Hankel matrix **H**(*k*) such that:

$$\mathbf{H}(0) = \mathbf{P}\mathrm{S}\mathbf{Q}^{\mathrm{T}} \tag{4}$$

where **P** and **Q** matrices are the left and right eigenvectors matrices of **H**(0), and the matrix S is the diagonal matrix of singular values. When noise exists, the order of a system is dependent on how noise affects singular values. Once the estimated order of the system 2*n* is identified, the rows and columns associated with the computational modes are eliminated to form a condensed version of the singular values and eigen vectors matrices, S_n, $\mathbf{P}_n$, and $\mathbf{Q}_n$ where

$$\mathrm{S} = \begin{bmatrix} \mathrm{S}_n & 0 \\ 0 & 0 \end{bmatrix} \tag{5}$$

Consequently, estimates of the state space matrices are found [9]:

$$\hat{\mathbf{A}} = \mathrm{S}_n^{-1/2}\mathbf{P}_n^{\mathrm{T}}\mathbf{H}(1)\mathbf{Q}_n\mathrm{S}_n^{-1/2} \tag{6}$$

$$\hat{\mathbf{B}} = \mathrm{S}_n^{1/2}\mathbf{Q}_n^{\mathrm{T}}\mathbf{E}_m \tag{7}$$

$$\hat{\mathbf{C}} = \mathbf{E}_n^{T}\mathbf{P}_n\mathrm{S}_n^{1/2} \tag{8}$$

where

$$\mathbf{E}_n^{T} = [\mathbf{I} \quad \mathbf{0}], \qquad \mathbf{E}_m = [\mathbf{I} \quad \mathbf{0}] \tag{9}$$

The eigen values and eigen vectors of the system can then be known from the identified system matrix $\hat{\mathbf{A}}$ using the eigen value problem equation $\hat{\mathbf{A}}\boldsymbol{\varphi} = l\boldsymbol{\varphi}$ (10)

where l represents the eigen values and j represents the eigen vectors.

Eventually, the matrix $\hat{\mathbf{C}}_n$ matrix is used to transform the computed eigen vectors of the state matrix output shapes (corresponding to non-physical states in the identified model), to displacement at the floors of the structure using the equation,

$$\boldsymbol{\gamma} = \hat{\mathbf{C}}\hat{\mathrm{F}} \tag{11}$$

where $\boldsymbol{\gamma}$ is the matrix of the output shapes and $\hat{\mathrm{F}}$ is the matrix of the eigen vectors of the state space matrix $\hat{\mathbf{A}}$. The ERA method is implemented using MATLAB®.

In the case of study of the research herein, the stiffness parameters of the structure are the main interest. Thus a technique that would evaluate such parameters from the modal parameters is required [1].

Stiffness Estimation Using Least Squares of the Eigen Value Problem Solution

By considering a lumped mass system with *n* degrees of freedom, the mass matrix is

$$\mathbf{M} = \begin{bmatrix} m_1 & 0 & \cdots & 0 & 0 \\ 0 & m_2 & \cdots & 0 & 0 \\ \vdots & \vdots & \ddots & \vdots & \vdots \\ 0 & 0 & \cdots & m_{n-1} & 0 \\ 0 & 0 & \cdots & 0 & m_n \end{bmatrix} \tag{12}$$

and the stiffness matrix is

$$\mathbf{K} = \begin{bmatrix} k_1 + k_2 & -k_2 & \cdots & 0 & 0 \\ -k_2 & k_2 + k_3 & \ddots & \vdots & \vdots \\ 0 & \ddots & \ddots & -k_{n-1} & 0 \\ \vdots & \cdots & -k_{n-1} & k_{n-1} + k_n & -k_n \\ 0 & \cdots & 0 & -k_n & k_n \end{bmatrix} \tag{13}$$

Then by considering the eigen value problem of such a structure [8]

$$(\mathbf{K} - l_j\mathbf{M})\boldsymbol{\varphi}_j = 0 \text{ or } \mathbf{K}\boldsymbol{\varphi}_j = l_j\mathbf{M}\boldsymbol{\varphi}_j \tag{14}$$

where l_j and $\boldsymbol{\varphi}_j$ are the j^{th} eigen value and eigen vector of the structure, respectively.

By considering the form of the mass and stiffness matrices as in Eqs. (12), (13), then Eq. (14) can be modified such that the stiffness coefficients can be assembled in a vector and is rewritten as

$$\mathrm{D}_j\boldsymbol{k} = \mathrm{L}_j \tag{15}$$

Eq. (15) can be applied for all of the *n* eigen values and eigen vectors identified. Thus, by gathering all of the equations corresponding to Eq. (15) in one big matrix equation, one would get,

$$\begin{bmatrix} \mathrm{D}_1 \\ \mathrm{D}_2 \\ \vdots \\ \mathrm{D}_n \end{bmatrix}\begin{bmatrix} k_1 \\ k_2 \\ \vdots \\ k_n \end{bmatrix} = \mathrm{D}\boldsymbol{k} = \mathrm{L} = \begin{bmatrix} \mathrm{L}_1 \\ \mathrm{L}_2 \\ \vdots \\ \mathrm{L}_n \end{bmatrix} \tag{16}$$

The stiffnesses vector $\boldsymbol{k}$ is thus computed by the relation,

$$\boldsymbol{k} = \mathrm{D}^{-1}\mathrm{L} \tag{17}$$

It is important to note that, the matrix D is not square and consequently a pseudo-inverse of this matrix is computed to obtain a least squares estimate of the stiffnesses.

ANALYSIS OF RESULTS

By applying the aforementioned non-destructive test, stiffness coefficients estimates of the different pier horizontal strips are obtained. The values of the stiffness coefficients estimates are evaluated once using clear response signals without noise. It is found that, the estimated values matched exactly the assumed exact ones. This, though shows the success of the test technique, yet may not be practical since noise is an expected factor. On consider- ing noisy signals, the stiffness coefficients estimates show slightly different values, depe- nding on the noise effect. Accordingly, showing the results of a single case with one noise pattern may not be indicative of noise as a fact and its consequent effect on the identification of the stiffness coefficients. Then, it seems appropriate to obtain mean values of the stiffness coefficients of the different pier strips. This is done for twenty noise patterns. Figure 4 shows a comparison between the values of the exact stiffness coefficients versus those estimated from clear signals and the mean estima-tes of the stiffness coefficients obtained from noisy signals. Thus, it can be concluded from Fig. 4 that the testing techni- que was successful in identify-ing the stiffness of the pier strips. Even with noisy signals, the error was within 2.6% of the actual stiffness value.

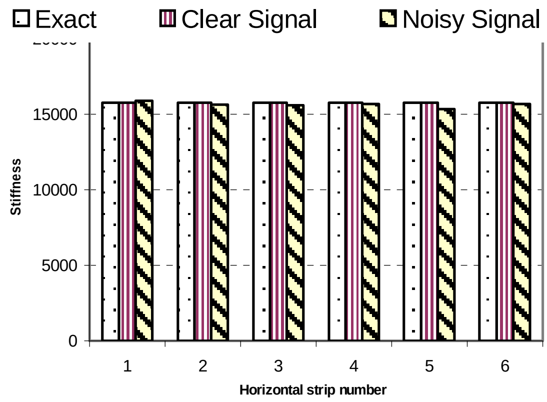

Fig. 4 Stiffness coefficients for the different horizontal strips (no damage)

In the case of having a 20% damage in the third pier strip from bottom, the test technique is found to be successful in locating damage in either cases of clear signals or noisy signals. On dealing with noisy signals, it can be seen that the mean estimate of the stiffness coefficient of the damaged
strip is very close to the exact value with an error of estimation of 1.5%, as shown in Fig. 5. This identifies the damage correctly in terms of location and almost exactly in terms of quantity.

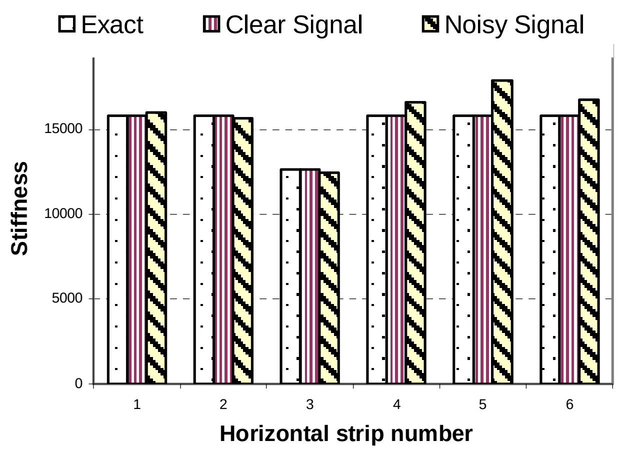

Fig. 5 Stiffness coefficients for the different horizontal strips (20% damage in the third strip from bottom)

Furthermore, looking at Fig. 5, one can notice that the stiffness coefficients of the 4th, 5th, and 6th pier strips are overestimated. However, and since the cross section dimensions of the pier are known, then a very good estimate of what the stiffness of the pier should be is available. This consequently means that the excess of stiffness can be practically neglected and that the focus in checking the integrity of the pier should be rather on the loss in stiffness. Table 1 shows the exact values of the stiffness coefficients of the pier strips as well as the estimated values on using the proposed test with clear signals and the mean estimates of the stiffness coefficients when using noisy signals.

Table 1. Estimated stiffness coefficients (kN/m) for the six different horizontal pier strips

	No damage			20% damage in 3rd strip from bottom		
Horizontal pier Strip	**Exact**	**Clear Signal**	**Noisy Signal**	**Exact**	**Clear Signal**	**Noisy Signal**
1	15791	15791	15880	15791	15791	16008
2	15791	15791	15648	15791	15791	15666
3	15791	15791	15611	**12633**	**12633**	**12438**
4	15791	15791	15688	15791	15791	16607
5	15791	15791	15374	15791	15791	17887
6	15791	15791	15693	15791	15791	16769

CONCLUSIONS

A non-destructive test that checks the integrity of full piers is introduced. A suggested idealized model for the pier is used. The pier is assumed divided vertically into horizontal strips with lumped masses. The pier is therefore modeled as a shear building where each lumped mass has a single degree of freedom. The test utilizes the ERA (Eigensystem Realization Algorithm) identification technique. The latter requires knowing the pulse response of the pier, measured using sensors attached to the tested pier at several levels along its height. This is done in order to identify the stiffness coefficients of the different pier strips. The proposed test shows capability of giving very good estimates of the stiffness coefficients for clear or low noise signals. The proposed technique managed also to identify a 20% loss in stiffness (damage) in one of the pier strips. The test introduced evaluates the integrity of piers through global vibration and thus the expected outcomes can define the integrity of the full tested pier. This is superior to obtaining results that can define the integrity of piers locally and is considered an advantage of the proposed test against other local tests. The suggested test can be extended in future to include other production quality control applications in construction sites.

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DEVELOPMENT OF A MODEL TO EVALUATE THE COST OF QUALITY IN THE CONSTRUCTION INDUSTRY

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ABSTRACT

In an increasingly global and competitive environment, an organization's long-term survival may depend on improved competitiveness. The construction firms have to achieve lower construction costs, better quality, greater customer satisfaction, a higher market share to be able to compete successfully.

Quality improvement program should be designed to minimize the total cost of quality in order to achieve the required level of quality with lower operation cost, this could be achieved by eliminating or reducing the non-conformance quality cost in the construction projects.

The main objective of this paper is to develop a quality cost measurement system in construction industry. The quality cost system is a valuable tool to provide a sufficient basis for taking action by identifying the areas and sizes of quality problems, evaluating the opportunities for quality improvement and enabling the organization to identify and eliminate non-conformance quality cost.

1- INTRODUCTION:

The control of quality needs to monitor the actual performance with the planned or desired achievement. It will be impossible to control quality without measuring it to determine whether the required level of the quality is achieved and to provide a sufficient basis for taking action.

TQM is a process of continuous improvement but suffer from overdose of flexibility in that it lacks a clear framework or common procedure. Most construction people prefer to have specific measurable objectives for quality.(Gunning ,1996)

2- THE IMPORTANCE OF MEASURING QUALITY COST

1- The cost of quality in many industries and services is big, which develops the need for a system to measure, control and reduce this cost. Bottorff (1997) stated that manufacturing organizations spend between 25 and 40 per cent of sales revenues on quality-related costs, Juran (1999) estimated that quality related costs range form 5 to 25 percent of company annual sales turnover. Also Harrington (1987) stated that Quality experts argue that a typical company can save more money by halving poor quality costs than by doubling sales.
2- The cost of quality is a valuable tool to identify the areas and sizes of quality problems because it presents the value and occurrences of these problems and give alert to the manger about the problem effect.

3- The cost of quality provides the top management with decision making tools to repair the failure in the traditional accounting system in providing a clear view of the organization efficiency. Gupta and Campbell(1995) mentioned that current accounting systems are incapable of providing decision-making information (including cost of quality information) necessary for strategic, as well as tactical, and operational decisions. While Ittner (1996) argued that ``more micro-level examination of quality cost behavior may provide a better understanding of the underlying economics of quality improvement, thereby allowing more informed decisions on the allocation of resources to quality-related activities''

4- The cost of quality provides the organization with a method to reduce the cost of its product or service by reducing the cost of poor quality. Therefore the organization can increase its profitability and market share

5- The cost of quality is a valuable tool in quality improvement because it can identify and prioritize, and evaluate the opportunities for quality improvement and enable organization to identify and eliminate non-value added activities.

6- The cost of quality provides the organization with a way to justify and evaluate the investment in quality and helps in evaluating the return on quality.

7- The cost of quality system leads to the development of a more advanced performance measure in the areas of customer satisfaction, production and design, because it identifies the cost related to claims of customer, warranty, etc . (Bottorff 1997).

8- The quality costs are, first a tool for focusing management attention on quality, and second a measure of the success of a quality improvement program.(Crosby ,1979) , while Juran (1999) stated that the language of money improve communication between middle manger and upper manager and it's the basic language of upper manger.

3- QUALITY COST IN CONSTRUCTION INDUSTRY

According to Dale and Plunkett (1999), quality costs are important because these can be extensive. In 1978, these were estimated by the UK's Government to be equal to 10 per cent of the UK's gross national product. One can refer to these figures to stress the significance of quality costs. Ledbetter tracked used Quality Performance Measurement System (QPMS) and estimated the cost of quality as 11.2 % of total labor expenditure (1994), while Barber (2000) determined the cost of failure only in two projects as 16 percent of total cost in one project and 23 percent of total cost in a second project.

In a study on construction projects in Australia, it was found that through spending 1 per cent more in prevention costs, the failure costs can be reduced from 10 per cent of construction costs to 2 per cent (Roberts, R. 1991). While The Construction Industry Development Agency in Australia (CIDA (1995) has estimated the direct cost of rework in construction to be greater than 10 per cent of project cost.

The pervious studies clearly show the need for a system to identify and measure the quality cost in the construction industry, without a formal systematic quality management system in place, quality deviations may not be identifiable. Consequently, information is lost and activities that need to be improved in order to reduce or eliminate rework cannot be ascertained. (Davis 1989)

The quality cost system will significant effect not only quality but also the cost of construction project, Latham (1994) considers it possible to achieve a 30% reduction in the construction cost in the UK within five years, TQM can provide the vehicle for achieving much of this. While The BRE (1982) stated that 15 per cent savings on total construction costs could be achieved through eliminating rework, and by spending more time and money on prevention.

4-QUALITY COST MEASUREMENT SYSTEM

The main function of the proposed quality cost measurement system is to capture those costs which are hidden and scattered in different cost elements in the construction projects and convert those cost into relevant information that could be used to satisfy the purpose of the system and to enhance the organization's strategy towards quality. The development of proposed quality measurement system goes through three processes, which are identification, quantification and classification of the quality related activities.

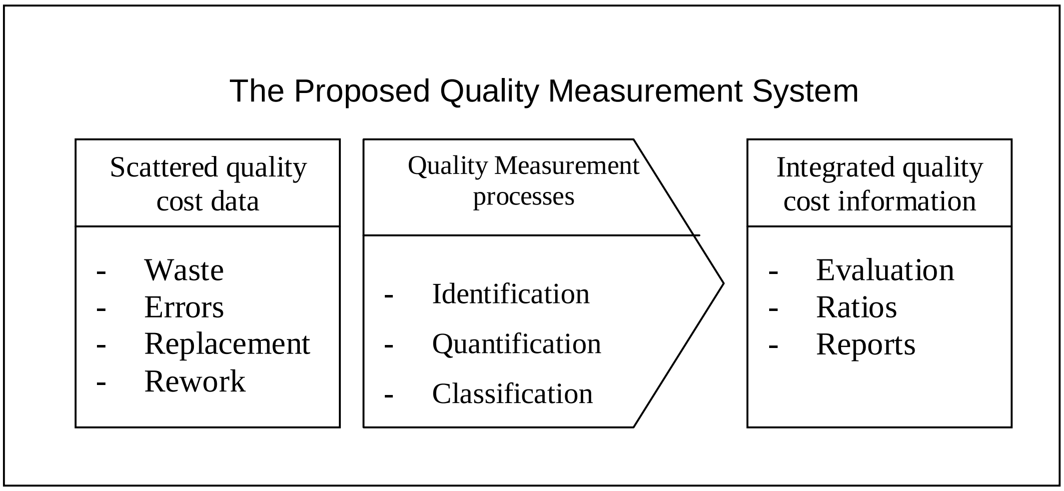

Fig. 1: Quality Measurement System

The quality measurement system processes are defined as follows:

1. Identification: is the process of identifying quality related activities and determining that a quality cost element has been occurred during the execution of the construction projects.
2. Quantification: is the process of estimating and measuring the value of the quality costs after this element has been discovered in the identification process.
3. Classification: is the process of classifying the identified quality cost elements to a relevant category (e.g. prevention, appraisal, and failure) in order to represent this cost element in a form could be used to satisfy the scope of the measurement.

The proposed model introduces a comprehensive and integrated methodology in order to help the construction firms make use of the collected data in evaluating their quality performance.

4-1 The Proposed Method for Identifying Quality Activities in the Construction Site:

The suggested model based on identifying non-conformance activities by defect type provides a simple and clear methodology. Also it suits the condition and complexity of construction process. This method depends on estimating the number and correction cost of each non-conformances.

In order to Identify quality related activities in the construction projects (which usually inherent in the construction process,) , an appreciate person like engineer or quality controller or quality supervisor is assigned to discover and identify these quality activities in the site.
This method was used by Barber and Tomkins (2000) to collect failure cost in construction site and they called this method work-shadowing method, they used a Key personnel such as engineers, foremen and other key operatives were shadowed for a period of time and the

quality problems they encountered were recorded, also similar approach was used by Haamarlund & Jacobson, ((1990a), (1990b) where they referred to the person who collect quality cost in site as a ``quality observer''.

Data were provided through the direct experience and observation by quality observer while supplementary information was found in the site diaries, from invoices and orders, the project program, allocation sheets and the bill of quantities. The quality failures data were supplemented by data concerning prevention and appraisal costs for the project, acquired by a quality observer. What should be measured is any disruption to the construction of the finished product, however that may be manifest.

The quality observer was required to monitor quality failures on site through self-monitoring and observation. This meant that the quality observer was personally responsible for recording incidents and suggesting possible causes for the manifested effects. Consequently, the process relied on an open, 'no-blame culture', where the tendency to hide or conceal quality incidents was mitigated in the knowledge that the exercise would be used in a constructive, problem-solving and learning spirit. Each quality incident was recorded and evaluated as it arose.

In the spirit of inclusiveness, all staff were encouraged to get involved in recording any incidents that seemed to be relevant to the exercise, relating either to their own activities or those of other parties in the supply chain (e.g. suppliers and subcontractors, designers and the client). Thus, the project manager, site agent, quantity surveyor and junior staff were all involved in recording and collating data. The incidents were then categorized by activity within PAF process model. Not only did this approach enable a more complete picture of quality failures to be established, it also developed data collection and performance measurement competencies among the participating staff. (Hall &Tomkins 2000).

4.2 The Proposed Method for classifying Quality Activities in the Construction Site:

The quality costs in the proposed model will be classified primarily according to the PAF model for the following reasons:

- The flexibility and ease of use of the PAF model.
- The ability of the model to work with the low-level quality environment.
- The model is based on capturing the tangible quality cost like errors and rework which the most frequency is happening in construction process.

The PAF model presents a list of cost elements under the model categories (prevention, appraisal, internal failure and external failure), this PAF works as guideline and general acceptance as reference for quality cost elements categories. However the PAF need to be modified and adjusted to suit the characteristics of the construction industry. It is required to adopt the proposed model to suite the construction.

Reservation should be taken by the firm to overcome the following points.
The distinct limit between internal and external failure in construction industry isn't existent, because the customer (the owner himself or the representative of the owner as owner engineer or the consultant) can internally discover failure during construction process and before the project finished, the difference between the internal and external failure had been modified to overcome this gap as following:

1. The internal failure: any failure discovered before the final turning over of the building to the owner regardless who discovered the failure.
2. The external failure: The failure, which has been discovered by the customer after the product completely, transfers to the owner.

Also the quality related activities in the Proposed model will be classified according to the maser WBS list of the firms, cause of quality failure (for failure category) and the ISO areas.

4.3 The Proposed Method for Quantifying Quality Activities in the Construction Site:

The quantification process is the process of estimating the costs of the discovered quality items. This process begins after identifying the quality cost items.
Fig. 2 illustrates that the quality cost is broken down to Direct cost categories namely material cost , equipment cost , labor cost , and indirect cost like site overhead and other indirect cost.

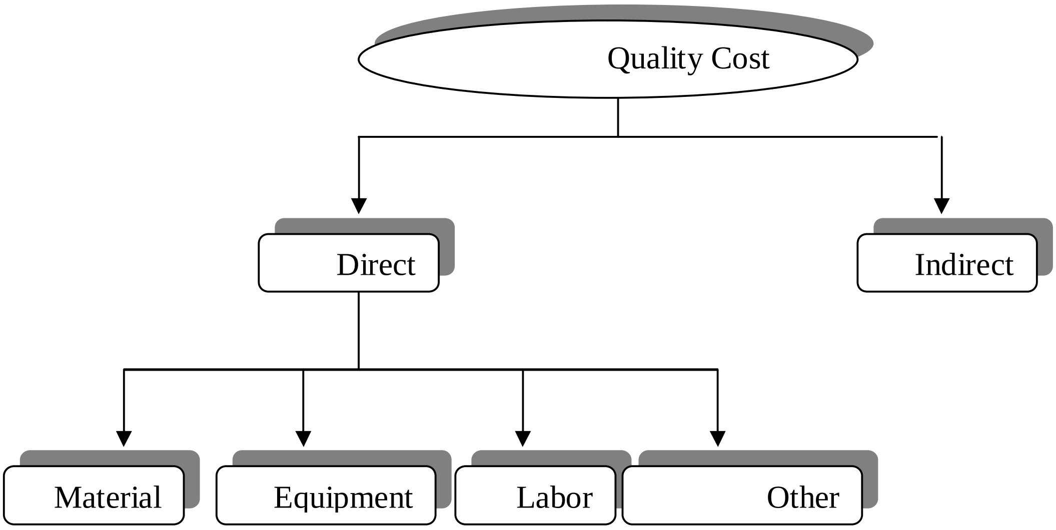

Fig. 2: Quality Cost Components

In the quantification process the quantity observer has to estimate the quantity of each category by breaking down the resource consumed by the quality cost activity, then he has to estimate the cost of these categories. As shown in Fig. 3.

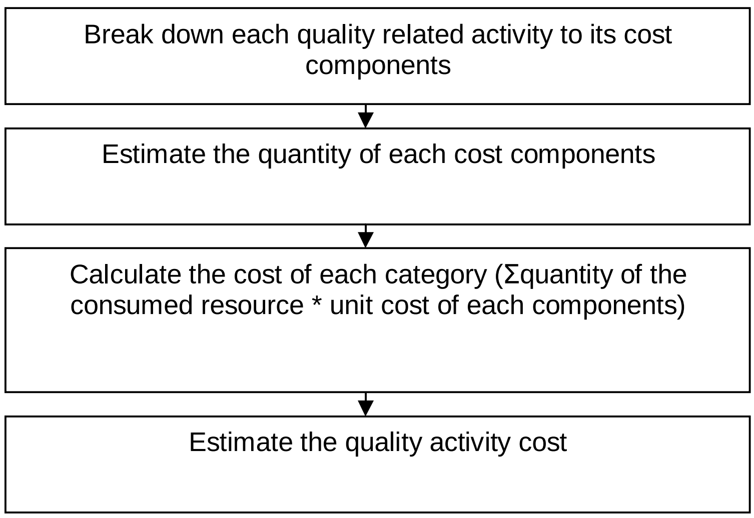

Fig. 3: Quantify The Quality Related Activity Cost

The whole integrated process of the proposed and the interrelated relationship between the major processes of the model are shown in Fig 4,

5- THE MODEL CONSISTS OF TWO BASIC PARTS:

- **Part 1:** a set of forms that used to collect the required data from the construction site to evaluate the value and effect of the quality and non-conformance quality in the construction firms, also the forms system should acquire information about the cause and root of the quality failure to use it in the process of analysis the quality in the construction industry. **As shown in appendix (A) .**

  The following areas are covered by the documentation system, which used to capture quality cost from construction site in order to fulfill the requirements of such system in measure quality (conformance and non-conformance) cost, and its applicability in failure analysis and continuous improvement.

  - **Item description:** in this area a brief description of the activity should be demonstrated by the person (quality observer) who fill the form in order to:
    The description could be used as additional source of information (specially in failure analysis) to review the condition that is surrounding the quality (failure) activity.

    This description allows the check of the accurate of assigning the other areas in the form, (because the activity was described the quality manager could review and modify the form filling).

  - **Classification code:** this area of the form is related with classifying the quality activity into his category (prevention, appraisal, and failure) then classify the activity into suitable sub-category (i.e. rework under internal failure or inspection under appraisal category) until the required level of detail.

  - **Quality activity location:** in this area the location of the quality activity should be determined (i.e. department, project, building, and floor), the important of this area to help in benchmarking between projects and departments, also this area helps in determining the location that the failure most occurrence in the project.

  - **Cost of the quality activity:** in this area the resource (material, equipment, human hours, money) of the quality activity are listed and the cost of this activity is calculated.

  - **Work break down structure:** the data of this area is to assign the quality cost activity to the type of work (concrete work, brickwork, ..etc) that this activity occurs in , the master WBS of the firm could be used , or special list containing the type of works that the projects of the firm deal with.

  - **ISO areas (optional):** the quality activity (both conformance or failure) should be assigned to its area in ISO standard , the importance of this area is that it makes a feed back process to the application of ISO in the firm quality system (if the firm applies ISO system) or helps the company to certify the ISO if the firm intends to pass ISO certification in the future.

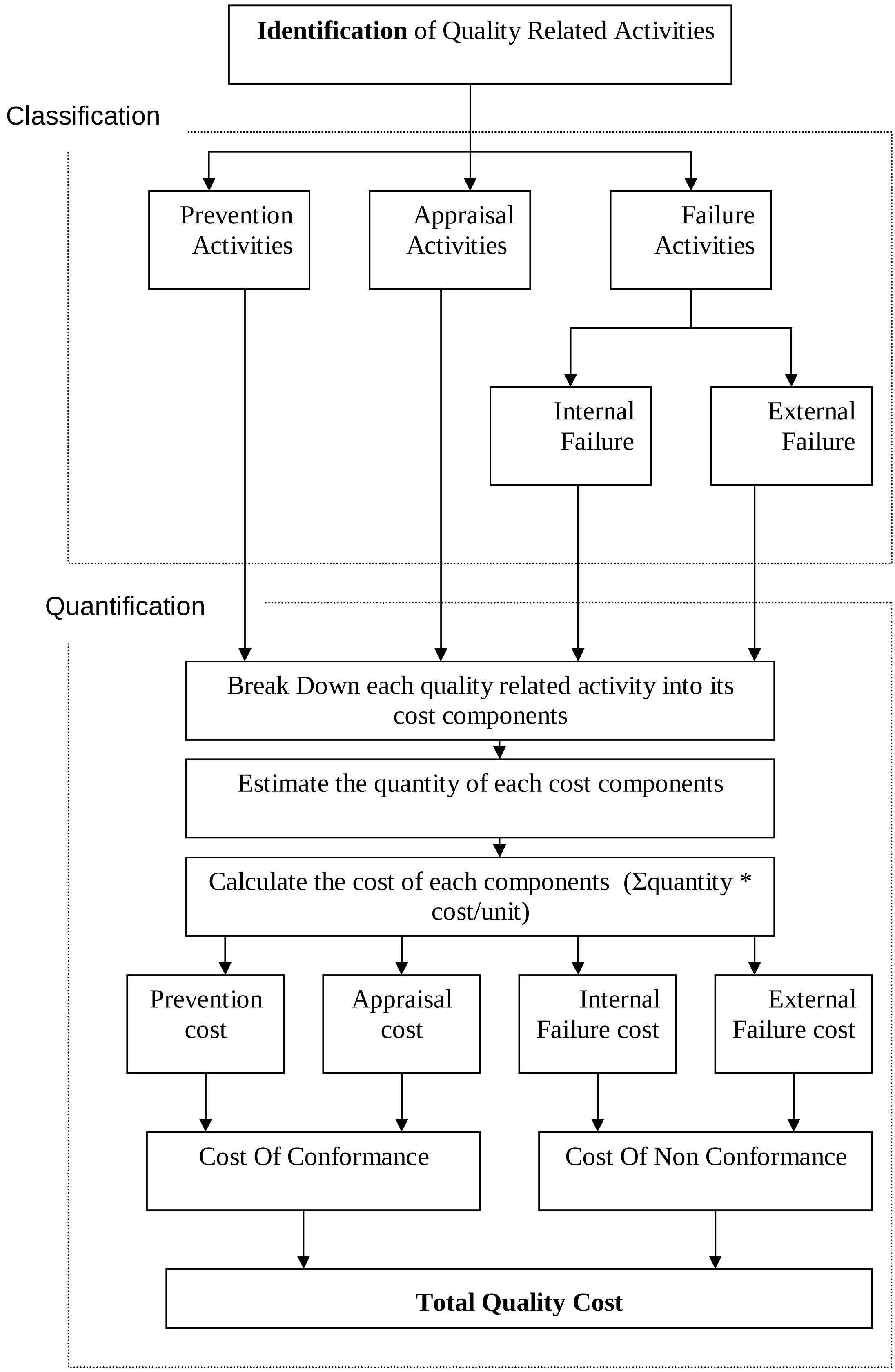

Fig. 4: The Proposed Model

Fig. 4 shows the integrated proposed model

- **Causes of failure:** this area contains the categories of the causes of failure in construction industry, this area focuses the manager attention on the failure that most occurs like planning, supplier …etc, (i.e. if the supplier is most failure category, the manager should give attention to select and inspect the supplier of the firm to prevent failure). Also a detailed description for the cause of failure should be mentioned by the site staff because this area is considered as valuable source of information.

- **Time consumed in failure:** this area presents the time consumed in the remedy of the failure activity that is discovered in site, and the total additional time required by the project as the failure happens (this time consists of time of fixed failure plus the time that the project delays as failure happens).

- **The failure responsibility:** the side that is responsible for the error happening should be determined (the consultant, the owner, and the supplier, .. etc).

- **Failure phase:** in this area the phase of the project in which failure is discovered and the phase of project in which failure happens should be determined

- **Part 2:** a simple program designed to collect and analysis the data from the documentation system and converts those data into useful information that could be used in the purpose of improvement, and decision-making.

6- QUALITY COST REPORTING

Quality cost reporting should be designed carefully in order to indicate the causes and roots of failure in the construction projects, and to enhance the corrective action process. The level of details and the contents of each report depend on the purpose of report and the user of the report. (The report that will be presented to the quality manager should be different from the report for senior or general manager).
Quality cost by them present insufficient information for analysis. a base line is required that will relate quality cost to some aspect that is sensitive for change . Those quality bases represent index to benchmarking between project phases and the different projects of the firm.

- Labor: quality cost per labor is a common index; the information about the cost of labor is usually available from accounting system.
- Project: quality cost from project cost is another common index. Project cost information is readily available from contract or project budget.
- Unit: quality cost per unit such as kilometers of road, or meters of concrete, is an excellent index for determination the origin of failure.

7- SUMMARY & CONCLUSION

Cost of quality is a valuable tool to measure quality because it identifies the areas and sizes of quality problems, provides the top management with decision making tools to repair the failures, leads to the development of a more advanced performance measure, and it presents information of the quality, failure, and efficiency of organization in a language attract the attention of the top management.

In an attempt to prepare the quality cost measurement system in order to assess the quality in the Egyptian construction industry, a quality cost measurement system was formulated. The suggested model philosophy is the integration between the PAF model as a base for classification quality cost activity with the identification and quantification process. This integration provides the environment, and the mechanism needed to satisfy the desirable characteristics for quality measurement in the construction industry.

The implementation of the PAF model in capture quality cost in the construction industry has many advantages due to the model flexibility and ease of use and the model ability to work with the low level quality environment.

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QUALITY COST FORM FOR CONSTRUCTION SITE

1- Description Quality Related Activity:

A-Activity description: --

-----.

B- Activity location(identify the location of quality activity)---------------------------------------
-- ---.

2- WBS List: (Remark the WBS that the quality related activity occurred in)

- 100 Foundation
- 200 Concrete Work
- 300 paint
- 400 sanitary work
- 500 doors

3-Classification:

A- The **PAF** classification: (Remark the classification category then type its code & name)

• **Prevention**	Code (-------------------) & Name (--------------------------).
• **Appraisal**	Code (-------------------) & Name (--------------------------).
• **Failure**	Code (-------------------) & Name (--------------------------).
	Cause of failure: (Choose one) • Design error • Construction error • Defect material • Equipment • Planning • Personal error
	Time Lost: • Time lost because of failure: ----------------. • Time of project delay:-------------------------.
	Responsibility of failure: • Owner • Contractor • Designer • Supplier • Sub-contractor
	Project Phase: (The failure discovered in the following phase): • Design phase • Execution phase • Installation phase
	Recommendation: -- ---

QUANTIFICATION

A-Material cost:

MATERIAL	QUANTITY	UNIT	COST PER UNIT
		Total Material cost	

B- Labor cost:

LABOR	QUANTITY	UNIT	COST PER UNIT
		Total Labor cost	

C-Equipment cost:

EQUIPMENT	QUANTITY	UNIT	COST PER UNIT
		Total equipment cost	

D- Other Expenditure:

------------------------------------- **Subtotal** : -----------------

E- Indirect Cost:

------------------------------------- **Subtotal** : -----------------

Total Quality Cost (----------------------------------)

Appendix (A) Quality Cost Form

A PROCESS APPROACH-BASED SYSTEM FOR MANAGEMENT SYSTEMS DOCUMENTATION

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ABSTRACT

During the last decade, the process approach has gained more concern and was adopted by many organizations as a concept in building their management systems. However, a very few number of organizations are correctly applying this concept because of the incomplete understanding. Scanning the quality management systems for many organizations revealed that most of them are theoritically implementing this concept but in fact, their systems are built and are running depending on the traditional documentations methods that appeared in the late 70s. This paper comprehensively explains the reasons for this misunderstanding and shows the basics and main differences among the process, activity and task. The paper also shows a new proposed documentation system that takes into considerations all needed components of the process model (input, process, output, resources and control). The tabulated format and the process flowcharting are the main features of the proposed system which was implemented effectively in a large governmental organization in the Arabian gulf region through its journey in certifying all of its departmental management systems. To assure the validity of the proposed system, it was comprehensively assessed by a well recognized international 3rd party organization. The paper also lists the main advantages and outcomes of the proposed system as a result of its implementation in the financial department of the organization. Finally, the paper cites the recommendations needed to to develop and use the proposed system in an effective manner.

Keywords: Process Approach, Quality assurance, Documentation, Management, EFQM, ISO.

INTRODUCTION

Process Model

Although the process model is widely used in many applications, the "Process approach" term is now preferably used after adopting it as the core idea in the new issue of the ISO 9001 standard in the year 2000. A process can be defined as " a set of interrelated or interacting activities which transforms inputs into outputs" [5]. These activities require allocation of resources such as people, machines and/or materials. Figure(1) represents the generic process model.

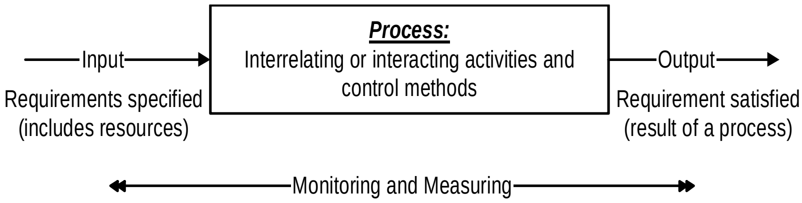

Fig. 1: Generic Process model

The process approach is considered as a "horizontal management approach" for managing all activities and tasks that share in producing an output and could be executed by one or many organizational functions within the organization, see figure (2). This horizontal management approach focuses on goals and avoids the traditional way of management which depends on the organizational hierarchy that leads the organization to be managed vertically with responsibility for intended outputs being divided among these organizational functions.

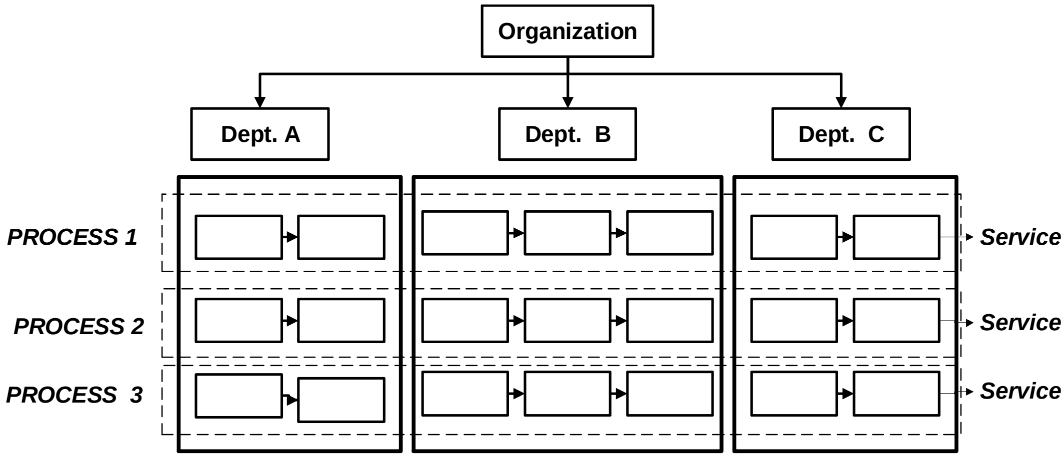

Fig. 2 :Process Linkages across departments (Horizontal Management)

The process approach can be applied to any management system regardless of the type or the size of the organization [5]. Examples of such management systems could be: Environment (ISO 14001), Quality assurance (ISO 9001), Occupational health & safety (OHSAS 18001), and Business risk.

The purpose of the process approach is to enhance the organization effectivness as well as efficiency in achieving its defined objectives. Other benefits of the process approach are:

- Structuring / restructuring the organization depending on its processes.
- Clarity and simplicity to track steps of any operation.
- Integrating processes.
- Time and cost reduction in delivering outputs.
- Ease of identifying improvement opportunities and the development of the process itself.

[5]

Process, Activity And Task

Although the old usage of the term "process", there is a lot of confusion that exists between the respondents to the simple questionnaire conducted by the authors. 73.1% of the total respondents (41) did not distinguish among the terms process, activity and task and they mixed them up. This result matched with the following statement by the international accreditation forum [9]: “Auditees frequently identify too many processes, some or all of them are activities, which do not fulfill the requirement of a process in the sense that ISO 9001:2000 uses the concept". Harrington [3] defined these terms as:

Process : "a logical, related, sequential (connected) set of activities that takes an input from a supplier, adds value to it, and produces an output to a customer".

Activities : "are things that go on within a process and usually performed by units of one (one person or one department".

Tasks : "are individual elements of an activity. Normally tasks relate to how an item performs a specific assignment".

Figure(3) shows the process hierarchy and as can be seen the more you go down, the more details you get. i.e. the task details is more than the activity which is also more detailed than the process.

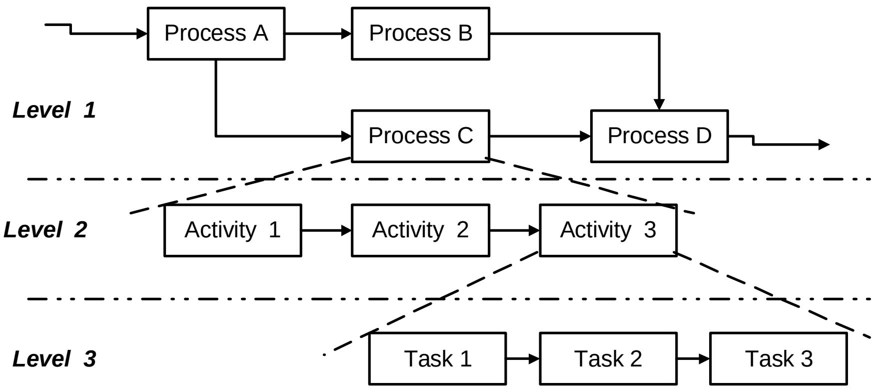

Fig. 3: Process Hierarchy

TRADITIONAL DOCUMENTATION OF MANAGEMENT SYSTEMS

The aim of any system documentation is to enable a common understanding and stable operation of the processes. Each organization determines which processes are to be documented, on the basis of:

- The size of the organization and its type of activities.
- The complexity of its processes and their interaction.
- The criticality of the processes.
- The availability of competent personnel. [10].

It worth mentioning that although the ISO 9001 standard (version of the year 2000) has significantly reduced the documentation requirements much less than 1994 version and has determined six procedures only to be documented, most organizations record the majority of their main and supporting processes to assure consistent implementation regardless the minimum documentation required by ISO 9001.

Most organizations built their documentation system depending on the commonly wel-known documentation triangle shown in figure (4). They un-intentionally correlate this hierarchy with the process hierarchy (figure 3) as follows:

Manual :used to define and to summarize the overall management system, approaches, policies, and responsibilities

Procedure :used to document the activities (level 2 in figure 3). It is used to define: Who, What, When and Where

Work instruction :used to document the tasks (level 3) – defines "how"

Forms and checklists :used as tools within the procedures and work instructions.

Level 1 of the process hierarchy (i.e. process level) is rarely considered in any documented management system. One of the aims of this paper is to put this level into consideration during the documentation stage of the management systems.

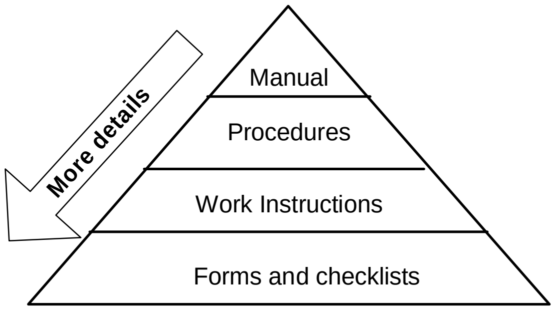

Fig. 4: Management Systems Documentation Hierarchy (Types)

After all, when it is decided to document processes, a number of methods can be used such as: graphical representations, text, checklists or flowcharts. The most commonly used method for documentation is the text. It is used to document procedures and work instructions and usually contains the following components:

1- Purpose. 2- Scope. 3- Definitions & Abbreviations.
4- Responsibilities. 5- Steps (tasks) & rules 6- References.
7- Related documents. 8- Issue & revisions history.

Our main focus will be given to item # 5 above , "steps & rules". In this item, the steps and their sequence are cited in such a way that enable all parties to follow these steps systematically.

By surveying the certified quality assurance systems for 18 organizations, it was found that the majority of these systems (72.2 %) were documented in a citation mode (i.e. logically sequential items). Moreover, none of these systems were found following the concept of the "process approach". The above mentioned finding is supported by the answers to the following question: "during the renewal and updating the systems from 1994 version to 2000 version, did you identify major changes between the 2 systems?". The respondents to this question were 11 of the certification bodies working in the market (those who internationally certify systems and they themselves are accredited from official organizations). 72.7 % of the respondents gave a percentage of 25 % and lower, while 18.2 % chose the range (25 : 50%), and 9.1 % chose 50 and more. (The answer to the previous question was based on the assumption that the certification body was the same for the old and new quality system). It should be noted that the certification bodies questionnaire was conducted using the CATI technique which is the Computer Aided Telephone Interviewing.
Certification bodies (or registrars as so called by Americans) have a common consensus that the traditional way of documenting the systems have many disadvantages (88% of the respondents).

These disadvantages can be summarized as follow:

1- Inconsistency in documentation in large systems.
2- Neither the efficiency nor the effectiveness can be measured by using the traditional way.
3- Difficulty to identify the quality requirements of both inputs and outputs.
4- It is not an easy task to match the job descriptions reports with these detailed in procedures. i.e. the JD is made independent of the procedures.
5- No clear performance measures.
6- The quality control within the process is not easily identified (inspection point).
7- Some steps may not be assigned to anyone due to the unintended use of passive statements.

Therefore, the following paragraphs will explain a newly proposed method for documenting the processes within any organization and based on the process approach model.

PROPOSED SYSTEM FOR DOCUMENTATION

The Need for a New System

The need for a new documentation method raised during a 5-day audit performed on a large governmental organization by the external assessors for the excellence program adopted by the government. This assessment was based on the 9 criteria of the European Quality Award (EFQM) and is done every 2 years for all governmental organizations to determine the excellence award winner.
One of the nine criteria is the "Processes" [1] and has a weight of 14 % of the total 1000 marks. During the audit, the assessors asked 2 questions in this criteria:

Q1: what is the average total duration for one of the main processes to produce the output? And how do you benchmark it to similar local or international organizations?

Q2: what are the inspection activities performed by the organization within the different stages of a process?

The authors were members of the team formed to prepare answers to these questions. The efforts exerted for the sake of providing these information were difficult enough to realize that the current used documentation methods are not capable to easily provide useful information or enable finding the data in an appropriate time.
After the assessment, the authors did their best to come up with a new documentation system that best satisfy the following requirements :

1- Avoid the main shortcomings of the traditional way of documentation.
2- Easy extraction of data and information.
3- The analysis of the processes should be the driving force for defining the amount of documentation needed for the organization. It should not be the documentation that drives the processes.

The following paragraph depicts the outline of this proposed system.

The Proposed System

The main features of the proposed system is a combination of tabular and graphical (flow charts) format. The suggested content of the table is designed in such a way that oblige those who prepare the documents to record their activities through a process-model concept. The second main idea that stands behind the proposal is to be compatible with the concept of "SIPOC" which is very common in the "SIX SIGMA" management [15]. SIPOC is the abbreviation of: **S**upplier – **I**nput – **P**rocess – **O**utput – **C**ustomer. Figure (5) shows this table (SIPOC table).

Sr.	(I) Activity / Task			(II) Inputs			(III) outputs			(IV) Controls	
	Step description	Responsible	Duration	What	Supplier	Specs.	What	Customer	Specs	Notes / resources	Cost (if app.)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)

Fig. 5: SIPOC table

Figure (5) shows the SIPOC table which contains the 4 pillars of the process model: input – process – output – controls. Each column of the table depicts one component as follows:

(I) Activity / Task
  a. Step description: the description of the activity or task performed.
  b. Responsible: step owner or the functional unit which perform the step.
  c. Duration: time elapsed in executing the task.

(II) Inputs

a. What: what is the nature of the input (description).
b. Supplier: who is / are the supplier(s) of the input.
c. Specification: specification or the quality requirements of the input.

(III) Outputs

a. What: description of the output.
b. Customer: to whom the output is delivered to.
c. Specification: specification or the quality requirements of the output.

(IV) Controls

a. Notes, resources, additional requirements.
b. Cost: the cost of the step. In some cases, it is very useful to record the cost associated with each step. This would be very helpful in estimating the total cost of the process , or the cost of specific tasks/activities..

In many cases, it is useful to use the graphical representation to show the workflow of any group of activities / tasks. Therefore, the authors completed their system by suggesting the use of flow charts (Process Mapping) together with the SIPOC table in those procedures that have too many sequential steps (i.e. more than 15:20 steps).

The suggestion of the flow charts usage was accompanied by certain arrangements and formatting (figure 6). For example, the document symbol when added to the left hand side of the "task box" will indicate: Input document, while it is an output document when it is pasted to the right hand side of the "task box". The second example is adding two small boxes attached to the right hand side of the "text box" to show the responsible for conducting the task and the customer who receives the output respectively.

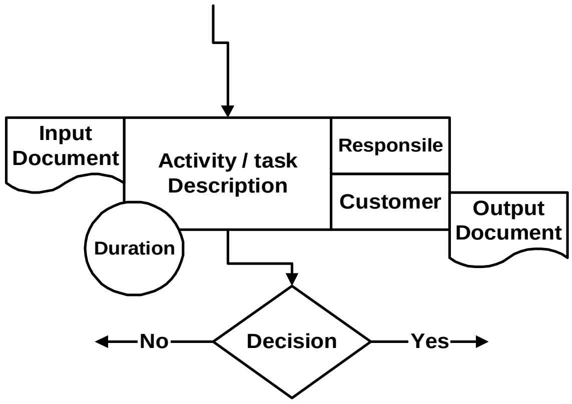

Fig. 6: Symbols used in Flowcharts (Process Mapping)

The above mentioned 2 items of the proposed system (table + flowchart) are used to document level 2 and 3 only therefore, the proposal requires documenting the process level (level 1 documents) in such a way that best describe the interaction between the processes of the management system. Referring to figure (3) “process hierarchy”, the process level documentation (or accurately processes’s interaction) should be either presented in the management system manual (as required by item 4.2.2.c, ISO 9001:2000) [10] or presented as a key figure in each procedure which used in documenting the activity level.

The value added chain diagram (VACD) shown in figure (7) is strongly recommended by the authors to document the processes’ interaction.

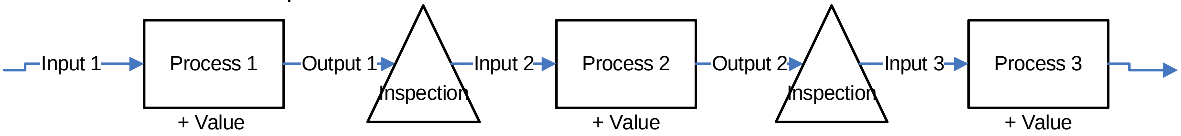

Fig. 7: Value Added Chain Diagram (VACD)

IMPLEMENTING THE PROPOSED SYSTEM

To verify the effectiveness of the newly proposed system, it was applied to develop the management system of the financial departement in a large governmental organization. This organization adopted a policy of having a separate quality assurance system for each of its 31 departments. The financial departement employs approximately 211 employees occupying 5 sections. The overall final quality assurance system contained 95 different procedures. The documentation stage started in September 2005 and lasted for 4 months.
Before starting the stage of documentation, 3 workshops were held to train the documentation team on documentation in general and on the new method in specific. During the training workshops the selection of those processes that need documentation according to the SIPOC table and those that may come at later priority was completed. This selection was conducted using the following criteria "Process Selection Matrix" (see figure 8).

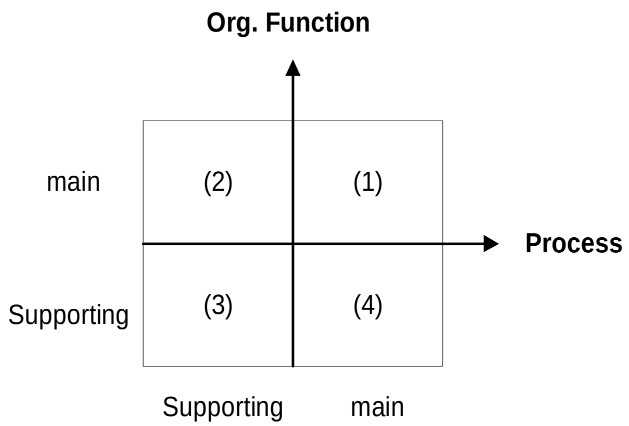

Fig. 8: Process Selection Matrix

Taking figure (8) into consideration, the procedures which lie in the quarter # 1 (i.e. main process in a main organizational function) were categorized to be "class A" document and consequently the SIPOC table was obligatorily used. The same was applied for procedures in the quarter # 4 (main process in a supporting organizational function). Those procedures which lie in quarters # 2 & # 3 were left to the choice of the documentation team and process owner to decide whether to use flow charts only or to use the complete system (table + flowchart).

The new system is distinguished by its rich format and features. The most apparent and important feature was of the new system the analytic ability of the tabulated procedures, which can lead to many explanatorily reports.

The comprehensive job-description report was one of these explanatorily reports. To activate this important report, the authors put a simple methodology to standardize the job acronyms and added a coded structure starting from the department director down to each position. This was implemented a while after the project start to deal with problems that raised from different naming of jobs that hinder the filtering function.

After coding of all jobs in-relation to the organization chart, it was possible to aggregate the tables of all processes into one major table which led easily to sort the jobs or filter it to extract every job report. It worth mentioning that when communicating these reports to directors, they re-structured the reports with the help of affinity function to show the activity level (less details) because the reports were derived from the SIPOC table which based on the task level (i.e very detailed and comprehensive).

The second main analytic report was generated by summing the tasks durations that constitute an activity, which helped in determining if there was a lack of staff or overstaffing (taking into consideration the frequency of occurrences of the task being executed as well as the number of employees occupying a job). When communicating these reports to directors, modifications were made to the allocated task times and some discarded / forgotten activities during the documentation phase were added. Tackling this issue and updating the procedures has resulted in more meaningful reports and it was easier to determine the total activities' durations apart from the sense of guesstimates.

Tackling improvements initiatives was another main feature of using SIPOC table. The cycle time reduction was one of the improvements opportunities where “longer time for implementing some functions” was the most requested by management to be improved. Therefore, and based on the design of the SIPOC table, the finance department was able to generate different scenarios to reach the optimum duration of the process of developing the monthly payment-roll for approximately 11'000 employees. This process is composed of different sub-processes such as overtime calculations, penalties, rewards, promotions, unpaid vacations, etc. Thus, the finance department started aggregating data for: pending times, reasons of rejections or rework, frequency, inputs/outputs specifications, and used forms/ tables/ resources then with the aid of process flowcharting and tabulating the processes, the total time for different scenarios was calculated and the best scenario was chosen and implemented.

Other main issue that was raised during implementing the new system was adding new fields to the SIPOC tables based on the feedback from the process owner. The most important one was adding the "Cost" column to the table to identify the cost incurred by each task which could generate very meaningful reports. The second was defining the different customer categories through assigning "EX" code for external customer and "IN" code for internal customer. These codes enabled identifying the different categories of customer and determine the relative importance of each according to different criteria and also helped in setting control points for monitoring and measuring the effectiveness of the process according to the definition of quality as meeting customer requirements.

Finally, usage of the SIPOC table in the finance department was very innovative as an initial structure -as a data collection sheet- with adding more fields -as needed- to collect another explanatory data, such as the well-known 5 M^s (material, method, manpower, machine, measurement) used in the "cause and effect" analysis tool (fish-bone diagram). This case was faced during the study of "delaying the issuance of cheques" problem where 117 cases were studied with the help of the SIPOC table to identify the following parameters and then collecting the associated data: type of contract, type of payment, department concerned for receiving service or product, auditing method, attached documents types, accountant in charge of processing the payment, software used, review method, communication method among departments. In brief, using tabulated formats of the procedures helped in collecting beneficial data for later root cause analysis.

VALIDATING THE NEW SYSTEM

Before validating the new system, it is better to distinguish between the two terms: "verification" and "validation" processes. Verification is the effort done - prior to release - to ensure that the outputs have met the input requirements, while validation is the activity related to ensuring that the resulting product/service is capable of meeting the requirements for the intended use. The validation conditions could be real or simulated. Both verification and validation processes should provide objective evidence of complete fullfillment. Figure (9) explains the above mentioned difference.

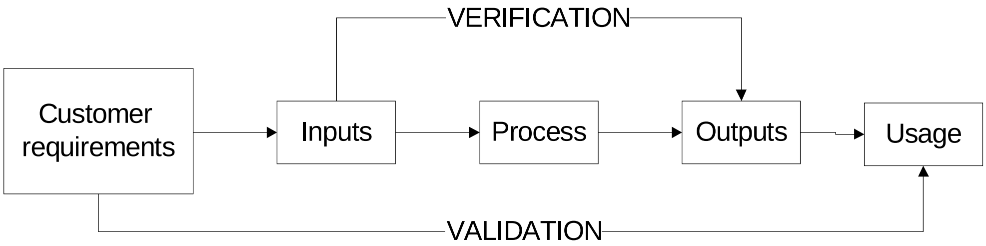

Fig. 9: The Difference between Verification and Validation

The validation of the proposed documentation system was conducted through an independent international 3rd party organization. This organization is one of the certification bodies (or called registrars) which are officially accredited from the accreditation boards in Europe and north America. The experts (lead auditors) of the selected registrar assessed the system within the process of the financial department certification which was conducted in three sequential stages: desk study, pre-assessment, and assessment. The desk study assessment was the most comprehensive stage where all documents of the management system were subjected to complete checking against the requirements of the ISO 9001 documentation requirements. By the end of the validation process, the financial department had a registered quality assurance certificate which is considered the validation's objective evidence of complete fullfillment.

MAIN ADVANTAGES AND OUTCOMES

According to the attained results and the inherent advantages in the suggested system, using SIPOC table has many advantages as can be seen from the implementation stage (paragraph 5). The most important outcome can be derived from cells/columns of the table (figure 5) in the form of reports such as:

i. Job descriptions reports: they could be extracted from columns # 2 (step description), # 3 (responsible), and column # 6 (supplier).
ii. The total duration reports for an activity / process using columns #2 and # 4, i.e. step description and duration. Therefore, the result of this report could lead to reducing wait, transport, or idle times.
iii. Quality requirements reports for both inputs and outputs. (column # 7 "input specification" and column # 10 "output specification"). Thus, reliability could be increased.
iv. Identifying both internal and external customers with their inputs / outputs (column # 9). i.e identifying quality requirements with respect to customers, hence a definite control points can be established and the data collection process can be planned.
v. Cost reports (column # 12) which lead to avoid non-value adding activities.
vi. Resources needed to any process. (column # 2 and # 11) such as softwares, hardwares, facilities, ………etc.

As can be seen, these reports cover the famous trilogy of: cost, time and quality in addition to the characteristics of customers, process owners and resources needed. Moreover, the use of the new system made it easier to answer the assessement questions of European Quality Award (EFQM) related to the process criteria.

The second main advantage of the proposed system is the ease of identifying the measures needed for monitoring the processes performance to determine their effectiveness and efficiency. In other words, the following measures were easy to be established and measured as performance indicators : i. On time delivery. ii. Lead time.
iii. Process costs. iv. Conformity with requirements (inputs and / or outputs).
v. Customer satisfaction (internal or external customers).

In general, key performance indicators could be established on the process level or the activity level according to the performance that needs monitoring. In other words, when it is required to measure the corporate level performance, main process KPIs are required while the activity

KPIs are established when monitoring individual organizational functions performance. Regarding our case study, the organization has defined 15 main processes that can be provided in the field of municipal services “i.e.: planning, designing, building and managing the municipal infrastructure and other related facilities and services“. These main processes had generated 34 corporate level KPIs which were derived from approximately 360 activity-level KPIs. No doubt that the use of the SIPOC table was a very helpful tool for identifying the indicators associated with the financial department which in turn were a part of the overall 34 corporate level key performance indicators.

Note: establishment of a new key performance indicator requires identifying the following items:

- Description of the indicator.
- formula / equation used.
- Measuring frequency.
- Benchmarks.
- Balance score card perspective.
- Source of data.
- Target.
- Approvals.

For more details refer to reference [13].

Finally, the tabular (matrix) format of the new system permits transferring the data to another process presentation formats. One of the wellknown softwares for this purpose is ARIS (Architecture of integrated information systems) that can transform the table format to equivellent graphical models such as: business process design, business process management, and process control process application.

SUMMARY, CONCLUSION & RECOMMENDATION

As previously shown, this paper explain the proposed documentation system which was built mainly to conform with the concept of the process approach which is the core concept adopted by the international organization for standardization (ISO). The paper started by explaining the process model and its components and how that most organizations are claiming implementation of the process approach although they are following the traditional documentation methods. The proposed system is mainly consists of a table called “SIPOC” table which has 4 main headings of: activity, inputs, outputs and controls. The 2nd part of the system is processes flowcharting. The system was successfully implemented in a governmental organization and was validated by an independent entity. The successful implementation is claimed based on the following achieved results:

1. The enormous number of important reports that were extracted such as: job description reports, time, quality requirements, cost, customer requirements and resources.
2. The wide variety of the established key performance indicators (KPIs) that covered both the process and activity levels.
3. Its effective use as a data collection tool which was the main input for the root cause analysis for some problems.

To properly use the proposed system, the authors recommend the following methodology for documentation the management systems:

1. Identify the processes necessary for the effective implementation of the management system using the process selection matrix shown if figure 8.
2. Understand the boundary conditions (interactions between these processes).
3. Document the processes using the “SIPOC” table and process mapping.
4. Establish key performance indicators that best control the process.
5. Measure and monitor the performance and the quality of the outputs.
6. For cases of inability to meet requirements, hypothetically test different scenarios using process mapping and then choose the best option and set the control points with the aid of the SIPOC table.

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TOWARDS AN UNDERSTANDING OF THE CONCEPT OF THE ISLAMIC HOUSE

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ABSTRACT

In Islam, the house is a place to rest, relax the body and mind, and enjoy legitimate worldly delights. Within the house realm we also worship, teach, learn and propagate the message of Islam. The Islamic house is a microcosm of Islamic culture and civilization in that individuals and families bred and nurtured therein constitute the fundamental units of the Islamic community. Central to the standards by which a house may be categorized as Islamic are the holiness and purity of its philosophy, vision and utility, accompanied by convenience, efficiency, safety, awareness of the surroundings, and anything else that Islam reckons as indispensable for living a decent and accountable family life. The overall physical appearance is therefore inferior and matters only when it comes into complete conformity with the said criteria. Thus, designers, planners, builders and final users alike, ought to perceive the house phenomenon as a sheer means, an instrument, a carrier of the spiritual, not a goal itself. Certainly, it is the *tawhidic* (God's oneness) spirit of Islam that serves as the sole force which furnishes the Islamic house with both its essence and identity, integrating itself with and visibly holding sway over a number of indigenous and newly emerging geographical, climatic, social, economic and technical factors and features.

This paper briefly examines some aspects of the phenomenon of the Islamic house, whose significance easily comes to the fore under all circumstances. The themes discussed are: the importance of the house institution in Islam; the factors that make a house Islamic; pragmatism in designing an Islamic house. These themes constitute what could be described as an ideological aspect of the subject in question. As a practical aspect of the paper, a four-steps strategy for designing an Islamic house is proposed.

Keywords: The House, Islam, Muslims, House Design

INTRODUCTION

In this paper, the phenomenon of the Islamic house is viewed as a cultural, environmental, structural and, above all, a religious one, because it is the message of Islam that presided, by and large, over the inception and proliferation of what later became known as the Islamic house. True, there were many other factors at play, however, Islam with the hierarchy of its standards and morals was the sole force that furnished the Islamic house with both its essence and identity, rendering the indigenous geographical, climatic, cultural and other inherited factors and features as rather ancillary. Having said this, extremely little has been written on the subject of integrating Islamic values into the sphere of housing. There are many studies that treat the subject of the Islamic house, but such efforts are confined only to certain cultural, social, environmental or built environment perspectives. It is almost impossible to find a study that deals with the matter from a broader spiritual perspective, integrating the same with and thus infusing a new dimension into the rest of social, cultural, environmental and built environment perspectives. This paper is a small step in that direction.

THE SIGNIFICANCE OF THE HOUSE IN ISLAM

In Islam, the house is a place to rest, relax the body and mind, and enjoy legitimate worldly delights. In the house we are surrounded with privacy, protection and security. Within the house realm we also worship, teach, learn and propagate the message of Islam. The house is one of the fundamental rights that must be enjoyed by every Muslim. Allah, be He exalted, says in the Qur'an: "It is Allah Who made your habitations homes of rest and quiet for you…" (Qur'an 16:80)
The Islamic house is a microcosm of Islamic culture and civilization in that individuals and families bred and nurtured therein constitute the fundamental units of the Islamic *ummah* (community). The house institution, therefore, has a potential to take up the role of an educational and training center able to produce, in concert with other societal establishments, individuals capable of transforming the whole communities they belong to. Thence, the same persons would contribute, somehow or other, their decent share to making this earth a better place for living.
By the same token, if misconstrued and its role perverted, the house has a potential to become a breeding ground for virtually every social disease, which if left unchecked could one day paralyze entire communities and drug them to the bottommost. In this case, the only remedy for the predicament will be the restoration of the position and role of the house in society and with it the position and role of every individual as well as the family institution. On the word of Isma'il Raji al-Faruqi [4] the family is indispensable for the fulfillment of the divine purpose. "Regardless of which is cause and which effect, civilization and the family seem to be destined for rising together and falling together."
It goes without saying that the house institution - as both a concept and sensory reality - plays a foremost role in Islam. Its contributions to advancing and sustaining the wellbeing and interests of Muslim communities are irreplaceable. Such a point has been demonstrated in the most striking way by the lifestyles and civilizational achievements of the first generation of the Muslims in Madinah - the prototype Islamic city-state – which came to pass under the aegis of the Prophet Muhammad and revelation. So important is the subject of the house and housing in Islam that some aspects thereof have been even shored up with the power of the Islamic law (*shari'ah)*. That is predictably the case taking into account the fact that the house is the physical locus of the family without which the total realization of the divine purpose on earth becomes virtually impossible.
Erecting houses in Islam rests within the category of permissibility (*ibahah*) and remains there so long as something does not come about and causes it to infringe some of the divinely prescribed norms and, hence, renders it prohibited. Cases that can make building houses forbidden (*haram*) are building on illegitimate places, building for the purpose of spreading depravity and wickedness, building for a sheer display of might or affluence, building for the purpose of causing harm to others, and the like.
However, if observing the objectives of the *shari'ah* - i.e., preserving religion, self, mental strength, progeny and wealth - is meant foremost to be realized by constructing and activating houses, then the whole matter becomes highly praiseworthy and so correspondingly rewarding. In other words, it becomes an integral part of one's worship paradigm *('ibadah)* whereby one duly discharges a considerable number of duties which have been entrusted to him in his capacity as a vicegerent on earth. Allah Almighty says in the Qur'an that He has created both Jinns and men only that they may worship and serve Him. (Qur'an 51:56)
For Imam Abu Hamid al-Ghazali, having a dwelling falls within the necessary minimum that must be sought by everyone since lack of it causes the people to be displeased with God and even sometimes to deny him [2]. By saying this, al-Ghazali apparently had in mind an Islamic dictum according to which if an obligation (*wajib*) cannot be performed without something the latter then becomes an obligation (*wajib)* itself. In addition, possessing no dwelling causes people to be exposed to many a social and environmental hazard, which goes against yet another Islamic principle established by the Prophet, that is, "There is neither harming nor reciprocating harm". [11] The prohibition covers all forms and degrees of deliberate harm or injury to either humans or the kingdom of flora and fauna. Hence, one of the definitions of the house is that the same is "a haven that surrounds us with privacy, security, refuge and protection from the slings and arrows of life outside it." [8]

The Prophet Muhammad said that one of the former prophets made a war exempting only three categories of people from following him. Exempted were persons who have built their houses but have not yet erected the roof. It was more important that they attend to the unfinished business than to go to the war [12].
Possessing dwelling places on earth is a gift to man from his Lord and Cherisher, which nobody can deny him. Nor can he for any reason deny himself not only this Allah's gift but every other gift and favor given to him. On the contrary, man is asked to enjoy them in many ways so that the effects of Allah's favor and bounty upon him could be seen, for Allah is beautiful and loves beauty [12].

WHAT MAKES A HOUSE ISLAMIC?

An ideal Islamic house is the one whose plan, design, form and function have been inspired primarily by Islam, are permeated with the Islamic spirit, and stand for the embodiment of the Islamic principles and values. Such a house has been perceived and formed in order to facilitate, foster and stimulate its occupants' ceaseless *'ibadah* (worship) activities entrusted to them by God, thus helping them to elevate their status over that of the angels and honorably live up to their reputation as the vicegerent on earth.
Central to the standards by which a house may be categorized as Islamic is the holiness and purity of its philosophy, vision and utility, accompanied by convenience, efficiency, safety, awareness of the surroundings, and anything else that Islam reckons as indispensable for living a decent and accountable family (social) life. The overall physical appearance is therefore inferior and matters only when it comes into complete conformity with the said criteria. The house, thus, ought to be perceived by designers, planners, builders and final users alike as a sheer means, an instrument, a carrier of the spiritual, not a goal itself. The goal is a much nobler and heavenly one.
Viewing the possession of houses as a goal, rather than one of the noble means by which the noblest goals are attained, means that our perception of building, in general, and of the house, in particular, has been adulterated by some damaging *jahiliyyah* (ignorance) elements. In this case, the whole scenario may, in the long run, prove disastrous not only for the future of individuals but also for the future of the Muslim community as a whole. The reason for this is that under some unfavorable circumstances the Islamic idea of the house and its splendid goals will become distorted and misleading, resulting in the people to drift away from purposeful moderation and lose their focus and orientation, in the end becoming liable to warp even the character and role of their very existence on earth. No sooner does this come about than breeding the causes, which the Prophet Muhammad has singled out as responsible for every upcoming cultural and civilizational slump of the Muslims, happens next. The causes highlighted by the Prophet are: exaggerated love of this world and having aversion to death [1]. Verily, the more people fritter away their time, energy and resources on building and exceedingly delighting in their houses, the greater affection do they develop for the upshots of their engagements and this world in general; and the more they are attached to this world, the 'farther' and more detested death and the Hereafter appear.
'The dwellings in which you delight' has been referred to in the Qur'an (9:24) as one of the potential hindrances in Allah's cause, in that man's heart is prone to clinging to it in this world together with wealth and prosperity, commerce, and kith and kin. And if it be that any of these turns out to be a hindrance "…then wait until Allah brings about His decision: and Allah guides not the rebellious." (Qur'an 9:24)
Since Islam accounts for both a worldview and a comprehensive way of life, it draws no distinction between the religious and secular realm along ideological lines. God's words of guidance are bidden to be evenly exalted, adhered to, implemented and made supreme in each and every department of human existence. The word 'Islamic' employed before 'house' thus does not denote a mere cultural phenomenon, philosophy or just another religious conviction, but a genuine faith and its enduring all-inclusive belief and value system. The word 'Islamic' is an adjective delineating a phenomenon vital for human social, psychological, and, of course, spiritual advancement. That phenomenon is the house institution which imbibes and reflects the special qualities inherent in Islam, and whose plan, design, form and function are – to a large

extent - dictated by the latter. In view of that, the notion of the Islamic house encompasses not only its conception and functionality but also – to a lesser degree, though - its arrangement of spaces and some of its structural elements. Religious and secular functions are by no means separable in Islam, and, as such, not in any sector of Islamic built environment either.

If one wanted to understand and appreciate really the phenomenon of the Islamic house, one first and foremost must possess an intimate knowledge of Islam whose major precepts and values the house exemplifies. Next, one ought to disengage himself for a moment and as much as he could from whatever he has formerly perused or has been told about the Islamic house, exerting an effort himself to experience it in its totality as if he is one of its users/inhabitants. One must recognize and feel the spiritual and sensory aura the Islamic house exudes within and without its immediate realm. Not to one or a few of its aspects, and not to a single and static moment of time, should one's comprehension and appreciation of the Islamic house be restricted. Rather, one's thoughts and attention must take in all its aspects and dimensions, honoring in the process its remarkable spiritedness and dynamism conditioned and governed by neither the time nor space factors. Finally, whatever one's approach in studying the Islamic house might be, one should never try to extricate it from the contexts which presided over its commencement, rise, dominance and survival. The Islamic house ought to be viewed as a revolutionary world phenomenon as universal, omnipresent, perpetual and revealing as the standards and values that gave rise to it. The Islamic house was as responsive to the climatic, geographical and cultural requirements as any other architectural expression; nevertheless, it never treated them apart from the exigencies of a higher order. By means of its architects' and structural engineers' outlook on life, their vision, abilities and ingenuity - on the one hand - and by steadfastly combining aesthetic and utilitarian ends - on the other - the Islamic house never appeared to be dissociating man's corporeal, psychic and spiritual needs.

That relying solely on the five senses while studying the phenomenon of the Islamic house would be an inapt method, could be corroborated by the following statement of Abu Hamid al-Ghazali as regards investigating, grasping and experiencing the essence or the fundamental nature of a thing: "The eye perceives the outer and the surface of things, but not their inner essence; moreover, it perceives only their shapes and their forms, not their real nature." [9]

Le Corbusier [10] wrote about the extent architecture can hold sway over our senses, experiences and thoughts: "The Architect, by his arrangement of forms, realizes an order which is a pure creation of his spirit; by forms and shapes he affects our senses to an acute degree and provokes plastic emotions; by the relationships which he creates he wakes profound echoes in us, he gives us the measure of an order which we feel to be in accordance with that of our world, he determines the various movements of our heart and of our understanding; it is then that we experience the sense of beauty."

Humans are not the only creatures that build. Many a creature that we classify low down the hierarchy of the animal kingdom build elaborate structures. However, it has been suggested that it is awareness and imagination that single out humans as superior to other animals in architectural achievement [14]. Whereas other creatures act on the environment instinctively without reasoning or training - exactly as preordained by Allah, the Creator of the universe - man does the same willingly and at his own discretion. Since his actions are preceded with thinking and rationalizing, man clearly demonstrates through architecture - and through every other engagement of his, indeed - his philosophy of himself and life. No sooner does a paradigm shift occur in the latter - no matter how (in)significant - than it sets about to reflect itself proportionally on the very character of the former, thus revealing and immortalizing one's relationship with himself, his peers, other creatures and, of course, his Creator.

Since in every action of his - including building - man tends to express, consciously or otherwise, his outlook on life and on the whole of the universe, Islamic domestic architecture is then nothing but a medium of expression of Islamic doctrine. Although Islam did not prescribe any specific form of the human habitat, yet its norms and ethos served as the chief force in bringing about both the genesis and swift expansion of what later became known as the Islamic house. The unity of Islamic domestic architecture diffused with fluctuating variations throughout the Muslim world is actually a facet of the Muslim ideological unity because of their unique and distinct view of reality, of existence, of space and time, of history, of the *ummah* (community) and of their organic relation thereto. Regardless of how big the apparent variations in terms of Islamic housing might be as dictated by the dissimilar climatic, ecological and cultural factors

dominant in the Muslim lands, yet the housing core is kept always intact and unchanged. Such is the life of the Muslims that they cannot avoid identifying their spiritual disposition with the disposition of the utility of the objects they create and make use of.
Indeed, Islam is a universal way of life which came to raze people's erring living patterns and furnish them with such as are based upon the *tawhidic* paradigm instead. Therefore, how could such a comprehensive religion, world-view and culture – all in one - omit to determine man's habitat, while it determined in a strikingly clear manner the style of clothing, of eating, of sleeping, of socializing, of leisure and recreation, etc.? "Nay, it did; and it even buttressed its influence with the power of Law… ", Isma'il al-Faruqi [5] infers. Perhaps one the best testimonies to this is the fact that the domestic architecture which originated with the advent and assertion of Islam on the world scene never existed before, even though the peoples that later became instrumental in molding and perpetuating its conspicuous identity lived where they were for centuries before embracing Islam and possessed the cultures and civilizations of their own.

PRAGMATISM IN DESIGNING AN ISLAMIC HOUSE

Islam is a complete way of life. Its lessons, and the lessons of the Prophet Muhammad whose primary task was to explain to mankind and put into practice the teachings of Islam, are universal and timeless, in that their source is God's revelation, and God's knowledge is above all the limitations of time and space. It is the nature of Islam that provides humanity with basic rules of morality and guidelines of proper conduct in all spheres of life, including the built environment. Upon such principles and guidelines people can establish laws and regulations in order to regulate their worldly life in accordance with their time, region and needs. Since every age has its own problems and challenges, the solutions deduced from the fundamental principles and permanent values of life have got to be to some extent different. This way, never can the Islamic life paradigm be rendered obsolete or irrelevant. It is precisely because of this that the Prophet Muhammad's teachings on any subject are still alive - and will be forever – holding sway over the life of individuals, families and communities across the globe. Islam is based on essential human nature, which is constant and not subject to change according to time and space. It is the outward forms which change while the fundamental principles, the basic values and the essential human nature together with men's basic needs remain unchanged.
The Muslims of today are bidden to identify the general Islamic guidelines and principles pertaining to housing. Next, they must be fully aware of the implications of the dilemmas and challenges their time and the diverse regions in which they live entail. They cannot be trapped in a historical episode overly romanticizing it and attempting to emulate the housing solutions the Muslims of that particular period successfully evolved. If something was just the thing during a period and in a particular ecological setting the same by no means can be the same in every subsequent period and in different ecological settings. Technological advancements rapidly change; demands of different eras fluctuate, even under the same ecological conditions; climate exigencies must be painstakingly heeded; and, lastly, human psychology also changes with the change of time and space posing a number of exigencies of its own. No house design which served as a solution for an age and place that can be simply 'parachuted' to another age and place without properly adjusting it to its rigorous environmental and socio-cultural requirements. To do that is to betray the dynamic spirit of both the common sense and the perpetual message of Islam. Blind and ignorant imitation and following, even in the sheer religious matters, are categorically rebuked by Islam.
Taking hold of the general Islamic guidelines and principles with reference to designing a house - on the one hand - and studying the needs of different times and situations so that the former can be accurately understood and applied – on the other - the Muslim architects in reality perform a degree of *ijtihad*, i.e., forming an independent opinion or judgment within the framework of an available text. In doing so, if one excels one receives two rewards from God, but if one for whatever reason fails to deliver, after he tried his best, one is bound to receive one reward from God – as propounded by the Prophet Muhammad in one of his traditions [13]. Based on this tradition, in no way can a serious, enlightened, accountable and willing person be a loser, as far as the execution of matters ordained by God is concerned. Verily, this divine

assurance should serve to the Muslim architects and designers as a starting point to look carefully and critically at the state of housing and how houses in the Muslim world are planned and designed, as well as to start contemplating the prospects of finding out much better solutions which will be inspired by and infused with the values of Islam and will be responsive to the exigencies of different times and regions.

At the start, the Muslim architects and planners ought not to be bound by a single historical structural device or solution. The past is to be viewed all the time as such, i.e., the past. It is to be neither excessively venerated or idealized nor completely disregarded. The past must be put in its true perspective with such notions as wisdom, pragmatism and practicality leading the way. In their daunting search for contemporary Islamic housing, the Muslim architects and designers must be driven by a clear principled vision, a free spirit and an insatiable thirst for ingenuity, which must be shrouded in strong determination, self-belief and quest for excellence. However, should some modern structural devices or solutions appear to bear a resemblance – partly or totally - to the ones used in the past, one is not to shy away from reviving them within the existing contexts. History of the Islamic house is not to be looked down at as entirely outmoded and worthless. As we are against blind and ignorant imitation of the past, we are likewise against disengaging ourselves from it completely ignoring the numerous lessons that we can learn therefrom. Indeed, much can be learned from history because the protagonists of any historical episode while solving their problems possessed the same vision and objectives as we do today while solving the problems of our own. On the other hand, however, we have to be extremely mindful and selective as to what exactly to benefit from history and in which areas and how far we are to emulate our predecessors, because most of their problems were the product of the circumstances under which they operated, whereas our problems are the product of the circumstances under which we operate. Hence, seldom can their solutions be ours.

There is no such thing as a standardized Islamic house which can be reproduced anytime and anywhere. If truth be told, there is nothing as such in the whole body of the Islamic built environment. "*Mashrabiyas*", courtyards, bent entrances, inward-looking design, etc., are by no means the exclusive representations of the Islamic house. If viable, a house can be Islamic without those structural solutions. Nor are domes, minarets, praying niches, heavy calligraphy, etc., the exclusive representations of the mosque institution. If need be, a bare structure without those structural solutions is capable very much to serve the projected purpose of the mosque institution. The same goes to every other aspect of the Islamic built environment.

Therefore, the Muslim architects and designers should not hesitate for a single second to unleash their Islamic spirit, desire, imagination and creativity in order to conceive and create such house plans and designs that will be compatible with the requirements of both the religious message and modernity. Undoubtedly, the given solutions will have to vary from one region to another, somewhere more and somewhere less. But the essence of all the possible designs – including those adopted as best solutions in history – will remain one, because of the same worldview and the same religious spirit and foundation that underpin the Muslims' presence and bind all the Muslim peoples regardless of their different geographical locations, cultures and historical appearances. Whatever conception and form are eventually given to such houses, the same is absolutely qualified to be branded as "Islamic". On account of its site, sheer exterior, or association with a historical moment, no house can be more Islamic than others. What matters – let's reiterate again – is function, that a house is imbued with the soul and purity of Islam, and that it stands for an embodiment of the Islamic values and principles, insofar as fulfilling the individual, family and some societal obligations therein is concerned.

A FOUR-STEP STRATEGY FOR DESIGNING AN ISLAMIC HOUSE

The earlier discussion focused on what could be categorized as an ideological aspect of the subject of the phenomenon of the Islamic house. What follows is a rather practical aspect of the theme at hand.

It is an undeniable fact that when one embarks on designing a house, one may look at the process from several angles. From the perspective of what could be described as an Islamization of the house phenomenon, we would like to suggest that the whole process of designing houses goes through the four following steps.

First Step

All the needs and requirements of the users (the Muslims) of a future house are to be identified and meticulously studied. All the big and small issues are to be considered. Although the needs and requirements - i.e., the functions of a house - have to be prioritized yet there is no such thing as an irrelevant need or a requirement. No single member of a household is to be neglected in the process, or his/her needs underestimated. House form is given only after the needs and requirements of the users of a house are correctly understood. A house users' needs and requirements translate into a house function which, in turn, is enveloped by form. Thus, the whole of architecture is sometimes said to be "the will of an epoch translated into space". [15]

The societal role of the house must be given its due as well. Hence, much attention is to be given to the implications of positioning a house vis-à-vis the rest of a neighborhood and the whole of nature, to the concept of neighbors and visitors who are kin and neighbors and visitors who are strangers, and so forth.

House form is a framework for human individual, family and - to an extent - social life activities. Giving a framework to a given set of needs and requirements, so as to enfold and shield them, giving them thus a sense of recognition and a chance for realization, is tantamount to giving a body its clothing: neither too baggy nor too tight will be appropriate, whereas leaving the body naked would mean denying it humanity as well as a chance for self-realization and achievement. Accurate measurements of certain body parts are to be taken first, followed by a process of sewing a dress in full accordance with the taken measurements.

The corollary of this opening step is that houses cannot be planned and designed hastily and haphazardly. Nor can a rigid uniformity be imposed on house users who may possess different needs and requirements. There should never be a perfect type which can go to a mass production line in order to satisfy the housing needs of an area. Unremitting inter and cross-professional studies and research activities thus appear to be inevitable.

Moreover, a degree of public involvement in resolving housing problems must be allowed. Residential architecture is a public art in that the end result always goes back to the public. Architecture in general should never be allowed to become an art of a privileged group, because it affects the life of all people, directly or indirectly.

Iftekhar Uddin Chowdhury [3] wrote on this: "A shelter provides people with functional, social and spiritual needs. The life of an individual and family unfolds in the space within the shelter. Any attempt at formulating housing and space standards should start by recognizing the quality of space that have to be provided in the family home to satisfy these needs." Hence, designing a house is one of the most difficult tasks in the field of architecture. "A proper understanding of the nature of human needs is of crucial importance in the formulation of housing and space standards." [3]

Second Step

Once the exact needs and requirements of the users of a future house - i.e., the functions of a house - have been ascertained, the following step is to find out what Islam has to say about all of them. This cannot be done cursorily, offhandedly or artificially. Rather, it must be done earnestly and meticulously. No subject matter is to be overlooked or taken too lightly, as Islam - which is a comprehensive way of life - hardly ever treats any aspect of human existence as a trivial one. Anything that is either by far or otherwise pertinent to human development Islam considers of paramount importance: from such subjects as entering and leaving toilets to such as are related to human interaction with nature and the Creator.

This is a critical step, indeed, because houses are erected in order to function and meet a purpose, not in order for their form to be admired by neighbors, passers-by or their owners. There must be a common ground between the character and function of an Islamic house and between the character and spiritual disposition of its planner(s) and designer(s). Islamic house designs are expected to facilitate, foster and stimulate the Muslims' *'ibadah* (worship) daily practices entrusted to them by their Lord and Cherisher.

The corollary of this step is that the Muslim architects, and all the other professionals in the field of built environment, must enhance considerably their knowledge of Islam: its *shari'ah* (Law) and worldview (*aqidah*). This may appear as a daunting task to many, however, needless to say that it is incumbent upon every Muslim - male and female - to know the rulings of Islam pertaining to the obligations and teachings they have to adhere to in their life. Every Muslim must know what is lawful and unlawful regarding matters of daily life like purification of body, praying, fasting, eating, drinking, dressing, adornment, household, work, family, community, nature, and so on. It is an individual duty (*fard 'ayn*) upon each and every Muslim to acquire some basic knowledge about all these matters, regardless of their professional fields of interests. The Prophet Muhammad has said that seeking knowledge is every Muslim's never-ending responsibility: from the cradle to the grave [11].

While Islamizing the function of a future house, giving it eventually a form so as to enfold and shield the function, the Muslim planners and designers can draw on their own familiarity with the rulings of Islam, provided the same is adequate. Otherwise, trustworthy religious scholars, who are both qualified and broad-minded, should be consulted as many times as needed. Here like in the case of the first step, unremitting inter and cross-professional studies and research activities appear to be inevitable. This is bound to lead gradually to narrowing down the glaring gap separating the religious scholars and their fields of interest from the secular ones and their own fields of interest. This way, every scholar will become aware as to his/her role in society and his/her obligations toward society, nature and God. Certainly, the religious scholars will have to widen their interests and concerns, becoming what they are actually always meant to be: the guardians of societies. But to secure that accolade they ought to reevaluate themselves and their undertakings, striving to be a more practical, approachable, people-friendly, and less dogmatic and idealistic lot. Whereas the secular scholars will have to think of Islamizing their knowledge, wherever there is a conflict of interests and as much as possible, realigning their scientific goals and aspirations with the goals and aspirations of the Muslim community to which they belong.

Should this scheme become a reality, a new contemporary genre of Islamic residential architecture will then unavoidably emerge. As an expression of a new emerging culture and psychology, a house will be then classified as Islamic only because of its noteworthy role and functions in making communities - in concert with other societal establishments - better places for living, and not because of some of its blindly reproduced traditional form components which, as a result, sometimes appear as obsolete due to rapid technological advancements, at other times as a misfit because the same form components served the purpose of a different time and space, and yet at other times as an extravagant adornment unessential to a house's structure and serviceability.

It is certainly because of this that Sinan, the architect of Suleyman the Magnificent and the Ottoman golden age, said that architecture is the most difficult profession, and he who would practice it correctly and justly must, above all things, be pious [7].

Third Step

When the function of a future house has been ascertained, and when the same has been thoroughly studied from the standpoint of the Islamic values and teachings, the next step is to make sure that a house goes well with the environment, geography, culture and technology of an area where a new house will be planted. When planning and designing not only houses but also each and every private or communal building, the Muslim architect and structural engineer must, first and foremost, be concerned about how the end result of their efforts will fare when juxtaposed with the existing universal setting in terms of both function and outward appearance: will it go well with it, or will it appear as if something of a misfit, oddity, or even offensiveness. Like in the case of the first two steps, here too some inter and cross-professional studies and research activities are needed.

The natural environment is simultaneously an obstruction and a help, and architects seek both to invite its aid and to drive back its attacks. If rightly conceived and seriously pondered, the placement and form of houses in relation to their sites with arrangement of their axes and spaces may well be turned into a device for controlling natural light, ventilation, heating, cooling, insulation, acoustics, etc. In his book "Natural Energy and Vernacular Architecture" Hassan

Fathy [6] wrote: “The built environment produces changes in the microclimate. The configuration of buildings, their orientations, and their arrangement in space create a specific microclimate for each side. To this must be added the building materials, surface textures and colors of exposed surfaces of the buildings, and the design of open spaces, such as streets, courtyards, gardens, and squares. These man-made elements interact with the natural microclimate to determine the factors affecting comfort in the built environment: light, heat, wind, and humidity.”
However, if the specific requirements of a climate, geography, culture and technology are not met then a new house comes into view rather like an alien element if one critically looked at it against the backdrop of its surroundings. The situation becomes even worse if a house is ‘parachuted’ from one context to another which is totally different from the former. Verily, little genuine comfort is such a house able to generate to its users whose character has nothing or, at best, has very little in common with the character of the house. Such a house can appear even as an extremely funny thing to an insightful observer. ‘Parachuting’ a house plan and design from one context to the other different one is like asking a person to move from one place to another which has different climate, geography and culture, without allowing him to get prepared for all the necessary acclimatization processes that await him. Surely, the person will suffer greatly should he insist on changing nothing. He will get much attention from the locals but for all the wrong reasons. Seldom could anyone survive under the circumstances.

Fourth Step

After the completion of the first three steps, the next step for an architect is to embark on designing a house. He should be bound only by the findings he made at the first three stages. All the other factors are to be relegated to a nonessential as they are all irrelevant. The Muslim architects must possess a free and ingenious spirit ready to be unleashed whenever needed. In the process of designing an Islamic house history and ancient traditions can be made a point of reference, but only if the guidelines which we have explained earlier are duly adhered to. Even some ‘borrowings’ can be made from other cultures provided the same goes well with the ideals that underpins the phenomenon of the Islamic house as both a concept and sensory experience. After all, materials and all the ‘earthly’ ingredients are sheer matter; what counts is the spirit of the heavenly given dispensation infused into the structural components.
In the end, whatever a designed house may look like yet it is an Islamic house. A house is Islamic owing to the processes that accompanied its conception as well as the evolution of whatever form that is eventually given to it. Houses may vary in plan and design. One must not be obsessed by a certain design or certain structural devices and solutions. Such thing as inflexible uniformity has no place in Islamic domestic architecture and in Islamic architecture in general, because it curtails utility and stifles innovation and creativity.
Having been successfully developed, a house type - if deemed suitable - can be somewhat standardized and mass produced in a region. The same type can be further exported to other regions, after it has been modified accordingly in order to meet the necessary specifications of the new regions.

CONCLUSION

It could be concluded that Islam as a comprehensive lifestyle influenced the planning and designing of the dwellings of its adherents. As a matter of fact, not only did Islam merely influence the systems of building houses, but it also laid a solid foundation, in some instances in form of laws, for what became known as the Islamic house. Hence, only such houses as embody through their essence and diverse functions the values and teachings of Islam could be categorized as “Islamic”. All this, however, in no way implies that the creativity and design freedom of Muslim architects and designers are stifled under the charge of Islam. On the contrary, they are very much stirred and encouraged to thrive with the only difference that certain divine precepts now govern them, lest people’s imagination and enthusiasm at some point should become disoriented and deceptive and so perilous to the well-being of men.
This is so because Islam is based on essential human nature that will never change. Moreover, Islam provides people with basic rules of morality and guidelines of right behavior. Upon such

rules and guidelines people can institute laws and regulations in order to regulate their worldly life in line with the requirements of their time, region and needs. Since every age has its own problems and challenges, the solutions deduced from the fundamental standards and permanent values of life are bound to be somewhat different. The numerous implications of this life's fundamental principle have been clearly suggested in the paper while discussing the quintessence of the Islamic house and how open-minded Muslim architects and designers should be when embarking on planning and designing Islamic houses.

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EVALUATION OF ENERGY CONSUMPTIONS FOR OFFICE BUILDING. (Case Study of Reconditioned Building)

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ABSTRACT

Presented paper focuses on the minimum requirements of HVAC (Heating, Ventilating and Air-Conditioning) central systems that used for specified buildings, which affected by application, utilization, and occupation changes. Investigation is devoted to the analysis and validation of both field case studies and simulation tasks. This is accomplished by using two approaches. First approach is directly related to the survey data collected from fields and in-operation systems, these data are either collected manually or from the BMS-systems then they compared with the output results of the Second approach, which is the numerical simulation methodologies utilizing different Hourly Analysis Simulation Programs [HASP].The present work dealt with private multi-story (ten floors) office building located in Cairo. The building is centrally air-conditioned using eight air-cooled 20 T.R direct expansion packaged units feeding each floor individually and each operates on weekly-programmed bases. The footprint area of the building is about 330-m2/floor including services. The building had three exposed façade facing north, east and south exposures on which the (Widow Wall Ratio) WWR at north is about 75% (frameless reflected double structure glazing) and the WWR at east and west is about 25% using standard double glass aluminum windows. Remain exposure is totally coupled with other building. The lighting intensity inside building is totally controlled by dimming systems and actuated from the BMS on which the lighting intensity ranged from 15 to 45 W/m2. Finally the total power consumptions inside different spaces are ranged from 45 to 65 W/m2. Building simplifications and occupancies, lighting, appliances, visitors and operation period's schedules were prepared then it fed to computer based on Cairo Weather data and executed using HASP programs. Collected data were tabulated and compared with field measured collected data.

Keywords: Energy Simulation, Energy efficiency, HVAC Modeling, HVAC System Design.

INTRODUCTION

In many of the developing countries buildings owners convert the use of part of their buildings from residential applications into commercial applications especially when their buildings are located in a luxury zones. Such action makes it difficult to select the code compliance route to be followed and as example is given in this paper. This investigation is a part of series of case-studies that were carried out to set, assist and evaluate the minimum requirements for attaining energy-efficient design of air-conditioning installations in such mentioned buildings for the new Egyptian Energy Efficiency Building Code **[EEEBC]** applicable for New Commercial Buildings. Moreover these investigations are devoted to the analysis and validation of the simulation tasks[1], which was related to collected data form field works and in-operation projects to assist and optimizing energy efficiency for office buildings. Previous work was reported, by, among others, Khalil [2, 3] , Medhat [4,5] and Kameel et al [6].

Historical Background

This particular case study dealt with ten years old multi-story building located in a very luxury and highly crowded zone in Cairo where outdoor air pollution is sensible. The building has a footprint area of 900-m^2/floor. The building was built similar to common residential buildings including two wings, the right wing includes two sold apartments with total area 570-m^2 and the left wing includes one rental apartment with total area 330-m^2 as indicated on Figure 1.

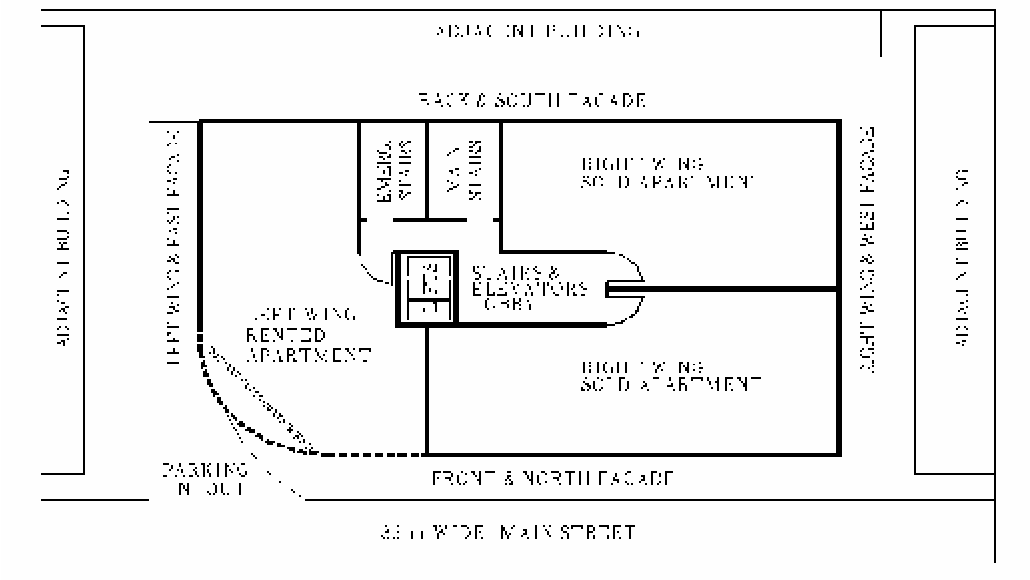

Fig. 1: Original Design Layout of the Case-Study Building

Since 1994 and till 1996 the building was air-conditioned using standard split unit room air conditioners that were located on the side and back façades. At the end of 1996 the building owner separated and split the left wing vertically from the right wing by continuous brick walls. This action split and physically separated all rented apartments vertically from the sold apartments. Each individual floor on the left wing was air-conditioned using individual air-cooled direct expansion self-contained packaged unit. Each unit was installed on external frames outside the building envelop at the back façade of left wing as shown in Figure 2.

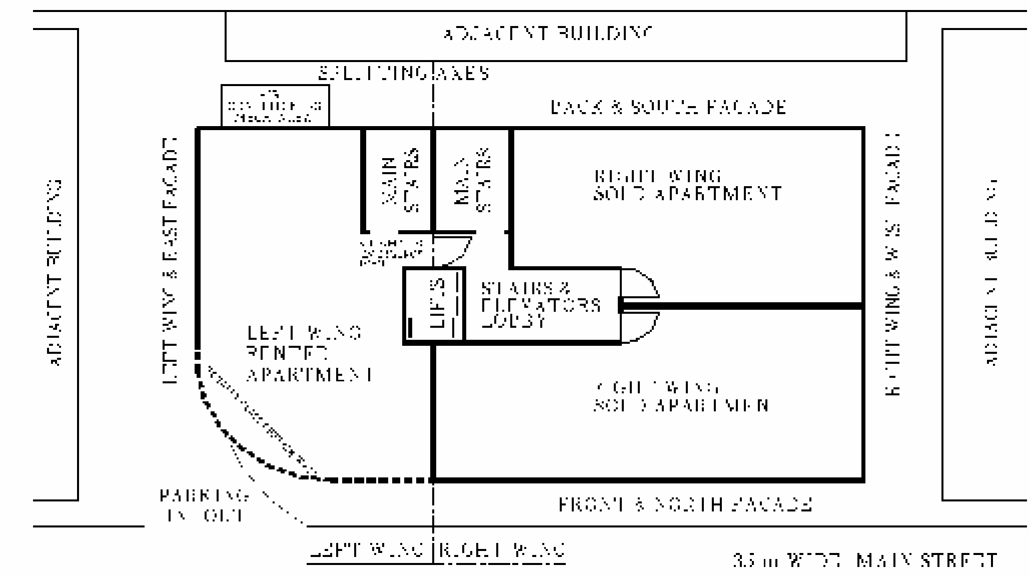

Fig. 2: Modified Layout of the Case-Study Building

The new constructed luxurious and rented apartments were used for foreign visitors as condominiums. At the end of 1998 it was found that different companies, clinics, private offices, and also residential occupy most of these luxury apartments. Finally at the end of 2002 an international local company bought this left wing completely (from basement to roof) from the owner and redesigned the interiors to meet the requirements of its headquarter. Till now all left wing is totally split from the right wing and centrally air-conditioned using air-cooled packaged system that, utilize constant speed fans and controlled via BMS-system.

Design Criteria and Parameters

The building has ten-floor levels including basement, ground, first, second, third, fourth, fifth, sixth, seventh and roof levels with three exposed façades facing north, east and south exposures on which the WWR at north is about 75% (reflected Bronze double structure glazing, curtain wall) and the WWR at south and east exposures are about 25% using standard double glass windows. Building roof is thermally insulated by 0.05-m polystyrene. Design criteria and construction drawings and documents of each phases changes on the building (especially the left wing) were arranged by the design consultants and tabulated as follows:

Parameter	1-st Case Common Residential	2-nd Case Luxury Residential	3-rd Case Individual Office	4-th Case Headquarter Office
System Criteria				
Indoor, DBT, Deg.C	25-27	25-27	22-25	20-24
Indoor RH%	50-55	50-55	45-50	40-50
Outdoor, DBT, Deg.C	37-40	37-40	38-41	38-41
Outdoot WBT, Deg.C	24-26	24-26	24-26	24-26
Number of Occupants	42-50	55-60	105-115	85-100
Fresh Air Per Person, L/s	8-10	8-10	9-11	10-15
Type of Cooling System	Split	Package	Package	Package
Actual Cooling Capacity kW	7*44.65	7*53.84	7*62.00	7*68.60
Applied Cooling Capacity kW	7*45.00	7*62.00	7*62.00	7*70.00
Building Power Consumptions				
HVAC Control System	Elec	Elec./Sch	Electronic	BMS
Zone Cooling Load, kW	127.75	162.19	183.68	198.66
Fresh Air Cooling Load, kW	7.70	10.15	15.82	22.96
Cooling System Fans, kW	9.80	35.00	35.00	55.00
Parking Ventilation System,kW	6.00	6.00	6.00	6.00
Lighting Loads, kW	59.57	59.57	73.71	86.24
Advertising Banners Power, kW	4.20	4.20	14.00	16.80
Domestic Utilities Power, kW	21.00	23.80	10.92	9.59
Equipment Power, kW	10.50	11.69	74.34	92.35
Other Miscellaneous Power, kW	1.54	2.17	8.29	10.43

While the actual monthly power consumptions were collected from the electricity bills of each of the apartments as till now each floor (apartment) at left wing has its own electric meter. While the central air conditioning equipment has one electric feed riser with one meter. Costs of power consumptions related to central air conditioning systems are divided on all floors according to the superimposed in-service hour meter that connected to each packaged unit. The in-service HVAC-System is controlled by BMS.

Occupation levels, expected schedules, and application types of each space were gained from the building security people and according to the annual contracts with the owner.
Now the lighting intensity inside building is totally controlled by dimming systems and actuated from the BMS on which the lighting intensity ranged from 10-to-40 W/m$^{2.}$ While different power consumptions inside spaces are ranged from 45-to-65 W/m^2.

HVAC-System Simulation Design

Building simplifications and occupancies, lighting, appliances, visitors and operation period's actual schedules were carried out in detailed manners for each construction phases then it fed to computer based on Cairo Weather data and executed using E20-II-HAP programs [6, 7]. Collected data were tabulated from both programs in a convenient way. Obtained data that were manually collected from BMS-system arranged and compared with the numerical simulation methodologies utilizing the E20-II- HAP software programs [6, 7]. While it was found that, the case-study original designs were carried out using E20-II-HAP Packaged software.
Out door design conditions are hourly base inputted to programs with maximum dry-bulb temperature of 42.2 ^{0}C with maximum wet bulb temperature of 26 $^{0.}$C. Indoor design conditions are ranged from 21-Deg.C to 24^0 C with RH 45% -to- RH 50% for occupied spaces and from 24^{0}C to 28 ^{0}C with RH 30% -to- RH 60% for corridors, elevator battery, stores and services zones. While parking area and all enclosed electro-mechanical services use mechanical Push-Pull ventilation system. Ventilation and exhaust rates comply with ANSI/ASHRAE Standard 62-1999[8], ventilation for acceptable indoor air quality. Some assumptions were considered for air distribution systems for zones that have special process to be individually treated and controlled, while air leakage limits on duct works were classified to be match with the equation:
L / s / m^2 = 0.009 x [P^0.65]
Where it's applicable for low static pressure (P) ducts network [1] (Within *700 Pa –to- 800 Pa*).

Cooling and Heating Thermal Load calculations for the purpose of sizing system and equipment were carried out using Cooling loads temperature difference [CLTD] method with the aid of E20-II- HAP software and according to internationally recognized procedure and methods. Based on hourly analysis concepts thermal load calculations shows that the maximum cooling capacity of the building is 490 K.W (139.36 T.R) without any reserve (stand-by) capacity.
Many computer runs were executed on Visual-DOE-3 software program[7] using different unit's manufacturers and types such as:

- Air-cooled 2-Circuits-2-Reciprocating Compressors,
- Air-cooled 2-Circuits-2-Scroll-compressors,
- Water-cooled 2-Circuits-2-Reciprocating Compressors,
- Water-cooled 2-circuits-2-Scroll Compressors.

The total fan motor power required for both of constant air volume CAV were selected to the nearest standard motors according to best efficiency of fan selection which indicated a power in range of 1.52-to- 2.5 watts for each required liter per second.

RESULTS AND DISCUSSIONS.

This paper is devised to evaluate the EEEBC design parameters and control criteria's of air-conditioning installations and to assist to achieve the minimum acceptable efficiencies for energy-efficient designs. Although the HVAC systems plays a vital role in Cairo and also contributes to a significant portion in total building energy consumption, till now requirements on this aspects varied widely from a project to another one that has similar characteristics and applications.

Most of the utilized air cooled standard packaged units in Egypt are typically locally manufactured or imported and are required as per code of practice to be certified from International laboratories or Certifying Institutes such as ARI, UL, or TUV. These certifications are related to a certain operating conditions comparable to foreign or different outdoor operating conditions. This means that even at the very early stages of equipment selection the actual efficiencies are lower than the nominal efficiencies [1,7]. Moreover, this case study dealt with the effects of outdoor fresh air quantities especially in Cairo, with the presence of pollution, these fresh air rates contributes strongly in the cooling capacities demand leading to sensible increase in the HVAC power consumption especially when feeding fresh air is related to manual adjustment without any logic set points or automatic controlling. For these reasons local designers are recommended to incorporate into the designs of the HVAC installations all appropriate requirements for outdoor air in accordance with the relevant local Codes and recommendations, international limits and any other relevant health requirements.

The energy consumption in a commercial building of the type described above would have the itemized utilization as indicated in Figure 3 through Figure 6. The Air conditioning service would require almost 50% of the energy; this was also in turn distributed to individual equipment/systems as shown in Figure 3 through Figure 6.

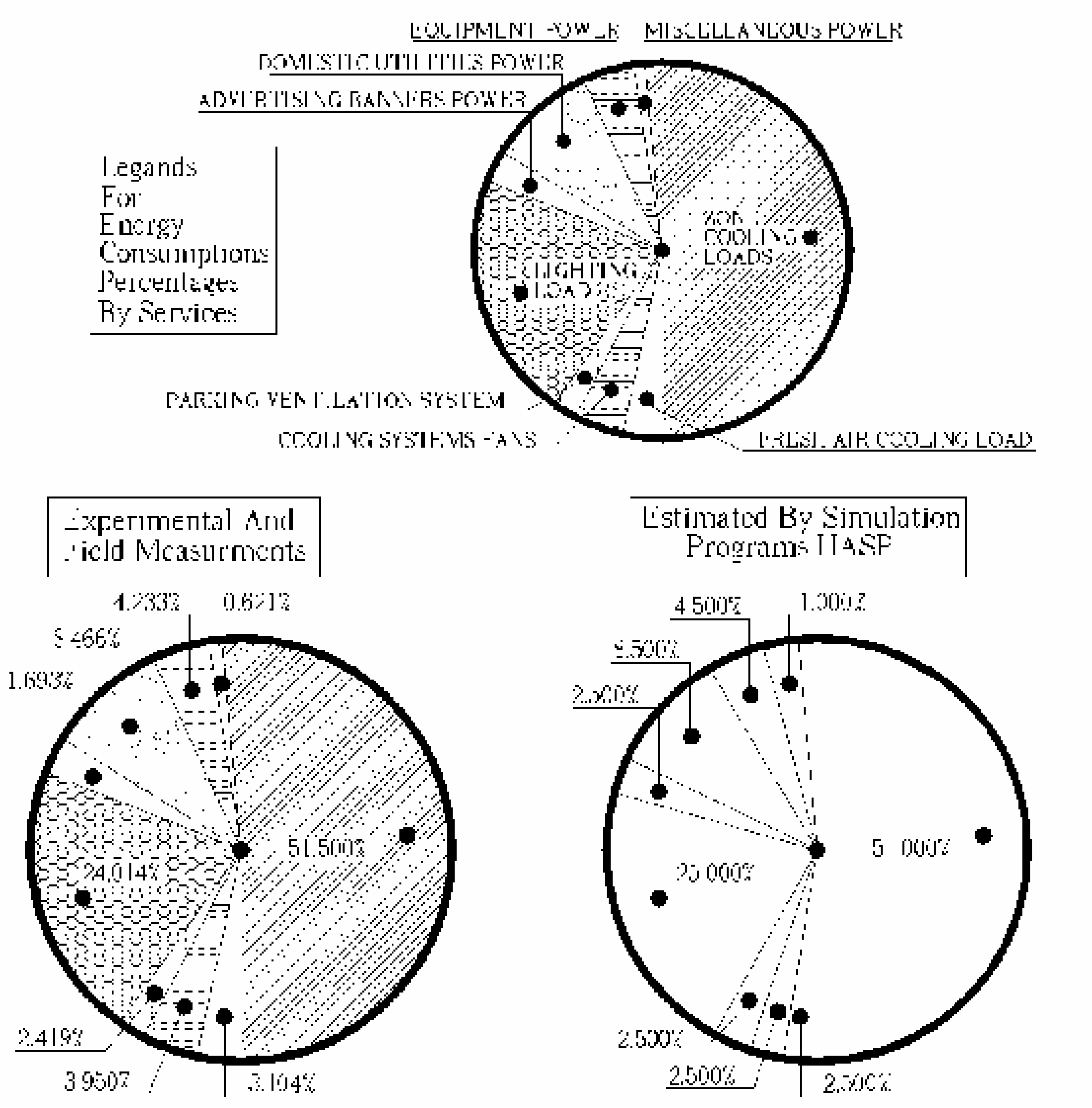

Fig. 3: Legend of Energy Distributions and Building Energy Consumption by Service First Case Left Wing (Common Residential)

When comparing results gained from simulation programs presented at the right hand side on Figure 3 through Figure 6 with the collected data. "Field measurements presented on the left hand side in the same Figures". Its clear that there are major changes in power distributions between first and second cases compared with the third and fourth cases especially for equipment power which is increased sensibly while the air conditioning system appear as deduced sensibly but when evaluate its percentage related to the total power consumption value it is actually increased in value. Also the changes in air conditioning power consumption percentages between simulated and measuring data for fourth case shows that there are still lack of information about the actual actions and operating schedules stored in the BMS systems compared with the fed data to computer simulating programs.

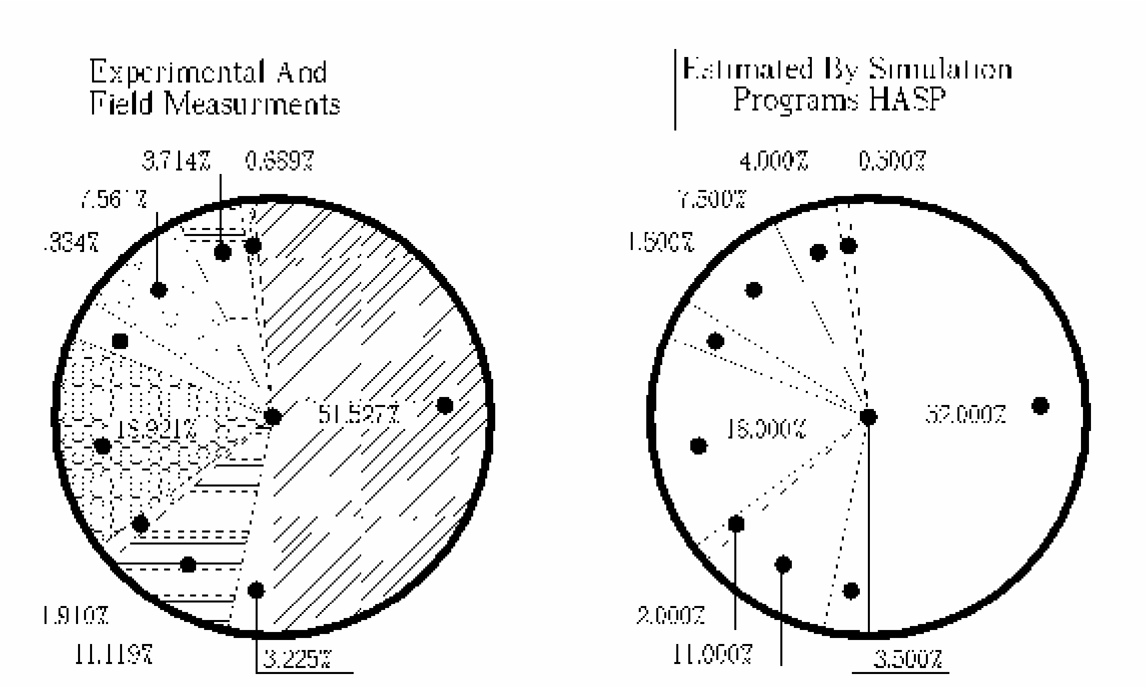

Fig. 4: Building Energy Consumption by Service
Second Case Left Wing (Luxury Residential)

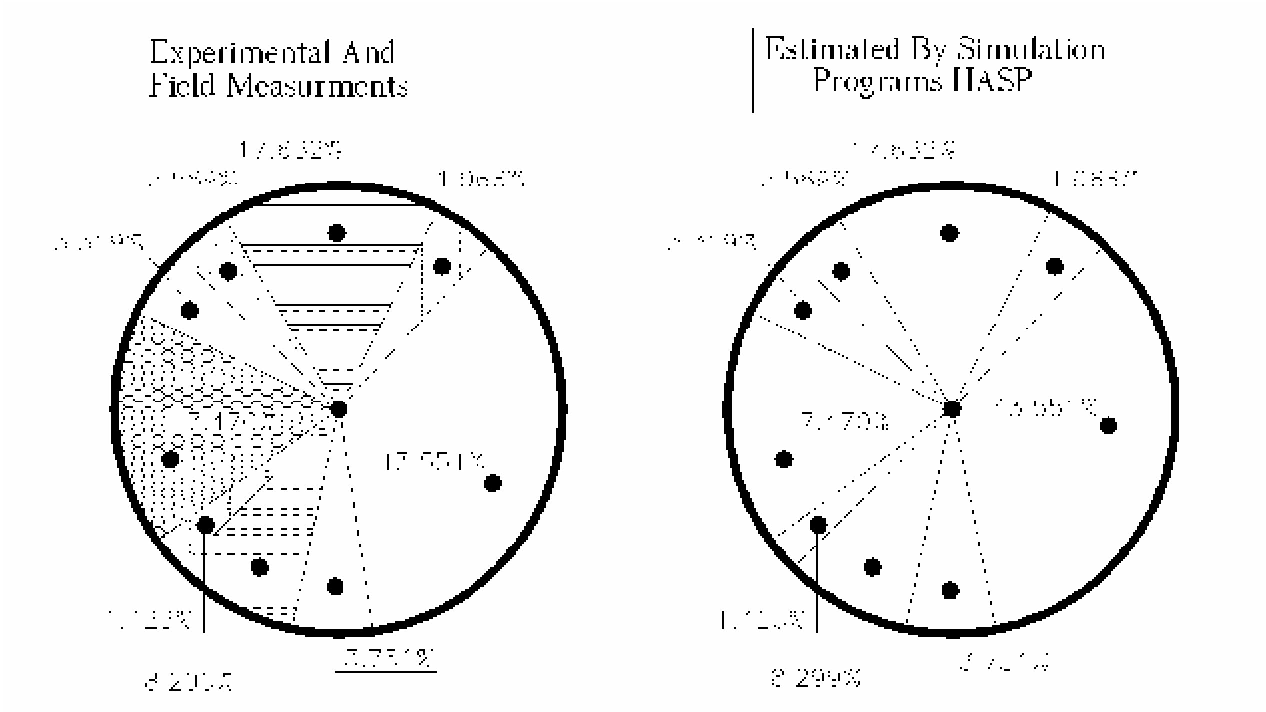

Fig. 5: Building Energy Consumption by Service
Third Case Left Wing (Individual Offices and Residential)

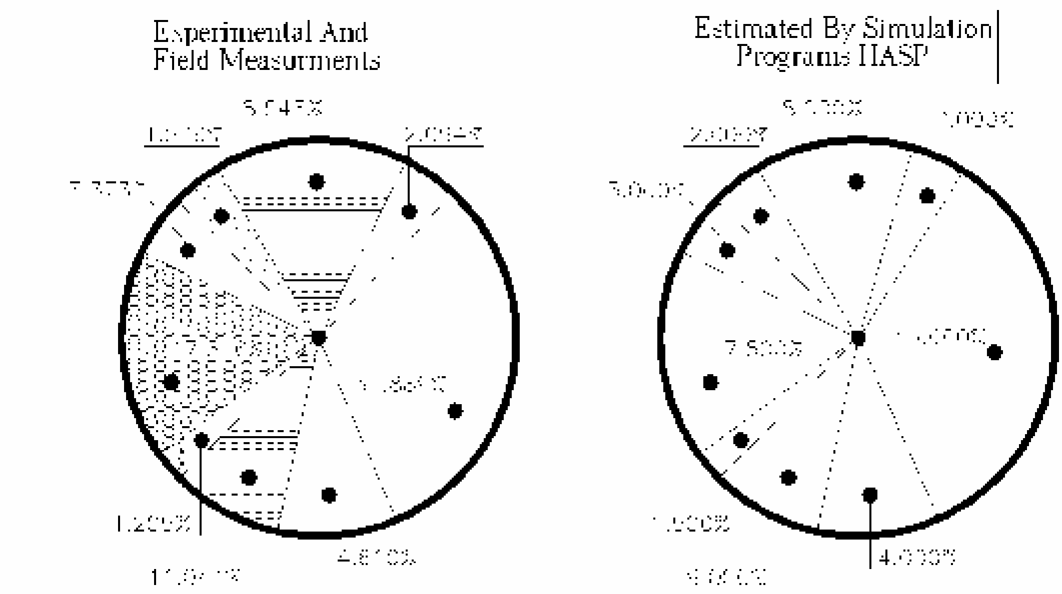

Fig. 6: Building Energy Consumption by Service
Fourth Case Left Wing (Headquarters Office Building)

Comments

On Figures 3, 4. 5, & 6 for Field Measurements

Total Power Consumption at First Case	248.060 K.W		☞ Figure 3
Total Power Consumption at Second Case	313.770 K.W	With 26.390% Increase	☞ Figure 4
Total Power Consumption at Third Case	421.760 K.W	With 70.020% Increase	☞ Figure 5
Total Power Consumption at Fourth Case	495.030 K.W	With 100.760% Increase	☞ Figure 6
Power Consumption Per Unit Area at First Case	117.00	Watts Per Square Meters	☞ Figure 3
Power Consumption Per Unit Area at Second Case	145.50	Watts Per Square Meters	☞ Figure 4
Power Consumption Per Unit Area at Third Case	199.00	Watts Per Square Meters	☞ Figure 5
Power Consumption Per Unit Area at Fourth Case	235.00	Watts Per Square Meters	☞ Figure 6

On Figures 3, 4. 5. & 6 for Simulation Program

Total Power Consumption at First Case	240.645 K.W	☞ Figure 3
Total Power Consumption at Second Case	328.246 K.W	☞ Figure 4
Total Power Consumption at Third Case	425.110 K.W	☞ Figure 5
Total Power Consumption at Fourth Case	485.795 K.W	☞ Figure 6

SUMMARY AND CONCLUSIONS

This paper addresses the factors that should be taken into consideration when investigating the building energy code compliance when they are converted from nearly residential to mixed or commercial within the same footprint .The main conclusions of this investigation can be summarized as:

- During normal operations criteria, there is a good agreement in annual energy consumption behaviors and curve shapes between numerically simulated data and the experimentally collected data.
- While the predicted energy demands in some tasks are 2% to 3.5% lower than the actual measured demands; according the actual operations of the energy consumption and behaviors are slightly offsets towards high-expected limits than the predicted consumption. These deviations may be due to the actual reduction effects on EER of HVAC equipment and continuous operation during part loads for long periods and also the ambient effects on the total performance of equipment. In addition to the previous main two factors the presence of thermal cooling loads required for fresh air, which in this case study was constant and fixed all the times in programs.
- One of the main conclusions that were found during this investigation is that: during survey of similar project and case studies there were a great deviations in most of the characteristics of the field-measured data and also in the results of simulation programs. These cases made conflicts with the proper judgments of different systems.

From previous investigations on office buildings in Cairo and comparing them with similar buildings in Alexandria, there are a lot of negotiable recommendations and implemented requirements that shall be revised, adopted carefully and applied in the EEEBC codes based on the practical investigation only and field collected data according to exact situation in EGYPT. These recommendations are presented as follows,

1- Outside air control during non-occupations shall be considered by providing motorized or gravity dampers to shutoff or reduction down to 10% of fresh air quantities during periods of non-use.

2- Control system shall be equipped with automatic control during Off-Hours capable of accomplishing a reduction of energy use through equipment shutdown, or increase in the set point temperatures especially for air handling units of the packaged units.

3- It is important during preliminary design phases to evaluate the actual EER of all anticipated utilized equipment according to the actual weather in EGYPT and introduce the estimated expected connected power and energy demands.

4- It's recommended strongly in Cairo to utilize water-cooled packaged systems with stringent water treatment systems and high efficient filters against air-cooled systems to reduce the major part of total power consumptions.

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