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ANALYSIS OF THE MECHANICAL PROPERTIES OF MORTAR MADE WITH BLAST FURNACE SLAG AND CALCINED CLAY AGGREGATES

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ABSTRACT

The main objective of this research is to investigate the possibility to substitute partially or completely the natural fine aggregate by the blast furnace waste (slag fine aggregate) and waste bricks (calcined clay fine aggregate) in relation to the weight of the dune fine aggregate at different proportions (0, 25, 50, 75 and 100%). The effects of blast furnace slag aggregates (slag fine aggregate) and waste bricks (calcined clay fine aggregate) on the density, workability and mechanical properties of the mortars were analysed. In the same way for this study, the physical and chemical properties of dune fine aggregate, blast furnace waste (slag fine aggregate), waste bricks (calcined clay fine aggregate) and cement were investigated. After crushing, the artificial fine aggregate (blast furnace slag and calcined clay fine aggregate) was sifted in order to use it as fine aggregate. The results obtained show that at a mass of slag fine aggregate and calcined clay fine aggregate 25% in relation to the weight of the natural fine aggregate, the mortar tested has an acceptable mechanical strength. The reuse of this recycled material (blast furnace waste and waste bricks) in the industry would contribute to the protection of the environment.

Keywords: Blast furnace slag, calcined clay, fine aggregates, mechanical properties.

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INTRODUCTION

Fine aggregates are inert granular materials used for the manufacture of the mortar or concrete. For a good mortar mix, fine aggregates need to be clean, hard, strong particles free of absorbed chemicals and other fine materials that could cause the deterioration of mortar. Unfortunately, the majority of the natural sands used (rolled sands : sand of river, dune sand or sand of sea and crushed fine aggregates) are selected for reasons of the price and the availability [1].
The environmental impact of the production of the raw ingredients of mortar and concrete (such as cement and fine and coarse aggregates) is considerable. The scale of the problem makes it prudent to investigate other sources of raw materials in order to reduce the consumption of energy and available natural resources [2].
The demand for aggregates knows, a considerable growth in connection with the development of construction in Algeria. To surmount it, it is be necessary to ensure a rational exploitation of the artificial aggregates available to the country by a valorization of mineral waste available : blast furnace slag, calcined clay, etc...
The quality of aggregates strongly influences mortar's freshly mixed, hardened properties, mixture proportions and economy. Consequently, selection of aggregates is an important process. Although some variation in aggregate properties is expected, characteristics that are

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considered when selecting aggregate include: particle shape, surface texture, abrasion, unit weights, voids, absorption and surface moisture [3].
The Algerian mineral waste industry (slag, bricks, tiles, ceramics, etc ….) has particular problems with its very high level of mineral waste who remains without being to exploit until now.
Mineral waste (blast furnace slag and waste bricks) represents residue that could be used with minimal processing, largely as construction material, low value industrial mineral [4].
The Use of the artificial resources (waste industrials) such as the slag and waste bricks (calcined clay) makes it possible to increase the manufacture of building materials, to limit the use of natural aggregates and to value the waste products and being able to find valuable solution of the protection of the environmental [5].
The purpose of this paper is to analyze the influence of partial or complete substitution of natural aggregates (dune sand) by artificial aggregates (slag fine aggregate and calcined clay fine aggregate) at different proportions (0, 25, 50, 75 and 100%) on the mechanical response of mortar.

CHARACTERISTICS OF USED MATERIALS

Natural sand (Fine aggregates)

The sand's equivalent measured by the French AFNOR standard NF P18 shows that the dune fine aggregate 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. This natural sand was taken from M'sila region. The absolute density and porosity were 2,56 and 35,94%, respectively. The sand equivalent value (sight/test) was 76/77.

Artificial fine aggregate (Waste bricks and Blast furnace slag)

In this study the calcined clay (artificial fine aggregate) used is waste bricks obtained by calcination of the clay to 850°C and used in the proportions of 0, 25, 50, 75 and 100% by the mass (weight) of natural sand to study its effect on mortar properties. The waste bricks before their use as artificial fine aggregates were crushed and to sift. The absolute density, apparent density and porosity were 2,32, 1,3 and 43,96%, respectively. The water content value was 2,35%. The blast furnace slag used is an industrial by-product from iron and steel industry (Metallurgic unit of El-Hadjar-Annaba). It is used like a artificial fine aggregates for the mortar. The absolute density and porosity were 2,44 and 38,68%, respectively. The water content value was 2,14%.

Cements

The Portland cement (CEM II) with mineral addition was used in this experimental study. The absolute density, apparent density and porosity were 3,1, 1,8 and 41,93%, respectively. The Blaine specific surface area (fineness) was 3424 cm^2/g. The finenesses (specific surface area) of the cement with mineral admixture studied was determined by Air Permeability Apparatus. The Chemical composition of cement, clay sand, slag sand and dune sand used is shown in Table 1.

Table 1: Chemical Composition of Cement, Clay Fine Aggregate, Slag Fine Aggregate and Dune Fine Aggregate Used

Constituents %	SiO_2 (%)	Al_2O_3 (%)	Fe_2O_3 (%)	CaO (%)	MgO (%)	K_2O (%)	Na_2O (%)	SO_3 (%)	LOI* (%)
Cement	22,51	5,76	3,52	57,58	1,92	0,38	0,07	1,77	5,67
Clay fine aggregate	63,65	10,32	5,37	11,54	2,11	1,44	0,40	2,35	2,82
Slag fine aggregate	38,24	7,12	2,55	34,92	4,37	0,65	0,43	1,35	1,68
Dune fine aggregate	87,85	0,98	0,69	6,12	0,19	0,35	0,01	0,04	2,62

(*Loss on ignition)

The chemical composition of the cement, clay fine aggregate, slag fine aggregate and dune fine aggregate used in this research have been determined by the testing method "X-ray Fluorescence Spectrometry (XRF)".

MECHANICAL TESTS (Flexural and compressive strengths)

The mortars samples were subjected to flexural and compressive mechanical tests. Mechanical strength was determined at 7, 14 and 28 days on 4 x 4 x 16 cm prisms specimens with 65% water-cement ratio and 1:3 cement/Sand (By mass). The moulds with fresh mortar test specimens were compacted into these moulds with the use of a vibrating table. After casting, the steel moulds containing the specimens were covered with plastic wrap to prevent loss of water by evaporation. After 24 h the specimens were demolded and cured in water at 20 $\pm$ 2 °C and above relative humidity of 95% RH till test. Three specimens were tested per specimen age.
Tables 2 and 3 give the mixes of the sands made by partial and full substitution of the natural fine aggregate (dune fine aggregate) by artificial fine aggregate (waste bricks and blast furnace slag) at various ponderal contents (0, 25, 50, 75 and 100%) and the ponderal composition of the mortars used in this experimental work.

Table 2: Mix Composition of Mixed Fine Aggregates (Clay Fine Aggregate or Slag Fine Aggregate) Studied

Mix of fine aggregates substituted	Artificial fine aggregate "Clay or Slag fine aggregate" %	Natural fine aggregate "Dune fine aggregate" %
$S_{0\%}$	0	100
$S_{25\%}$	25	75
S_{5O}	50	50
$S_{75\%}$	75	25
$S_{100\%}$	100	0

Table 3: Ponderal Composition of Mortars Studied with Dune Fine Aggregate-Clay Fine Aggregate and Dune Fine Aggregate-Slag Fine Aggregate

Mix of Mortars studied	Mixed sands (g)		Cement (g)	Water (ml)
	Clay or Slag fine aggregate	Dune fine aggregate		
$M_{0\%}$	0	1350	450	292,5
$M_{25\%}$	337,5	1012,5	450	292,5
M_{5O}	675	675	450	292,5
$M_{75\%}$	1012,5	337,5	450	292,5
$M_{100\%}$	1350	0	450	292,5

RESULTS AND DISCUSSION

Effect of the quantity of artificial fine aggregate substituted on the workability of mortar

Water is a very important part of the mix and the volume of water used can dictate the strength of the finished mix. The method for measurement of fluidity with the "cone" of fresh mixed mortar is used to test the water content of the mortar. A cone made of steel is used for this test. The mixed mortar is placed into the cone through the top, a bar is used to compact the mortar, and remove air voids, within the cone. During the substitution of natural sand (dune fine aggregate) by artificial fine aggregate (calcined clay and slag), water-cement ratio was maintained constant.

Figure 1 presents the effect the quantity of artificial fine aggregate substituted on the workability of mortar. From the results obtained (Figure 1), the following conclusions may be drawn :

* a significant difference of the workability (slump test) beyond 25% of the substitution between the various mortars tested.

* a significant decrease of the workability (slump) beyond 25% of the replacement of the artificial fine aggregate (waste bricks and blast furnace slag) in relation to the weight of the dune fine aggregate.

The difference observed between the workability of mortars tested, depends of the content of the artificial fine aggregate incorporated with the natural fine aggregate (difference of the density and the porosity between the different fine aggregates studied). The Substituted artificial fine aggregate (clay fine aggregate and calcined clay fine aggregate) presents a high porosity compared to the natural fine aggregate (dune fine aggregate), this is mainly due at the variation of the physical properties for each type of fine aggregate.

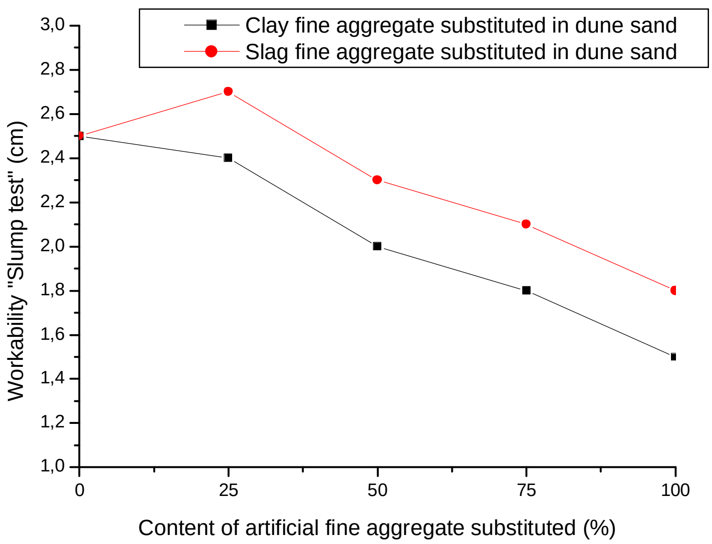

Fig. 1: Variation of Workability as a Function of the Quantity of Artificial Fine Aggregate Substituted

Effect of the quantity of clay fine aggregate substituted on the density of mortar

Figure 2 shows the effect of the clay fine aggregate substituted on the density of mortar. The substitution method by the incorporation of the artificial waste (slag sand or calcined clay sand) with the dune fine aggregate influences the density of the mortars studied. The increase of the percentage of the artificial sand at different percentages (50, 75 and 100%) in relation to the weight of the dune fine aggregate strongly decreases the density of mortar (variation of the porosity of fine aggregate). The difference observed between the density of mortars, depends of the percentage of the artificial fine aggregate (lightweight density) incorporated with the dune fine aggregate (low density of the calcined clay and slag). The best result of the workability is obtained for the substitution of 25% of artificial fine aggregate.

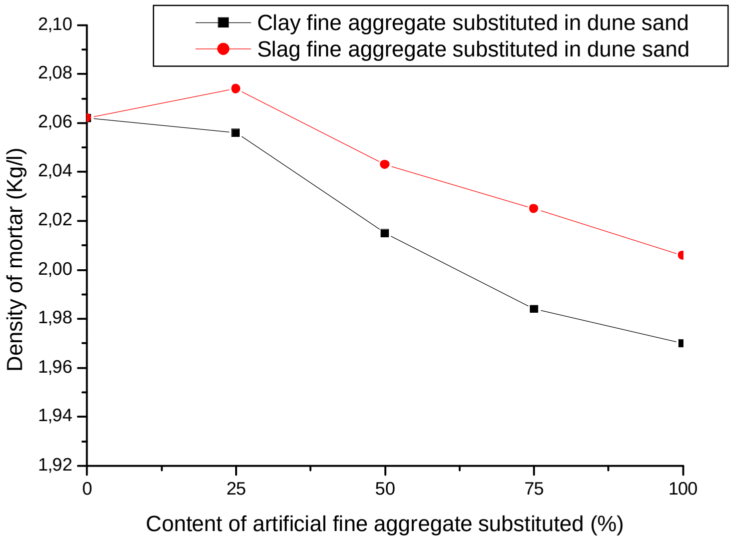

Fig. 2: Variation of Density as a Function of the Quantity of Artificial Fine Aggregate Substituted

Effect of the quantity of clay sand substituted on the mechanical strengths of mortar

The developments of compressive and flexural strengths of the test specimens are showns in Figs. 3 and 4. The compressive and flexural strengths increases with curing time for all hardened mortars. The increase of the percentage of the artificial fine aggregate beyond 25% by the substitution method in relation to the weight of the dune fine aggregate decreases the mechanical strengths for all samples tested (the chemical composition and the granulometry of fine aggregates). The difference observed between the mechanical responses of mortars, depends of the percentage of the artificial fine aggregate (lightweight density) incorporated with the dune fine aggregate (low density of the calcined clay and slag), the porosity and the chemical composition of calcined clay and slag. The results obtained show that the substitution of the fine aggregates by 25% of artificial fine aggregate (calcined clay and slag) by ratio to the ponderal weight of fine agregate gives a acceptable mechanical strengths of the examined mortars.

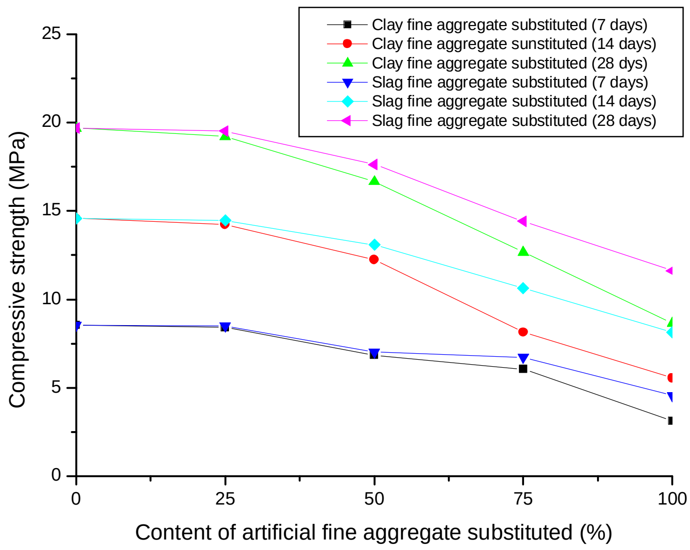

Fig. 3: Variatin of Compressive Strength of Mortars as a Function of the Quantity of Artificial Fine Aggregate Substituted

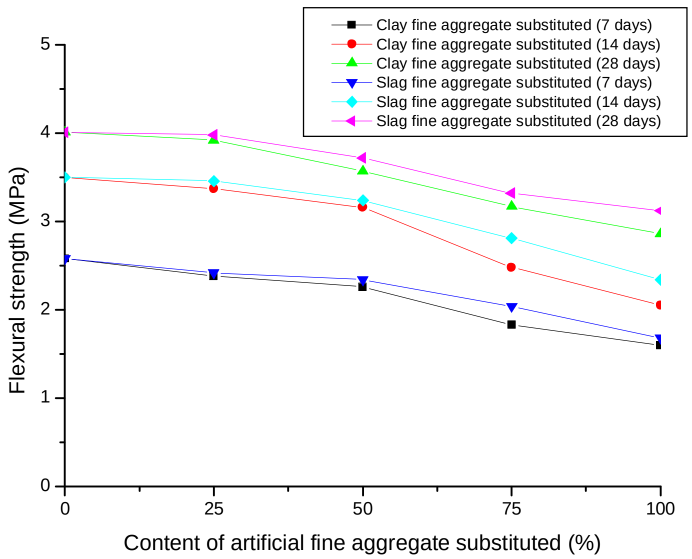

Fig. 4: Variation of Flexural Strength of Mortars as a Function of the Quantity of Artificial Fine Aggregate Substituted

Conclusion

Based on the experimental results, the following can be concluded :
* The substitution of the dune sand by the clay fine aggregate (calcined clay) or slag fine aggregate (blast furnace slag) influences appreciably on the water demand necessary to have a acceptable workability (fluidity or consistency) of fresh mortar.
* a significant decrease of the workability (slump) beyond 25% of the replacement of the artificial fine aggregate (waste bricks and blast furnace slag) in relation to the weight of the dune fine aggregate.
* The increase of the percentage of the artificial fine aggregate at different percentages (50, 75 and 100%) in relation to the weight of the dune fine aggregate strongly decreases the density of mortar (variation of the porosity of fine aggregate).
* The difference observed between the density of mortars, depends of the percentage of the artificial fine aggregate (lightweight density) incorporated with the dune fine aggregate (low density of the calcined clay and slag).
* The best result of the workability is obtained for the substitution of 25% of artificial fine aggregate.
* The increase of the percentage of the artificial fine aggregate beyond 25% by the substitution method in relation to the weight of the dune fine aggregate decreases the mechanical strengths for all samples tested (the chemical composition and the granulometry of fine aggregates).
* The difference observed between the mechanical responses of mortars, depends of the percentage of the artificial fine aggregate (lightweight density) incorporated with the dune fine aggregate (low density of the calcined clay and slag), the porosity and the chemical composition of calcined clay and slag.
* The results obtained show that the substitution of the fine aggregates by 25% of artificial fine aggregate (waste bricks and blast furnace slag) by ratio to the ponderal weight of fine aggregate gives a acceptable mechanical strengths of the examined mortars.

The best results of the workability, density and mechanical strengths are obtained for the substitution of 25% of slag fine aggregate in comparison with the clay fine aggregate (low porosity and better pozzolanic reactivity of slag).

ACKNOWLEDGMENTS

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UTILIZATION OF BURNT RICE AND COTTON STRAWS IN PRODUCING LIGHTWEIGHT - THERMALLY CHARACTERIZED CLAY BUILDING UNITS

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ABSTRACT

Utilization of Burnt Rice Straw BRS or Burnt Cotton Straw BCS in building clay bricks industry shares in the environmental protection against the pollution caused by the accumulation of such solid waste, in addition to saving energy consumed in cooling instruments. One method of increasing the insulation ability of brick is generating porosity in clay body. Combustible organic types of pore-forming additives are most frequently used for this purpose. El-Saff clay, (Giza Governorate, Egypt) representative sample as well as BRS and BCS were investigated, from their physical, mineralogical and chemical composition. El-Saff clay is silty and well sorted with a mineralogical composition of montmorillonite, kaolinite and illite clay minerals in addition to, quartz, feldspar, gypsum and calcite as the non-clay minerals. BRS is mainly composed of feldspars; BCS is mainly composed of quartz and calcite in addition to some feldspar minerals. Specimens fired at 800 ^{o}c of clay / BRS with 5, 10, 15, 20, 25 and 30% of BRS have a reduction of 22.22, 35.74, 39.73, 49.17, 50.5 and 56.2% of the bulk density of a blank clay specimen, while they have a reduction of 68.2, 75.1, 86.9, 88.7, 90.5 and 93.5% of the thermal conductivity of a blank clay specimen. A mix of clay/10% BRS or clay/5% BCS at a lower firing temperature (800^{o} C) completely agree with the limits of the Egyptian Standard Specifications for building units made from clay No.43 for year 1982, No.1756for year 1989 and No. 1524 for year 1993. Increasing the firing temperature has little effect on the product obtained; this saves on fuel used in firing process.

Keywords: Rice Straw, Cotton Straw, clay brick.

INTRODUCTION

Recently, intensive work has been done on investigating possibilities for using different wastes as additives in the production of new building products. Due to environmental regulations, the demand for high insulation ability of bricks is increasing. The aim of this work is to investigate

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the effect of utilizing some agricultural wastes such as Burnt Rice Straw BRS and Burnt Cotton Straw BCS on the thermo-physical properties (drying shrinkage, firing shrinkage, bulk density, porosity, water absorption and thermal conductivity) and mechanical properties (compressive strength), as supplementary materials to the fired clay specimens. The samples were tested by using the standard test methods and compared with the specifications and with the clay specimens made of pure clay. The results obtained are useful in showing possibilities for the solution of environmental problems.

EXPERIMENTAL PROCEDURES

The mineralogical composition of the studied raw materials was examined using a Philips PW 1050/70 X-ray diffractometer. The powder X-ray diffraction method was used for the identification of non clay minerals. The oriented aggregates of the <2mμ fraction method was used for the identification of clay minerals through processes of untreating, glycerol solvating and heating at 550 ^oC for 2 hrs. The identification of clay and non-clay minerals was based on the Mineral Powder Diffraction File Data Book (ICDD, Pennsylvania**,** 1993). The semi-quantitative estimation of the separated clay minerals was calculated according to the method of Johns et al. 1954. Philips PW-1400 X-ray florescence spectrometer and traditional chemical analysis method were applied for studying the chemical composition of the clayey raw materials in addition to the chemical composition of BRS and BCS. Ten clay/BRS mixes with 0, 2.5, 5, 7.5, 10, 12.5, 15, 20, 25 & 30% BRS additives in addition to four clay/BCS mixes with 5, 10, 15, & 20% BCS additives were prepared. The physical properties of the clay raw material in terms of plasticity and drying behavior as well as their properties with various percentages of BRS and BCS were examined. The plasticity measurements were carried out adopting the method of Pfefferkorn, 1924; based on deformation caused by the action of a piston on clay cylinders of different water contents. The values of water content corresponding to a compressibility = 3.3 is considered as the plasticity coefficient (PC) i.e. water of plasticity. The drying behavior was calculated according to ASTM C67-2003. The obtained data were used for drawing Bigot's curves (Lach, V., 1965) from which the drying sensitivity coefficients (D.S.C.) were calculated for each mix. Physical and mechanical properties were studied after firing the dried specimens at 800, 850 and 900^oC with 1 ^oC/min rate of firing and 2hrs soaking time. Finally, various physical, mechanical and thermal standard test techniques were utilized.

RESULTS AND DISSCUSION

The studied El-Saff clay sample contains clay > silt > sand specimens. They had a grain size distribution ranging from silty clay, to sandy silt clay according to the classification of Picard (1971); this means that El-Saff clays have high percentage of sand and silt fractions compared with the most predominant highly plastic Egyptian clays. Accordingly, El-Saff clay has no need for sand additives to be workable during the molding process. Table 1 shows the grain size composition of the studied clayey raw material.

Table 1: Grain Size Composition of the Examined El-Saff Clay Sample.

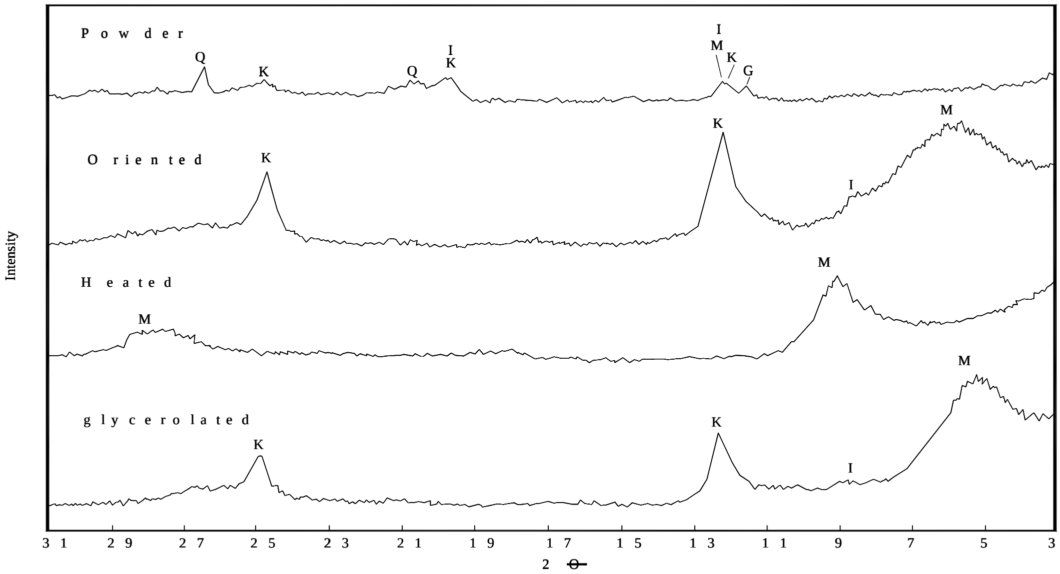
Clay sample	Grain Size (micron)										
	Sand Fraction					Silt Fraction					Clay Fraction
	2000 to 1000	1000 to 500	500 to 250	250 to 125	125 to 63	63 to 32	32 to 16	16 to 8	8 to 4	4 to 2	< 2
El-Saff clay	0.0	0.0	0.0	0.0	1.5	3.3	4.1	10.3	10.6	14.4	55.80

Burned straws produced from the controlled burning process were found to be soft and easily pulverized. The specimens of burned rice and cotton straws are angular yet highly cellular; hence the reactive surface is reflected by the specimen size distribution not the internal porosity of the individual specimens. In this study, BRS & BCS were ground manually using a mill found in (HBRC). Grinding was continued until reaching suitable grading lying in the limits of loose fill thermal insulation materials. The grading of the powder was then determined as shown in Table (2)

Table 2: Sieve Analysis for BRS and BCS.

Sieve diameter(mm)		6.35	4.75	2.38	1.19	0.60	0.43	0.30	0.15	0.08	0
% Pass	BRS	100	99.21	98.03	92.88	71.02	51.99	40.89	17.17	8.45	0
	BCS	100	93.02	81.00	63.21	55.31	30.52	23.23	15.89	7.22	0

The X-ray diffraction patterns of El-Saff clay powder and the separated clay fraction sample are shown in Figure (1). The powder clay sample shows that it is mainly composed of quartz (Q) and gypsum (G) as non-clay minerals in addition to kaolinite (K) and illite (I)as clay minerals. The oriented, heated and glycerolated clay fraction separated samples show that they are mainly composed of montmorillonite (M) (84%), kaolinite (15.7%) and illite (0.3%) in a descending order of abundance. Accordingly, Montmorillonite clay minerals represent the major clay mineral in the studied clay sample; this indicates that EL-Saff clay behaves as plastic clay, since montmorillonite is responsible for the plastic behavior of clays (Searle, et al, 1959). It is noticed that



the mineralogical composition of EL- Saff clays agrees with that of the most Egyptian clays. Such clays could be used and workable in several building material industries.

Fig.1: X- Ray Diffraction pattern of the Studied El-Saff Clay Sample.

Figure (2) shows the mineralogical composition of both burnt rice straw (BRS) and burnt cotton straw (BCS). It was noticed that burnt rice straw (BRS) is mainly composed of feldspars, while burnt cotton straw (BCS) is mainly composed of quartz and calcite in addition to some feldspar minerals. Generally there is a kind of similarity in the mineralogical composition of the powder samples of both clay and the burnt straws samples, such similarity may encourage for making several mix designs between them.

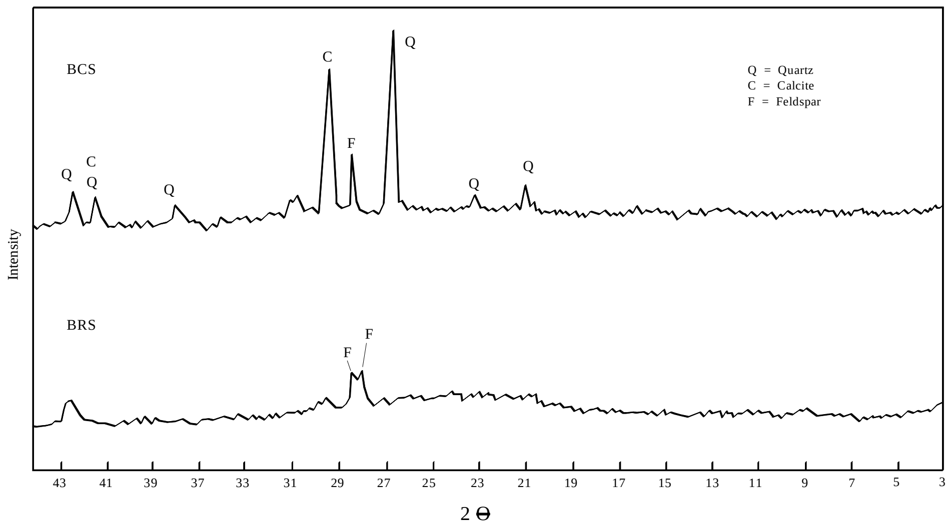


Fig. 2: X- Ray Diffraction Pattern of the Studied Burnt Rice BRS and Burnt Cotton Straws BCS

Table (3) shows the chemical analysis of the studied El-Saff clay sample in addition to the burnt rice and cotton straws (BRS & BCS) powder samples. The data of chemical analysis indicates that El-Saff clays contain a relatively high percentage of silica SiO_2 (46.15%) and moderate percentage of alumina $Al_2 O_3$ (18.47%), in addition to low percentage of CaO (1.25%), high Fe_2O_3 (9.86) and low % of L.O.I (14.95 %). It can be noticed from the results of chemical analysis that burnt rice or cotton straws have lower contents of SiO_2 and Al_2O_3 and high percentage of loss on ignition L.O.I (50.48 and 69.32% respectively). Generally, the chemical composition of the studied clay and burnt rice or cotton straws are in quiet agreement with their mineralogical composition.

Table 3: Chemical Analysis of the Studied El- Saff Clay Sample, BRS and BCS.

Oxides	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	Na_2O	K_2O	SO_3	L.O.I	TiO_2	P_2O_5	Total
El-Saff clay	46.15	18.47	9.86	1.25	2.76	1.51	1.16	2.33	14.95	1.21	0.15	99.80
BRS	31	0.8	0.17	7.24	1.6	0.63	3.8	0.85	50.48	2.71	--	99.28
BCS	17.11	2.66	0.38	3.22	1.55	0.32	1.84	2.15	69.32	0.06	1.26	98.87

Figures (3) and (4) show the plasticity curves for the studied El-Saff clays with various percentages of burnt rice and cotton straw additives BRS and BCS respectively. It could be noticed that addition of burnt rice straw to El-Saff clays decreases their original plasticity. At the same time, as the amounts of BRS additives increase, the values of plasticity also increase. Such behavior could be attributed to the size and the plastic nature of both clays and BRS or BCS additives. The type of clay minerals and its specimen size are the principal factors affecting the plasticity of clays , (Platen & Winkler,1958) . Also, it's well known that, the more the plastic clays the good brick properties could be obtained, (Searle & Grimshaw, 1959 and Abdel Ghafour, 1995). On the other hand it was noticed that addition of BCS to EL-Saff clay samples has the same effect of BRS on the plastic behavior of the applied mixes. But it was also obviously noticed that the effect of BCS on the plasticity of the studied clays is greater in diminishing their plasticity coefficient values if compared with the same percentage of BRS additives.

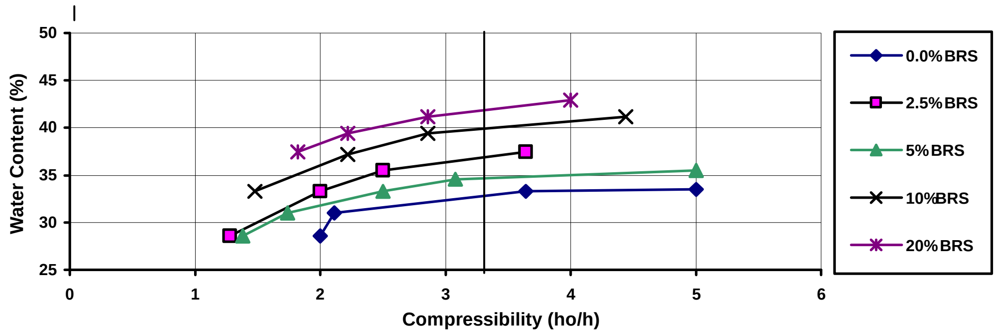


Fig. 3: Plasticity Curves for the Studied Clay Mixes with Various Percentages of Burnt Rice Straw (BRS) Additives.

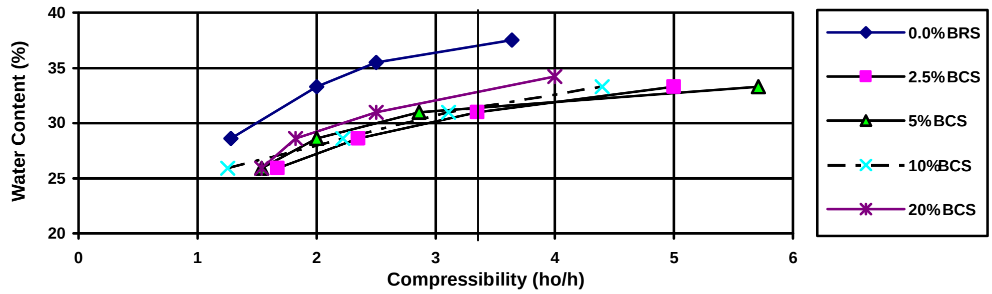
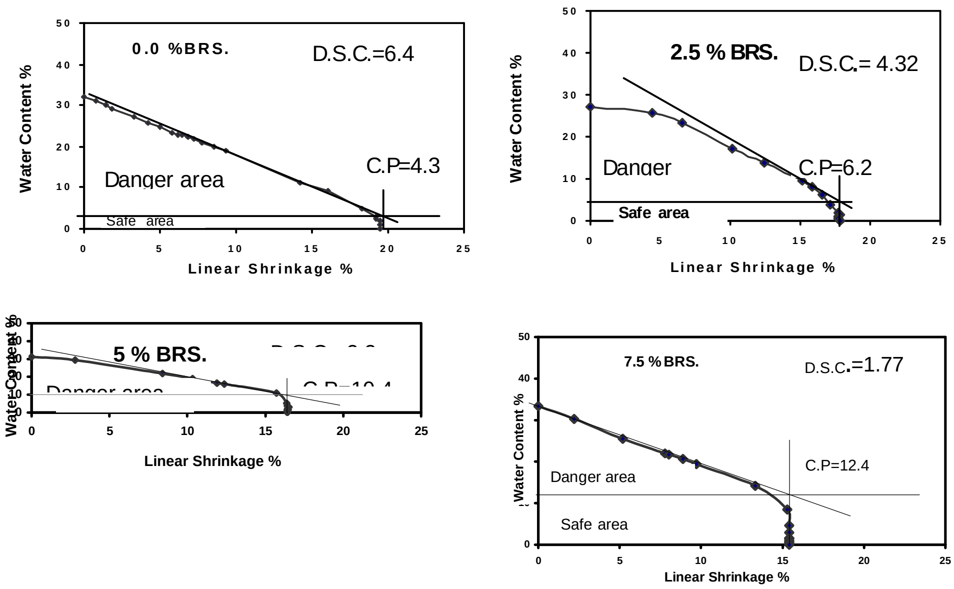


Fig. 4: Plasticity Curves for the Studied Clay with Various Percentages of Burnt Cotton Straw (BCS) Additives.

Plastic pastes of El-Saff clays as well as their mixes with burnt rice and cotton straws were molded into cubes of 5cm side length. The molded specimens were left to dry gradually at room temperature and then in a dryer at 110^0 C for 24 hours. The drying behavior was calculated as the method in ASTM (C67-C66). The obtained data was used for drawing Bigot's curves (Lach, 1965) from which the drying sensitivity coefficient (D.S.C.) was calculated for each studied clay /BRS and clay /BCS mixes. In such Bigot's curves the percentage of the linear shrinkage and its corresponding water content for clay /BRS and clay /BCS mixes during drying process were determined and plotted in figures (5) and (6) respectively. Figures (7-a) and (7-b) represent the relationship between the amounts of BRS or BCS additives and the drying sensitivity coefficient (D.S.C.) respectively.



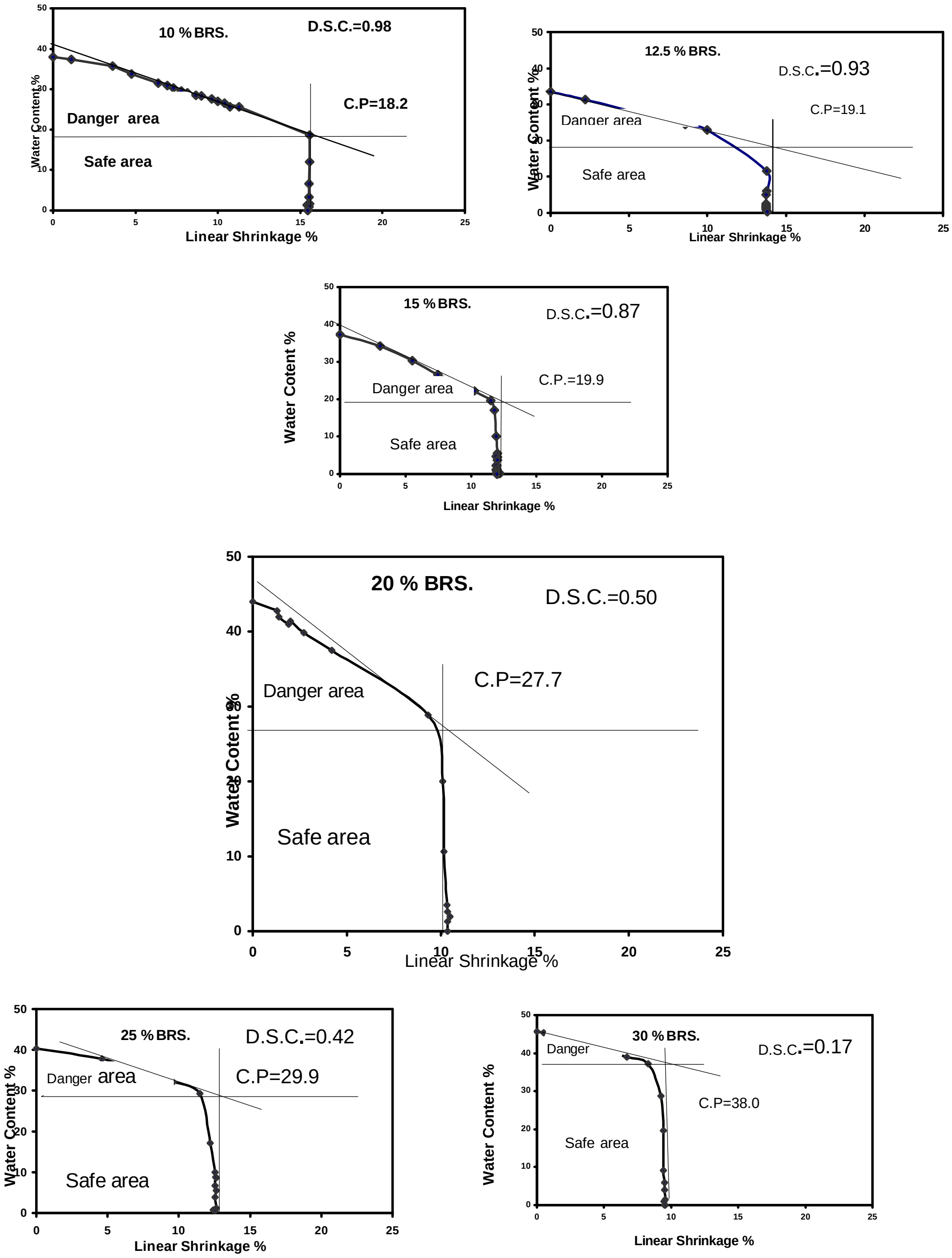


Fig. 5: Bigot's Curves of the Studied Clay Specimens with Different Percentages of Burnt Rice Straw (BRS) Additives.

The drying sensitivity coefficient (D.S.C.) of El-Saff clays indicates that such clays are very sensitive upon drying (ASTM C 67-2003) due to their high content of clays specially montmorillonitic clay minerals. It is noticed that as the percentage of BRS or BCS increase, the D.S.C. of El-Saff clays decreases from very sensitive to insensitive upon drying process. It is obviously noticed that addition of 10% BRS decreases the (D.S.C.) of El-Saff clays to 0.98 making it insensitive upon drying. Generally, addition of more than 10% till 30% of BRS increases the insensitivity of such clays upon drying from 0.98 to 0.17 respectively. This behavior is attributed to the role of burnt rice and cotton straws, which diminishes the action of clay minerals during the drying process. On the other hand, addition of 10% BCS decreases the (D.S.C.) of El-Saff clays to 0.73 while addition of more than 10% until 20% of BCS increases the insensitivity of such clays upon drying from 0.73 to 0.50

respectively. The optimum expected or maximum percentage of additive could be depending on the physical and mechanical properties of the fired clay specimens.

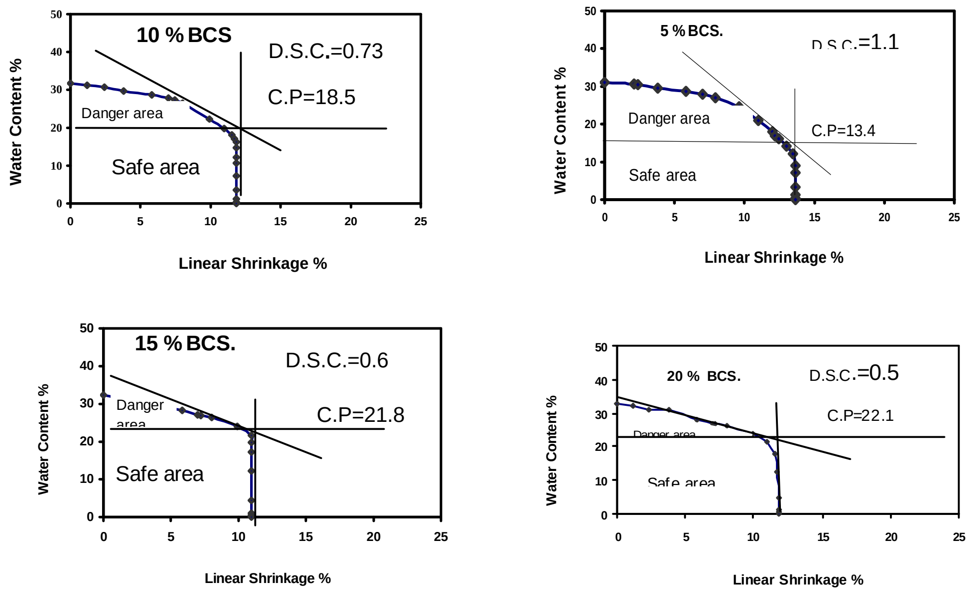


Fig. 6: Bigot's Curves of the Studied Clay Specimens with Different Percentages of Burnt Cotton Straw BCS additives.

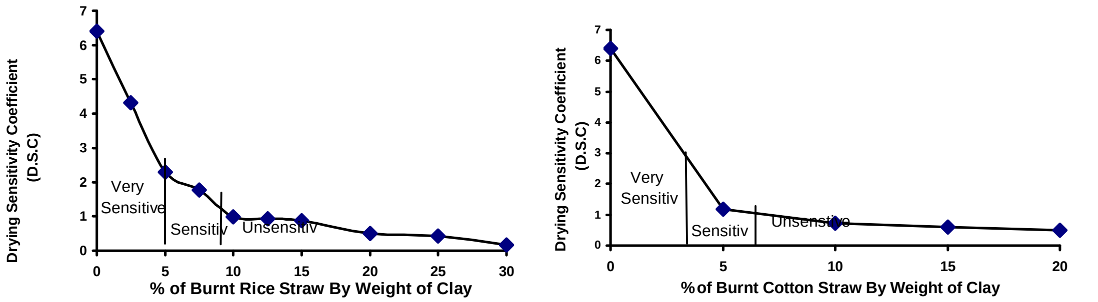


Fig. 7: Relationship between the Percentages of Burnt Rice and Cotton Straw (BRS) and (BCS) within the Studied Clay Specimens and the Drying Sensitivity Coefficient (D.S.C).

Physical properties

The dried green specimens were fired in an electric oven at temperatures between 800-900^{o}C. Physical and mechanical properties were studied for the fired specimens at 800, 850 and 900^{o}C with 1^{o}C/min rate of firing and 2hrs soaking time. Tables (4 and 5) and figures (9 to 23) show the results of the physical and mechanical properties of the studied fired clay/BRS and clay/BCS specimens with their graphical relationships.

Table 4: Physical and Mechanical Properties for the Clay Specimens with Different Percentages of Burnt Rice Straw (BRS) at Different Firing Temperatures.

BRS (%)	Firing Temp. (^oC)	Firing Volume Shrinkage (%)	Bulk Density (gm/cm^3)	Apparent Specific Gravity	Apparent Porosity (%)	Water Absorption (%)	Compressive Strength (kg/cm^2)	
							Dry	Wet
0.00	800	18.09	1.818	2.00	16.93	8.48	316	241.8
	850	14.24	1.802	1.982	18.32	9.24	247.65	195.6
	900	6.81	1.762	1.939	22.40	11.55	178.9	148.5
2.5	800	21.17	1.543	1.651	20.30	12.3	233.1	177.6
	850	17.62	1.535	1.645	29.88	18.22	223.25	165.2
	900	6.80	1.530	1.578	35.25	22.34	142.3	133.2
5.0	800	20.24	1.448	1.598	22.85	14.3	210.2	156.89
	850	16.69	1.404	1.567	31.08	19.79	162.2	140.95
	900	5.50	1.361	1.562	37.50	24.0	124.03	119.7
7.5	800	16.4	1.333	1.573	25.80	16.40	143.1	117.3
	850	14.38	1.320	1.510	34.50	22.85	123.2	102.3
	900	4.66	1.312	1.507	37.98	25.20	103.7	95.35
10.0	800	14.02	1.171	1.398	27.71	19.82	127.6	83.3
	850	12.60	1.158	1.383	34.71	23.50	103.2	78.20
	900	4.46	1.153	1.375	36.16	25.9	89.85	76.3
12.5	800	12.80	1.119	1.238	29.35	23.70	107.6	78.3
	850	11.92`	1.107	1.193	36.60	28.67	96.9	74.8
	900	2.87	1.105	1.158	36.99	31.95	71.55	64.2
15	800	11.70	1.102	1.218	34.79	28.56	85.9	65.2
	850	10.30``	1.086	1.178	37.63	31.95	71.6	56..25
	900	2.71	1.065	1.128	43.31	38.39	68.20	50.30
20	800	10.00	0.969	1.141	40.26	41.48	65.20	55.6
	850	4.17	0.916	1.129	47.12	41.73	56.6	44.40
	900	2.55	0.901	0.945	62.86	66.53	42.30	36.8
25	800	7.42	0.895	1.026	44.04	42.93	53.6	46.82
	850	4.11	0.892	0.983	54.3	54.29	44.30	37.90
	900	1.58	0.840	0.880	66.17	66.17	32.0	23.40
30	800	5.36	0.796	0.935	50.8	54.32	39.30	30.20
	850	3.10	0.789	0.916	59.8	65.26	20.20	18.80
	900	0.44	0.751	0.799	67.64	84.65	14.70	12.60

Table 5: Physical and Mechanical Properties for Clay Specimens with Different Percentages of Burnt Cotton Straw (BCS) at Different Firing Temperatures.

BCS (%)	Firing Temp. (^oC)	Firing Volume Shrinkage (%)	Bulk Density (gm/cm^3)	Apparent Specific Gravity	Apparent Porosity (%)	Water Absorption (%)	Compressive Strength (kg/cm^2)	
							Dry	Wet
0.00	800	18.09	1.818	2.00	16.93	8.48	316	241.8
	850	14.24	1.802	1.982	18.32	9.24	247.65	195.6
	900	6.81	1.762	1.939	22.40	11.55	178.9	148.5
5.0	800	3.92	1.436	1.586	18.7	12.55	145.0	74.3
	850	3.09	1.417	1.230	20.49	15.03	117.5	26.4
	900	2.35	1.412	1.575	26.25	16.67	55.6	32.1
10.0	800	3.35	1.280	1.467	19.16	13.06	78.0	45.0
	850	2.94	1.265	1.416	22.04	15.57	64.2	25
	900	2.23	1.249	1.346	33.68	25.02	24.78	24.27
15	800	2.96	1.101	1.265	27.5	30.45	15.8	11.7
	850	2.22	1.002	1.202	35.22	32.64	12.75	9.4
	900	1.95	0.977	1.091	40.92	37.5	5.29	3.87
20	800	2.85	0.866	1.041	44.4	42.63	5.39	5.36
	850	2.12	0.821	0.959	47.48	50.98	4.64	4.23
	900	1.79	0.800	0.955	52.06	54.04	4.25	2.78

§ Firing Volume Shrinkage

The firing shrinkage of the studied fired specimens was generally decreased by increasing the firing temperature. This behavior could be attributed to the grain size and the textural composition of El-Saff clays which are considered as sandy silt clays. Such firing volume shrinkage of the fired specimens could be explained also, due to the role of the burnt rice or burnt cotton straws which are brittle burned filler materials which do not suffer any shrinkage by firing with the clay mixes at the applied firing temperatures. Accordingly, addition of burnt rice and cotton straws limits the effect of firing shrinkage obtained by the clay minerals within the clay/ BRS and clay/BCS specimens during and after firing process.

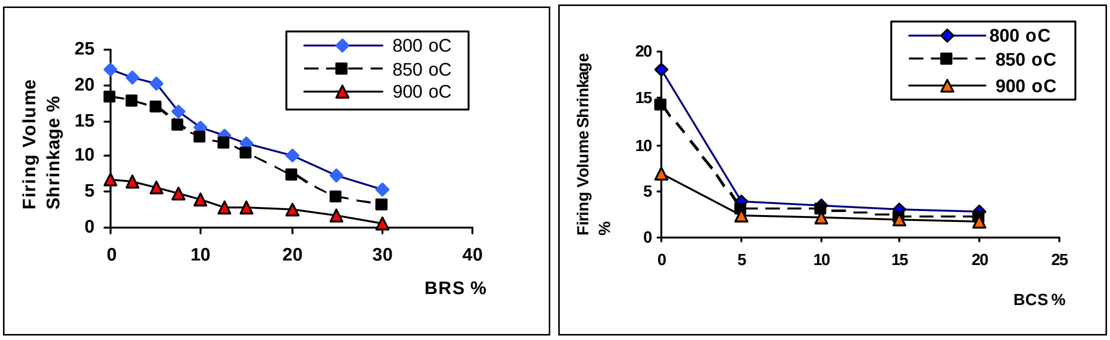


Fig. 9: Firing Volume Shrinkage of Clay Specimens Fired at 800, 850 and 900 °C with Different Percentages of Burnt Rice Straw (BRS) or Burnt Cotton Straw (BCS).

§ Bulk Density

Bulk density also decreased by increasing the additive content and as well as by increasing the firing temperature for all mixes. It was also noticed that the additive content has a greater effect than that of firing temperature on decreasing the bulk density values. The behavior of firing shrinkage was well reflected on the bulk density values of the studied fired specimens. It should be recorded that, the high content of sand and silt fractions in the clayey material and the added percentage of waste materials (BRS & BCS) gives the fired specimens more open pores during the release of gases and water vapor through the firing process. This behavior could be reflected on the porosity and consequently the other studied physical and mechanical properties.

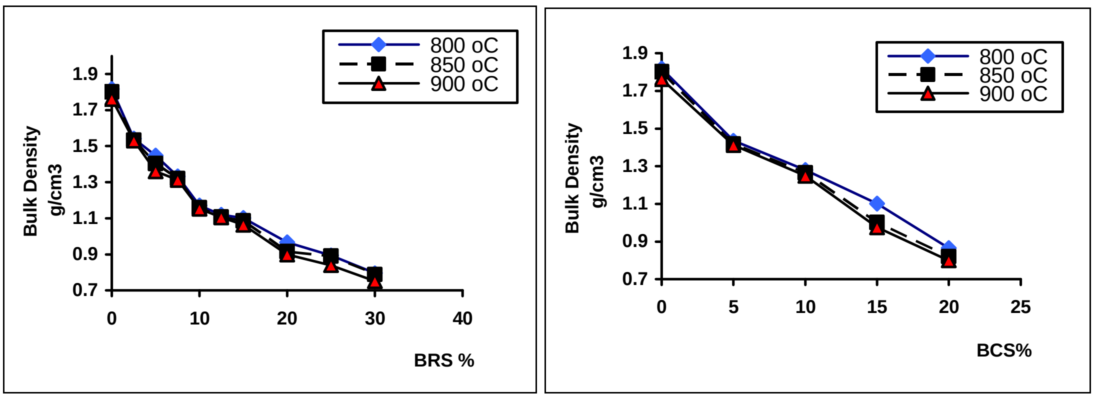


Fig. 10: Bulk Density of Clay Specimens Fired at 800, 850 and 900 °C with Different Percentages of Burnt Rice or Cotton Straws (BRS) or (BCS).

§ Porosity

It was noticed that, as the percentage of additives increases the values of porosity also increase. At the same time, as the firing temperatures increase the porosity values increase.

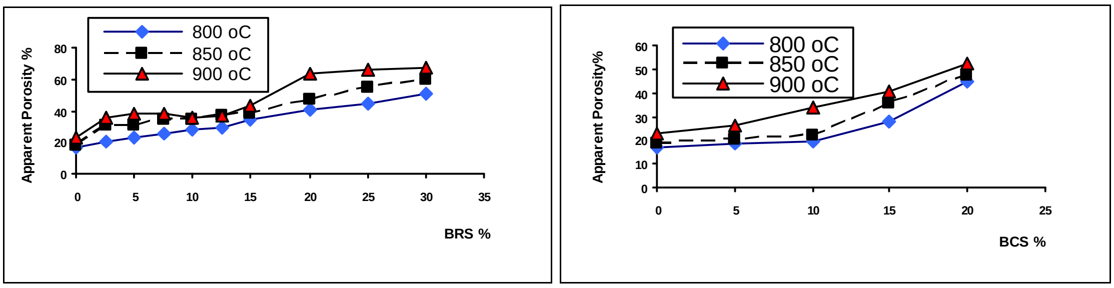


Fig. 11: Apparent Porosity of Clay Specimens Fired at 800, 850 and 900 °C with Different Percentages of Burnt Rice and Cotton Straws (BRS) or (BCS) Additives.

§ Water Absorption

The percentages of water absorption were generally increased by increasing the additive content and as well as the firing temperature with all the fired specimens, It should be reported that, the evolving of various gases during firing process such as carbon dioxide and sulphates leave some kind of porous cavities responsible for the highly characteristic percentage of water absorption. Such gases were evolved from clayey material itself (SO4) or from the burnt additives (CO2). Also the high content of sand and silt fractions within EL-Saff clays add another kind of porosity due to the melting of clayey material leaving stable sandy and silty material at the applied temperatures of firing (800 – 900 ^{o}C). On the other side, the additive materials have a great ability to absorb water or release moisture until they come into equilibrium with the relative humidity of the surrounding air. Such behavior was noticed during plasticity test, molding process and after the drying and firing processes. This may explain the abrupt increase of water absorption values of the fired specimens with 30% BRS at the highest temperature of firing (900^{o}C). Finally, the increase of firing temperature and the percentage of burnt rice or cotton straw additives increases the pore spaces and accordingly the values of water absorption of the fired clay specimens. The values of water absorption given by the fired specimens could be considered as a direct reflection to the firing shrinkage and bulk density values.

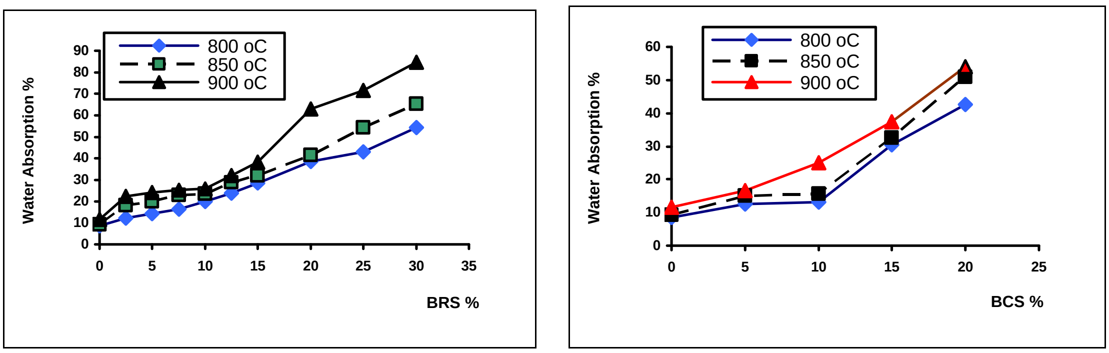


Fig. 12: Percentages of Water Absorption of Clay Specimens Fired at 800, 850 and 900 °C with Different Percentages of Burnt Cotton Straw (BCS) Additives.

Mechanical properties

§ Compressive Strength

The results indicate that the strength is greatly dependent on the amount of agro-waste additives in the brick and the firing temperature. El-Saff fired clay specimens have the highest dry and wet compressive strength since they have a vitrification with temperature of about 800^{o}C, which produces molten phases acting as strong binding material after firing, (Abd El-Ghafour). It was noticed that the dry and wet compressive strength values of the fired clay specimens decrease with increasing the firing temperature. Also, increasing of burnt rice or cotton straw additives is associated with decreasing in compressive strength values. The decrease of compressive strength values could be attributed to the role of sand and silt fractions included within the textural composition of the clayey raw material. Also, the added brittle burnt rice or cotton straw plays an important role in decreasing compressive strength values. It should be recorded that, the fusion phase which takes place by the clay minerals during the firing process, while sand or silt contents haven't any chance for fusion during the applied range of firing temperature, so, pores will be formed after cooling process. Such behavior could explain the higher percentage of porosity and the lower values of compressive strength in the fired specimens with 30 % burnt rice straw and 20% burnt cotton straw additives. Moreover, the descending in strength especially with the higher percentages of BRS or BCS additives may be attributed to the burning of the high amounts of organic matters (50.48% and 69.32% respectively) included within the burnt rice straw leaving porous framework inside the fired specimens, so, the compressive strength of clay/ BCS specimens have smaller values than that of clay/ BRS , therefore, a reasonable compressive strength values could be achieved at a lower percentages of burnt additives with a lower range of firing temperature.

Finally, the fired specimens with 10% burnt rice straw BRS or 5% burnt cotton straw achieved a higher and a reasonable dry and wet compressive strength values (127.6kg/cm^2 & 83.3 kg/cm^2, dry and wet, for BRS and 145.0 kg/cm^2 & 74.3kg/cm^2 dry and wet for BCS respectively.) at the lower applied firing temperature (800^o C).It's well known that fluxes or impurities and vitrification of clays are dependent upon each other. Feldspars which are included in the clayey raw material and the BRS or BCS represent the main source of alkalis within the studied mixes. Such alkalis act as a fluxing agent leading to a high degree of vitrification in the clay body. Accordingly, the relative differences in the composition of the studied mixes, their impurities and fluxes may explain the differences in the obtained compressive strength values. Accordingly, addition of dry burnt rice or cotton straws to -the highly plastic El-Saff clays shouldn't exceed 10% or 5%, respectively, with firing temperature not more than 800 ^{o}C. So, it is concluded that, the compressive strength values of all fired specimens decrease by increasing the percentage of additives at all firing temperatures. Figures (13) represent the relationship between the percentages of BRS or BCS additives and the compressive strength at different firing temperatures respectively.

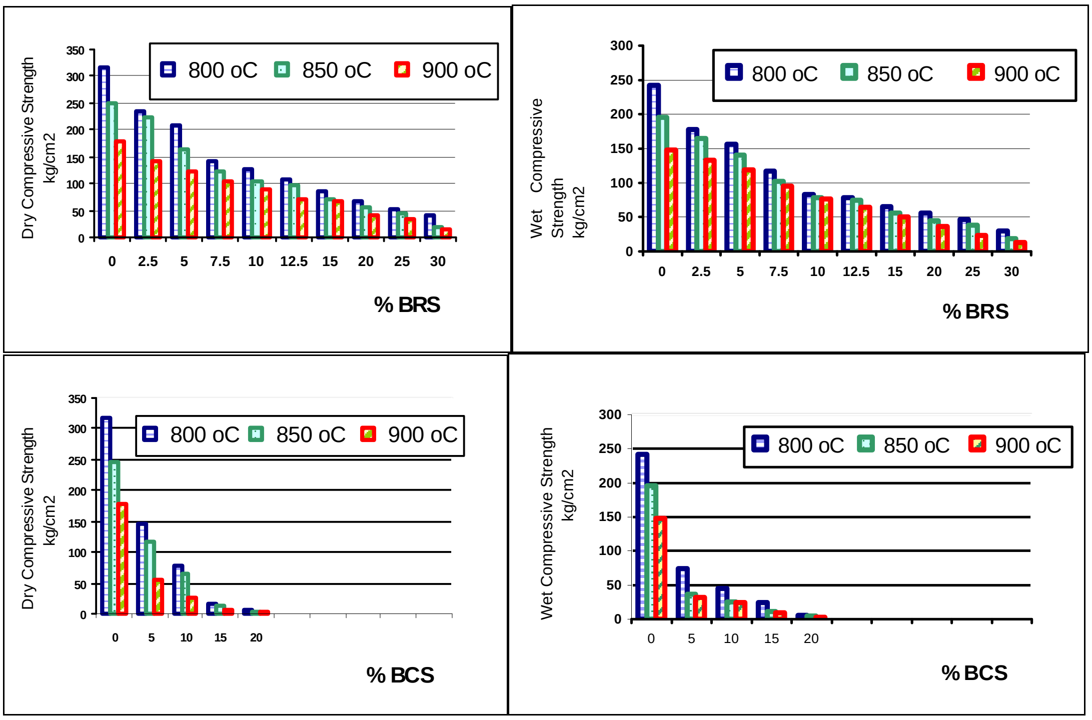


Fig. 13: Dry and Wet Compressive Strength Values of the Fired Clay Specimens with Different Percentages of Burnt Rice and Cotton Straws BRS and BCS Additives at Different Degrees of Firing.

Thermal properties

Table (6) and figures (14) & (15) show the relation between BRS or BCS content and the thermal conductivity of the fired clay specimens with different percentages of additives.

It should be recorded that the high content of sand and silt fractions in the clayey material and the added percentage of waste materials (BRS & BCS) gives the fired specimens more open pores during the release of gases and water vapor through the firing process. This behavior could be reflected on the porosity and consequently the bulk density and thermal conductivity. This means that the more addition of burnt rice and cotton straws, the less bulk density and less thermal conductivity values could be obtained.

Table 6: Thermal Conductivity of the Fired Clay Specimens with Different Percentages of Burnt Rice Straw BRS or Burnt Cotton Straw BCS Firing at 850 °C.

Additive material	Percentage %	Thermal Conductivity (w/m°c)
Burnt Rice Straw (BRS)	0	1.000
	2.5	0.658
	5	0.318
	7.5	0.275
	10	0.249
	12.5	0.184
	15	0.131
	20	0.113
	25	0.095
	30	0.063
Burnt Cotton Straw (BCS)	5	0.298
	10	0.215
	15	0.119
	20	0.101

Finally, the fired specimens with 10% burnt rice straw BRS or 5% burnt cotton straw BCS achieved a higher and a reasonable dry and wet compressive strength values and lower bulk density and thermal conductivity. It was noticed that as the percentage of additives increase the values of thermal conductivity decreases. At the same time, as the thermal conductivity decreases the bulk density also decreases. Accordingly there is a direct relationship between the percentage of BRS or BCS additives from one side with the values of thermal conductivity and the bulk density on the other side.

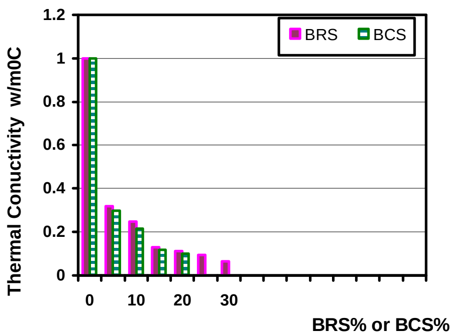


Fig. 14:Thermal Conductivity of Clay / BRS and clay/BCS Specimens Fired at 850 °C.

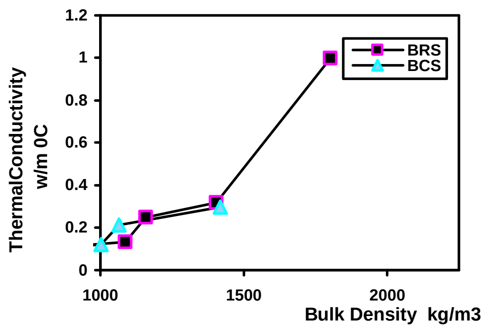


Fig. 15 Relationship between BRS or BCS Content and Thermal Conductivity of clay/BRS and clay/BCS Specimens Fired at 850 °C.

Uses of clay/BRS or clay/BCS building units in wall sections at different climatic regions of Egypt.

The most important properties of the clay/BRS or clay/BCS specimens were the lowest bulk density and thermal conductivity, bulk density of clay with 10%BRS or of clay with 5%BCS were less than that without additives by about 30.8% and 27.4% respectively, while for thermal conductivity they were less by about 75.1% and 70.2% respectively.

External walls are generally rated by their insulating value, known as the R-value. Materials with higher R-values are better insulators; materials with lower R-values must be used in thicker layers to achieve the same insulation value. The R-value of any building element such as wall or roof can be computed from conductance of the various components which make up the building elements. The total thermal resistance may calculate from the following equation (B. Givoni et al, 1968):

$$R_T = 1/h_{ao} + \sum^n_i L_i/k_I + 1/h_{ai} \qquad (1)$$

Where:

h_{ao} : the external heat transfer coefficients (W/m^{2o}c), it is equal to 20 & 18 (W/m^{2o}c) for roof and wall respectively.

h_{ai} : the internal heat transfer coefficients (W/m^{2o}c), it is equal to 8 (W/m^{2o}c) for both roof and wall.

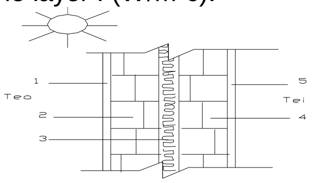
L_I : is the thickness (cm) and K_I is the thermal conductivity of the layer I (W/m^{o}c).

Wall section consists of the following:

1& 5: 2cm thickness of cement plaster (C.p) or other materials (Perlite, Vermiculite).

2 & 4: Each of 12.5-cm thickness of clay brick.

3: Space may be filled with: Air (A), or looseBRS or BCS.



Cross section of brick wall

To calculate the R-values of different wall sections, the next equation is considered:

$$R = R_{air} + L_1/K_1 + L_2/K_2 + L_3/K_3 .+ R_{inside}$$

For solid pure clay brick:

$R_{12\ cm}$ = 0.055 + 0.02/1.0 + 0.12/1.0 + 0.02/1.0+ 0.125 = 0.34 m^{2}c^o/W.

$R_{25\ cm}$ = 0.055 + 0.02/1.0 + 0.25/1.0 + 0.02/1.0 + 0.125 = 0.47 m^{2}c^o/W.

$R_{37\ cm}$ = 0.055 + 0.02/1.0 + 0.37/1.0 + 0.02/1.0 + 0.125 = 0.59 m^{2}c^o/W.

, for clay brick with 10% BRS:

$R_{12\ cm}$ = 0.055 + 0.02/1.0 + 0.12/0.249 + 0.02/1.0+ 0.125 = 0.702 m^{2}c^o/W.

$R_{25\ cm}$ = 0.055 + 0.02/1.0 + 0.25/0.249 + 0.02/1.0 + 0.125 = 1.224 m^{2}c^o/W.

$R_{37\ cm}$ = 0.055 + 0.02/1.0 + 0.37/0.249 + 0.02/1.0 + 0.125 = 1.706 m^{2}c^o/W.

, and for clay brick with 5% BCS:

$R_{12\ cm}$ = 0.055 + 0.02/1.0 + 0.12/0.298 + 0.02/1.0+ 0.125 = 0.622 m^{2}c^o/W.

$R_{25\ cm}$ = 0.055 + 0.02/1.0 + 0.25/0.298 + 0.02/1.0 + 0.125 = 1.059 m^{2}c^o/W.

$R_{37\ cm}$ = 0.055 + 0.02/1.0 + 0.37/0.298 + 0.02/1.0 + 0.125 = 1.462 m^{2}c^o/W.

Table (8) shows the physical and mechanical properties of clay bricks in both of the Egyptian Standard Specifications(ESS, 1989) and the clay/BRS or clay/BCS specimen which agree with them**.**

Table 7: Limits of The Physical and Mechanical Properties of Clay Bricks in the Egyptian Standard Specifications and the clay/BRS or clay/BCS Specimen which Agree with them.

Property	Limits in ESS	Clay with 10%BRS	Clay with 5%BCS
Bulk Density (gm/cm^3)	Not less than 1.6 (for load-bearing wall)	1.171	1.416
Water Absorption (%)	- Not more than 16% (for load-bearing wall)	19.82	12.55
Compressive Strength (kg/cm^2)	-Not less than 81.0 (for load-bearing wall) - Not less than 41.0 (for non load-bearing wall)	127.6	145
Thermal Conductivity (w/m^{o}c)	-----------	0.249	0.298

Table (8) shows the recommended wall sections, which may be used to give the required thermal resistances in different regions according to the Egyptian Code for Energy (2005).

Table 8: Recommended Wall Sections at Different Climatic Regions of Egypt.

Region			Orientation Of wall	Required R value ($m^2\ {}^oc/w$)	Recommended Wall section				
No.	Name	Cities			Exterior Plaster	Brick 12 cm	Insulation	Brick 12 cm	Interior Plaster
1	Northern coast	Alexandria Salloum, Dumyat, Marsa-Matruh, Port said, El Arish.	N,S	0.35 to 0.65	Cement plastering	Clay with 10% BRS or 5%BCS	Without Insulation	0	Cement plastering
2	Delta& Cairo	Zagazig, Shebeen El kom, Tanta, Banha, Mansoura.	N						
5	Eastern coast	Hurgada Marsa Alam	N,S						
1	As mentioned	As mentioned	E, W	0.66 to 1.15	Cement plastering	Clay with 10% BRS or 5%BCS	Without Insulation	Clay with 10% BRS or 5%BCS	Cement plastering
2	As mentioned	As mentioned	E, W, S						
3	North Wadi	Giza, Fayoum, Benisweif, Minya.	E, W, N,S						
4	South Wadi	Asyot, Kena, Kom Ombo, Lauxer,	N,S						
5	As mentioned	As mentioned	E, W						
6	High Hills	Saint Carteen, El – Toor	E, W, N,S						
7	Desert Region	Siwa, Farafra, Aldakhla, Alkharga.	N,S						
8	South of Egypt	Aswan, Toshki.	N,S						
4	As mentioned	As mentioned	E, W	1.16 to 1.706	Cement plastering	Clay with 10% BRS or 5%BCS	2.5 cm BRS or BCS powder	Clay with 10% BRS or 5%BCS	Cement plastering
7	As mentioned	As mentioned	E, W						
8	As mentioned	As mentioned	E, W						

It was noticed that in most climatic regions R-value of a wall of 25 cm thickness with 10%BRS or 5% BCS will be sufficient to agree with the requirements of the Egyptian Code of Energy (2005) , while east and west walls in regions 4, 7 and 8 needs only 2.5 cm of BRS or BCS powder in between the wall section.

CONCLUSIONS

The problem of rice and cotton straws disposal was highly concentrated due to the increased amounts of straws production which cause a great environmental problem. The industrial application of 10% BRS or 5% BCS with highly plastic clays produces suitable building clay

bricks by firing at a temperature as low as 800^o, so, the huge amounts of rice and cotton wastes can be recycled in the building brick industry. Moreover, a triple target could be achieved; first, an environmental protection against the pollution caused by the accumulation of rice and cotton straw wastes, second, saving a part of the clayey raw material needed urgently for building brick industry and third energy saving could be achieved through reducing the temperature of brick firing to about 800 ^oC.
Finally, mixes with various percentages of BRS or BCS more or less save the workable water, improve the drying sensitivity, in addition to the modification of some physical and mechanical properties of the fired specimens into reasonable ones under a normal applied range of firing temperature (800^{o}C).

From the above test results, it's concluded that:

1. El-Saff clays are considered as highly plastic clays with a clay fraction greater than silt and sand fractions.
2. El-Saff powder clay sample (non-clay fraction) is composed of quartz, gypsum and calcite, while, the powder burnt rice straw is composed of quartz, calcite, dolomite and gypsum. The contents of SiO_2 and Al_2O_3 in the clayey material are relatively higher than in the rice straw, while the values of CaO and L.O.I have an opposite trend.
3. Addition of BRS or BCS additives to El-Saff clay decreases its plasticity coefficient, but does not affect greatly the plastic behavior of the studied mixes. BRS or BCS can be used in clay brick making with a suitable percentage (not more than 10%), and firing at 800^{o}C.
4. Addition of BRS or BCS to the studied sensitive clay decreases its drying sensitivity coefficient (D.S.C.) from very sensitive to insensitive during the drying process.
5. As the amounts of BRS or BCS additives increase the values of plasticity also increase. Addition of 10% BRS or BCS decreases the (D.S.C.) of El Saff clays making it less sensitive upon drying.
6. The more addition of BRS or BCS the less firing shrinkage and bulk density values in addition to the more water absorption values will be obtained, so, the increase of BRS or BCS additives relatively raises the water absorption of the fired specimens.
7. The compressive strength values of El-Saff fired clay specimens with BRS or BCS additives generally decrease with increasing firing temperature. An increasing of rice or cotton straw additives decreases the compressive strength values.
8. The mix with 10% BRS achieved reasonable compressive strength values at all firing temperatures.
9. All the results of physical and mechanical properties for percentages of BRS and BCS less than 10%, completely agree with the limits of the Egyptian Standard Specifications for building units made from clay No.43 for year 1982, No.1756for year 1989 and No. 1524 for year 1993, and it is also agrees with the Egyptian Code for Design and Construction of Buildings ECP 204 - 2005.
10. All the physical and mechanical properties of the fired clay building bricks, except water absorption for BRS or BCS percentages higher than 10% completely agree with the limits of the Egyptian Standard Specifications for building units made from clay No.1756, for year (1989) and The Egyptian Code for Design and Construction of Buildings ECP 204 - 2005 .
11. Bulk density and thermal conductivity properties for all fired specimens of clay / BRS or clay / BCS additives have less value than that without BRS or BCS additives which encourage to be used as a thermal barrier in the external walls.
12. Fired specimens of clay / BRS with 5, 10, 15, 20, 25 and 30% of BRS have a reduction of 22.22, 35.74, 39.73, 49.17, 50.5 and 56.2% of the bulk density of a blank clay specimen, while they have a reduction of 68.2, 75.1, 86.9, 88.7, 90.5 and 93.5% of the thermal conductivity of a blank clay specimen.
13. Fired specimens of clay / BCS with 5, 10, 15 and 20% of BCS have a reduction of 22.1, 40.9, 44.4, and 54.4% of the bulk density of a blank clay specimen, while they have a reduction of 70.2, 78.5, 88.1 and 90.0% of the thermal conductivity of a blank clay specimen.
14. A mix with 10% rice straw BRS or 5% cotton straw BCS achieved reasonable physical and mechanical properties at a lower firing temperature (800^o C).

15. Utilization of BRS or BCS in building clay bricks industry shares in the environmental protection against the pollution caused by the accumulation of such solid waste, in addition to saving the great need of clayey raw material for making clay building bricks and also saving the energy required for brick manufacturing by reducing firing temperature to 800° C.
16. Using a wall of 25 cm thickness with 10% BRS or 5% BCS realize a sufficient thermal resistance in all regions except east and west walls in regions 4, 7 and 8.
17. It was noticed that in most climatic regions(1, 2, 3, 5 and 6), R-value of a roof insulation of 10 to 15 cm thickness of fired clay with 30%BRS or 10 cm to 12.5 cm of BRS or BCS powder was sufficient, while regions 4, 7 and 8 needs 15 cm of BRS or BCS powder to agree with the requirements of the Egyptian Code of Energy.

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PHYSICO-THERMAL PROPERTIES OF COMPOSITE- FILLED CEMENT PASTES

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ABSTRACT

The effect of limestone filler on the hydration characteristics as well as the fire resistance of composite cement pastes up to 800°C has been investigated. The hydration characteristics of cement pastes assessed by the determination of combined water content as well as bulk density, total porosity and compressive strength. The effect of fire on the cement pastes were investigated from the weight loss, bulk density, total porosity and compressive strength up to 800°C for two hours. Although partial replacement of basalt both by silica fume and limestone does not offer additional enhancement on the hydration characteristics of used control cement paste,it does not reduce the fire resistance of hardened cement pastes up to 600°C due to the filling role of limestone. From the above findings it can be concluded that composite cement pastes containing 80% OPC, 5-0% basalt, 10% silica fume and 5-10% limestone can be used as fire resisting cement.

Key words: Composite -Filled cement, Fire resistance, Basalt, Silica fume, Limestone filler.

INTRODUCTION

In pozzolanic-filled cement pastes, pozzolana improves the microstructure of hardened cement pastes due to its reaction with free lime librated from the hydration of cement forming additional cementatious materials which fill some of the open pores [1]. However, the most obvious disadvantage for most pozzolanic materials, especially natural pozzolans, is that its tendency to increase water requirement and lower the rate of strength development. Therefore, for structural applications their proportion in blended Portland cements is generally limited to 30% or less [2]. While filler acts as nucleation sites for precipitation of cement hydration products, as a result it significantly enhances the hydration of Portland cement and accelerates initial stage [3] as well as later stage of chemical hardening [4]. Fine and ultrafine fillers can reduce water requirement of a concrete mix thus increasing concrete strength [5].

The effect of fire on basalt-silica fume composite cement pastes up to 800°C has been investigated in previous work [6]. It was concluded that cement pastes resist firing with basalt content, also the replacement of basalt with silica fume greatly enhances the thermal stability of OPC paste up to 600°C. The binary pozzolana system is better than one pozzolana. The

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cement blend composed of about 80% OPC, 5-10% basalt and 5-10% silica fume was recommended to resist firing.

Limestone filler improves the fine-particle packing of cement granular skeleton and a larger dispersion of cement grains which can considerably enhance stability and workability of fresh concrete [7]. It may significantly improve several cement properties such as mechanical properties, water demand, workability as well as durability [8,9]. Limestone filler accelerates the hydration of Portland clinker minerals (especially the C_3S) at early ages, modifies the Ca/Si ratio of C-S-H and forms calcium carboaluminate hydrates, resulting in an improvement in early strength [10,11]. It also acts as the nucleation sites for the precipitation of CH and CSH reaction products at early ages of hydration [12] and it reacts with C_3A to form calcium carboaluminate hydrate [13].

It is known that limestone decomposes at 700-900°C [14] and there is a lake of knowledge about the role of limestone filler in pozzolanic cements with regard to fire resistance. The aim of this work is to investigate the effect of limestone filler on the hydration characteristics as well as the fire resistance of basalt-silica fume composite cement pastes up to 800°C.

EXPERIMENTAL WORK

The starting materials used in this work were OPC, limestone, Silica fume and basalt. The chemical analysis of the starting materials is shown in Table 1. The basalt and limestone were ground to pass 90μm sieve. Mix. composition, mass% of the investigated composite -filled cements is shown in Table 2.

Table 1: Chemical analysis of the starting materials, mass%.

Oxide	OPC	Basalt	Silica fume	Limestone
SiO_2	20.88	50.19	94.64	0.26
Al_2O_3	6.08	13.56	0.97	0.16
Fe_2O_3	3.18	11.28	0.93	--
CaO	63.00	10.07	0.55	54.59
MgO	1.50	4.24	0.35	0.29
SO_3	1.60	0.04	0.10	0.05
Na_2O	0.02	3.06	0.20	0.11
K_2O	0.04	0.49	0.25	0.03
L.O.I	--	4.41	2.01	43.72

Table 2: Mix composition of investigated composite -filled cements, mass%.

Symbol	OPC	Basalt	Silica fume	Limestone
B1	100	00	00	00
B3 (Control Mix.)	80	20	00	00
X1	80	10	05	05
X2	80	05	05	10
X3	80	05	10	05
X4	80	00	10	10

Each dry mix was homogenized in porcelain ball mill using four balls for one hour in a mechanical roller to obtain complete homogeneity. The water of consistency, initial and final setting times of each blend were determined according to ASTM specifications [15,16]. The dry mix was mixed using the corresponding water of consistency. To study the hydration characteristics of cement pastes, the specimens were demoulded after 24h casting and cured under tap water. Bulk density was determined from weight both in air and suspended in water of the hydrated pastes using Archimedes principle [17]. The determination of compressive strength of hardened cement pastes was carried out according to ASTM Designation: C 109-80 [18] using a manual compressive strength machine. After the compressive strength determination, the broken pieces of each cube were used for stopping the hydration using microwave drying technique [19]. The combined water (Wn), was determined from the loss in weigh of dried samples by ignition at 1000°C. The total porosity of the hardened cement pastes was calculated depending on bulk density and the water content results [20].

To investigate the fire resistance of cement pastes, another set of specimens were demoulded after 24h casting and cured under tap water for 28 days, dried at l05°C for 24h. A group of three cubes from each mix were fired in furnace with rate 20°C/min and kept for 2hrs at each temperature in the rang 200-800°C at intervals of 100°C and allowed cool to room temperature in the furnace. The ignition loss of each mix at all treatment temperatures was determined on the ignited weight bases. The compressive strength was determined using a hydraulic press. Chips of cubes were used for determination of bulk density and total porosity after immersing overnight under kerosene (ISO 5018:1983).

RESULTS AND DISSCUSION

1- Hydration Characteristics of Composite - Filled Cement Pastes

1.1- Water of consistency and setting time

The water of consistency, initial and final setting times of composite -filled cement pastes made from OPC, basalt, silica fume and limestone as a function of mix composition are graphically plotted in Fig. 1. It was found that water of consistency of all pozzolanic-filled cement pastes are higher than that of control cement paste as a result of silica fume which greatly affects the water demand of cement paste. The water of consistency increases by substituting basalt with silica fume more than with limestone. This is due to that silica fume has very high specific surface compared to limestone. The setting times for all composite -filled cement pastes are shorter than that of the control cement paste due to the presence of limestone and silica fume that shorten the setting time. The silica fume reacts with CH and the limestone acts as filler or as nucleating agent that accelerates the rate of hydration. Therefore, the setting time of cement pastes with silica fume and limestone is accelerated.

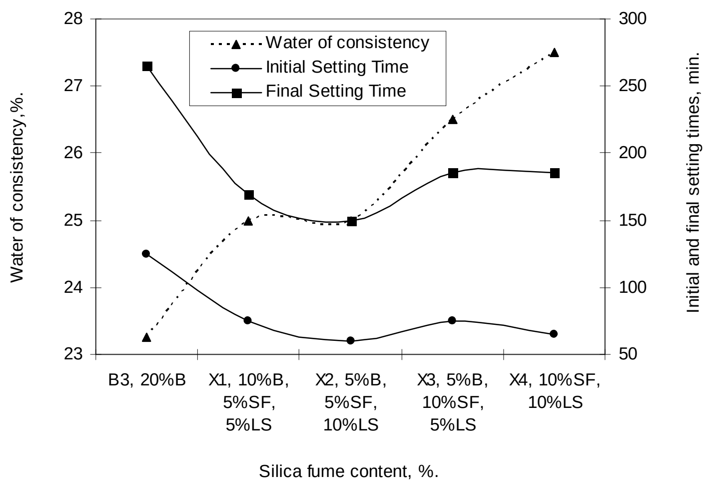

Fig. 1: Water of Consistency, Initial and Final Setting Times of Composite- Filled Cement Pastes as a Function of Mix Composition

1.2- Chemically combined water content

The combined water content of composite -filled cement pastes made from OPC, basalt, silica fume and limestone as a function of curing time up to 360 days are graphically plotted in Fig. 2. The combined water content increases with curing time for all cement pastes due to the progress of hydration. The combined water content for all cement pastes increases sharply after both 7 and 90 days due to that at early ages both of silica fume and limestone behaves like an inert material diluting the portland cement. Befor 7 days silica fume starts to react with CH librated from the hydration of clinker to form additional hydrated cementitious products. After 90 days limestone reacts with C_3A to forms calcium carboaluminate hydrates.

The combined water contents of the cement pastes containing silica fume and limestone are higher than those with basalt only at all hydration ages. The combined water content for all cement pastes increases with partial substitution of basalt by both silica fume and limestone. This is due to that silica fume is a very active pozzolana that fills some of pores of the paste so it activates the hydration of C_3S and reacts with CH librated from the hydration of clinker to form additional hydrated cementitious products. Also, limestone reacts with C_3A to form calcium carboaluminate hydrates which contain higher amount of water than the calcium silicate hydrates.

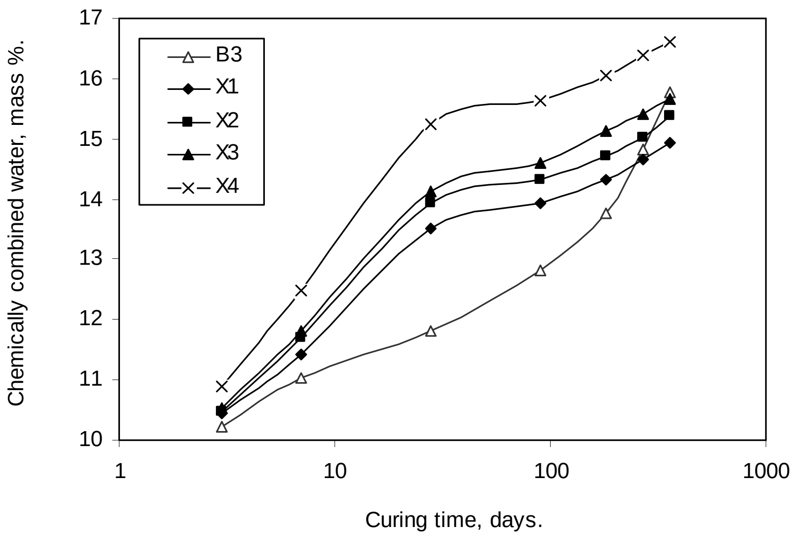

Fig. 2: Combined Water Content of Composite - Filled Cement Pastes as a Function of Curing Time Up to 360 Days

1.3- Bulk density and total porosity

The bulk density and total porosity of composite -filled cement pastes made from OPC, basalt, silica fume and limestone as a function of curing time up to 360 days are graphically plotted in Fig. 3. The bulk density increases whereas the total porosity of cement pastes decreases with curing time up to 360 days for all cement pastes as a result of hydration of clinker phases and pozzolanic reaction. As the hydration progresses the hydration products fill some of pores that tend to decrease the porosity and increase the bulk density. The bulk density is greatly related to both basalt content and the amount of mixing water. As the basalt content decreases and mixing water increases the bulk density decreases. Therefore, Mix. B3 with the highest basalt content and less mixing water has the highest values of bulk density compared to the other mixes. Whiles, Mix. X4 with 0 % basalt and 10% of both silica fume and limestone has the lowest values of bulk density as a result of the high value of water of consistency of this mix.

The total porosity increases with silica fume content for all composite -filled cement pastes due to the high specific surface area of silica fume which needs more mixing water. The water demand of cement paste increases with silica fume content so the total porosity increases. The sharp decrease of total porosity after 7 days reflects the pozzolanic activity of silica fume which consumes CH to form additional hydration products.

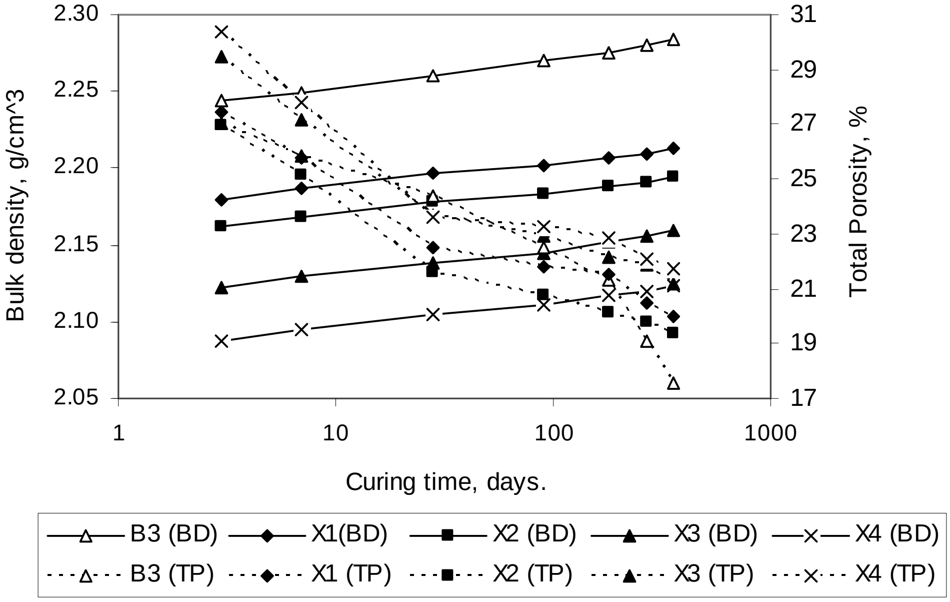

Fig. 3: Bulk Density and Total Porosity of Composite - Filled Cement Pastes as a Function of Curing Time Up to 360 Days

1.4- Compressive strength

The compressive strength of composite cement pastes as a function of curing time up to 360 days are graphically plotted in Fig. 4. The compressive strength increases up to 360 days for all cement pastes due to the progress of hydration and formation of more cementing materials leading to increase the compressive strength. The compressive strength for all cement pastes increases primary with basalt and secondary with limestone contents due to the pozzolanic activity of basalt, combined filling and latent pozzolanic role of limestone and decrease the water of consistency.

It can be concluded that composite -filled cement pastes containing 5-10wt% basalt, 5% silica fume and 5-10% limestone show an ultimate compressive strength comparable to that of the control cement paste due to the combined pozzolanic activities of basalt, silica fume and that of limestone at later ages as well as the decrease of water of consistency, increase of bulk density and decrease of total porosity.

2- Fire Resistance of Basalt–Limestone-Silica fume Composite -Filled Cement Pastes

2.1- Ignition loss

The ignition loss of OPC and composite -filled cement pastes as a function of firing temperature up to 800°C are graphically plotted in Fig. 5. Substitution of basalt with other materials such as limestone and silica fume increases the weight loss at all firing temperatures due to the increase of mixing water as well as the decomposition of limestone after 600°C. The weight loss is sharply increased after 600 up to 800°C due to the decomposition of limestone. X3 cement paste (80% OPC, 5% basalt, 10% silica fume and 5% limestone) exhibits the higher weight loss up to 600°C.

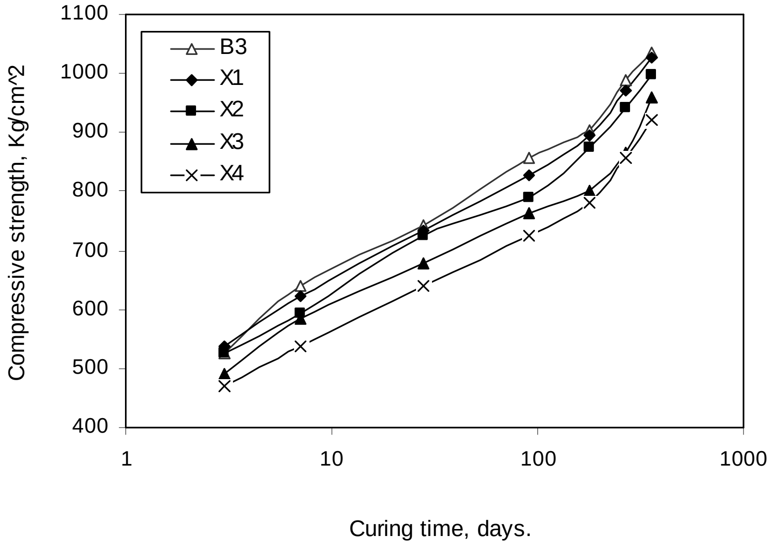

Fig. 4: Compressive Strength of Composite - Filled Cement Pastes as a Function of Curing Time Up to 360 Days

As the amount of limestone increases X4 (80% OPC, 10% silica fume and 10% limestone) the weight loss increases after 600°C, due to the decomposition of more limestone content. Cement paste with only 20% basalt (B3) shows the lower weight loss of all cement pastes due to the low water of consistency.

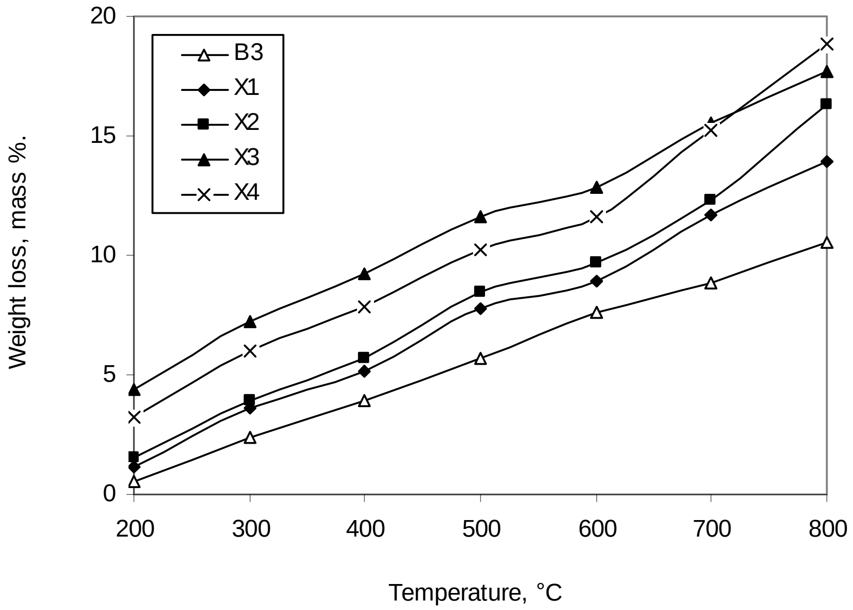

Fig. 5: Ignition Loss of OPC and Composite - Filled Cement Pastes as a Function of Firing Temperature Up to 800°C

2.2- Total porosity

The total porosity of OPC and pozzolanic cement pastes made from OPC, basalt, silica fume and limestone as a function of firing temperature up to 800°C are graphically plotted in Fig. 6. Total porosity of all mixes decreases sharply with firing temperature in the range 25-200°C. This is mainly due to that the treatment temperature enhances the hydration of cement clinker phases, the reaction of basalt and silica fume with CH librated from the hydration as well as limestone with C_3A to form hydration products that fill some pores of the cement matrix. Therefore, the total porosity decreases. The total porosity is nearly the same from 200 up to 400°C due to that no further loss of free or bound water is occurred. After 400°C, the total porosity of all cement pastes increases sharply due to the decomposition of CH librated during the hydration of Portland cement in addition to the decomposition of limestone as well as the partial decomposition of the hydration products. The calcinations of $CaCO_3$ gives CO_2 gas which tends to increase the total porosity of the cement paste containing limestone.

X2 cement paste (80% OPC, 5% basalt, 5% silica fume and 10% limestone) gives the higher values of total porosity up to 600°C. This is mainly due to the presence of 10% limestone. The calcium carboaluminate decomposes and gives CO_2 and CaO. On the other side X4 with 10% limestone and 10% silica fume gives the higher total porosity at 800°C due to the increase of water of consistency in addition to the increase of limestone content. The two factors tend to increase the total porosity from the decomposition of limestone in addition to the increase of free water which affects the total porosity. Also, total porosity increases with firing temperature from 400-800°C for all mixes as a result of the formation and enlargement of microcracks as well as the dehydroxylation of CH and the calcinations of limestone

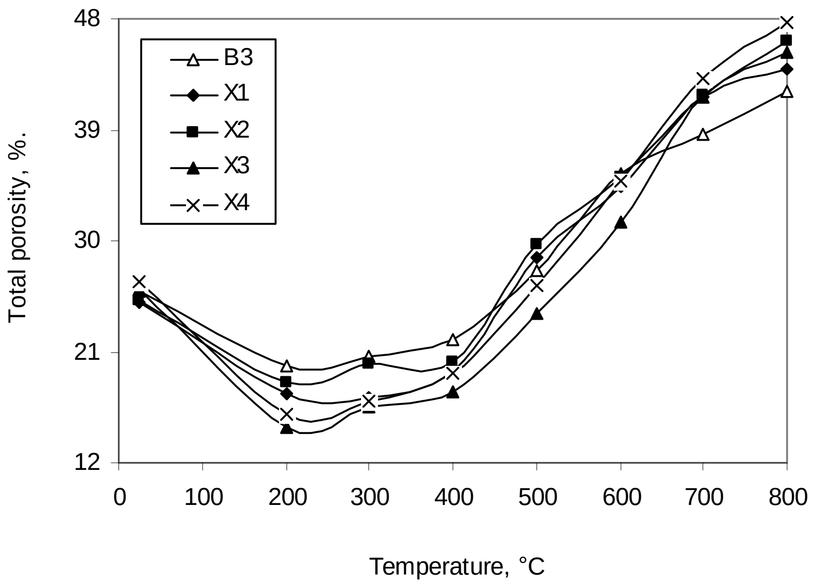

Fig. 6: Total Porosity of OPC and Composite -Filled Cement Pastes as a Function of Firing Temperature Up to 800°C

2.3- Bulk density

The bulk density of OPC and composite - filled cement pastes made from OPC, basalt, silica fume and limestone as a function of firing temperature up to 800°C are graphically represented in Fig. 7. The bulk density of all cement pastes decreases sharply from room temperature up to 200°C. This is mainly also due to the removal of free as well as weakly bound water leaving pores that tends to decrease the density of the cement pastes. It is clear that the bulk density decreases with firing temperature for all mixes as a result of the formation and enlargement of microcracks results from internal cracks and the dehydration of CH crystals.

Basalt acts as a nucleating agent that accelerates the hydration of cement, consumption of CH by silica fume to form CSH as well as consumption of limestone through reaction with C_3A to form calcium carboaluminate hydrates as shown from the high bulk density values of X1 mix (80% OPC, 10% basalt, 5% silica fume and 5% limestone). Replacement of basalt by either silica fume or limestone reduces the bulk density due to the increase of water of consistency which increases the total porosity and then decreases the bulk density. The limestone has lower density than basalt in addition to its decomposition to CaO and CO_2. As the limestone and silica fume contents increase on the expense of basalt the bulk density decreases. The sharp decrease of bulk density of X2 at 800°C is mainly due to the increase of limestone content (10%) which is decomposed at 800°C to CaO and CO_2. Also, X4 behaves in similar behavior but with lower bulk density than X2 due to the increase of silica fume in addition to 10% limestone.

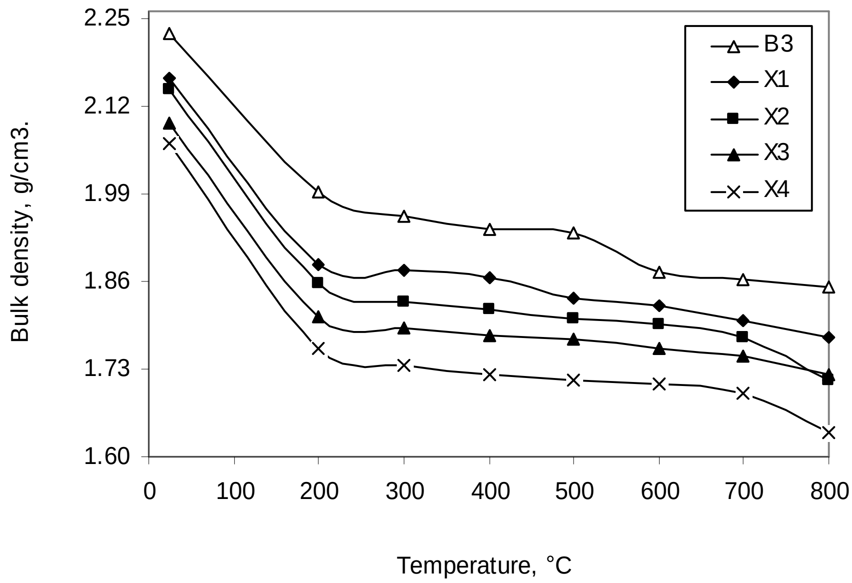

Fig. 7: Bulk Density of OPC and Composite -Filled Cement Pastes as a Function of Firing Temperature up to 800°C

2.4- Compressive strength

The compressive strength of OPC and composite -filled cement pastes made from OPC, basalt, silica fume and limestone as a function of firing temperature up to 800°C are graphically plotted in Fig. 8. The compressive strength of hydrated cement paste increases sharply up to 200°C as a result of enhancing the hydration of unhydrated cement clinker, as well as the pozzolanic reactions of basalt and silica fume with CH to form extra CSH. Limestone reacts with C_3A to form calcium carboaluminate hydrates. Also, this is due to the self autoclaving which accelerates the hydration of cement pastes.

The compressive strength of all blended cement pastes shows nearly high thermal stability in the temperature range 300-600°C compared to OPC with 20% basalt which shows steep decrease of compressive strength. This can be attributed to the pozzolanic activity of basalt and silica fume reducing the amount of CH and due to the thermal stability of the hydration products in this temperature range. The compressive strength of all cement pastes increases with silica fume content. This is mainly due to that silica fume acts as nucleating agent and accelerates the hydration of cement, reacts with librated CH to form CSH in conjugation with basalt or even in absence of basalt as shown from the high compressive strength values of X3 as well as X4 respectively. The compressive strength of all composite cement pastes decreases sharply after 600°C as a result of the decomposition of limestone and due to the dense microstructures, which led to the buildup of internal pressure. Also, the cement contains 5% limestone which decomposes to CaO and CO_2 that gives higher strength than the control cement which contains only 20% basalt. This is mainly due to that the decomposed CaO reacts with the SiO_2 and Al_2O_3 giving solid state cementatious bodies. Therefore the compressive strength increases.

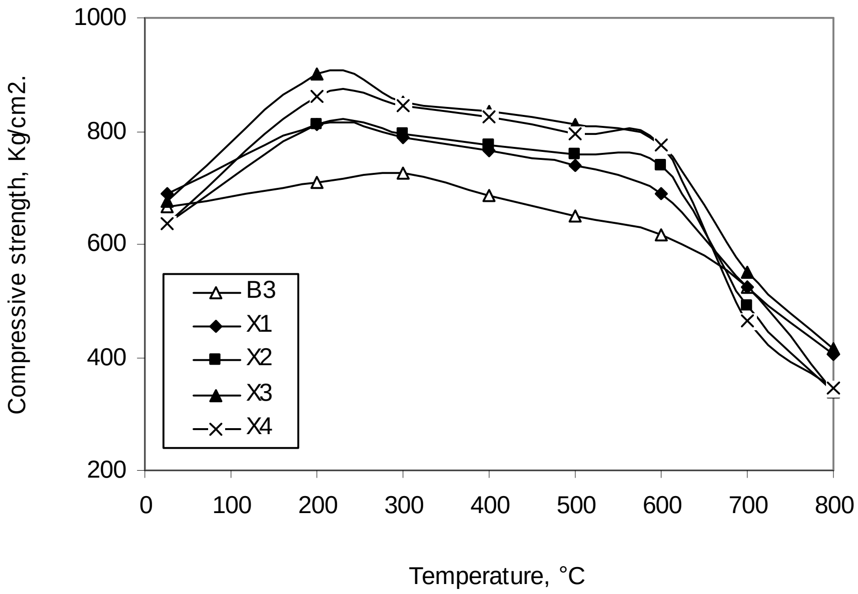

Fig. 8: Compressive Strength of OPC and Composite -Filled Cement Pastes as a Function of Firing Temperature Up to 800°C

2.5- Phase composition

XRD patterns of composite cement pastes made from OPC and 20% basalt fired at different temperatures up to 800°C are shown in Fig. 9. The results indicate that the CSH peak decreases where as that of calcium silicate minerals larnite and hartrurite increase with firing temperatures. This indicates that CSH gel transforms at high temperature to the thermally stable forms larnite and hartrurite minerals [21]. The peak of larnite exceeds that of hartrurite at 800°C. This indicates the thermal stability of the former than the later mineral. The intensity of CH peaks decrease with firing temperature due to thermal decomposition of CH as well as partial conversion of CH to CC as indicated from the intensity of CC peak at 500°C. This may be due to the physico-chemical processes connected with the liberation of water from hardened cement pastes in the temperature range 200-500°C which are accompanied by a partial carbonation of CH [22]. The presence of portlandite at 800°C is mainly due to the rehydration of some decomposed portlandite. The intensity of basalt minerals as albite and augite peaks increase with firing temperatures up to 800°C. This indicates the thermal stability of basalt minerals.

XRD patterns of the pozzolanic hardened cement pastes of OPC, B3, X3 and X4 fired at 500°C are shown in Fig. 10. The results indicate that the addition of basalt has no clear effect on the of CH peak while it decreases to a minimum value with silica fume addition. This is mainly due to the rehydration of some decomposed portlandite and basalt acts as nucleating agent accelerating the hydration of OPC. Also, this indicates that basalt has a slow pozzolanic activity in comparison with silica fume because it does not consume the liberated CH at early ages up to 28 days. Addition of limestone filler refines the microstructure of hardened cement pastes and improves the pozzolanic reaction of basalt and silica fume with CH.

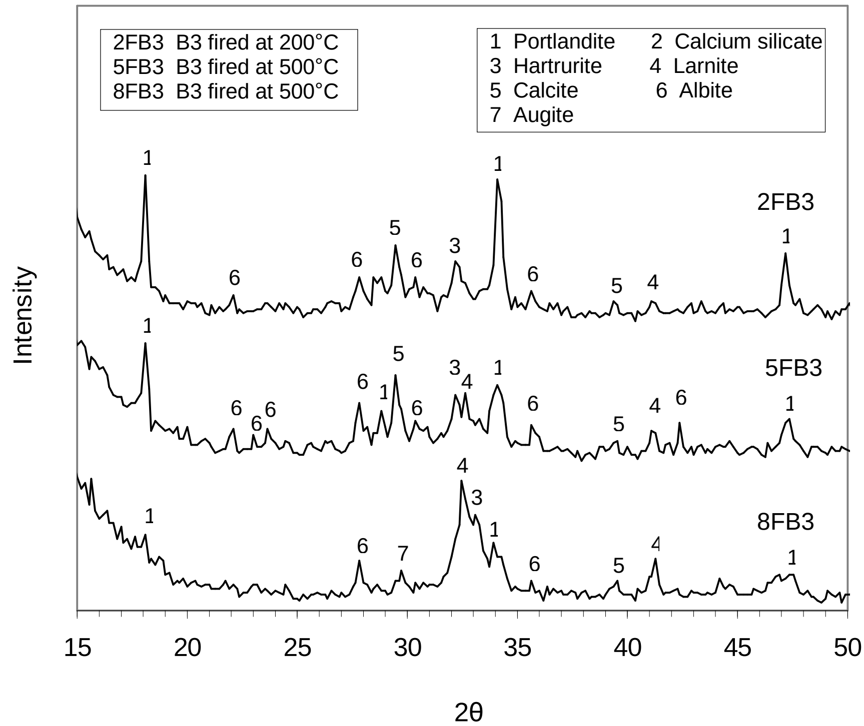

Fig. 9: XRD Patterns of Pozzolanic Hardened Cement Pastes Made from OPC and 20% Basalt Fired at Different Temperatures Up to 800°C

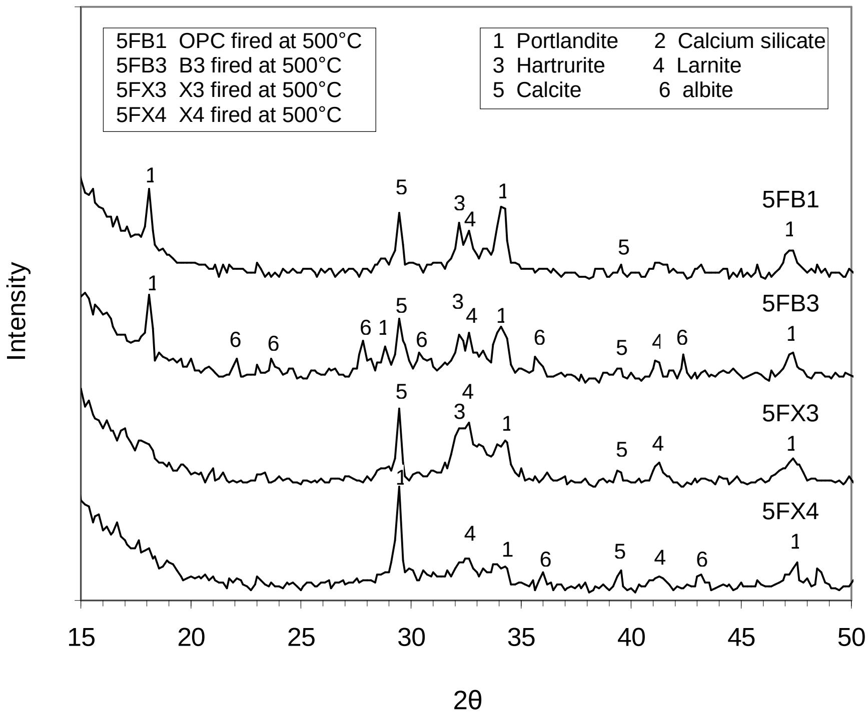

Fig. 10: XRD Patterns of the Composite -Filled Cement Pastes of OPC, B3, X3 and X4 Fired at 500°C

2.6- Visual inspection

The typical visual inspection of the investigated hardened OPC, pozzolanic as well as pozzolanic-filled cement pastes fired at different temperatures is shown in Fig. 11. It was observed that small surface cracks took place in OPC as well as B3 (80 % OPC+20 %basalt) hardened cement pastes fired at 500°C. This is mainly due to the physico-chemical processes connected with the liberation of water from hardened cement pastes in the temperature range 200-500°C. At 800°C, hardened cement paste made from OPC is completely destroyed as a result of the rehydration of decomposed portlandite. It was noticed that basalt improves the fire resistance of hardened cement paste. This is mainly due to that basalt consumes portlandite forming additional CSH gel which transform at higher temperature to the thermally stable larnite and hartrurite minerals as indicated from the XRD patterns of hardened cement pastes made from OPC and basalt.

It was seen that cement pastes made from OPC, basalt, silica fume and limestone fired at 500° and 800°C exhibit good fire resistance. This is mainly due to that both silica fume and limestone filler refine the microstructure of hardened cement paste. Also, silica fume together with basalt consume liberated portlandite. It can be concluded that binary pozzolana system is more effective than the single pozzolana under fire. Also, the presence of filler enhances the fire resistance of hardened cement paste containing binary pozzolana system.

It was found that the portlandite content diminished and CSH content enhanced through the hydration of pozzolanic-filled cement pastes made from 80% OPC, 5-0% basalt, 10% silica fume and 5-10% limestone. As a result, these mix compositions exhibited the higher fire resistance behavior and suffer from little cracking after exposure to fire up to 600°C in comparison with all other cement pastes as seen from the compressive strength values.

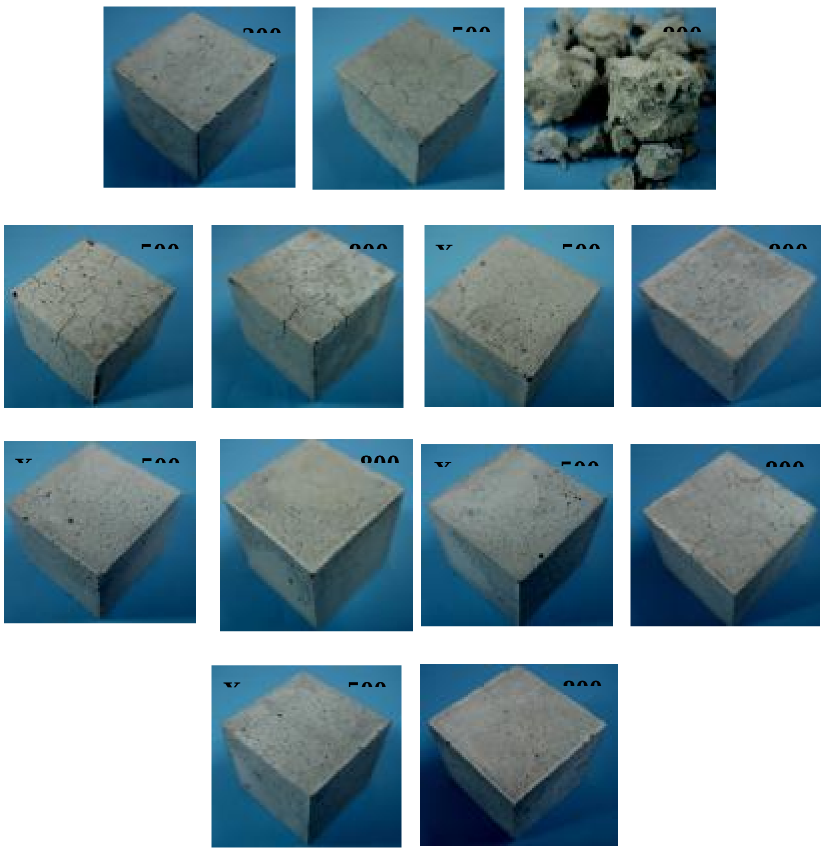

Fig. 11: Typical Visual Inspection of the Investigated Hardened OPC and Composite - Filled Cement Pastes Fired at 300°, 500° and 800°C

CONCLUSIONS

From the above findings it can be concluded that:

1- Partial replacement of basalt both by silica fume and limestone does not offer additional enhancement for the hydration characteristics of used control cement paste. The composite -filled cement paste containing about 10-5wt% basalt, 5% silica fume and 5-10% limestone shows an ultimate compressive strength comparable to that of the control cement paste (20% basalt).

2- Partial replacement of basalt by silica fume in presence of limestone does not reduce the fire resistance of hardened cement pastes up to 600°C due to the filling role of limestone.

3- Composite -filled cement pastes containing 80% OPC, 5-0% basalt, 10% silica fume and 5-10% limestone can be used as fire resisting cement.

AKNOWLEDGMENT

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HEAT ENDURING CEMENT-GLASS MORTAR

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ABSTRACT

The present work is an experimental investigation concerned with developing plastering cement mortar characterized by an accepted heat endurance to act as a heat barrier to rather sensitive materials like Advanced Composite Materials (ACM). For this purpose, finely ground waste glass and finely ground granulated blast furnace slag were introduced into the mortar mixtures. A total of eleven mixtures were cast, each of which comprises six groups of mortar cubes. For all mixtures, compressive strength is evaluated at ages extending from one week to three months. After 28 days of water curing, the mixtures' compressive strength was evaluated after exposure to elevated temperatures ranging from 200 ^{0}C to 800 ^{0}C. The retained strength after heat exposure is regarded as the heat endurance measure. A mineralogical study was conducted on the mortar specimens before and after exposure to 800 ^{0}C. As a consequence, the formed cementitious phases and the heat induced transformations are clarified. Test results show that by employing either the ground glass or the ground slag and owing to their pozzolanic nature and superior thermal stability a recognized enhancement in strength and heat endurance could be achieved.

Keywords: Heat endurance, Cement mortar, Ground slag, Ground glass, Advanced Composite Materials.

1. INTRODUCTION

Nowadays, the use of advanced composite material (ACM) like carbon fiber-epoxy resin system to retrofit or strengthen the structural elements is a common practice in many countries. The application of ACM are expected to grow more with the reduction in its cost. However, its resistance to fire and elevated temperature is questionable, because of the known low melting point of the employed resins. Hence providing an external fire barrier is an essential requirement for adequate performance. Through the present work, the ground waste glass (GWG) is introduced in the mortar mixes as a partial replacement of sand to improve its heat endurance. The finely ground glass was reported to have a dual effect on the generated mortar. One regarding its pozzolanic nature, as it combines with some of the calcium hydroxide (CH) liberated during cement hydration to form the cementitious product calcium-silicate-hydrate (CSH), which leads to strength enhancement. Secondly, improving the resistance to elevated

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temperature due to the formation of wollastonite mineral (Ca Si O_3) with robust crystalline structure embedded in the glass matrix offered by glass softening criteria under high thermal loads [1].

Plastering with glass cement-mortar as an external heat barrier was reported to have an excellent protection to reinforced concrete slabs strengthened by externally bonded ACM strips and heated with temperature up to 570 ^oC [2].

Ground granulated blast furnace slag (GGS) is also employed through the present investigation as a partial replacement of the mixtures sand, either alone or in combination with GWG. The GGS in addition to its pozzolanic activity is contributing to heat endurance.

2. Experimental Program

2.1 Materials

All mixtures were prepared employing natural siliceous sand has a fineness modulus of 2.6, and an ordinary Portland cement of Blain surface area 3200 cm^2/ gm, complying with the Egyptian standard specification (ESS) 1109-2002 for aggregate, and ESS 4756-2005 for cement.

The used waste glass is obtained from fluorescent lamps industry. It was finely ground to a powder form with grain size ranging from 200 μm to 75 μm. The granulated blast furnace slag was delivered from El-Tebin factory of steel in the form of water quenched fine grains. The slag was then finely ground in a laboratory ball mill, the fraction passing 100 μm sieve is used through this work. Table 1 shows the chemical composition of the employed mineral admixtures. It is worth to mention that using the GWG in a powder form is an efficient way to overcome the vulnerability of alkali-silica reaction since glass is not stable in the alkaline media offered by the hydrated cement [3,4]. With increasing the fineness of GWG, its efficiency would be improved due to increasing its pozzolanic activity. This would also help to refine the pore structure at elevated temperatures, as the remaining part after the pozzolanic reaction would go soft and act as a binder for the dehydrated cement paste constituents [1].

Table 1: Chemical Composition of the Employed Mineral Admixtures

Oxide	fluorescent glass	blast furnace slag
Si O_2	74	23.47
Al_2 O_3	2.3	9.05
Fe_2 O_3	0.1	3.82
Ca O	6.2	52.79
Mg O	3.11	5.39
Na_2 O	12.82	0.52
K_2 O	0.43	0.31
S O_3	0.01	1.78
L.O.I.	1.03	1.41

2.2 Mixes features

The adopted design of the mortar mixtures is based on the following considerations:

- All the mixtures are normally designed and no attempt was made to produce high strength mortar. The reason of that, to produce a moderate void ratio which facilitate the escape of built up pore pressure during exposure to elevated temperature.
- The cement content was kept constant through all mixtures, and the ground glass and/or the ground slag were employed as a partial replacement of sand. In this way a satisfactory level of strength would be acquired. Also the richness of the mortar mixture in cement would facilitate plastering, which is the proposed method of applying the external heat barrier.
- The choice of ground glass was based on its established contribution to heat endurance of the cementitious mixtures. The ground slag selection was based on the expected stability at elevated temperatures, and the utilization of its pozzolanic nature for pore refinement and strength enhancement.

The proportions of all the mixtures are identical; the water – cement ratio and the sand – cement ratio were maintained constant at 0.485 and 2.75, respectively. The GWG was employed as a replacement of sand by five ratios: 20%, 40%, 60%, 80%, and 100%. The GGS was used as a replacement of 20% of the sand weight in one mix. Ternary blends of cement, GGS, and GWG were also investigated. The slag (GGS) and the glass (GWG) were used as a sand replacement, the first one at a constant ratio of 20%, while the second one with four ratios of 20%, 40%, 60%, and 80%. Proportions of all mixtures are outlined in Table 2.

Table 2: Experimental plan- Mixes proportions & types of mineral admixtures and the replacement ratio

Mix	Proportions of mortar mixtures			Replacemnt of sand, %	
	Cement	Sand	Water	GWG	GGS
Mc	1	2.75	0.485	_	_
M1				20	_
M2				40	_
M3				60	_
M4				80	_
M5				100	_
M6				_	20
M7				20	20
M8				40	20
M9				60	20
M10				80	20

3. METHODS OF INVESTIGATION

In this work, it was tried to produce cement mortar mixture with sufficient ability to withstand elevated temperature (heat enduring). These mixtures can be beneficially utilized in protecting ACM as mentioned before. For this purpose, the following tests were conducted:

- Compressive strength at three different ages; namely 7, 28, and 90 days. The compressive strength at each age is the average of testing three companion cubes with 50 mm side length.
- The retained strength after elevated temperature exposure. This test is the prime test in this investigation. It measures the strength retrogression normally associated with thermal loads, or in other words the retained strength that are hoped to be large enough to appreciate the investigated mortar mixtures.

Three temperatures were adopted, namely 200 ^{0}C, 600 ^{0}C, and 800 ^{0}C. The testing procedure sequenced as follows: 9 mortar cubes with 50 mm side length in addition to the 9 cubes for compressive strength evaluation were cast for each of the mixtures presented in Table 2. After 28 days of water curing, all the specimens were prepared by drying them at oven at 100 ± 5 ^{0}C. The specimens were then exposed to the predetermined temperatures with 100 ^{0}C interval. After 3 hours of exposure to the elevated temperature, the electrical furnace was turned off and specimens were allowed to cool down for about 2 hours at the furnace environment. The specimens were then brought out the furnace and left in the room environment to cool down. The specimens were then tested in compression and their results were compared with the corresponding strengths after 28 days of water curing.

- Mineralogical study

The X-ray Diffraction Technique (XRD) is adopted to identify the different cementitious phases formed after curing for 28 days, i.e. before exposure to elevated temperature, due to introducing the mineral admixtures as a partial or full replacement of the mix's sand. Also, the changes in the cementitious phases after exposure to 800 ^{0}C temperature (chemical transformations) are thoroughly investigated. The control mixture and three other ones, namely M3, M5, and M9 were selected for testing. After performing the compressive strength test, the crushed mortar cubes of each of the selected mixes were finely ground and thoroughly mixed. Then a representative sample corresponding to the selected mixtures was taken and ground to a very fine powder that passes (75µm) sieve and was tested immediately after that.

4. RESULTS & DISCUSSION

4.1 Compressive strength

The effect of replacing the sand with the mineral admixtures GWG, and GGS having different pozzolanic activities is investigated for different replacement ratios. Figs. 1, 2, and 3 show respectively the 7 day, 28 day, and 90 day compressive strength for the control mixture and the first group of mixtures incorporating GWG and the second group with both GWG and GGS, from which the following observations are drawn.

- The GGS has a relatively higher pozzolanic activity than the GWG at least at early ages, as could be realized from comparing the strengths of the corresponding mixtures of the two groups.
- The compressive strengths of the mixtures containing GWG show a limited enhancement in strength at 7 days age. Their strengths were up to 9% higher than that of the control mixture at 60% replacement ratio, beyond which their strengths suffered reduction. At 28 days age, their strengths followed more or less the same trend as at 7 days, but at 90 days age the enhancements in strength were recognized at all replacement ratios with a maximum increase of 33% over the control mixture strength.
- The mixtures containing GGS in addition to the GWG showed an increase in their compressive strength over that of the control mixture till replacement ratios of 60%, 80%, and 100% at 7 days, 28 days, and 90 days age respectively. The maximum recorded increases

were 18%, 35%, and 40% at these testing ages corresponding to the mixture M6 with 20% GGS. The other replacement ratios, i.e. with 20% GGS and different ratios of GWG exhibited satisfactory strength enhancement although less than the mixture M6.

It may be concluded that whenever the compressive strength is a concern, the GWG owing to its pozzolanic nature can be favorably employed as a replacement of sand either partially or fully, thus justifying the desired strength enhancement and at the same time recycling the waste glass which is an environmental issue. The fineness of the ground glass apparently is an important parameter influencing the pozzolanic activity or the rate of strength gain. Further enhancement of the mixture could be achieved by adding another type of pozzolana like GGS as could be seen from test results.

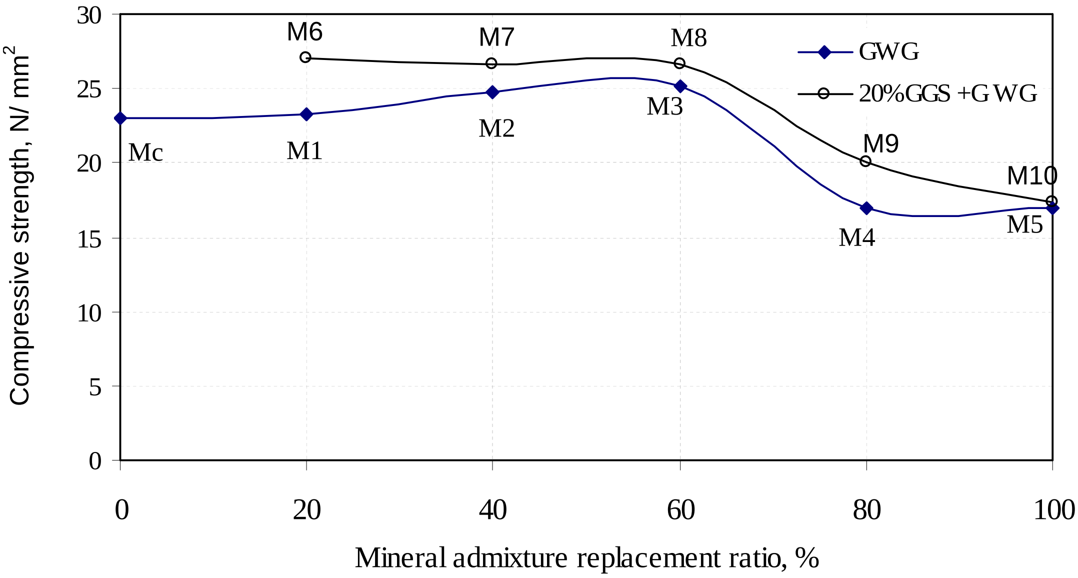

Fig. 1: 7 Day Compressive Strength of the Investigated Mortar Mixtures

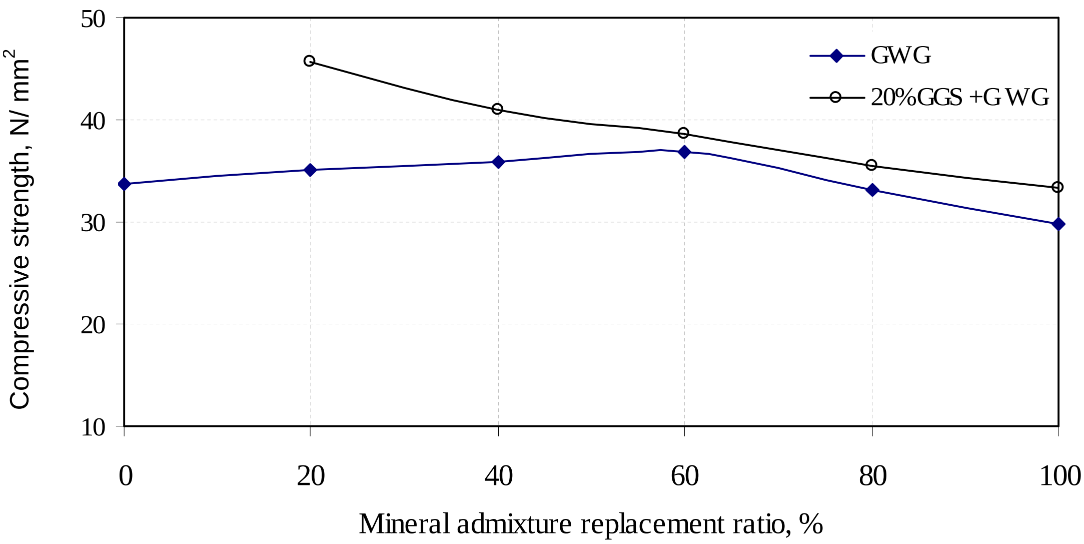

Fig. 2: 28 Day Compressive Strength of the Investigated Mortar Mixtures

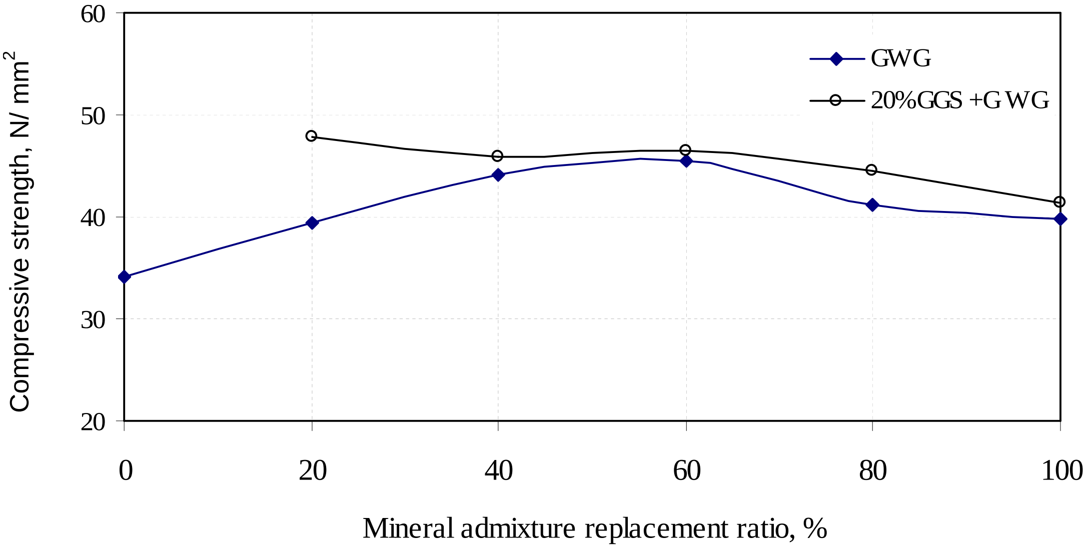

Fig. 3: 90 Day Compressive Strength of the Investigated Mortar Mixtures

4.2 Retained compressive strength after thermal loading

The behavior of the mortar specimens under different thermal loads or heat exposure temperatures is investigated, the results are presented in Figs. 4, 5, and 6 for temperatures of 200 ^{0}C. 600 ^{0}C, and 800 ^{0}C respectively. The test results are presented in two different forms in each of these figures; either as the retained compressive strength values after heat exposure, or the ratio of these strengths to the 28 day ones (before heat exposure). Fig. 4 shows that after heating to 200 ^{0}C, the retained strength ratio of all mixtures including the control one is higher than the 28 day compressive strength. That is partly attributed to the evaporation of free water which leads to friction increase between failure planes, or possibly this level of heat catalyze hydration of the non-reacted cementitious products. i.e. advancement in chemical bounding process. Anyhow, the ratio of strength increase is more pronounced for the waste glass (WG) group of mixtures although the strength of the other group of mixtures incorporating 20% slag is still the highest.

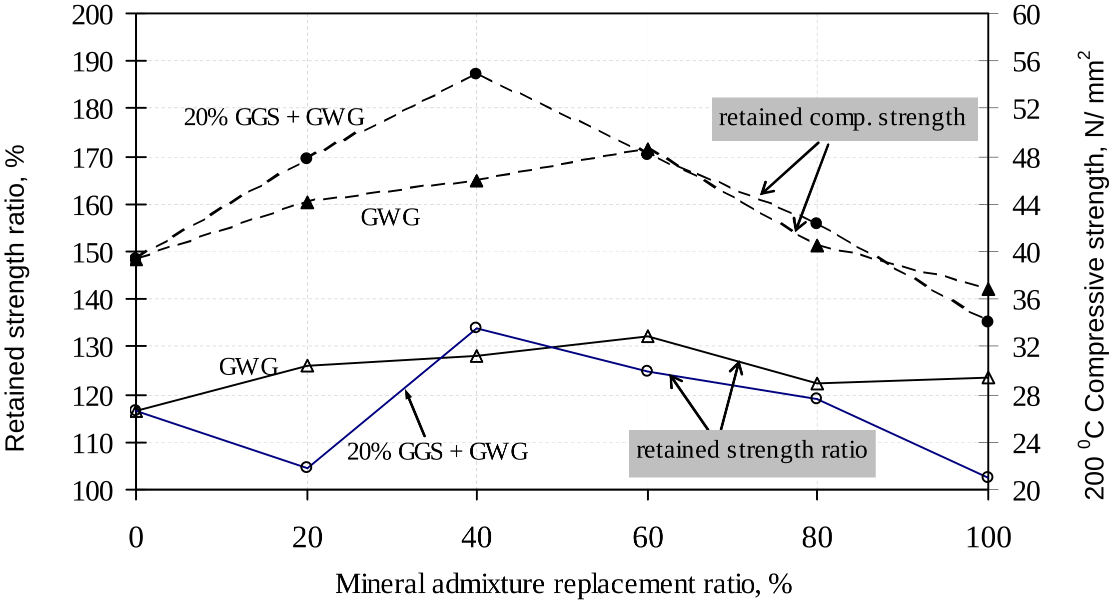

Fig.4: Retained strength after 200 ^{0}C heat exposure

Heating the mortar specimens to higher temperatures is normally associated with multiple chemical and physical transformations which would affect the stability of the internal structure and consequently the strength. The most recognized causes of these transformations include; dehydration (decomposition) of the cementitious compounds, different expansion values of the constituents (thermal mismatch), and internal pore pressure. The last effect is thought to be alleviated in this investigation due to adopting a relatively high water/ cement ratio which would generate more connected pores. This remark is confirmed by the test results as no spalling or disintegration was noticed in any of the tested samples upon heating. Therefore, the observed variations in the mortar mixtures compressive strengths are attributed to the other two causes of transformation.

At 600 ^{0}C, the possible chemical transformations include; decomposition of the cementing compound CSH with its different phases, dehydration of calcium hydroxide (CH) into free lime, and $\alpha - \beta$ quartz transformation. These changes would affect the volume occupied by these cementitious products and when combined with the weakened cohesion between the mixture constituents due to the different expansions experienced by each of them they may develop micro cracks or hairs inside the mortar mass and consequently degradation of the compressive strength would result.

Fig. 5 shows that the retained strength of the control mixture is drastically reduced to 47.6 % of the 28 day compressive strength after heating to 600 ^{0}C. On the other hand, the mixtures containing WG were able to retain a considerable part of their strengths ranging from 65% to 79%. Moreover, the retained strengths are in direct proportion to the GWG content. Adding 20% slag in beside the WG, the second group of mixtures, efficiently enhanced the retained strength values till 80% total replacement ratio. The retained strengths ranged from 62% to 95%. Apparently the molten glass is efficient in healing the heat-developed micro cracks and flows. Also, the GWG is able via its pozzolanic nature to combine with some of the free CH to form the cementing compound CSH, and thus reducing the amount of CH as a main source of instability upon heating. It was explained[1] that decomposition of CH into CO and the subsequent reaction between it and the humidity from the surrounding air during specimen cooling is likely to cause micro cracks due to the volume increase associated with this process. The pozzolanic activity which reduces the CH partly explains the superior resistance of the mixtures incorporating ground slag to degradation at elevated temperatures.

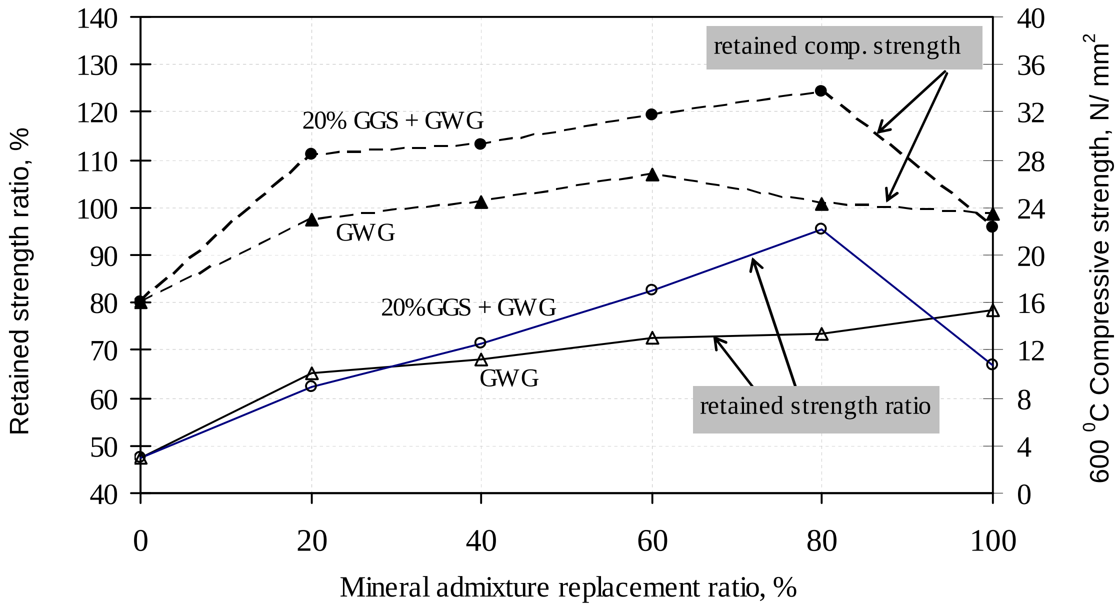

Fig.5: Retained strength after 600 ^{0}C heat exposure

Further heating the mortar specimens to 800 ^{0}C would result in more chemical and physical transformations. For instance, decomposition of different forms of calcium carbonate, re-crystallization of new compounds, and also more relative expansions between the mortar constituents. The impact of these transformations on the mixtures compressive strengths is shown in Fig. 6. As shown in the figure, different levels of retained strength are generated. The control mixture retained about 20% of its 28 day strength, while that of the GWG group of mixtures ranged from 21% to 45%, and that of the second group of mixtures containing slag ranged from 26% to 60%.

These results show that, it is possible to increase the retained strength more than twice that of the control mixture by merely replacing the mixture sand with waste glass. Further enhancement is achieved when 20% ground slag is introduced within the mixture constituents, as the retained strength is about 3 times that of the control mixture. The interpretations mentioned before about the role of GWG and slag in alleviating the strength retrogression upon heating are valid here. Also, the new formed re-crystallization products, particularly the mineral compound Wollastonite ($Ca\ Si\ O_3$ (β–)), share in preventing a catastrophic drop in strength.

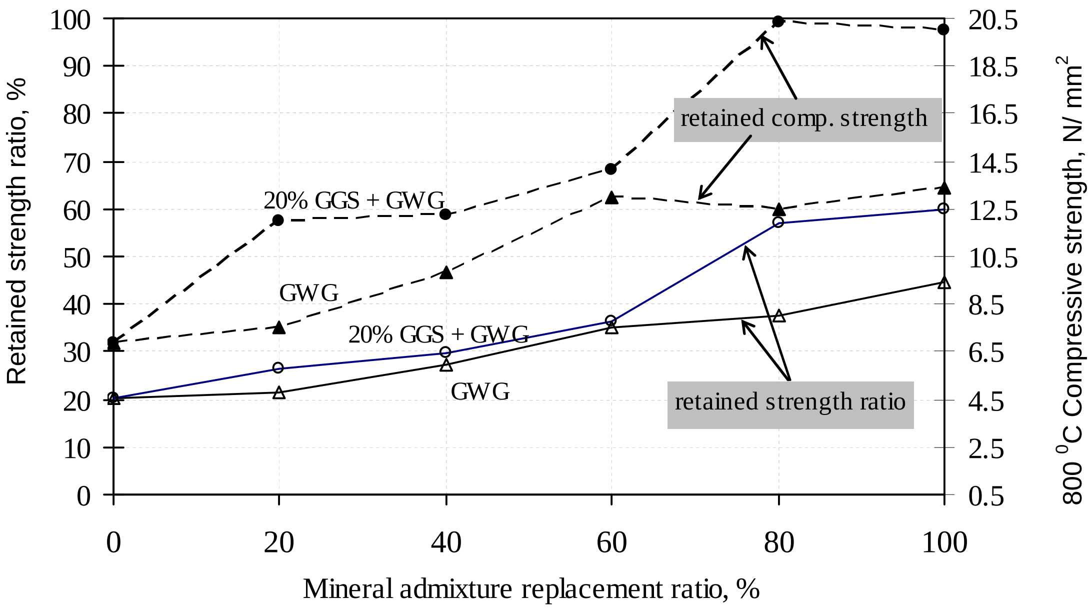

Fig.6: Retained strength after 800 ^{0}C heat exposure

4.3 XRD analysis

The X-ray diffractograms of some of the investigated mortars mixtures at ambient temperature condition and after heating to 800 ^{0}C are shown in Fig.7.

The calcium hydroxide main peak identified at d spacing 2.628 Å (2θ = 34.1^o) and the other confirming peaks demonstrate the close relationship between these peaks and the contents of the added mineral admixtures. It is clear that the calcium hydroxide decreases with increasing the replacement ratio, which is an evidence of their pozzolanic activity. The observed reductions in the peaks of quartz are normally anticipated due to replacing the sand by the mineral admixtures. It was explained before that, the CH and the quartz are main causes of thermal instability, therefore the reductions in their quantities are generally acknowledged.

After exposing the specimens to 800 ^0C the calcite (calcium carbonate) and the CH phases almost disappeared, Fig. 7. Fortunately, a new hydrothermal calcium silicate compounds are produced. The figure confirms the formation of the wollastonite which is a cementing compound serves to retain a considerable part of the strength after heating.

It is worth to mention that, the XRD technique enables identifying the crystalline phases only. It is expected that some amorphous phases have been formed in addition to the molten glass matrix which contribute to the stability (retained strength) of the mixes.

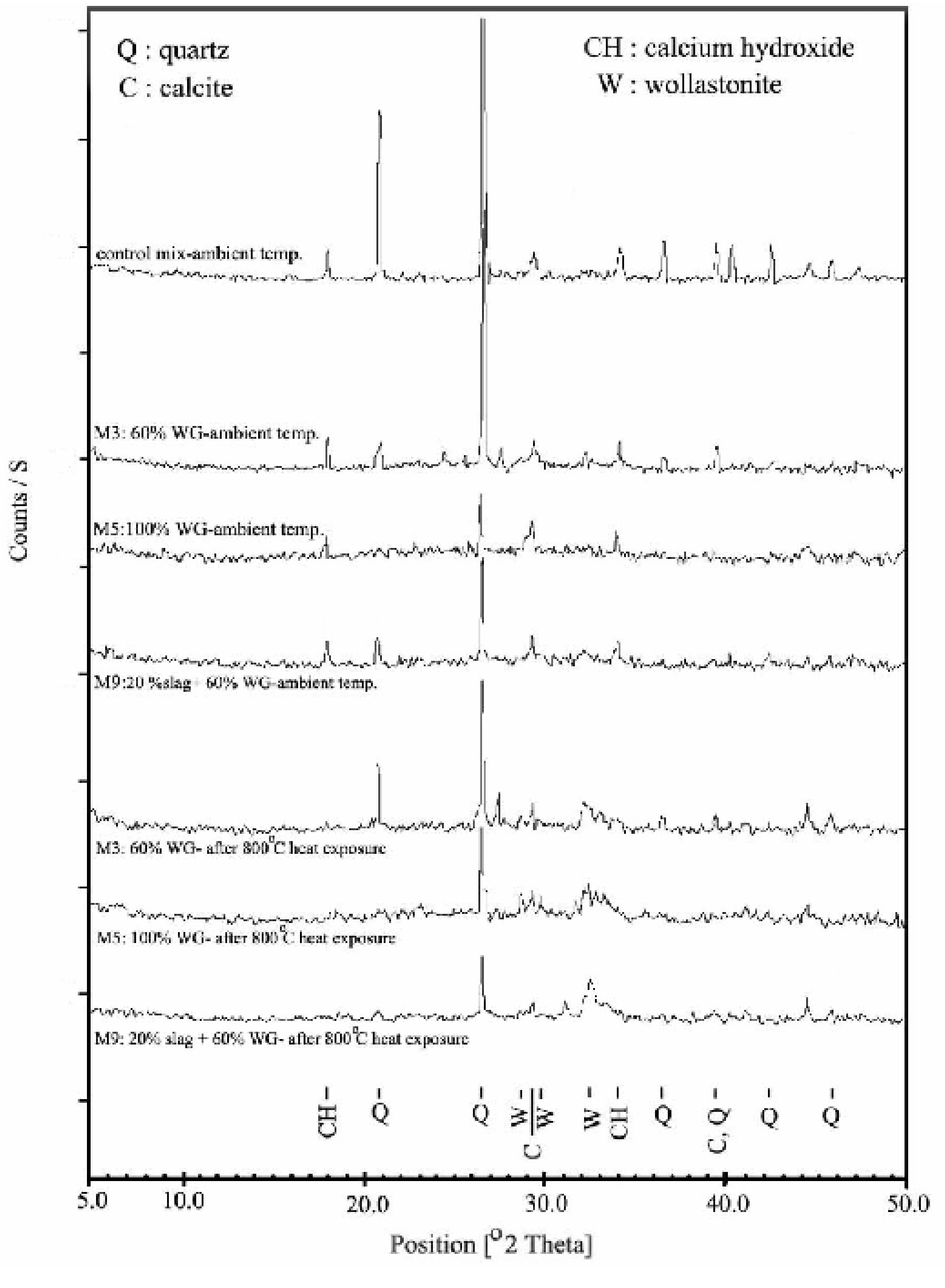

Fig. 7: X-ray Diffract grams of some of the Investigated Mixes before and after Exposure to Elevated Temperature.

6. CONCLUSIONS

The production of a plastering cement mortar that is able to withstand elevated temperature (heat endurance) while maintaining an accepted level of strength is a concern, especially when it is used as a heat barrier for ACM. Proceeding to this target was done through replacing the mortar mixtures sand with GWG in one group of mixtures. Replacement has been done by 20% GS and GWG in the second group of mixtures. Test results reveal that:

- By replacing the mixtures' sand with GWG, the retained strengths at temperatures of 200 ^{0}C, 600 ^{0}C, and 800 ^{0}C were respectively up to 132%, 79%, and 45% of the 28 day compressive strength compared to 117%, 48%, and 20% respectively of the control mixture.
- The mixtures containing 20% GS in addition to GWG were superior to the corresponding mixtures containing GWG only in all aspects. Their compressive strengths at all ages were the highest as well as their retained compressive strengths after heating. The recorded values of the retained strength were up to 134%, 95%, and 60% of the 28 day strength at 200 ^{0}C, 600 ^{0}C, and 800 ^{0}C respectively.
- More research work is recommended to investigate the heat enduring characteristics of mortar mixtures containing higher ratios of the ground slag than adopted in this investigation, as its contribution to the mixtures' stability at elevated temperature is promising.

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PLASTIC AND FREE SHRINKAGES CRACKING OF BLENDED WHITE CEMENT CONCRETE

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ABSTRACT

The objective of the present research work is to determine experimentally the effect of partially substituted of concrete made of white Portland cement with pozzolanic admixtures such as, metakaolin, MK, or silica fume, SF up to 15% by cement weight on the plastic shrinkage cracking and free shrinkage. Also, plastic shrinkage cracking and free shrinkage for concrete made of regular plain gray Portland cement were determined as a reference. The experimental results showed that, plain white Portland cement concrete showed less number of plastic cracks but higher average crack width compared to other concrete mixtures with MK or SF. It has been found that, plain concrete with gray Portland cement matrix showed higher number of plastic cracks than plain white Portland cement matrix. Also the results indicated that, free shrinkage of concrete modified with MK or SF was higher compared to plain white cement matrix and free shrinkage behavior of plain white cement as well as gray cement matrix was comparable.

Keywords: Plastic Shrinkage, Free Shrinkage, White Cement Concrete, Pozzolanic Admixtures.

INTRODUCTION

Early age deterioration of concrete is a persistent problem that arises from rapid complex volume changes such as autogenous shrinkage, drying shrinkage, and thermal deformation. These volume changes cause tensile stresses in the material when strength is relatively low. There is a competition inside the material between the development of tensile stress and the development of strength that evolves with time. At stake in this competition is the potential for premature cracking. The induced stresses may cause immediate cracking or linger as residual stresses that serve to limit capacity of the concrete material. Such premature deterioration affects integrity, durability, and long-term service life of concrete structures.

The brittleness and low tensile strength of cement-based materials are detrimental to their durability. Such materials, the prominent construction materials, are sensitive to cracking. They particularly suffer from shrinkage cracking, especially when the shrinkage is restrained. A typical example is the one of large area structures such as slabs on grade. Are also affected: toppings, linings and cement-based overlays. Their cracking induces debonding, the latter being the main cause limiting their durability [1-4].

If concrete is restrained against shrinkage, tensile stress develops and can cause cracks. Plastic shrinkage cracks are widely evident in bridge decks, industrial and parking garage floors

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and highway pavement slabs, that have large thickness and exposed areas. The development of plastic shrinkage cracks leads to rapid deterioration of the structures when they are exposed to drying and wetting or freezing and thawing conditions [5-8]. In addition, Plastic shrinkage is the dimensional change that occurs in all fresh cement based materials within the first few hours after placement when the mixture is still plastic and has not yet achieved any significant strength. Freshly cast concrete shrinks primarily due to water evaporation. This shrinkage has been attributed [9, 10] to negative capillary pressure that leads to a volume contraction of the cement paste. The stresses are generated by a complex series of menisci which are formed in the water filled concrete pores when water is eliminated from the paste mainly by evaporation. If concrete is restrained against shrinkage, tensile stress develops and can cause cracks. Plastic shrinkage cracks are widely evident in bridge decks, industrial and parking garage floors and highway pavement slabs, that have large thickness and exposed areas.

The use of silica fume in concrete has become widespread all over the world in the production of high-strength and durable concretes. In spite of its increasing use, little data has been published on the effect of silica fume on autogenous volume changes and drying shrinkage. Autogenous volume changes should be taken into account because of their importance in the interior of mass concrete and other exposure conditions. The experimental research program outlined in this paper is designed to investigate the influence of silica fume on expansion of cement pastes and drying shrinkage of mortar bars. Silica fume was varied from 0 to 30% by weight of cement [11].

Metakaolin is a largely amorphous dehydration product of Kaolinite. During the proper calcination process of Kaolinite, dehydroxylation leads to the metakaolin state. Metakaolin is a pozzolan, probably the most effective pozzolanic material for use in concrete. Pozzolans are natural or industrially produced materials which react with the lime released from the hydration of Portland cement. Metakaolin is a product which is manufactured for use rather than a byproduct. It is formed when china clay, mineral kaolin, is heated to a temperature between 600 and 800°C. When used to replace cement at levels of 5 to 10 % by weight, the concrete produced is generally more cohesive and less likely to bleed. As a result pumping and finishing process require less effort. The compressive strength of hardened concrete is also increased at this level of replacement. Metakaolin has been successfully used to improve the durability of glass fiber reinforced cement composites [12].

White Portland cements are commonly used for architectural use when genuine white concrete is desired or as base cement when colors are desired. Plastic shrinkage cracking is a critical problem to these architectural precast members. Thus, the objectives of this study were to investigate the plastic and free shrinkages of white Portland cement concrete, concrete incorporating SF and concrete incorporating MK compared to regular plain gray Portland cement concrete

EXPERIMENTAL DETAILS

An experimental program was designed to investigate the plastic and free shrinkage of concrete containing gray and white blended cement

Materials
Cement

Type I white Portland cement and gray Type I Portland cement were used for making concrete. Their physical properties and chemical composition are given in Table 1.

Silica fume and metakaolin

The SF used was a dry, uncompacted powder from the production of silicon metal with SiO2 content of 93.6%. The amount retained on a 45-mm sieve was 1.8%. The MK used in this research produced by calcining kaolin at temperature of 850 ^{o}C for two hours and consists predominantly of silica and alumina. Detailed physical properties and chemical composition of MK and SF are also given in Table 1.

Aggregate

The coarse aggregate used was crushed stone with a maximum nominal size of 6.35 mm with specific gravity of 2.65 and volumetric weight 1.6 t/m^3. Fine aggregate used was natural sand less than 5 mm with specific gravity of 2.65 and volumetric weight of 1.57 t/m^2. The gradation of coarse and fine aggregates satisfied ASTM C 33 requirements.

Superplasticizer

A superplasticizer, SP, of sulfonated, naphthalene formaldehyde condensate type was used in all the concrete mixtures. The superplasticizer is a dark brown solution with solid content of ~ 40% and specific gravity of 1.1.

Table 1: Chemical Composition and Physical Properties of Cementitious Materials

Chemical composition (%)	White Cement	Gray Cement	SF	MK
SiO_2	22.57	20.39	93.6	58.52
Al_2O_3	4.74	5.6	0.5	35.54
Fe_2O_3	0.3	3.43	1.5	1.15
CaO	64.28	63.07	0.9	1.24
MgO	2.03	2.91	0.6	0.19
Na_2O	0.5	0.38	0.04	0.25
K_2O	0.11	0.35	-	0.05
SO_3	2.66	0.7	0.3	0.06
C_3A	-	9.04	-	-
Phosphrous pentaoxide (P_2O_5)	-	-	-	0.09
Titanium (TiO_2)	-	-	-	0.04
Loss on ignition	1.72	2.06	2	2.74
Physical properties			-	
Specific gravity	3.15	3.15	2.23	2.34
Specific surface (cm^2/gm)	3000	2750	230000	3600

Concrete mixtures and specimen preparation
Mixture proportions

Mixture proportions of the concrete studied are given in Table 2. There are eight different concrete mixes. The first and second mixes contain plain gray and plain white cement respectively. The third, fourth and fifth mixes contain SF incorporating 5%, 10% and 15% by mass of the total cementitious materials as cement replacement respectively. The remaining mixes are similar to the previous mixes but their mixes contain of MK instead of SF.

Table 2: Proportions of the Concrete Mixes

Mix	Gray Cement (kg)	White Cement (kg)	SF (kg)	MK (kg)	Sand (kg)	Crushed Stone (kg)	Water (liter)	SP (liter)
GC	350	0	0	0	675	1215	140	105
WC	0	350	0	0	675	1215	140	105
SF5	0	332.5	17.5	0	672	1210	140	105
SF10	0	315	35	0	669	1205	140	105
SF15	0	297.5	52.5	0	666	1200	140	105
MK5	0	332.5	0	17.5	673.5	1212	140	105
MK10	0	315	0	35	672	1209	140	105
MK15	0	297.5	0	52.5	670	1206	140	105

Preparation and curing of concrete specimens

The concrete was mixed in a laboratory electrical pan mixer. Mixing was done in accordance with ASTM C 192. The materials were placed in the mixer in the following sequence: first the coarse aggregates were placed in the mixer, and then the fine aggregates, followed by cement and SF, or MK. The SF, or MK was added and mixed with the cement separately before being transferred to the mixer. The superplasticizer was added to the mix water and thoroughly stirred to a uniform color. Some of the mixing water was added and mixing continued for about 2 min. Thereafter, the remaining mixing water was added and mixing continued for another 2–3 min until the mix became homogeneous. The concrete was then cast in to the moulds in two layers with each layer being vibrated for one minute. A straight aluminum angle was used to consolidate and level the concrete surface. The concrete surface was leveled in one direction. After casting, all the specimens for the total shrinkage measurement were left in the casting room (~ 30 ^oC), covered with plastic sheet for approximately 24 h, then demolded and cured in a moist-curing room at (~ 30 ^oC) and >95% relative humidity until required for testing.

TEST SPECIMENS

For plastic shrinkage cracking test, the specimens were slabs of dimensions 610 mm x 915 mm x 19 mm. Two specimens were cast for each test. Free shrinkage specimens were two specimens of the dimensions 101.6 mm x 101.6 mm x 254 mm.

Determination of concrete properties

Slump of fresh concrete

The slump of the fresh concrete was determined immediately after the mixing according to ASTM C 143. The slump of the concrete is not less than 100 mm.

Plastic shrinkage

Plastic Shrinkage slabs were exposed to uniform breeze of air condition using a suitable fan. A bowl filled with water was placed next to the specimens to measure the rate of plain water evaporation in that particular breezy condition provided by the fan. Even though this is not a standard ASTM procedure, other researchers have successfully used this test method to assess the plastic shrinkage behavior of concrete [13, 14]. First reading for plastic shrinkage specimen was taken after two hours of placing the concrete. The number of cracks and crack width were measured for the entire slab. Consecutive readings were monitored every two hours up to eight hours. A hand-held microscope was used to detect the crack and measure its width and its length.

Dry shrinkage

Dry, free, shrinkage cracking tests were performed according to ASTM C-157. A digital extensometer with 254 mm gage length was used to measure the length change on studs embedded in concrete. Values of dry shrinkage were read every 24 hours for the first two weeks and then three times a week

RESULTS AND DISCUSSION

Plastic shrinkage

Observations for cracks in plastic shrinkage slabs started after 2 hours of placing and finishing. There was no significant difference between concrete mixtures as to when the first crack appeared except for, plain gray Portland cement concrete, for which the first crack appeared late. First crack was noticed in all the samples within the range of 2 – 3 hours (Fig. 1). Total number of cracks as well as their widths was monitored for comparison. These measurements were repeated every two hours for up to 8 hours. It should be noted that two identical specimens were made for observation of plastic shrinkage cracking. Crack characteristics were observed to be similar in both the specimens. Figure 1 and 2 presents the total number of cracks and their average crack widths, respectively, for different concrete mixtures. Plain white Portland cement concrete showed less number of cracks but higher average crack width compared to other concrete mixtures with silica fume. However, this increased average crack width (1 ½ to 2 times) was less pronounced compared to significantly lower number of cracks observed with plain white Portland cement matrix. When plain white cement matrix was modified with partial replacement of cement with silica fume there was a significant increase in number of cracks. About 200 to 400 cracks were observed with silica fume concrete compared to 15 with plain white cement matrix. These results are similar to earlier observation with gray cement matrix modified with silica fume [15-16]. It can be concluded that the addition of silica fume increases the content of calcium silicate hydrate, which is the most important factor causing shrinkage at 28 days [11]. The average crack widths for these mixtures were smaller. White cement matrix modified with 5 – 15 percent of silica fume showed similar behavior. While there was a clear indication that the number of cracks increased with the addition of silica fume (5% replacement) it did not show a definite trend with increased silica fume content (10 and 15%). Plain concrete with gray Portland cement matrix showed higher number of cracks than plain white Portland cement matrix.

Figures 3 and 4 presents the total number of cracks and their average crack widths, respectively, for different concrete mixtures with metakaolin. Plain white Portland cement concrete showed less number of cracks but slightly higher average crack width compared to other concrete mixtures. However, this increased average crack width (1 ½ to 2 times) was less pronounced compared to significantly lower number of cracks observed with plain white Portland cement matrix. When plain white cement matrix was modified with partial replacement of cement with metakaolin there was a significant increase in number of cracks. These results are similar to earlier observation with gray cement matrix modified with MK [17, 18]. About 200 to 500 cracks were observed with metakaolin concrete compared to 15 with plain white cement matrix. The average crack widths for these mixtures were smaller. White cement matrix modified with 5 – 15 percent of metakaolin showed similar behavior. While there was a clear indication that the number of cracks increased with the addition of metakaolin (5% replacement) it did not show a definite trend with increased metakaolin content (10 and 15%). This trend was similar to the one observed with silica fume. Plain concrete with gray Portland cement matrix showed higher number of cracks than plain white Portland cement matrix. Effect of plastic shrinkage cracking on plain gray cement matrix modified with metalaolin was not studied in this investigation.

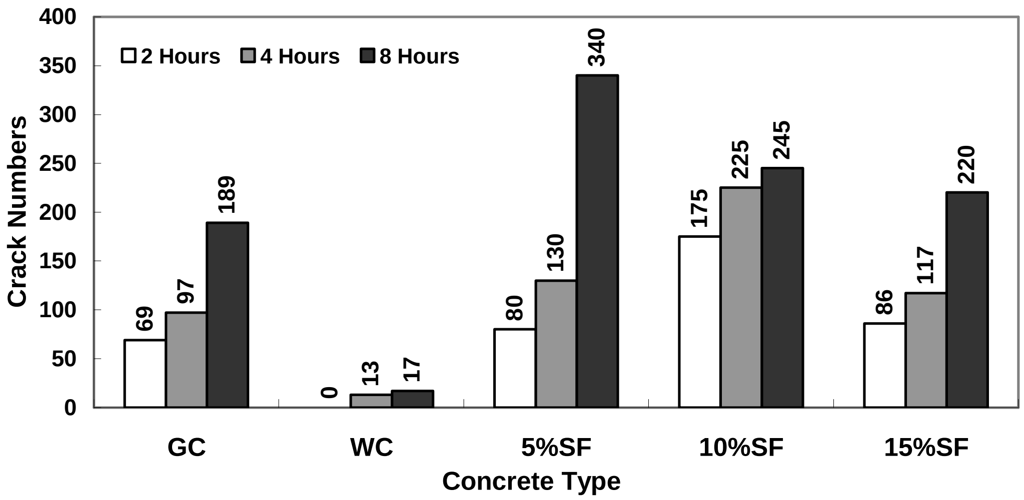

Fig. 1: Number of Cracks with Time for Concrete with Silica Fume.

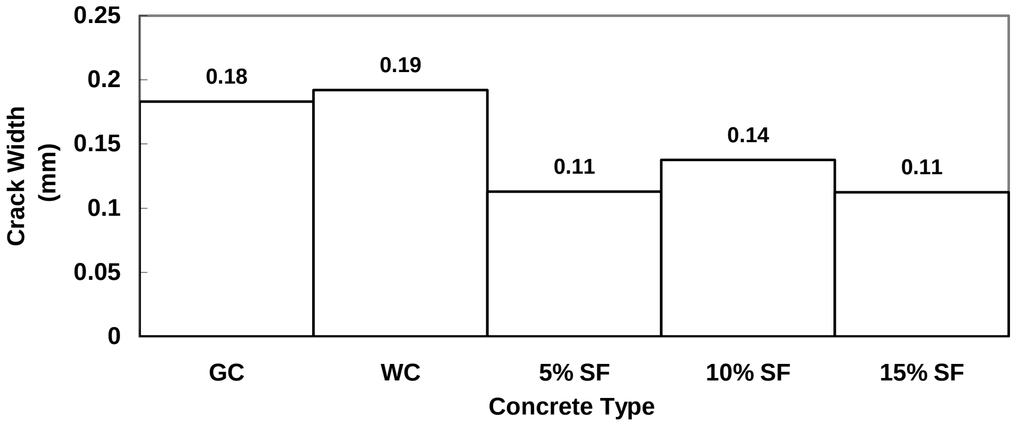

Fig. 2: Average Crack Width at 8 Hours for Concrete with Silica Fume

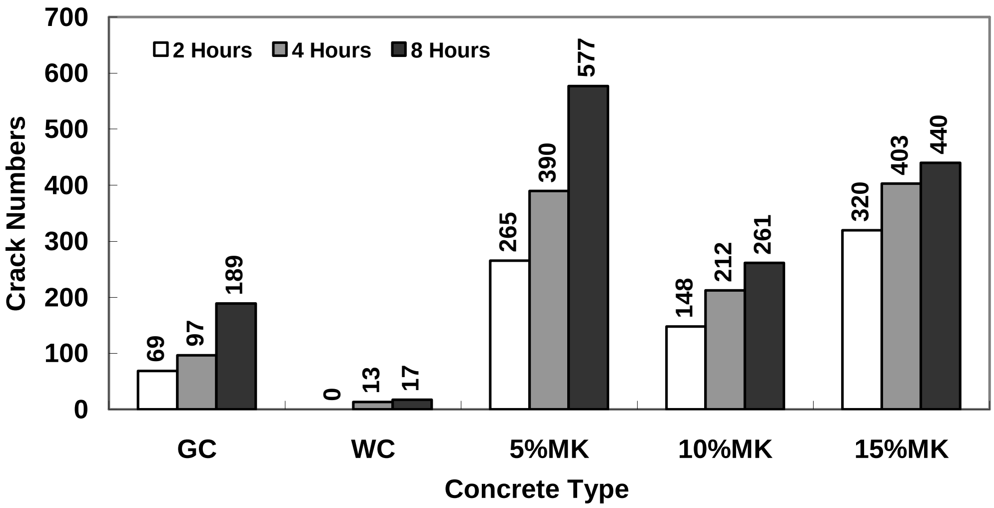

Fig. 3: Number of Cracks with Time for Concrete with Metakaolin.

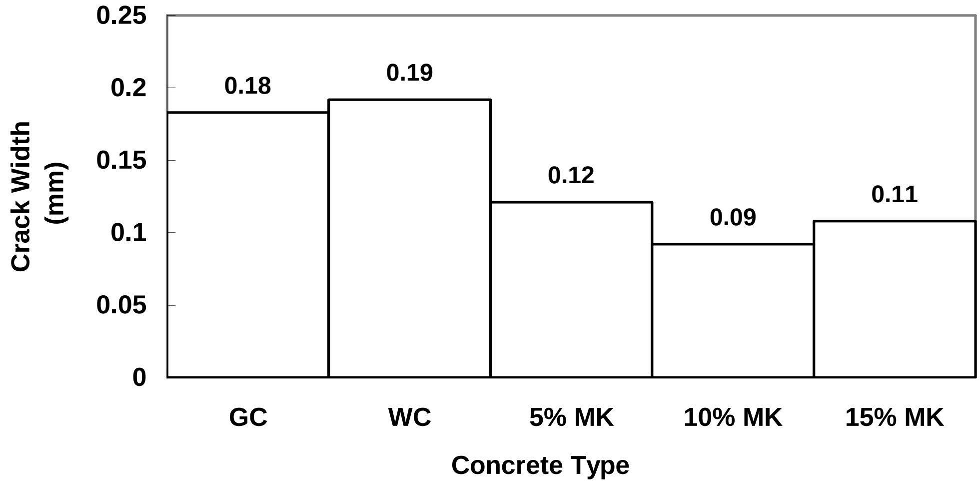

Fig. 4: Average Crack Width at 8 Hours for Concrete with Metakaolin

Free shrinkage

Free shrinkage tests alone can not offer sufficient information on the behavior of concrete structures since virtually all concretes are restrained in some way, either by reinforcement or by structure. However, dry shrinkage tests can provide necessary information on how the free shrinkage stresses develop.

Figure 5 presents the free shrinkage cracking behavior of concrete with silica fume. Shrinkage readings were started after demolding (24 hours). It should be noted that much of autogenous shrinkage occurs before 24 hours and as such the results do not reflect the effect of autogenous shrinkage. The results indicate that the addition of silica fume increases the free shrinkage of concrete. These results are similar to earlier observation with gray cement matrix modified with silica fume [11, 19-23]. Free shrinkage behavior of plain white cement as well as gray cement matrix was comparable.

Figure 6 presents the free shrinkage cracking behavior of concrete with metakaolin. The results indicate that the addition of metakaolin increases the free shrinkage of concrete. These results are similar to the ones observed with white cement matrix modified with silica fume. Free shrinkage behavior of plain white cement as well as gray cement matrix was comparable.

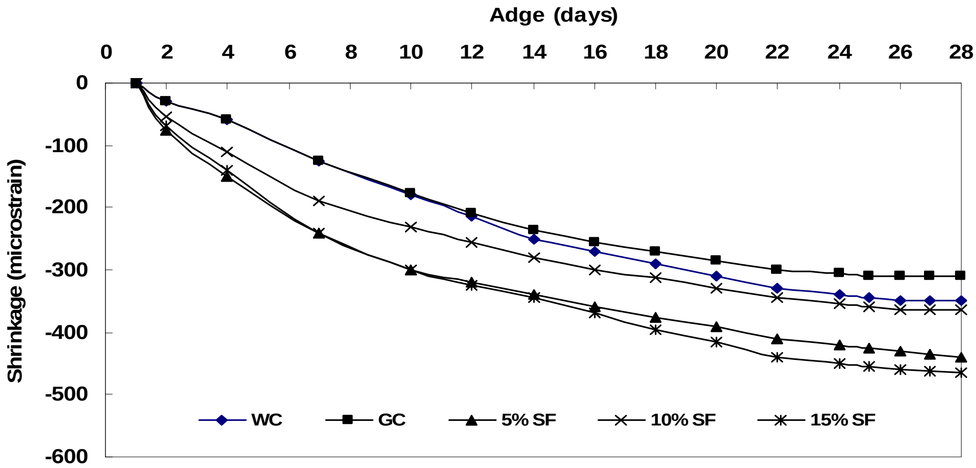

Fig. 5: Free shrinkage behavior of concrete with silica fume

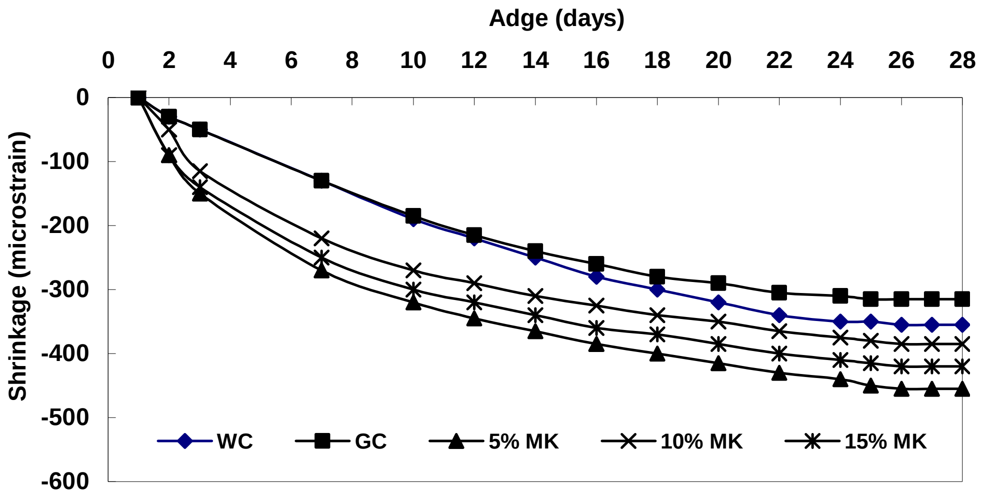

Fig. 6: Free Shrinkage Behavior of Concrete with Metakaolin

CONCLUSIONS

The following conclusions can be drawn from the present study:

1. Plain white Portland cement concrete showed less number of plastic cracks but slightly higher average crack width compared to other concrete mixtures with MK or SF.
2. Plain concrete with white Portland cement matrix showed lower number of plastic cracks than plain gray Portland cement matrix but slightly higher average crack width compared to plain gray Portland cement matrix.
3. Free shrinkage of blended white Portland cement concrete with MK or SF was higher compared to plain white cement matrix.
4. Free shrinkage behavior of plain white cement and plain gray cement matrix was comparable.

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DEVELOPMENT METHOD FOR EVALUATING CONDITION RATING SYSTYEM OF THE BRIDGE COMPONENTS

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ABSTRACT

Most of the current methods of assessment the condition of the bridge during the inspection process depend on the subjectivity opinion of the inspectors. No calculations are performed, but the condition of the component is a degree. This degree provides by the inspectors during the inspection process to evaluate the condition of the bridge. Some of known rating methods provide common words such as low – medium – high – very high to classify the defects.
In this paper an extensive literature search of condition rating systems in various countries is presented. A developed calculated method is suggested to standardize the assessment of the condition rating of the bridge and minimize the dependant on the inspector's expert. The proposed method depends on classifying the defects according to the martial deficiency and the structure severity. The martial deficiency depends on the type of defects, importunacy of bridge components, and the quantity of the defects. The structural severity is evaluated according to the design cods limits. Proposed limits to correlate the condition rating for any components and its defects are suggested. These limits may be modified according to the conditions of each country. Using these limits, a chart was developed to facilitate the use of the propose rating system. This chart can be used as a base of computer program which can be developed to calculate the condition rating of any components of the bridge from specific input data from the inspector report.

Keywords: Bridge Management, bridge condition assessment, rating system, inspection.

INTRODUCTION

Several method of assessment the condition rating of the components of the bridge and the total condition of the bridge were found in literature. Each country have developed method to evaluate the condition of it bridges. The condition rating of the bridge is the main factor which decides the plan and the strategy of maintenance, the priority and the necessary activities according to the available budget.
The revision of literatures appears that many methods of assessment the condition of the bridge depend on the subjectivity opinion of the inspectors. No calculations are performed, but the condition of the component is a degree numbers. This degree provides by the inspectors during the inspection process to evaluate the condition of the bridge. Also, in some countries a failure of bridges happed after an inspection process which didn't determined the actual condition of these bridges.

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In this paper, some calculations are suggested to calculate the condition rating number of the component of the bridge according to certain factors.

CONDATION RAGING SYSTEM IN DEFFERENT COUNTRIES

Pennsylvania Department of Transportation system

The classification system of defects depends on the priority method, McClure and Hoffman 1990, [1] which uses the following components: activity ranking, activity urgency, bridge critically and bridge adequacy. In general rule, activities are divided to five groups from A to E based on relative importance to the safety and stability of the existing structure. Deficiency points are assigned to each group. The AF group represents the activity of fatigue defects. Also, the importance of the bridge to the road network and it effect on the delay traffic service (ATD) is also another factor. The severity of a deficiency is a reason to increase the priority for repair. The priority codes used ranges from 0 to 5 where the 0 represent the critical safety deficiency and 5 represent the routine nonstructural bridge maintenance. Then, the condition rating of each critical component in the bridge assesses the bridge degradation. (McClure and Hoffman 1990) [2]. Although it is numerically possible that the summation of deficiency point of the component for a single bridge exceed 100, the deficiency point is limited to maximum 100. the higher than this is higher the priority. The evaluation of the bridge is calculated by the total deficiency rating (TDR) by the following:

$$\mathbf{TDR = \phi\,(LSC + BCD + RLD+AAD+WAD)} \qquad (1)$$

Where

ϕ = functional classification factors depends on the type of road.

$$\mathbf{LSC = LCD + WD + VCOD + VCUD} \qquad (2)$$

LSC = Level – of – service capabilities, LCD = Load capacity deficiency, WD = Clear deck deficiency, VCOD = Over clearance deficiency, VCUD = Under clearance deficiency.

$$\mathbf{BCD = SPD + SBD +BDD} \qquad (3)$$

BCD = Bridge condition deficiency, SPD = Condition deficiency for superstructure, SBD = Condition deficiency for substructure, BDD = Condition deficiency for deck, RLD = Maintenance life deficiency, AAD = Approach roadway alignment, WAD = Adequacy of the waterway. All these terms can be calculated by BMTG,1987- equation [2].

The inventory system PONTIS

The Federal High way Administration and AASHTO manual [3,4] developed "Guidelines for condition rating" degradation system for each element of the bridge, superstructure, substructure and deck: from 0 to 9. The condition rating of any item 0 represents dangerous condition but 9 represents excellent condition. The structure is classified according to the rating condition.

- The bridge is considered structurally deficient if any of the deck, superstructure, or substructure condition rating is equal to or less than 4 (poor).
- Functionally obsolete (high priority for corrective action) If the deck, the under clearance, or waterway appraisal is equal to or less than 3 .
- High priority for replacement if the structure appraisal or waterway appraisal is equal or less than 2.

The system permits to evaluate the defects according to sense and experience of the inspectors.

Canadian system

The Highway Engineering Division in CANDA developed a condition rating system that depends on the observed material defect (MCR) and the resulting effect on the ability of the component to perform its function in the structure (PCR). The condition rating number is varies between 1 to 6 and assigned to each component of the structure.
All the components of the structure are classified as primary, secondary or auxiliary. The defects are classified as high, medium, severe, and very severe according to the ratio of severity in all over the components, Branco et al. [5].

Japanese system

In Japan (Yokoyama et al.1996) [6], a list of deficiency ratings is specified. Each element of the bridge is evaluated by assigning a deficiency rating for each kind of defect such cracking, corrosion, deformation etc. Combined with these, a demerit rating d*i* for each component of damage is calculated from the pre-defined deficiency rating within the BMS.
For example the d_I for corrosion of steel girder is given as 70, thus d_{II}=0.5x70=35, and so on. The demerit rating for each structural element is tabulated and by process of reduction a final overall rating is arrived at end subtracted from 100 (the perfect bridge) to give the final bridge condition rating. These have been compared to condition ratings provided by experienced bridge inspectors and the process is continually refined year by year with the aim of getting a calibration factor of 1.0. The demerit rating d_I corresponding to deficiency rating I for each component of damage was assigned within the BMS, and the remainder of the demerit rating are calculated from:

$$d_{II} = d_I \times \alpha_{II}\ ; \qquad d_{III} = d_I \times \alpha_{III}\ ; \qquad d_{IV} = d_I \times \alpha_{IV}\ ;\ d_{ok} = 0 \qquad (4)$$

Where $\alpha I = 1$ for serious damage; αII = 0.5 for damage in large area; $\alpha III = 0.2$ for damage need follow–up investigation. $\alpha IV = 0.05$ for slight damage. Inspection data is recorded.
once the condition rating of the bridge have been determined for each bridge, the strategy of rehabilitation plan is prepared. The rehabilitation cost of each bridges is calculated and a list of all bridges in ascending order is prepared.

United Kingdome

Surrey county Council has a system in place which aims to determine a maintenance priority Number (MPN) for each bridge [6]. Each structure is broken down into its elements (42 for Surrey's Bridges and 33 for Highway Agency Bridges). Each bridge is awarded a condition value (CV) between 1 and 5. condition 1 is good and condition 5 is critical. Then. The CV is converted to a condition factor (CF) by using the following relation :

CV	5	4	3	2	1
CF	1	2	4	7	10

The CF uses to determine the MPN: The location of each element in the structure is given a location factor (LF) between 5 to 10 depending upon its importance as follows. A road factor (RF) is assigned to each type of road between 9 to 14 depending upon its relative importance. Values 9 to 10 are for Motorway and Truck road bridges. Values 11 to 14 are assigned to County Road and Rights of way.
Each element in each structure can therefore have MPN calculated for it, and by using it the MPN for structure can be calculated. By the repeated evaluation of MPN each inspection, a graph can be plotted which represents the deterioration of the structure along its live.
The maintenance priority Number is then calculated from

$$\mathbf{MPN = CF \times LF \times RF/14} \qquad (5)$$

In a national level the concept of the bridge condition index (BCI) is being used in an attempt to provide a means of monitoring the change in condition state of the bridge element, or a single bridge within an entire bridge stock (Blakelock et al. 1998) and could be used to address the problem of prioritization of funding.

$$\mathbf{BCI = 100 - F1 \times (F2 \times S(E_{fp} \times S_f) / N_p \times F3 \times S(E_{fg} \times S_f) / Ns)} \qquad (6)$$

Where ,
BCI = condition index; E_{fp} = element factor for primary elements =1 to 10; E_{fs} = element factor for secondary elements; S_f = extent/severity factor=1 to 10; N_p = number of primary elements on the bridge; Ns = number of secondary elements on the bridge; and F1,F2 and F3 are series of factors. The factor BCI varies between 100 (all elements are A1 (as a new)) to 0 (nonfunctional bridge).

National Bridge Inventory

The specifications for the national bridge inventory draft version – 10/16-06 [4] provides graphical interpolation charts for each items of the bridges where the items of the bridges includes: deck – superstructure, substructure, and culvert. Also, the charts is deferred according to the basic materials of the bridge (concrete – steel – timber) . The condition rating system is measured on a 9 (best) to 0 (worst) scales. For example the condition rating for concrete deck is: good condition (degree 7) if spalls and patched areas is up to 2% with insignificant cracking. Satisfactory condition (degree 6) if spalls and patched areas is 2-10% with insignificant cracking, or up to 2% with minor cracking. Fair condition (degree 5) if spalls and patched areas is 10-25% with insignificant cracking, between 2-10% with minor cracking, or up to 2% with moderate cracking. Poor condition (degree 4) if spalls and patched areas is 25-50% with insignificant cracking, between 10-25% with minor cracking, between 2-10% with moderate cracking or up to 2% with severe cracking. Serious condition (degree 3) if spalls and patched areas is 25-50% with insignificant cracking, between 10-25% with minor cracking, between 2-10% with moderate cracking or up to 2% with insignificant cracking.
For the previous limits it can be notice that:

- The draft version still classified the structure defects into insufficient, minor, moderate, and severe types.
- The draft version does not taken into considering the position of the defects in the members (main – secondary, or auxiliary member) and its is effects on the over all structure system.
- Each items of the bridge has a separate graph according to its base materials.
- The only reference for the structure defects is the cracking in the concrete and the corrosion in the steel bridges without any other consideration such as the deflection or the vibration of the bridge.

McClure and Hoffman Method

McClure and Hoffman 1990, De Brito et al. 1994, and Andrey 1987 [5] proposed a conditional rating system of the base on three fundamental aspects: Rehabilitation urgency: this rating is the inspector's responsibility and is performed during a regular inspection visit. This rating corresponding to the " activity urgency " of the Pennsylvania system.

0 – Immediate action required, if the possible collapse is expected at any time.
1- Short-term action required (up to 6 months maximum), if for example corrosion of steel bars reveals and losses of the adherence with the reinforcement totally exposed .
2- medium –term action required (up to 15 months), if for example rust in the member of joint)

3- long –term action required (in next visit) , for example if the area of deck affected has only medium aesthetic value).

Importance to the structure's stability: this rating is constant for each defect type as a function of its location. This rating corresponding to " bridge maintenance activity" in Pennsylvania system.

A – Eminently structural defect associated with main structural elements (beams, columns, deck, abutment, and foundation).
B – Semi structural defect associated with main structure elements or structural defects with secondary structural elements (bearing, joints, etc.)
C – Semi structural defect associated with secondary structure elements or defect in nonstructural elements (wearing surface, drainage system, sidewalks, handrails, and curbs, etc.)

Volume of traffic affected by defects: this rating, which is inherent in each bridge, must be updated periodically but also depends on the inspection visit); this rating to a signification of the criteria " bridge Critically" and " Bridge adequacy" of the Pennsylvania system.

$\alpha - t.v \text{ X } d.l. \text{ X } k \geq n1$ (vehicle km / day); (7)

$\beta - n1 > t.v \text{ X } d.l \text{ X } k \geq n2$ (vehicle km / day); (8)

$\gamma - t.v \text{ X } d.l \text{ X } k < n2$ (vehicle km / day) (9)

Where

$t.v$ = average daily traffic volume over the bridge in two direction (vehicle / day).

$d.l$ = average detour length caused by the total disruption of the bridge (km).

k = Coefficient that provides the degree of obstruction to normal traffic over the bridge caused by each defect or by the most defects in the bridge.
= 0 , 0.5 , 1 without interpolation values and it provided by the inspector during the inspection visit.

$n1,n2$ = fixed parameters defined by the bridge's authorities and calibrated as experience is gained ; at a first stage, the values 3.000 and 15.000 respectively , are proposed.

For each possible defect determine urgency rate form first term and the rate of importance to the structure's stability from second term and the effectiveness in traffic volume from the third term then the rate of defects for example (1Bα). Then inter the Table No. or Table No. to determine the priority group of the defect. From table No. it can be determine the priority of the repair or maintenance of the defects. Table No. suggested time periods for repair or rehabilitation of defects.

PROPOSED METHOD

The proposed method is based on evaluating the condition rating of the bridges independence on the different inspector’s opinion, different qualification experts, or different selection of repair method. It depends on the concept of the total number of points assigned to each defects is not the only information needed to determine the maintenance work. Several factors should be evaluated before repair and rehabilitation decisions, however, a decision support system might be needed to help reaching adequate repair decision.
The maintenance should be defined as the solution for simple problem. The repair and replacement are the solution of great problems. The repair / maintenance / strengthening should be evaluated by the computer program according to specific role and can’t depend on the inspector’s opinion to standardized the process of the inspection.

The defects of concrete bridges

The proposed method classifies the deficiency of bridge elements according to the following:

1- Type of defects (TD) as shown in Fig.1: The condition rating system should be evaluated according to the effect of the member on the material of the member and the structural behavior of the member as follows:

- Material defects (MD): as shown in Fig.2 which is defined as the ratio of loss of material cross section or surface area of bridge component such as delamination, spalling, scaling, voids, etc. then,

$$\textbf{MD} = \frac{\textbf{Area / volume of the defects of the bridge component}}{\textbf{Area / volume of the original component}} \qquad (10)$$

- Structural defects (SD): is the effect of the defects on the structure behavior of the bridge components and it defined as the ratio between the exceeding structure defects and the permit limits according the code of practice. The structure defects as shown in Fig.3 include deformation, deflection, settlements, cracks, etc. This means that the defects may cause for example excessive deflection, then, the structural defect is the exceeding ratio of existing deformation due to defect to the permitted deflection according to the code of practice. Also, the structural defect of the cracks is the exceeding ratio of the existing crack width to the permitted crack width according to the code of practice, and so.

$$\textbf{SD} = \frac{\textbf{Existing structural defects of the bridge component}}{\textbf{Permits value of the structural defects}} \textbf{ - 1} \qquad (11)$$

The position of the defect (PD):

The condition rating system should be evaluated according to the importunacy of the members in the structural system as follows:

- Primary elements.
- Secondary elements / Deck.
- Auxiliary elements.

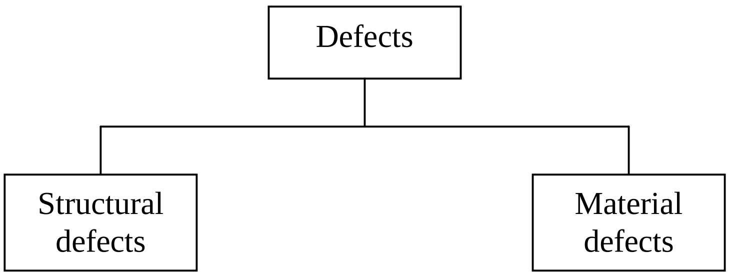

Fig.1: Type of Defect (TD) in Bridge Structures.

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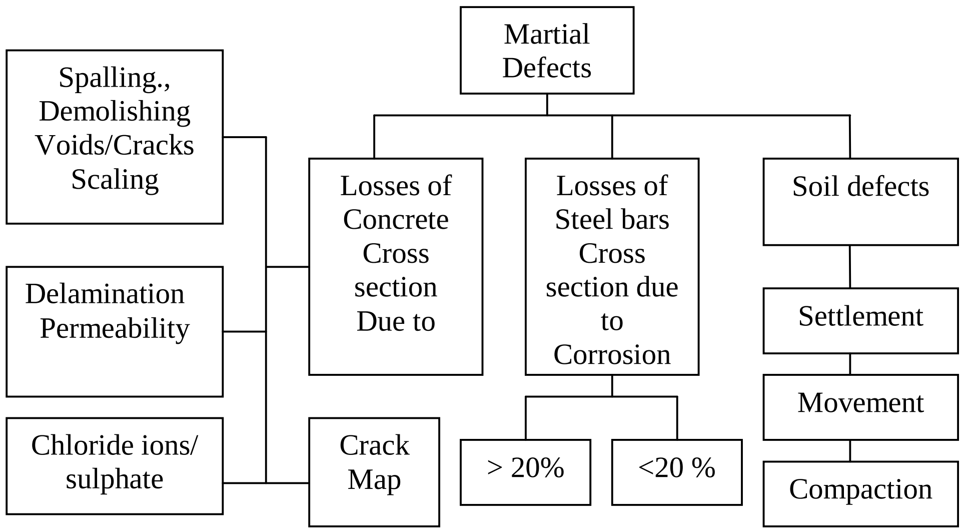

Fig. 2: Material Defects Types in Primary and Secondary/Deck Elements

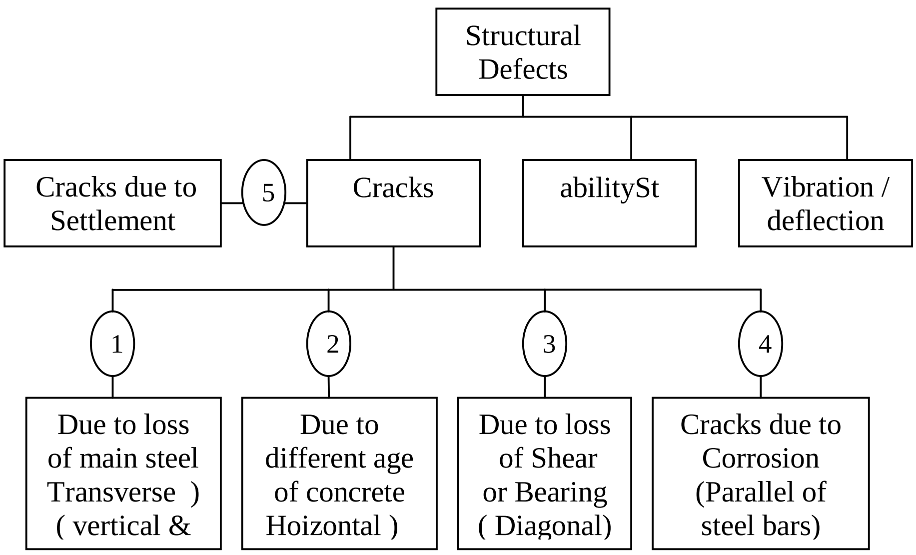

Fig. 3: The Type of Structural Defects According to its Causes.

Evolution of the proposed method

The proposed method suggests that the condition rating number varies between 1 to 9 may be assigned to each component of the structure as shown in Fig. 4. This degradation is compatible with PONTIS system. All the components of the structure are classified according to the importanacy of the member in the structural system such as primary, secondary or auxiliary. The defects are classified twice, one according to the ratio of material deficiency (MD) and other according to ratio of the structural deficiency (SD). Using Fig. 4 to evaluate the condition rating of the bridge components. Fig. 4 presents the relationship between the previous factors, MD, SD and the type of defected member in the bridge. This chart presents a proposal limits for words low, medium and high severity which its used in the common methods.

This developed method is considered a modification of the pontis method which takes the same number of degradation but has more definitions of the defects. The method depends on evaluating the defects according to the ratio of defect areas and the allowable limits of design cods.

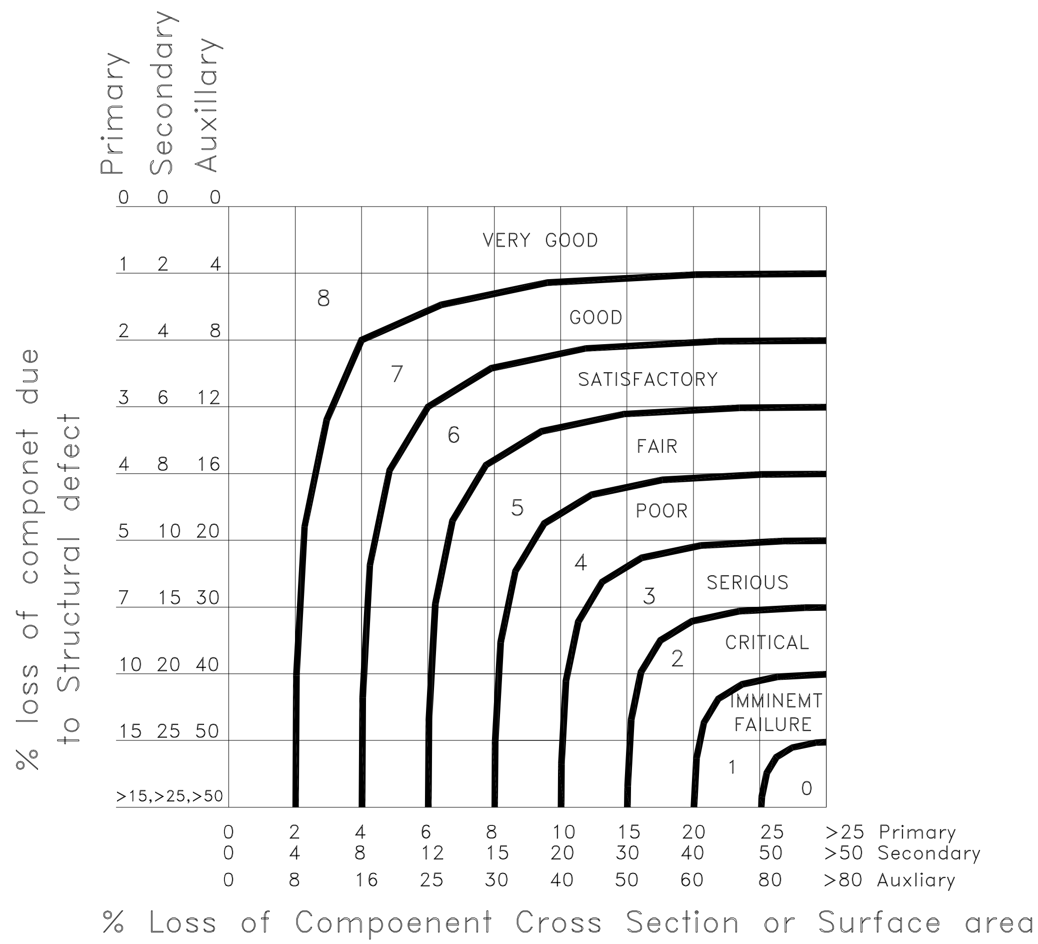

Fig. 4: The Relationship between the Material, Structural Defects and the Proposed Condition Rating.

CONCLUSION

Therefore, the following procedure suggested to determine the condition rating of the component of the bridge:

- Reveal the defects and classify it as a primary, secondary / deck or auxiliary components of the bridge.
- Determine the type of defects (material – structural) and cause of the defects.
- Determine the diagnosis method to determine the material and the structural defects.
- Determine the ratio of defects in case of material defects (MD) by equation (10) and the structural defects (SD) by equation (11).
- By using Fig. 4 and knowing MD and SD, it can be determined the condition rating factor of any component of the bridge.

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EFFICIENT SHEAR REINFORCEMENT FOR CONCRETE BEAM-COLUMN JOINTS UNDER EARTHQUAKE LOADS

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ABSTRACT

Current ACI and CSA code provisions recommend large amount of shear reinforcement for concrete beam-column joints in the form of closed hoops and cross ties to ensure confinement of the joint core. The closed hoops, the cross ties, and the bends required for anchoring the beam bars cause significant congestion in the joint. Double-headed studs are proposed as shear reinforcement in lieu of conventional closed hoops and cross ties. Five full-scale beam-column joint specimens were tested under cyclic loading. One specimen was designed without joint shear reinforcement and used as shear-deficient control specimen. Another specimen was designed according to the Canadian Code provisions and the joint was reinforced with closed hoops and cross ties. The joints of the remaining three specimens were reinforced with two different arrangements and amounts of double-headed studs. The use of headed studs proved to be a viable option for reinforcing exterior beam-column joints. In comparison with the shear deficient-joint, the specimens reinforced with headed studs exhibited considerable enhancement in the behaviour, in terms of strength, ductility, energy dissipation, and joint contribution to lateral drift.

Keywords: Beam-column joints, Confinement, Cyclic loading, Ductility, Energy dissipation, Shear, Stud reinforcement.

INTRODUCTION

In many earthquakes worldwide, reinforced concrete beam-column joints, particularly exterior ones, have been repeatedly identified as critical elements which fail prematurely, forming weak links in framed structures. Poorly detailed joints frequently fail either by diagonal tension cracking resulting from high shear forces when insufficient shear reinforcement is provided, or by pullout of the beam flexural reinforcement before the joint reaches its full capacity, when adequate anchorage is not available [1]. Therefore, recent design codes require that beam-column joints be provided with closed hoops (stirrups), and cross ties, as shear reinforcement and for confinement of the concrete core. The codes also require that the beam flexural reinforcement be extended inside the joint with sufficient anchorage length. The legs of hoops and the ties are anchored to the concrete by hooks or bends. The beam reinforcing bars are bent at 90 degrees at the end of the joint to achieve proper anchorage. The hooks and bends, and the necessary overlaps within the joint, lead to congestion of the reinforcement causing construction difficulties, hence, increase in labour cost, and to reduction in the contribution of concrete to the strength, hence, a weak joint with reduced ductility. Congestion may also lead to limitations on the beam bar sizes relative to the joint dimensions. Furthermore, experimental and analytical work has shown that the yield strength cannot be developed fully in a stirrup leg adjacent to a hook or a bend [2]. This is

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because the high compressive stress at the inner face of the bends may cause crushing or splitting of the concrete, resulting in slip before the yielding force can develop.

A special type of reinforcement known as headed studs has been developed at the University of Calgary [3] for reinforcing thin concrete flat plates against punching shear in areas around the columns. A stud is a straight bar with an anchor head at one or both ends. Figure 1 shows a double-headed stud. The area of the head is 9 to 10 times the area of the stem. The heads eliminate the need for the development length required in conventional reinforcement and ensure that full yielding of the stem develops, with negligible slip, immediately behind the head [4]. Stud shear reinforcement has been used in flat slabs, footings, and raft foundations of hundreds of structures around the world. Double-headed studs have also been proposed for many other applications. Several recent investigations have shown that stud reinforcement provides better confinement and hence enhanced ductility of concrete elements. Tests have been carried out on corbels reinforced with double-headed studs as primary tension reinforcement [5] and on dapped-end beams with single- and double-headed studs used as shear reinforcement to the dapped-end zones [6]. Tests have also been conducted to demonstrate the effectiveness of double-headed studs in confining concrete in columns [7] and shear walls [8] under cyclic loading.

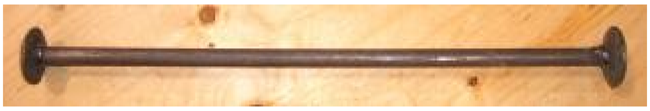


Fig. 1: A Double-Headed Stud

The research reported herein investigates the efficiency of using double-headed studs in lieu of conventional closed hoops and cross ties in concrete beam-column joints. The paper presents results of an experimental investigation into the behaviour of joints reinforced with double-headed studs under quasi-static cyclic loading producing high level of inelastic deformations similar to what may be experienced during severe earthquakes. Two different arrangements and amounts of studs are investigated. A comparison is made with the behaviour of a joint without shear reinforcement and with that of a joint conventionally reinforced according to the Canadian standards.

SHEAR STRENGTH PREDICTION MODELS AND CODE PROVISIONS

Two analytical models, known as the diagonal compression strut mechanism and the truss mechanism, are commonly used to describe the flow of forces in the joint core and to determine the amounts of vertical and transverse reinforcement required for developing the shear capacity of the joint [1]. Although the two models are widely debated, they constitute the basic concepts for the design and detailing of beam-column joints by different international standards. The provisions of the ACI Code, ACI 318-05 [9] and the Canadian Standards, CSA A23.3-04 [10] are based mainly on empirical formulation adopting the diagonal strut mechanism, in which the joint shear capacity is limited by the strength of the strut. Thus, the codes require large amounts of transverse reinforcement (closed hoops and cross ties) to ensure confinement sufficient enough to produce high strength of the strut.

The strut-and-tie model (STM), first proposed by Schlaich et al. [11], presents a more rational approach for explaining the transfer of forces and determining the amount of reinforcement needed in discontinuity regions. Hwang and Lee [12] proposed a "*softened*" strut-and-tie (SST) model for prediction of the shear strength of exterior beam-column joints under cyclic loading. The SST model accounts for the nonlinear constitutive relation and softening of concrete due to cracking. In the SST model, the resultant of the vertical and the horizontal shear in the joint is transmitted through inclined struts, a vertical tie, and a horizontal tie (Fig. 2). The vertical and the horizontal ties are represented, respectively, by the column intermediate vertical bars and the closed hoops and cross-ties in the joint. Proportioning the resultant among the struts and the ties is done through distribution factors. The forces in the ties are used to determine the required reinforcing area. The compressive stresses resulting on the nodes are checked against the "*softened*" strength of concrete. Hwang and Lee [13] adopted a softening coefficient, x, proposed by Zhang and Hsu [14] as:

$$x = \frac{5.8}{\sqrt{f_c'}} \frac{1}{\sqrt{1+400\,\varepsilon_r}} \leqslant \frac{0.9}{\sqrt{1+400\,\varepsilon_r}} \tag{1}$$

where f_c' is the characteristic compressive strength of concrete and e_r is the principal tensile concrete strain. To quantify softening of the concrete, the coefficient, x is multiplied by the concrete strength, f_c', and the corresponding strain, e_o. Assuming $e_r = 0.005$, Hwang and Lee [13] proposed the following approximation:

$$x = \frac{3.35}{\sqrt{f_c'}} \leq 0.52 \tag{2}$$

Application of the SST model leads to a smaller amount of joint reinforcement than that required by the code.

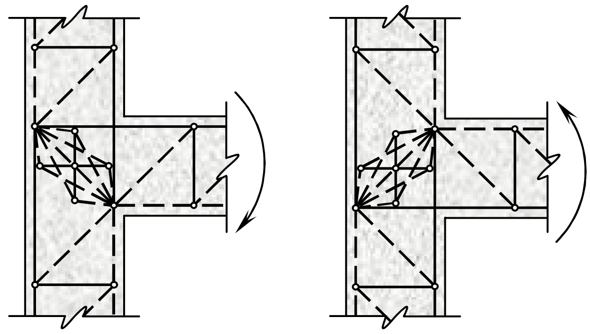


Fig. 2: SST Model by Hwang and Lee [12]

EXPERIMENTAL PROGRAM

A total of five full-scale beam-column joint specimens were tested under cyclic loading. Each specimen represented an exterior beam-column joint subassembly isolated at the points of contra-flexure from a typical multi-storey, multi-bay reinforced concrete frame. All the five specimens had the same concrete dimensions and the same reinforcing details in the beams and the columns. The beam length from the column face to the point of contra-flexure was 1.65 m. The column was 3.0 m high. Both the beam and the column had the same cross section dimensions of 300 mm width and 400 mm depth. The beam was reinforced with 4-20M bars top and bottom, while the column was reinforced with 3-20M bars on each side and 2 intermediate 15M bars. Size 10M closed hoops and stirrups were placed in the column and the beam in accordance with CSA A23.3 Code [10]. The specimens differed only in the reinforcing details of the joint. One specimen did not have any shear reinforcement in the joint and was used as shear-deficient control specimen. The joint of another specimen was designed according to the CSA A23.3-04 provisions and reinforced with conventional closed hoops and cross ties. The joints of the remaining three specimens were reinforced with two different arrangements and amounts of double-headed studs as described below.

Specimen Description and Designation

Specimen C-1 was a shear-deficient control specimen not provided with any transverse shear reinforcement in the joint (Fig. 3a).

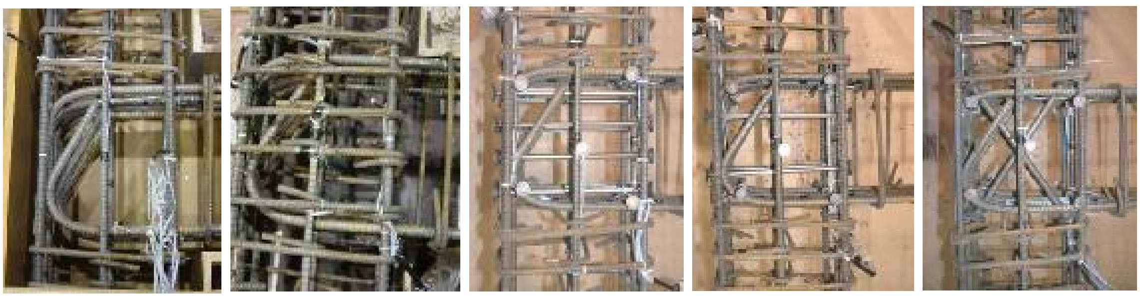
Specimen C-2 was another control specimen representing a joint detailed according to the CSA A23.3-04 specifications. The transverse reinforcement in the joint consisted of 4-10M ordinary closed hoops and 4-10M cross ties designed to resist the shear caused by the ultimate flexural capacities of the beam and column plus 25% over strength. Figure 3b shows the highly congested reinforcement in the joint.

Specimen SS-1 represented a joint where the conventional hoops in specimen C-2 were replaced by double-headed studs of equivalent reinforcing area. Eight ½ in. (12.7 mm) dia. double-headed studs were used in the in-plane direction (Fig. 3c). Five ⅜ in. (9.5 mm) dia. double-headed studs were provided to confine the joint concrete core in the out-of-plane direction. The area of these five 9.5 mm dia. studs was less than that of the confining reinforcement provided by the closed hoops and cross ties in specimen C-2.

Specimen SS-2 was similar to specimen SS-1 except that the horizontal double-headed studs were designed according to the SST model (Fig. 2) of Hwang and Lee [12]. According to this model, four ½ in. (12.7 mm) dia. double-headed studs were needed as shown in Fig. 3d to serve as a horizontal tie. The two 15M intermediate bars of the column served as the vertical tie. Similar to specimen SS-1, five 9.5 mm dia. double-headed studs were provided to confine the joint core in the out-of-plane direction. The nodal concrete stresses due to forces in all struts were checked against the softened concrete strength as recommended by Hwang and Lee [13]. The nodal stresses were found to be lower than the crushing strength.

Specimen SS-3 represented a joint reinforced by diagonal double-headed studs according to a strut-and-tie model developed by the authors as shown in Fig. 4. The area needed for the diagonal ties was provided by two ¾ in. (19.1 mm) dia. double-headed studs for each direction as shown in Fig. 3e. Again, five 9.5 mm dia. double-headed studs were provided in the out-of-plane direction. It should be noted that the area provided by these five studs was chosen arbitrarily and was kept constant for all the three specimens SS-1 to SS-3 in order to examine the effects of the layout and the amount of the in-plane double-headed studs on the shear strength of the joint.

Table 1 gives the concrete strengths of the five specimens on the day of testing. Table 2 summarizes the results of tension tests conducted on the various reinforcing bars and headed studs used in the specimens.



a) Specimen C-1 b) Specimen C-2 c) Specimen SS-1 d) Specimen SS-2 e) Specimen SS-3

Fig. 3: Reinforcing Details in Beam-Column Joint Specimens

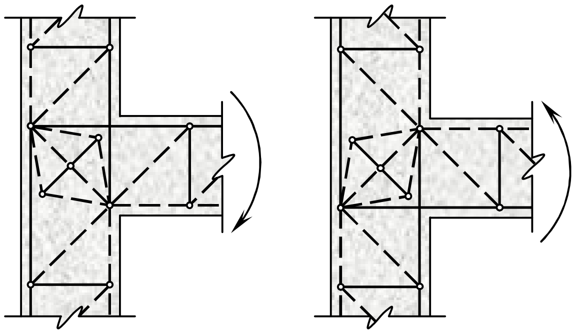


Fig. 4: SST Model with Diagonal Ties in the Joint

Table 1: Concrete Strength on the Day of Testing

Specimen	C-1	C-2	SS-1	SS-2	SS-3
Concrete strength (MPa)	35.5	34.5	36.3	36.7	33.7

Table 2: Properties of Reinforcing Steel

Reinforcement type	Cross-sectional area (mm^2)	Stress at yield (MPa)	Strain at yield	Ultimate stress (MPa)	Strain at ultimate stress
10M Bars	100	484	0.0034	760	0.0800
15M Bars	200	491	0.0034	772	0.0790
20M Bars	300	495	0.0034	782	0.0790
⅜ in. (9.5 mm) Studs	71	414	0.0032	531	0.0190
½ in.(12.7 mm) Studs	127	540	0.0026	591	0.0042
¾ in. (19.1 mm) Studs	287	576	0.0037	648	0.0017

Test Setup

Figure 5a shows schematically the deformed shape of a typical exterior beam-column joint subassembly in a multi-storey moment resisting frame subjected to lateral loads. The inter-storey drift angle, q, is defined as the column relative lateral displacement, d_{col}, divided by the column height, H. The drift angle can also be calculated using the beam-tip vertical displacement, d_{beam}, that would happen if the beam free body rotation were allowed as shown in Fig. 5a. For testing a beam-column joint specimen, it is practical to have the column ends pinned with restrained lateral displacement while the beam tip is displaced vertically (Fig. 5b). In this case, the column shear, V, from the test is given as:

$$V = \frac{F\left(L/2\right)}{H} \tag{3}$$

where F is the vertical force acting on the beam tip, and L and H are the beam length and column height, respectively. The corresponding column shear in the actual frame, V_{actual}, is calculated as:

$$V_{actual} = V - \theta\left(F/2 + P\right) \tag{4}$$

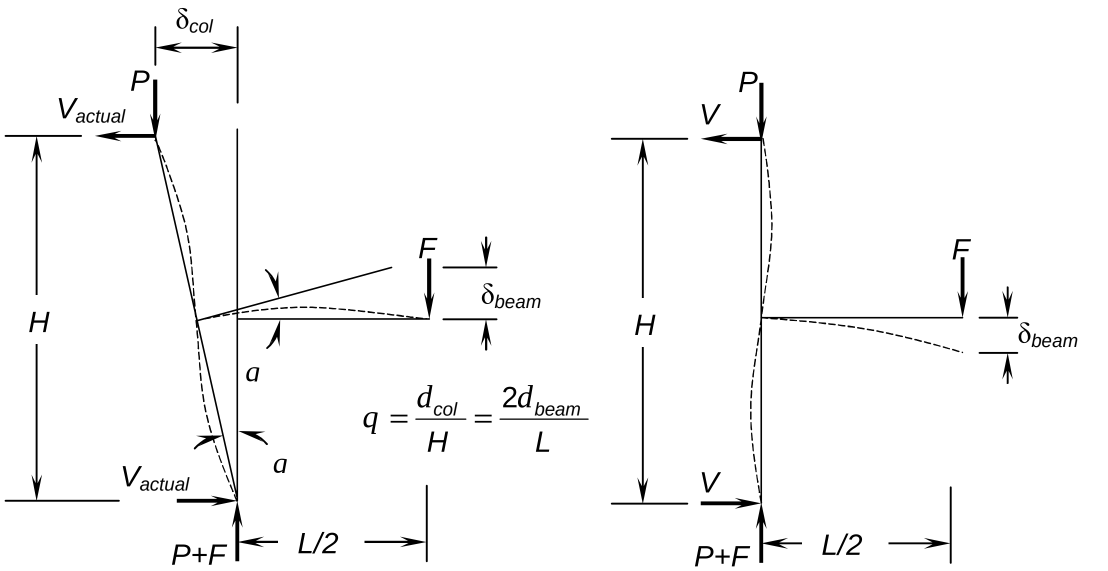
where P is the axial load on the column.

Figure 6 shows a schematic view of the test setup. A hydraulic jack was used to apply an axial force P = 360 kN to the column causing a compressive stress of 10% of the concrete strength. A

250 KN actuator was used to apply a force *F* at the beam tip in a displacement-controlled mode. The steel testing frame supports the reaction of both the hydraulic jack and the actuator. The column was pinned laterally at its top and bottom to the testing frame. The maximum moment, *M*, on the beam at the column face was equal to $F\,l$, with l = 1650 mm. The testing frame was laterally braced to the concrete frame of the structures lab. Lateral supports also restrained the frame in the out-of-plane direction. A roller support was provided under the column, while a spherical seat was transferring the axial force to the top of the column.

Loading Routine

After application of the 360 kN axial load at the top of the column, a quasi-static cyclic load was applied at the beam tip in a displacement-controlled mode. Quasi-static cyclic loading gives conservative estimate of the strength as the dynamic forces due to earthquakes increase the strain rate and, hence, the strength and stiffness [1]. The loading history consisted of series of three identical displacement cycles. The displacement amplitude was progressively increased from one series to another by a 5 mm increment in each direction (Fig. 7). This loading sequence was intended to produce high levels of inelastic deformations similar to those experienced by beam-column joints in a severe earthquake.



a) Inter-storey drift angle in idealized structure b) Inter-storey drift angle in test specimen

Fig. 5: Inter-Storey Drift Angle

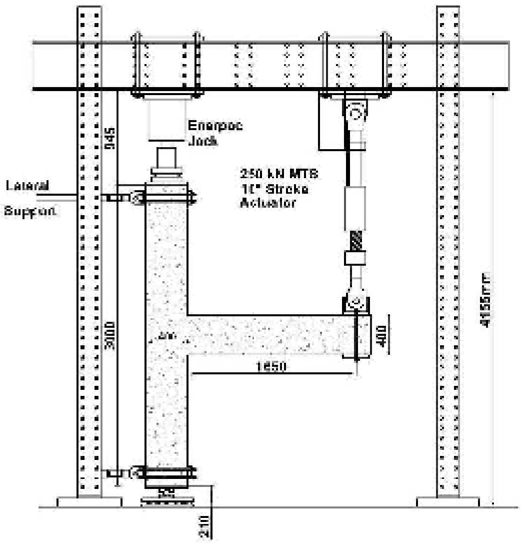


Fig. 6: Test Setup

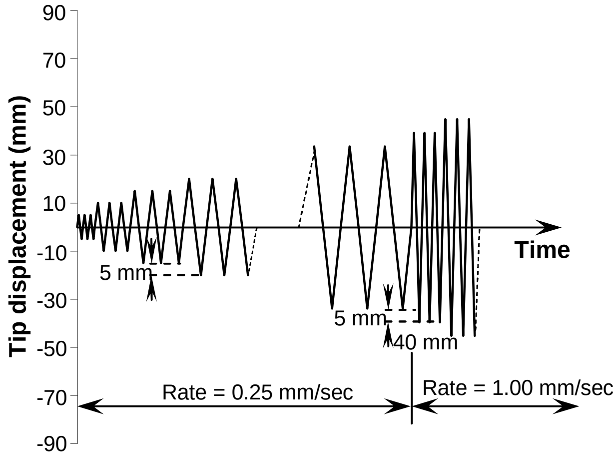
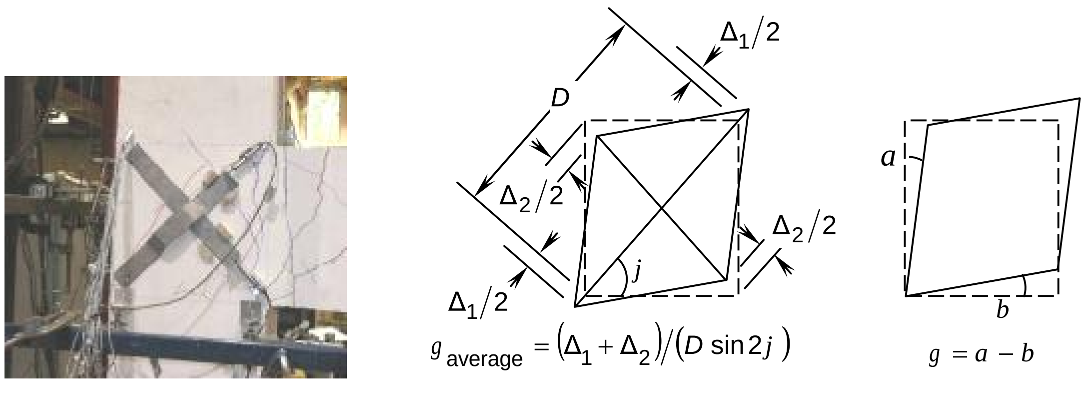


Fig. 7: Loading Routine

Instrumentation

The axial load on the column, *P*, and the vertical load, *F*, on the beam tip were measured by two load cells. Displacement transducers and spring potentiometers were used to measure the displacement at various locations. One spring potentiometer was attached to the beam tip at the point of application of the load *F* to measure the deflection, d_{beam}, under the load. Plotting the load-displacement hysteresis loops will give an estimate of the ductility and energy dissipation of each specimen.

Figure 8a shows the displacement transducers mounted diagonally on the joint to measure the joint shear deformations. The shear deformation angle, g, is calculated directly from the readings of the two diagonal transducers or from the deformation angles a and b with the vertical and the horizontal directions, respectively (Fig. 8b).



a) Displacement transducers b) Shear deformation angle

Fig. 8: Measurement and Calculation of the Joint Shear Deformation Angle

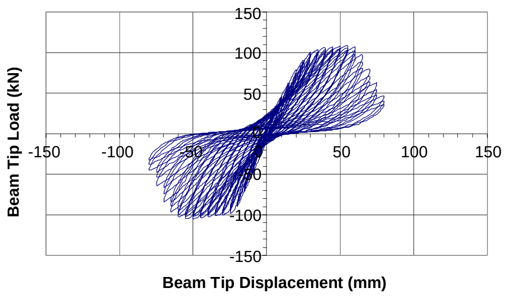
In order to measure the rotation of the plastic hinge in the beam, three displacement transducers were mounted on the top and bottom of the beam at the potential location of the plastic hinge, covering a distance of 600 mm from the column face. Electrical resistance strain gauges were used to measure the strain in the shear reinforcement in the joint and in the flexural reinforcing bars in the beam at locations near the column face, at mid-width of the joint and just before the bend at the end of the joint.

RESULTS AND DISCUSSION

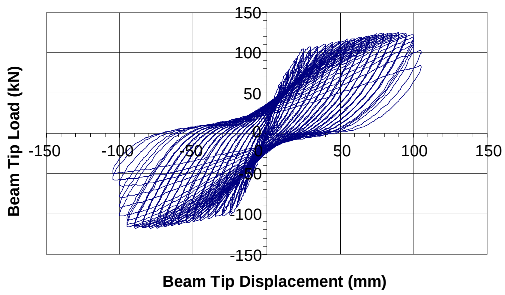
The hysteresis loops describing the beam-tip load-displacement relationship are plotted in Fig. 9 for each specimen. The modes of failure of all specimens are shown in Fig. 10. The load-displacement envelopes are compared in Fig. 11. The envelopes of the actual storey shear, Equation (4), versus the drift angle are plotted and compared in Fig. 12. The contribution of the joint to the storey drift angle in all specimens is compared in Fig. 13. The total cumulative energy dissipated in all specimens is presented in Fig. 14.

Specimens Behaviour

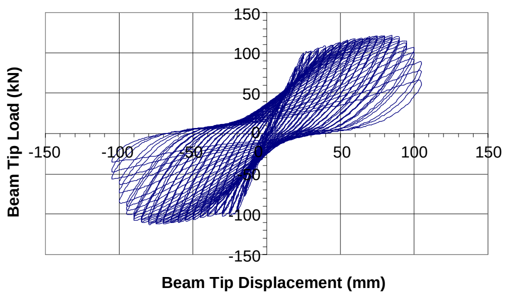
Specimen C-1: Since this specimen was designed to be shear-deficient without transverse reinforcement in the joint, failure was expected to occur in the joint. The first flexural crack appeared at 5 mm displacement in the beam at the column interface. The first diagonal crack appeared in the joint at 10 mm displacement. Diagonal cracks in the joint reached 0.1 mm width at 15 mm displacement. Yielding of the beam bars was first observed at 35 mm displacement at a 103.5 kN load, and the plastic hinge started to form in the beam initiating degradation of stiffness and increase in the area of the hysteresis loops (Fig. 9a). At 40 mm displacement, the crack widths reached 0.8 mm inside the joint. An ultimate load of 108.9 kN was reached at 55 mm displacement. At that load, the joint cracks became unstable and the load dropped till complete failure of the joint. Failure of the joint started with the formation of two major diagonal cracks propagating from the centre towards the upper and lower corners of the joint on the column edge. The two cracks passed through the centres of the bends of the beam bars in the direction of the high compressive stresses exerted by the bends on the joint concrete core. The cracks continued to propagate parallel to the column reinforcement and were accompanied by bulging of the joint in both the in-plane (at the column edge) and the out-of-plane directions. Bulging of the joint occurred due to an increase in the volumetric strain that resulted from lack of confinement of the joint core. Fig. 10a shows the specimen at failure.



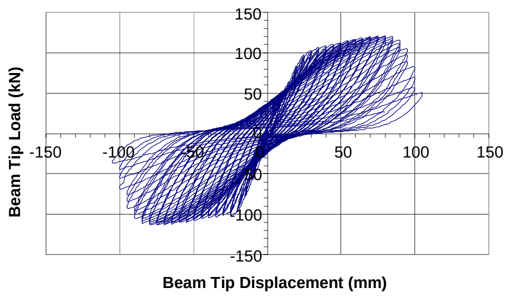
a) Specimen C-1



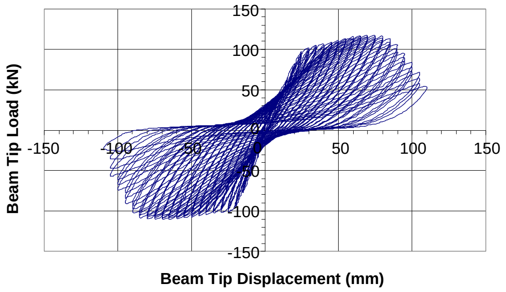
b) Specimen C-2



c) Specimen SS-1



d) Specimen SS-2



c) Specimen SS-3

Fig. 9: Load-Displacement Hysteresis Loops for the Test Specimens

a) Specimen C-1 b) Specimen C-2 c) Specimen SS-1 d) Specimen SS-2 e) Specimen SS-3

Fig. 10: Failure Modes

Specimen C-2: This specimen was expected to fail in the beam. Similar to specimen C-1, the first 5 mm displacement caused a flexural crack to appear in the beam at the interface with the column. At 10 mm displacement, the first diagonal crack appeared at the top corner of the joint, extending from the first flexural crack at the beam-column interface. At 15 mm displacement, three diagonal cracks were observed in each direction within the joint, the largest of which was 0.22 mm wide. Yielding of the beam bars was first observed at 30 mm displacement at a 105.8 kN load, at which the plastic hinge started to form in the beam. After yielding, stiffness degradation started, accompanied by an increase in the area of hysteresis loops (Fig. 9b). The crack widths kept increasing and the largest crack reached 0.7 mm width at 60 mm displacement. A peak load of 123.8 kN was reached at 90 mm displacement, at which the joint cracks stabilized while the beam cracks continued to widen. The load dropped till failure of the beam at the plastic hinge. Figure 10b shows the failure at the plastic hinge. Throughout the test, no yielding was observed in the closed hoops or in the cross ties reinforcing the joint. However, high levels of strain were recorded. The strain in the closed hoops reached 2100 με in the in-plane direction and 2500 με in the out-of-plane direction. The strain in the cross ties reached 1000 με.

Specimen SS-1: The first beam crack appeared in this specimen at the column interface at 5 mm displacement. No cracks appeared in the joint for displacement less than 20 mm. Yielding of the beam bars started at 35 mm displacement at a load of 105.4 kN and the plastic hinge started to form in the beam initiating degradation of stiffness and increase of the area of hysteresis loops (Fig. 9c). At 40 mm displacement the largest crack width recorded was 0.36 mm wide. At 80 mm displacement the crack width reached 1.0 mm. A peak load of 121.5 kN was reached at 85 mm displacement. No change was observed in the load when the displacement reached 90 mm. Two major diagonal cracks started to appear in the joint at 90 mm displacement. However, the cracks in the beam started to widen rapidly, at a much higher rate than the increase of crack width in the joint, and the maximum load continued to decrease till failure occurred in the beam at the plastic hinge at 105 mm displacement. Bulging of the joint occurred only in the out-of-plane direction. No yielding occurred in the shear studs, and the maximum strain recorded was 2500 με at 100 mm displacement. The studs perpendicular to the joint yielded after cracking at 95 mm displacement, and the maximum strain recorded was 4000 με. Figure 10c shows failure of the beam at the plastic hinge accompanied by wide cracks at the joint.

Specimen SS-2: The first beam crack appeared at the column interface at 5 mm displacement, and the first joint crack developed at 10 mm displacement and reached 0.24 mm width at 15 mm displacement. Yielding of the beam bars and formation of the plastic hinge, initiating the stiffness degradation and the increase of the area of hysteresis loops, started at 35 mm displacement at a load of 105.8 kN (Fig. 9d). The maximum crack width reached 0.4 mm, 0.54 mm, and 1.6 mm at displacements of 40 mm, 60 mm, and 80 mm, respectively. A peak load of 120.6 kN was reached at 75 mm displacement and remained constant till 85 mm displacement. The peak load started to drop and the cracks in the joint continued to increase in width, while the beam cracks were almost stable until failure took place in the joint. The two major cracks that initiated failure of the joint in specimen C-1 were also observed in specimen SS-2 at 80 mm displacement. Appearance of these cracks was followed by bulging of the joint indicating an increase in the volumetric strain. Figure 10d shows failure of the specimen at the joint. The two upper horizontal studs yielded at 80 mm displacement and the strain reached 2900 με at 100 mm displacement. However, the two lower studs did not yield and the strain reached 1700 με. The studs in the out-of-plane direction yielded following extensive cracking that occurred at 80 mm displacement and the strain exceeded 4500 με.

Specimen SS-3: Figure 9e shows the load-displacement hysterisis loops for this specimen. Failure was expected to occur in the beam since the joint was designed to carry the beam ultimate capacity plus 25 percent over strength. The first beam crack was recorded at the column interface at 5 mm displacement, and the first joint crack was seen at 10 mm displacement and reached 0.06 mm width at 15 mm displacement. Yielding of the beam bars was first observed and the plastic hinge started to form at 35 mm displacement and a 104.4 kN load. The crack width continued to increase and the largest crack reached 1.4 mm width at 60 mm displacement. A peak load of 115.9 kN was reached at 75 mm displacement and kept almost constant till 80 mm displacement, after which the joint cracks continued to widen while the beam cracks became stable. Failure started in the joint by formation of two major cracks similar to those observed in specimens C-1 and SS-2 (Fig. 10e). The peak load dropped till complete failure of the joint. The joint core was not badly damaged. No yielding was observed in the diagonal studs. The maximum strain reached was 2800 με. The out-of-plane studs yielded at 80 mm displacement and the strain reached 16700 με.

Load-Displacement and Storey Shear Response

Envelopes of the load-displacement hysteresis loops are presented for all specimens and compared in Fig. 11. Envelopes for the relationship between the actual storey shear and the inter-storey drift angle are plotted and compared in Fig. 12. The figures show clearly that specimens SS-1, SS-2, and SS-3, reinforced with double-headed studs in the joint had much better behaviour than the shear-deficient specimen C-1 and were close in their behaviour to specimen C-2 reinforced with closed hoops according to the code provisions.

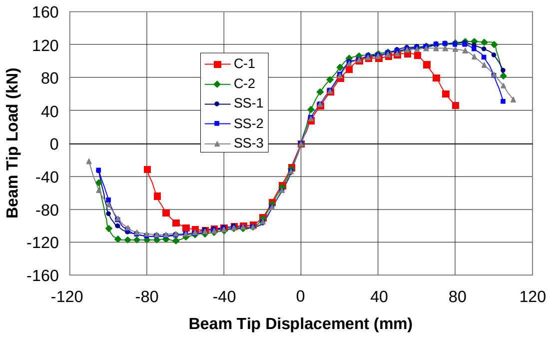


Fig. 11: Load-Displacement Envelopes

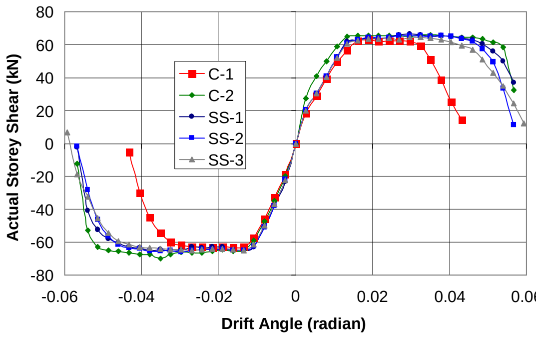


Fig. 12: Storey Shear Versus Drift Angle

Specimen SS-1 exhibited a response very close to that of specimen C-2 and failed in a desirable mode at the plastic hinge in the beam. However, the joint of specimen SS-1 experienced more cracking than the joint in specimen C-2. This is attributed to the smaller confinement provided by the five 9.5 mm dia. double-headed studs in the out-of-plane direction in specimen SS-1 in comparison to the confinement provided in specimen C-2 by the cross ties and the closed hoop legs perpendicular to the joint plane.

Specimen SS-2 was designed according to the SST model by Hwang and Lee [12]. The area of the four 12.7 mm dia. studs provided was enough to carry the force in the horizontal tie, and the concrete was checked against the softened strength calculated using the simplified Equation (2). In the test, the tie did not experience strain levels higher than what was predicted. However, the joint failed by crushing of concrete at a load close to the joint strength. This is attributed again to the lack of sufficient confinement. The SST model does not take into account the effect of confinement in the out-of-plane direction. The test results used by Hwang and Lee [12] to verify the SST model were for specimens reinforced with cross ties and closed hoops with reinforcing area sufficient to provide good confinement of the joint in the out-of-plane direction. If the joint of specimen SS-2 was reinforced with 2 closed hoops and 2 cross ties (No. 10 bars with yield strength of 484 MPa, see Table 2) in place of the double-headed studs, the force in the steel perpendicular to the joint would have been 290 kN. In comparison, the five 9.5 mm dia. studs with yield strength of 414 MPa provide a force of 147 kN, and hence, less confinement. This underlines the importance of confinement in the out-of-plane direction of the joint.

The amount of shear reinforcement provided in the joint of specimen SS-1 is twice the amount provided in the joint of specimen SS-2. However, at almost the same peak load, the steel strain levels were high and of comparable magnitude in both joints. This indicates that the stud shear reinforcement in specimen SS-1 carried a higher force than that in specimen SS-2. The increase in the force is attributed to the better in-plane confinement provided by the eight headed studs in specimen SS-1. This proves that, contrary to the conclusions of Hwang and Lee [12, 13], the transverse reinforcement in a beam-column joint is needed not only to contribute to the joint shear resistance, but also to provide confinement to the joint concrete core and compression strut.

Specimen SS-3 exhibited the least desirable behaviour of the three specimens reinforced with studs. As the diagonal studs were placed in the direction of the principal tensile stresses, they fulfilled their role as ties and experienced strains well below the yield strain. However, their effect in confining the compression struts was not great. The nodal zones at the beam bars bends, which had the highest compressive stresses and accompanying perpendicular principal tensile stresses, were not fully affected by the compression cones of the stud heads. This again indicates the importance of the confinement to the joint core.

Joint Contribution to the Storey Drift Angle

The joint contribution, q_j to the storey drift angle, q (Fig. 5) can be calculated from the following equation of Alcocer and Jirsa [15]:

$$q_j = g\left(1 - \frac{h_c}{L} - \frac{h_b}{H}\right) \tag{5}$$

where g is the joint shear deformation angle (see Fig. 8) and h_b and h_c are the depth of the beam and the column cross-section, respectively.

The joint contribution is expressed as percentage of the total drift angle and presented in Fig. 13 for all the specimens. As expected, the joint of specimen C-1 had the largest contribution due to its large deformations. In specimen C-1, the joint contribution increased from zero to a maximum of 29.6 percent at a drift angle of 0.0162 radians, corresponding to the start of yielding of the beam bars at 35 mm displacement. Because of the increase in the beam deformations after yielding, the joint contribution decreased to 21.6 percent at a drift angle of 0.0324 radians, which was reached at the peak load sustained by the specimen at the beam tip. Because of the extensive joint cracking that followed the peak load, the joint contribution increased rapidly till failure.

The joint contribution in specimen C-2 remained practically constant after reaching 10 percent at yielding of the beam bars. Before reaching the peak load, the joint contribution in specimens SS-1 and SS-2 was smaller than that in specimen C-2 and ranged between 3 to 8 percent, indicating higher stiffness of the joint. Due to excessive cracking after the peak load, the joint contribution increased slightly in specimen SS-1 and dramatically in specimen SS-2.

The joint contribution in specimen SS-3 was close to that of C-1 at small drift angles. This shows that the diagonal studs were still not in effect. After early cracking of the joint, the studs became effective and played their role as ties, hence controlled the crack width. Consequently, the joint contribution decreased and became close to that of C-2 for drift angles between 0.02 and 0.04 radians. For drift angle larger than 0.04 radians, the lack of confinement to the joint core induced larger cracks and crushing started to take place causing an increase in the joint contribution till failure. The joint in specimen SS-3 had the largest contribution at large drift angles.

Cumulative Energy Dissipation

The ability to dissipate energy is the most important factor in seismic design since the higher the ability of the structure to dissipate energy the higher its chances to survive an earthquake. The energy dissipated in each load cycle is defined by the area enclosed by the hysteresis loop in the load-displacement diagram. The cumulative dissipated energy is the sum of the energy dissipated in the hysteresis loops. The cumulative dissipated energy for each of the five specimens at different displacement levels is presented in Fig. 14. As can be seen, specimen C-2 had the highest energy dissipation. The energy dissipated by specimens SS-1 and SS-2 was very close to that of specimen C-2. The total energy dissipation was about 90 and 81 percent of that of C-2 for SS-1 and SS-2, respectively. Specimen SS-3 dissipated about 70 percent of the total energy dissipated by C-2. Naturally, specimen C-1 had the lowest energy dissipation due to lack of shear reinforcement and confinement in the joint. These results are consistent with those of the joint contribution to the drift angle shown in Fig. 13, since the larger the joint deformation, the more the pinching in the hysteresis loops, and hence, the smaller the enclosed area in each loop.

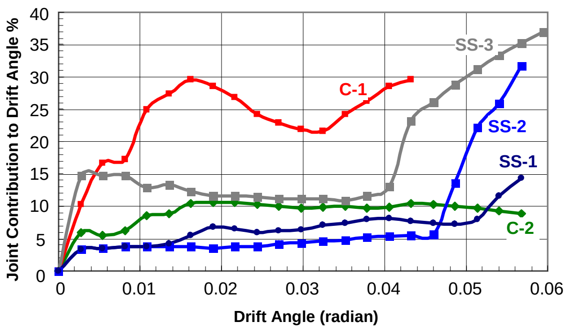


Fig. 13: Joint Contribution to Drift Angle

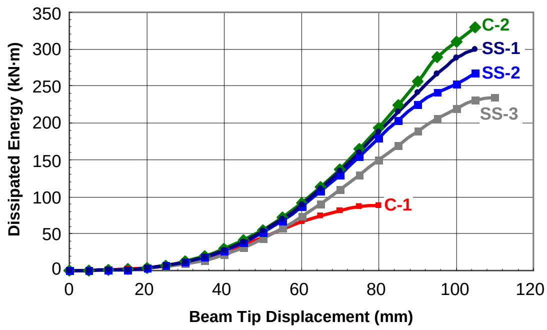


Fig. 14: Dissipated Energy vs. Displacement

CONCLUSIONS

Use of double-headed studs is a viable option for shear reinforcing of exterior beam-column joints. The three test specimens reinforced with shear studs in the joint achieved considerable enhancement in their behaviour under cyclic loads in comparison to a shear-deficient joint, and exhibited a performance close to that of a joint reinforced with closed hoops and cross ties according to the code. Further studies are in progress to improve detailing of the joint reinforcement with studs in order to achieve a desirable mode of failure. The following are important conclusions drawn from the present research:

1. The transverse reinforcement in the joint is important for both the shear resistance and confinement.

2. A strut-and-tie design model is proposed with diagonal ties within the joint. The model is not possible to develop without the use of double headed studs.
3. Design according to the softened strut-and-tie model or the proposed model with diagonal ties should take into account the need for out-of-plane confinement.
4. Use of double-headed studs as shear reinforcement reduces congestion in the joint considerably and makes assemblage of the cage much easier.
5. Replacing conventional closed hoops and cross ties in a code designed joint by horizontal studs of equivalent reinforcing area is effective and leads to a competitive behaviour of the joint. However, sufficient out-of-plane confinement is necessary to control excessive joint cracking.
6. The proposed diagonal stud arrangement performs well as ties, but is not sufficient for confining the joint nodal zones.

ACKNOWLEDGMENT

This research was financially supported by the Natural Sciences and Engineering Research Council of Canada (NSERC) and the Cement Association of Canada (CAC). This support is gratefully acknowledged. The authors also wish to express their gratitude to Decon Canada for the donation of the headed studs.

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BEHAVIOR OF STRENGTHENED MASONRY WALLS

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ABSTRACT

Masonry walls are one of the oldest construction methods. Lots of buildings were built with un-reinforced masonry all around the word. Cracking and failure of un-reinforced masonry walls can be caused by differential settlement, earthquakes, structural deficiencies, and inplane and out of plane loads. Seismic performance of un-reinforced masonry needs to be studied. Un-reinforced masonry walls exhibit a brittle response to lateral load due to limited tensile and shear strengths and lack of ductility and energy absorption capacity. Methods of repair and seismic upgrading of these walls are needed. The main objective of this paper is to study the behavior of un-strengthened and strengthened masonry walls. The walls are strengthened using glass or carbon fiber reinforced polymers (GFRP or CFRP). Three specimens are constructed using clay brick units. These walls are tested under combined vertical and horizontal loads. Horizontal displacements at the top of the walls were monitored along with the vertical displacements at the heal and the diagonal displacements. The ultimate load capacity and the mode of failure were also monitored. The efficiency of the strengthening techniques were demonstrated via the increase in strength and deformation ability of the walls without increase in their stiffness.

Keywords: Brick Walls, Strengthening, Behavior, FRP Reinforcement.

INTRODUCTION

Most of existing buildings in Egypt were constructed with un-reinforced masonry. The masonry elements in these buildings have been designed to resist gravity loads in bearing masonry construction, with no consideration of the forces generated by seismic events. The earthquakes cause severe inplane and out of plane forces to masonry walls resulting in damages range from minor cracks to complete collapse.

The use of fiber reinforced polymers (FRP) for strengthening of brick walls may prove to be a good alternative. Fiber reinforced polymers (FRP) have good corrosion resistance and low weight. They are easy to handle. However, fiber reinforced polymers are generally more expensive than many traditional materials. Meier et al. [1] indicated that for bridge strengthening, a 94 kg of steel could be replaced with 4.5 kg of carbon fiber composite sheets. Hadad et al. [2] tested masonry walls with openings. Some of the walls were strengthened using GFRP strips around the openings only. They concluded that GFRP is an efficient method for increasing the load carrying capacity and ductility of masonry walls with openings. Tumialan et al. [3] presented results of an experimental program on the flexural behavior of masonry walls strengthened with externally bonded fiber-reinforced polymer (FRP) laminates. They found that strength and ductility of walls can be significantly increased by strengthening them with FRP laminates. Velazques-Dimas et al. [4] investigated the flexural behavior of slender

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masonry walls retrofitted with vertical glass fabric composite strips and subjected to cyclic out of plane loading. The walls were capable of supporting pressures of up to 25 times their weight and deflect up to 1/20 times the wall height.
In this project two types of fiber reinforced polymers are used to strengthen the masonry walls. These are glass fiber reinforced polymers (GFRP) and carbon fiber reinforced polymers (CFRP). Description of the specimens, the test setup, the instrumentation, and discussion of the results are presented in this paper.

RESEARCH SIGNIFICANCE

This paper shows the experimental tests on masonry walls strengthened with GFRP and CFRP sheets and tested under static load up to failure. The project describes the performance and mode of failure of such walls. The horizontal load- horizontal displacement at the top of the walls, the horizontal load- vertical displacement at the heal of the walls, the lateral load versus the diagonal displacement, the ultimate load capacity and the mode of failure are described in the paper.

EXPERIMENTAL PROGRAM

Tests were carried on 3 clay brick masonry walls. One of the walls was strengthened by placing GFRP sheets on a complete one face and was referred to as WG. Another was strengthened by placing CFRP sheets on a complete one face and was referred to as WC. The plain wall (control wall) was referred to as WP. The three walls were subjected to a constant vertical load of 8 ton and an increasing horizontal load up to failure. The dimensions of the used local perforated clay bricks are 220*100*60 mm. Each brick had ten holes of 25 mm diameter each. As shown in Figure 1.0, each wall is 940 mm long (4 bricks), 1050 mm high (15 courses) and 60 mm thick with an aspect ratio of 1.12. A reinforced concrete beam of 100*100 mm cross section is cast on the top of each wall

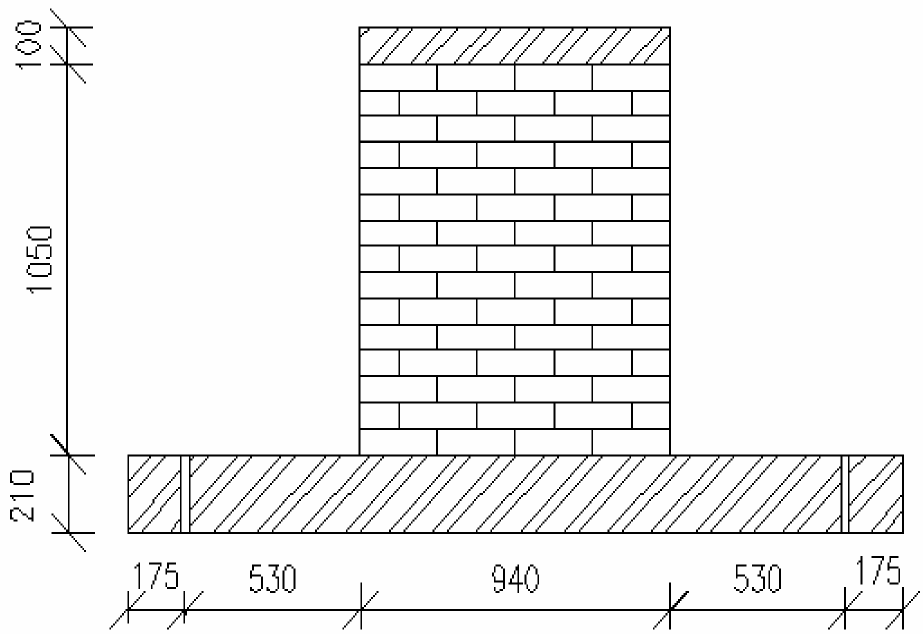

Fig.1: Schematic Drawing of Tested Masonry Walls

Material Caracteristics

The mechanical properties of masonry walls depend on the characteristics of the bricks and mortar. Masonry prisms made of 3 courses high were built in the same time with the tested walls using the same type of mortar and bricks. The tested prisms gave an average compressive strength of 5.91 MPa. Compressive strength test was performed on unit brick which gave strength of 11.36 MPa. Mortar cubes of 50*50*50 mm dimensions were tested in compression after a period of 28 days of curing to reveal an average strength of 30.61 MPa.
The walls were strengthened using unidirectional glass fiber reinforced polymers (GFRP) or carbon fiber reinforced polymers (CFRP). The GFRP comes in rolls of 500 mm width and the CFRP comes in rolls of 300 mm width. They were placed on the walls in the horizontal direction. Their properties, as reported by the producing company, are shown in Table 1.0.

Table 1.0: Properties of GFRP and CFRP Fabrics

	Area density	Effective thickness	Tensile strength	Ultimate deformation	Modulus of elasticity (fiber modulus)
GFRP	600 gm/m^2	0.23 mm	1700 MPa	0.028	80,000 MPa
CFRP	200 gm/m^2	0.117 mm	3800 MPa	0.0155	240,000 MPa

Test Arrangement

Figure 2.0 shows the loading and instrumentation located on the tested walls. The walls were subjected to inplane constant vertical load of 8 ton

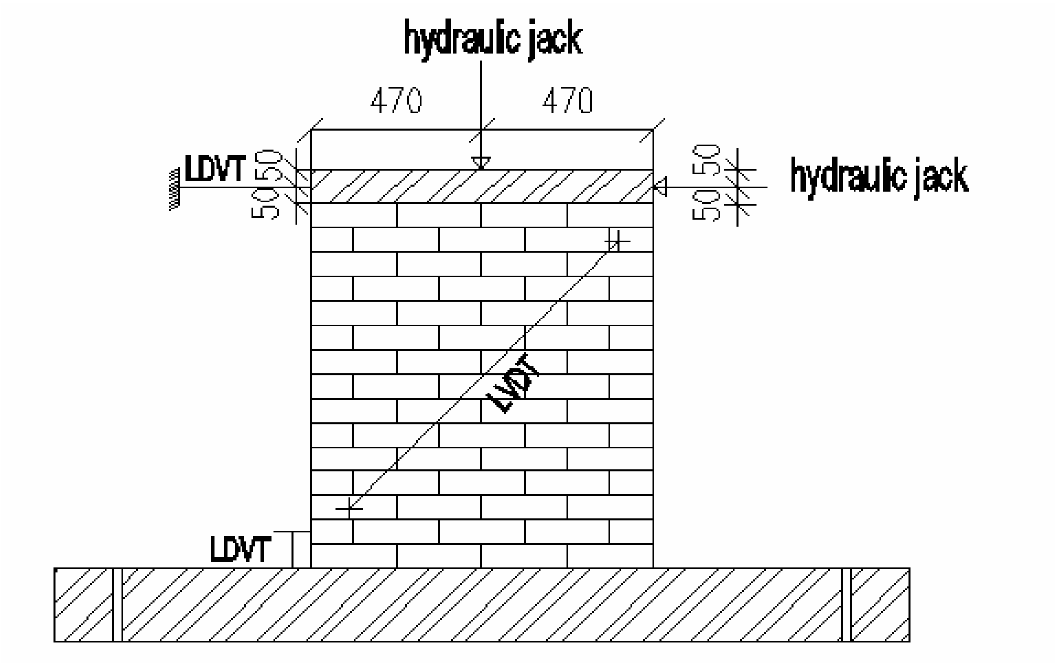

Fig. 2: Schematic View of Test Setup

and monotonically increasing lateral load. The vertical load was applied at the middle of the wall using a hydraulic jack supported at the top steel beam of the testing frame. The horizontal lateral load was applied through a hydraulic jack connected to a calibrated load cell at the level of the top reinforced concrete lintel. Linear variable differential transducers (LVDT) were attached to the wall to monitor the response. All the instrumentations were monitored through a computerized control station. The recorded data included the horizontal lateral load versus the horizontal displacement, the diagonal strain and the vertical strain at the heal.

TEST RESULTS

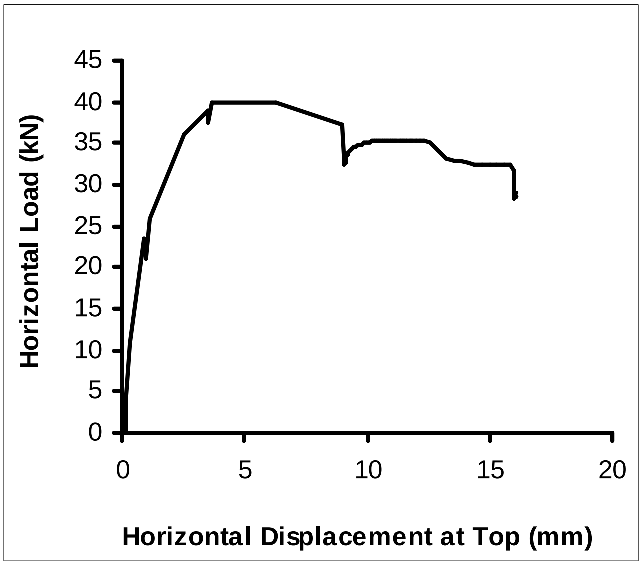

Fig. 3: Horizontal Load versus Horizontal Displacement for Wall WP

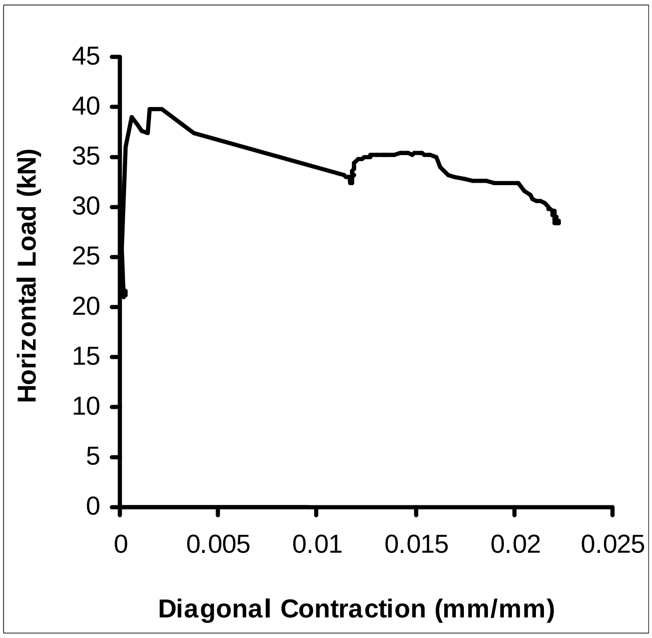

Fig. 4: Horizontal Load versus Diagonal Strain for Wall WP

Fig. 5: Crack Pattern of Wall WP

Figure 3.0 shows the lateral load against the lateral deflection of the solid plain wall (WP). The maximum attained lateral load is 40.0 kN and the maximum attained horizontal displacement is 16 mm. the recorded maximum diagonal strain for wall WP is 0.0222 as clear from Figure 4.0. Diagonal crack appeared in wall WP, as shown in Figure 5.0. This diagonal crack forms when the tensile stresses developed in the wall under combination of vertical and horizontal loads exceed the tensile strength. With pushing the wall the crack opened causing the collapse of the wall.

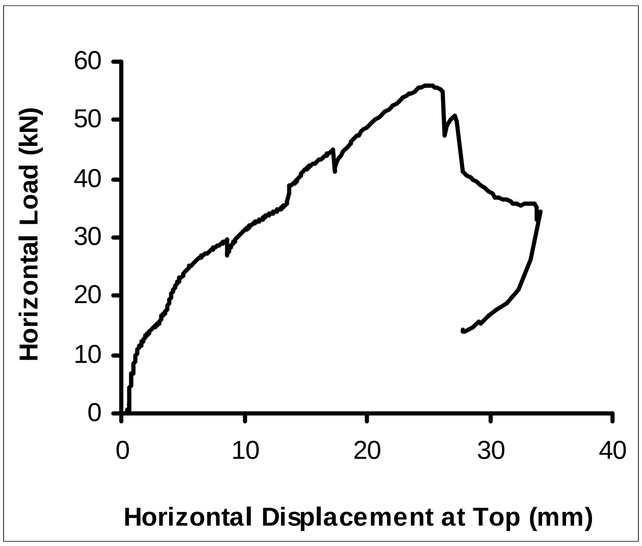

Fig. 6: Horizontal Load versus Horizontal Displacement for Wall WC

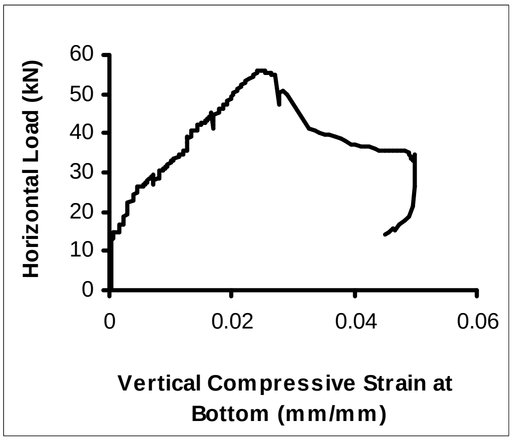

Fig. 7: Horizontal Load versus Vertical Strain at the Heal for Wall WC

Fig. 8: Failure Mode for Wall WC

Wall WC resisted maximum horizontal load of 55.0 kN (see Figure 6.0) and accommodated displacement of 34 mm. The maximum vertical strain at the heal is 0.0499 (see Figure 7.0). The failure of the wall occurred due to overturning moments which caused crushing of the bricks in the zone of the heal as given in Figure 8.0.

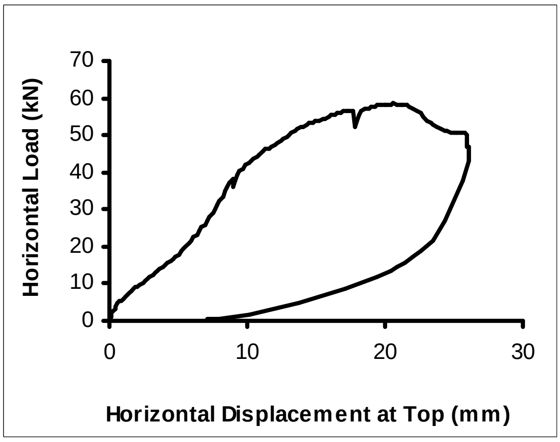

Fig. 9: Horizontal Load versus Horizontal Displacement for Wall WG

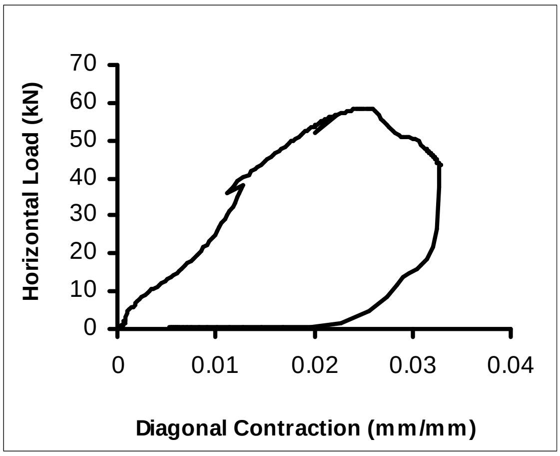

Fig. 10: Horizontal Load versus Diagonal Strain for Wall WG

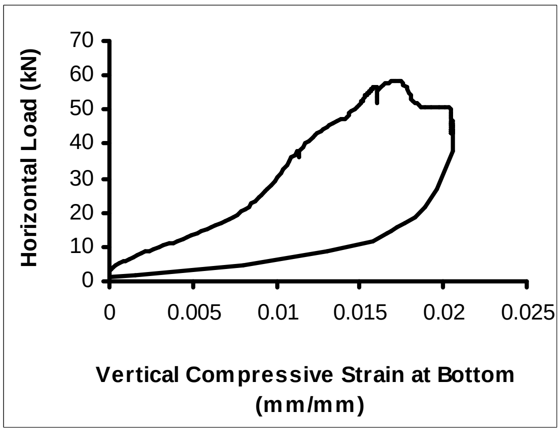

Fig. 11: Horizontal Load versus Vertical Strain at the Heal for Wall WG

Fig. 12: Crack Pattern of Wall WG

As recorded in Figure 9.0, the maximum horizontal load was 58.0 kN for wall WG and the maximum horizontal displacement was 26 mm. The diagonal strain for wall WG reached 0.033 as seen in Figure 10.0 while the vertical strain at the heal reached 0.0205 as seen in Figure 11.0. Similar to wall WP, splitting crack developed in the diagonal direction of wall WG (see Figure 12.0). Final failure occurred when the wall was divided into two parts and the GFRP laminate was ruptured.

Comparing Figures 3.0, 6.0, and 9.0, we can notice that the fiber reinforced polymers increased the lateral load capacity and the displacement of the walls. The increase in the maximum load reached 37.5% for wall WC and 45% for wall WG. The maximum horizontal displacement increased by 112.5% for wall WC and by 62.5% for wall WG. Thus, the maximum displacement is higher for the case of wall strengthened by carbon fiber reinforced polymer. The diagonal strain was 48.6% larger for reinforced wall (wall WG) compared with plain wall (wall WP) as proved by Figures 4.0 and 10.0. Figures 7.0 and 11.0 show that wall WC attained higher vertical strain at the heal compared with wall WG. The vertical strain at the heal of wall WC was 143.4% higher than that of wall WG.

CONCLUSIONS

1. The use of fiber reinforced polymers to reinforce brick walls results in gain in the wall strength and ductility (top horizontal displacement, diagonal strain and vertical strain at the heal).
2. The increase in the top horizontal displacement and vertical strain at the heal is higher for the case of walls reinforced by CFRP.

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AN APPROACH TOWARDS, APPROPRIATE ARCHITECTURAL SPACE DESIGN & DISASTER PREVENTION PLANS FOR THE ELDERLY & VISUALLY IMPAIRED PEOPLE

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ABSTRACT

Appropriate Architectural design for both indoor and outdoor spaces should concern all targeted groups of users, including children, elderly people as well as visually impaired people, by which these groups become the focus of concern in socio-cultural & human aspects for the sake of their physical & psychological development.

Any society should maintain its coherence by observing these groups by care & love in order to achieve a successful melted & highly productive society. For that reasons, this study was prepared to pursue some architectural design fundamentals helping visually impaired people concerning supportive element e.g. entrances, corridors, passageways, & toilets.

Keywords: Appropriate design, Visually Impaired People, Barrier free issue, supportive elements.

INTRODUCTION

It is important for buildings used by the general public to generalize and satisfy a broad range of needs and requirements of their users, whereas buildings whose users are more specific (facilities for specified users) have to consider and make design allowances for the needs and characteristics of those specific users.

There are also buildings where some sections are used by the general public, and other sections are only accessible to specific users. In these cases, the facilities should be designed according to their usage. Examples of this include regional community areas or day service centers collocated with special nursing homes for the elderly.

In the case of buildings whose users and usage are specified to a certain degree, an input by potential users in the design stage will help to ensure the building meets their specific needs as much as possible, & when constructing a building, the evacuation of elderly and disabled people must be fully examined, and incorporated into the plan. Buildings in the 21st century are large and complex, and it is not always clear where the evacuation routes are. Evacuating elderly and disabled people requires considerable time and effort, and ensuring this in a smooth process is becoming increasingly difficult. The basic idea of disaster prevention plans is that access and evacuation routes must be easy for all people to understand, and this of course includes the elderly and disabled. That is, a clear understanding when entering the building, and a clear understanding when evacuating in an emergency must be the basis of any disaster prevention plan.

For their safe evacuation, the fact that an emergency (fire, etc.) exits must be conveyed to building users without delay. There is a need to look into various ways of informing them of any

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danger, such as sound, lights, and other forms of personal help (e.g., from workmates in a workplace). However considering the extent of overtime and other work patterns in the modern office, relying only on colleagues to inform and provide assistance is quite risky indeed.
In a fire, the first evacuation priority is to be able to move away from the source of the fire. If a route has been secured, evacuees can make their way to the evacuation floor, and from there out of the building. When preparing specific designs, architects must confirm there are no obstacles to wheelchair users when passing through firewalls.
There is a need to secure refuges and temporary shelters for the safety of elderly and disabled people who require considerable time or assistance from others to evacuate after escaping through the fire doors. From this perspective, it would be effective to prepare temporary shelters separate from the line of evacuation in fireproof sections such as emergency elevator lobbies, emergency stairs and attached rooms, and balconies.
To move to the evacuation floor, people will have to use the stairs, elevators, and other means depending on the extent or state of their disability, and careful consideration must be given to establishing equipment and facilities according to the characteristic needs of users.
Most of these points can be achieved by a physical response (including equipment), but some will also need a personnel response.

1. Promoting the development of an environment that facilitates access by elderly and physically disabled people

Concept of buildings that facilitate access by elderly and physically disabled people

(1) What are buildings accessible to all?

In revising design standards, the author envisaged a range of citizens as possible as building users , and this was according to the background information supplied by the Egyptian code for the elderly and disabled people (ministry of housing , utilities and urban development , 2004) . That is, to promote buildings that are easily accessible and usable by all people. The following are the ideas of what constitutes easily accessible buildings [1-2].
• Buildings accessible to all are those buildings that are connected with regional life, regardless of whether they are public or private, and whether they are for work, recreation or learning.
• While it is by no means a simple task to develop buildings that can be used by all people equitably, a range of parties including citizens, building contractors and administrative agencies need to work together from all perspectives to develop a physical environment that gives full consideration to ease of access and use by elderly and physically disabled people.
• Designing buildings that can be used equitably by all people requires a thorough study of how the various users will actually use the buildings. Building owners and architects must therefore seek the input of citizens and users as required to understand their needs, and from this, develop buildings that can be easily accessed and used by as many people as possible.
• Creating a barrier-free building demands not just work on the internal sections, but a comprehensive approach that integrates the physical and human aspects of the building's development so that it can be used continuously and safely from the road or building grounds through to the intended destination inside the building.

(2) Measures for elderly and physically disabled people

The elderly, physically disabled, children and infants, and foreigners have special needs when using buildings, and it is these needs in particular that we have to understand. For example, wheelchair users have differing needs depending on their disability level, and wheelchair propulsion. Visually impaired people also have a range of needs in building use according to the age at which they became visually impaired, the extent and particulars of their visual impairment, life experience after impairment, vocational experience, and how often they go out of the building. So we have to have a firm understanding of these diverse needs. In the case of hearing impaired people, there is a difference in the means of communicating to people who are profoundly deaf and those who have a moderate hearing impairment, so there is a need to learn about these differences.

The following points should be noted,(according to the Egyptian code for the elderly & disabled people [3].
• While these design standards are the objectives of the development, there are cases where the development methods will be different according to the needs of users, how the facilities are used, construction costs, and the location and sitting of the development, and this will require the ingenuity of the building owners and architects. They must strive to avoid simply applying the design standards uniformly [2].
• A barrier-free response must be achieved through building space and equipment, but at the same time, assistance in the form of welfare devices and staff (e.g., carers, sign language interpreters, and guides) should also be examined. For example, where the building is likely to be used by children or intellectually disabled people, consideration should be given to positioning staff to provide any necessary assistance.
• Elderly and disabled people tend to be restricted especially when evacuating during a fire, earthquake or other disaster, so full consideration should be given to securing safe evacuation routes, establishing evacuation areas, and alarm systems. Special care should be taken in planning buildings used primarily by elderly or disabled people.

(3) Integrating the physical and non-physical aspects

To encourage and help elderly and physically disabled people to take a more active role in the community, there is a need to provide support in both the physical and non-physical aspects, so the following points should be considered;

• While a barrier-free response should essentially be done through the physical aspects of the building, there is also a need to further facilitate access through non-physical measures as well, such as operation and management of the facilities, and personnel measures.
• Users' needs may expand and diversify after the facilities have started to be used, so consideration should be given to facilities maintenance, management and operation so that renovations can be undertaken at a later stage if required.
• Safety during an emergency is a priority issue, so there is a need to construct a disaster prevention system that encompasses both physical and personnel support.

Key points in general building plans

(1) Process of planning buildings accessible to all users

Building plans and designs greatly varies depending on the owners, architects, and building use and size, but the following standards are considered to be critical when applying these design standards[4].

- Development guidelines must be based on the equitable use by all people.
- Development guidelines look into the barrier-free standards of the overall building. One point requiring careful consideration when applying design standards is determining the barrier-free standards to set for the overall facility. Development guidelines will probably vary depending on such factors as building use, size, and position.
- In that case, the objective should not merely be the development of the building section or unit space alone. If focus is concentrated on sectional development, there is a possibility that the overall accessibility of the building will be disjointed and incomplete, so it is vital to be mindful of accessibility and ease of use throughout the building as a whole.
- In existing buildings, a thorough examination should be carried out into such aspects as users' needs, structural and cost constraints when extending or renovating, and the possibility of securing alternative routes when it is difficult to upgrade the primary route. Some times building improvement can be a simple matter of effective staff placement and equipment and systems installation or upgrading. For existing buildings it is also crucial to prioritize development areas according to the usage of the facilities[3].

Understanding users' characteristics and needs

• As stated earlier, to understand the characteristics of users or to grasp their needs based on the use of the facility, listening to their views as required and seeking their input into the planning process is very important.

Examine the application of development standards indicated in design standards

• Compliance and conformity with barrier-free access standards.
• The measures and responses listed in the design standards are not all-inclusive; often architects will need to structure their designs on a regional or individual facility basis.

Appropriate management for barrier-free measures

• Careful consideration should be given to maintenance and management after the development is completed. This includes properly maintaining tactile tiles for visually impaired people, internal and external floor material, positioning of bollards, and information board [3-5].

(2) Key points in building plans

From a minimum level to a more comfortable level of accessibility for elderly and disabled people . as shown in figures no. (1) & (2).

1) Planning for a continuous movement flow

• Being able to move safely from the road and passageways within the building grounds to the destination room, etc. is fundamental. Priority areas for development will vary according to the function of this movement flow. For example, the flow in a restaurant would be from the dining room to the toilet, in sport facilities it is to the spectators' seats, and in theaters it is applied on the vertical movement to audience seats, dressing rooms, and the stage. In hotels and inns, there is a need for easy accesses to guestrooms or communal baths , see fig (3).

2) Detailed safety plan

• There is a need for appropriate measures to alleviate the risk presented by steps, and prevent people from stumbling or colliding into protruding objects while using the facilities.
• Other than when there are alternative movement means or there are no particular functional difficulties, special care should be taken to keep steps to a minimum [4].

3) Appropriate dimensions

• There is a need to examine and plan for suitable spatial measurements, such as the space required for various kinds of movement based on an understanding of users' needs, the space required for wheelchairs to turn around; the height of openings and switches, and the position of signs [5].

4) Considering economic efficiency, flexibility, and general efficiency

• Space planning and facilities that can be used by all users rather than designing specifically for elderly or disabled people will lead to lower construction costs, and the more efficient use of space [6].
• Consideration should be given to ease and efficiency of use, including the provision of a suitable number of parking spaces for wheelchair users, multifunctional toilet cubicles, and a large number of toilet cubicles that are slightly bigger than normal, and integrating signs in

adjacent or collocated buildings. Meeting facilities, theaters and the like may need to adopt a flexible seating arrangement, e.g. movable or detachable seats, in response to fluctuating numbers of users.

5) Ensuring ease of use and recognition

• While making the overall building user-friendly is fundamental, consideration should also be given to the fitting of switches and door knobs that can be readily used and recognized by children or elderly people, visually impaired people, or those with upper limb disabilities.
• Building signs should be designed to be simple, clear and easily understood by intellectually disabled people and foreigners.

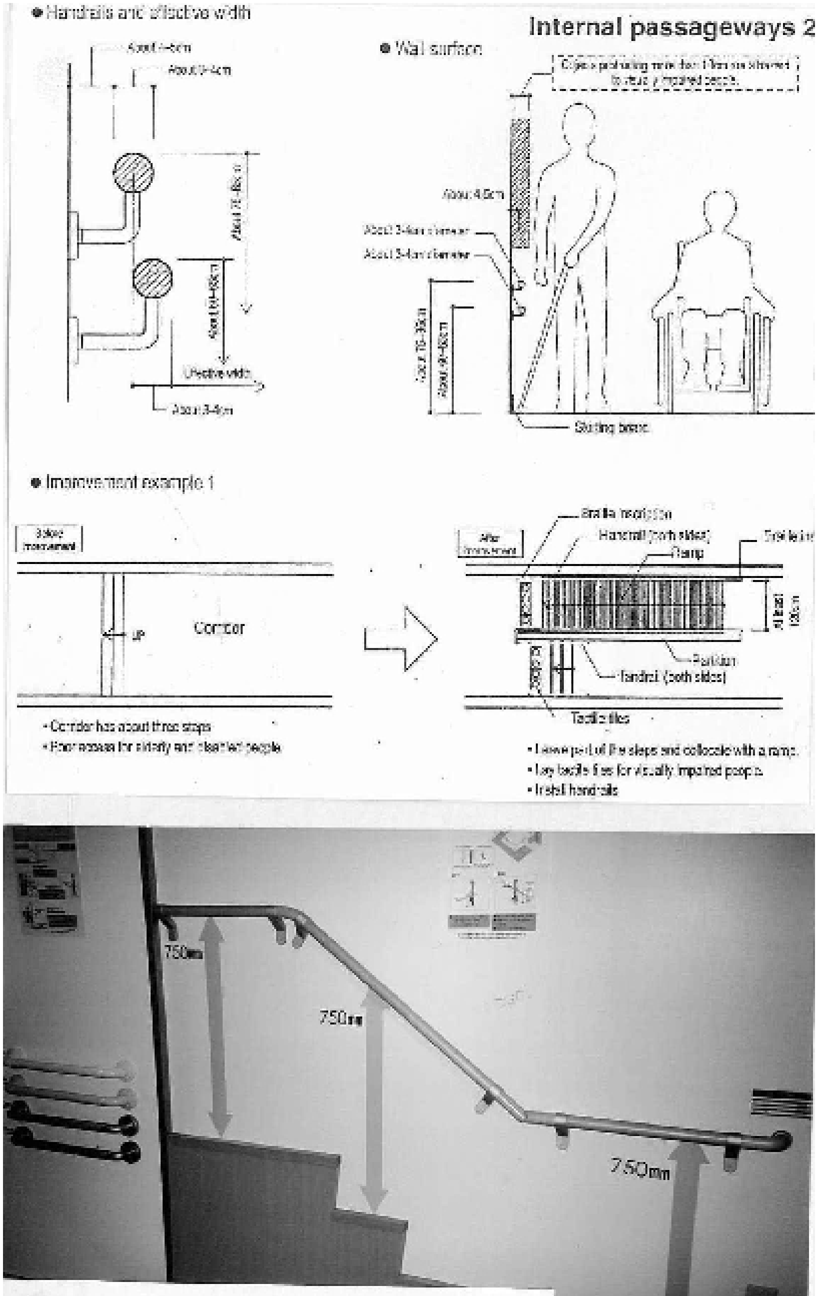


Fig. 1: Internal passageways measures for disabled people

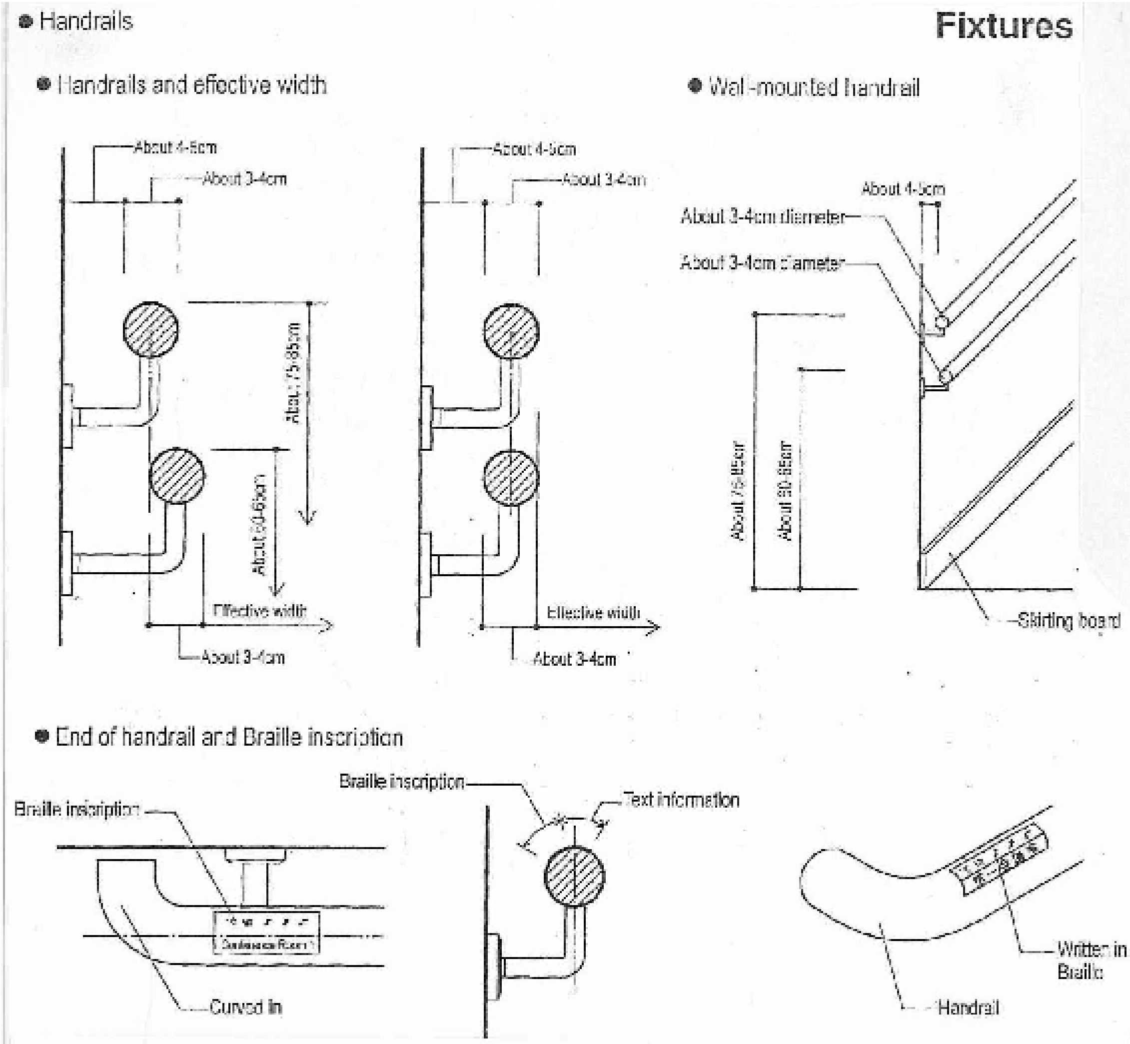
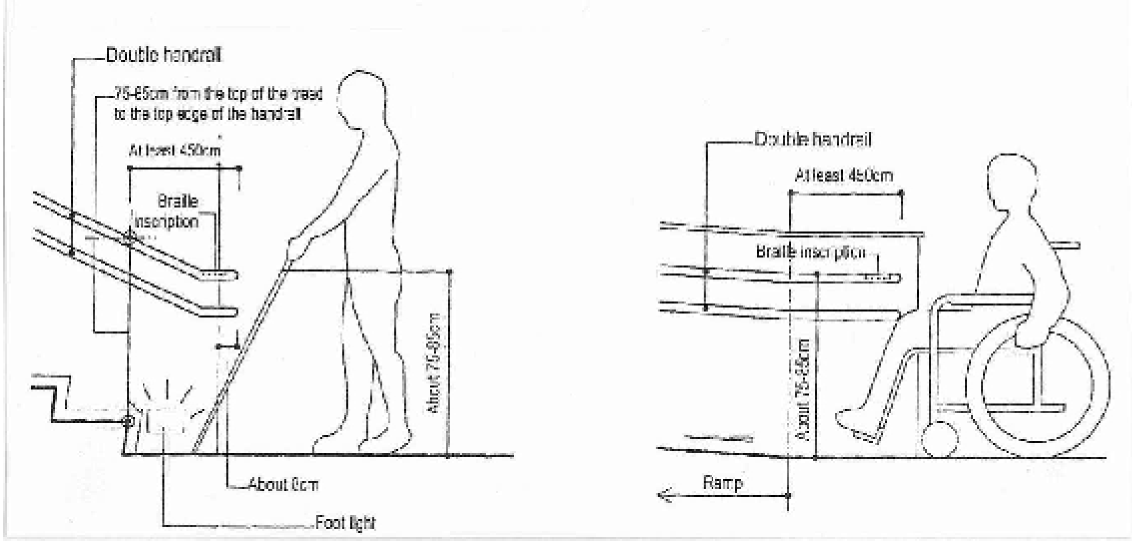


Fig. 2: Handrail detail for visually Impaired people

Handrail on a ramp



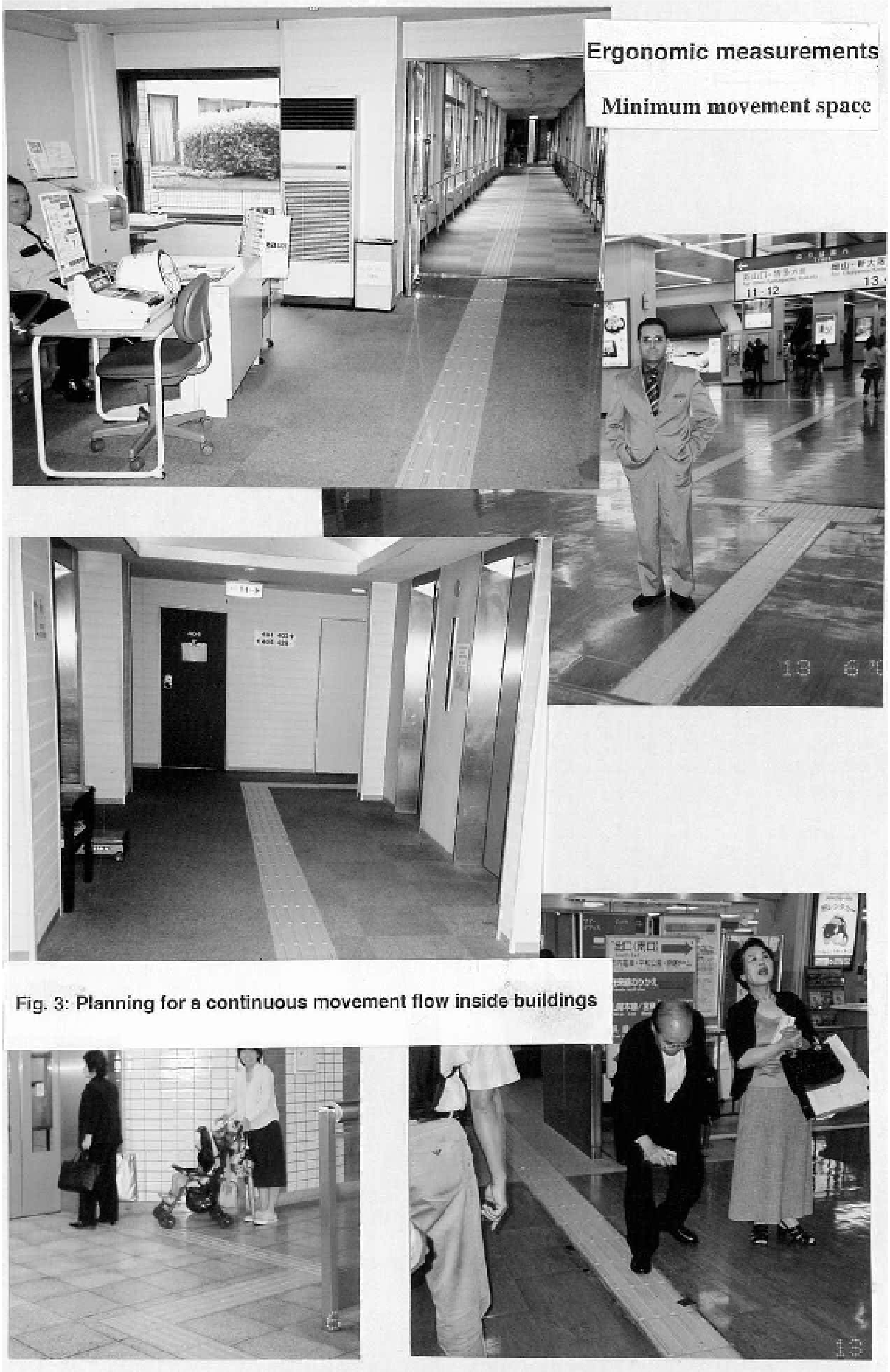


Fig. 3: Planning for a continuous movement flow inside buildings

6) Staff placement according to users' needs

• It is desirable to examine situations where from a use or location perspective, personal assistance is essential; for example, guidance and assistance for visually impaired people, sign language interpretation for hearing impaired people, special guidance for intellectually disabled people, and assistance and guidance for all facility users during an emergency [4].

2. Building entrances

• It is essential that building entrances should provide easy access for elderly and disabled people, and clear display information about the building facilities.
• Staff should be readily available to provide information and assistance to elderly and disabled people using the building if required. Entrances should be designed with the building usage, management, and users in mind.
• Entrances should not have any steps that may obstruct access by wheelchair users, but where this is unavoidable; ramps or lifts should also be installed. Thorough examination at the design stage is therefore absolutely critical, as shown in fig (4)

Design standards for building entrances

The design of building entrances should be as follows.
Entrance must be wide enough to allow easy access by wheelchair users, and should have enough space to the front and rear for wheelchairs to turn around. Doors should be easy for users and people with an upper limb disability to open and close as well as glass panels in doors which should be made of safety glass (laminated glass or tempered glass) to prevent breakage and injury in an impact, besides. Revolving doors which prevent access by wheelchair users should not be as the only means of entry. The clearance between hinged doors in the entrance enclosure must be large enough for wheelchair users to pause and wait safely, & tactile guiding tiles for visually impaired people should be laid continuously from the entrance to the reception or other information area. Tactile guiding tiles for visually impaired people are sometimes not necessary in entrance enclosures. If, however, a change of direction is required in the entrance enclosure, tiles should be laid. Lighting should be installed for safe access at night. & the floor should be a non-slip surface for a reception counter, intercom system, or other information facility should be positioned near the entrance. & finally, Information about the building facilities must be appropriately displayed for hearing impaired people, see figure no. (5) & (6).

Passageways in the building

• Passageways from the boundary of the building grounds and car park to the building entrance and between buildings within the same grounds must provide easy access for elderly and disabled people (where the land has a characteristic form, easy access is to be provided from the porch to the building entrance).
• Pedestrian passageways should be separated from roads, steps should be removed using lifts, passageways must be wide enough for wheelchair users and night lighting and guidance systems installed for visually impaired people.
• The basic concept regarding passageways in the building grounds is that people with various mobility restrictions should be able to use the passageways in the same way as people without those restrictions.

Design standards for passageways

The design of passageways in building grounds should be as follows;
In principle, the lines of movement of pedestrians and vehicles should be separated to ensure the safety of elderly and disabled people,[3-4] where there is a difference in level between the

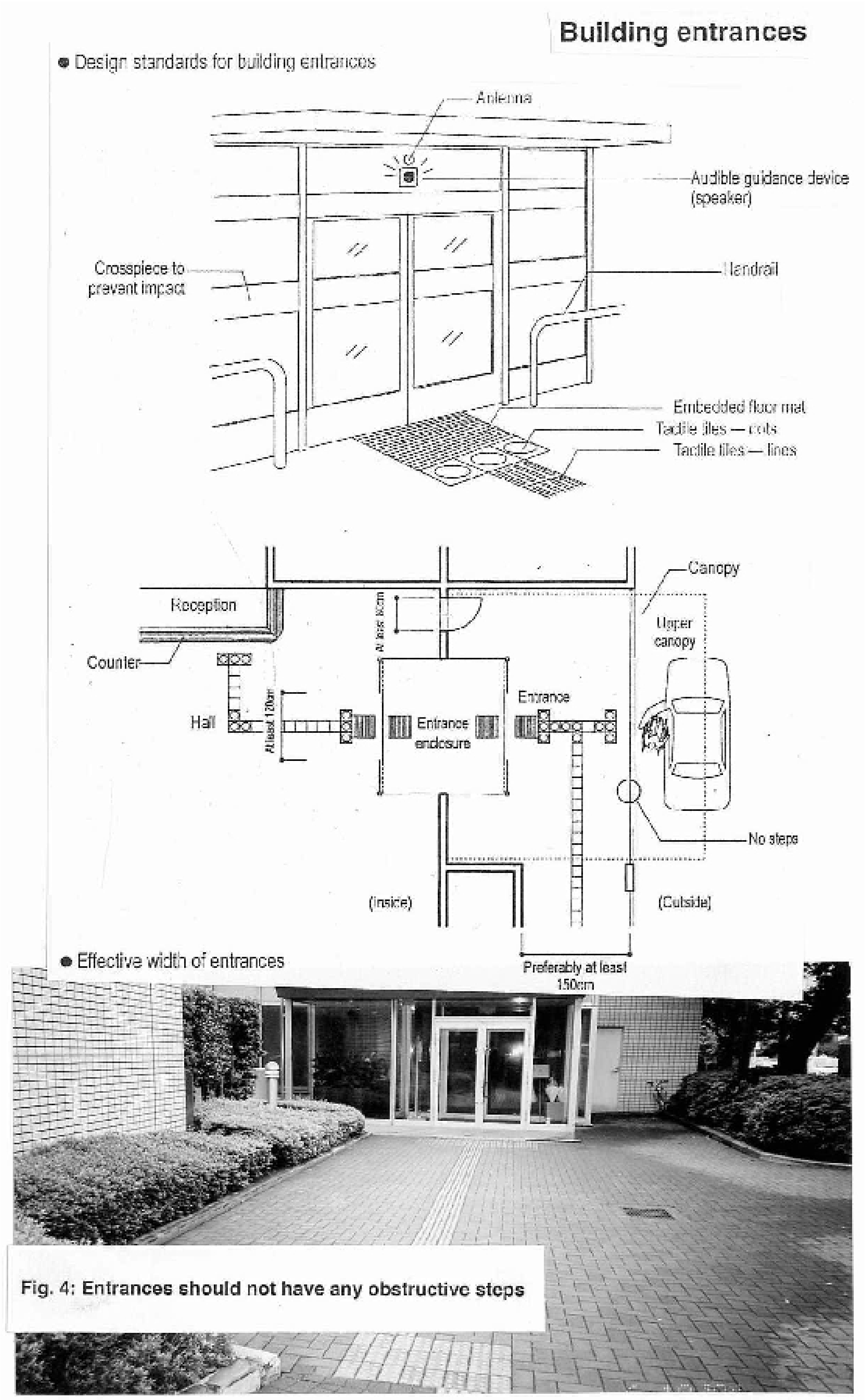


Fig. 4: Entrances should not have any obstructive steps

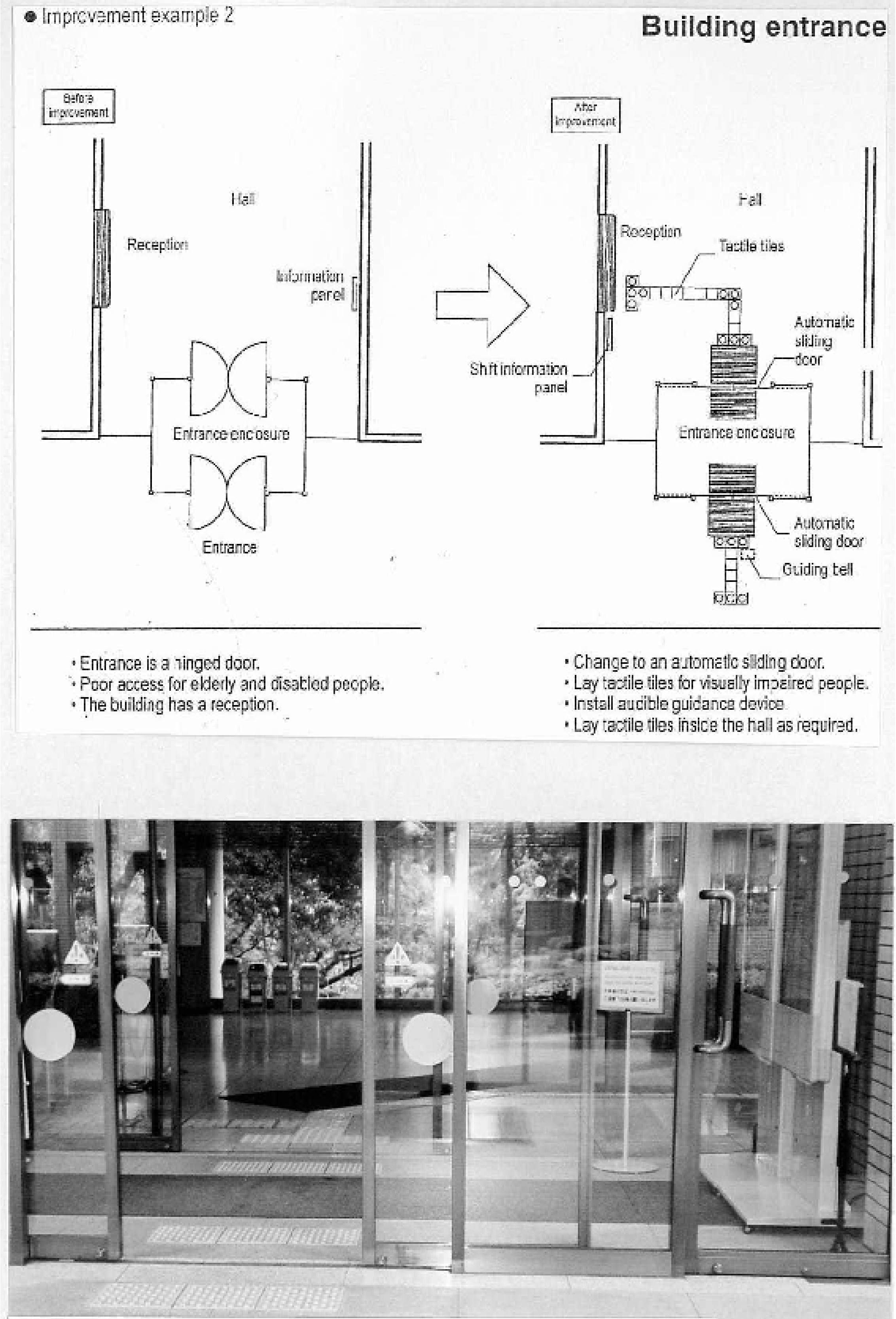


Fig. 5: An important example for an entrance, not using revolving doors

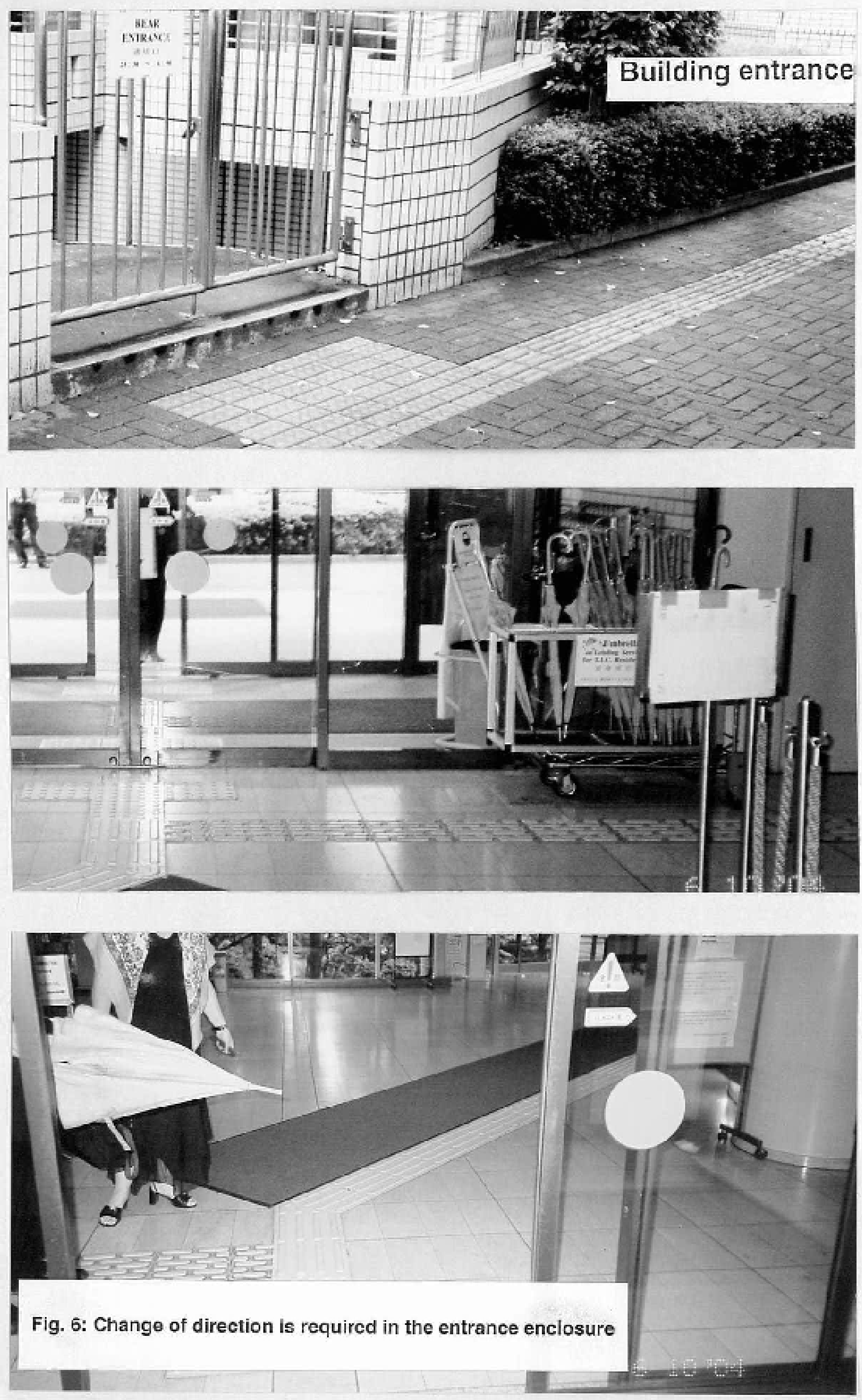


Fig. 6: Change of direction is required in the entrance enclosure

road and passageways along the boundary of the building grounds, consideration should be given to the passage of wheelchair users. There should be no steps along passageways. Where there are steps, there should also be ramps or wheelchair lifts & when building a separate main passageway for elderly and disabled people, care should be taken so that where possible the route is not markedly different from the passageway for other users, passageways must be wide enough for elderly and disabled people to pass safely as shown in fig (7) & (8). Any monuments, bollards, and planters should be positioned where they will not become an obstacle for wheelchair users or visually impaired people moving along the passageway, tactile tiles should be laid to guide visually impaired people from the boundary to the building entrance or information facility, & passageways should have a non-slip surface, and in areas of heavy snowfall, snow melting systems should be installed as required to prevent ice forming on the passageways.

Design standards for ramps on passageways

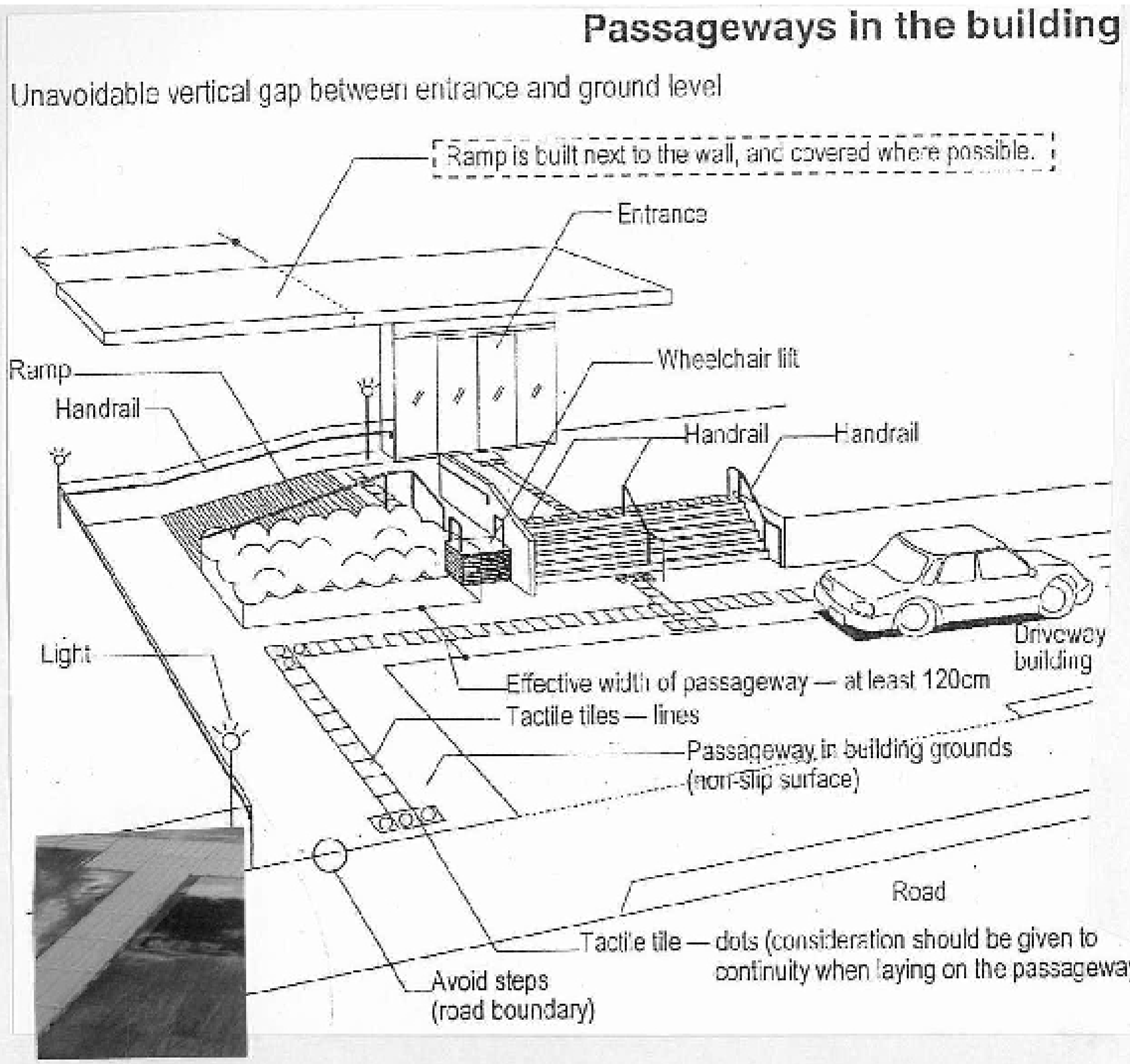
Care must be taken in the placement, grade, effective width (width excluding obstacles such as drains) and landings of ramps so that wheelchair users can travel up without difficulty, and travel down safely, as in fig (9).
Measures should be adopted that will enable visually impaired people to detect that they are approaching a ramp. If the ramp has a sharper incline, tactile tiles should be laid at the upper end to warn visually impaired people, (fig 10).
At times people with prostheses or who are partially paralyzed may find it easier to use stairs, so stairs with gently inclining handrails should be collocated with ramps, handrails and raised edges should be installed along open sides of ramps that are not flanked by a wall or barrier to prevent people from falling or dropping canes over the edge, as well as the surface of ramps should be non-slip, and easily detectable by visually impaired people.
• In cases where passageway cannot be established, ramps that conform to the following should be installed.

Internal passageways

Passageways inside buildings must follow a simple way to understand flow plan and be clearly signed to enable users to reach their destination with minimum difficulty. This is especially important for elderly and disabled people so they do not lose their way, and do not have to travel any further than is absolutely necessary, & passageways must be wide enough so elderly and disabled people can move along safely, and wheelchair users or people with canes can pass without obstruction, and steps and objects protruding from walls should be avoided. For visually impaired people in particular, protruding objects and posts that cannot be detected with a cane must be avoided. Also, internal passageways should also contain suitable guiding handrails etc. depending on building usage and users' needs, see fig (11).

Design standards for internal passageways

Should be easy to follow and provide easy access, floors should have a non-slip surface , & steps should be avoided, but where there are steps, ramps or lifts should also be installed. Fire extinguishers and telephone stands should be positioned where they will not obstruct movement, and posts and other protruding objects should be removed to facilitate access, & Passageways must be wide enough for access by wheelchair users and people with canes. Where a passageway is not wide enough for a wheelchair to turn around, areas with enough space to turn around must be secured along the passageway, besides handrails and tactile tiles should be installed as required. To ensure smooth access, there should be no steps in exits to balconies and other areas outside, and there should be enough space to the front and rear of these exits for wheelchairs to turn around. Where there is enough room for a rest area, seats can be installed for people to rest, [3-4]. Space should also be set aside for wheelchair users.



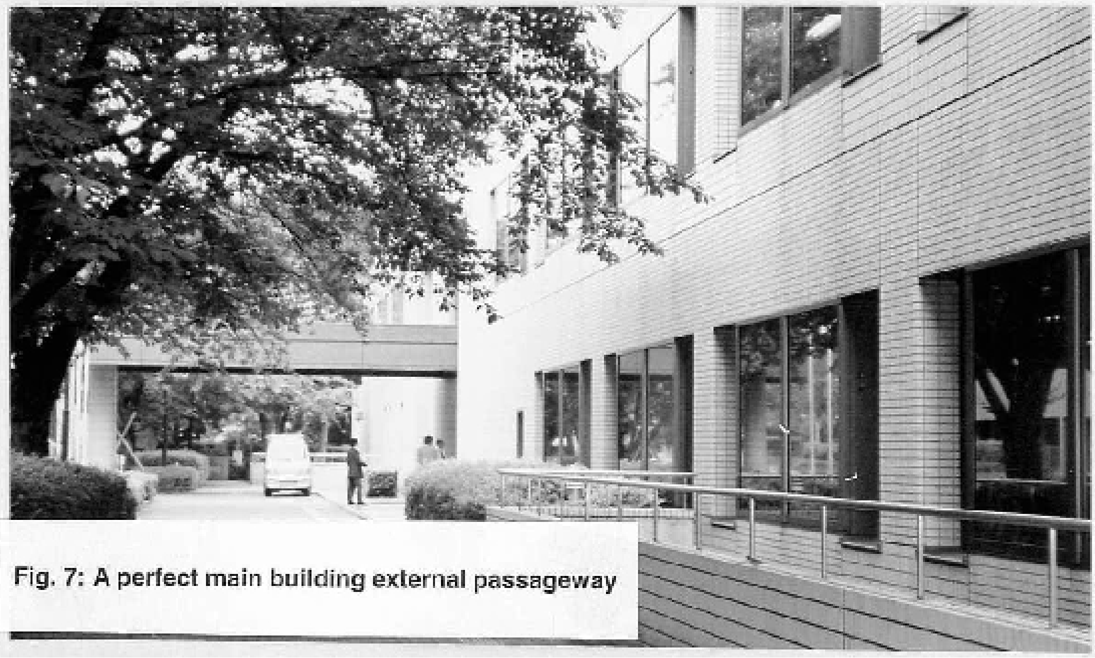


Fig. 7: A perfect main building external passageway

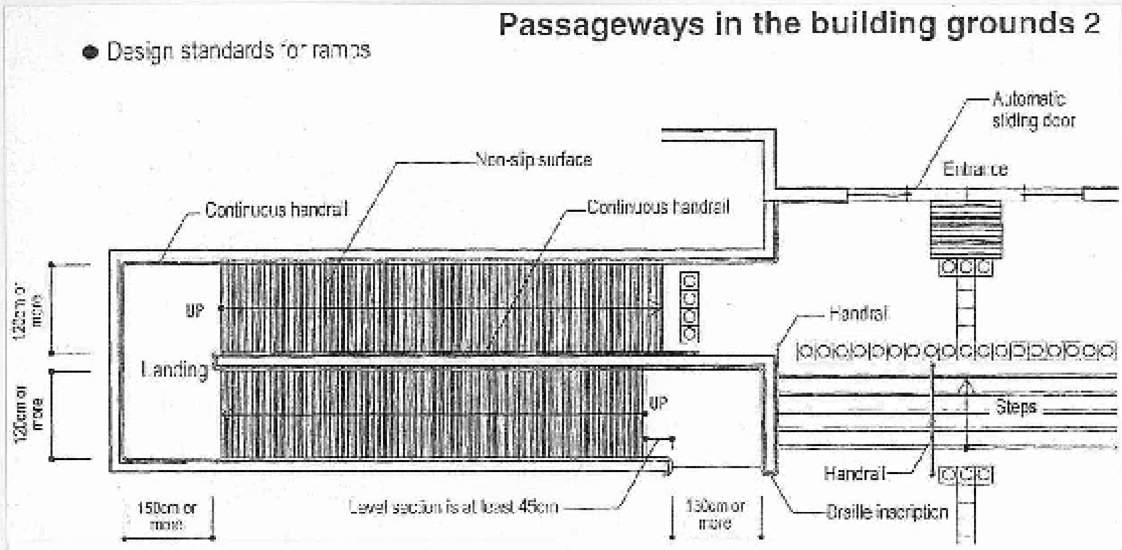
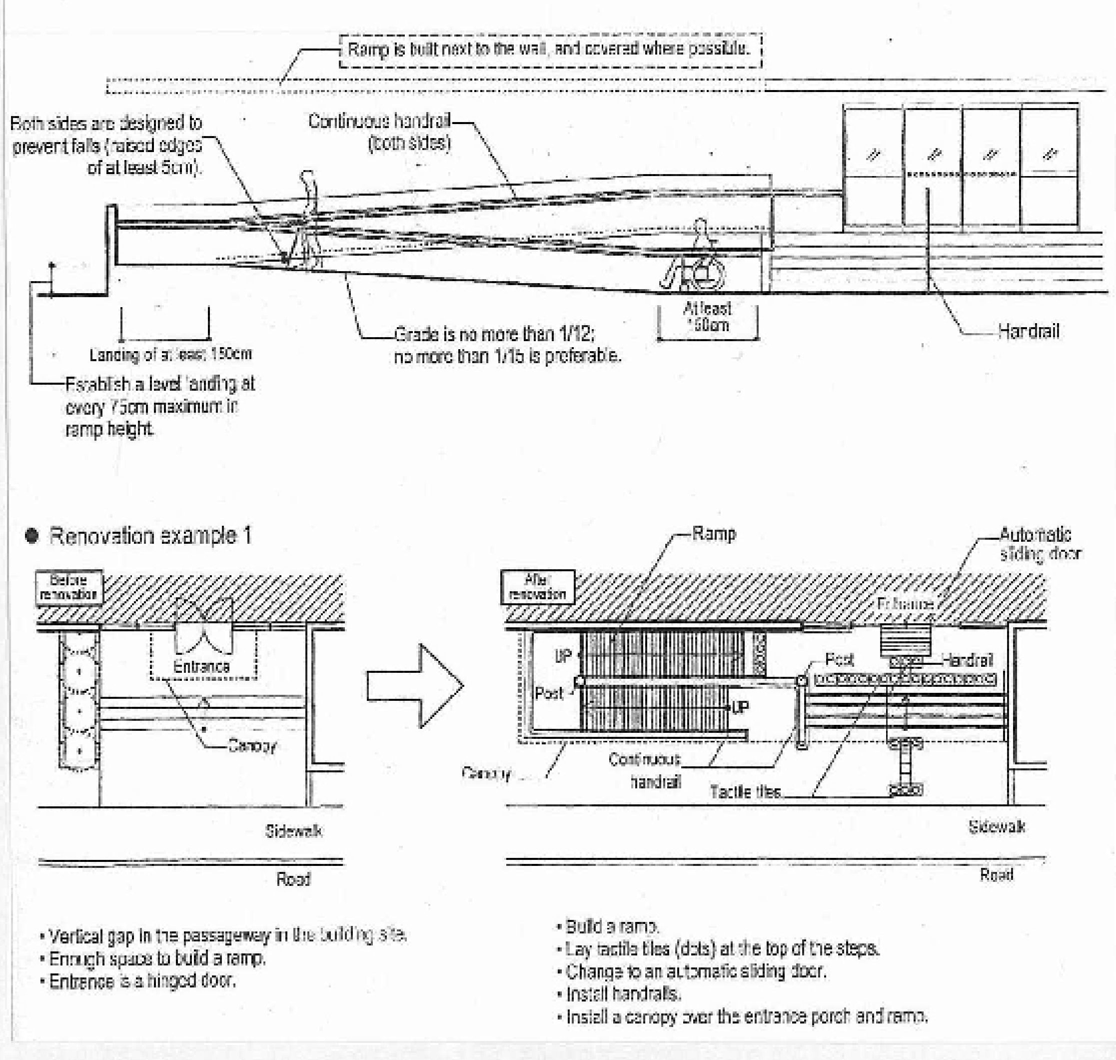


Fig. 8: Safe entrances & ramps for a building



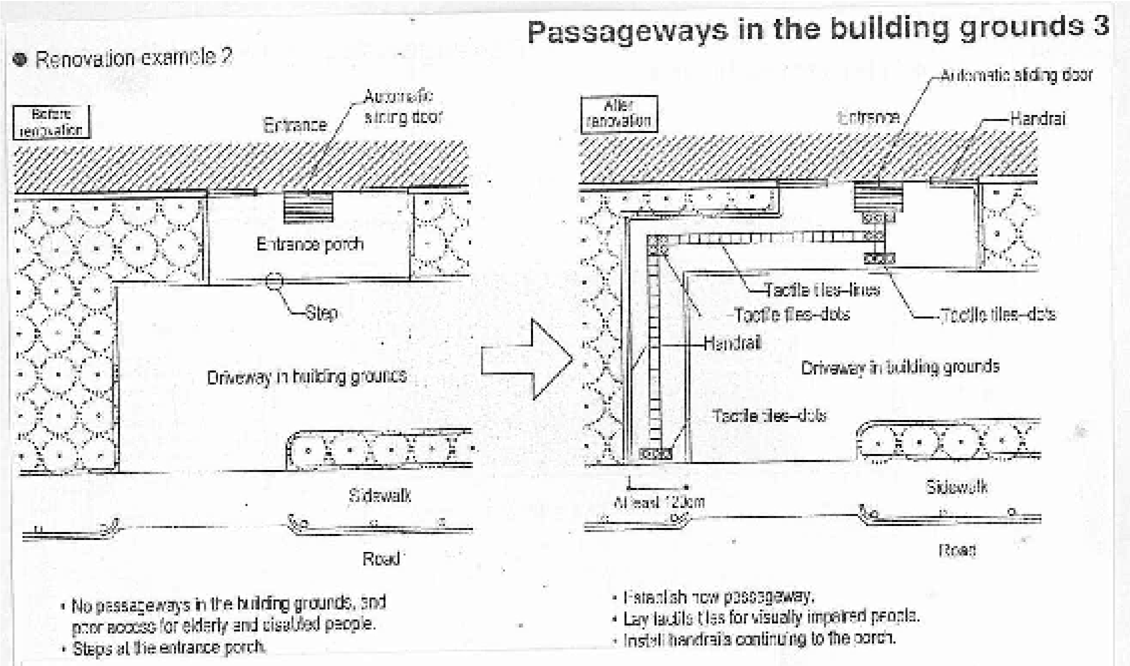
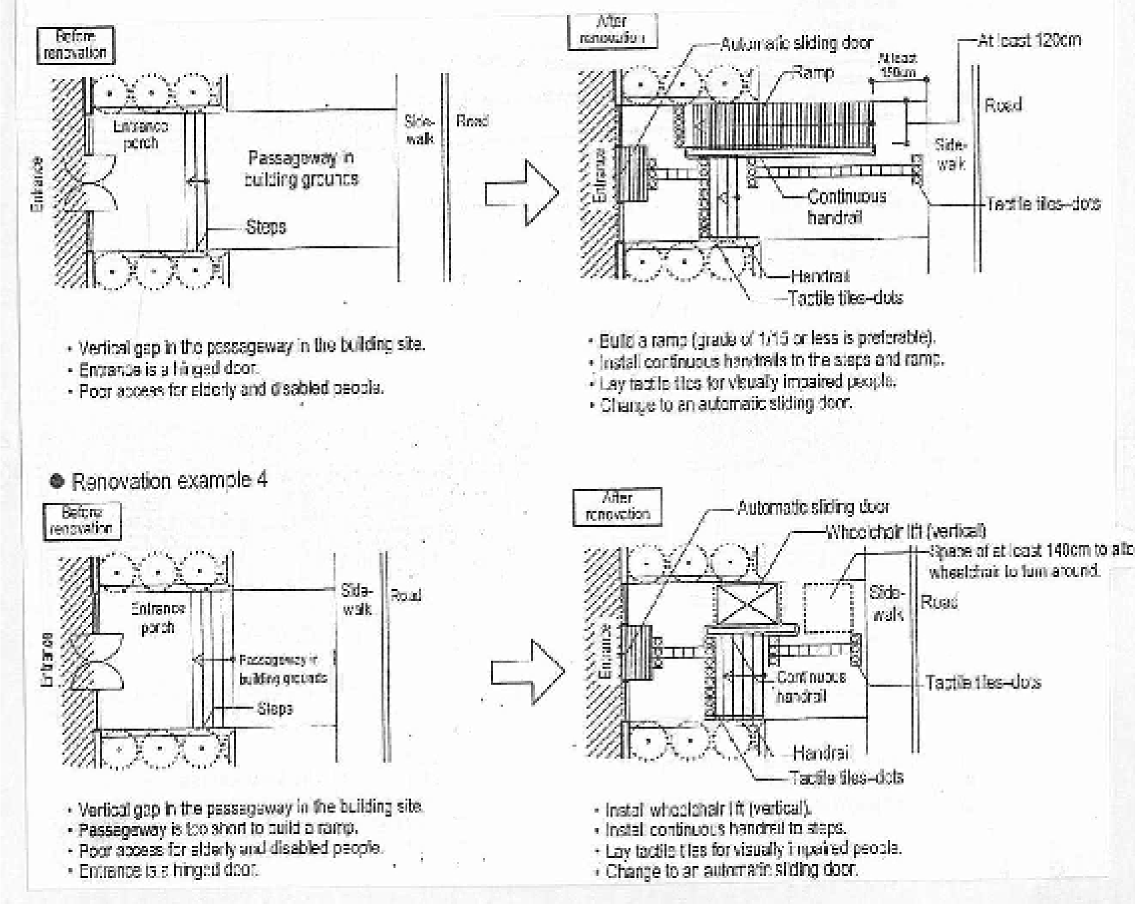


Fig. 9: An effective & improved accessible entrance passageway



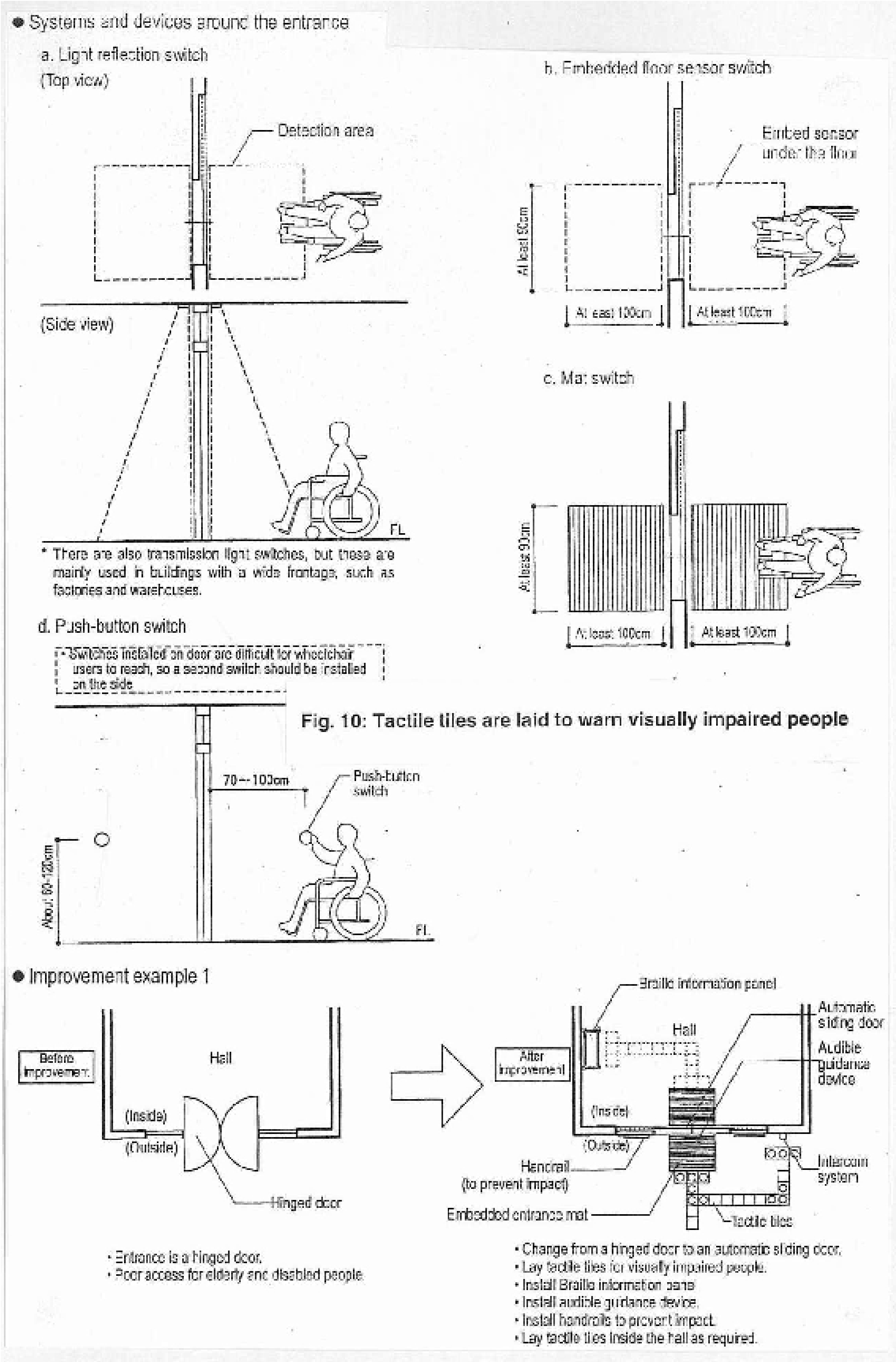


Fig. 10: Tactile tiles are laid to warn visually impaired people

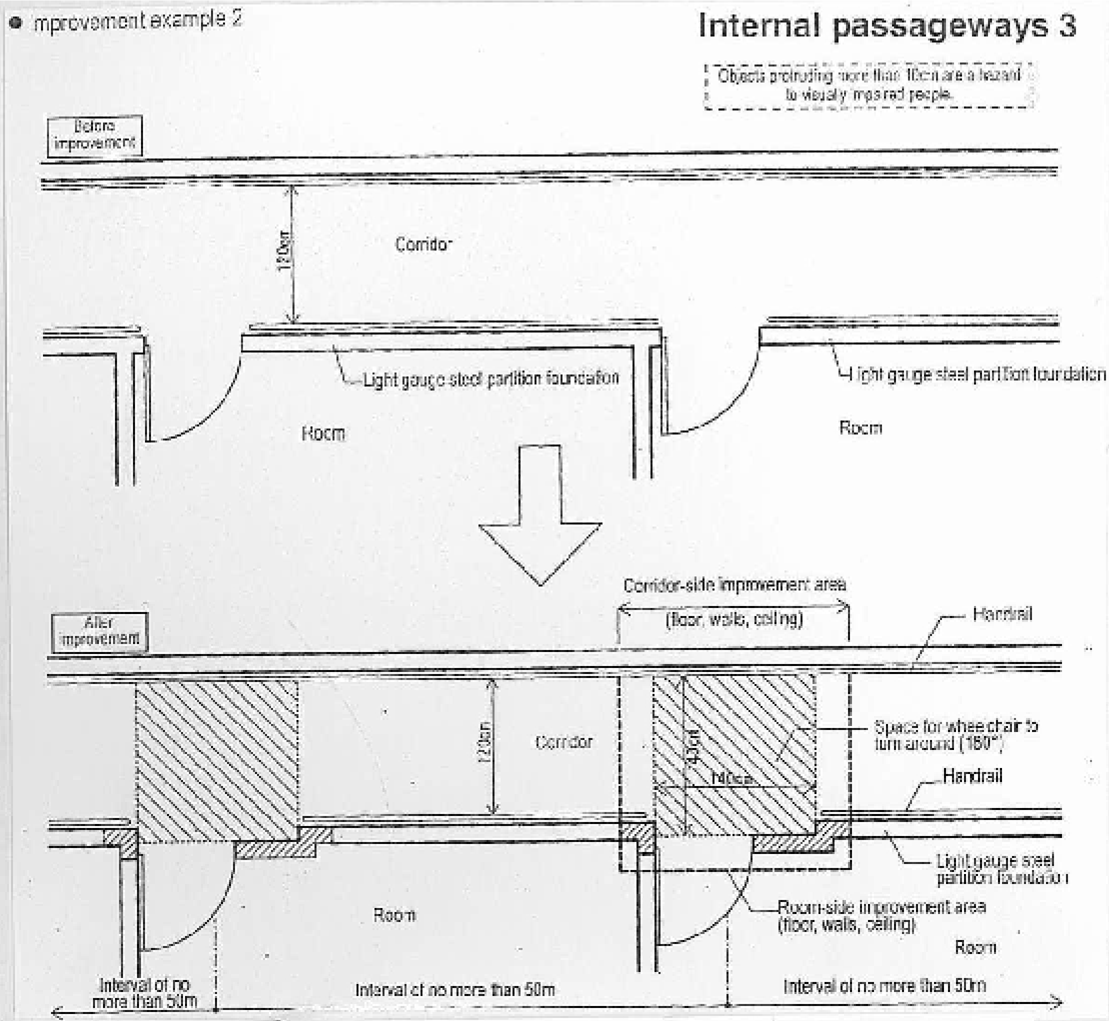


Fig. 11: Internal passageways containing suitable guiding handrails and tactile tiles

Stairs

• Stairs are areas of frequent falls, and other accidents, safety features such as nosing and handrails should be installed. Measures should be adopted so that visually impaired people are aware of the existence of the steps.
• The shape and the grade of the steps should be easy for elderly and disabled people to climb, and the steps should be wide enough for easy access by people on crutches, and elderly and disabled people together with their carers as well as internal stairs, such access considerations for elder1y and physically disabled people should also be given to external stairs used on a regular basis.

Design standards for stairs

Circular stairs tend to be a hazard to visually impaired people because they can lose their bearing, they can easily stumble due to the different tread measurements between the inner and outer sides, and they have to combine vertical movement with a turning motion. For this reason, either straight or return stairs should be used, & straight and return stairs should have landings to minimize the danger in case of a fall. Handrails should be installed for use especially by people with walking difficulties, elderly people, and visually impaired people. Treads and landings should have a non-slip surface, & nosings should be easily distinguishable by elderly and visually impaired people, and should not protrude to prevent stumbling, as fig (12). Tactile tiles should be laid at the top of the stairs to ensure they are easily detectable (tiles are not necessary in landings where handrails continue on from the steps).

Elevators and escalators

• Elevators are effective means for the safe and easy vertical movement of elderly and disabled people. Elevators should be located in areas where they can be accessed by all people and are easy to find. Signs should be suitable positioned, & for the convenience of users, escalators alone are not sufficient. In principle, elevators should be provided for disabled people , as shown in fig (13).
• Escalators should be structured for ease of access by elderly and disabled people, and as shown in fig (14)

Design standards for elevators

Buildings with more than two stories and that are used by many people should have elevators to facilitate access by elderly and disabled people, & they should be located next to the main route of access [4].
The elevator lobby should have enough space for a wheelchair to turn around in front of the elevator, and preferably wheelchair users should be able to move straight into and out of the elevator.
Elevator frontage, shape and size of cages, and positioning of control panels should take into account their use by elderly and disabled people, where buttons in the elevator lobby and control panels in the elevator cages should be positioned and structured for ease of use by visually impaired people, as well as the audible information systems such as emergency call system and overload warning buzzer, there should also be corresponding visual systems for hearing impaired people.
Signs and information displays should provide clear directions to the elevator entrance.

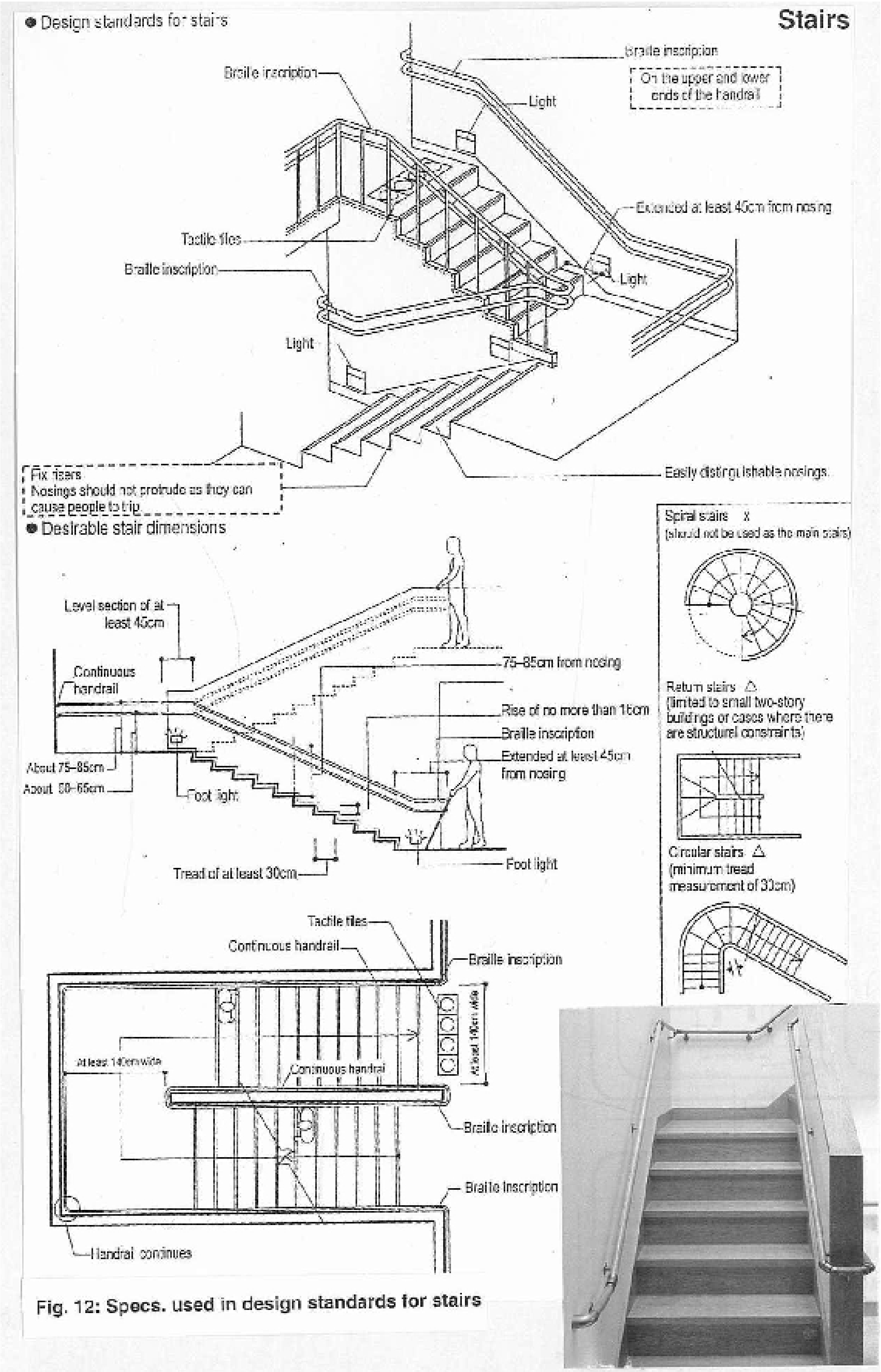


Fig. 12: Specs. used in design standards for stairs

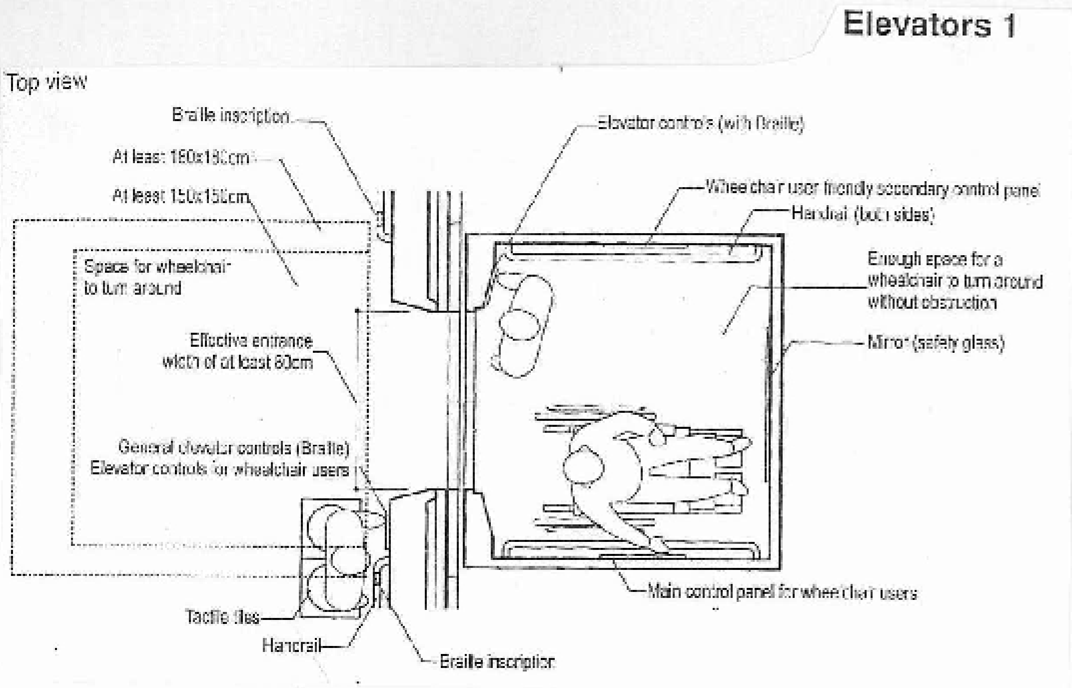
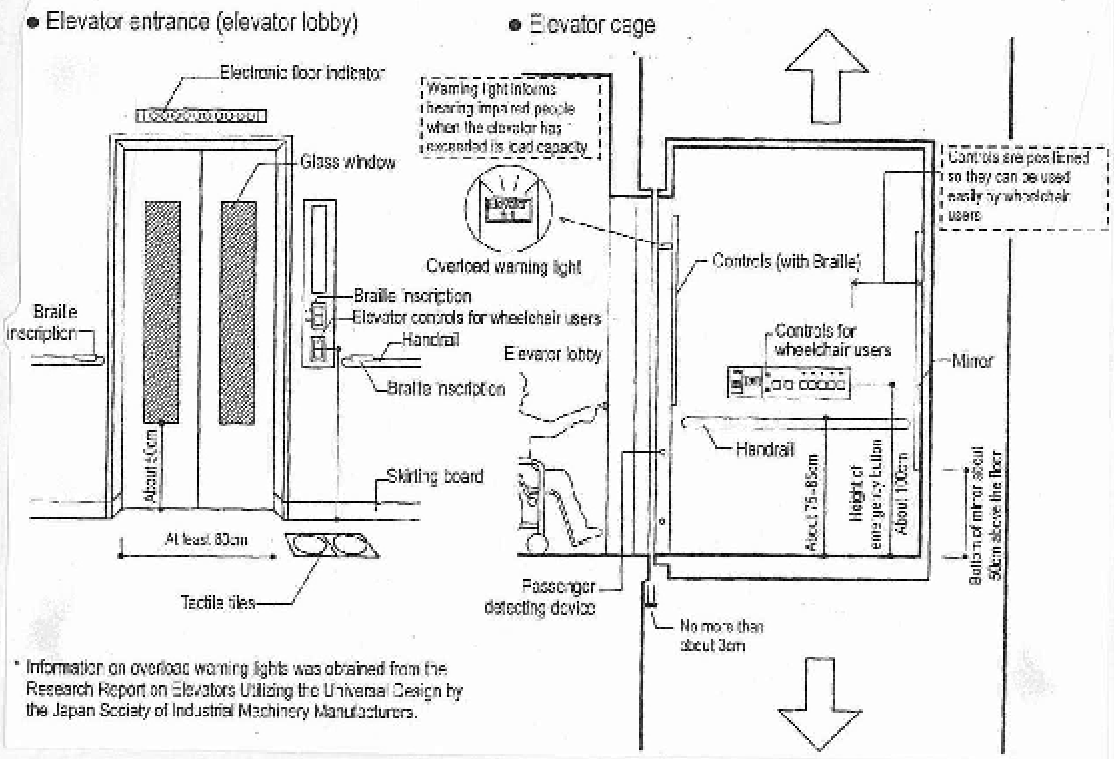


Fig. 13: Specs. used in design standards for elevators of visually impaired people



* Information on overload warning lights was obtained from the Research Report on Elevators Utilizing the Universal Design by the Japan Society of Industrial Machinery Manufacturers.

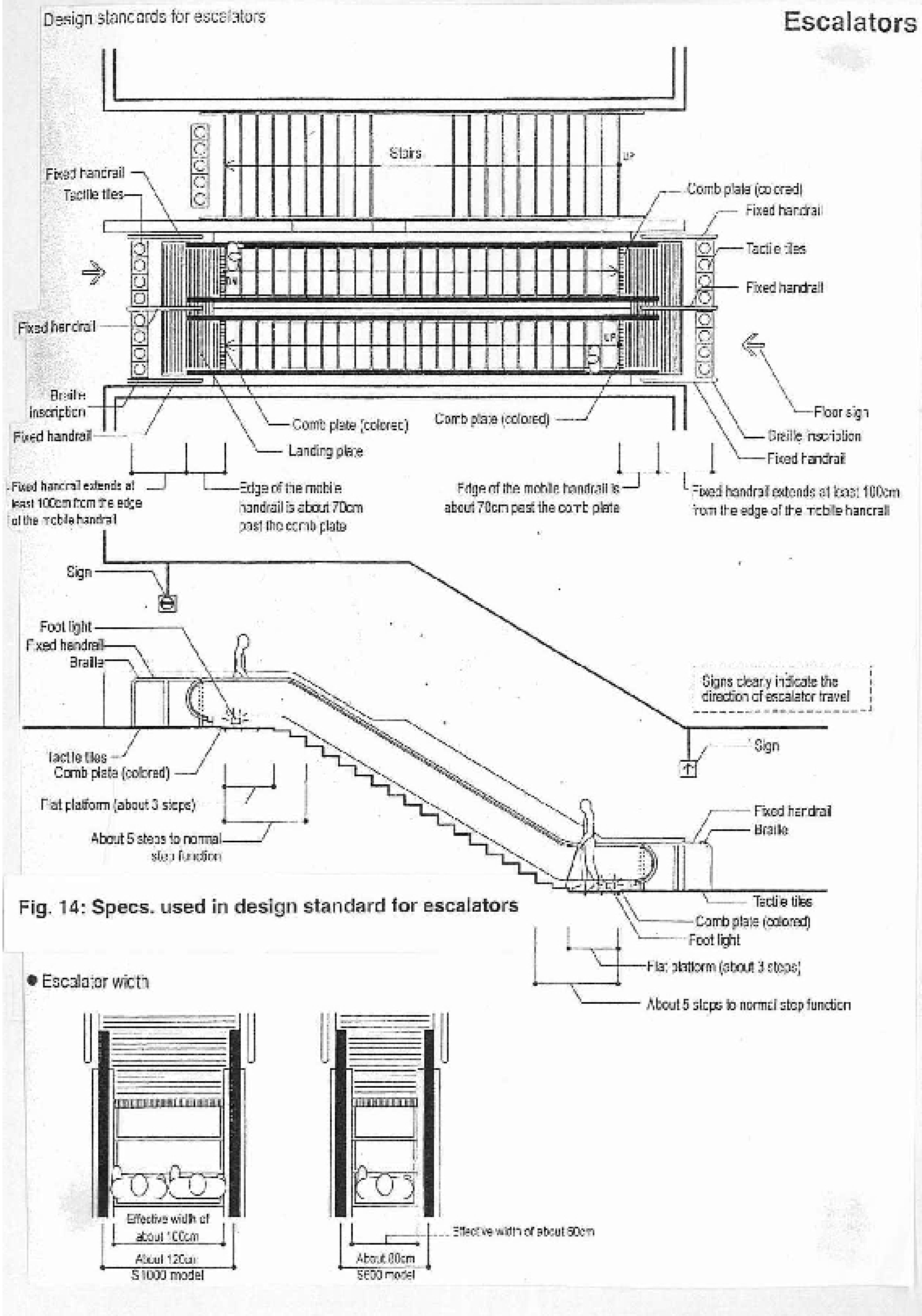


Fig. 14: Specs. used in design standard for escalators

Toilets and washrooms

• Multifunctional toilets for people with various functional restrictions are essential for promoting the social participation of elderly and disabled people [4].
• In addition to multifunctional toilets, .there is a need to install facilities that are accessible to elderly and disabled people in other toilets and washrooms.
• Multifunctional toilets should be located in areas that can be readily seen by elderly and disabled people, be large enough for a wheelchair to turn around, and have grab bars, flush basin for ostomates, faucets, and nappy changing sheets to meet the diverse needs of users.
• Disabled people's needs vary greatly when using the toilet, for example, depending on their disability; they have various ways of moving on to the toilet seat. Where a single kind of multifunctional toilet is unable to meet all users' needs, one solution is to have several kinds of multifunctional toilets located in different places within the building according to the characteristics of the expected users.
• The location of toilets must be marked so that elderly and disabled people can readily identify them, and information about the toilet layout and functions should be clearly displayed.
• Routes to toilets should consider ease of access for elderly and disabled people. Depending on how the building is used, a large number of wheelchair users may need to use the toilets at the same time, so in this case; several multifunctional or wheelchair accessible toilets are required.

Design standards for multifunctional toilets

The design of multifunctional toilets should be.
With or near general toilets so they can be easily located by wheelchair users, & they should provide easy access to both disabled and able-bodied people.
Toilet cubicles should have enough space for a wheelchair to turn around or for a helper or carer to move and assist as required so that wheelchair users, elderly and disabled people have as free access as possible, & there should be no steps at the entrance to toilets or toilet cubicles and along access routes and they should be wide enigh for wheelchairs to pass.
Entrance doors should be easy for wheelchair users to open and close as they pass through, & cubicles should have grab bars to help wheelchair user's move from their wheelchair to the toilet seat, while the paper holder and emergency button should be placed where they can be reached from the toilet seat or the wheelchair while remaining seated.
Basins in multifunctional toilets should have easy-to-use faucets, and should have enough clearance underneath for easy wheelchair accessibility.
When installing several multifunctional toilets, the position of toilet bowls should allow an approach from the front, but consideration should also be given to moving on to the toilet seat from the side, as shown in fig (15-17).
Door handles should be easy to operate, & floors should have a non-slip surface, besides that the equipment should be easy to operate and understand.
The entrance to multifunctional toilets should have signs indicating they are suitable for use by elderly and disabled people & one toilet unit per one floor should be a barrier free.

Design standards for non-multifunctional toilets

The design of male and female non-multifunctional toilets should have at least one seat-style toilet that can be used by wheelchair users. Grab bars should be fitted to provide support when getting on and off the toilet seat, and cubicle doors should be either out- swinging hinge doors or sliding doors.
Routes to toilets should consider the convenience and needs of elderly and disabled people, and passageways should be wide enough for easy access and should not have any steps.
The location and gender of toilets should be clearly marked on direction boards in Braille for visually impaired people, & at least one washbasin in toilets should be accessible by wheelchair users, e.g., low enough to use while seated, easy-to-use faucets, and enough clearance underneath for easy wheelchair accessibility, as in fig (18).

It is preferable that cubicle, doors be fitted with a display device that can indicate to visually impaired people whether the toilet is occupied or vacant, & it is preferable that cubicle doors be fitted with a display device that can easily indicate to hearing impaired people whether the toilet is occupied or vacant , fig (19).

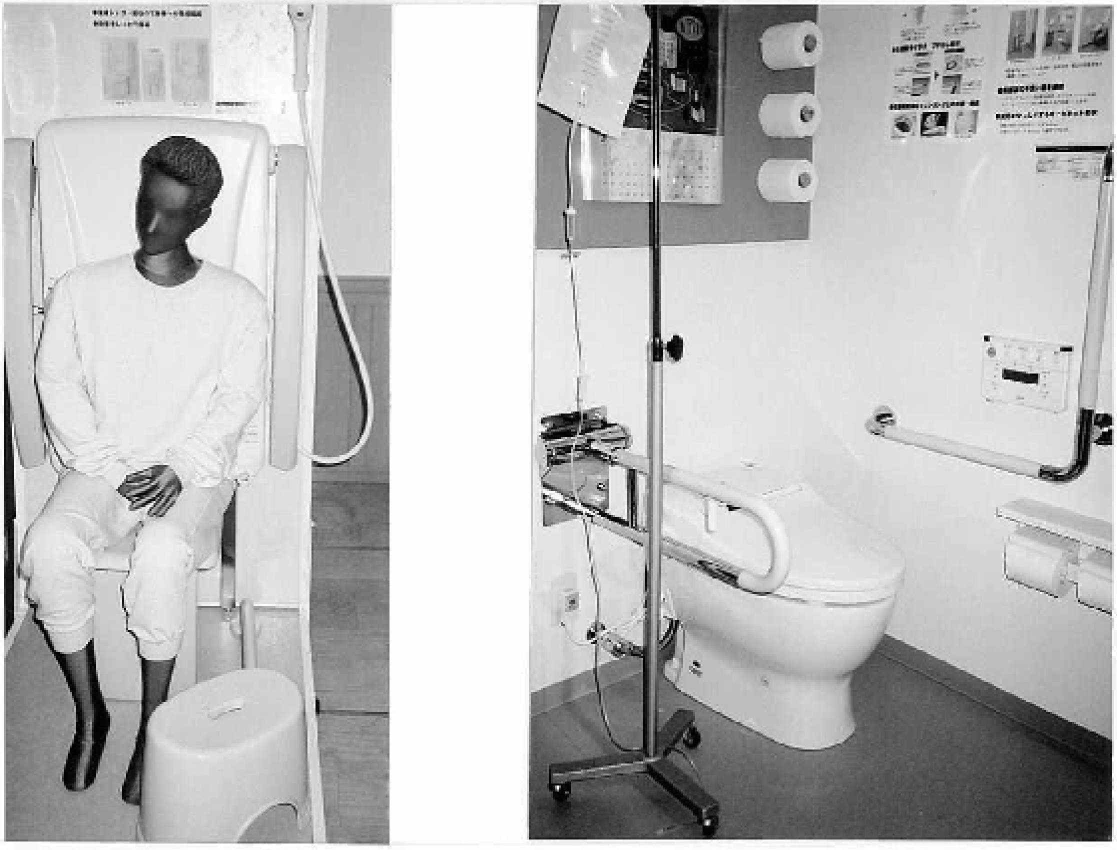


Fig. 15: Design standards for toilets used by visually impaired people

Fig. 16: Design standards for special toilet cabinets used by visually impaired people

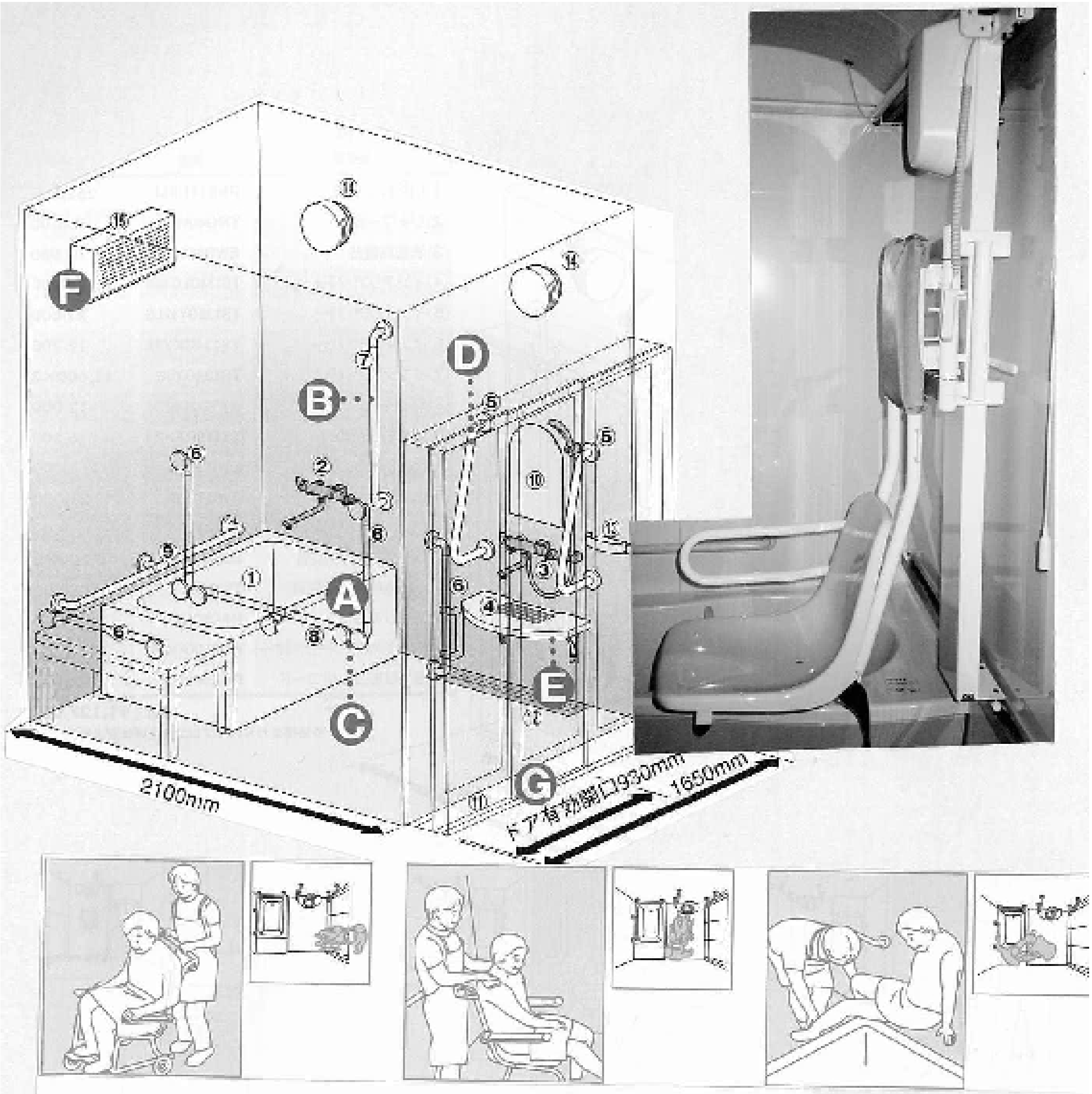
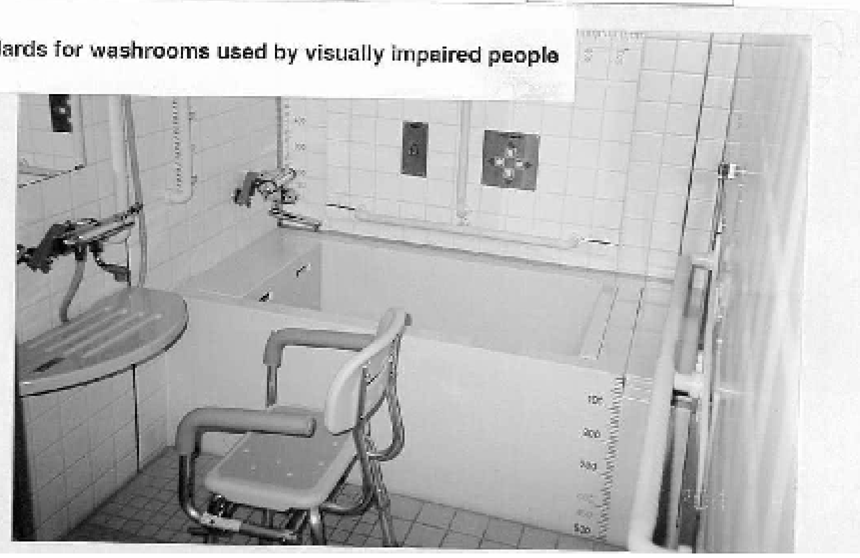


Fig. 17: Design standards for washrooms used by visually impaired people



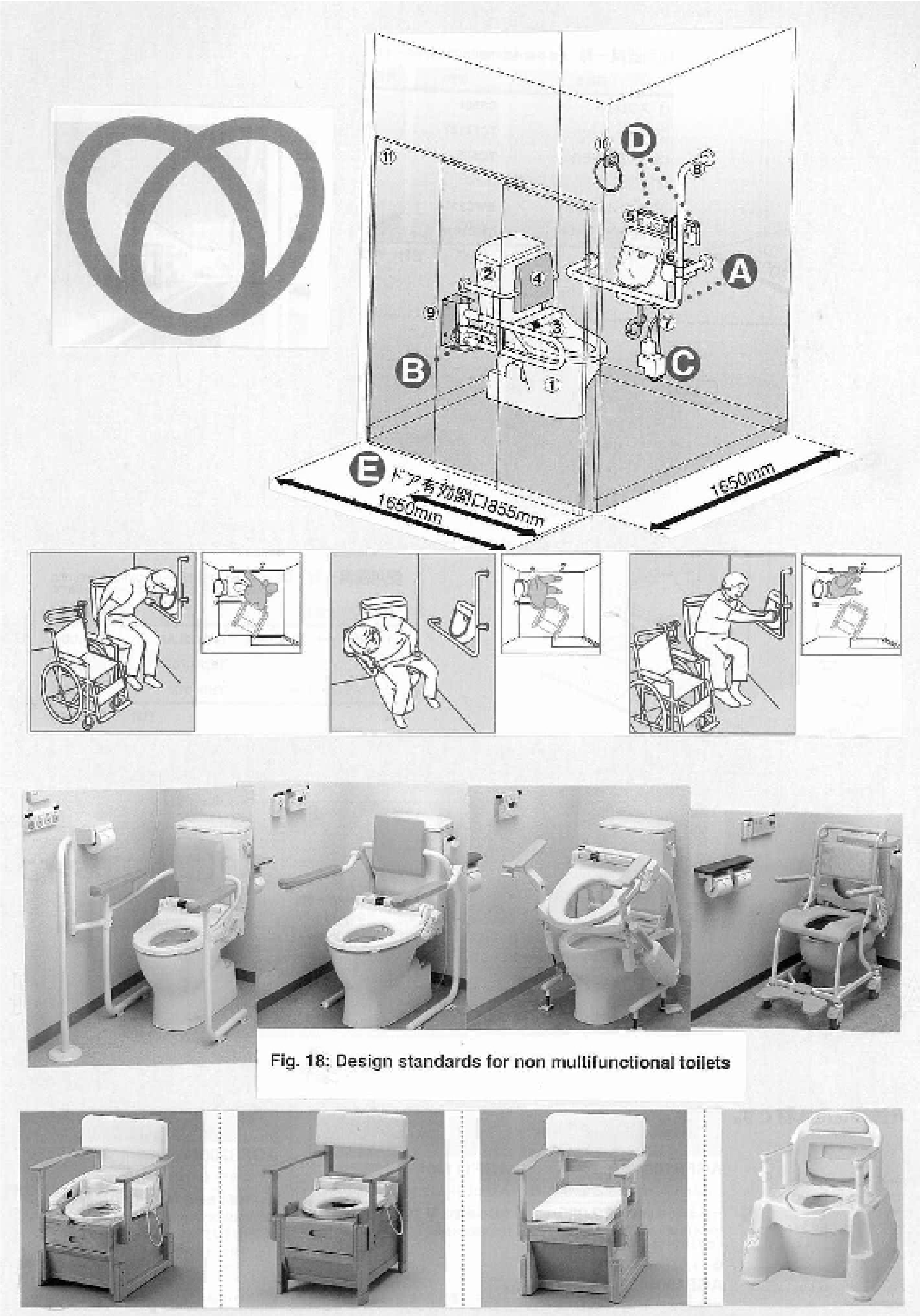


Fig. 18: Design standards for non multifunctional toilets

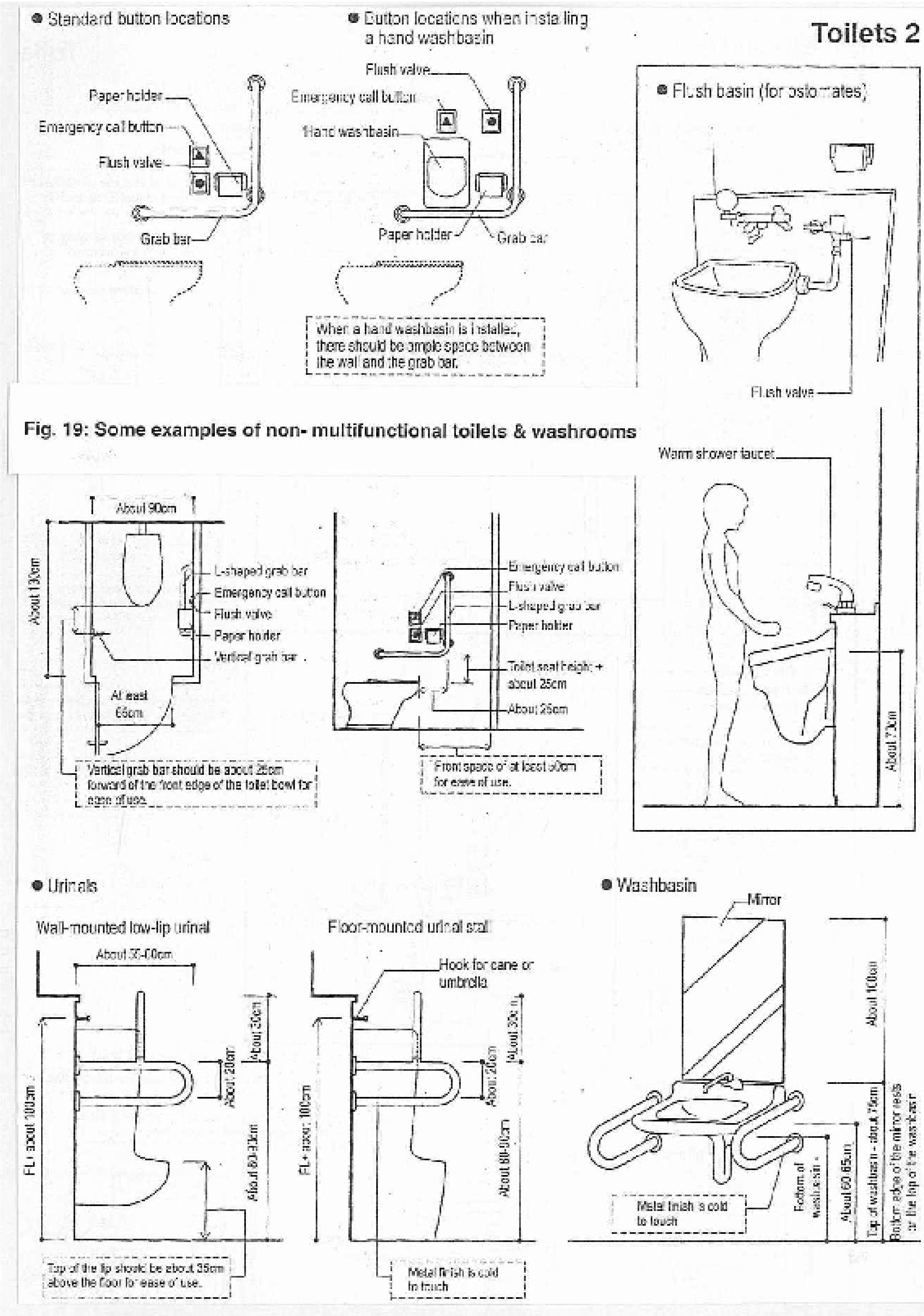


Fig. 19: Some examples of non- multifunctional toilets & washrooms

3. Tactile tiles

Tactile tiles are used extensively in such areas as roads, public facilities and train stations to assist the movement of visually impaired people both indoors and outdoors, and come various shapes (shape and pattern of the raised sections to convey information through the soles of the feet), colors and materials.
Tactile tiles shall be laid in areas on roads, elevated crossings, bus stops, tram stops, and passageways in car parks where there is a recognized need to facilitate the movement of visually impaired people.
The color of tactile tiles shall be yellow or another color that contrasts with the surrounding road surface so the tiles can be readily detected, as shown in fig (20).
Audible devices for visually impaired people shall be installed with tactile tiles in areas where they are considered necessary to facilitate the movement of visually impaired people.
Tactile tiles are tiles laid on the road surface to guide visually impaired people or alert them or make them aware of the existence of steps or other obstacles.
Tactile tiles were developed to improve the convenience of visually impaired people through the use of their sense of touch primarily in the soles of their feet.
Visually impaired people walk along the road armed with general information such as facilities and road structure, and individual information such as the experience of walking along the same route, and road directions before and while walking. Tactile tiles provide more accurate and immediate information on location and direction to them to supplement this broad range of information.
In principal, the types of tactile tiles are as follows;

- Line tiles

Line tactile tiles have raised parallel lines, and are laid on the road surface to indicate direction to visually impaired people.

- Dome tiles

Dome tactile tiles have raised domes, and are laid to warn or make visually impaired people aware about the existence of steps.

- Material

Material used in tactile tiles must be strong, slip-resistant, durable and hardwearing.
Commercially available tiles are made of various materials, but in choosing the type of material, full consideration must be given to safety and ease of walking, durability, weather-resistance, ease of installation, economic efficiency, and maintenance. A slippery surface is closely linked to ease of walking; if the surface is slippery, steps tend to be much shorter requiring greater muscle use, and this in turn causes fatigue.
Public comment called for when formulating the standards listed the main problems with tactile tiles as slippery surface, easy to break or chip, and easy to trip over. In this light, tactile tile material must consider ease of walking and be slip- and trip-resistant.

- Color

The basic color of tactile tiles is yellow, however, if yellow tiles do not provide adequate contrast with the, surrounding pavement, a sufficiently contrasting color other than yellow must be used. Some colors may be difficult to detect because of the weather, brightness or color combination, so decisions on color must take into account and reflect the views of people living alongside the road and general users.

. Tactile tiles

Fig. 20:Continuous tactile tiles used in various spaces

Differences in floor material, texture & brightness Improves VIP accessibility

Yellow is used for most tactile tiles because it provides a good contrast with the surrounding asphalt paving, and can therefore provide suitable guidance to visually impaired people (people with poor eyesight).
Yellow is also used because it is widely associated with tactile tiles and people generally believe that yellow is the most appropriate color, but when the road surface is a similar color, a luminance ratio is necessary to ensure the tactile tiles are easily detected.

Recommendations

- General standards for designated facilities (overall standards for facilities used by the general public or mainly elderly or physically disabled people)

Designated facilities	
Corridors	1) Does the floor have a non-slip surface? 2) Tactile tiles etc. (section adjacent to the upper end of stairs
Stairs	1) Have handrails been installed? 2) Does the floor have a non-slip surface? 3) Are the steps easy to distinguish? 4) Are the steps designed to prevent stumbling? 5) Tactile tiles, etc. (landing section adjacent upper end of the stairs) 6) In principle, the main stairs are not circular stairs.
Ramps	1) Have handrails been installed? 2) Does the floor have a non-slip surface? 3) Is the ramp easy to distinguish from the corridor. 4) Tactile tiles, etc. (landing section adjacent to incline section)
Toilets	1) Are there toilet cubicles for wheelchair users (at least one)? (1) Are seat-style toilets and handrails appropriately positioned? (2) Is there sufficient space for easy access by wheelchair users? (3) Are there appropriate signs? 2) Are there floor urinals (at least one)?
Passageways	1) Does the floor have a non-slip surface? 2) Areas with steps (1) Have handrails been installed? (2) Are the steps easy to distinguish? (3) Are the steps designed to prevent stumbling? 3) Ramps (1) Have handrails been installed? (Exempt for sections with a grade of no more than 1/12 and a height less than 16cm, or an incline section of no more than 1/20) (2) Is the ramp easy to distinguish from the passageway to the front and rear?

- Access routes (standards relating to one or more access routes to public rooms, toilet cubicles and parking spaces for wheelchair users)

Entrances	1) Are entrances at least 80cm wide? 2) Are doors easy for wheelchair users to pass through, & are there level sections in front of and behind the doors?
Corridors, etc.	1) Are corridors at least 120cm wide? 2) Are there sections with enough space for a wheelchair to turn around at no more than 50m intervals? 3) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?
Ramps	1) Are ramps at least 120cm wide (at least 90cm when collocated with steps)? 2) Is the grade no more than 1/12 (no more than 1/8 when the height is 16cm or less)? 3) Are there landings at least 150cm wide positioned every 75cm maximum in ramp height?
Passageways	1) Are passageways at least 120cm wide? 2) Are there sections with enough space for a wheelchair to turn around at no more than 50m intervals? 3) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?
Tactile tiles	1) Tactile tiles or audible guidance devices (Exempt when proceeding straight ahead in an entrance enclosure) 2) Are tactile tiles laid next to the road? 3) Are tactile tiles laid in the section adjacent to the upper end of steps/incline?

Accessibility Standards Checklist

Designated facilities	Check items	
Entrances	1) All entrances (excluding entrances to toilets and bathrooms, and entrances collocated with entrances that meet the standards)	
	(1) Is the entrance at least 90cm wide?	
	(2) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
	2) At least one building entrance	
	(1) Is the entrance at least 120cm wide?	
	(2) Does the door open and close automatically, and is there a level section in front of and behind the door?	
	3) Entrances to wheelchair accessible guestrooms (in principle, at least 2% of all guestrooms)	
	(1) Is the entrance at least 80cm wide?	
	(2) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
Corridors	1) Are corridors at least 180cm wide (at least 140cm when there are sections with enough space for a wheelchair to turn around at no more than 50m intervals)?	
	2) Does the floor have a non-slip surface?	
	3) Tactile tiles etc. (section adjacent to the upper end of stairs or ramps)	
	4) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
	5) If there are any out-swinging side doors, are they set into alcoves?	
	6) Are any protruding objects set back so they do not obstruct visually impaired people passing by?	

Designated facilities	Check items	
	7) Are rest facilities suitably installed?	
	8) Items 1) and 4) above do not apply to areas where there is no obstruction to wheelchair users.	
Stairs	1) Are stairs at least 140cm wide (not including handrails up to 10cm wide)?	
	2) Are risers no more than 16cm high?	
	3) Are treads at least 30cm?	
	4) Are handrails installed on both sides of the stairs?	
	5) Does the floor have a non-slip surface?	
	6) Are the steps easy to distinguish?	
	7) Are the steps designed to prevent stumbling?	
	8) Tactile tiles etc. (landing section adjacent to the upper end of the stairs)	
	9) The main stairs are not circular stairs.	
Ramps	1) Are ramps at least 150cm wide (at least 120cm when collocated with steps)?	
	2) Is the grade no more than 1/12?	
	3) Are there landings at least 150cm wide positioned every 75cm maximum in ramp height?	
	4) Are handrails installed on both sides? (Exempt for inclining sections with a height less than 16cm)	
	5) Does the floor have a non-slip surface?	
	6) Is the ramp easy to distinguish from the corridor etc. to the front and rear?	
	7) Tactile tiles etc. (landing section adjacent to the upper end of the incline section)	
	8) Items from 1) to 3) above do not apply to areas where there is no obstruction to wheelchair users.	
Elevators	1) All elevators and elevator lobbies used by the general public	
	(1) Do the elevators stop at every floor? (Floors containing public rooms or guestrooms, toilets, parking spaces and bathrooms for wheelchair users, floors above ground level).	
	(2) Is the entrance to elevator cages and shafts at least 80cm wide?	
	(3) Are elevator cages at least 135cm deep?	
	(4) Is the elevator lobby level and at least 150cm square?	
	(5) Are elevator cages equipped with devices that display the intended destination floor and current location?	
	(6) Is the elevator lobby equipped with devices that indicate the direction the elevator cage is moving?	
	(7) Elevators in buildings used by the general public:	
	• Do they satisfy all conditions above from (1) to (6)?	
	• Are elevator cages shaped to allow wheelchairs to turn around?	
	2) At least one elevator or elevator lobby	
	Is the elevator cage shaped to allow wheelchairs to turn around?	
	(5) Is the elevator cage at least 135cm deep?	
	Are elevator controls in the elevator cage and lobby easy for wheelchair users to use?	
	Is the elevator cage equipped with devices that display the intended destination floor and current location?	
	(10) Elevators in buildings used by the general public:	
	• Is the entrance to elevator cages and shafts at least 90cm wide?	
	• Is the elevator lobby level and at least 180cm square?	
	(11) Elevators used by the general public or mainly by visually impaired people:	

Designated facilities	Check items	
	• Do they satisfy all conditions above	
	• Are elevator cages equipped with audible devices indicating the floor, and that the door is closing?	
	• Are elevator controls in the elevator cage and lobby easy for visually impaired people to use?	
	• Are elevator cages and lobbies equipped with audible devices that indicate the direction the elevator cage is moving?	
Elevators or escalators with a special structure or form of use	1) Elevators:	
	(1) Is the elevator a wheelchair lift?	
	(2) Is the floor area of the elevator cage at least 0.84m2?	
	(3) Is the floor area of the elevator cage adequate? (Nhen there is a need for wheelchairs to change direction in the elevator cage)	
	2) Escalators:	
	(1) Is the escalator accessible by wheelchair users? (Stipulated in the Proviso to Ministry of Construction Notification 1417 No.1 of 2000)	
Toilets	1) Are there wheelchair accessible toilet cubicles (in principle, at least 2% of toilets on each floor)?	
	(1) Are seat-style toilets and handrails appropriately positioned?	
	(2) Is there sufficient space for easy access and use by wheelchair users?	
	(3) Entrance (same for toilets with cubicles for wheelchair users)	
	• Are entrances at least 80cm wide?	
	• Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
	(4) Are there appropriate signs?	
	2) Are there floor urinals (in principle, at least 2% on each floor)?	
	3) Wheelchair accessible toilet cubicles in guestrooms (in principle, at least 2% of all guestrooms) (exempt when there are public toilets on the same floor)	
	(1) Are there wheelchair accessible toilet cubicles?	
	• Do they satisfy conditions 1) (1) and (2) above?	
	• Are entrances at least 80cm wide? (same for toilets with wheelchair accessible cubicles)	
	• Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors? (same for toilets with cubicles for wheelchair users)	
Passageways	1) Are passageways at least 180cm wide?	
	2) Does the floor have a non-slip surface?	
	3) Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
	4) Are there ramps or elevators in addition to stairs?	
	5) Stairs:	
	(1) Are stairs at least 140cm wide (not including handrails up to 10cm wide)?	
	(2) Are risers no more than 16cm high?	
	(3) Are treads at least 30cm?	
	(4) Are handrails installed on both sides of the stairs?	
	(5) Are the steps easy to distinguish?	
	(6) Are the steps designed to prevent stumbling?	
	6) Ramps	
	(1) Are ramps at least 150cm wide (at least 120cm when collocated with steps)?	
	(2) Is the grade no more than 1/15	
	(3) Are there landings at least 150cm wide positioned every 75cm	

Designated facilities	Check items	
	maximum in ramp height? (Exempt when the grade is 1/20 or less)	
	(4) Are handrails installed on both sides? (Exempt for sections with a height less than 16cm and a grade of no more than 1/20)	
	(5) Is the ramp easy to distinguish from the passageway to the front and rear?	
	7) Items 1), 3), 4), and from 6) (1) to (3) above do not apply to areas where there is no obstruction to wheelchair users.	
Car parks	1) Are there parking spaces for wheelchair users (in principle, at least 2% of parking spaces)?	
	(1) Is the width at least 350cm?	
	(2) Are they clearly marked?	
	(3) Are the parking spaces located close to the public rooms?	
Bathrooms	1) Are there wheelchair accessible bathrooms (at least one)?	
	(1) Are bathtubs and handrails appropriately positioned?	
	(2) Is there sufficient space for easy access and use by wheelchair users?	
	(3) Entrances	
	• Are entrances at least 80cm wide?	
	• Are doors easy for wheelchair users to pass through, and are there level sections in front of and behind the doors?	
	2) Wheelchair accessible bathrooms in guestrooms (in principle, at least 2% of all guestrooms) (exempt when there are public bathrooms)	
	(1) Are there wheelchair accessible bathrooms?	
	• Do they satisfy conditions 1) (1)-(3) above?	

- Paths for visually impaired people:

Designated facilities	Check items	
Access routes to the facility	1) Tactile tiles or audible guidance devices (Exempt when proceeding straight ahead in an entrance enclosure)	
	2) Are tactile tiles laid next to the road?	
	3) Are tactile tiles laid in the section adjacent to the upper end of steps/incline?	

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EFFECT OF CLIENT REGULATIONS UPON THE QUANTITY SURVEYORS ROLE IN CONTROLLING PROJECTS

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ABSTRACT

Throughout the world, Quantity Surveyors are working on projects as diverse as housing, commercial property developments, hospitals, mosques, highways, dams and airports. Whenever any building project is proposed, it is important that the cost involved is known in advance. These include site preparation cost, construction, labor, material and plant costs, professional fees, taxes and other charges as well as the likely running and maintenance costs for the new building. The Quantity Surveyor is trained to evaluate these costs and to advise on alternative proposals. This research reports on the finding of an empirical study of 900 cases representing the payment documents for the projects of 2005 within a big client governmental organization in The United Arab Emirates. It shows that the release period of the payment documents is double the allowed period. It was aimed to find out the main reason for that phenomenon. The findings showed that the role of the quantity surveyor was the key factor, therefore, a review of the relevant literature was undertaken prior to the preparation of the new system for evaluating the quantity surveyors who will join the organization's staff or those who will be hired by the consultants supervising the projects owned by the organization. The study also showed that the international organizations such as universities and societies are having separate divisions for the quantity surveying profession which is not the case in the Arab countries including Egypt. Many steps are required from the academic and profession institutions in Egypt and the Arab countries regarding the strengthening of the quantity surveyor role through academic syllabuses, codes of practices, contractual documents…etc.

Keywords: Quantity Surveyor, POMI, RICS, Cost, Takeoff, Management code of practice

INTRODUCTION

Although each project has a specific goal, but during the project construction phases, the roles of the different parties vary based on many factors such as stakeholders' objectives, contract type, the project stage, project duration, available resources, …..etc. One of the jobs that plays an important role during the different project stages is the "Quantity Surveyor" which is one of the roles that is needed for the main project parties i.e. client, consultant and contractor.

The role of the Quantity Surveyor "Q.S." is, in general terms, to manage and control financial and contractual matters within construction projects and involves the use of a range of management procedures and technical tools to achieve this goal [12].

Although the above mentioned definition is accepted worldwide and especially in the Commonwealth countries (Britain, Australia, Canada, India, …..), but in many other countries (including Egypt) this expression "Quantity Surveyor" denotes quantities takeoff only. Consequently, this reflects the required qualifications and educational background that are required to hire a quantity surveyor. Adopting the previous definition, it is obviously noted that the duties of the Q.S. are identical to the job which is called "Technical office engineer" in Egypt.

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The profession of the quantity surveyor developed during the 19th century from the earlier "Measurer", a specialist tradesman, who prepared standardized schedules for a building project in which all of the construction materials, labor activities and the like were quantified and against which competing builders could submit priced tenders [12] i.e. the traditional role of Quantity Surveying is concerned principally with the cost and legal management of construction projects. This remains central to what most Q.S.[S] do, however, in the past 20 years or so, the profession has evolved rapidly and in some ways dramatically. For example, procurement, a term not used until the 1980's, became an important area of activity, largely because of the increasing options that were available.

Also, increased importance and emphasis has been placed upon design and cost planning as a tool that is effective in meeting client's objectives. This was coupled with whole life costing, value management and risk analysis and management were other tools being used to add value for the client [11]. Therefore, no job was ready or has the ability to acquire the duties and assignments of these new terms and fields of project management except the quantity surveyors. This study will explain, in detail, the duties of the quantity surveyor and how to evaluate his competencies.

THE RESEARCH PROBLEM AND OBJECTIVES

The particular problem that has been acknowledged is that the monthly payment documents for contractors are very much delayed in the contracts and purchasing department (C&P) within a client organization (owner) which resulted in declining of most of contractors to bid in new projects. Taking into account that the city where the organization operates in United Arab Emirates is undergoing rapid growth in the past few years and the demand for construction services is more than supply, the problem gets worse since most contractor bid with an extremely high prices in an attempt to avoid winning the tender and in the same time to satisfy the important client. The complicated procedures of finalizing the monthly payments which resulted in these noticeable delays in this governmental organization which construct projects with a total value of 600 million $ were the main driving force for this study which has the following objectives:

1- Finding out the possible causes for such delays in processing the monthly payment of contractor.
2- Recommending the suitable and feasible solution.
3- Emphasizing on the effect of the human factor competency in eliminating the problem causes.
4- Showing how the clients' rules affect the construction market professions.
5- Measuring the effects (positive or negative) of implementing the recommended solutions.

RESEARCH METHODOLOGY

The methodology adopted in this causes is a straight forward one. It starts with citation of the problem and its symptoms supported by some statistics extracted from the log of the C&P department. The statistical measures show that the delay in processing the payments is not acceptable from the time point of view. The investigation shows that the educational background of the quantity surveyors has a big effect on their efficiency. Therefore, the duties of the Q.S. were zoomed taking into consideration that these duties vary according to the Q.S. employer (client, consultant or contractor).

Based on the wide range of the Q.S. duties, the academic syllabus of the quantity surveying path is highlighted in addition to focusing on the international societies and organizations that serve this profession. Then the study shows how the Q.S. duties and his academic education affects the establishment of an evaluation process for the quantity surveyors who will take a role in the projects owned by the organization of our study. Finally, the effect of implementation of some recommended solutions is recorded and discussed.

PAYMENTS STATISTICS IN AN EMIRATI GOVERNMENTAL ORGANIZATION (CASE STUDY)

As mentioned earlier in the “research problem”, there were many complaints from contractors regarding the delay of the monthly payments in a big governmental organization. A comprehensive study was performed to pinpoint the reasons for these delays and recommending suitable solutions. The study started by identifying the flow of the monthly payment documents. This flow is shown in Fig. 1.

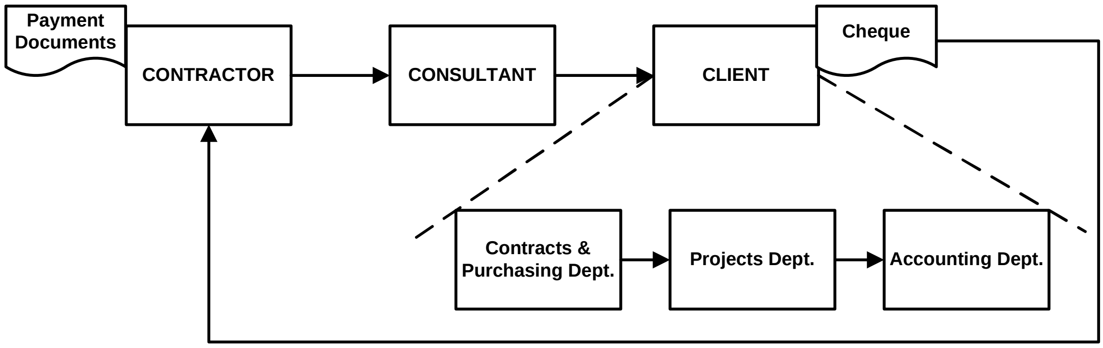


Fig. 1: Flow of Monthly Payment Documents

The cycle shown in Fig. 1 is supposed to take 10 days at maximum within the Contracts & Purchasing dept, of the client organization, but the study of 900 cases representing the projects of 2005 shows that the release period of the payment documents jumps to 21 days as an average and a median of 12 days (which means that 50% of the payments is released within 12 days). All data associated with these 900 cases were captured from the register of the contracts and purchasing (C&P) department. Table 1 shows the work load for 9 quantity surveyors within the department and some statistical measures for the time elapsed in completing the release of payment cycles while Fig. 2 explains the performance of each Q.S. and how far are they from the target.

Table 1: Some Statistical Measures for the Time of Processing the Payments by Different Q.S^{s}

Quantity surveyor ID	# of processed Cases	Average Time of release	Median	Standard Deviation	Minimum	Maximum
AF	77	30.23	23	35.2	0.5	263
AH	57	12.24	11	11.66	0.5	61
HM	88	14.9	10	16.08	0.5	83
KR	51	27.76	14	38.86	0.5	205
MA	109	17.15	7	34.65	0.5	233
MN	126	13.80	7	23.77	0.5	203
MT	146	18.76	15.5	20.5	0.5	168
MW	104	28.22	14.0	37.78	0.5	252
RP	142	16.35	11.5	18.54	0.5	176

Note: all times are in (days)

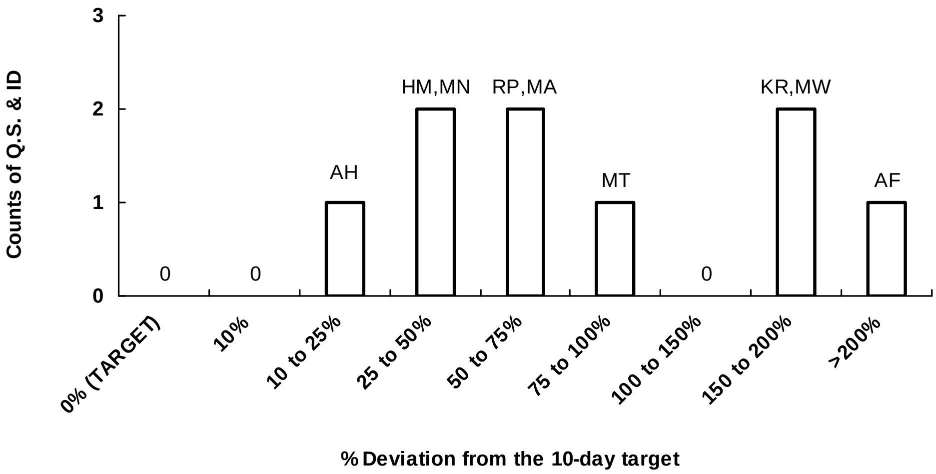


Fig. 2: Quantity Surveyors Performance

Navigating deeply in the data and taking into consideration that the less the processing time, the better the performance of the quantity surveyor, none of the quantity surveyors has achieved the target of 10 days of processing time. It also can be seen that up to 50% deviation from target (i.e. 15 days) there are only three Q.S.^{S} out of nine who achieved this relatively big percentage of deviation, while other Q.S.^{S} deviate from target by more than 50%.

Re-considering the data in Table 1 and the Median of each Q.S., it can be observed that the quantity surveyors HM, MA & MN have Medians of 10, 7 and 7 respectively which means that nearly 50% of the payments processed by these quantity surveyors were released within the time policy of 10 days. However, the overall average for each Q.S. is greater than 10 days.

Based on these results, a further comprehensive investigation was performed to identify the factors which affect the processing time of the monthly payments within the C&P department. The investigation was conducted by the quality team which identifies the following as possible causes for the delay:

1. The submitted documents (payment attachment) are incomplete.
2. Too much workload is probably assigned to the quantity surveyors.
3. Some quantity surveyors may be inefficient.
4. The target of the processing time (10 days) may not be realistic.

The quality team was further asked to recommend the possible solutions for the above mentioned probable causes of delay. Table 2 summarizes the suggested solutions and shows whether they were agreed to be implemented or not.

Table 2: Recommended Solutions for the Possible Causes of Payments Delay

Sr.	Possible Cause of Delay	Recommended Solutions	Impleme-ntation
1	The submitted payment documents are incomplete	1-1 Awareness sessions for contractors.	No
		1-2 Preparing checklists for the complete submission.	Yes
2	Possible high work load for Q.S.	2-1 Increase Q.S. employees.	No
		2-2 Increase working hours per week.	No
3	Possible inefficiency of Q.S^{S}	3-1 Determining the minimum requirements to be fulfilled by the new joining Q.S.^{S}	Yes
		3-2 Preparing a comprehensive test evaluation sheets for qualifying new Q.S^{S} during interview.	Yes
4	The target of the processing time (10 days) may not be realistic	4-1 Reducing processing time was infeasible because of many	No

Sr.	Possible Cause of Delay	Recommended Solutions	Impleme-ntation
		contractual and strategic issues.	

Many of the recommended solutions were totally implemented as seen in Table 2 while others were not. For example, item (1-2) was implemented through a one page checklist for the complete submission of the monthly payments and their attachments. This checklist was distributed for all contractors who have current projects with the organization and was planned to be delivered to contractors of new projects during the pre=construction meeting (added as an item in the meeting agenda).

The second implemented solution was the most important recommended one. It stated that "Determining the minimum requirements to be fulfilled by the new joining Q.S.^{S}" (see item 3-1 in table 2). The importance of this recommended solution came from the result which concluded from the previous analysis: "better quantity-surveyor performance was very much correlated to their academic educational background". In other word, those Q.S.^{S} who received an academic education related to quantity surveying syllabus have better performance than those who had the traditional civil engineering syllabus.

This result was deduced with the aid of the data in Table 1, Fig. 2 and data related to the quantity surveyors' academic education.

Therefore, and based on the later mentioned finding, the following paragraphs will focus on highlighting the role of the quantity surveyors in construction projects and from different points of views for the sake of getting a clearer image why quantity surveying background positively affects the Q.S. performance. This would help in two directions; the first is determining the suitable qualification of competent Q.S. and secondly, in preparing an evaluation test form for these Q.S.S who have the desire to join the C&P department within the organization.

THE ROLE OF QUANTITY SURVEYORS ACCORDING TO THEIR EMPLOYER

Depending on the employer (owner, consultant or consultant), the quantity surveyor plays different roles resulting in different duties and outputs. Typical duties of the Q.S. when his employer is a *contractor* are [2]:

1. Dealing with all commercial matters.
2. Capturing chances to get maximum benefits from the project (money-time).
3. Utilizing approved suppliers and subcontractors.
4. Preparing subcontract documentations.
5. Administer the payments, insurances and bonds.
6. Produce monthly cost value reconciliation.
7. Prepare quarterly forecast reports.
8. Undertake re-measurement and prepare, submit and agree monthly valuation and variations.
9. prepare cash flows and claims documentations
10. Identify areas of waste or loss on materials.
11. Ensure proper archiving of site documentation.
12. Monitor all project correspondence for commercial issues.

The duties of the *client's quantity surveyor* are much different and are centralized around reviewing and checking of payments documents during the construction phase as follows:

1. Verify retention money details / contract price.
2. Verify Back-up calculations & documentations for any new rates.
3. Verify detailed back-ups of Day-works and additional works.
4. Verify detailed back-ups of final quantities and the method of measurement.
5. Verify quantities reconciliation, provisional sums, material on site, claims, variations, adjustment item and overheads in the final certificate.
6. Checking key dates of instruction / direct payment details with the original invoices.
7. Consultant confirmation letter on fulfilling the requirements to process advance payment.

8. Verify advance payment recovery details.
9. Verify project code and contractor payee code.
10. Correct dates in the payment certificate / appendices.
11. Verify taking over certificate/defects liability certificate for retention release.
12. Statement at discharge and laboratory clearance attachment and details in final certificate.
13. Verify approval on the new rates in the final certificate.
14. Correct dates in the payment certificate / appendices.
15. Verify BOQ rates in the final certificate.
16. Verify previous paid amount in the final certificate.

Finally, the British code of practice [1] was the main source for extracting the duties of the *consultant's quantity surveyor*. Table 3 summarizes these duties according to the project stage.

Table 3: Duties of Consultant's Quantity Surveyor as Described in the British Code of Practice [1]

Q.S. Duties	Stage/ page #
- Preparing capital expenditure program through the client team.	Inception stage - Page 4
- Preparing preliminary estimate of project costs.	Feasibility stage - Page 10
- Financial and contractual advice through the design team.	Strategy stage - Page 29
- Discussing the following in the pre-start construction meeting: 1- Adjustment to tender figures. 2- Agree procedures for valuations. 3- Clarification of items related to day-work rates and taxes.	Pre-construction stage - Page 46 & 48
- Measuring the value of work executed by main contractor. - Agree monthly valuations with contractor. - Agree the final account.	Construction stage - Page 54
- Evaluating / calculating the cost implications for changes in the client's brief.	Construction stage - Page 63
- Physical measuring of the works carried out on site and the costing the quantity of work against the rates in the BOQ. (With the contractor).	Construction stage - Page 67
- Establishing with the PM the cost monitoring and reporting system and providing feedback to the other consultants and the client on budget status and cash flow.	Responsibilities - Page 97
- Incorporates the effect of the approved recommendations into the cost plan.	Changes during the design development process – page 142
- Cost monitoring and reporting with the assistance of and input from the design team, other consultants and contractors.	Cost control reporting – page 166

For any quantity surveyor to have the ability to perform the above mentioned duties whether working with a contractor, consultant or client, he/she should receive an academic education that copes with these required high qualifications.

Many countries especially in the Commonwealth countries (Britain, Australia, Canada, India and East Asia… ….etc) have realized the importance of the Q.S. role and established – a long time ago - separate divisions for quantity surveying specialty in their academic institutions. The syllabus in one of these institutions [11] in Britain is:

First Year

In Year One studies concentrate on the principles of knowledge on which a Quantity Surveying is based; Law, Economics, Management and Construction Technology. An overview of the Construction Industry and the parties involved is also provided together with Information and Communication Technology.

Second Year

In the second year the program deepens the student's understanding of Quantity Surveying via modules in Construction Technology, Building Economics, Contract Administration, Development Law, Measurement, and Construction Management & Procurement. These skills are combined in a Project module.
Third and Final Year
In the third and final year students become familiar with more advanced techniques of analysis, evaluation, appraisal and forecasting, and the ability to apply these techniques to solve practical problems. Modules include Measurement, Construction technology, Development Economics and Project Management. Students also undertake an Interdisciplinary Professional Development Project and write a Dissertation. Students also undertake a Field Study Visit to another member state of the European Union [11]. Similar courses are available in countries which have separate quantity surveying specialty in their academic institutions.
Therefore, it was decided internally within the organization (our case study) that candidates who have quantity surveying degree will attain higher score than other academic degrees. This decision was reflected in the Q.S. selection criteria and the evaluation form which will be shown later in Fig. 3.

QUANTITY SURVEYORS AND INTERNATIONAL INSTITUTIONS

As previously mentioned, the traditional role of Quantity Surveying is concerned principally with the cost and legal management of construction projects. Due to the rapid evolvement of the quantity surveying profession in the past 20 years or so, many professional institutions and societies are established to cope with these fast changes and to be the entity that offer many services to the profession such as training courses, conferences, reference books, global network of professionals and strong links with the business community.
The professional institution with which most English-speaking Quantity Surveyors are affiliated is the UK based Royal Institution of Chartered Surveyors (RICS). This institution is considered the most famous one in the world and its efforts are noticeable very well. One of these efforts is issuing the Principles Of Measuring (International) in 1979 for works of construction (abbreviated POMI), which is considered the main reference for measuring the works and preparing the bills of quantities. To join the RICS and be a member, Q.S. has have to have certain qualifications and should pass a series of tests including oral ones through representatives spread in most of countries. There are 3 categories of memberships in the RICS which are: MRICS - a chartered surveyor (member of RICS), TechRICS - a technical surveyor (technical member of RICS) and with the right experience progress could be made to FRICS (a fellow of RICS).
In addition to this well known institution of RICS, there are many entities which plays the same role of RICS but locally and on a smaller scale than RICS. Examples to these institutions are:
Australian Institute of Quantity Surveyors (AIQS), Canadian Institute of Quantity Surveyors (CIQS), Hong Kong Institute of Surveyors (HKIS), Institute of Quantity Surveyors of Kenya (IQSK), Institute of Quantity Surveyors Sri Lanka (IQSSL), New Zealand Institute of Quantity Surveyors (NZIQS), The Institution of Surveyors, Malaysia (ISM).

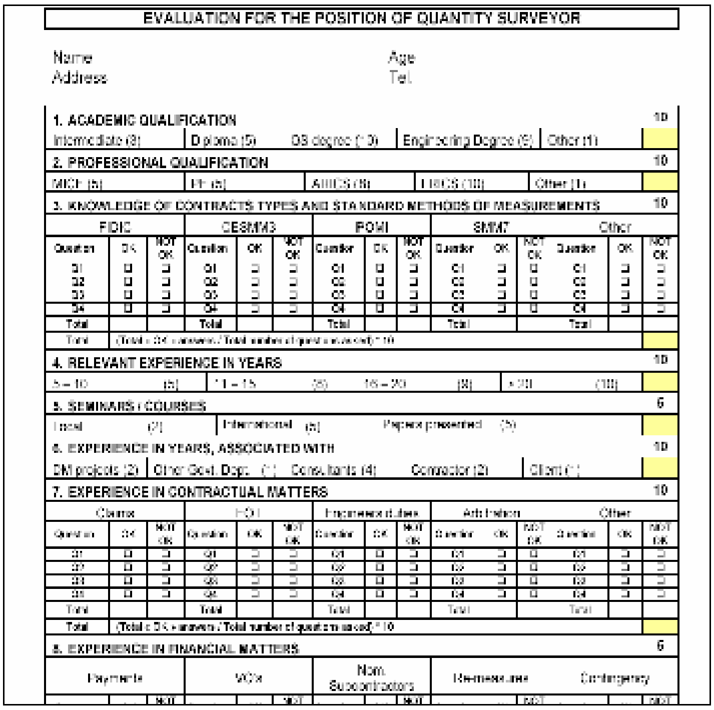
EVALUATION OF QUANTITY SURVEYORS

Up to this point, the study has discussed the problem of delaying the payments within the C&P department and as a result of the problem analysis, some recommended solutions were suggested. One of the recommendations was to determine the qualifications required for new Q.S. employees who want to join the staff of the C&P dept (see item # 3-1 in table 2). The second recommendation was to establish evaluation forms which can be used in the interview of new candidates (see # 3-2 in table 2). This effort was done jointly between the quality team and the head of tenders and quantity surveying.
Many factors were taken into consideration before establishing the form, such as:

1- The academic qualifications, where quantity surveying degree attains higher score than other degrees.

2- The professional qualifications, where the membership of the RICS takes the biggest score amongst others. Besides, the fellow member gets higher score than other types of member.
3- Awareness of contract types.
4- Awareness of standard methods of measurements such as: POMI [6], CESMM [3], SMM7, …etc.
5- Contractual matters as: claims, arbitrations, engineering duties,….
6- Financial matters as: payments, variation orders, contingency, …..
7- Awareness of conditions of contracts as: final accounts, penalties, insurances, …..
8- Knowledge of planning techniques.
9- Other miscellaneous personal skills and experience.

Fig. 3 shows a portion of the newly established form which is used in candidates evaluation.



EVALUATION FOR THE POSITION OF QUANTITY SURVEYOR

Name Age
Address Tel.

1. ACADEMIC QUALIFICATION 10
Intermediate (3) | Diploma (5) | BS degree (10) | Engineering Degree (6) | Other (1)

2. PROFESSIONAL QUALIFICATION 10
MICE (5) | FE (5) | AIIRCS (8) | FRICS (10) | Other (1)

3. KNOWLEDGE OF CONTRACTS TYPES AND STANDARD METHODS OF MEASUREMENTS 10

FIDIC			CESMM3			POMI			SMM7			Other		
Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK
Q1	☐	☐	Q1	☐	☐	Q1	☐	☐	Q1	☐	☐	Q1	☐	☐
Q2	☐	☐	Q2	☐	☐	Q2	☐	☐	Q2	☐	☐	Q2	☐	☐
Q3	☐	☐	Q3	☐	☐	Q3	☐	☐	Q3	☐	☐	Q3	☐	☐
Q4	☐	☐	Q4	☐	☐	Q4	☐	☐	Q4	☐	☐	Q4	☐	☐
Total			Total			Total			Total			Total		

Total (Total = OK answers / Total number of questions asked) * 10

4. RELEVANT EXPERIENCE IN YEARS 10
5 – 10 (5) | 11 – 15 (6) 16 – 20 (8) | > 20 (10)

5. SEMINARS / COURSES 6
Local (2) | International (5) Papers presented (5)

6. EXPERIENCE IN YEARS, ASSOCIATED WITH 10
DM projects (2) | Other Govt. Dept. (1) Consultants (4) Contractor (2) | Client (1)

7. EXPERIENCE IN CONTRACTUAL MATTERS 10

Claims			EOT			Engineers duties			Arbitration			Other		
Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK	Question	OK	NOT OK
Q1	☐	☐	Q1	☐	☐	Q1	☐	☐	Q1	☐	☐	Q1	☐	☐
Q2	☐	☐	Q2	☐	☐	Q2	☐	☐	Q2	☐	☐	Q2	☐	☐
Q3	☐	☐	Q3	☐	☐	Q3	☐	☐	Q3	☐	☐	Q3	☐	☐
Q4	☐	☐	Q4	☐	☐	Q4	☐	☐	Q4	☐	☐	Q4	☐	☐
Total			Total			Total			Total			Total		

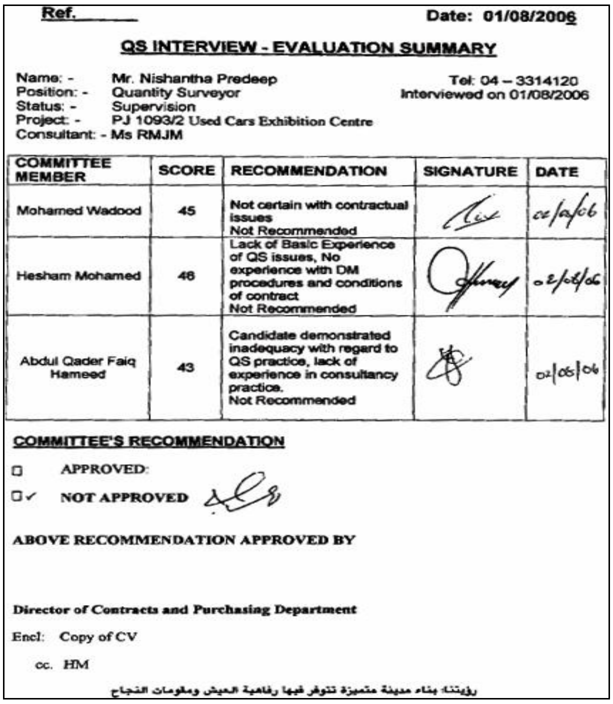
Total (Total = OK answers / Total number of questions asked) * 10

8. EXPERIENCE IN FINANCIAL MATTERS 6
Payments | VO's | Nom. Subcontractors | Re-measures | Contingency

Fig. 3: Part of the Quantity Surveyor Evaluation Form

It worth mentioning that this form was used for two purposes, the first is for evaluating the Q.S.[S] who want to be employed by this organization, while the second purpose is using the form for evaluating consultants' quantity surveyors. The later purpose needs more explanation, where all projects of the organization are supervised through outsourced consultants. These consultants employ their quantity surveyors after getting approval from the organization which conducts a complete test for them because these Q.S.[S] will be responsible for the projects owned by the organization although they are employed by the consultant. The organization aimed from this action to assure competency of the Q.S.[S] who will daily manage its projects through the consultant.

Fig. 4 shows the summary report for the evaluation of one of quantity surveyors who had the desire be hired by one of the consultant who supervises a project owned by the organization. (As seen from Fig. 4, this Q.S. did not pass the test and was not recommended to be hired).



Ref.______ Date: 01/08/2006

QS INTERVIEW - EVALUATION SUMMARY

Name: - Mr. Nishantha Predeep
Position: - Quantity Surveyor
Status: - Supervision
Project: - PJ 1093/2 Used Cars Exhibition Centre
Consultant: - Ms RMJM

Tel: 04 – 3314120
Interviewed on 01/08/2006

COMMITTEE MEMBER	SCORE	RECOMMENDATION	SIGNATURE	DATE
Mohamed Wadood	45	Not certain with contractual issues Not Recommended		02/08/06
Hesham Mohamed	46	Lack of Basic Experience of QS issues, No experience with DM procedures and conditions of contract Not Recommended		02/08/06
Abdul Qader Faiq Hameed	43	Candidate demonstrated inadequacy with regard to QS practice, lack of experience in consultancy practice. Not Recommended		02/08/06

COMMITTEE'S RECOMMENDATION

☐ APPROVED:

☐✓ NOT APPROVED

ABOVE RECOMMENDATION APPROVED BY

Director of Contracts and Purchasing Department

Encl: Copy of CV

cc. HM

رؤيتنا: بناء مدينة متميزة تتوفر فيها رفاهية العيش ومقومات النجاح

Fig. 4: A Copy of an Actual Q.S. Evaluation Summary

IMPLEMENTATION EFFECTS

Although the recommended solutions were suggested for the sake of avoiding the problem of delays within the case-study organization, some drawbacks did appear. The most noticeable one was the high salaries required by the Q.S.^{S} after realizing that the client organization applies comprehensive tests which not all the Q.S.^{S} can pass. This increase in salaries was noticed for both kinds of Q.S.^{S}, those who want to join the staff of the organization, and those who will work with consultants supervising the organization's projects. No doubt that these increments were added to the contract price resulting in increased overheads and fees.
The second effect was expected, which is the inability of the majority of the civil engineers with moderate experience to pass the evaluation tests. It requires them to re-submit for retesting which again led the consultants to seek hiring new graduate Q.S.^{S} from East Asia (Sri-Lanka, India, Malaysia,…..) who succeed in passing the test easily rather than others.

Some further actions taken after implementing the previous recommendations of Table 2, the first action was by the quality team who has established a departmental-level key performance indicator (KPI) to measure the processing time of the payments. This departmental-level KPI is now used together with other indicators to get the overall performance level of the contracts and purchasing department. This KPI which is measured quarterly and compared with the previously determined target of 10 days processing time, is monitored by the department of administrative development and quality within the organization.

The second important action which is now in its final stages and will have a great effect is the automation of the payments submission process. This automation will be through a web-based developed software called "PRISM" (Project Information system for management). This software was mainly developed by an outsourced IT company to improve, control and monitor the projects by enhancing projects' communications. At the time being, the contractor can submit his documents through this web-based software, where all revisions and approvals are online and are timely traceable. Fig. 5 shows a typical screen of the software (January 2007 is the user acceptance testing period of this software program).

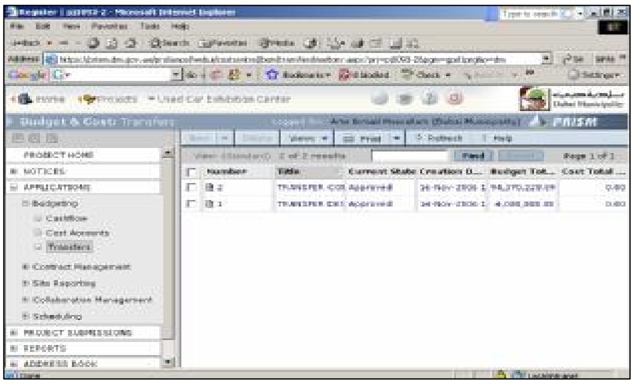


Fig. 5: Typical Screen from the Newly Web-Based Established Software (PRISM)

SUMMARY & CONCLUSIONS

This research paper discussed the phenomena of delayed payments within one of the largest governmental organizations in the United Arab Emirates. The data analysis showed that the human factor (quantity surveyor) was the most important cause behind such a problem. Therefore, the study aggregated the different duties which the Q.S. is doing in any construction project in addition to focusing on the Q.S. academic path for the sake of establishing a system for evaluating the Q.S. candidates to join the projects owned by the this organization (as employees or working with the consultants hired by organization). Some further actions were taken to support the efforts toward reducing the processing time of monthly payments such as establishing departmental level key performance indicator (KPI) and starting the automation process of payment submission through web-based software.

Many conclusions could be derived from this study, which can be summarized as follows:

1) Although the profession of quantity surveying is not well known in Egypt, there is a big need for quantity surveyors especially in the Arabian Gulf Area as a result of the construction booming there.
2) The duties assigned to the quantity surveyor are different according to his employer whether owner, consultant or contractor.
3) The quantity surveyor has a great effect on the contractual and financial matters.
4) The newly graduate civil engineer does not have the same capabilities of the newly Q.S. graduate regarding all financial and contractual matters.
5) There is a lack of Quantity surveying education either in Egypt or in the Arab countries.
6) There are great number of international institutions and societies that serve the Q.S. profession.

RECOMMENDATIONS

Although some drawbacks were noticed due to the implementation of the Q.S. evaluation process (overheads and salaries increase) and taking the above mentioned conclusions into consideration, some recommendations can be raised as follows:

1) Educational prospective:
   As previously mentioned, some nationalities monopolized the field due to their specialty in quantity surveying. These nationalities profiteer the chance that certified Q.S.[S] who has quantity-surveying background are widely demanded and raised their salaries. Therefore, it is strongly recommended that Arab academic institutions establish such syllabus in their colleges and universities in cooperation with well-known certifying agencies (like the royal institute of chartered surveyors or similar). It should be taken into consideration that the

quantity-surveying syllabus is similar to a big extent with the construction management syllabus which is applied in few universities either in Egypt or in the Arab countries.

2) Stakeholder prospective:
The client (owner) organizations or developers are recommended to adopt the same methodology in hiring Q.S.^{S} or in qualifying those who will work for the consultants supervising their projects.
3) Codes of Practice:
Since the Egyptian code of practice for project management is in its early preparation stages, the author strongly suggest and recommend that this pioneer Arabic code addresses clearly the role of the Q.S. and take a step toward changing the common concept in the Egyptian construction market that Q.S. is not a highly educated person and his duties in not only limited to quantities takeoff.
4) Future research:
For further research works, it is recommended to study the role and duties of the Q.S. according to the type of contract.

ACKNOWLEDGEMENT

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SOURCES OF WORKFLOW UNCERTAINTY AND VARIATION IN RESIDENTIAL CONSTRUCTION PROJECTS

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ABSTRACT

Recent growth in the US residential construction has created increased pressures to homebuilders and trades. According to previous research, this has resulted in longer cycle times and more quality problems. This paper presents an in-depth investigation into the cycle time performance of a large residential construction project in order to identify problems (and patterns of problems) that lead to delays. The researchers collected detailed data on nine single-family houses in a large residential development in the Phoenix area, built by one of the largest homebuilders in the US. The paper first presents the schedule performance of the nine units which reveals significant delays. To better understand the reasons for delays, we then examine the planned and actual work performed on a weekly basis, and identify the reasons for activities not completed as planned. The analysis indicates that limited labor availability and quality problems caused extensive workflow disruptions and increased cycle time. Then we examine why the production control system failed to manage effectively the workload and quality issues. Finally, the paper discusses how the use of the Last Planner System can benefit homebuilders by reducing defects, increasing labor availability and reducing construction cycle time.

Keywords: Residential Construction, Cycle Time, Delays, Production Control, Lean Construction.

INTRODUCTION

Residential construction in the United States is currently one of the fastest growing sectors of the construction industry. This growth is partly attributed to historically low interest rates. Furthermore, the demand for homes in this decade is expected to steadily increase as the number of households increases from 105 millions in 2001 to 115 millions in 2010 [11]. The growth of the residential construction industry has created increased pressures to homebuilders and trade contractors to build as many homes as possible within a shorter period of time [12]. According to an industry publication [13], homebuilders estimate that when accounting for the cost of money, contractor costs, management costs and sales opportunities, each day off the construction schedule in the US is worth between $50 and $500. Many homebuilders have fully or partially adopted production homebuilding to leverage the economy of scale on material procurement as well as getting homes built in larger numbers and in less time.

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Despite the market pressures for higher speed, cycle time performance has shown little or no improvement. According to the US census bureau [14], the average time from start to completion for single residential units built for sale increased from 5.5 months in 1999 to 5.9 in 2005. According to the Joint Center of Housing Studies of Harvard University [9], the average cycle time for National builders (more than 2,500 units each year) has been reduced from 130.6 days in 1999 to 126.1 days in 2004. The cycle time of regional builders (between 500 and 2,500 unit each year) has increased from 93.6 days in 1999 to 119.7 days in 2004.

Researchers of residential construction, have also identified the need to improve the efficiency and quality of the residential construction [5, 8]. Bashford et al. [6] estimated that the average construction cycle time in the Phoenix area is 152 days. This lengthy construction cycle and the resulting work in process are largely due to non- productive time during the construction process. A multi-year study of on-site home building operations in the Phoenix metropolitan area found that actual construction operations consumed 25 to 40% of available working time (8 hours per day, 5 days per week). Houses sit idle over 50% of the available work time [1]. Bashford et al. [7] also studied the effect of inspection failures on the cycle time. The researchers examined the four critical-path inspections (pre-slab, framing, drywall and final inspection) and found that failures to pass critical-path inspections add on average 12 days to the cycle time (pre-slab 2, framing 1, drywall 4 and final inspection 5) due to rework and re-inspection. Inspection failures constitute only a portion of the cycle time problems. Furthermore, they reflect an underlying problem with quality during the construction process. Quality problems during the construction process (generally referred to as construction defects), result in rework and additional cost and time (when the problems are corrected) or reduced quality of the new homes (if the problems are not corrected). The cost of rework is estimated to be an average of 12% of the total project cost [10].

OBJECTIVES AND METHODOLOGY

The preceding discussion highlights the critical need to understand the causes of increased cycle times and quality problems in residential construction. This paper presents an in-depth case study of the construction process at a large residential subdivision in the Phoenix area. The objectives of the study are the following:

- Describe the actual situation on the project and the cycle time performance.
- Identify the reasons for increased cycle time.
- Understand how the production control system managed (or failed to manage) these issues.

The authors collected data over a five month period (from Feb 05 to June 05), on the construction of nine single family houses. These units were part of a larger residential development that included about 500 houses built over four years. About 100 houses were constructed at the time the study was conducted. For each house, the study collected the following data:

- Plan and actual schedule data were collected from schedule records and direct observation.
- Data on work activities completed vs. planned were collected weekly from approximately the start of drywall to the completion of each house. The number of work activities completed every week divided over the total work activities planned for that week is the ‘Percent Plan Complete’ (PPC).
- Information on the reasons for not completed activities (from the beginning of drywall to the end of the project).

- Other data were collected through observation of the progress and problems of other houses in the subdivision, discussions with other superintendents, and informal interviews with trades, inspectors, customers, etc

The information was collected primarily through direct observation of the construction process. A unique element of this case study is that one of the researchers was working as an intern assistant superintendent on the site, and helped supervise the nine units from the beginning of drywall till completion. Thus, the observer was at the same time a participant in the project which enabled a first hand understanding of the project situation and the problems that occurred. Another unique element of this case study is that in order to understanding how the overall delays resulted, we tracked and analyzed the reasons for delayed activities on a weekly basis.

Because of the small sample of houses from the same subdivision, this case study does not allow generalizations regarding the frequency of the problems on other residential projects. However, the case study approach provides an in-depth understanding of the production factors that resulted in the long delays and the mechanism by which these delays were brought about. The case study enables analytical generalization (regarding the causality of the phenomenon) rather than statistical generalizations. Thus, the lessons learned from the case can be generalized to residential projects.

The next section presents an overview of the project organization and market conditions. Then, the paper presents and discusses the cycle time performance (planned and actual construction schedule for each house) of the nine units. Then, the paper analyzes the activity delays on a weekly level and the reasons for these activities not completed as planned. The analysis identifies the causes of the increased cycle time, and the conclusions propose a pro-active approach to production management to prevent the problems identified on this project.

PROJECT OVERVIEW

In recent years, Phoenix, Arizona, became the second largest new home market in the US. The increased market demand has resulted in increased workload for contractors, labor shortage and occasionally material shortages (concrete, steel, lumber). The developer in this study is one of the largest and most successful developers in the US with operations in more than 28 states. In recent years the developer has received several awards and is recognized as one of the strongest homebuilders.

The developer organizes the construction process in two phases, each with a different superintendent. Phase 1 includes the operations from the beginning of construction to the start of drywall. Phase 2 starts with drywall (install, tape, sand, texture, cleanup), and ends with the completion of the house. Phase 2 includes two major work sequences: interior work and exterior work. Table 1 summarizes the major activities in each phase.

Table 1. Construction Process Overview

Phase 1	Phase 2-interior	Phase 2-exterior
Grading	Drywall (interior)	Drywall (exterior)
Layout House	Trim Carpentry	Stucco
Underground Utilities/ plumbing/ Electrical	Interior paint	Exterior paint
Slab on grade	Cabinets and countertops	Roof tile
Framing	Plumbing trim	Fence grade
Top out plumbing	Mechanical & Electrical trim	CMU (Concrete Masonry Unit) wall
Load roof	Flooring (ceramic tile, vinyl & carpet)	Gates
Framing inspection	Install utility meters	Site concrete (driveway, garage slab, etc.).
Electrical & Mechanical Rough in	Inspections, punchlist, touch up, cleanup	Inspections, punchlist, touch up, cleanup
Insulation	Appliance installation	

The detailed schedule includes about 170 activities (including procurement activities). About 30 activities are quality control related including city and company inspections, walkthroughs, and punchlist. The schedule has some slack built-in, as the units were scheduled for 80-86 work days (for both one and two story houses). For example, the duration assigned for framing is 8 days for a 1,600 sqft unit and 10-12 days for a two-story 2,600 sqft unit. For a framing crew of six to seven workers, the average duration for a single story 1,600 sqft house was five work days. The last two weeks of the schedule are dedicated to punchlist and inspection activities.

The critical path includes phase 1 and phase 2 interior work. Framing and drywall are the longest activities. The homebuilder does not perform any work with their own labor forces—all work is performed by subcontractors. The homebuilder's superintendents supervise and coordinate the process. A detailed schedule is developed for every house by the home office. The schedule is monitored and adjusted by the superintendents in the field. Each superintendent supervises several houses. The project manager is responsible for the overall development and she/he can change the priorities of the subcontractors.

CYCLE TIME AND SCHEDULE PERFORMANCE

The construction cycle time was measured as follows: the starting point was the day the "house layout" activity was performed. The finish milestone was the day the house was officially accepted as 'complete' by the company after all inspections were completed. Figure 1 shows the planned and actual start and finish of the nine units in ordinal work days. Day 1 is a different calendar day for each unit. Negative numbers indicate that construction started before the planned start. For example, unit 1 started 51 days before the planned start date, and was completed 91 work days after the planned start.

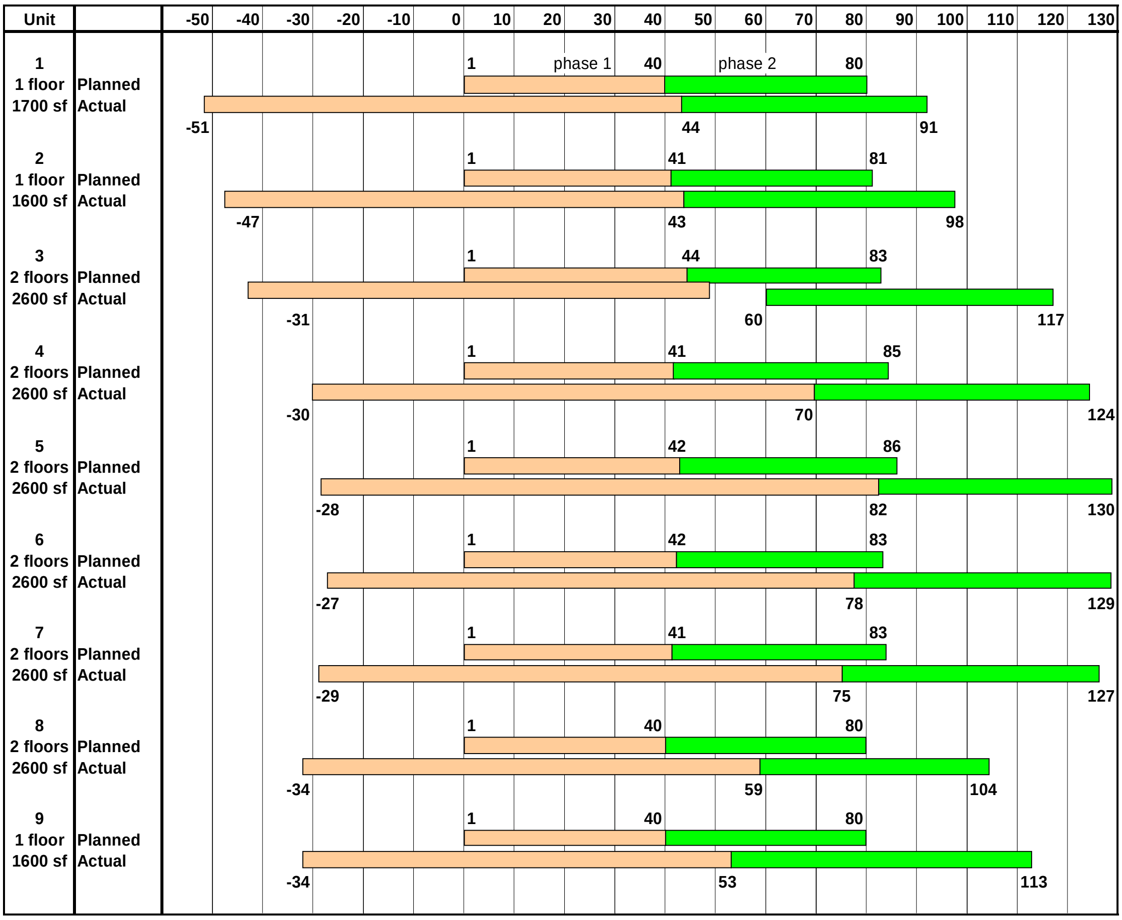

Figure 1: Planned and Actual Duration for the Nine Houses.

As shown in Figure 1, all units started several weeks earlier than originally planned. The early start was not a result of changing the priority of the particular units, but it was influenced by attempts to take advantage of crew availability in the area and "do everything possible, as soon as possible," and strong motivation to report progress. As a result, activities started ahead of time where possible. However, the gains from the early start did not result in early completion in any of the units. In phase 1, the average increase in cycle time was 57 work days (138% increase). Phase 2 started (drywall start) on average 22 days later than scheduled and had an average increase of 10 days (24%). The overall average increase in cycle time for both phases was 66 days (82%).

DELAYS BY ACTIVITY

Table 2 presents the timing of the major activities for all nine units. The numbers indicate how many work days earlier or later than the planned date the activity took place. Negative numbers indicate ahead of schedule.

Table 2. Timing of Major Activities (Days earlier/later than Baseline Schedule)

		Unit									
	Activity	1	2	3	4	5	6	7	8	9	Average
Phase 1	Layout house	-51	-48	-31	-30	-29	-28	-30	-34	-33	-35
	Install undergr. plumb.	-44	-45	-29	-31	-29	-30	-27	-33	-28	-33
	Pour Slab on Grade	-28	-25	-22	-26	-26	-22	-24	-31	-30	-26
	Start Framing	-22	-23	3	3	4	0	-2	-7	-3	-5
	Finish framing	-24	-23	3	0	2	-1	0	-5	-3	-6
	Rough Electrical	-17	-11	4	0	11	9	3	6	0	1
	Load roof	-13	-5	11	25	32	18	22	13	10	13
	Frame Inspection	-9	-7	12	26	29	25	23	11	9	13
	Insulation	5	4	16	29	40	36	34	19	15	22
Phase 2	Drywall start	4	6	16	29	40	36	34	19	13	22
	Drywall finish	7	12	28	38	46	43	41	25	15	28
	Stucco start	17	9	29	32	50	45	55	30	16	31
	Carpentry	4	6	22	40	46	42	40	25	19	27
	Painting start	5	7	23	39	44	40	38	24	19	27
	Electric trim start	3	4	22	35	48	43	44	38	28	29
	Plumbing trim start	3	5	28	38	44	43	39	30	24	28
	Carpet	11	-3	27	33	49	45	43	25	20	28
	CMU wall start	19	18	37	37	48	47	43	33	19	33
	CMU complete	15	24	36	46	49	60	58	34	25	39
	House Complete	11	17	34	39	44	46	44	24	33	31
	TOTAL DELAY	62	64	65	69	72	73	73	58	66	67

Delays to start activities were a major factor for increased cycle time. The activities that contributed most to the delays were the following:

- Framing: On average, framing started 5 days ahead of schedule, while the previous activity (Slab on Grade) started 26 work days ahead of schedule. Thus, it contributed a delay of 21 work days (26-5).
- Loading the roof with tile contributed a delay of 12 work days: it started 13 days later than scheduled (on average), while the previous activity (rough electrical) started 1 day later than scheduled.
- Insulation contributed a delay of 9 work days—it started 22 days late, while frame inspection started 13 days late.

- Drywall started closely after insulation (22 days late), but was completed 28 days late due to framing and drywall rework.

Clearly, the domino effect cannot be ignored in this process. Delays in the start of non-critical path activities were less of an issue (such as CMU wall delays) because of the float available. Only in one case (unit 4) CMU wall was delayed long enough to delay the completion of the house.

WEEKLY PPC

To better understand the reasons for the increased cycle time we further examined the planned and actual activities on a weekly basis. The researcher on site tracked on a weekly basis the PPC—that is, activities completed vs. activities planned for that week (based on the updated schedule). PPC is a Lean Construction metric used in the Last Planner system® (LPS®) to measure the effectiveness of the production control system [2, 3]. PPC less than 100% reflects a failure in the production planning process or in the execution of the task. The project in this study did not use the LPS® to plan and control production work. The PPC metric was only used by the researchers to measure the percent of activities completed as planned.

The weekly PPC for the nine units was recorded from the point when the phase 2 superintendent was assigned to the unit (approximately from the beginning of drywall) until the completion of the unit (except unit 9). In several cases, the superintendent of phase 2 was assigned two or more weeks after the phase had started. If an activity was completed within the week it was planned for, it was counted as 'complete,' even if it was completed later than planned (e.g. Friday instead of Tuesday). The number of work activities in any week varied from 2 to 20.

Table 3 summarizes the PPC for the nine units for the weeks tracked. The calculation of the average excludes the final week (FW) of the project, when PPC is always 100%. The 0% PPC in units 5 to 8 is due to the failure to complete the 'hang drywall' activity due to framing problems and rework in these units. As a result, subsequent activities such as stucco and drywall taping were not performed as planned.

Table 3: PPC Data Collected—Units and Weeks

	Week										
Unit	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**Avg PPC**
1	31%	79%	71%	50%	100%	31%	44%	22%	FW*		54%
2	33%	65%	75%	63%	100%	35%	74%	17%	FW		58%
3	57%	50%	61%	100%	79%	25%	14%	FW			55%
4	100%	25%	40%	38%	31%	56%	44%	38%	82%	FW	50%
5	0%	20%	53%	38%	78%	73%	43%	89%	78%	FW	52%
6	0%	22%	69%	100%	64%	50%	63%	83%	100%	FW	61%
7	0%	0%	60%	42%	53%	82%	60%	45%	85%	FW	47%
8	0%	29%	0%	86%	50%	50%	35%	100%	100%	FW	50%
9	67%	42%	67%	69%	100%						69%
										Average all units	**55%**

* FW = Final Week

The average PPC for all units tracked was 55%. This means that 45% of the work activities planned for the week were not completed within that week. It is not surprising then to have the long cycle times discussed earlier. The next section presents the reasons for activities not completed as planned.

REASONS FOR PLAN FAILURES

Every activity not completed as planned is considered a 'plan failure.' For each plan failure, the researchers identified one primary reason (for example, lack of material), which also has other deeper causes (such as management failure to order the material). The reasons for plan failures provide an understanding of the sources of uncertainty, variability and delays on the project. Table 5 summarizes the reasons for plan failures.

Table 5: Reasons for Plan Failures

Unit	Reasons for Plan Failures						Unit total
	Prerequisite	Labor	Quality	Material	Priority	Weather	
1	24	10	3	3	6	1	47
2	25	19	2	3	2	2	53
3	7	12	11	5	0	0	35
4	21	17	6	3	4	0	51
5	22	8	1	2	1	0	34
6	10	12	11	1	3	0	37
7	21	23	6	2	2	0	54
8	11	19	5	1	1	0	37
9	4	11	0	2	2	0	19
Total	144	131	42	26	21	3	367
% Total	40%	36%	12%	6%	6%	1%	100%

Pre-requisite work

Incomplete or incorrect pre-requisite work was responsible for 40% of plan failures. For example, problems in completing 'hang drywall' (a prerequisite) resulted in 'stucco' being rescheduled for the following week. Problems with labor availability and work quality were the primary reasons for the pre-requisite work not being completed as planned. More than one third of the prerequisite work failures (about 35%-40%) were due to quality problems with the prerequisite work. For example, in almost all units, drywall and exterior stucco were not performed as planned because the prerequisite work (framing and drywall) was delayed due to quality problems.

Labor availability

Labor availability was a major reason for plan failures. In these cases, the pre-requisite work was completed, but the crews either did not show up to work on the particular unit, or reduced labor was allocated and the work was not completed as planned. The most common cause of the labor problem was that the available crews were sent to work on units that were considered more urgent, often without the superintendent's knowledge. Even activities in progress were sometimes interrupted for a few days while the crew was sent to work on another house. Part of the reason was that the subcontractors had very high workload (due to the market growth) and not enough labor. Furthermore, labor availability was significantly reduced by quality problems, rework, and emergencies that disrupted the work flow. Based on the field observations, it was estimated that more than one third (33%) of labor unavailability was due to quality problems on other houses.

Quality

12% of plan failures were due to quality problems and rework that delayed the completion of the planned activity. The sources of such problems were either errors in the execution of the activity (e.g., wrong drywall texture applied) or defects in previous activities that were not corrected earlier (e.g., the poor finish of the slab on grade in some cases delayed the flooring activities, and plumbing errors caused flooding and extensive rework). Delays of one trade often caused out-of-sequence work (roof tile and exterior painting, flooring and plumbing trim) which often damaged installed work. Finally, in several cases the wrong material was installed and later removed and reinstalled. This was the result of the crews and superintendent not knowing what were the correct options for each house.

Material

About 6% of the plan failures were attributed to material problems—either the material was not ordered as planned (electric riser), or it was not delivered as expected, or it was stolen (ceramic tile). There are several causes for these problems.

- Management failures to order material on time. The division of work into two phases contributed to the late delivery and installation of the electric riser on every house—the superintendent of phase 1 was responsible for ordering the riser, but this was not done on time, as the installation is part of phase 2.
- Market factors caused shortage of material and increased lead times in the case of CMU blocks, and occasionally lumber for framing. Such problems could have been avoided if the company procured the material from other suppliers at a higher price, but the developer selected not to do so.

Change of priorities

6% of the activities not completed were tasks intentionally not performed and postponed for a later time. For example, the superintendents ordered appliances as late as possible to avoid theft, and delayed carpet installation when custom carpet was required to avoid damaging the carpet. These are not plan failures—simply the schedule was not updated to reflect the change.

Weather

The effect of weather on phase 2 was small as the exterior activities were performed in late spring/early summer. Weather had a greater effect on the phase 1 activities.

In summary, labor shortage and quality problems were the primary reasons for the plan failures and delays. The market growth contributed to labor shortage. In Arizona, employment for specialty building contractors grew from 118,500 in 2003 to 131,700 in 2004 and 150,400 in 2005—a 27% increase in 2 years. Labor shortage resulted in lower skilled workers entering the workplace as well as high workload and production pressures. On the project, these factors contributed to quality problems, out of sequence work and delayed completion of activities.

Quality problems had a major contribution to plan failures, directly and indirectly—they delayed prerequisite work (35-40% of the prerequisite work failures were due to quality problems), and increased rework of downstream work (e.g., the poor finish of slab caused flooring rework, plumbing problems caused extensive interior rework). The rework further reduced the labor availability. Based on the field observations, it was estimated that more than one third (33%) of labor unavailability was due to quality problems on other houses. This raises the question “how did the project organization manage these issues?” The next section discusses how the

production management system failed to effectively manage these factors, and further exacerbated these problems.

PRODUCTION CONTROL PROBLEMS

The organization managed to build the project, although with delays. However, there was significant amount of waste both in time (which is reflected in the increased cycle time), and resources (due to rework). To minimize waste it is important that the production control system produces reliable work assignments by ensuring that the required information, prerequisite work, resources, and labor are available so that the work can be completed as planned. On this project, the production control system was characterized by low reliability of weekly work plans and poor management of quality.

Low reliability of weekly work plans

The weekly planning was updating the schedule based on work completed. Work not completed was moved to the following week. Thus, the weekly schedule reflected the activities that 'should' be performed, but there was no consideration whether the work 'could' reliably be completed. Although the superintendent was calling subcontractors requesting to start work, the 'promises' from the subcontractors were either not made, or not kept. The allocation of labor was primarily based on the urgency of the unit.

Poor management of quality

Despite the large number of scheduled inspections, the project had a large number of quality problems which did not identify or correct early. Although the quality problems were corrected (the study did not look into defects identified after completion), they had a significant direct and indirect effect on the ability to complete the work as planned. This happened due to several weaknesses of the production control system on the project.

- Poor execution of the work. Low skill trades and production pressures contributed to many quality problems.
- Management failure to provide correct directives. Often the superintendent and crews did not know what was the correct option for each house. Combined with errors in the material deliveries, it resulted in the wrong material being installed and later removed and reinstalled (handrails, trim, cabinets).
- Poor management of the handoffs between trades. The completed work was not checked for quality. As a result, quality problems with prerequisite work were discovered after they affected downstream activities, and delayed or caused rework to several activities.
- No learning. The same problems were repeated in several houses.
- Lack of supervision. The high workload of the superintendents was a major factor in the lack of effective supervision. Phase 1 superintendents supervised 20-30 houses each, and phase 2 superintendents supervised 10-20 houses each. In this case study, the phase 2 superintendent for several units was assigned weeks after drywall had started. As a result, the superintendents spent most of their time dealing with urgent problems and had very little time to plan ahead.

Under conditions of resource (labor) constraints, even a reliable production management system would not avoid delays. Consequently, under these conditions it was critical to manage effectively the workload, the quality and the handoffs between trades, in order to prevent rework and better utilize the available labor. However, the project organization did not take any measures to prevent these problems. On the contrary, the workload of the supervisorsremained high, thus reducing the ability to manage the quality. The next section provides some brief recommendations on how the production system could have mitigated some of the identified problems.

CONCLUSIONS & RECOMMENDATIONS

The primary goal of the study was to understand the causes of increased cycle time in residential projects. This case study found significant delays and long cycle times. The case illustrates how an external condition (the market growth and subsequent labor shortage) combined with a project condition (deficiencies in production control) resulted in quality problems and delays. The situation described in this case study is not extreme or particular to this builder. The problems observed on this project are similar to other residential development projects performed under the same market conditions—increased cycle time and rework are not limited to the particular project or developer.

Implementation of Lean production practices provide an opportunity to reduce waste and add value in construction projects. The use of the Last Planner System (LPS) on this project could address the identified problems, improve the production system and reduce the waste in time and resources (Ballard and Howell 1994).

The Last Planner system (LPS) is the Lean Construction's method for production control [2,3,4]. LPS focuses on generating "quality work assignments" as the primary means to reduce variability and increase process speed and productivity [2]. The next section describes the LPS criteria of "quality assignments" and discusses how their use could prevent some of the problems identified on this project.

- Definition: the work should be specific enough to understand the requirement and completeness of the assignment. This would ensure that the crews performing the work understand the scope of work and the quality requirements.
- Soundness: All preconditions and resources needed for the activity (materials, design, prerequisites) are satisfied and available. Any item unavailable is a "constraint" that needs to be removed. Implementation of this strategy would avoid having crews working on defective prior work, as in the case of drywall crews who repeatedly installed work on defective framing.
- Size: The assignments should be correctly sized based on the capacity of the crew. This would prevent excessive workload and pressures to crews.
- Sequencing: the assignment should be sequenced in the correct constructability order. The correct work sequence prevents damages to other trades' work.

Application of these criteria would reduce quality problems and rework, improve the handoffs between the trades and make the best possible use of the available labor, primarily by not allowing effort to be spent on defective work. In turn, this would increase labor availability, and reduce the cycle time. The combination of fewer defects and increased labor availability would reduce the project cycle time.

The use of the Last Planner system could have alleviated these problems to some degree. However, the homebuilders' motivation for such changes is weak. From the financial perspective, despite the situation in production, the is a very successful company enjoying great market success, as indicated by the financial results, customer surveys, company celebrations and bonuses distributed.

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