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Housing & Building Research Center

FOREWORD

Founded in 1954, the Housing and Building Research Center (HBRC) has always been a support to the Ministry of Housing, Utilities, and Urban Communities as it presents its research and development branch.

Today, after 50 years of continuous endeavor in the field of housing and building, HBRC has taken its role as a nationwide initiator in the field of applied research. In addition, collaborations have already started with international bodies aiming at the exchange of experience between Egyptian researchers and international pioneers in the corresponding fields. Thus, on this occasion of celebrating its 50^{th} anniversary, the HBRC introduces its scientific journal entitled "**HBRC Journal**".

The HBRC journal is a multidisciplinary one that covers a diversity of scientific fields related to the construction industry. The journal, which is open to all interested researchers, will be refereed aiming to include the most recent developments, worldwide.

The ministry intends to support the HBRC journal by all means to help achieving its aims and I am confident that the HBRC journal will promptly take its position worldwide through publishing refined, high quality papers.

My deep wishes for the HBRC journal to advance quickly and keep up with the quality of published researches as seen in its first issue so as to help in the development of the construction sector in Egypt.

Minister of Housing, Utilities & Urban Communities

Prof. **Mohamed Ibrahim Soliman**

PREFACE

On the occasion of celebrating its 50^{th} anniversary, the HBRC is pleased to introduce the first issue of its scientific journal entitled "**HBRC Journal**". HBRC journal is a periodical, intended to be issued twice a year, in December and June.

The journal covers a variety of scientific branches, namely: material science, architecture, housing and building physics, structural engineering, geotechnical engineering, in addition to construction engineering and management. More than 200 papers have been received by the scientific committee of the conference entitled "Future Vision and Challenges for Urban Development", held during the period 20-22 December 2004 celebrating the same occasion. The first issue of the HBRC Journal contains selected papers from the conference, covering all the themes of the journal.

For contributions to subsequent issues of the journal, please refer to the information given in the attached "Call for Papers", in which full details have been provided.

Our main goal is to enrich the Egyptian scientific society with a refereed high quality journal to share in the scientific development of our country, Egypt. Thus, any comments are more than welcome to help us achieve our target.

Chairperson of the Advisory Board

Prof. **Omaima A. Salah El-Din**

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MECHANICAL PROPERTIES OF LOCALLY PRODUCED HYBRID FRP BARS AS CONCRETE REINFORCEMENT

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ABSTRACT: Using non-metallic fiber reinforced polymer bars as concrete reinforcement has gained an increasing interest in the last decade. FRP bars provide a promising alternative for conventional steel reinforcement, especially in harsh environment. Extensive testing is still needed to produce FRP bars with improved tensile characteristics with regard to the brittle nature of the currently available products. This research describes the process of manufacturing hybrid FRP bars using locally available glass fiber roving, nylon thread and polyester. Bars having a diameter of 9.5 mm were produced with a fiber volume fraction of 63%. The mechanical properties of the bars including tensile, compressive, shear, tensile splitting and bond strengths were experimentally evaluated by conducting the previous tests using new testing approaches. The results showed that the produced bars were much cheaper and yielded improved mechanical properties, which make them an attractive alternative as concrete reinforcement.

Keywords: GFRP, nonmetallic, bars, hybrid, nylon, testing.

INTRODUCTION

Recent research and development regarding the use of composites for reinforcing concrete structures almost involved the application of fiber reinforced polymers as nonmetallic reinforcing bars. Many researches and field applications have demonstrated the benefits of using FRP bars in specified applications that make use of their noncorrosive and nonconductive behavior [1,2]. A wider range of application and gaining more experience in using FRP bars are parallel demands that contribute to reducing the construction cost using this advanced material. However, this is restricted by potential difficulties that are mainly related to durability and fire resistance considerations. Other disadvantages of the material such as its natural anisotropy and lack of ductility have unfavorable impact on the structural behavior. Also, a lower modulus of elasticity than that of steel, as in the case of GFRP bars, can be detrimental when deflections rather than strength control the design.

The tensile behavior of an FRP bar of a specified diameter is influenced by the type, volume fraction and orientation of the reinforcing fiber(s) as well as the production process[3]. This behavior is characterized by a linear elastic constitutive law and limited elongation at failure for bars made of one type of fibers. The percentage elongation may be increased by mixing fibers of different moduli in a hybrid bar. Theoretically, a nonlinear behavior may be attained near the ultimate load due to successive spreading of damage and redistribution of load from stiff to less stiffer fibers causing degradation of the modulus prior to failure. The experimental results reported by Tamužs and Tepfers [4] showed that successful load transfer requires a uniform distribution of the fibers in the cross section. Also, the fiber-matrix adhesion that controls the load transfer between the fibers should be moderate to reduce the shock wave effect, resulting from the failure of high modulus fibers, on the adjacent low modulus fibers. It was concluded that improving the fiber-matrix adhesion by curing the bars under hot pressing vacuum along with non-uniform distribution of the mixed fibers restricted the improvement of ductility in hybrid bars.

The current research is concerned with the production of hybrid FRP bars using locally available materials and simple equipment. Previous experience with in-house manufacturing of FRP bars by applying the regular steps of molding and pultrusion techniques showed that the process was not fairly easy [5,6]. It was reported that, the fiber volume fraction in molding operations was limited to 25-38 percent, while this ratio increased to 60 percent by applying the pultrusion technique, which improved greatly the tensile characteristics.

RESEARCH SIGNIFICANCE

The cost effectiveness of FRP using carbon fibers as reinforcement is questionable with regard to the high cost of the fibers. Despite the unique high stiffness provided, designers have to be concerned about brittleness and impact resistance. On the other hand, E-glass fibers and polyester are commonly used in producing commercially available FRP bars because of the appealing combination of cost, processability and performance. However, the lower modulus of GFRP bars compared to steel and lack of ductility are yet major deficiencies. Therefore, innovative bar design considering the shape and fiber choice and hybridization with other fibers is urgently needed. Also, adopting reliable and reproducible test methods is needed for conducting quality control tests as well as design purposes.

Based on the above needs, the current research efforts were devoted to: (1) Producing hybrid FRP bars using polyester, glass fiber roving and nylon thread. The bars were expected to achieve better ductility, while tensile strength and modulus would not be significantly changed due to the introduction of the nylon thread with regard to its excellent mechanical properties in terms of strength, modulus and toughness. (2) Evaluating the short term mechanical properties of the produced bars in terms of tensile, compressive, shear, tensile splitting and bond strengths to demonstrate their efficiency.

MECHANICAL PROPERTIES: SATE OF THE ART

This section summarized briefly the basic findings concerning the evaluation of the short-term mechanical properties of FRP bars, focusing on test configurations and potential difficulties in performing the tests. .

Tensile Behavior

The design of a suitable gripping mechanism for testing GFRP bars in tension have represented a challenge due to local stress concentrations near the grips. With regard to the low transverse strength of the FRP bars, premature failures within the grip zone are more likely to occur. A failure away from the grips seems to be the clear experimental indication that the full strength of the tested bar is achieved [7]. The technical literature showed that several types of anchors have been used. According to ASTM D3916 test method [8], aluminum tab grips with sandblasted circular surfaces are used. The gripping system suggested by ACI 440K-99 [9] used epoxy filled steel pipes with an internal diameter at least 10-14 mm greater than the bar diameter and a gripping length not less than the greater of f_u / 350 or 250 mm, where f_u is the ultimate tensile force developed in the bar. The gripping clamp type anchor [10,11] consists of two or four blocks held together by prestressing bolts to apply a specified level of lateral clamping pressure. Handling this type of anchor is difficult and requires high precision and experience and considered impractical for routine tensile testing [12]. A modified wedge-bond anchor consists of a steel tube with an inner thread. The steel tube is gripped at the end by steel wedges in a conical steel housing. Pull-out of the FRP bar from the steel pipe was reported due to lack of confinement [12]. A trial has been reported to improve the transverse strength by wrapping the ends of the FRP bar using carbon fiber laminate providing lateral confinement, yet the test bars failed in the gripping zone [5].

According to the ACI 440K-99 test procedures, the modulus of elasticity is calculated from the stress-strain curve values at 20 and 60 percent of the tensile strength. The ultimate strain may be determined from the test if the strain gage measurements are available up to failure, otherwise, it may be extrapolated assuming a linear stress-strain relationship up to failure.

Compressive Behavior

Tests conducted by Wu [3] on FRP bars having a length to diameter ratios varying from 1:1 to 2:1 showed that the compressive strength was in the range of 317 to 470 MPa for GFRP bars having a tensile strength in the range of 552 to 896 MPa. The compressive strength increased as the tensile strength increased and the mode of failure patterns included transverse tensile splitting, fiber micro-buckling and shear failure. The test results also showed that the compressive modulus of elasticity was about 80 percent of the tensile modulus for bars containing 55 to 60 percent fiber volume fractions. Gripping and aligning procedures as well as premature failure resulting from end brooming were reported as potential difficulties that influence the accuracy of determining the compressive strength and modulus [1,2].

Bond Behavior

Critical design parameters, such as development length and transfer length, depend directly on the bond between concrete and reinforcement. Also, deflections and cracking width and spacing are dependent on bond. The test methods reported in the technical literature included cantilever beam, spliced reinforcement beam, notched beam, hinged beam and the frequently used direct pull-out test [6]. Unlike reinforcing steel, the bond of FRP bars appears not to be significantly influenced by the concrete compressive strength [7,8]. The experimental work conducted by Nanni et al. [7] showed that the bond of FRP bars tended to be controlled by the mechanical action due to surface treatment of the bar surface rather than adhesion and friction. It was also demonstrated that specimens with a smaller bar diameter of 6.3 mm developed higher bond strength compared to similar bars of a 12.7-mm diameter. Also, a higher bond strength was achieved by a shorter embedded length of *5D* compared to a longer length of *10D*, where *D* is the bar diameter.

MANUFACTURING PROCESS

Materials

Resin

Isophthalic polyester belonging to the Vipel ® F737 series resins produced by AOC, USA was used. The resin is suitable for various fabrication processes such as hand lay up, winding and pultrusion and extensively used in fabricating water pipes and other corrosion resistance applications. The resin has excellent mechanical properties in terms of tensile strength (86 MPa), tensile modulus (3.4 GPa) and tensile elongation (4.0%) according to ASTM D638 test method and a specific gravity of 1.12 as reported by the manufacturer.

Fibers

Glass fiber roving formed from continuous untwisted strands that are bonded together with a polyester-compatible size was used. According to the manufacturer, the fiber combines the mechanical properties of traditional E-glass and the acid corrosion resistance of ECR-glass. The glass fiber, produced by CAMELYAF, Turkey has a specific gravity of 2.54, A Tex of 2400, tensile strength of 3250 MPa, tensile modulus of 69 GPa and tensile elongation of 4.5% according to the manufacturer's data sheet.

A lower modulus, yet highly ductile nylon thread obtained from the workshops of the Suez Canal Authority, Egypt was used in manufacturing the hybrid bars. Twisted strands of this thread are used in manufacturing durable tying ropes. The thread has improved mechanical properties to suit this application by adding 30% by weight of fiberglass reinforcement to the polymer to increase the tensile strength and stiffness as well as impact and chemical resistance. The thread has a nominal diameter of 0.35 mm, specific gravity of 1.4, tensile strength of 650 MPa, elastic modulus of 9 GPa, and an elongation of 14% as reported in the product data sheet.

Manufacturing

Bars having a length of 2.00 m and a nominal diameter of 9.5 mm were produced using nylon fiber fractions of 0, 5,10 and 20 percent of the total volume of reinforcing fibers. The glass roving had a cross sectional area of 0.945 mm^2 (=Tex /1000 ρ_f). A number of 40 roving bundles were needed to produce a bar with the specified diameter. In hybrid bars, one glass roving was replaced with 10 nylon fibers that yielded the same cross sectional area. The production process goes through three stages as described in the following:

Stage (I): continuous fibers are tensioned between two hooks that are initially 2.20 m apart. While one hook is fixed the other is movable and connected to a handle, Fig. (1). The movable arm is pushed in, so that the sagging fibers are wholly laid in a 2.0 m long half tube basin containing 200 gm of polyester. After the fibers are thoroughly wet out, the arm is pulled out and rotated to twist the fibers and squeeze out extra resin. The process continues until the clear distance between the hooks is 2.0 meters (at this distance the fibers are twisted at an off-axis angle of 30 degrees). A slight tension force is applied to prevent sagging of the bar under its own weight and the movable arm is then locked using a piece of wood. Peroxide dose of 0.5% by weight causes the resin to acquire sufficient strength within 20 minutes, after which the bar is cut at the ends using a saw.

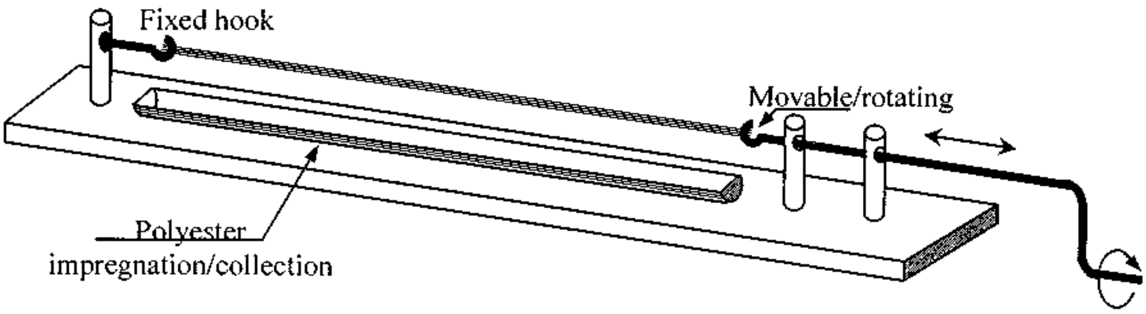

Figure 1. Sketch for the twisting equipment showing end hooks and polyester basin

Stage (II): the bar surface is covered with a thin layer of polyester and soaked in sand with particle size 0.7-1.2 mm. Sand is applied as a surface treatment to improve the bond strength.
Stage (III): after about 20 minutes, a thin layer of polyester is applied to improve the adhesion of sand particles to the bar. This final layer was also found to improve the end quality for saw-cut bars. The average diameter of sand coated bar was 12 mm.

The control bar (zero nylon fiber content) had a weight of 132.5 gm/m'. It was concluded that the fiber weight fraction was 80 percent as the weight of fibers in the bar was 211.2 gm. The corresponding fiber volume fraction was estimated to be 63 percent based on the reported specific gravity values.

SHORT TERM MECHANICAL TESTS

The short term mechanical properties of the produced bars including tensile, compressive, bond, shear and tensile splitting strengths were evaluated. The tests were conducted utilizing a 500 kN universal testing machine. A maximum loading range of 50 kN was set, which provided a measuring accuracy of 0.05 kN. A gradually increasing load at a rate of 5 kN/minute was applied in all tests. All the reported strength results represent the average of three tested specimens.

Tensile Test

The use of conventional grooved and teethed steel clamps for testing unprotected FRP bars have been shown to be completely unsuitable for this purpose. However, the excellent results obtained in the current work for the transverse strength encouraged a trial to use these grips to test the produced bars. The grooved grips provided a gripping length of 100 mm. After mounting the test specimen that had a total length of 576 mm (*40D*+200 mm), a 3-point support differential transformer type extensometer was attached at mid-height of the test specimen. Strain measurements were recorded up to 70 percent of the ultimate load. At this loading level, the extensometer was detached and the test was continued till failure. A total of 27 tests were conducted. Following the recommendations of the ACI 440k-99 report, The results of 15 tests were disregarded due to failure near the gripping zone. Thus, the results reported in Table (1) for the tensile strength and modulus of elasticity, corresponding to different nylon fiber contents (0, 5, 10 and 20 %), are the average of results obtained from three successful tests. Strain measurements showed that the stress-strain relationship was fully elastic. The ultimate strain was extrapolated for the control bar. Failure was sudden, noisy and explosive in nature. For this reason, safety precautions should be followed strictly by the testing personnel.

To avoid the common problems due to gripping of the test specimen, a flexural test specimen was developed to load the FRP in tension. The specimen (40x100x400 mm) consists of two blocks (40x100x198) made of a polymer mortar consisting of one part of polyester and two parts of sand by weight. The two blocks are connected by an FRP bar near the bottom of the specimen and a high tensile steel bar (15 mm in diameter) located near the top. The two bars were located so that the distance between their centerlines was 75 mm. The bars extended along the whole length of the specimen to provided maximum resistance against slip. The specimens were manufactured using a wooden form consisting of two cells separated by 4-mm thick plywood provided with two holes to pass reinforcing bars. Fig. (2) shows the test set up and dimensions of the test specimen. The specimens were tested in flexure under four-point loading over a clear span of 300 mm. Two concentrated loads, 50 mm apart, were gradually applied up to failure. According to the dimensions of test configuration, the tensile force in the FRP bar was equal to the applied load. The bar strain was measured by a demountable extensometer having a gage length of 50 mm and demec points fixed to both sides of the specimen at the same level of the FRP bar. The average strain was recorded at load intervals of 2 kN up to failure that occurred suddenly, but gently and thus it was possible for the testing personnel to follow up measuring the strain up to failure.

Table 1. Mechanical properties addressing the tensile behavior

Mechanical Properties	*Test*	*Nylon Fiber Content, %*			
		0	5	10	20
Tensile strength, MPa	*axial tension*	735	665	590	550
	flexural	750	680	630	580
Modulus of elasticity, GPa	*axial tension*	44.70	41.30	38.45	34.20
	flexural	40.23	38.89	34.51	28.20
Ultimate strain ($x10^{-3}$)	*axial tension*	16.44	N. A.	N. A.	N. A.
	flexural	19.2	21	40	58

In order to obtain a consistent stress-strain diagram, the following requirements should be satisfied during the whole course of testing (i) no slip should occur between the bar and the holding block, (ii) the holding blocks should be free from potential damage, such as cracking, that causes a drop in the applied load and (iii) the compression bar remains elastic. The tests showed that the first two requirements were perfectly satisfied, while the third was guaranteed as the cross sectional area of the compression steel bar was 2.5 times greater than that of the FRP bar.

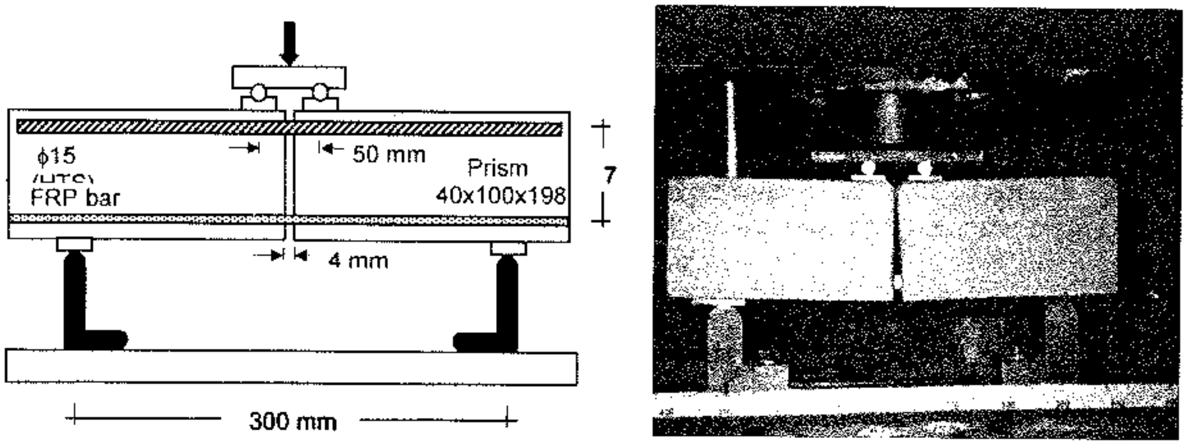

Figure 2. Test set-up and specimen at rupture of FRP bar

The results reported in Table (1) shows that the tensile strength obtained in the flexural test were 2-7 percent higher than the corresponding results in the axial tensile tests. On the other hand, the modulus of elasticity obtained from the flexural tests was 82-94 percent of its value determined from the axial tensile tests. Based on the axial tensile test results, it can concluded that the use of nylon fibers contents of 5, 10 and 20 percent reduced the tensile strength by 10, 20 and 25 percent, respectively. The corresponding ratios for the reduction of the modulus of elasticity were 8, 14, and 23 percent. Based on these figures, it can be concluded that the proposed flexural test can be used safely to estimate the tensile characteristics in terms of strength and modulus of elasticity. Fig. (3) shows the stress-strain curves up to ultimate loads. Each data point on the curves represents the average of three test results. It can be seen that the brittle nature of control bars did not significantly alter due to the use of 5 percent nylon fibers and the increase in the ultimate strain was limited to 10 percent. As the nylon fiber content was increased to 10 and 20 percent, the ultimate strain increased effectively to be 2 and 3 times the control value, respectively. Based on these results the use of nylon fiber content of 10 percent of the total fiber volume is recommended due to the moderate reductions in both the tensile strength and elastic modulus and the improved ductility.

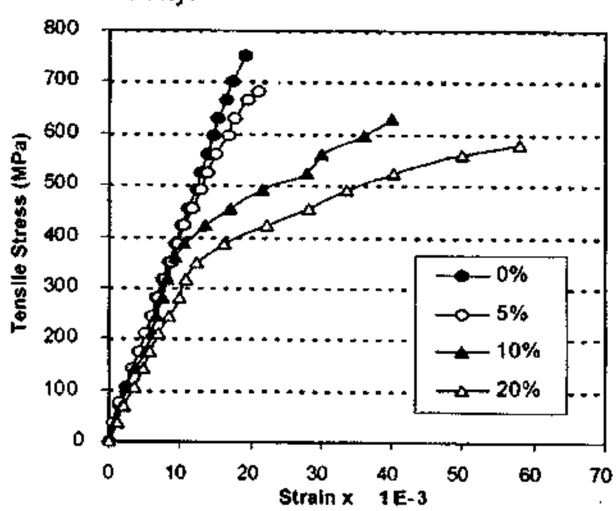

Figure 3. Tensile stress-strain relationship for FRP bars with different nylon fiber contents

Compression Test

FRP reinforcement should not be relied upon to resist compression because of the limited contribution of the bars at concrete crushing [1]. However, compression tests were performed to characterize fully the behavior of the produced bars and the influence of the nylon fiber. Two sets of specimens were prepared for testing in compression. The specimen length to diameter ratio, *h/D*, was one in the first set and two in the second. Testing problems due to gripping, alignment and end brooming of the test specimen were solved by casting two bearing blocks that confined fully the bar ends. The test specimen having a total length of 50 mm was vertically fixed to the base of a 50-mm side cubic steel mould. A mortar consisting of equal parts by weight of polyester and sand was poured to a thickness of 15.5 mm (*h/D*=2) or 20 mm (*h/D*=1). The mortar acquired sufficient strength within 20 minutes after which the mould was stripped and the second block was cast. The specimens were tested in the compression test the jig shown in Fig. (4) with a fixed lower platen, while the upper is connected to a spherical seating.

Table 2. Short-term mechanical properties

TEST		*Nylon Fiber Content, %*			
		0	*5*	*10*	*20*
Compression	*h/D*= 2	185	206	184	149
	h/D= 1	274	285	275	169
Transverse tensile		60	68	66	62
Shear		209	215	205	199

The results reported in Table (2) shows that the compressive strength was not significantly influenced by nylon fiber contents up to 10 percent. Increasing the nylon fiber content to 20 percent reduced the compressive strength by 19 % for *h/D*=2 and 38 % for *h/D*=1. For nylon fiber contents up to 10 percent, the compressive strength varied from 25% to 31% (*h/D*=2) and from 37% to 47% (*h/D*=1) of the corresponding tensile strength determined in axial tensile tests. These figures shows that the compressive strength of the produced bars, expressed as a fraction of the corresponding tensile strength, is satisfactory and lies within the limits reported in the technical literature. The tests showed that most of the specimens failed in shear due to the off-axis alignment of the fibers, while a few specimens failed due to fiber micro-buckling.

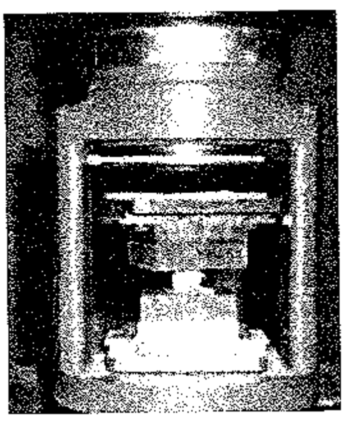

Figure 4. Set-up of compression test

Bond test

Push-out tests were performed using the specimens shown in Fig. (5). This configuration for bond testing is known to give higher bond strength compared to pull-out tests as both concrete and reinforcement are compressed. Actually, both push-out tests and the frequently used pull-out tests do not realistically simulate the stress state in flexural cracked members. However, the proposed push-out test has the advantages of robustness, avoiding gripping problems and the possibility of conducting the test in a compression test machine.

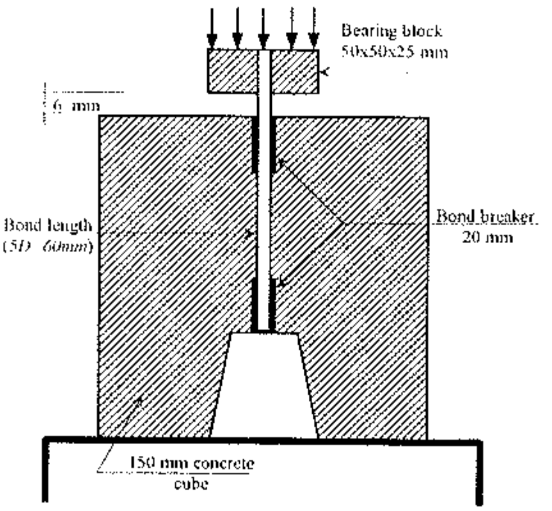

Figure 5. Dimensions of test specimen

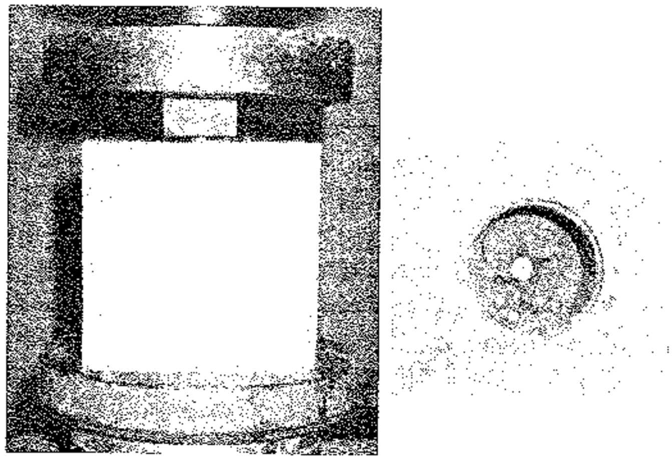

Figure 6. Penetration of FRP bar through concrete

The produced FRP bras with zero nylon fiber content and deformed steel bars of the same diameter (12 mm) were tested to evaluate the bond strength. Testing the bond of steel reinforcement provided a benchmark for judging the capacity of the proposed surface treatment of the produced FRP bars. This means that the proposed test was intended to provide comparative results rather than absolute conservative results that can be applied directly into design. Designing the test specimen to determine a suitable embedded length was based on assuming a uniform bond stress distribution. For an embedded length of *5D*, the ultimate bond stress is limited to $\sigma/20$, where σ is the compressive strength determined for a specified free loaded length to diameter ratio. This means that a bond strength higher than 13.7 MPa should not be expected based on the compressive test results reported in Table (2) for *h/D* ratio of 1:1. However, this limit was guaranteed by utilizing an *h/D* ratio of 0.5:1 as the free loaded bar length was 6 mm as shown in Fig. (5).

A normal strength concrete mix yielding a 28-day compressive strength of 29 MPa was used in casting 150 mm cubes. A 70-mm height plastic cap was fixed to the base of the steel mould, then the reinforcement bar with a total length of 130 mm was fixed vertically to the top of the plastic cap. 20 mm bond breakers were provided, so that the embedded length was five times the bar diameter. The mold was stripped 24 hours after casting the concrete and a bearing block was cast using a polymeric mortar to prevent end brooming of FRP bars. A clear bar height of 6 mm, which was sufficient to cause significant slip, was left between the block and the upper surface of the concrete cube. The test specimens were cured until tested at 28 days after removing the plastic cap allowing the unloaded end to go down freely. Fig. (6) shows the test set-up and the penetration of the unloaded bar end.

Three test specimens were tested for each reinforcement type. The average test results showed that the sand coated FRP control bars developed bond strength of 14.56 MPa versus a strength of 6.6 MPa developed by steel. This result demonstrated the efficiency of the continuous sand coating in developing a superior bond resistance in comparison with the discrete ribs in conventional steel reinforcement.

Transverse Tensile Test

The transverse tensile strength of the produced bars was determined by performing splitting tensile strength using the compression test jig. The sand cover was removed along bar specimens that were 50 mm in length. The specimens were laid horizontally and loaded uniformly, Fig. (7). The tests showed that the specimens were highly compressible, as they continued to deform, while the load continued to increase without reaching a peak. Therefore, a criterion for ending the test had to be specified. The test was ended when the bar diameter was compressed to 80 percent of its initial value. The corresponding average loads are reported in Table (2). Obviously, it was not convenient to compute splitting tensile strengths, as the specimens did not actually split. However, transverse stresses, computed by dividing twice the recorded loads by the loaded perimeter, were reported to demonstrate the behavior in transverse loading in comparison with other results reported in the literature. The results demonstrated the role of the off-axis fibers in carrying out considerable amount of transverse deformations without developing noticeable cracking. Independent of the nylon fiber content, the transverse strength was about 9 percent of the axial tensile strength of the control bar.

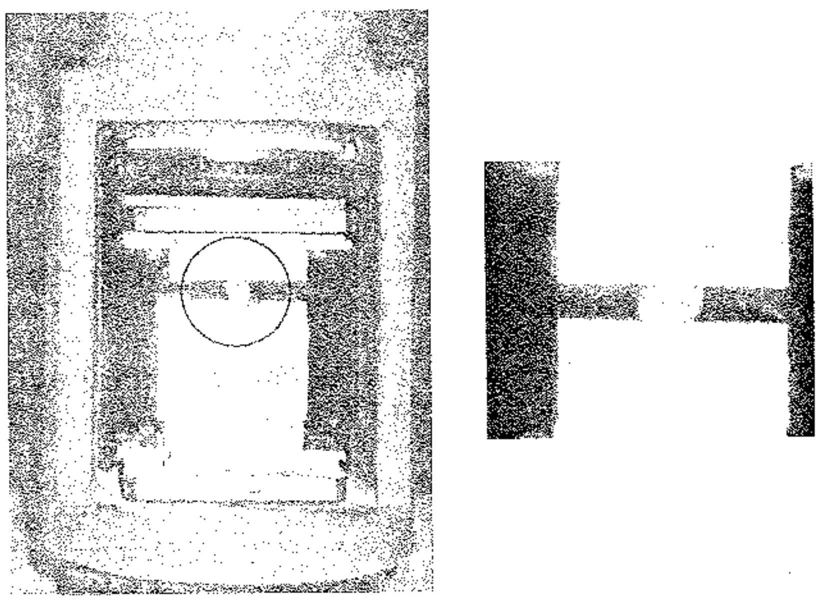

Figure 7. Test set-up for transverse loading and tested specimen at failure

Shear Test

The quality of fiber-matrix adhesion can be well reflected by the results of shear tests. Also, the performance of the bar at bent portions and crack intersections is influenced by its shear resistance due to the combination of tensile and shear stresses at these locations. Tests have been performed on 12 samples having 30-mm length. Two steel prisms (30x30x18 mm and 30x30x10 mm) provided with a hole (13 mm in diameter) across the thickness were used to load the bar in transverse shear as shown in Fig. (8). The prisms were mounted 2-mm apart in a shear test. The results reported in Table (2) represent the average of 3 specimens and show that the shear strength was about 28 percent of the axial tensile strength of the control bar independent of the nylon fiber content.

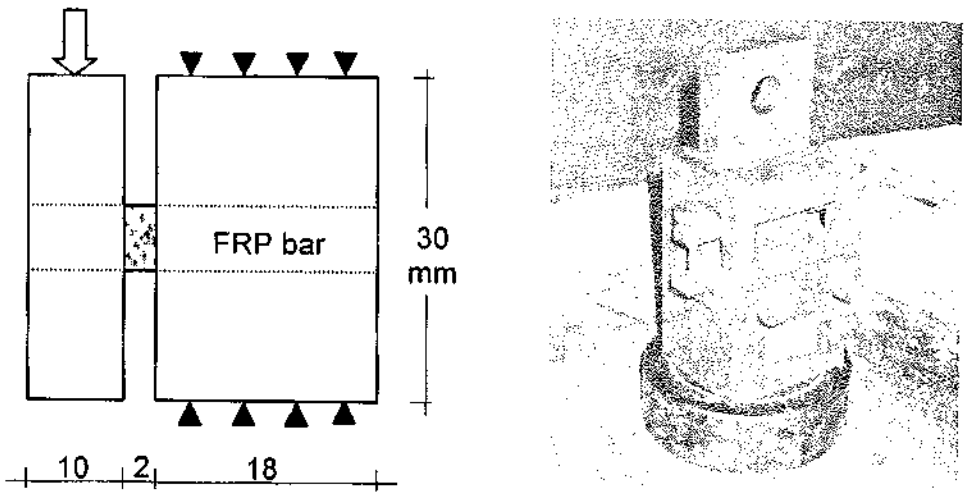

Figure 8. Shear test set-up and dimensions of loading steel prisms

CONCLUSIONS

The current research studied the feasibility of producing hybrid FRP bars with an improved ductility. Glass fiber roving and polyester were used to manufacture the control bars. Different volume fractions (5, 10, and 20 percent) of the glass fibers were replaced by nylon fibers, keeping the total fiber volume fraction fixed at 63 percent. Efforts have been made to develop new approaches for testing the short-term mechanical properties to overcome certain problems related to gripping and alignment of test specimens. Based on the reported results, the following conclusions can be summarized:

(1) The proposed technique for manufacturing FRP rods is fairly simple and the mechanical properties of the produced bars are comparable to those reported in the literature. Control bars had a tensile strength of 735 MPa, modulus of elasticity of 44.7 GPa and an ultimate strain of 1.6%.

(2) Sand coating was efficient in developing higher bond strength compared to conventional steel with discrete ribs.

(3) Replacing 10 percent of the glass fibers by an equal volume of nylon fibers is recommended to attain an effective ductility improvement, while the tensile strength and modulus of elasticity are decreased moderately.

(4) Nylon fiber contents up to 10 percent had no significant effect on the compression, shear and transverse tensile strengths of the produced hybrid FRP bars.

(5) Compared to axial tensile tests, the proposed flexural test provided a safe estimate for the tensile strength and modulus of elasticity. Common gripping problems were avoided and strain measurements could be tracked so easily up to failure as the rupture of the bar was not explosive.

(6) The proposed configuration for a push-out test to evaluate the bond resistance was successful. However, it is not recommended to use the results of the test for direct design application. The results may better be used to explore the efficiency of different surface treatment configuration.

ACKNOWLEDGMENTS

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DURABILITY OF AIR- COOLED SLAG CEMENT PASTES IN SEAWATER

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ABSTRACT: In the operation of a blast- furnace the iron oxide ore is reduced by means of cock to metallic iron, while the silica and alumina constituents combine with the lime and magnesia to form a molten slag which collects on the top of the molten iron. When the slag is allowed to cool slowly, it solidifies into a crystalline material known as "air-cooled slag". This slag has no or very little, cementing properties. On the other side, when the blast – furnace slag is cooled rapidly by quenching with water it is called "granulated slag". It is mainly composed of amorphous glassy material and can be used in blended cements. The aim of the present work is to study the hydration characteristics of air-cooled slag as pozzolana in blended cements and the durability of cement pastes in seawater. The results revealed that the air cooled slag showed a hydraulic activity as seen from the results of the combined water, free lime contents and compressive strength of hardened cement pastes. Also, the air-cooled blended cement pastes have a good resistance in seawater with comparison of sulphate resistant cement pastes up to one year.

Keywords: Blast-furnace, Slag, Crystalline, Amorphous, Aggregates, Pozzolana, Combined water.

INTRODUCTION

Blast-Furnace slag issues from the blast- furnace as a molten stream at temperature of 1400-1500°C which collects on the top of the molten iron. When the slag is cooled slowly, it solidifies into a crystalline material known as "air-cooled slag" which has no or very little cementing properties. When the slag is colled rapidly by quenching with water it is called "granulated slag" It is amorphous and can be used in blended cements. The air-cooled slag is a crystalline material containing gehlenite (C_2AS) and akermanite (C_2MS_2) in addition to β-dicalcium silicate (β-C_2S) [1].

The hydration of Portland slag cement is more complicated than that of Portland cement. No hydration products can be observed when ground slag is placed in water due to the formation of acidic films as small amount of ca++ ions is released into solution. In a calcium hydroxide solution, the hydration reaction occurs removing this film and continued hydration takes place [2, 3].

The hydraulic activity of slag in the presence of Portland cement and $Ca(OH)_2$ was studied [4]. The hydraulic reactivity was higher for the slag cement mixture than for the slag-lime mixture indicating those Ca++ ions during cement hydration reaction was necessary and effective to improve the hydraulic reactivity of slag.

Mostafa et al [5] investigated the hydraulic reactivity of air-cooled and granulated slag. The air-cooled slag exhibited significant reactivity in comparison with granulated slag at room temperature.

Regourd [6] investigated the resistance of Portland slag cements in seawater. Since granulated slags are vitreous products and more siliceous than portland cements, their presence increases the resistance of Portland cement to chemical attack.

In blended cements the presence of slag, fly ash or fillers improves the resistance to seawater [7, 8].

The aim of the present work is to study the hydraulic activity of air- cooled slag as blended cement and the durability of its cement in seawater in comparison to sulphate resisting cement (SRC) up to one year.

EXPERIMENTAL

Materials

The materials used in this investigation were ordinary Portland cement clinker (OPC), air-cooled (AC) slag, and sulphate resisting cement (SRC) provided from Helwan Cement Company, Helwan, Egypt. The chemical analysis and surface area each starting material is given in Table (1). All of the starting materials were ground separately using a aboratory small steel ball mill having a capacity of five kilograms. The mix composition of the prepared cements is seen in Table (2).

Table 1. Chemical analysis of starting materials, wt %.

Oxides (%)	Starting Materials			
	OPC Clinker	AC Slag	SRC	Gypsum
CaO	65.0	35.57	63.57	41.27
SiO_2	21.6	32.69	21.79	5.95
Al_2O_3	5.00	8.17	3.97	0.64
Fe_2O_3	3.05	8.79	4.98	0.09
MgO	2.17	2.70	2.11	0.77
SO_3	0.73	0.50	1.94	26.93
K_2O	0.12	0.22	0.11	0.02
Na_2O	0.33	0.93	0.31	0.32
Cl^-	0.01	0.03	0.01	0.25
L.O.I	1.46	4.42	0.49	23.88
Surface Area (cm^2/g)	3240	3260	3300	3820

Table 2. Mix composition of blended cements; (Wt, %).

Mix No.	Starting Materials (%)		
	OPC Clinker	A C Slag	Gypsum
OPC	95	0	5
A_1	85	10	5
A_2	75	20	5
A_3	65	30	5
A_4	55	40	5

Preparation of cement paste

A certain weight of cement was placed on a smooth, non-absorbent surface and a crater was formed in the center. The required amount of mixing water was poured into the crater by aid of trowel. the dry cement was troweled over the remaining mixture for about three minutes. the fresh cement paste was moulded into cubic inch specimens; the specimens with their moulds were cured in humidity chamber at constant temperature 23 + 2 °C for 24 hours, then demoulded immersed under tap water until the required time of testing of 3,7,28 and 90 days.

Preparation of mortars

Mixing was carried out on the cement powder and sand with a ratio of 1:2.75 and a water/cement ratio (O.485) by weight according to ASTM-C:109-93.

Methods of Investigation

Water of consistency and setting time : The water of consistency, initial and final setting times of the blended cements were determined using a Vicat Apparatus[9,10]. The quantity of water of consistency will be that required to give a paste which permitted the settlement of the plunger to a point 5 to 7 mm from the bottom of the vicat mould. The needle of initial setting time lowered gently into contact with the surface of the test block then released, and allowed sinking into the paste replaced the plunger. Process would be repeated until the needle did not pierce it by about 5-7 mm from the bottom. The period between the time of mixing with water and the time at which the needle ceased to pierce the test block, would be the initial setting time.

For the determination of the final setting time, the initial setting needle of the Vicat Apparatus was replaced by the needle with an annular attachment. The paste would be considered as finally set when, upon applying the needle gently to the surface of the test block. The needle made an impression thereon, while the attachment failed to do so. The time interval from the addition of water to the mix and impression is the final setting time.

Stopping of the hydration The stopping of hydration is made as described elsewhere[11] . The stopping solution was prepared as a 1:1 mixture by volume of methyl alcohol and acetone. A representative sample (10 g) from hardened cement paste was ground in a porcelain mortar under the surface of the stopping solution, then, filtered through sintered glass furnnel (G.4). Washing of the contents of the funnel was carried out three times with the stopping solution, finally with 50 ml of fresh ethyl ether and dried at 70°C in the drier.

Determination of the combined water content (Wn, %): Two representative samples of the dried specimens, about 2g each, were weighed in platinum crucibles, and ignited for 0.25 hour at 850°C in a muffle furnace cooled in a desiccator then weighed. The combined water content was the ignition loss on the ignited weight basis.

$$Wn = \frac{[W_1 - W_2]}{W_2}$$

Determination of the free lime content :The free lime was determined as described elsewhere [12]

Resistance against attack of seawater :The hardened cement pastes were tested for their resistance against attack of seawater. The seawater resistance of the hardened cement pastes is assessed by determining the changes in compressive strength, total chloride, total sulphate, combined water and free lime contents for immersed samples up to one year. A commercially produced sulphate resisting cement sample was tested in comparison with blended cement samples such as OPC, A_1, A_3 , A_4, just for comparison. One-inch cubic samples of the cement pastes were cured first in tap water for 28 days (zero time); then immersed in seawater as aggressive media after one, 3,6,9 and 12 months. The concentration of chloride ion and TDS in seawater were 2.34% ,0.27 and 46320 ppm respectively. The seawater from Suez gulf as aggressive media was renewed every month with fresh seawater.

RESULTS AND DISCUSSION

Hydration Characteristics

Water of consistency and setting time : The water of consistency as well as the initial and final setting times of cement pastes are plotted in the Fig. (1). It can observed that the water of consistency of air-cooled slag cement pastes is slightly higher than that of Portland cement; this may be due to the water requirement of cement with the increase of gypsum content due to the formation of sulphoaluminate hydrates (ettringite or mono-sulphate). The initial and final setting times of the air-cooled slag cement pastes are longer than that of ordinary Portland cement pastes. This result is mainly due to the replacement of Portland cement clinker content with air-cooled slag. The dilution of clinker with air-cooled slag leads to delaying of rapid hydration. The air-cooled slag does not take part in the initial reaction to the same extent as clinker does and performs as a diluent action in the system [(13)].

The initial and final setting times are increased with air-cooled slag content of the cement blend from (A_1 to A_4). This is mainly due to the presence of SO_3 more than the required amount used in the reaction with C_3A in addition to the relatively low hydraulic activity of air-cooled slag. The remaining unreacted SO_3 may retard the hydration reaction of the clinker fraction of the blended cement pastes.

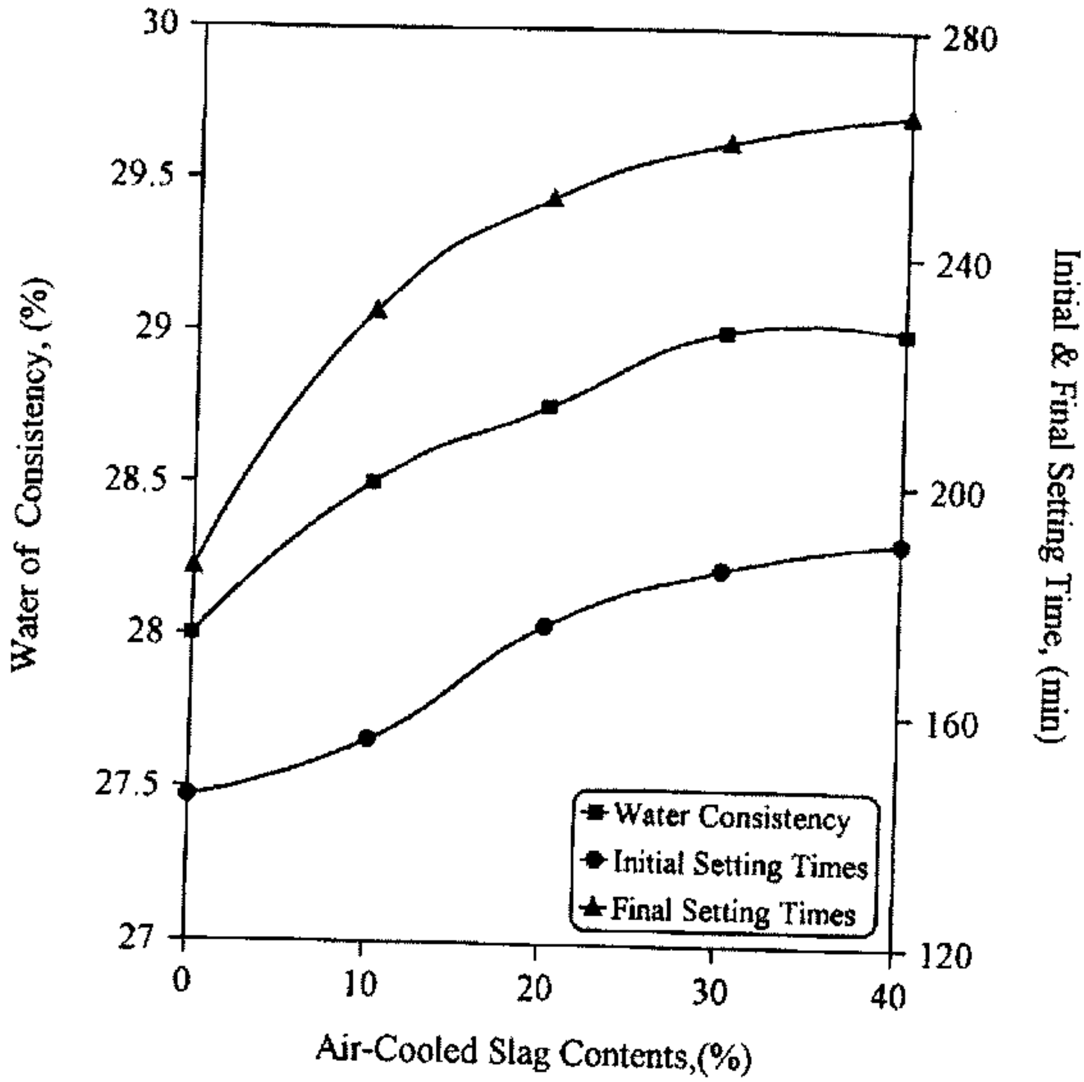

Fig.(1): Water of Consistency, Initial and Final Setting Times of Ordinary Portland Cement Pastes Made with Various Proportions from Air-Cooled Slag

Chemically-combined water contents: The data representing the combined water contents (Wn,%), for the air-cooled slag cement blends as well as ordinary Portland cement pastes hydrated for 3.7,28 and 90 days are plotted in Fig.(2) .Evidently, the combined water content increases gradually with curing time for all cement pastes due to the progress of hydration. In addition, the combined water content of blended cement pastes decreases with air-cooled slag content from (A_1 to A_4) at each hydration age. Hence, the retardation of the hydration reaction of these cement pastes takes place and consequently the combined water contents (Wn) decreases. The hardened Portland cement paste has higher Wn-values than those made of the slag cement blends (A_1-A_4). The clinker constituents have a higher contribution to the hydraulic properties as compared with air-cooled slag. Therefore, a rapid hydration reaction of clinker occurs as compared with air-cooled slag especially at early ages of hydration (3-days). The combined water contents are relatively high for Portland cement due to the formation of calcium sulpho-aluminate hydrates, where their amount decreases with slag content in the slag-cement blends (A_1-A_4).

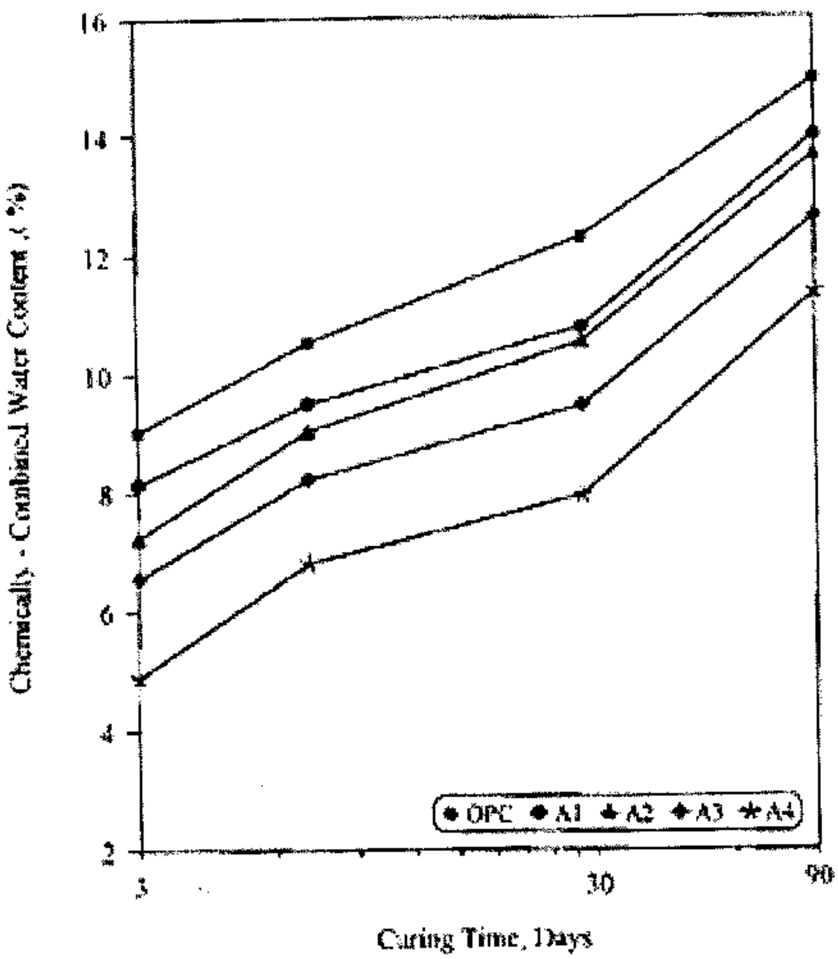

Fig.(2) : Variation of Chemically- Combined Water Contents of OPC Pastes Made with Various Proportions of Air-Cooled Slag with Time of Hydration

Free lime content :The free lime contents of the hydrated air-cooled slag cement pastes as well as the Portland cement pastes are represented in Fig.(3). These results show that the free lime content increases for all cement pastes with curing time up to 90 days and decreases with slag content. Therefore the decrease of free lime content for the slag cement blends is due to the less reactivity of air-cooled slag grains which may also consume a little amount of the free lime, produced from the hydration of Portland cement.

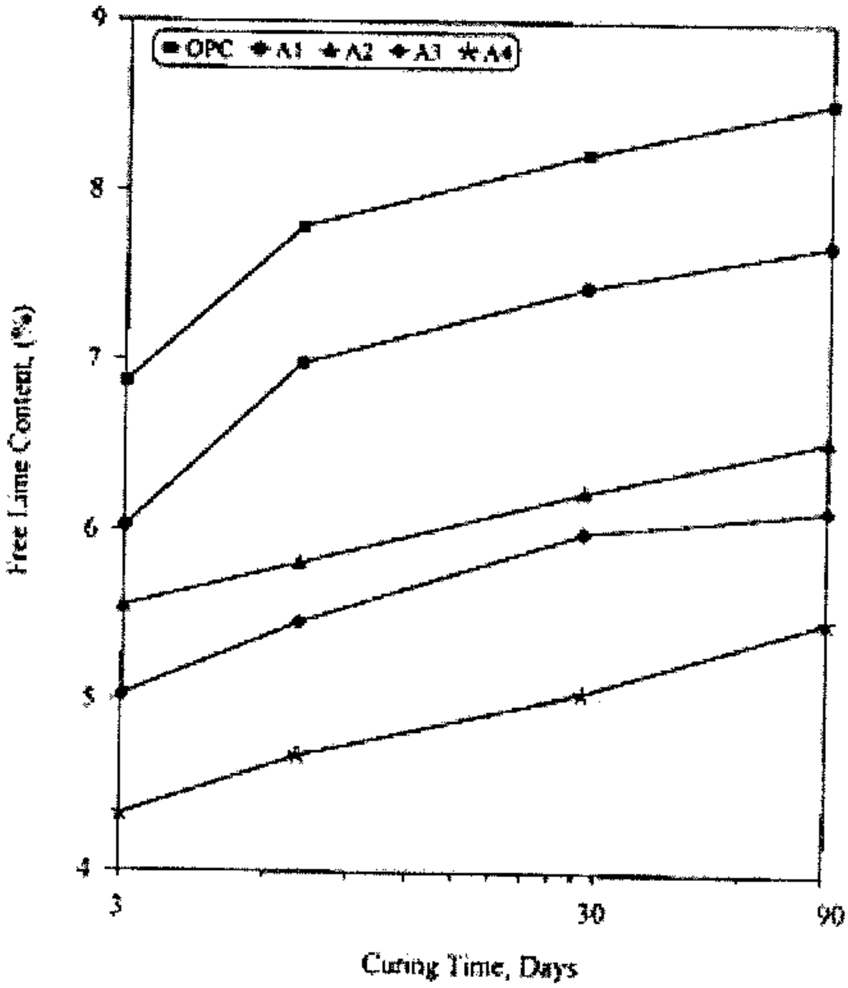

Fig. (3) Free Lime Contents of OPC Pastes Made with Various Proportions of Air-Cooled Slag up to 90 Days.

Compressive strength : The values of compressive strength of the hardened mortars of air-cooled slag cements and Portland cement cured for 3,7,28 and 90 days are represented in Fig.(4). The compressive strength of cement mortars increases with hydration time. This is due to the precipitation and later accumulation of hydrated phases in the available pores; as the hydration proceeds a hardened mass with higher strength was obtained. The higher strength is due to the formation of a dense structure as a result of crystallization of a highly polymerized CSH (tobermorite-like phase). Also the compressive strength decreases with the slag content which decreases the amount of hydration products deposited in the pores.

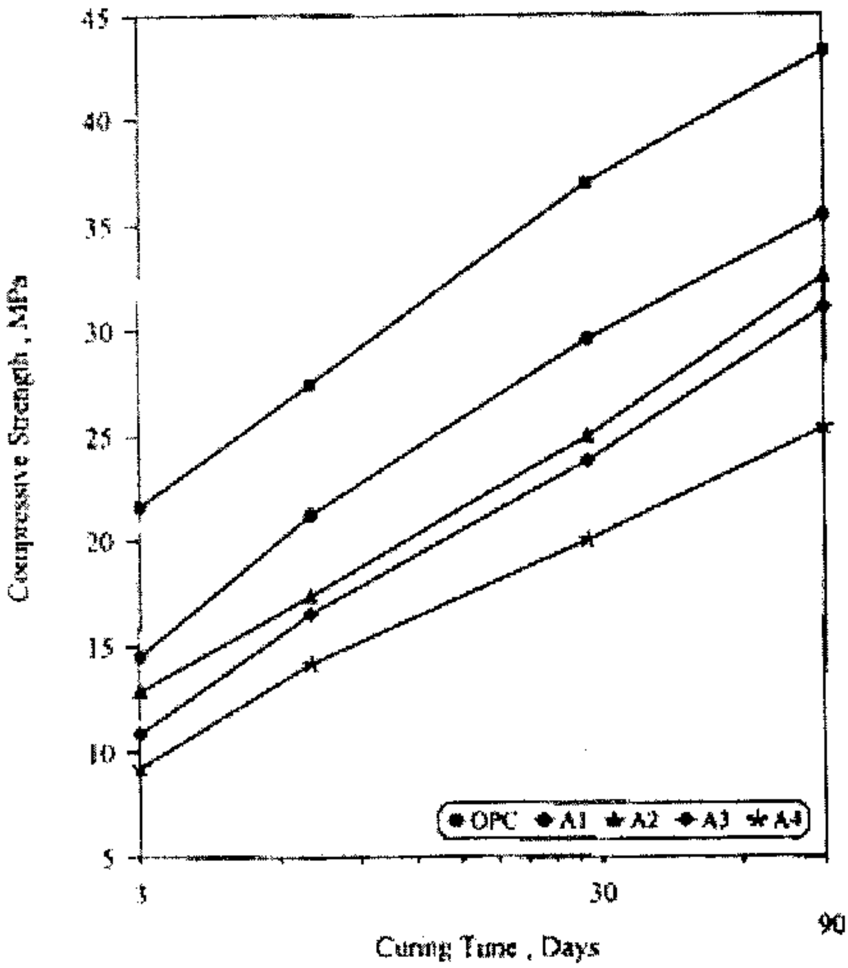

Fig (4) : Compressive Strength of OPC Mortars Made with Various Proportions of Air-Cooled Slag Versus the Age of Hydration

Seawater Attack

Chemically-combined water content : The results of combined water for hardened air-cooled slag cement pastes, in addition to OPC and SRC pastes immersed in seawater are graphically represented in Fig.(5). Evidently, the combined water content increases progressively with curing time. The extent of increase is higher for OPC than the others. The Progress of combined water content of OPC is mainly due to the formation of ettringite and chloroaluminate, which contain larger amounts of combined water. Also, the combined water content decreases gradually with slag content. This is due to the decrease of C_3A content for the part of OPC clinker, which correspond to the formation of calcium sulpho-aluminate hydrate. Therefore, OPC cement pastes possess higher values of combined water than those of SRC, A_1, A_3 and A_4 pastes respectively. The sulphate resisting cement pastes give lower values of combined water than those of OPC pastes due to the decrease of C_3A content.

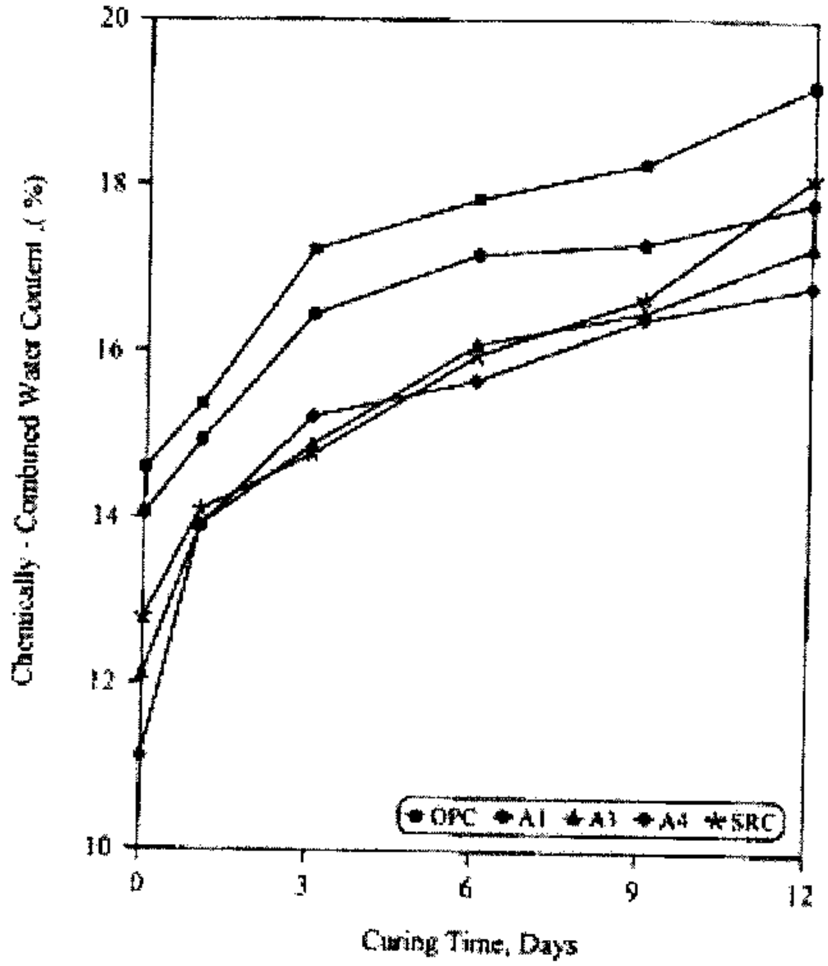

Fig.(5) : Chemically- Combined Water Contents for both SRC and OPC -Slag Cement Pastes Containing Various Proportions of Air-Cooled Slag Versus Curing Time in Sea Water.

Free lime content : The free lime contents of the cement pastes immersed in seawater up to one year are graphically plotted as a function of curing time in Fig. (6). The results indicate that the free lime content of hydrated air-cooled slag cement pastes decreases gradually up to one year in seawater. The decrease of free lime content in seawater is mainly attributed to the reaction of the liberated lime with magnesium chloride and sodium chloride as well as sulphate to form $CaCl_2$ and gypsum giving chloroaluminate, sulphoaluminate hydrates and magnesium hydroxide.

The cement pastes made of mixes (A_3,A_4) have lower free lime contents than those of OPC and SRC pastes. The free lime contents of blended cement pastes decrease with the fraction of OPC clinker. This is mainly due to the reduction of OPC clinker fraction in addition to the pozzolonic reaction of air-cooled slag with calcium hydroxide. The SRC pastes have relatively the same free lime contents as those of OPC pastes in seawater at all ages of hydration. This is mainly due to the relatively similar percent of C_3S in both types of Portland cement pastes; since the C_3S phase is the main source of liberated lime during hydration. Air- cooled slag cement pastes with

10% air-cooled slag gives higher values of free lime than those containing 20 and 30%.

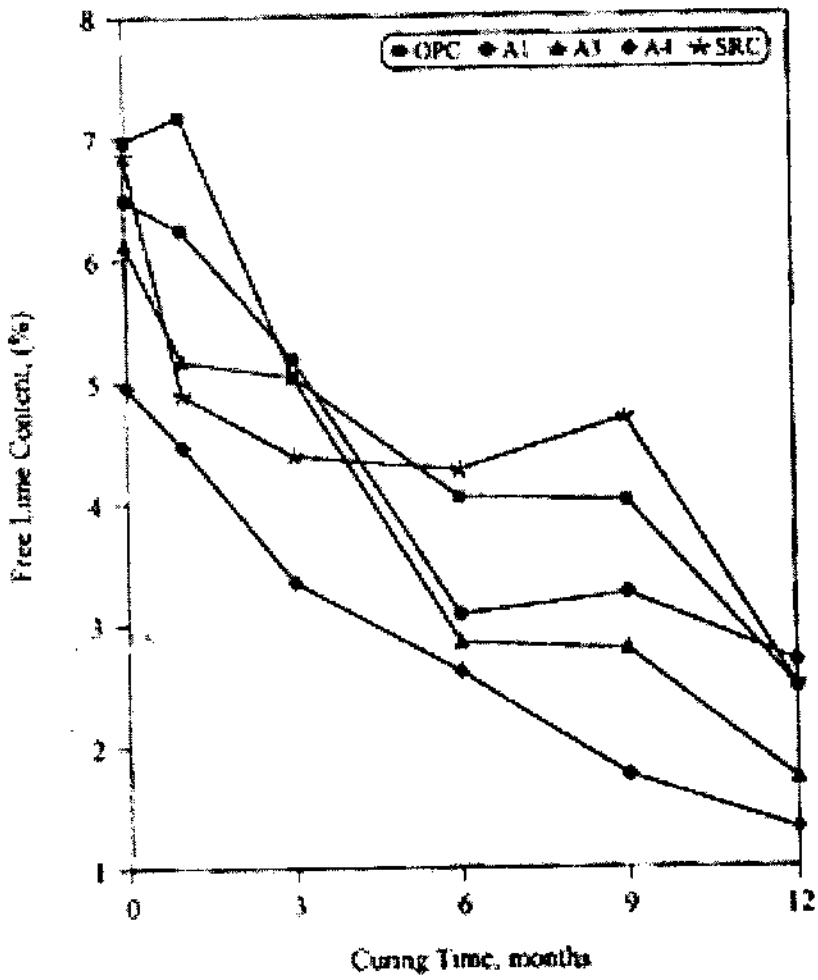

Fig (6) . Free Lime Content for both SRC and OPC -Slag Blended Cement Pastes Made with Various Proportions of Air-Cooled Slag Versus Curing Time in Sea Water

Total sulphate content : The total sulphate contents of the blended cement. OPC and SRC pastes are graphically represented in Fig.(7). Obviously, for all cement pastes the total sulphate content increases with immersion time in seawater. Seawater contains magnesium ions as sulphate and chloride. The sulphate reacts with calcium aluminate as well as calcium silicate hydrates to produce calcium sulpho aluminate hydrates, silica gel and $Mg(OH)_2$. With the increase of OPC clinker in the cement blend, the reaction of sulphate ions is enhanced in the OPC paste, and therefore, it has higher values of total sulphate up to 12-months, followed by those of mixes (A_1), (A_3), (A_4) and finally by SRC pastes. A_4 (40% air-cooled slag) exhibits the lowest values of total sulphate content up to 6 months. the decrease of sulphate content in SRC pastes is mainly due to the decrease or absence of C_3A in the anhydrous cement.

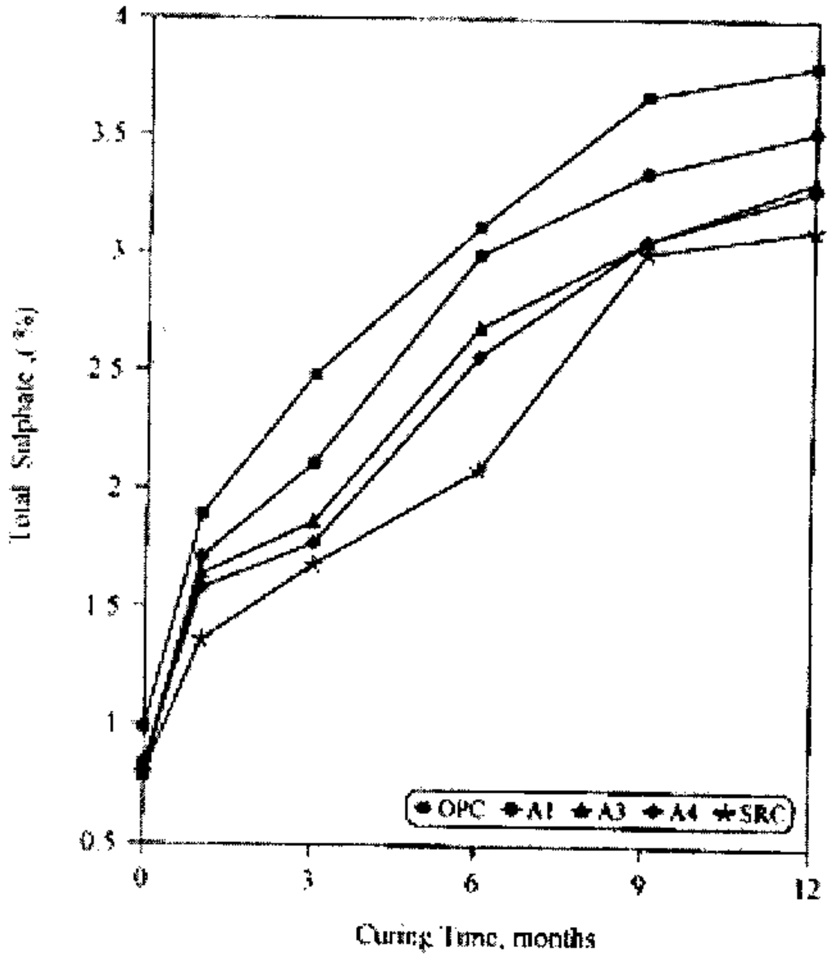

Fig.(7) Total Sulphate for both SRC and OPC -Slag Cement Pastes Containing Various Proportions of Air-Cooled Slag Versus Curing Time in Sea Water.

Total chloride contents : The total chloride contents of all cement pastes are graphically plotted in Fig.(8). Generally the chloride content increases with curing time due to the formation of $CaCl_2$ and then chloro-aluminate hydrates. The chloride content is directly proportional to the amount of C_3A in the cement as well as the free $Ca(OH)_2$. The Portland cement paste as well as the pastes of blend A_1 made with 10% air-cooled slag and 85% OPC clinker possesses the highest values of total chloride content especially at later ages of curing. On the other side, SRC and mix A_4 have the lowest values of chloride due to the reduction of C_3A as well as $Ca(OH)_2$ liberated during the hydration of OPC clinker. It may be concluded that the replacement of 40% air-cooled slag instead of OPC clinker in air- cooled slag cement improves the resistivity of this hardened cement paste against chloride attack.

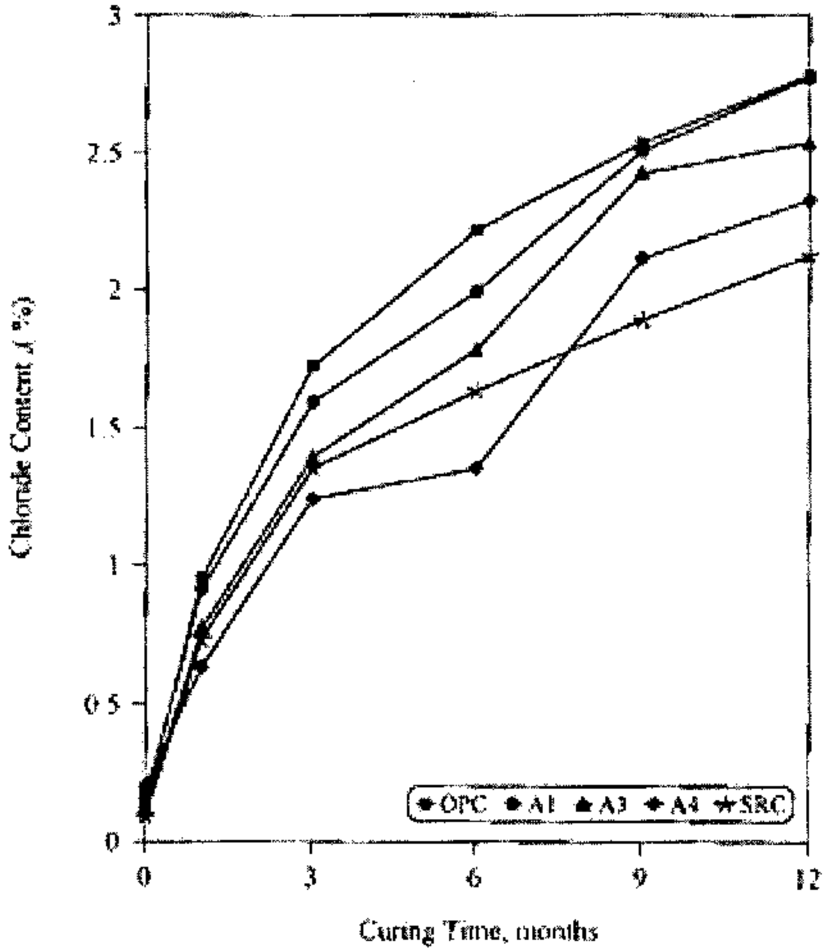

Fig (8) : Chloride Content for both SRC and OPC -Slag Cement Pastes Containing Various Proportions of Air-Cooled Slag Versus Curing Time in Sea Water.

Change in compressive strength : The change in compressive strength of blended cement pastes as well as SRC and OPC pastes immersed in seawater up to one year is graphically plotted as a function of curing time in Fig(9). It is evident that the value of compressive strength of the hardened cement pastes increases up to 3-months and then decreases up to one year. The lower strength change after one year immersion in seawater is for the blended cement paste (A_4) (40% air-cooled slag), followed by the paste (A_3) (30% air-cooled slag) and (A_1) 10% air-cooled slag and later the SRC and OPC pastes. The values of compressive strength changes of OPC paste are the highest. Therefore, the durability of the mix (A_4), (A_3) is higher than SRC. Evidently, the lower compressive strength change of the blended cements A_3 and A_4 results from their retardation effect towards the attack of sulphate and chloride ions in seawater due to the decrease of clinker content and then C_3A as well as liberated $Ca(OH)_2$. This means that both of ettringite and chloro-aluminate hydrates not pre-emptied in the voids and therefore exerted no expansive force as well as softening; but in the other side, SRC and OPC pastes deterioration is associated with the chemical reactions between components of seawater and the constituents of Portland cement clinker. The aggressiveness of seawater is higher than $MgCl_2$ and $MgSO_4$ solutions, especially at latter ages of immersion. This is due to the presence of the two salts together in addition to NaCl solution in the marine environment.

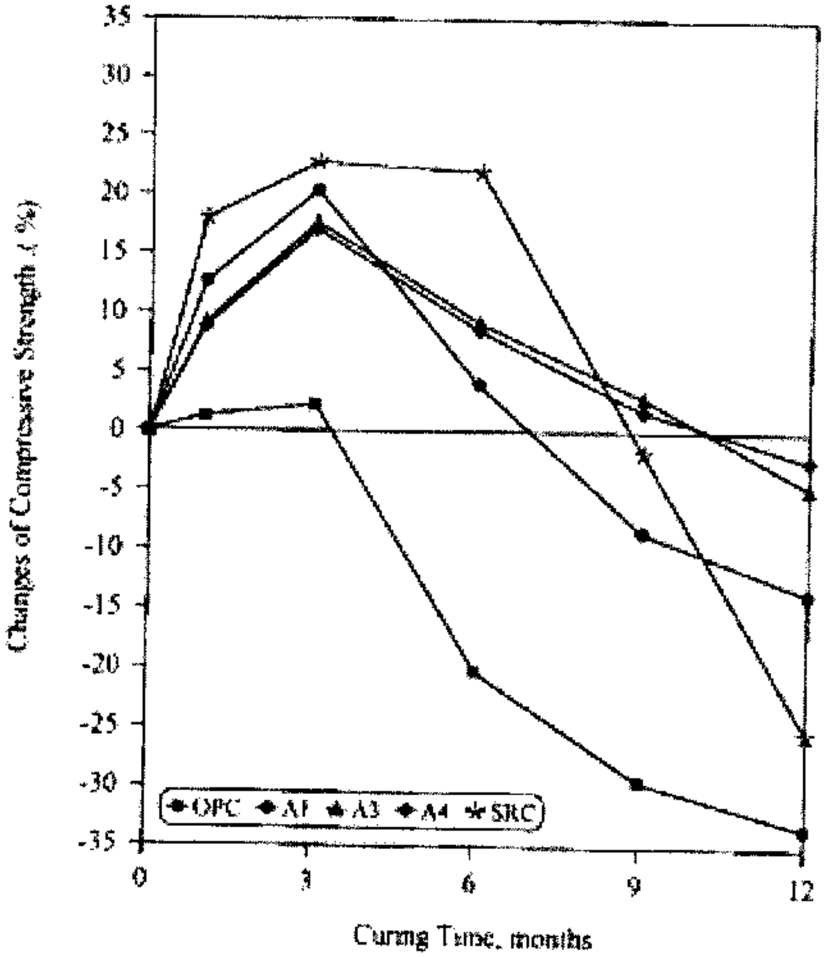

Fig. (9) . Changes of Compressive Strength for both SRC and OPC-Slag Cement Pastes Containing Various Proportions of Air-Cooled Slag Versus Curing Time in Sea Water.

CONCLUSION

It can be concluded that the air-cooled slag as a waste product from the steel manufacture shows low hydraulic activity in comparison to the granulated slag. This is clear from the increase of liberated lime with curing time up to 90 days. This means that the air-cooled slag does not consume any liberated lime[14]. This acts as filler and not pozzolanic. Also, the blended air-cooled slag made with 30-40 wt% slag shows higher durability in seawater than those of SRC and OPC up to one year.

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RETROFITTING OF UNREINFORCED MASONRY WALLS USING GLASS FIBER REINFORCED POLYMER LAMINATES

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ABSTRACT: The paper compares the structural response of unreinforced masonry (URM) walls with and without retrofit on one side using glass fiber reinforced polymer (GFRP) laminates. Six walls are experimentally investigated using ASTM diagonal tension (shear) standard test procedure. The objectives of the study included: (1) to compare between the improvements in performance provided by retrofitting a single-wythe wall versus that provided by retrofitting a triple-wythe wall, and (2) to compare the performance enhancement provided by the GFRP for two mortar-to-brick strength ratios. The results from the tests are compared with current codes for evaluating the design strength of URM walls. The strength enhancement provided by the GFRP in the experiments is also compared with current standards for estimating the design strength contribution of GFRP. The study shows that the application of GFRP on only one side of a triple-wythe wall prevents brittle catastrophic failure and may potentially improve the in-plane seismic response of a URM wall.

Keywords: Diagonal tension; Glass fibers; Multi-wythe; Unreinforced masonry.

INTRODUCTION

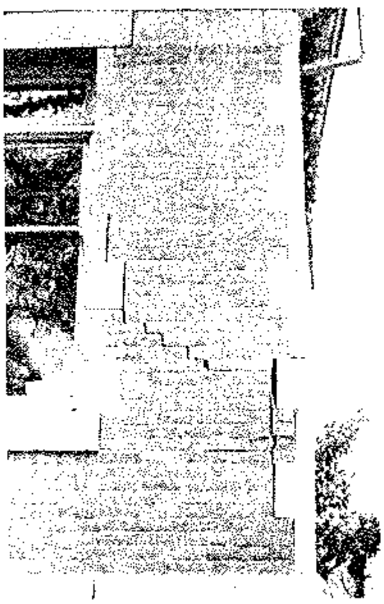
Observations from previous earthquakes show that buildings that depend on unreinforced masonry (URM) walls to resist seismic forces often experience severe damage compared to other structures. Since damage to masonry buildings usually results in falling of bricks, masonry buildings pose a serious life-safety threat even if complete collapse does not occur. Furthermore, masonry structures are prevalent allover the world. For example, masonry constitutes approximately 70% of existing buildings in USA in 2001 (Tumialan, 2001). Though most of these structures exist in East USA, a large number were built in the earthquake region along the West coast prior to consideration of seismic forces.

Because of the prevalence of URM, the development of appropriate retrofitting techniques is an important component of the effort to mitigate the disastrous effects of earthquakes on urban areas. Masonry structures generally depend on structural walls to resist lateral forces. If the shear forces caused by the inertia of upper floors are high, the shear capacity of URM walls can be exceeded and the wall may fail in-plane, Figure 1.

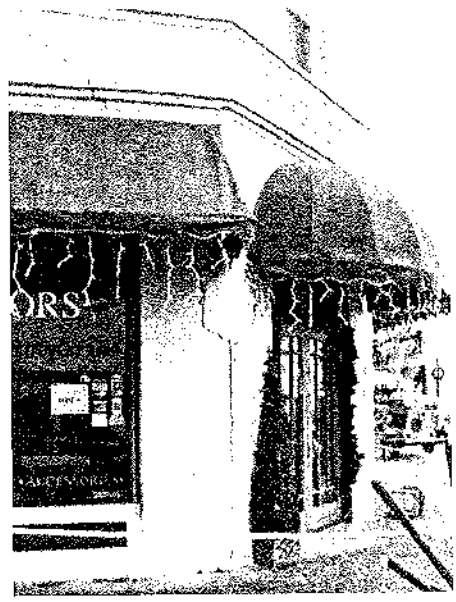
URM is also frequently used to infill reinforced concrete (RC) frames. Prior to damage of infill walls, they contribute significantly to the lateral stiffness of the structure. Collapse of the infill wall causes significant threat to life and may allow the RC frame to drift to potentially unstable displacements. An example of URM infill failure is shown in Figure 2.

An experimental investigation of the behavior of infilled RC frames with URM walls is currently under development at the University of California, Berkeley (UCB), Figure 3. This paper summarizes the first phase of the study at UCB which globally aims at understanding the behavior of infilled RC frames using shake-table and pseudo-dynamic experiments and developing retrofitting techniques of these vulnerable structural systems.

The weakness of URM structures has long been recognized. As a result of increased consciousness of the potential effects of future earthquakes, much effort has been put forth to retrofit all types of structures in the Western USA over the last decade. Recently, glass fiber reinforced polymer (GFRP) laminates, with their transparent appearance, have become popular for both concrete and masonry retrofits. Oftentimes, URM structures are historically significant, requiring minimal changes to exterior architectural appearance. Consequently, engineers often have limited access to the structural elements that require strengthening. Previous research focused on use of GFRP on both sides of the wall or on one side of a single-wythe wall. In practice, historical structures typically have multi-wythe walls supporting gravity and seismic forces and frequently, engineers have access to only the interior side of the wall. This reality prompted the investigation of triple-wythe URM walls retrofitted on one side with GFRP.



(a) Diagonal shear cracks in a corner pier next to a window in a URM building.



(b) Diagonal shear cracks throughout a URM building.

Figure 1. Observations from December 22, 2003 San Simeon, California earthquake (http://peer.berkeley.edu/san_simeonEQ/).

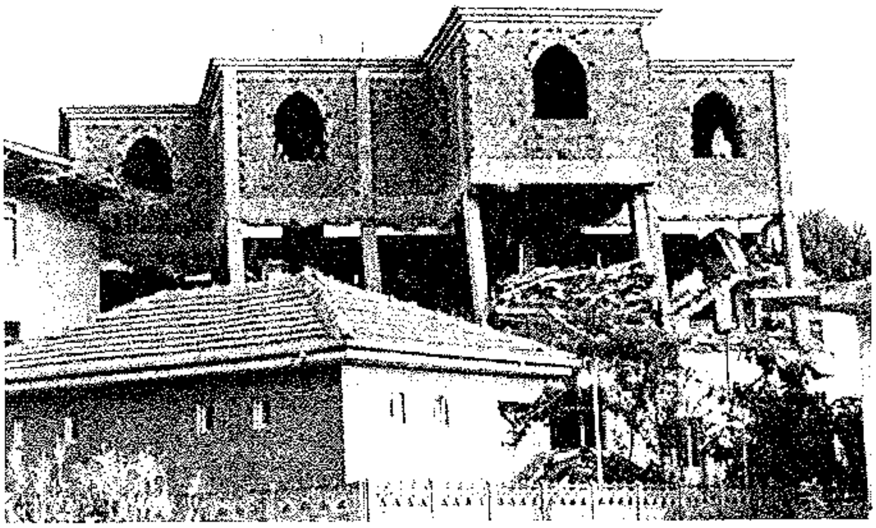
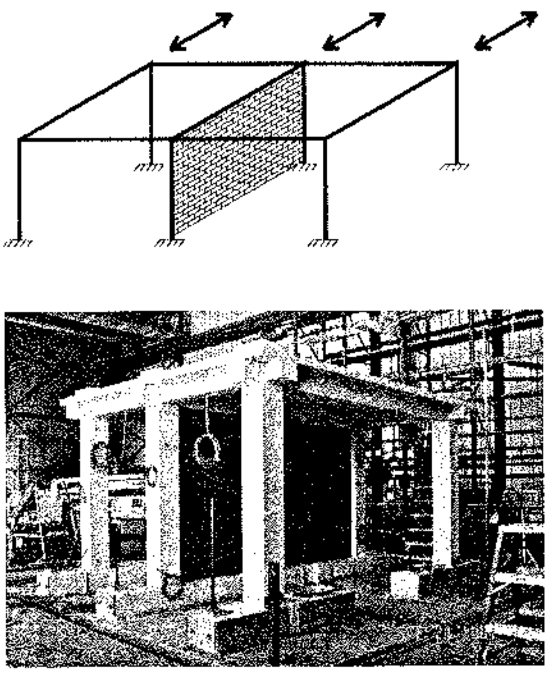
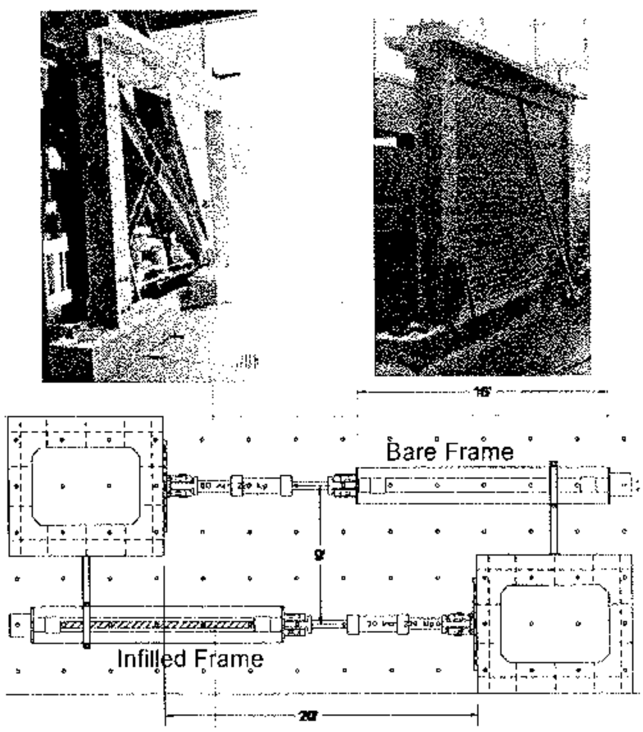


Figure 2. RC frame with collapse of brick infill, August 17, 1999 Izmit (Kocaeli), Turkey earthquake (http://nisee.berkeley.edu/images/servlet/EqiisListQuake).



(a) Shake-table test with a URM infill wall in middle frame.



(b) Pseudo-dynamic test of URM infilled and bare RC frames.

Figure 3. Planned experiments at UCB for URM infilled RC structural systems.

The objective of the experiments conducted in this study is to evaluate the effectiveness of GFRP in enhancing strength and ductility when applied to only one side of a triple-wythe URM wall. In this study, two variables are investigated. First, a comparison is made between the improvements in performance provided by retrofitting a single-wythe wall versus that when retrofitting a triple-wythe wall. Second, performance enhancement provided by the GFRP is compared for two mortar-to-brick strength ratios.

EXPERIMENTAL STUDY AND MATERIAL PROPERTIES

Two pairs of control and retrofitted specimens are tested with Type N (weak) mortar for both single-wythe and triple-wythe URM, and one pair of triple-wythe specimens is tested with Type M (strong) mortar. The same type of brick is used for all the walls. The test matrix in Table 1 shows the performed wall tests.

Table 1. Test matrix

Wall No.	Wythes	Mortar	Retrofit
1	1	N, Weak	None
2	1	N, Weak	GFRP
3	3	N, Weak	None
4	3	N, Weak	GFRP
5	3	M, Strong	None
6	3	M, Strong	GFRP

A diagonal tension test slightly modified from ASTM E 519 was chosen, because of its similarity to the in-situ stress state that would exist in a panel of bricks in a URM wall (ASTM, 1988). To simplify the test setup, a panel 30"×30" was tested as opposed to the 48"×48" panel recommended by ASTM guidelines. The panels represent a prototype middle section of either a URM wall with shear and axial load forces, Figure 4(a), or an infill wall bounded by a RC frame with a compression strut forming along the diagonal, Figure 4(b). Regardless of the prototype chosen, a combined compression and shear force exists at the center of the panel, as shown in Figure 4(c). To model this stress state, the wall panel is compressed in-plane across the diagonal using the test setup shown in Figure 4(d). This setup creates the shear stress state shown in Figure 4(e), which induces the diagonal tension cracking typically observed in prototype buildings.

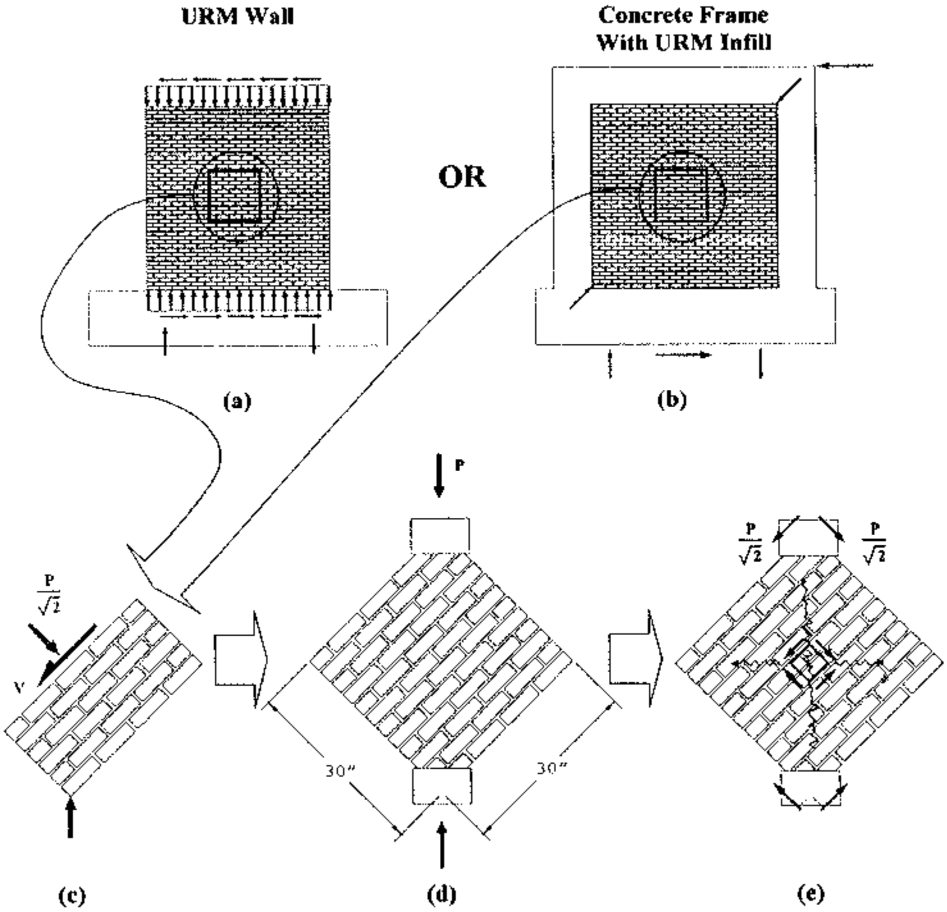


Figure 4. Justification of adopted testing configuration. (a) URM wall. (b) RC frame with URM infill. (c) Free body diagram of the brick panel in URM wall or URM infill prototypes. (d) Test setup. (e) Stress conditions inducing vertical cracking in the test panel.

The behavior of a URM wall depends on the properties of the brick units, Table 2, the mortar, Table 3, and the mortar-brick interface. Several material tests were conducted to better understand the wall panel behavior. Table 4 lists average results for each material test as well as the coefficient of variation (COV) for each test result.

Table 2. Physical brick properties

Gross area		29.50 in^2
Net area		25.08 in^2
Compressive strength	Gross	6933 psi
	Net	8155 psi
Absorption	24 hr. cold soak	6.04
	5 hr. boil	9.61
Saturation coefficient		0.63
Initial rate of absorption (IRA)		40.41

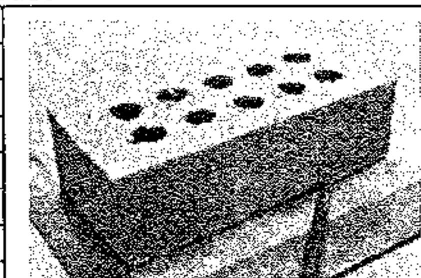


Table 3. Actual mix proportions of the used mortar

Mortar type	Proportion by volume		
	Type I/II Portland cement	Type S, Hydrated lime	Sand
M	1	0.25	3.5
N	1	1	6

Table 4. Material test result (CF: Cored Face, SF: Smooth Face)

Test		Curing period [days]	Number of specimens	Average strength [psi]	COV [%]
Brick compression		-	5	4341	24
Brick modulus of rupture		-	5	794	25
Brick split tension		-	4	516	18
Mortar compression	Type N	7	3	736	2
		28	3	1179	2
	Type M	7	3	2910	0
		28	3	4155	2
Mortar split tension	Type N	7	3	113	9
		28	3	186	3
	Type M	7	3	393	13
		28	3	487	9
Masonry prism	Type N	28	5	2906	4
	Type M	28	5	3472	4
Direct shear (Interface)	Type N: CF	-	2	156	11
	Type M: CF	-	2	232	33
	Type N: SF	-	2	106	5
	Type M: SF	-	2	182	42

Professional contractors applied two GFRP sheets to one side of the brick panels. Hex-3R Wrap 101G manufactured by Hexcel Corporation was used in combination with Sikadur Hex 300 high modulus, high strength, impregnating resin to form a GFRP laminate. The manufacturer, Sika, reported material properties in Table 5. Calculations of GFRP design strength can be made using either the fiber properties or the laminate properties. In this paper, calculations are made according to the laminate properties. Note that ply thickness of 0.04" was the overall laminate.

Table 5. Reported FRP properties after standard cure and post-cure for a single ply

E-glass fiber	
Tensile strength	330,000 psi
Tensile modulus	10,500 ksi
Density	2.54 g/cc
Elongation	4.0%
Sikadur Hex 300 and Hex-3R Wrap 101 laminate	
Tensile strength	44,100 psi
Tensile modulus	2,270 ksi
Shear strength	9,000 psi
Shear modulus	470 ksi
Poisson ratio	0.189
Elongation	2.37%
Ply thickness	0.04 in.

Single-wythe panels were constructed in the pattern shown in Figure 4(d). Both vertical joints and horizontal bed joints were approximately 9/32 in. As is common in URM buildings, triple-wythe panels were constructed with headers placed every sixth course in the common bond brick pattern.

In the application of GFRP, Figure 5, the brick wall was sand blasted thoroughly to remove excess mortar and create a flat surface. The panel was then air blown to remove sanded clay particles. Next, a filler mixture of Cabosil (a compound of fine aggregates) and Sika epoxy resin was used to fill in large pores in the bricks and mortar. The first GFRP sheet was placed on the panel with the fibers aligned at ±45 degree angles to the edge. The contractors then rolled more resin over the first sheet to remove air gaps. The second ±45 degree sheet was placed on the first resin-covered sheet and again covered with more resin and smoothed and pressed to remove any air gaps.

Figure 5. Process of applying GFRP to URM wall.

The in-plane displacement instrumentation setup is shown in Figure 6. The out-of-plane distance from the wall to the gage was measured, so that by comparing displacements on each side of the wall, curvatures could be estimated. Another displacement transducer measured the displacement of the loading head and a load cell measured the vertical force applied monotonically along one diagonal of the specimen.

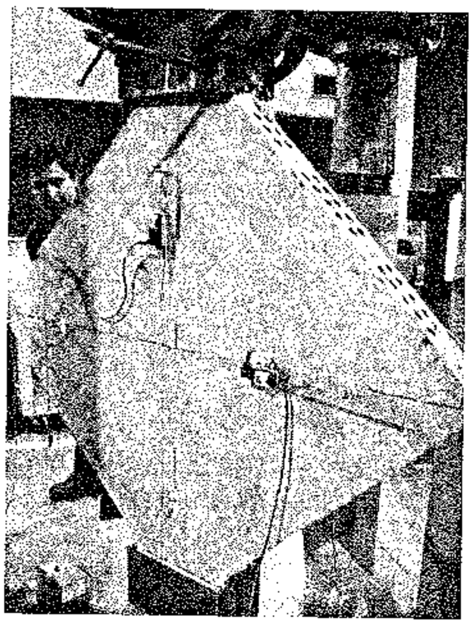


Figure 6. Diagonal tension test with instrumentation.

EXPERIMENTAL RESULTS AND DISCUSSION

The results of the diagonal tension tests are presented in two forms in Figure 7. The first is the shear force versus drift, where shear force is normalized by the thickness of the wall. The second is the brick shear stress versus shear strain. The labeled points in the top middle plot represent time after yielding obtained from test movies in units of [sec.; 1/30th sec.], because each video frame lasts 1/30 sec.

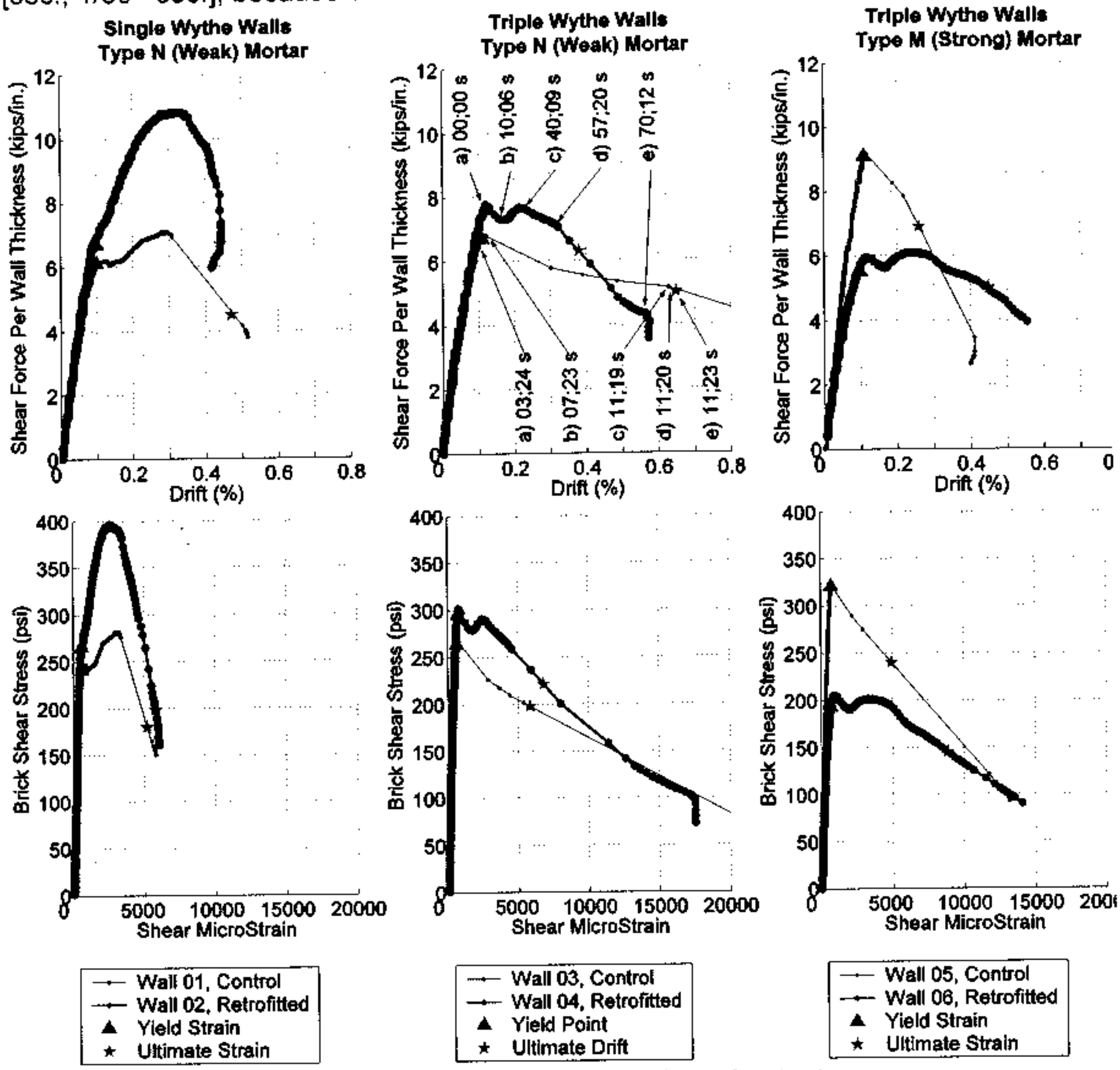
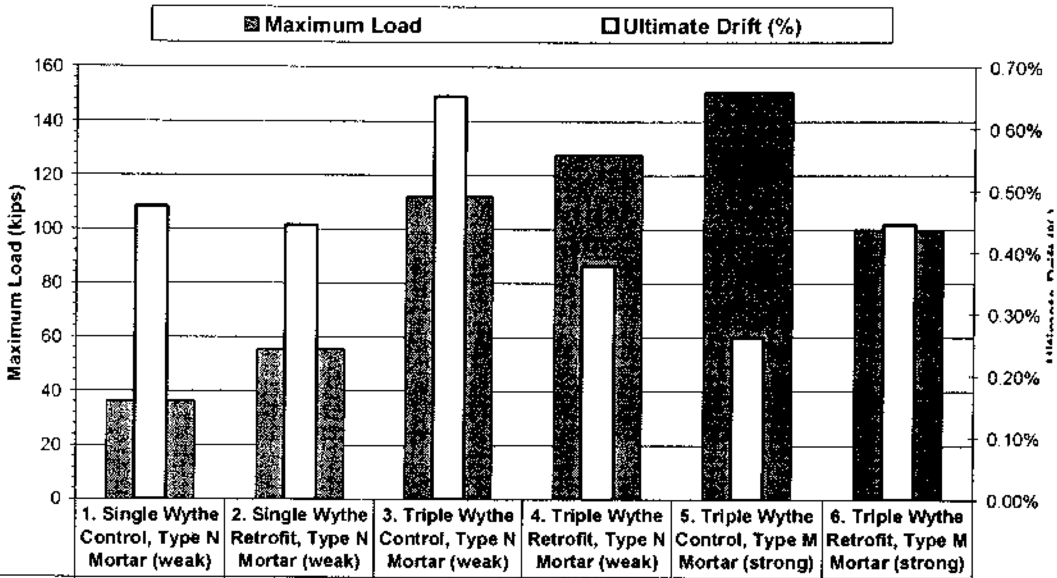


Figure 7. Results of all diagonal tension tests.

The results show that the elastic behavior of the panels is not significantly affected by the addition of GFRP. Moreover, the GFRP does not seem to greatly enhance the strength of the triple-wythe panels. As shown in Figure 8, the triple-wythe retrofitted wall with weak mortar is slightly stronger than the unretrofitted wall, but for the strong mortar, the retrofitted panel was actually much weaker. This inconsistency prevents general conclusions from being made regarding the effect of GFRP on strength of a triple-wythe wall. Figure 8 also shows that no conclusions can be made regarding effect of GFRP on ultimate drifts. The retrofitted wall 4 had a lower ultimate drift than the unretrofitted wall 3.

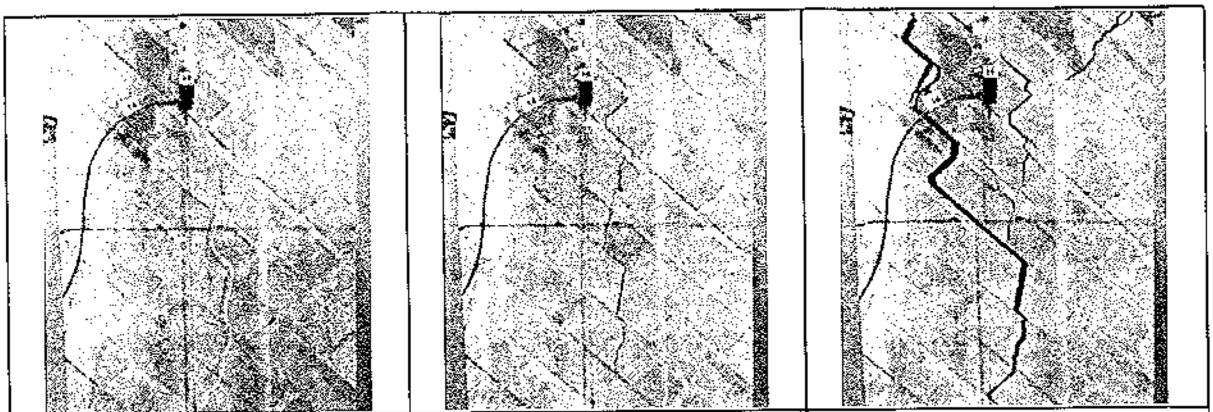
Wall 1 (single wythe, control) and wall 2 (single wythe, retrofitted) both yield at about the same drift near 6 kips/in. when initial cracking in the brick is observed. While the unretrofitted wall gains minimal strength after cracking, the retrofitted wall continues to resist load as the GFRP bears additional forces, resulting in a maximum load almost twice as large as the yield load. This behavior was not observed in triple-wythe walls because the GFRP could not continue to resist forces after bricks on opposite side failed.



	1. Single Wythe Control, Type N Mortar (weak)	2. Single Wythe Retrofit, Type N Mortar (weak)	3. Triple Wythe Control, Type N Mortar (weak)	4. Triple Wythe Retrofit, Type N Mortar (weak)	5. Triple Wythe Control, Type M Mortar (strong)	6. Triple Wythe Retrofit, Type M Mortar (strong)
Maximum Load	36	56	112	128	151	100
Ultimate Drift (%)	0.47%	0.44%	0.65%	0.38%	0.26%	0.45%

Figure 8. Comparison of ultimate drift and maximum load.

The two retrofitted triple-wythe walls showed an initial decrease in strength when the first bricks cracked followed by a slight increase in capacity as forces shifted to the GFRP after the cracked bricks lost stiffness. Rather than resisting additional forces as was the case for the single-wythe walls, the capacity of the retrofitted triple-wythe walls continued to deteriorate. The capacity decreased very slowly as indicated by the closely spaced data points in Figure 7. On the other hand, the unretrofitted triple-wythe walls cracked and then lost strength quickly as indicated by the widely spaced data points in Figure 7. Standard equations for calculating ductility do not show any improvement provided by the GFRP. However, observation of the experiments suggests an apparent advantage provided by the GFRP. In all cases, the unretrofitted specimens failed catastrophically, whereas the retrofitted specimens developed several cracks and failed gradually. To quantify this improvement in behavior, the top middle plot of Figure 7 is stamped with times that images were extracted from the video of the control (Figure 9) and retrofitted (Figure 10) specimens. In Figure 9(a), a small crack forms shortly after yielding of the control wall 3. Within 4 sec., the crack breaks open violently, and the two sides of the wall completely separate, Figure 9(c). In Figure 10(a), a tiny crack propagates shortly after yielding of the retrofitted wall 4. With the GFRP retrofit, the crack opens slowly, a second parallel crack opens (Figure 10(b)), and the wall continues to resist load for 70 sec. even after the crack becomes very large, Figure 10(c). The side with the GFRP showed that the GFRP never delaminated from the brick and though the crack propagated through all three wythes, no crack or damage was observed on the GFRP face. All three retrofitted specimens exhibited similar behavior. Accordingly, the GFRP clearly affects even a triple-wythe wall retrofitted on one side. Therefore, one can expect that GFRP reduces the brittleness of multiple-wythe URM walls under dynamic loading.



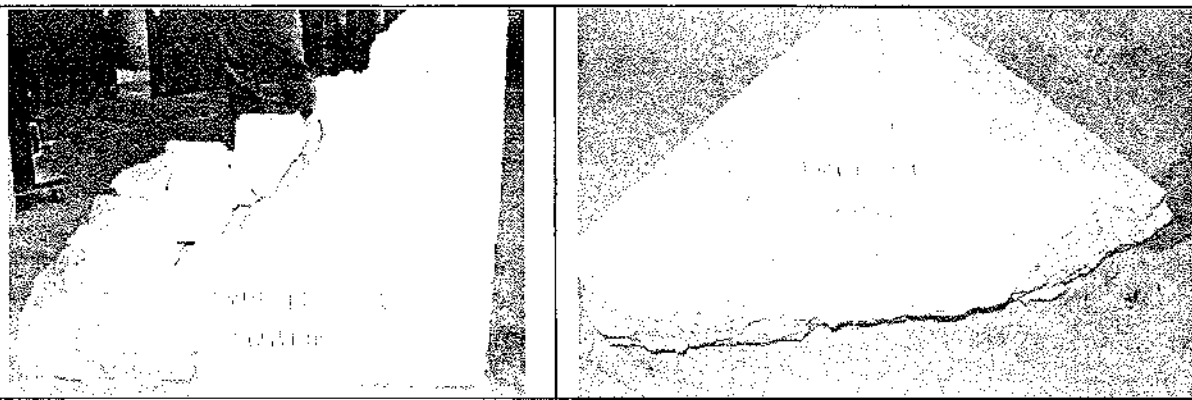
(a) 7;23 sec.	(b) 11;19 sec.	(c) 11;20 sec.

Figure 9. Control wall 3

(a) 10;06 sec.	(b) 57;20 sec.	(c) 70;12 sec.

Figure 10. Retrofitted wall 4.

One of the preliminary objectives of this research was to better understand how GFRP retrofit of URM panels would be affected by different ratios of brick to mortar strength. Bricks and mortar were chosen with the intention that walls 1 through 4 with Type N mortar would have much stronger bricks than mortar and failure would initiate in the mortar and zigzag around the bricks. Walls 5 and 6 were intended to have stronger mortar than brick and fail in the brick first and crack directly through the bricks. From the test results, once the diagonal crack formed, its propagation was significantly different for the two types of mortar. From Figure 11, it is clear that the crack in the wall with Type N traveled around the stronger bricks, and in the wall with Type M, the crack propagated directly through the bricks.



(a) Zigzag crack pattern, wall 3.	(b) Continuous crack pattern, wall 5.

Figure 11. Effect of mortar type on failure of URM panels.

TEST LIMITATION

There several limitations of the presented experimental research. Based on finite element analysis (FEA) of test panels, Figure 12 shows that most of tensile strain is concentrated within a small band near the center of the panel. Accordingly, local strain in this region is much higher than average strain across the gage length, in the interpretation of the test results. For details about FEA of test specimens, the reader is referred to (Nguyen, 2003).

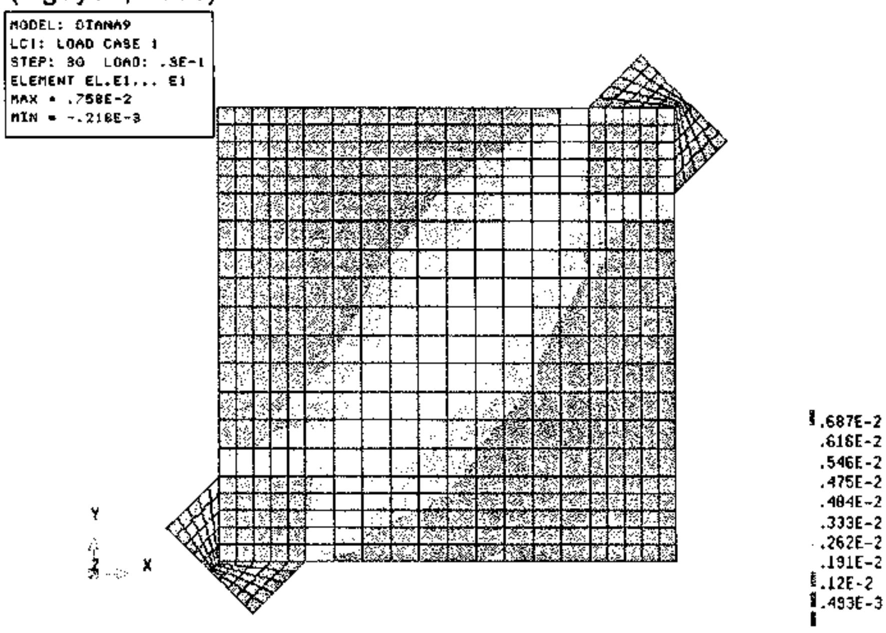


Figure 12. Principal tensile strain in homogeneous system using finite element analysis.

DESIGN IMPLECATIONS

In this study FEMA 356 (2000) and ACI 530 (2002) were used to determine the design shear strength of a hypothetical 10'×10' freestanding URM shear wall. The design strengths were compared with the scaled measured shear forces from the experimental panels. The left column of Figure 13 shows four of the considered design strengths superimposed on the experimental results for the three control specimens. The code plots include the predicted stiffness of the hypothetical wall including flexure and shear contributions. Due to scaling effects, the experimental shear force is assumed to be simply four times the actual shear force from the experiments.

After the code shear strength and stiffness were calculated, the GFRP contribution to shear strength was predicted. The right column of Figure 13 shows the design shear strength of the URM walls retrofitted with GFRP superimposed on the experimental drift-shear force relationship. The lines in the figure represent the design strength. The GFRP contribution is shown here using the AC 125 (ICBO, 2001) recommendations (labeled 5 and 7) and using the experimentally calculated yield shear strain, namely 660 micro-strain (labeled 6 and 8). These predicted GFRP contributions are then added to the FEMA 356 and ACI 530 Strength Design calculations for the unretrofitted URM wall (shown in the left column of Figure 13) to obtain the overall design strength of a retrofitted prototype. Note that the allowed masonry stress (100 psi) is used for the FEMA 356 calculations.

From Figure 13, it is somewhat difficult to make conclusive remarks regarding the accuracy of the design methods. This is attributed to the variability in test results, the small number of specimens tested, and the inability of the diagonal tension test to model real URM wall behavior, as discussed previously in conjunction with Figure 12.

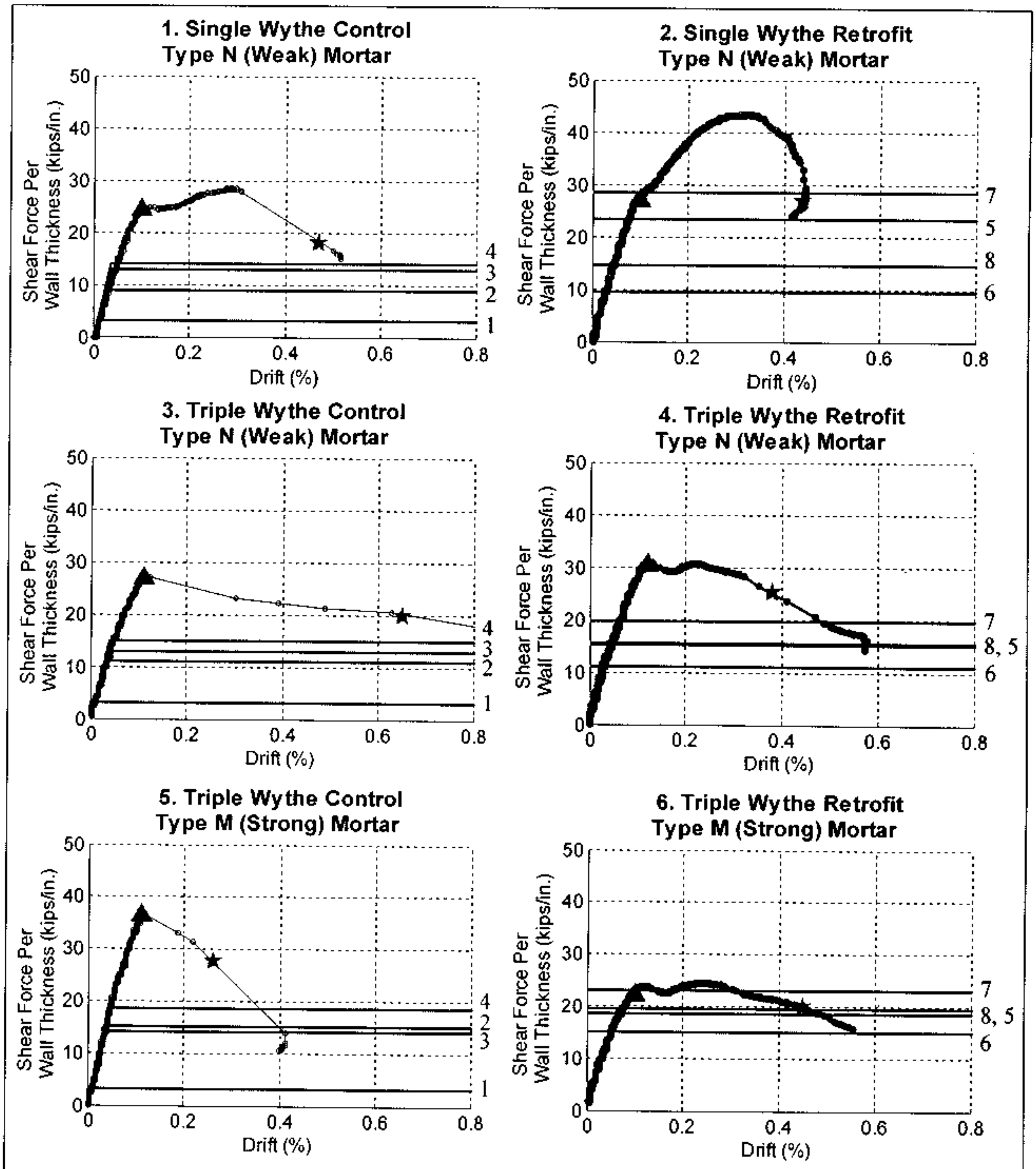
In conclusion, the design techniques for evaluating the shear strength of a URM wall are well established. However, more research is necessary to better estimate an appropriate method for predicting the additional shear strength provided by retrofitting a multi-wythe URM wall with GFRP on only one side.

CONCLUDING REMARKS

Conclusions from the experiments can be summarized as follows:

1. GFRP increased the strength of the single-wythe URM wall by about 50% but had minimal effect on ductility. However, the triple-wythe wall results did not consistently show an increase in ductility or strength provided by the GFRP.
2. Walls retrofitted with GFRP exhibited less catastrophic failures. Upon initial cracking, the strength capacity of the retrofitted walls deteriorated at much lower strain rates than the unretrofitted walls. However, it is difficult to directly apply this result to the retrofit design of a URM building unless further in-depth investigation is performed.
3. The strong Type M mortar most likely increased the sensitivity of the URM wall to flaws. Though historic URM buildings are more likely to have weak mortar relative to brick strength, testing the strong mortar showed that the addition of GFRP may decrease such sensitivity to flaws by increasing the ductility capacity of the wall. Thus, retrofitting a historic structure with GFRP may reduce sensitivity to flaws caused by poor construction practices or deterioration over time. The strength of the retrofitted wall with Type M mortar was 66% of the strength of the unretrofitted wall, yet the ductility was nearly two times greater.
4. Current design guidelines provide reasonable estimates of the shear strength of a URM wall. More research is necessary to evaluate current practices for predicting the additional shear strength provided by retrofitting a multi-wythe URM wall with GFRP on only one side.

The above conclusions should be considered in the limited context of the experiments conducted. Further research is needed to determine more definitively the advantages and disadvantages of using GFRP on only one side of multi-wythe URM walls.



1. FEMA356, default masonry shear strength	5. FEMA356, allowed stress & GFRP from AC125 (ε=0.004)
2. FEMA356, allowed stress	6. FEMA356, allowed stress & GFRP from yield shear strain
3. ACI530, allowable stress design	7. ACI530, strength design & GFRP from AC125 (ε=0.004)
4. ACI530, strength design	8. ACI530, strength design & GFRP from yield shear strain

Figure 13. Predicted behavior versus experimental results for a hypothetical wall.

ACKNOWLEDGMENTS

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DESIGN GUIDELINES FOR CONCRETE BEAMS REINFORCED WITH MMFX MICROCOMPOSITE REINFORCING BARS

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ABSTRACT: Corrosion of steel reinforcement in concrete structures and bridges is a major problem facing the departments of transportation worldwide. In the United States, maintenance and replacement costs are measured in billions of dollars. Salt environment in hot climate and the use of deicing salts in cold regions have resulted in steady deterioration of bridge decks due to corrosion. These concerns have initiated continual development of protective measures including the use of corrosion-resistant MMFX Microcomposite reinforcing bars.

This paper provides design guidelines for the use of MMFX steel as flexural reinforcement for concrete beams and slabs. The behavior of concrete beams reinforced with MMFX reinforcing bars is evaluated and characterized using cracked section analysis. Principles used for the design of MMFX-reinforced concrete beams are discussed. The behavior of concrete beams reinforced with MMFX is compared to the behavior of the beams reinforced with conventional Grade 60 steel. Using the principles of equilibrium and compatibility, the effect of reinforcement ratio on the strength of concrete beams reinforced with MMFX is examined. The ductility of concrete sections reinforced with MMFX steel throughout the entire loading range is evaluated and design limits for tension and compression controlled failure modes are proposed. In addition to the concrete bridges constructed recently using MMFX steel, this paper discusses the construction procedures of a new bridge at North Carolina where the concrete deck is totally reinforced with MMFX steel.

Keywords: MMFX, flexure, strength, ductility, design guidelines

INTRODUCTION

Corrosion is a significant problem currently facing civil engineering infrastructure. Corrosion of reinforcement can result in the need for costly repair or possible total replacement of the entire reinforced concrete structure. To address this issue, MMFX Steel Corporation of America has developed a new type of reinforcement called Micro-Composite Multi-Structural Formable reinforcing steel (MMFX). MMFX is a high strength, highly corrosion resistant steel ideal for use in civil engineering applications. Because it is a relatively new technology, the effective use of this type of reinforcement as flexural reinforcement for concrete members is still under development by many researchers. Currently there are several limitations imposed on its tensile stress-strain behavior for its immediate safe use for structures and bridges.

This paper examines the use of MMFX steel as flexural reinforcement for concrete members. The effect of the various limitations on the flexural strength and ductility of concrete sections reinforced with MMFX is considered using simple cracked section analysis. The analysis is verified by comparison with the results of experimental data reported by other researchers. Limitations on the resistance factor are proposed based on an acceptable level of the ductility of members reinforced with MMFX. A design chart is presented based on the actual stress-strain behavior of MMFX and the proposed resistance factors presented in this paper.

The scope of this paper is limited to the analysis and design of singly reinforced, rectangular concrete sections and assumes that reinforcement area is lumped at the centroid of the steel area. Concrete strengths under consideration were selected to be representative of the common range of strengths used for design. Concrete strengths were limited between 3,000 psi and 8,000 psi. The analysis in this paper follows the design concepts and recommendations of ACI 318-02.

BACKGROUND

Corrosion of reinforcement is a leading cause of deterioration of concrete structures and necessitates expensive rehabilitation, repair and replacement. It has been estimated that 43,000 highway bridges in the United States are in need of rehabilitation, repair or replacement due to the corrosion of reinforcement induced by the spreading of deicing salts alone[1]. As such, there is a need to develop reinforcing steel that is not susceptible to corrosion. MMFX Steel Corporation of America is currently producing Micro-Composite Multi-Structural Formable reinforcing steel (MMFX), which is highly corrosion resistant compared to conventional steel.

Corrosion of conventional steel is an electrochemical process that occurs at the microstructural level. The microstructure of conventional steels consists of bands of ferrite and iron-carbide. These bands form a microgalvanic cell within the steel in which ferrite is the anode and iron-carbide acts as the cathode. Electrons from the ferrite travel to the iron-carbide and react to form corrosion byproducts including rust[2]. This process is illustrated in Figure 1 and the associated chemical equations are presented in the same figure.

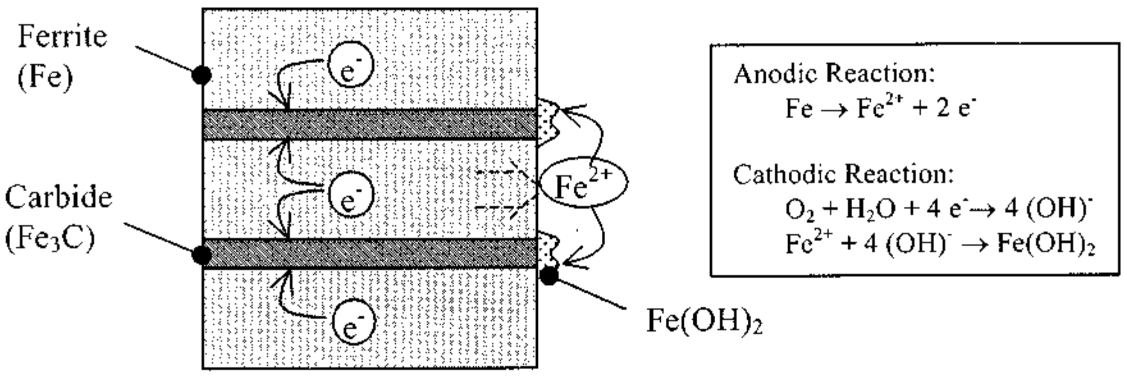

Figure 1: Schematic of a Microgalvanic Cell in Steel[2]

MMFX is designed to eliminate the mechanisms that cause corrosion. Its microstructure consists of alternating bands of austenite and martensite. Consequently, MMFX is practically carbide free and microgalvanic cell formation is minimized[2]. MMFX exhibits greater corrosion resistance than ASTM A615 steel reinforcement as demonstrated by the results of Accelerated Chloride Threshold testing conducted on both materials[3]. Furthermore, MMFX exhibits an ultimate tensile strength approximately 2.5 times that of conventional Grade 60 steel making it well suited for use in civil engineering applications.

MMFX Corporation proposes the use of equation 1 below, which is modified from that proposed by Vijay et. al., as a representation of the stress-strain behavior of MMFX steel[4,5].

$$f_{MMFX} = 165\left(1 - e^{-185\varepsilon_{MMFX}}\right) \tag{1}$$

where f_{MMFX} and ε_{MMFX} are the stress and the strain in the MMFX reinforcing bars, respectively. Using the 0.2% offset method, the yield strain of MMFX reinforcement using this equation is 0.006 in/in. Equation 1 above closely represents the actual tensile stress-strain curve of MMFX steel as can be seen in Figure 2. This equation also corresponds well with experimental stress-strain curves of several sizes of MMFX bars reported by NCSU[6].

Several limitations have been proposed to represent the tensile stress strain behavior of MMFX steel. The Concrete Innovations Appraisal Service (CIAS) has verified that MMFX steel can conservatively be represented by an elasto-plastic stress strain curve with a modulus of elasticity of 20,000 ksi and yield strength of 100 ksi as proposed by MMFX Corporation[4]. These values were deemed to be conservative based on the consideration of a series of tensile tests of No. 4 to 11 MMFX bars[4]. This stress-strain model is presented in Figure 2 as Model 1. The American Concrete Institute (ACI) in clause 9.4 of ACI 318-02 limits the yield strength of concrete reinforcement to 80 ksi[7]. To conform to this limitation, equation 1 presented above should be limited to a maximum stress of 80 ksi. This stress-strain model is presented in Figure 2 as Model 2. Figure 2 also includes experimental results from a tensile test of a No. 6 MMFX bar for comparison purposes[6].

This paper presents the strength, ductility and failure mode of beams reinforced with MMFX steel using each of the three models presented above to determine the validity and accuracy of predicting the behavior of beams reinforced with MMFX in comparison to results obtained experimentally.

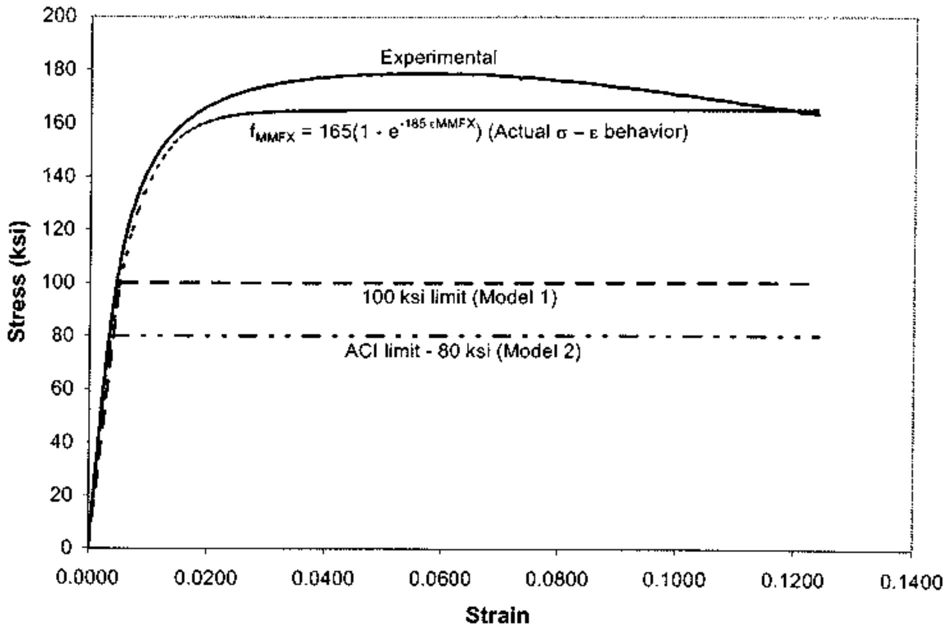

Figure 2: Comparison of Proposed MMFX Material Models

CRACKED SECTION ANALYSIS

A cracked section analysis was conducted to study the behavior of concrete beams reinforced with MMFX steel using the actual behavior and the two models discussed in Figure 2. Equilibrium and compatibility were used to determine how the different material models affect the moment capacity and ductility of sections reinforced with MMFX steel. The results were compared with experimental results reported by other investigators to determine the validity of the models in predicting the flexural behavior of beams.

Section Strength

Consider a concrete cross section as shown in Figure 3. At failure, the strain at the extreme concrete compression surface is to reach a value of 0.003. For a given reinforcement ratio, ρ, and concrete strength f_c' the initial location of the neutral axis, c/d, can be estimated. Consequently, the corresponding strain in the MMFX reinforcement, ε_{MMFX}, can be determined. From the strain in the reinforcement, each of the three material models discussed above can be used to calculate the stress in the reinforcement. Equating tension and compression forces using the ACI stress block factors yields:

$$0.85 f_c' \beta_1 \frac{c}{d} = f_{MMFX} \rho \qquad (2)$$

in which f_{MMFX} is the stress in the MMFX reinforcement at failure of the section corresponding to the strain in the reinforcement using each of the three material models discussed previously.

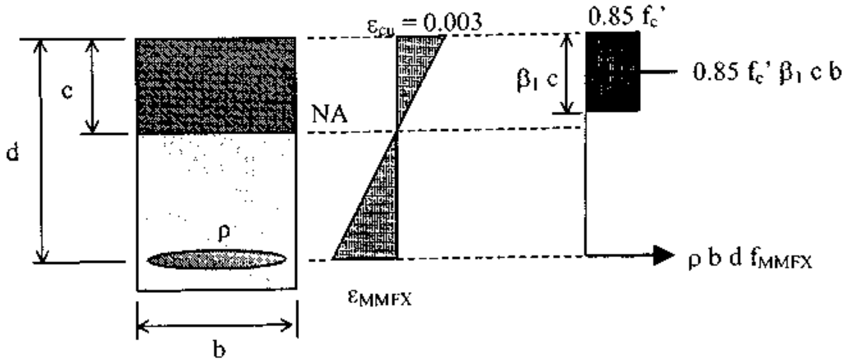

Figure 3: Equilibrium and Compatibility of an MMFX Reinforced Concrete Section at Failure

By iteration, the ratio of the neutral axis depth to the effective depth, c/d, can be determined based on compatibility and equilibrium of the section. The nominal flexural resistance of a concrete section reinforced with MMFX, M_n, can be determined using equation 3.

$$\frac{M_n}{bd^2} = \rho f_{MMFX}\left(1 - 0.5\frac{\beta_1 c}{d}\right) \quad (3)$$

where: f_{MMFX} is the stress in the MMFX based on any of the three models and the strain satisfying equilibrium and compatibility of the section
β_1 is a stress block factor as a function of concrete strength[7]

For each of the three material models being considered and a concrete strength, f_c', of 6500 psi the above relationship is plotted in Figure 4. Experimental results obtained at the University of North Florida (UNF) and Florida DOT are also plotted[8,9]. The above relationship is plotted for conventional Grade 60 steel reinforcement for comparison purposes. Also, the minimum reinforcement ratios using each of the three models as defined by ACI 318-02 clause 10.5 are presented as a reference[7].

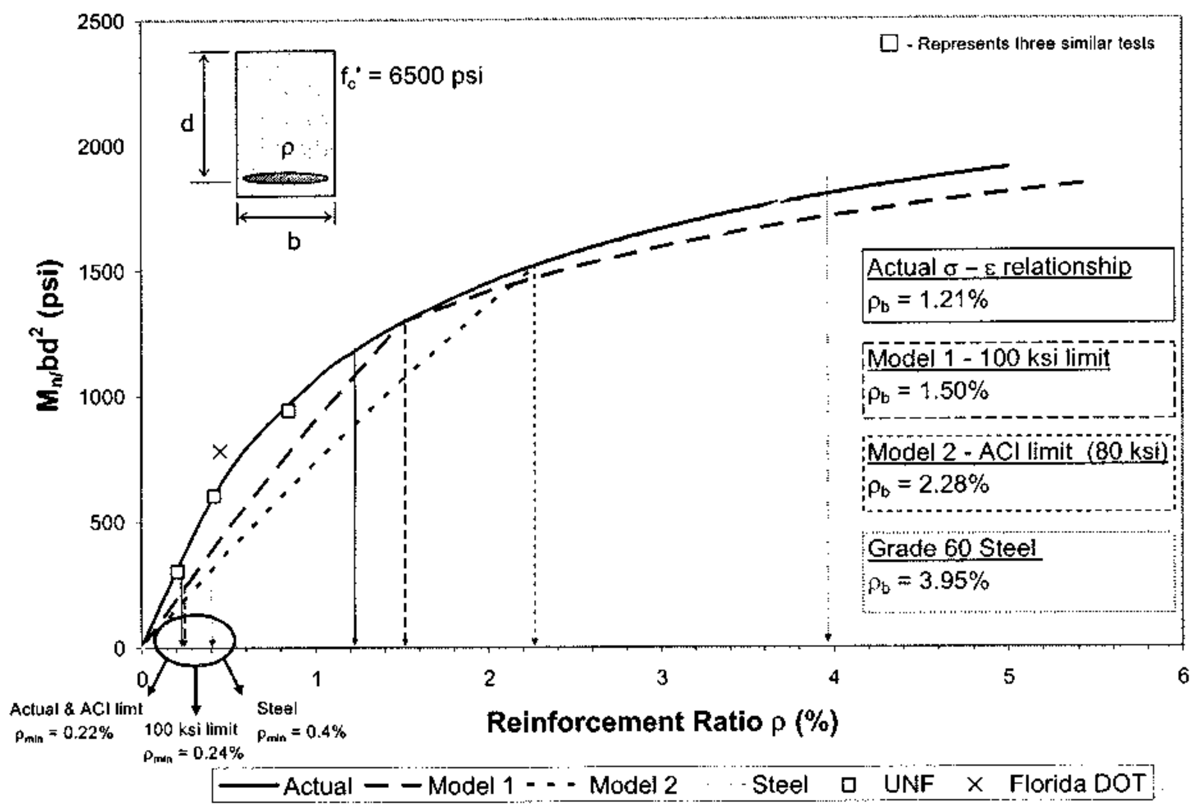

Figure 4: Effect of Stress-Strain Modeling of MMFX on Flexural Strength of Reinforced Concrete Sections

The following observations can be concluded from Figure 4:

1. Using the three stress-strain models for MMFX produces three values of balanced reinforcement ratio, ρ_b. The balanced ratio decreases as the specified yield strength for each model increases and the balanced ratio produced by all three stress-strain models of MMFX are lower than the balanced ratio of sections reinforced with Grade 60 steel.
2. The balanced reinforcement ratio provided by Model 1 is 1.5 percent. For higher reinforcement ratios the difference in the flexural strength of sections determined using the actual stress-strain relationship of MMFX and Model 1 is not significant. This is because the section behavior is governed by crushing of the concrete in both cases. For a reinforcement ratio less than 1.5 percent, however, use of Model 1 results in a lower flexural strength than the actual stress-strain behavior because of the reduction in the MMFX yield strength.
3. The balanced reinforcement ratio provided by Model 2 is 2.3 percent. For higher reinforcement ratios the difference in the flexural strength of sections determined using the actual stress-strain relationship of MMFX and Model 2 is not significant. This is because the section behavior is governed by crushing of the concrete in both cases. For a reinforcement ratio less than 2.3 percent, however, use of Model 2 results in a lower flexural strength than the actual stress-strain behavior because of the reduction in the MMFX yield strength.
4. Use of the actual stress-strain relationship for MMFX provides a balanced reinforcement ratio of 1.2 percent. Since the balanced ratios provided Models 1 and 2 are greater than 1.2 percent, both of these models will predict tension controlled failures for a range of reinforcement ratios while the actual behavior exhibits a compression controlled failure.
5. The actual MMFX stress-strain relationship most closely predicts the test results reported by UNF and Florida DOT. Models 1 and 2 significantly underestimated the flexural strength of the beams tested by the other researchers.

The difference in failure modes observed in Figure 4 may result in inaccurate prediction of ductility of beams reinforced with MMFX and could lead to insufficient warning prior to failure for these beams. This warrants a closer investigation of the ductility of sections reinforced with MMFX using the three different stress-strain models under consideration.

Section Ductility

The ductility of MMFX reinforced sections was examined by cracked section analysis using a similar procedure to that used for the strength analysis. For a given concrete strength and reinforcing ratio, and a series of strains at the extreme compression face of a section, c/d was iterated until equilibrium was satisfied. The concrete compression block was modeled using Hognestad's stress-strain curve with an ultimate strain of 0.003 and each of the three MMFX models were examined[10]. Figure 5 presents the moment-curvature diagrams for sections reinforced with MMFX with a reinforcement ratio of 1.2 percent, which corresponds to the balanced reinforcement ratio provided by the actual stress-strain behavior of MMFX, and a concrete compression strength of 6500 psi.

Figure 5 demonstrates that for a section reinforced with the actual balanced reinforcement ratio, Models 1 and 2 both predict considerably higher ultimate curvature than does the actual behavior. Using the definition of curvature ductility of ultimate curvature / yield curvature, MMFX stress-strain Models 1 and 2 predict ductilities of 2.5 and 1.3 respectively. The actual behavior, however, exhibits a balanced failure with a ductility of 1.0. This increase in ductility corresponds to an increase in the predicted deflection of beams and incorrectly suggests that the section exhibits sufficient warning prior to failure. Furthermore, the lower ductility predicted by the actual MMFX stress-strain relationship would necessitate use of a lower reduction factor due to the lack of warning prior to failure. For higher reinforcement ratios, the error in the prediction of ductility decreases.

For a reinforcement ratio of 0.5 percent, which is commonly used for design of reinforced concrete members, the actual stress strain behavior of MMFX predicts a ductility of 2.2. MMFX Model 1 and 2, however, predict much higher ductilities of 3.8 and 7.0 respectively.

From this discussion, it is evident that the various proposed limits on the tensile stress strain behavior of MMFX steel, while being conservative with respect to capacity, do not accurately predict the ductility of sections reinforced with MMFX steel. To correctly and accurately predict the ductility of MMFX reinforced concrete sections, the actual stress strain curve of MMFX in tension should be used for design. This necessitates a consideration of the resistance factor, ϕ, used in design of MMFX reinforced concrete members to ensure adequate warning prior to failure.

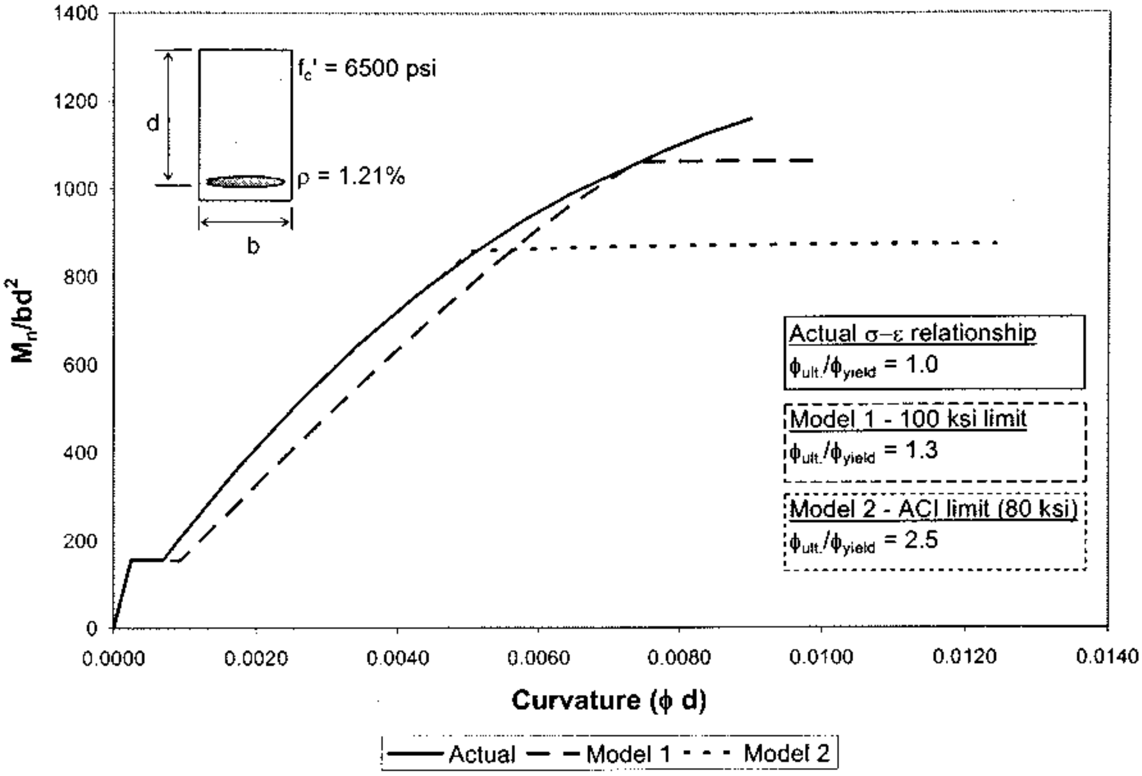

Figure 5: Effect of Stress-Strain Modeling of MMFX on Flexural Ductility of Reinforced Concrete Sections

EXPERIMENTAL VERIFICATION

The cracked section analysis presented above can easily be extended to predict the load-deflection behavior of any concrete beam member reinforced with MMFX steel. The moment-curvature relationship for any cross-section can be determined using cracked section analysis as presented above. The beam's moment diagram can be established by considering the support conditions and load configuration of the member. The moment-curvature relationship can then be superimposed on the moment diagram of the beam and the curvature diagram can be indirectly integrated to establish the load-deflection characteristics of the beam throughout the loading range.

In order to verify that the use of the actual MMFX stress-strain relationship accurately predicts the behavior of concrete beams reinforced with MMFX, the load deflection curve, as established by the above method, was compared to that determined experimentally by Yotakhong[11]. The cross section under consideration was 12" x 18" reinforced with 3 #6 MMFX bars at 15 ⅝" deep. The concrete strength was 7000 psi and the beam was tested in a four-point bending configuration with a 15 foot clear span and a 3 foot constant moment region centered along the beam length. The experimental and analytical load-deflection curves are presented in Figure 6.

As can be seen in the figure, the load deflection curve established using cracked section analysis and the actual stress-strain curve of MMFX steel accurately reflects the behavior observed experimentally. Results of the analysis coincide extremely well with the behavior determined experimentally. The cracked section analysis, however, predicts failure at a load of 72.8 kips with an ultimate deflection of 2.24 in. The tested beam failed at a load of 77.9 kips and a deflection at ultimate of 2.58 in. This discrepancy is due to the low ultimate strain value of 0.003 used to define failure of the concrete. If the cracked section analysis is extended to include higher concrete strains, the experimentally obtained ultimate load and deflection would be predicted very closely.

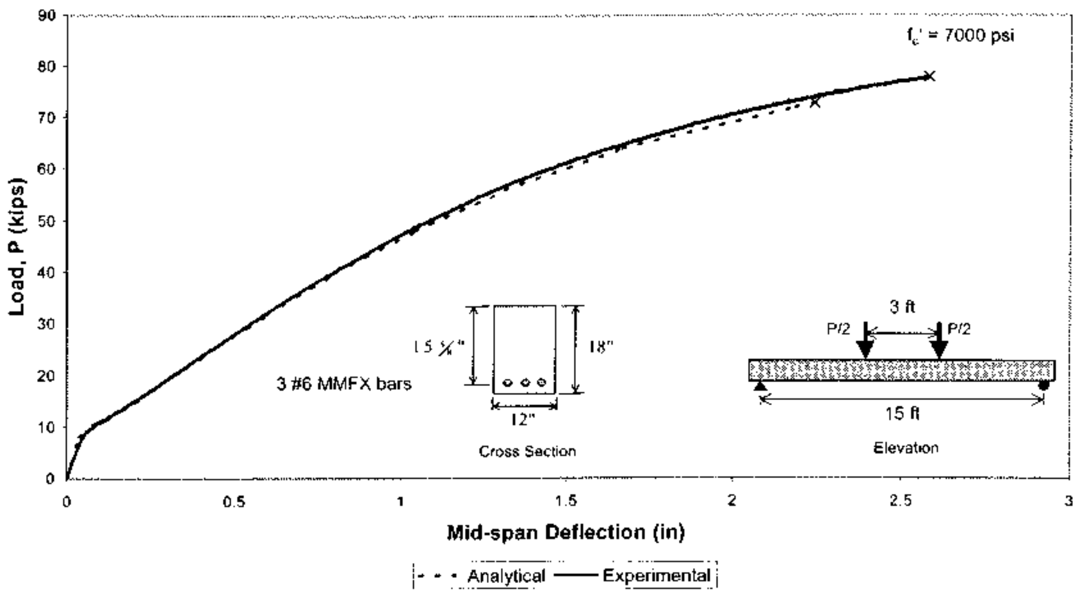

Figure 6: Load-Deflection Behavior of MMFX Reinforced Concrete Beam

DESIGN GUIDELINES

Resistance Factor

ACI 318-02 has included provisions to allow design of flexural members exhibiting both compression and tension controlled failures. Since sections exhibiting a compression controlled failure also exhibit lower ductility and provide inadequate warning prior to failure, ACI 318-02 Clause 9.3 requires that designers use a resistance factor, ϕ, of 0.65 for members with a failure strain in the reinforcement of less than 0.002. For tension controlled sections in which the reinforcement exhibits substantial plastic deformation prior to failure, those with a strain in the reinforcement greater than 0.005 at failure, a resistance factor, ϕ, of 0.9 is permitted. Between these limits, sections are considered to be in the transition region and the resistance factor varies linearly[7]. Since MMFX has a yield strain, ε_{MMFX} of 0.006, these limits need to be re-evaluated to ensure that designs of MMFX reinforced members also exhibit adequate ductility.

For a general reinforced concrete section and a specified reinforcement ratio, the moment-curvature relationship of the section can be established as discussed above. From the moment curvature relationship, the ductility, μ, of the section can be calculated as ϕ_u / ϕ_y where ϕ_u is the section curvature at ultimate and ϕ_y is the section curvature at yield. This procedure can be followed for a series of reinforcement ratios, for a given concrete strength.

Figure 7 presents the relationship between reinforcement ratio and ductility for sections reinforced with both steel and MMFX for a concrete strength of 6500 psi established using a series of reinforcement ratios. The figure demonstrates that sections reinforced with MMFX steel exhibit substantially lower ductility than do sections reinforced with conventional Grade 60 steel. To permit use of a resistance factor of 0.9, sections reinforced with MMFX should provide a ductility comparable to that of sections reinforced with conventional steel when the strain in the steel reinforcement at failure is equal to or higher than 0.005. As seen from Figure 7, this corresponds to a ductility, μ, of 2.24 and a failure strain in the MMFX reinforcement of 0.013 in/in. To maintain consistency with ACI 318-02, it is reasonable to use a reduction factor, ϕ, of 0.9 for sections reinforced with MMFX when the strain at failure in the section is 2.5 times the yield strain or 0.015 in/in. Thus, for MMFX reinforced sections, when using the actual stress-strain behavior of MMFX, for compression controlled failures, when the strain in the MMFX reinforcement at ultimate, $\varepsilon_{MMFX,u}$, is less than 0.006, a resistance factor, ϕ, of 0.65 should be used. For tension controlled failures, when the strain in the MMFX reinforcement at ultimate, $\varepsilon_{MMFX,u}$, is greater than 0.015, a resistance factor, ϕ, of 0.9 may be used. In the transition region between these two strain limits, ϕ should vary linearly between 0.65 and 0.9. These limits are presented graphically in Figure 8.

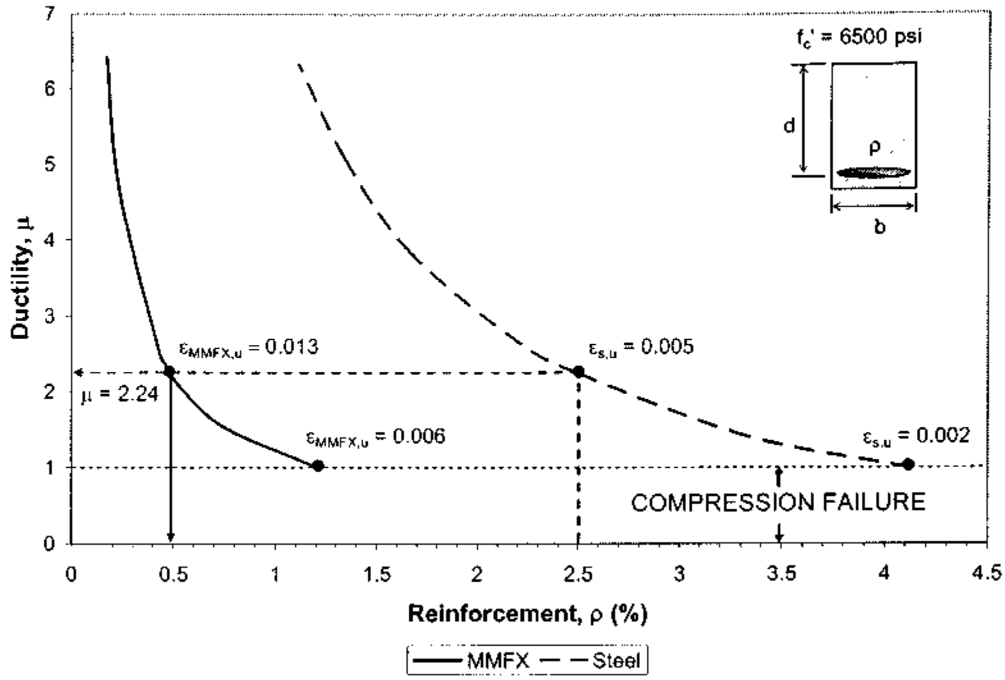

Figure 7: Effect of Reinforcement Ratio on Ductility of Reinforced Concrete Sections

The proposed strain limits for use with concrete members reinforced with MMFX are selected to ensure sufficient ductility and warning prior to failure of the members. At a reinforcement strain of 0.002 in/in, both members reinforced with conventional steel and those reinforced with MMFX may exhibit similar crack patterns and deflections. Beyond this strain conventional steel exhibits a pronounced yield plateau and substantial increases in crack width and deflection occur suddenly. MMFX, however, still maintains appreciable stiffness well beyond this strain limit and increases in crack width and deflection may occur much more gradually. Consequently, concrete sections reinforced with MMFX may exhibit larger crack widths and higher deflections at service load levels than do members reinforced with conventional steel. To provide adequate warning prior to failure, the proposed reduction factors can be used.

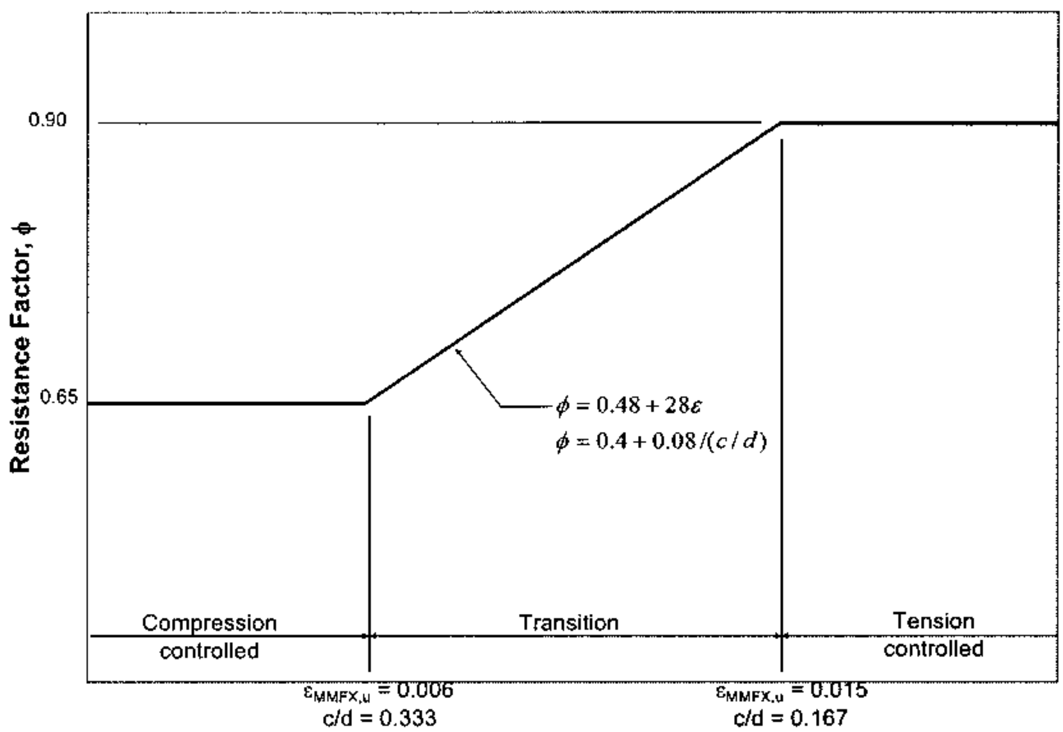

Figure 8: Resistance Factor for MMFX Reinforced Concrete Flexural Members

Design Chart

To assist design engineers and to facilitate the use of MMFX, the above resistance factors can be superimposed on the nominal capacity of sections reinforced with MMFX steel to develop a design chart for the use of MMFX reinforcing steel in reinforced concrete flexural members. Design charts were developed for MMFX reinforced concrete sections for concrete strengths of 3000 psi, 5000 psi, and 8000 psi and are presented in Figure 9. These concrete strengths are representative of the range of concrete strengths commonly used today. The minimum reinforcement ratios as recommended by ACI 318-02 Clause 10.5 are presented for information purposes. The design chart was developed using the actual MMFX stress-strain relationship.

The chart can be used to determine the MMFX reinforcement ratio for a given ultimate moment, M_u, in dimensionless form with the selected section width, b, and effective depth, d. The chart also accounts for the strength reduction factor, ϕ, for tension, T, transition, TR and compression, C, controlled failure modes. Alternatively, the designer may use this chart to determine the capacity of a given section with a known reinforcement ratio.

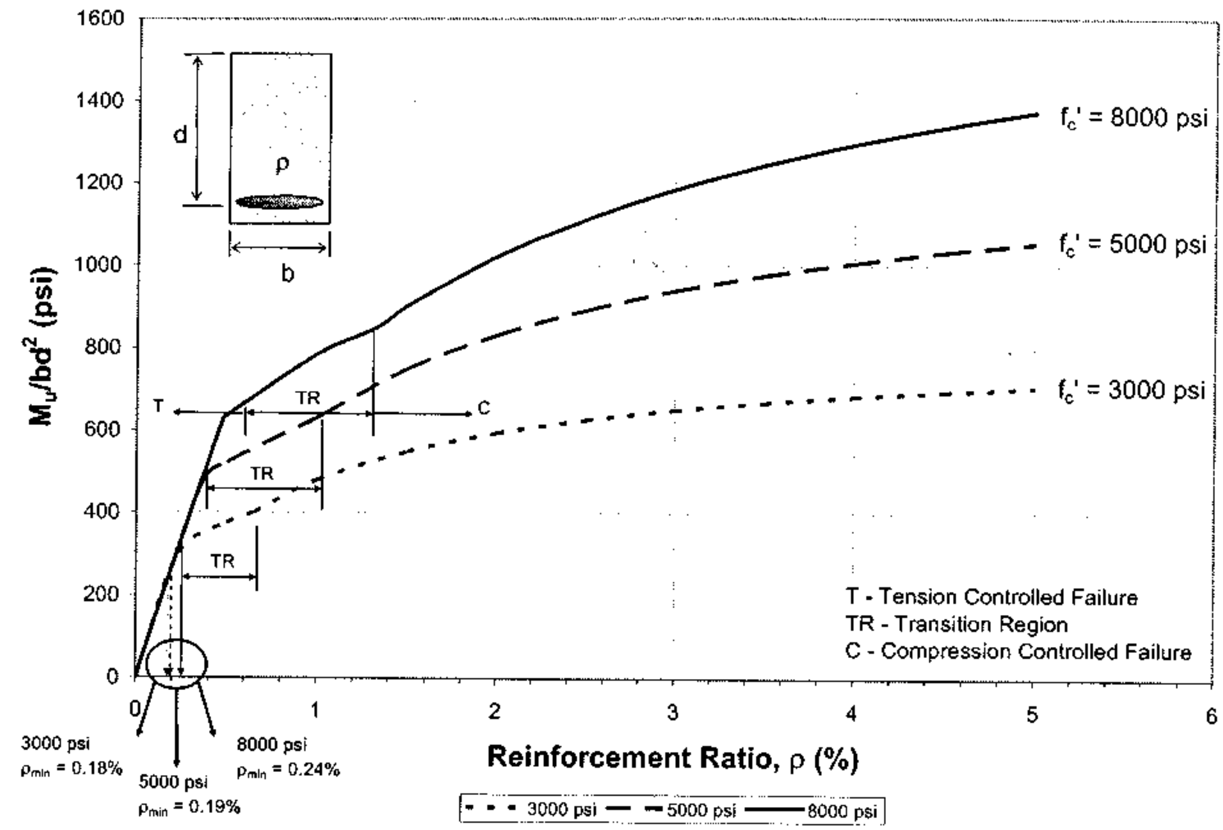

Figure 9: Design Chart for MMFX Reinforced Concrete Flexural Members

BRIDGE CASE STUDY

MMFX was used exclusively as reinforcement for the deck slab of a bridge under construction in North Carolina State located at I-95 and SR 1178 interchange at Four Oaks. The bridge is a skew bridge with a structural system composed of two simply supported steel girder spans of 120 feet and 126 feet, with the reinforced concrete deck running continuously over the two spans. The bridge cross-section consists of six built-up steel girders acting compositely with the reinforced concrete deck slab by means of shear studs as shown in Figure 10. The steel girders are 106.3" apart, with 15" wide by 1" thick flanges, and a 63" high by 0.6" thick web. A cross-bracing system was used every 21 feet through the entire bridge. The deck slab is 8.7" in thick and reinforced longitudinally and transversely with MMFX bars, as shown in Figure 11. The shear studs are 0.75" in diameter, 5" high, and spaced every 4" with three studs per row. Casting of the reinforced concrete deck slab is shown in Figure 12.

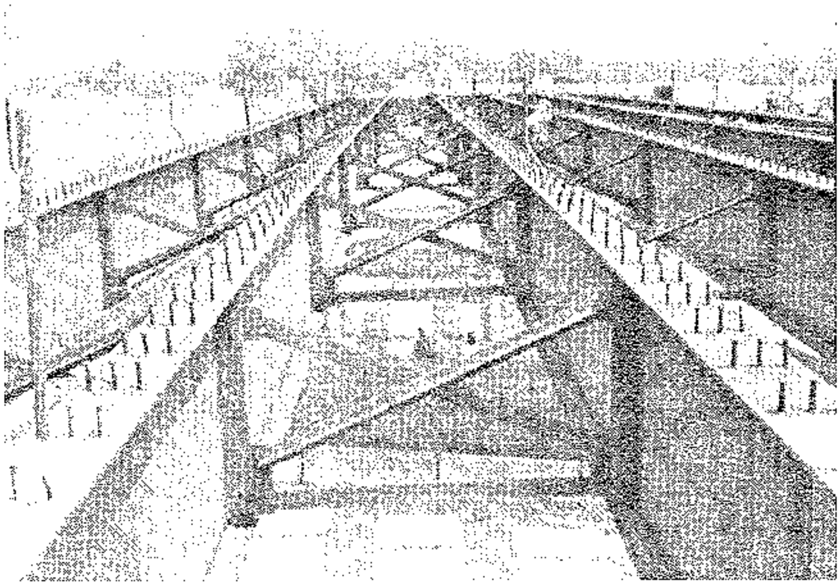

Figure 10: Cross Bracing and Shear Studs

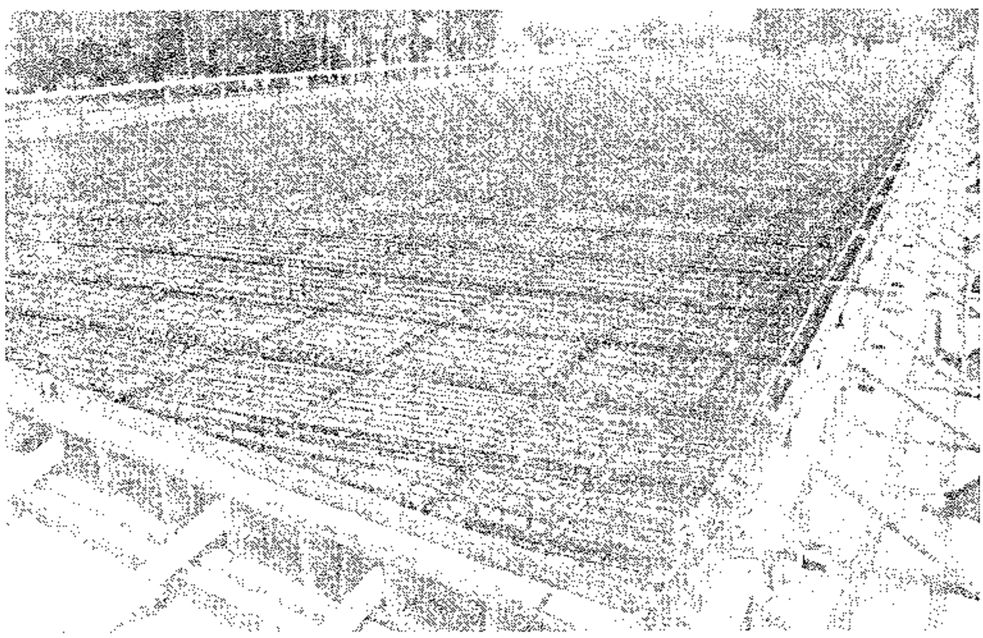

Figure 11: Deck Slab Reinforced with MMFX Steel

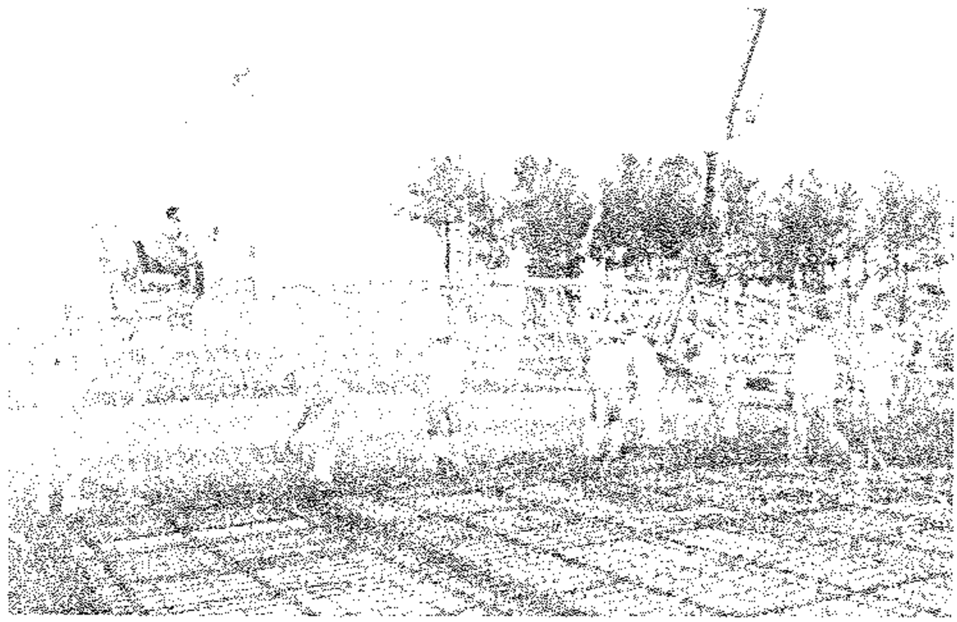

Figure 12: Casting of the Reinforced Concrete Deck Slab

CONCLUSION

MMFX is a high strength, corrosion resistant reinforcing steel that is ideally suited for use in civil engineering applications. Because the tensile stress strain relationship of MMFX is unlike that of conventional Grade 60 steel, several limitations have been placed on the material in an attempt to ensure safety and sufficient ductility. In considering concrete flexural members reinforced with MMFX steel, it has been demonstrated that the ACI limitation that steel reinforcing not exceed 80 ksi[7] and the MMFX Corporation's proposed idealized elasto-plastic model limited to 100 ksi[4], while being conservative with respect to strength, over estimate section ductility. As such, the actual stress-strain curve of MMFX in tension should be considered rather than following the ACI limitation or the 100 ksi limitation discussed.

Using the actual MMFX stress strain equation, and cracked section analysis, a procedure to predict the flexural behavior of sections reinforced with MMFX steel was discussed. Comparison of the load-deflection curve obtained using the cracked section analysis and the actual stress-strain behavior of MMFX with experimental results verifies the validity of the method to predict the load-deflection behavior of MMFX reinforced concrete beams.

Based on the findings of this paper, the resistance factor used for design of MMFX reinforced sections should be 0.65 for compression controlled sections with a strain in the MMFX reinforcement at ultimate, $\varepsilon_{MMFX,u}$, less than 0.006 in/in, 0.9 for tension controlled sections with a strain in the MMFX reinforcement at ultimate, $\varepsilon_{MMFX,u}$, greater than 0.015 in/in. Within the transition zone the resistance factor should vary linearly. These resistance factors are only to be used with the actual MMFX stress strain relationship and not with ACI limitation or the idealized elasto-plastic behavior proposed by MMFX Corporation.

This paper provides design charts for concrete sections reinforced with MMFX based on the actual MMFX stress-strain relationship and the proposed resistance factors for concrete strengths of 3,000 psi, 5,000 psi and 8,000 psi and reinforcement ratios up to 5 percent.

ACKNOWLEDGEMENTS

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STRENGTHENING REINFORCED CONCRETE SLAB-COLUMN CONNECTION USING CFRP SHEETS

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ABSTRACT: Due to the high cost of new construction and the increasing inventory of otherwise adequate structure, the need for repair or strengthening of reinforced-concrete structures is growing significantly worldwide. Fibre reinforced polymer technology has emerged during the past decade as a particular tool to upgrade existing structures. These materials are an excellent option for use as external reinforcement because of their light weight, resistance to corrosion, and high strength. This paper introduces a strengthening technique for slab-column connection in flat-slab structures to enhance its flexural strength. Six flat-slab column connections were fabricated and then tested under increasing gravity loading up-to-complete-collapse to investigate the effectiveness of using carbon fibre reinforced polymer (CFRP) sheets in enhancing their flexural capacity. The tested specimens consisted of two groups. The first group provided control specimens, on which three specimens had central column, eccentric column, and edge column, respectively. The second group was similar to the first group but with CFRP sheets installed around the columns on the tension side of the slab. Results from testing showed that the ultimate load carrying capacity increased by 32, 38 and 65 % for the tested specimens with central column, eccentric column, and edge column, respectively, when strengthened using CFRP sheets.

Keywords: Reinforced Concrete, Strengthening, CFRP, Experiments, Flat slab.

INTRODUCTION

Residential buildings, bridge decks, power stations and garages are usually composed of flat slab structures that are subjected to heavy loads. Cities and Municipalities depend on these structures to perform their activities and serve the public. The increasing age of these structures is becoming a major concern, especially when they are exposed to serve environmental conditions that cause corrosion of steel reinforcement embedded in concrete. A number of methods and precautions has been introduced to prevent corrosion of steel reinforcement. Among those methods are decreasing the permeability of concrete by using dense concrete, adding additives to concrete, coating concrete with impermeable layers, coating steel rebar with epoxy and cathodic protection of reinforcement, and providing thicker cover to the reinforcement. However, most of these protection measures increase the cost of the structure and do not prevent corrosion completely; rather they delay the problem, and they may even exacerbate the problem if not done properly. An ideal solution is to reinforce the slab by using fibre reinforced polymers (FRP) which is immune from corrosion. Of course, the new reinforcement must have adequate strength and stiffness and must be reasonably economical. It must also be able to withstand other environmental and short and long term loading effects. Among FRP types, carbon fibre reinforced polymers (CFRP) are known to be practically immune to chemical attacks and they have high strength and relatively high modulus. While durability and long-term performance of CFRP are reasonably well established, their ability to resist the bending moment at the slab-column connections needs investigation. This paper presents (i) a brief literature review pertained to strengthening slab structures; (ii) description of the experimental program on strengthening slab-column connections using CFRP sheets; and (iii) discussion of the results.

LITERATURE REVIEW

Few authors dealt with strengthening flat-slab structures. Abdel Rahman (2001) reported strengthening, using carbon fibre reinforced polymer strips, of a multi-storey building structure that was originally deigned as a residential building, 24 x 80 m in plan and made of flat slab supported on columns. The paper presents the design concept and constructional details of the application of carbon fibre reinforced polymer laminates in strengthening reinforced concrete flat-plate structure. El-Salakawy et al. (2002) presented the results of testing four full-scale specimens representing slab-column edge connections to examine their punching shear rehabilitation. Three slabs were tested to failure and then repaired, strengthened and tested again. In the originally tested slabs, which were chosen for repair, the first slab had an opening in front of the column and contained shear reinforcement, while the second slab had no opening and no shear reinforcement. The third slab had a 150 x 150 mm opening and no shear reinforcement. The slabs were repaired by replacing old damaged concrete by a new concrete of the same properties. Strengthening was carried out using shear studs for the two slabs, which originally did not have shear reinforcement. Vatovec et al. (2002) investigated the evaluation of carbon fibre reinforced polymer strengthening of an existing garage. Since 1995, carbon fibre reinforced polymers have been applied to strengthen concrete decks of a troubled posttensioned garage in Atlanta, USA. During the construction of the garage, design deficiencies were found. A remedial repair, involving heavily reinforced, 76 mm thick Gunite (Shotcrete) beams, was applied to the underside of the slab between drop panels in the east-west direction. Since then, delamination of Gunite beams and other structural problems repeatedly occurred. Epoxy injection and other limited repairs were done over the years in an attempt to remediate the problems. In 2000, due to the growing delamination concerns, backed up with nondestructive impact-echo testing results, and due to proven additional design deficiencies, a new and comprehensive remedial program was developed. The first phase of this program included an in-depth mechanical in-situ load test program to study the strength and the stiffness performance of the existing typical slab spans, including the effects of Gunite beams, the loss of Gunite beams due to delamination, and the CFRP strengthening of spans. The tests showed that the CFRP repair of the East-West spans with delaminated Gunite beams was warranted and that it performed well. Ebead and Marzouk (2002) predicted strengthening of two-way slab-to-column connections subjected to moment and cyclic loading. The strengthening steel plates were extended to twice the slab depth around the column. Steel bolts were used as vertical shear reinforcement. Eight bolts were believed ot be sufficient to transfer the horizontal forces from the steel plates. Binici and Bayrak (2003) presented a strengthening technique for increasing punching shear resistance in reinforced concrete flat plates using carbon fibre reinforced polymers. This strengthening method employed carbon fibre reinforced polymers in the vertical direction as shear reinforcement around the concentrated load area in a specified pattern.

EXPERIMENTAL PROGRAM

The experimental program included testing up-to-failure six specimens forming two groups. The first group consisted of three specimens with standard concrete slab-column connection system without Carbon Fibre Reinforced Polymer (CFRP) strengthening, while the second group was identical to the first group but with strengthening using CFRP sheets. Each tested specimen was composed of 2000 mm x 1000 mm x 150 mm slab, with 200 mm x 200 mm column-stub extending above the slab by 750 mm and below the slab by 400 mm. The dimension of the slabs and the column-stub were kept unchanged for all specimens, however the location of the column-stub was variable with the short direction of the specimens. These column locations were identified herein as central column for specimen S-1, eccentric column for specimen S-2 and edge column for specimen S-3. Specimens S-4, S-5, and S-6 are identical to specimens S-1, S-2 and S-3, respectively, except that the former specimens were strengthened in the tension side of the slab around columns using CFRP sheets. Figures 1, 2 and 3 show the dimensioning for specimens S-4, S-5 and S-6, respectively. High-early strength concrete, with a specified compressive strength of 35 MPa after seven days, was used. The concrete was ordered in two batches. The first batch was used in casting specimens S-1, S-2 and S-4, while the second batch was used to cast specimens S-3, S-5 and S-6. The results of testing concrete cylinders to determine the compressive and splitting strength of concrete at the time of testing are shown in Table 1. Steel reinforcing rebars, which were used for reinforcing the six slabs and all their column stubs, were # 10 rebars (diameter = 11.3 mm) with a yield strength of 400 MPa and modulus of elasticity of 200 GPa. Two meshes of rebars were used near the top and bottom surfaces of the slabs at equal spacing of 100 mm and clear concrete cover of 25 mm. Four rebars, one at each corner of the cross-section, were used to reinforce the column stubs. Figure 7 shows view of the steel reinforcement and wooden formwork for specimen S-1.

Table 1. Specimen Characteristics

Group	*Specimen No.*	*Column Location*	*Strengthening using CFRP*	*Concrete compressive strength (MPa)*	*Concrete splitting strength (MPa)*
I	S-1	Central	No	40.1	3.9
	S-2	Eccentric	No	40.1	3.9
	S-3	Edge	No	39.2	3.7
II	S-4	Central	Yes	40.1	3.9
	S-5	Eccentric	Yes	39.2	3.7
	S-6	Edge	Yes	39.2	3.7

Table 2. Tyfo SCH-41S System Properties

Description	Elongation percent	*Ultimate tensile strength, FRP*	*Tensile modulus, E*	*Composite thickness*
Primary carbon fibre, 0°; aramid fibre 90°	1.21%	876 MPa	72.4 GPa	1.0 mm per layer

Table 3. Epoxy Material Properties

Tensile strength	*Tensile modulus*	*Elongation percent*	*Flexural strength*	*Flexural modulus*
72.4 MPa	3.18 GPa	5.0 %	123.4 MPa	3.12 GPa

After concrete hardening, Specimens S-4, S-5 and S-6 were strengthened using CFRP sheets. The CFRP reinforcement material, which was applied horizontally to the columns and longitudinally and transversely along the slab, were Tyfo SCH-41S Composite Fibre System. This system consists of Tyfo S Epoxy and Tyfo SVH-41S reinforcing fabrics. The Tyfo SCH-41S is a unidirectional carbon fabric with aramid cross fibres. It has been stitched, with the carbon material oriented in 0° direction, and aramid fibres at 90°. The system properties are summarized in the Tables 2 and 3. The CFRP sheets were applied as recommended by the manufacturer. The carbon fibre reinforced polymer sheets were glued in the tension side of the slab per the sequence shown in Figures 1, 2 and 3 for specimens S-4, S-5, and S-6. Figures 4, 5 and 6 show details of the CFRP sheets used and the sequence of installation, for specimens S-4 with central column, S-5 with eccentric column and S-6 with edge column, respectively. It can be observed that the ends of the CFRP sheets in the shorter direction of the slab in specimen S-5 were bent vertically and glued to the vertical side of the slab, as shown in Figure 5. Similar details to those used in S-5 are utilized in specimen S-6 as shown in Figure 6. To facilitate testing the slab-column connection, each specimen was inverted by setting the slab corners over point supports, as shown in Figures 9 and 10, in such a way that the lower surface of the slab is the tension side on which CFRP were installed. The jacking load was applied over the column stub as shown in Figures 9 and 10.

Figure 8 shows view of specimen S-1 during gluing the CFRP sheets. The following steps was considered in each test: (1) the specimen was accurately placed over the four supports; (2) LVDTs were put in their marked position to measure deflections; (3) steel strain gauges, concrete strain gauges, LVDTs, and load cell were connected to the data acquisition panel; (4) the jacking load was applied in increments by means of a hydraulic jack connected to a 450 kN load cell placed on the upper column stub, as shown in Figures 9 and 10.

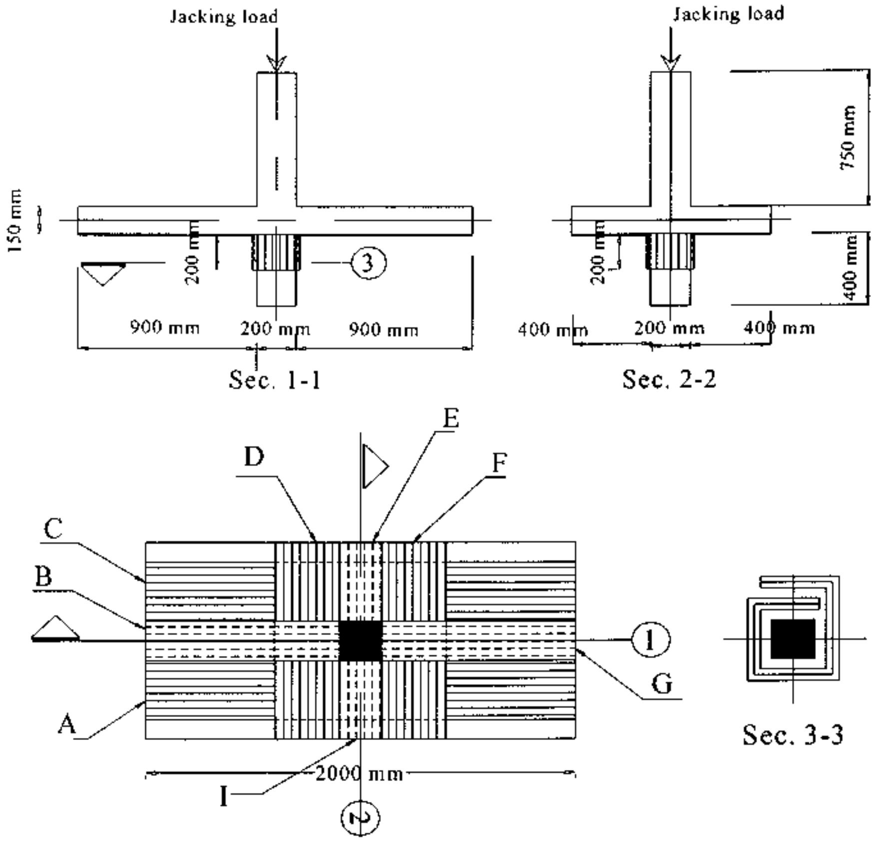


Figure 1. Geometry of Specimen S-4 with CFRP arrangement

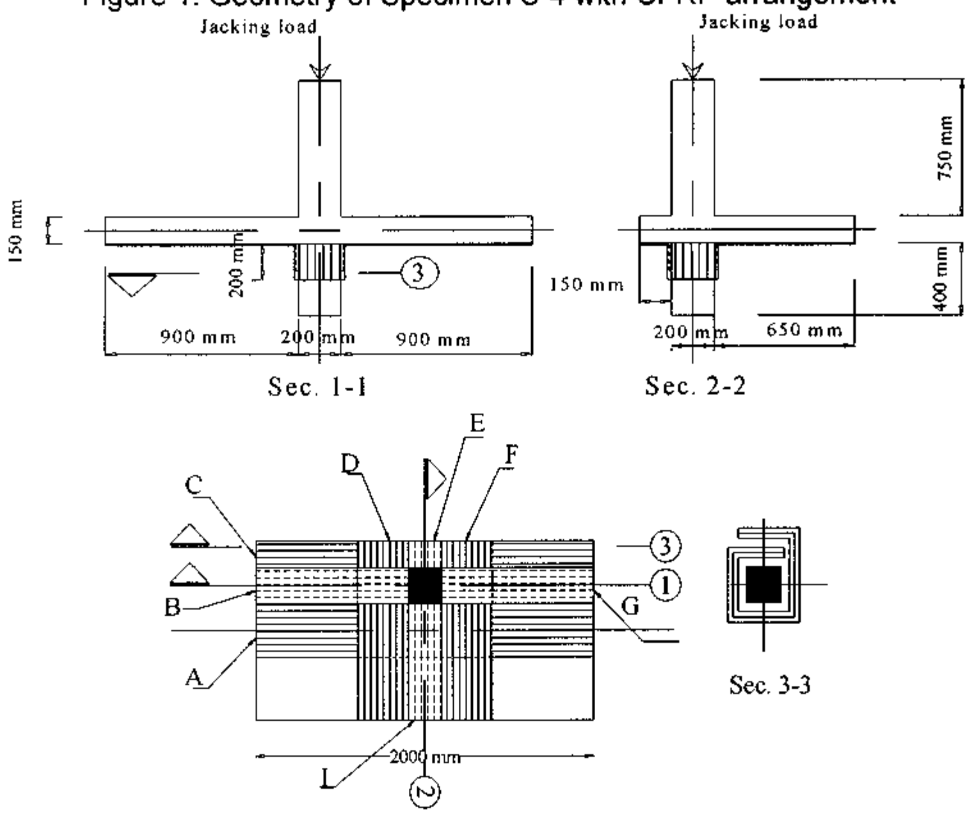


Figure 2: Geometry of Specimen S-5 with CFRP Arrangement

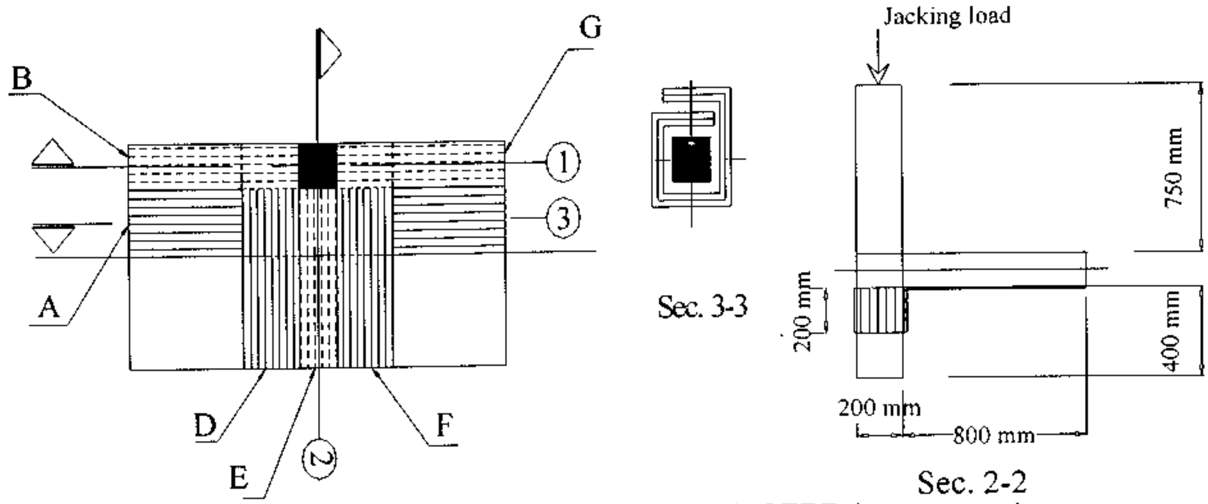


Figure 3. Geometry of Specimen S-6 with CFRP Arrangement

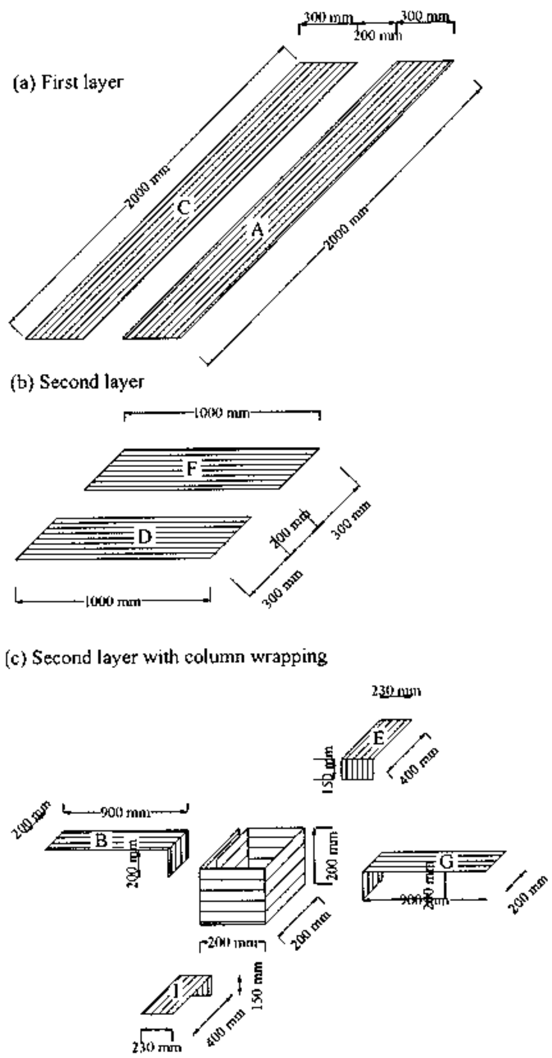


Figure 4. CFRP details for Specimen S-4

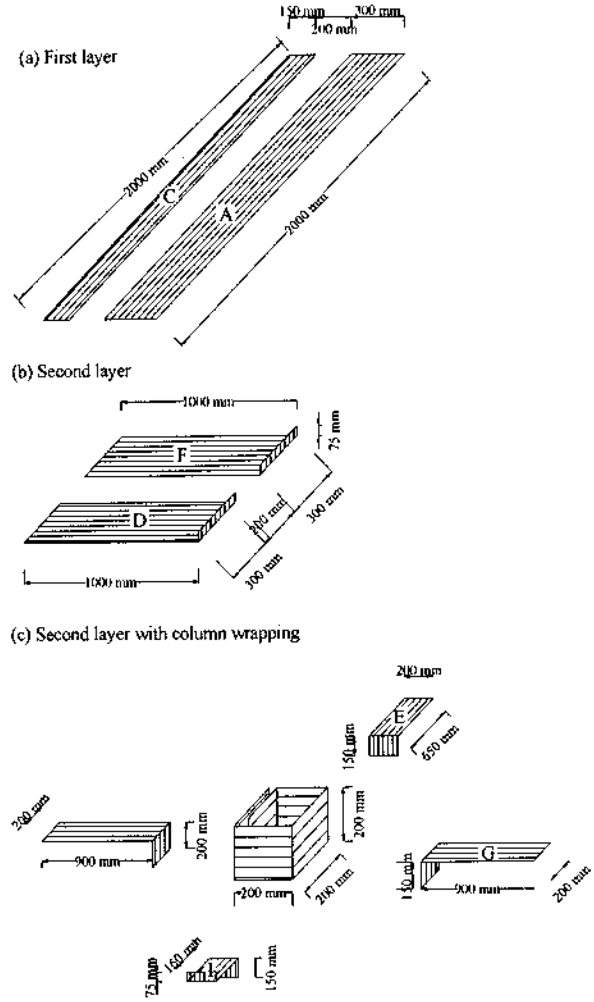


Figure 5. CFRP details for Specimen S-5

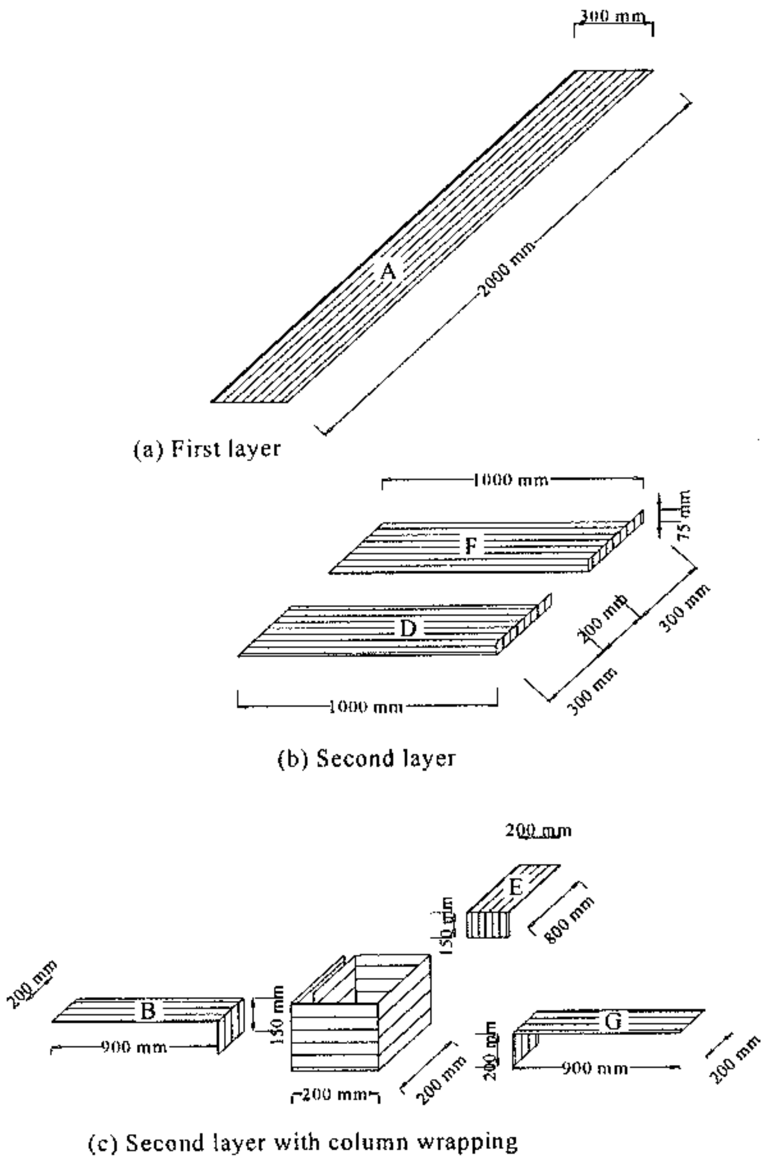


Figure 6. Details of CFRP Installation for Specimen S-6

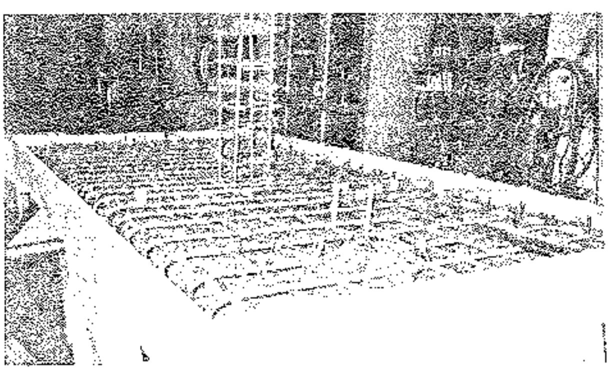


Figure 7. View of the Steel Mesh and Formwork for Specimen S-1

Figure 8. First CFRP Layer for Specimen S-4

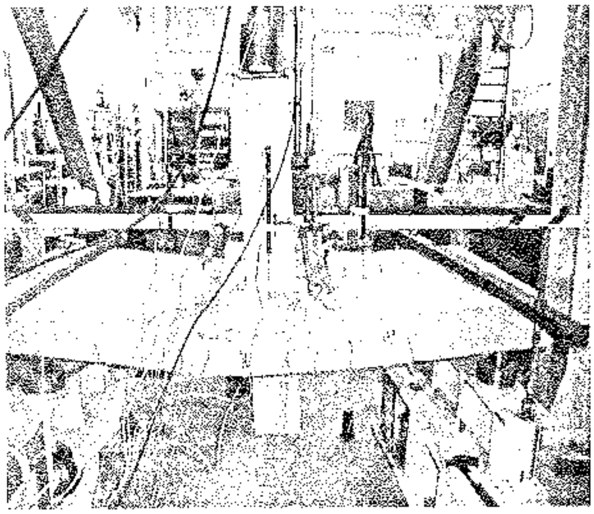


Figure 9: View of the Deflected Shape of Specimen S-1 after Failure

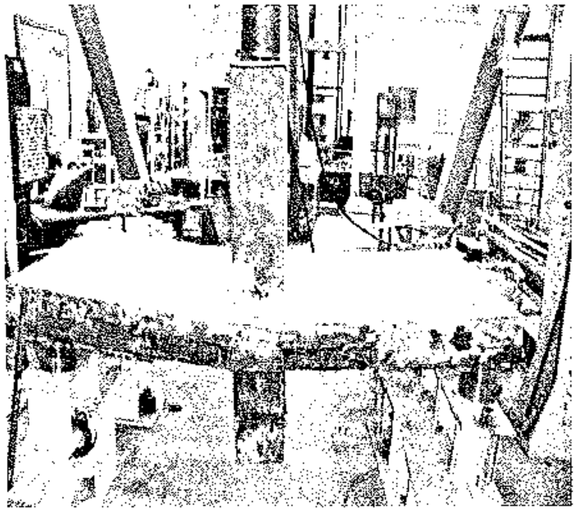


Figure 10: View of the Deflected shape of Specimen S-5 after Failure

RESULTS

The following subsections explain the load history, strain gauge readings, deflection and ultimate load carrying capacity of the tested specimens.

Load History

In specimen S-1 with central column, the first visible crack was observed at a jacking load of 125 kN. This crack appeared at the bottom surface (tension side) of the specimens at mid-span location. With the increase of the applied load, more tension cracks were observed away from the column location. These bottom cracks penetrated the slab thickness towards the compression side (top surface of the specimen) with increase in the applied load. Crack propagations on the vertical side of the free edge of the long direction of the slab were also observed. With increase in the applied load, slab deflections and strains were observed to be increasing and concrete cracks on the vertical side of long direction of the slabs continued to widen and penetrate into the slab thickness till concrete crushed at the top surface of the specimen at the mid-span location along a line parallel to the short direction of the slab at 164 kN. Figure 9 shows the permanent deflected shape of specimen S-1 after failure. Specimen S-2 exhibited crack pattern similar to that for specimen S-1, but with cracking load of 90 kN and ultimate load of 151 kN. The first visible crack was observed in specimen S-3 at a jacking load of 67 kN. This crack appeared at the bottom surface of the specimen at mid-span and propagated on the vertical side of the free edge of the slab toward its top surface. With increase of the applied load, the slab deflections and the strains were observed to increase and concrete cracks on the vertical side of the long direction of the slab continued to widen and penetrate into the slab thickness till concrete crushed at the top surface of the specimen at the mid-span location at 113 kN. At a jacking load of 154 kN, the first visible crack was observed in specimen S-4. This crack appeared at the bottom surface of the specimens at mid-span location. With increase in the applied load, more tension cracks were observed away from the column location. These bottom cracks penetrated the slab thickness towards the compression side (top surface of the specimen) with increase in the applied load. With increase in the applied load, slab deflections and strains were observed to increase and concrete cracks on the vertical side of the long direction of the slabs continued to widen and penetrate into the slab thickness till concrete crushed at the top surface of the specimen at the mid-span location at 217 kN. Similar crack pattern was observed in specimen S-5, but with a cracking load of 125 kN and ultimate load of 208 kN. Figure 10 shows the permanent deflected shape of specimen S-5 after failure. In specimen S-6, the first visible crack was observed at a jacking load of 105 kN. This crack appeared at the bottom surface of the specimens at mid-span location. With the increase of the applied load, more tension cracks were observed away from the column location. These bottom cracks penetrated the slab thickness towards the compression side (top surface of the specimen) with increase in the applied load. Cracks were observed to widen and penetrate into the slab thickness till concrete crushed at the top surface of the specimen at the mid-span location at 187 kN. Table 4 summarizes the cracking and ultimate loads obtained experimentally for all the specimens.

Figures 11 and 12 show the effect of column eccentricity on the value of the observed cracking load and the recorded ultimate load carrying capacity of the tested slabs. Figure 11 shows that with increase in column eccentricity, the cracking load decreases. Also, it was observed, from Figure 12 that the ultimate load carrying capacity of the slab-column connection decreased by about 8% for eccentric column specimen and about 31% for the edge-column specimens, when compared to that for the central-column specimen. Similar trend was observed in Figures 11, 12 for specimens strengthened using CFRP sheets. However, the changes in cracking load and ultimate load carrying capacity of the slab were observed to be less in case of specimens strengthened with CFRP sheets. Figures 11, 12 also depict a comparison between specimens with and without CFRP strengthening. The cracking load observed visually was shown to increase with the presence of CFRP strengthening, as presented in Figure 11. The ultimate load carrying capacity of the slab-column connection increased by 32% for the central-column specimen, 38% for the eccentric-column specimen and 65% for the edge-column specimen when using CFRP sheets for strengthening, as shown in Figure 12.

Table 4. The load at first crack and the ultimate load for each specimen

Specimen	*First crack load (kN)*	*Ultimate load (kN)*
S-1	125	164
S-2	90	151
S-3	67	113
S-4	154	217
S-5	125	208
S-6	105	187

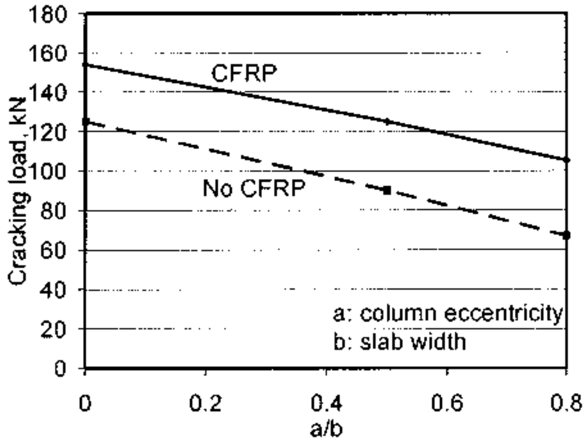


Figure 11: Cracking Loads

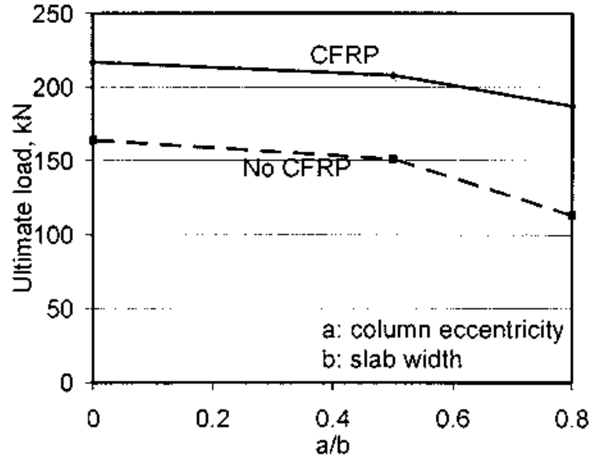
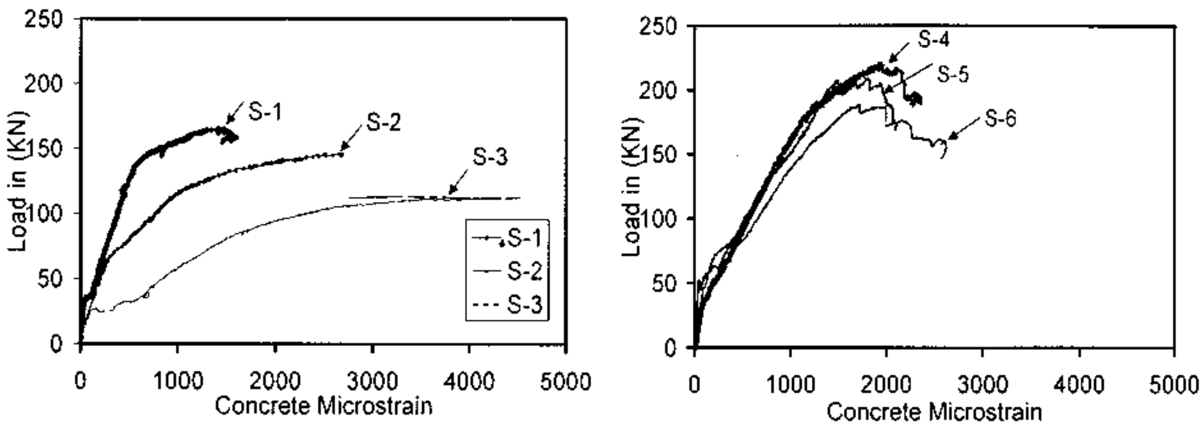


Figure 12: Ultimate Loads

Strain Measurements

Concrete normal strains were measured at certain locations along the slab top surface (the compression side) of the six specimens. Figure 13 presents the load-concrete strain relationship at the column corner in the long direction of the control specimens S-1, S-2, and S-3 (the control specimens) and for specimens S-4, S-5 and S-6 (strengthened using CFRP sheets). It can generally be observed that at any load increment, the concrete strain decreased as a result of the presence of the CFRP sheets. This may be attributed to the contribution of the CFRP sheets in increasing slab strength to applied loads. Results of concrete and steel strain readings in the short direction of the slab, not shown here, showed that strain readings in the short direction were much less than those in the long direction of the slab. This may be attributed to the chosen aspect ratio of the slab that represents floor panels with high aspect ratio. However, it was observed that strain readings for specimens strengthened with CFRP sheets were less than those of the corresponding control specimens.

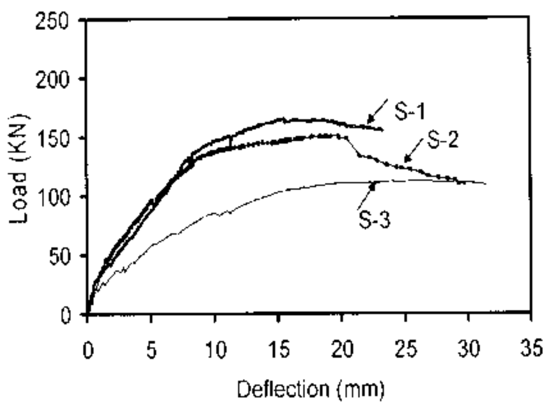


a) Specimens S-1 to S-3 b) Specimens S-4 to S-6

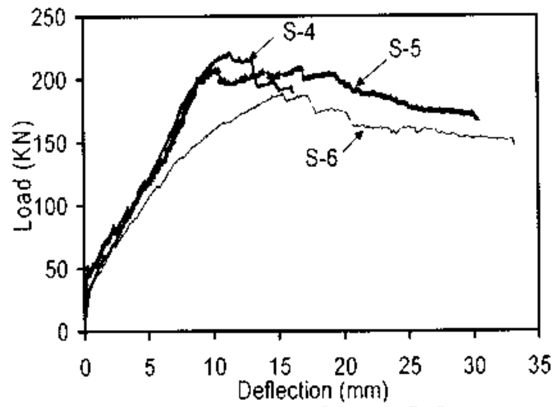
Figure 13. Load-Concrete Strain Relationships for the Tested Specimens

Deflection Measurements

Deflections were measured at certain locations along the slab of the six specimens. Figure 14 presents the load-deflection relationships for the tested specimens, at a point in the slab along the slab longitudinal axis passing through the column centroid and 400 mm away from it. It can be generally observed that at a certain load level, deflection decreased as a result of the presence of the CFRP sheets. This may be attributed to the contribution of the CFRP sheets in increasing slab stiffness. Figure 15 shows the effect of column eccentricity on the maximum deflection at failure. Compared to maximum deflection of specimen S-3 with central column, It can be observed that the maximum deflection increased by 29% for eccentric column and 96% for the edge column. Similar trend was observed in Figure 15 for specimens strengthened using CFRP sheets. However, the changes in deflection and ultimate load carrying capacity of the slab were observed to be less in case of specimens strengthened with CFRP sheets. Figure 15 also depicts a comparison between specimens with and without CFRP strengthening. It was observed that the maximum deflection at failure decreased by 7% for central-column specimens, 11% for eccentric-column specimen, and 34% for edge-column specimen as a result of using CFRP sheets to strengthen the tension side of the slab at column location.



a) Specimens S-1 to S-3



b) Specimens S-4 to S-6

Figure 14. Load-Deflection Relationships for the Tested Specimens

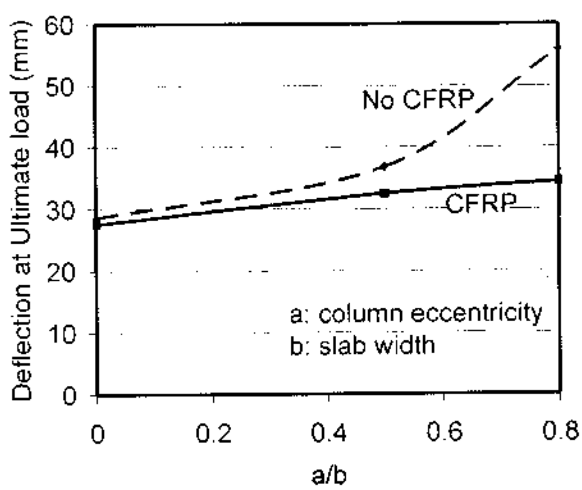


Figure 15. Deflection at Ultimate Load for the Tested Specimens

The flexural resistance (nominal moment M_n or theoretical flexural capacity) of the slab cross-section strengthened with bonded CFRP sheets can be calculated as follows based on equilibrium of forces and compatibility of strains as per *Canadian Standard for Design and Construction of Building Components with Fibre-Reinforced Polymers*, CSA-S806-02 (CSA, 2002).

$$M_n = A_f f_f\left(t - \frac{\beta_1 c}{2}\right) + A_s f_s\left(d_s - \frac{\beta_1 c}{2}\right) + A_{s1} f_{s1}\left(\frac{\beta_1 c}{2} - d_{s1}\right) + A_{s2} f_{s2}\left(d_{s2} - \frac{\beta_1 c}{2}\right) \quad (1)$$

Where A_f is the cross-sectional area of the CFRP sheets bonded to the tension side of the slab; A_s is the area of the steel reinforcement in the tension side of the section; A_{s1} is the area of the steel reinforcement in the compression side of the section; A_{s2} is the area of the steel reinforcement in the concrete deck; b is the width of the slab section; c is the distance from concrete top fibres to the neutral axis of the section; d_s is the depth of steel reinforcement in the tension side of the section at ultimate load; d_{s1} is the depth to the steel reinforcement in the compression side of the section at ultimate load; d_{s2} is the steel reinforcement in the concrete deck at ultimate load; ff is the stress in the CFRP sheets bonded to the tensions side of the slab at ultimate load; f_s is the stress in the steel reinforcement in the tension side of the section at ultimate load; f_{s1} is the stress in the steel reinforcement in the compression side of the section at ultimate load; f_{s2} is the stress in the steel reinforcement in the concrete block at ultimate load; and β_1 is the ratio of the equivalent compressive stress block depth to the depth to the neutral axis of the reinforced concrete section.

It should be noted that the above-mentioned equation can be easily used to determine the ultimate moment and hence the maximum load carrying capacity of one-way slab system similar to specimens S-1 and S-4, with central column and aspect ratio of 2. However, this equation overestimates the load carrying capacities of other slab specimens with eccentric or edge columns that collapsed with failure pattern similar to those for S-1 and S-4. An investigation is underway to determine a theoretical solution to the flexural load carrying capacity of slab-column connection with slab aspect ratios ranging from 1 to 2 and with different column eccentricities.

CONCLUSIONS

The experimental study presented in this paper supports the following conclusions: (i) the failure of all the specimens was of flexure type and ductile; (ii) The presence of CFRP significantly reduces slab deflection and increase both the cracking load and the ultimate load carrying capacity of the slab-column connection; and (iii) the ultimate load of the slab-column connection is affected by the eccentricity of the column; the smaller the eccentricity, the bigger the ultimate load carrying capacity and the cracking load.

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SEISMIC BEHAVIOR OF 3-D WALL PANEL SYSTEM

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ABSTRACT: Four half scale 3-D wall panels representing ground floor wall of a four story building were tested to investigate the seismic behavior and integrity of the system in lateral loads. The 3-D wall panel system consists of a three dimensional welded wire space frame integrated module with a polystyrene insulation core. The module is placed in position and shot-crete concrete is applied at both sides. The system is suitable for hot weather and rapid construction. The specimens were subjected to a quasi-static lateral displacement test to simulate seismic loading. The first specimen was considered as a reference, while the second and the fourth specimens were constructed with vertical and horizontal splices, respectively. The third specimen was constructed with an opening. The results shows that the 3-D system is capable to resist lateral loads successfully. The reinforcement details at the vertical and horizontal splices or around the wall opening were sufficient to keep the lateral resistance of the wall and to achieve system integrity.

Keywords: 3-D wall – seismic – shot Crete

INTRODUCTION

The 3-D wall panel system was introduced to the construction field in the U.S.A by 1989. The 3-D wall panel consists of a three-dimensional welded wire space frame integrated with a polystyrene insulation core. This reinforcement/insulation module is placed in position and concrete or mortar is applied to both sides as shown in Figure 1. The 3-D wall panel gains its strength and rigidity by the diagonal cross wires welded to the welded wire fabric mesh on each side. This produces very rigid truss behavior, which provides adequate shear transfer for composite behavior. The reinforcement/insulation modules are shop fabricated with highly automated equipment. This insures consistent dimensional control and high-quality welding. The standard 3-D panel is shown in Figure 2. Different thicknesses of insulation and concrete are available.

The 3-D wall panel can be used as integrated structural system consists of slabs and bearing walls or as structural walls and slabs elements. The 3-D bearing wall element can be subjected to concentric or eccentric (in plane or out of plane) vertical loads. The 3-D slab element can be subjected to bending moments and shearing forces. In this case, the 3-D panel should be provided with additional reinforcement according to the applied loads and spans.

The 3-D wall panel system was tested before. The tests were include lateral load and earthquake resistance tests[1]. Classical Compression and flexural tests were also investigated[2], [3].

The 3-D wall panel has to fulfill the following basic design assumptions:

- Adequate structural integrity of the building to resist the lateral loads (seismic or wind loads) should be achieved.
- The 3-D wall panel is subjected to vertical loads with in plane or out of plane eccentricities such that no tensile stresses are allowed.
- Under the combination of the vertical loads and the lateral loads no tensile stresses are allowed at the 3-D wall panel section as well as at the connection between the panel and the foundation.

The objectives of this research are to study the behavior of the 3-D panel system under seismic loads and to investigate the system integrity at splices and openings.

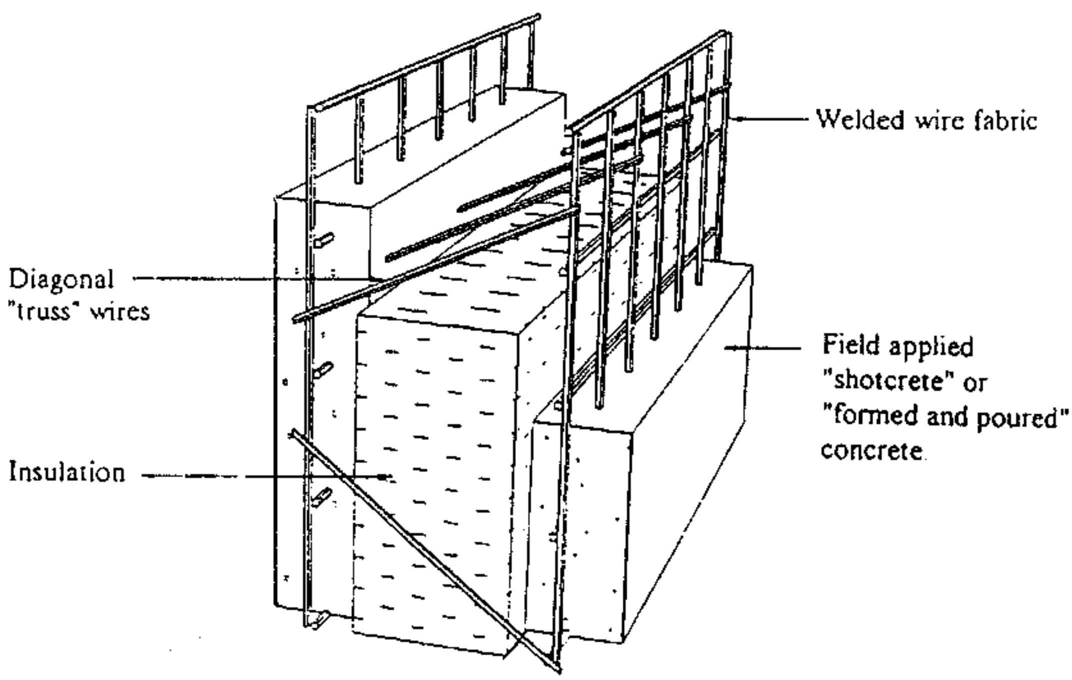

Figure 1. The 3-D Wall Panel

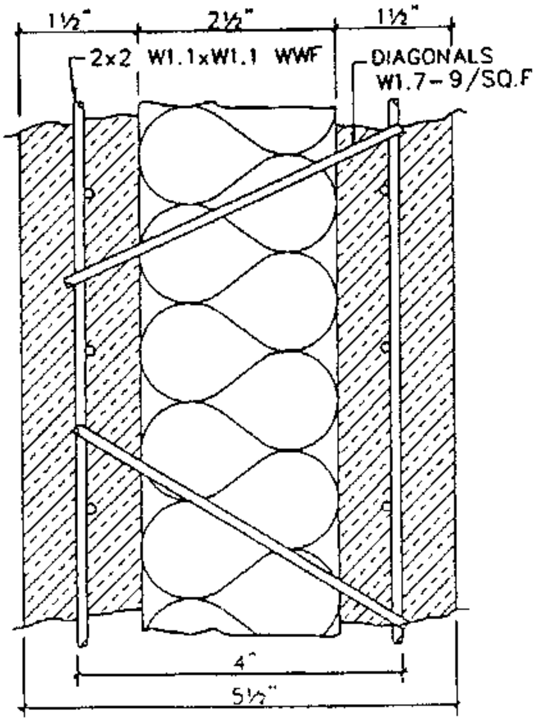

Figure 2. Standard 3-D Section

EXPERIMENTAL PROGRAM

Four half scale 3-D wall panels representing ground floor walls of a 4-story building were tested. The 3-D wall panels were prepared by October Company for Estate Investment and Development at the RC laboratory of Housing and Building Research Center (HBRC). The specimens were tested under the application of quasi-static horizontal displacements simulating seismic loads and constant vertical load. The following parameters were studied:

- The seismic behavior of the 3-D wall system
- The structural integrity of the wall with vertical splice
- The structural integrity of the wall with horizontal splice (at floor level)
- The behavior of the wall with opening

TEST SPECIMENS

All specimens were prepared using standard 3-D wall panel provided by October Company of module 1.20m width and 3.00m height. The thickness of the polystyrene was 10cm and the wire mesh at each side was ϕ 3mm spaced at 5cm. The two wire meshes was connected with ϕ 4mm cross-ties per 100cm^2. The thickness of each concrete layer was 5cm, then the overall thickness of the 3-D wall panel was 20cm. The 3-D wall panel had an I section in the plan, the web height was 120cm and the width of the flanges was 80cm as shown in Figure 3. The overall wall height was 160cm. The 3-D wall panel was rested at 60cm height RC base. All specimens were provided at the top with RC bad with 20cm height. Plate 1 Shows a general view of the tested specimen.

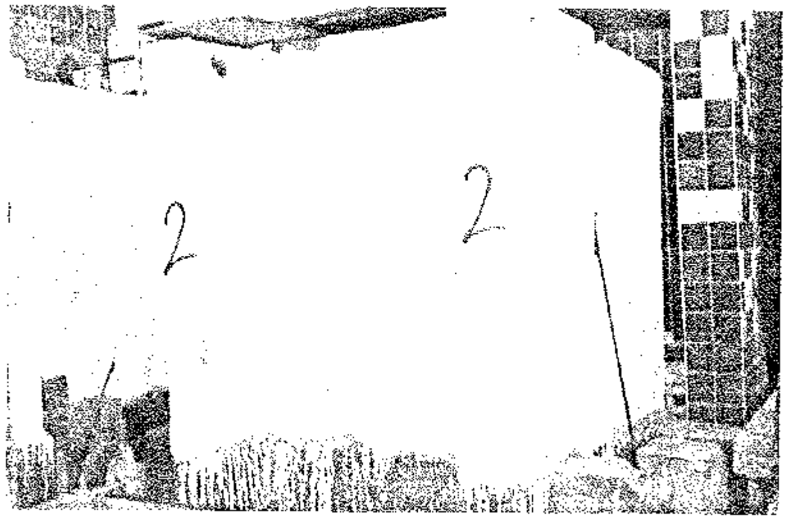

Plate 1. General view for specimen (W2)

PREPARATION OF SPECIMENS

After pouring the RC base, ϕ 12mm steel dowels with length of 50cm spaced at 40cm were drilled 10cm depth into the RC base and mounted with epoxy mortar. The 3-D wall panels were erected and kept in a vertical position. The concrete was pumped with 5cm thickness at both sides using a shotcrete. The characteristic strength of the concrete was 250 kg/cm^2. Then, The RC bad at the top of the 3-D wall panel was poured with concrete.

DESCRIPTION OF TESTED SPECIMENS

Four test specimens W1, W2, W3, and W4 were prepared as the following:

- Specimen W1: This specimen was prepared with the same dimension and details described above and is considered as the control specimen as shown in Figure 3.
- Specimen W2: The specimen was similar to specimen W1 but a vertical splice at the mid-width of the specimen was provided using a splice reinforcement mesh with 30cm width as shown in Figure 4. This splice represents the vertical connection between the wall panels.

- Specimen W3: The specimen was similar to specimen W1 but an opening 30cm width by 40cm height was executed as shown in Figure 5. This specimen represents the effect of openings on the behavior of the wall when subjected to seismic loads.

- Specimen W4: This specimen was similar to W1 but a horizontal splice at the mid-height was provided as shown in Figure 6a. This splice represents the connection of the walls at the floor level. The splice was provided with 45cm height wire mesh and dowels 2 ϕ 10mm spaced at 50cm as shown in Figure 6b.

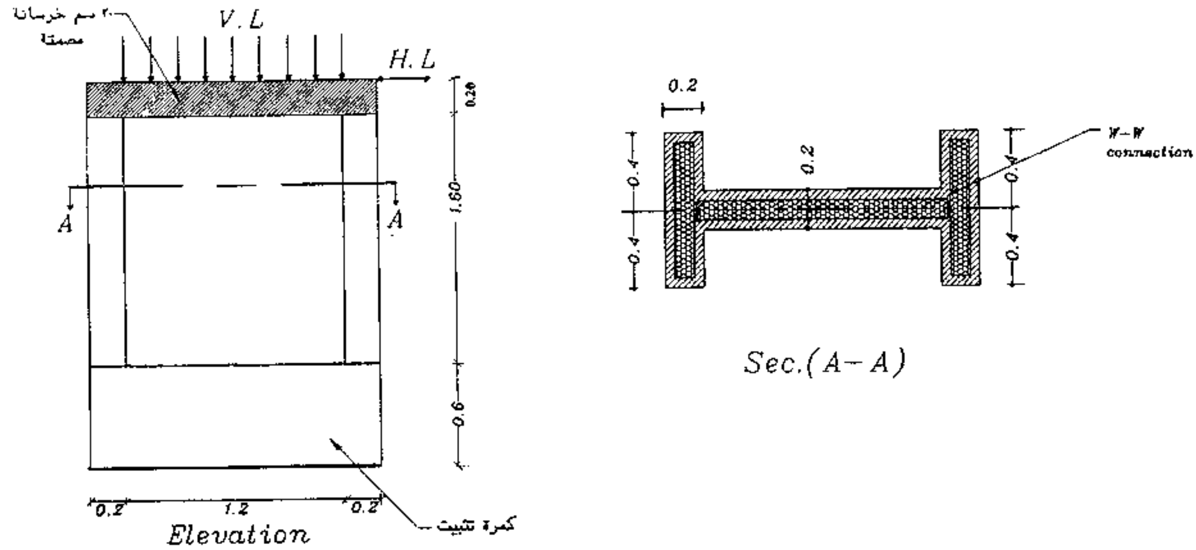

Figure 3. Dimensions of Specimen (W1)

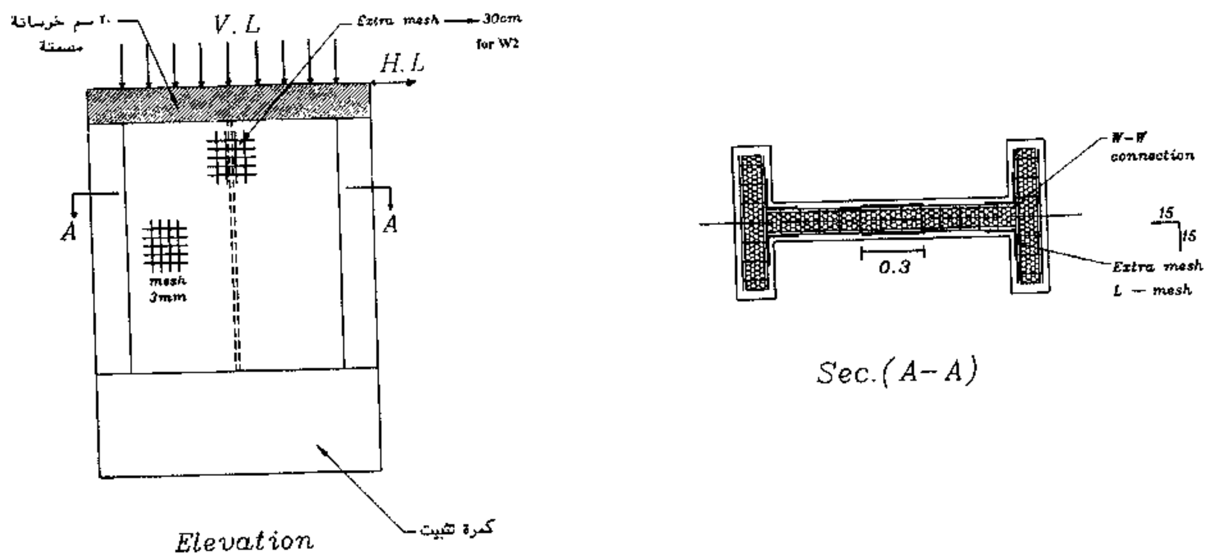

Figure 4. Dimensions of Specimen (W2)

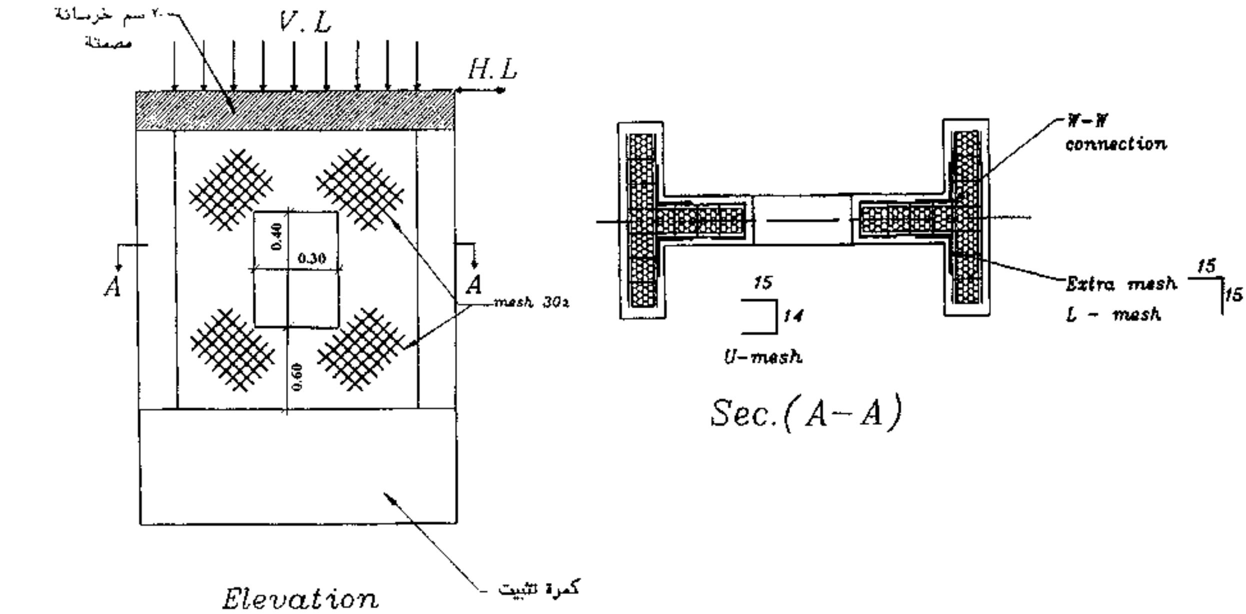

Figure 5. Dimensions of Specimen (W3)

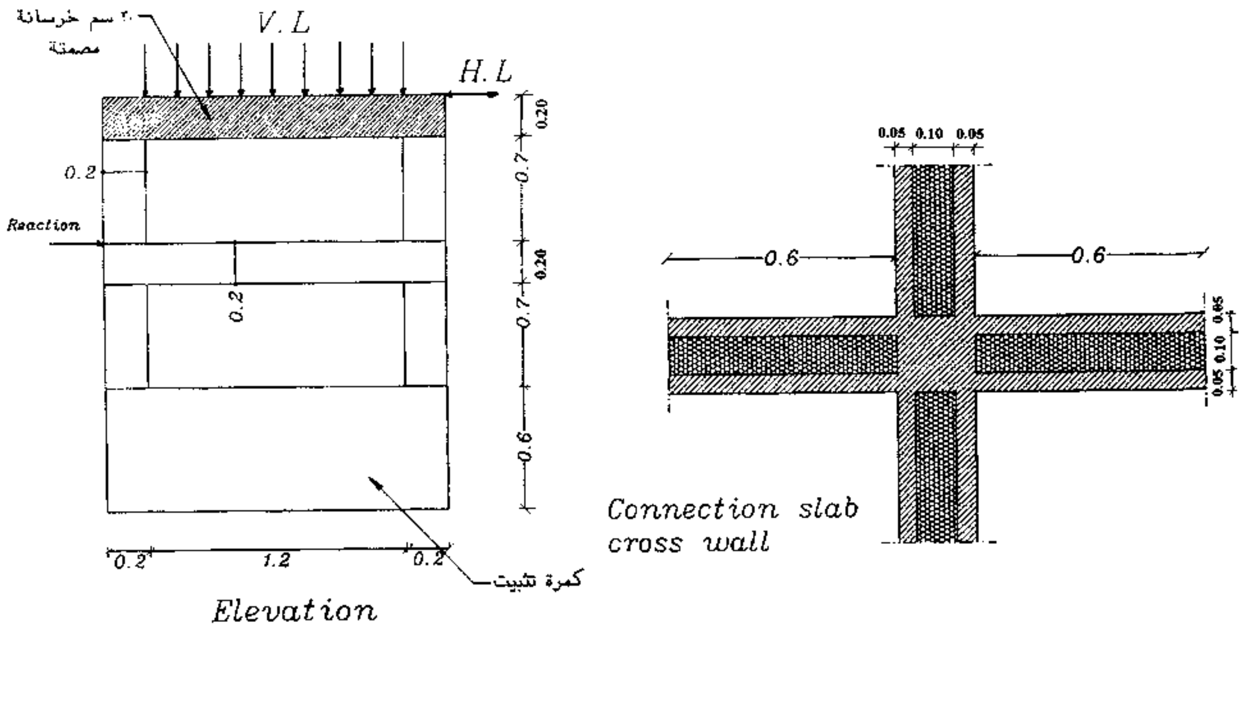

Figure 6a. Dimensions of Specimen (W4)

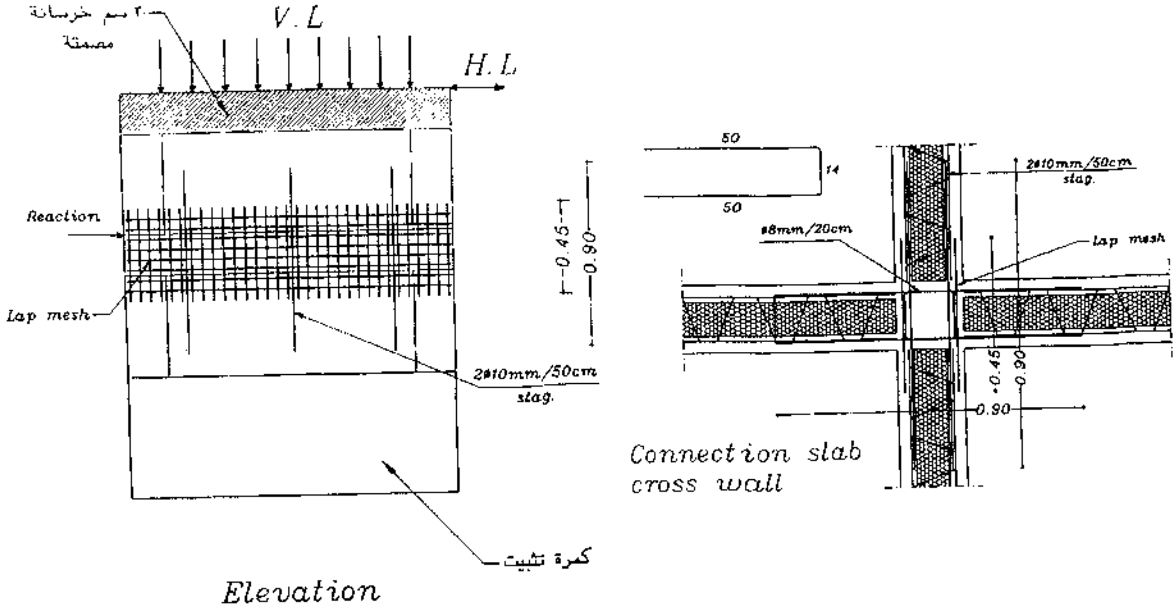

Figure 6b. Details of Additional reinforcement for Specimen (W4)

TEST SETUP

The test setup was prepared to apply quasi-static lateral displacement at the top of the specimens and constant vertical load as shown in Plate 2 according to the following steps:

- The RC base was fixed to the lab rigid floor using two 50mm anchor bolts spaced by 2.00m.
- The sliding of the RC base was prevented by applying horizontal load of 10 tons using hydraulic jack supported by a rigid horizontal reaction blocks.
- The walls were loaded vertically at two points 50cm apart by 7.5 tons for each point using two hydraulic jacks. The two jacks were connected together with one hydraulic pump. The oil pressure was adjusted to provide a constant vertical load of 15 tons (equivalent to working load). The wall was allowed to move freely in the horizontal direction at its top using ϕ 40mm rollers between the vertical jacks and the RC bad.
- The horizontal displacements were applied by a double acting hydraulic cylinder provided with electronic load cell and linear variable transducer. The hydraulic cylinder was hinged to the rigid horizontal reaction girder of the

laboratory main double portal 200 tons test rig. The connection between the hydraulic cylinder and the specimen was designed to allow for rotation.

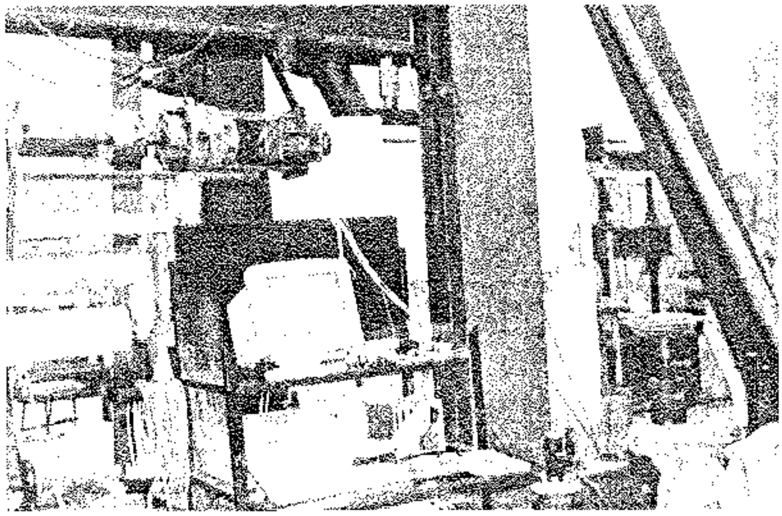

Plate 2. General view for test setup of specimen (W4) showing the horizontal and vertical loading devices

INSTRUMENTATION

The displacements and deformations of the test specimens were measured using linear Variable differential transducers (LVDTs). The horizontal displacement at the top of the test specimen was measured by +/- 100 mm stroke LVDT as shown in plate 3. Another +/- 10 cm LVDT was mounted at the wall base to record base sliding. The control displacement is the difference between the top displacement and the base sliding. The total deformations along the wall diagonals were measured using two +/- 50 mm stroke LVDTs. Two LVDTs of +/- 30 mm stroke recorded axial strains at extreme fibers of the specimen at its connection with the RC base as shown in Plate 4.

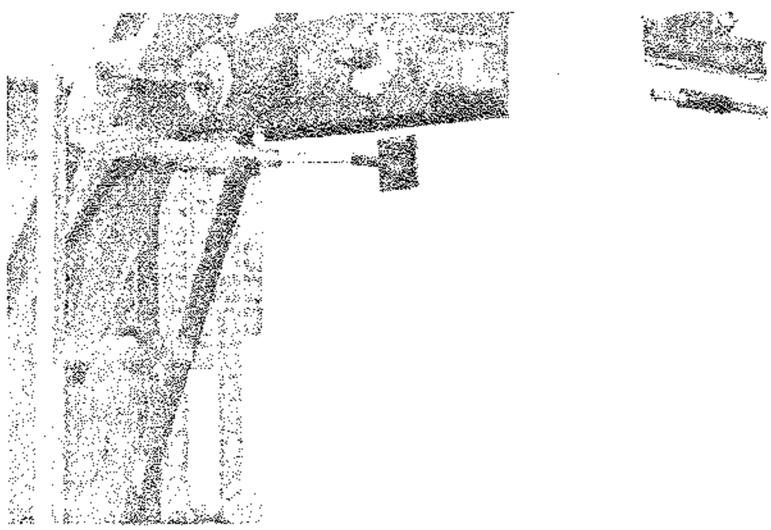

Plate 3. Measuring the horizontal displacement at the top level of the specimen

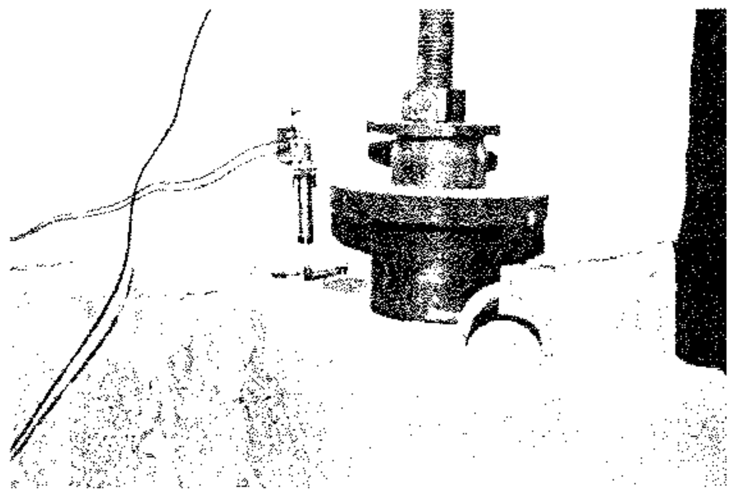

Plate 4. Measuring the axial displacement at the base of the specimen

The horizontal load was measured using +/- 68 ton electronic tension-compression load cell. The vertical load was monitored using mechanical gauge. The load and displacement measuring devices were connected and controlled by Lab View computer software program. The test was executed by on-line measurement and control computerized system.

TEST PROCEDURE

The test specimen was subjected to the vertical load up to 15 tons. All the load and displacement measuring devices were connected to Data Acquisition system. The system was programmed to perform Quasi-static displacement control test. The used displacement pattern is plotted in Figure 7. Lateral load values and corresponding displacements and deformations were recorded simultaneously during the test.

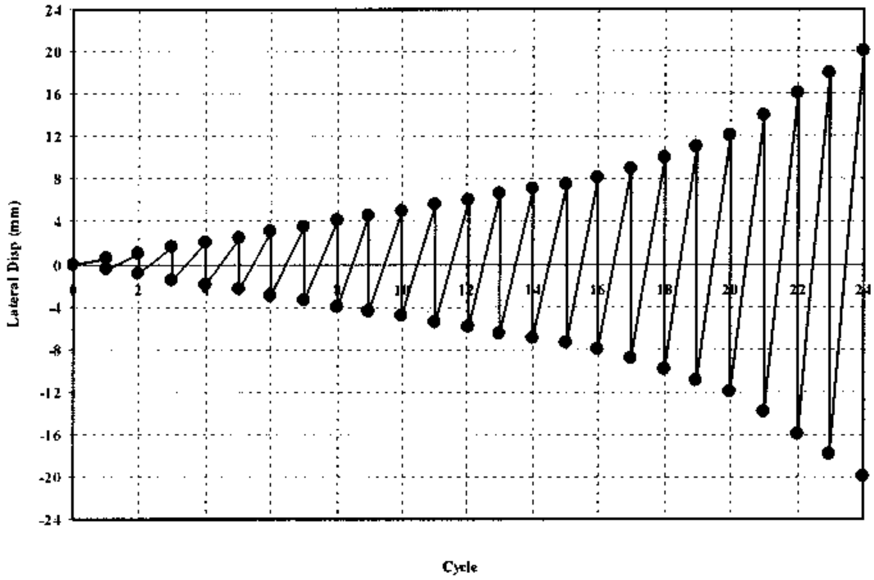

Figure 7. Lateral displacement pattern

TEST RESULTS

The test results of the tested specimens are presented and discussed.

CRACK PATTERN AND MODE OF FAILURE

In the reference specimen W1 no cracks appeared in the wall field and the failure occurred due to the overturning moments. Complete separation between the wall and the rigid base in the tension side was observed as shown in Plate 5. The ultimate load was reached when separation exceeds half wall length. Then, the load started to decrease as base separation increases. In specimens W2 and W3 with horizontal and vertical splices the same separation crack and mode failure was observed. In specimen W3 with window opening, diagonal cracks appeared around the opening. However, at ultimate load separation at tension sides governs the failure.

Plate 5. The separation between 3-D wall panel and its base at ultimate lateral load

ULTIMATE LOAD AND MAXIMUM DRIFT

In specimen W1 (reference) the maximum load was 16 t at maximum drift ratio of 0.25%. The test was stopped at this limit due to base sliding. In the other specimens the base was restrained by initial jacking against horizontal reaction blocks and prevented from sliding. In specimens W2, W3, and W4 the ultimate load ranges between 16 t to 20 t at drift ratios of about 0.375 %. Then the ultimate load degrades progressively with the increase of base separation and the test continued to a maximum drift ratio of 0.625 %.

HYSTERESIS LOOPS

The lateral load-displacement hysteresis loops of the tested specimens are shown in Figures 8,9,10, and 11. In specimen W1 the test was stopped at loop number 8 (drift ratio 0.25 %). In specimens W2 and W3, the load increased up to its ultimate value in the loop number 12 (drift ratio 0.375 %), then the load degraded steeply in the following cycles. In specimen W4 the ultimate load was reached at the cycle number 9 (drift ratio 0.28 %), then the load degraded in the same manner.

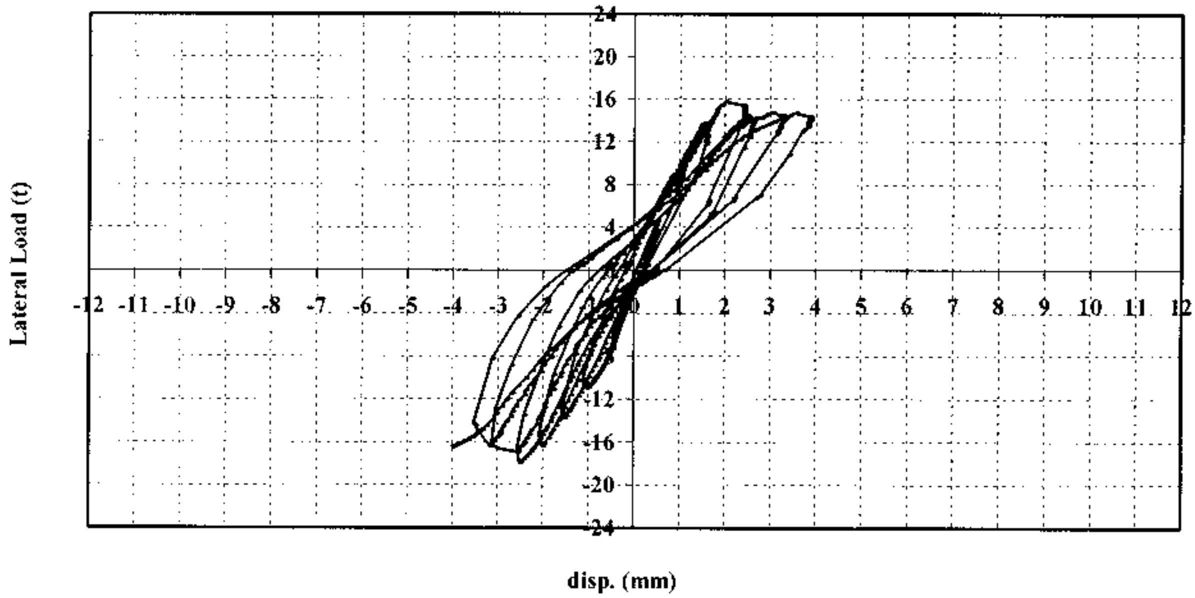

Figure 8. Lateral load- displacement hysteresis loops of wall (W1)

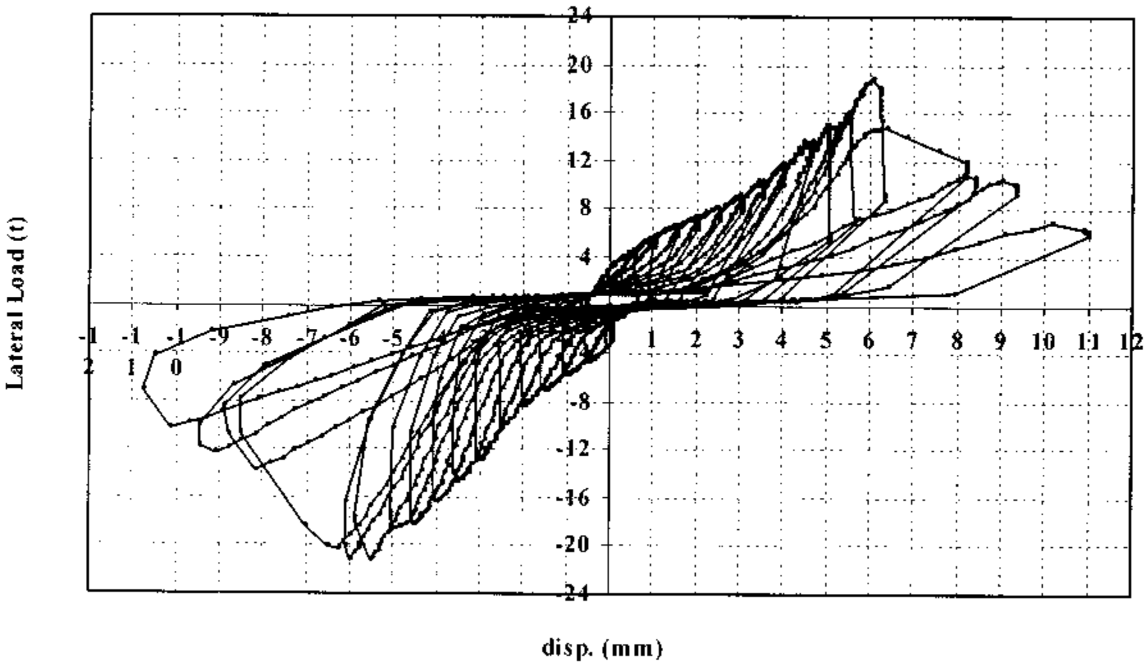

Figure 9. Lateral load- displacement hysteresis loops of wall (W2)

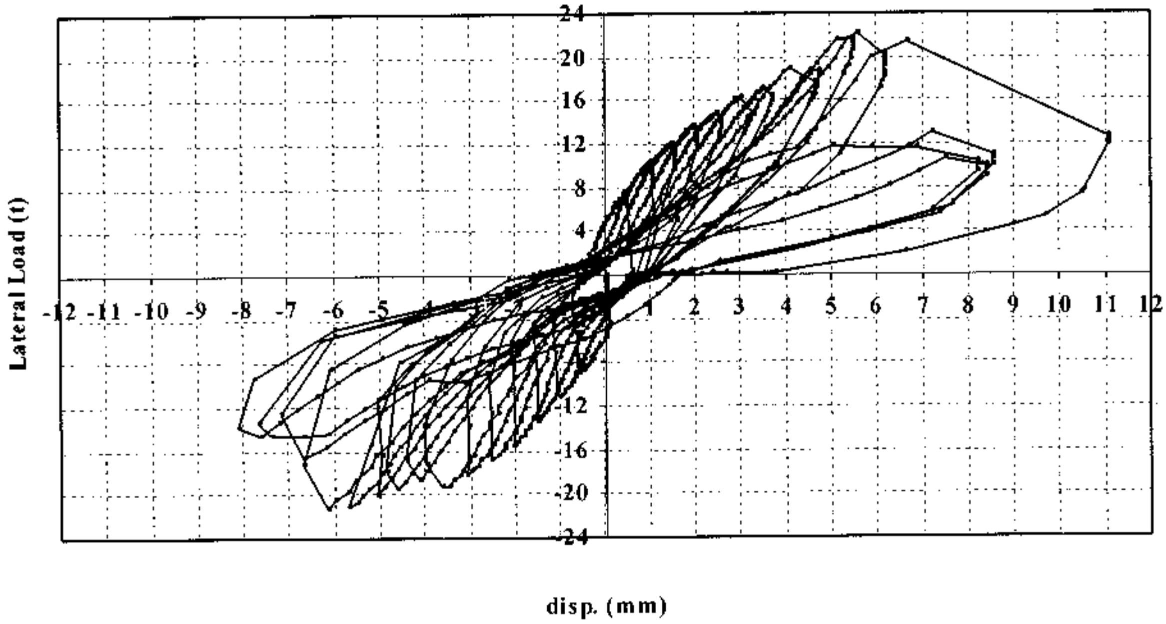

Figure 10. Lateral load- displacement hysteresis loops of wall (W3)

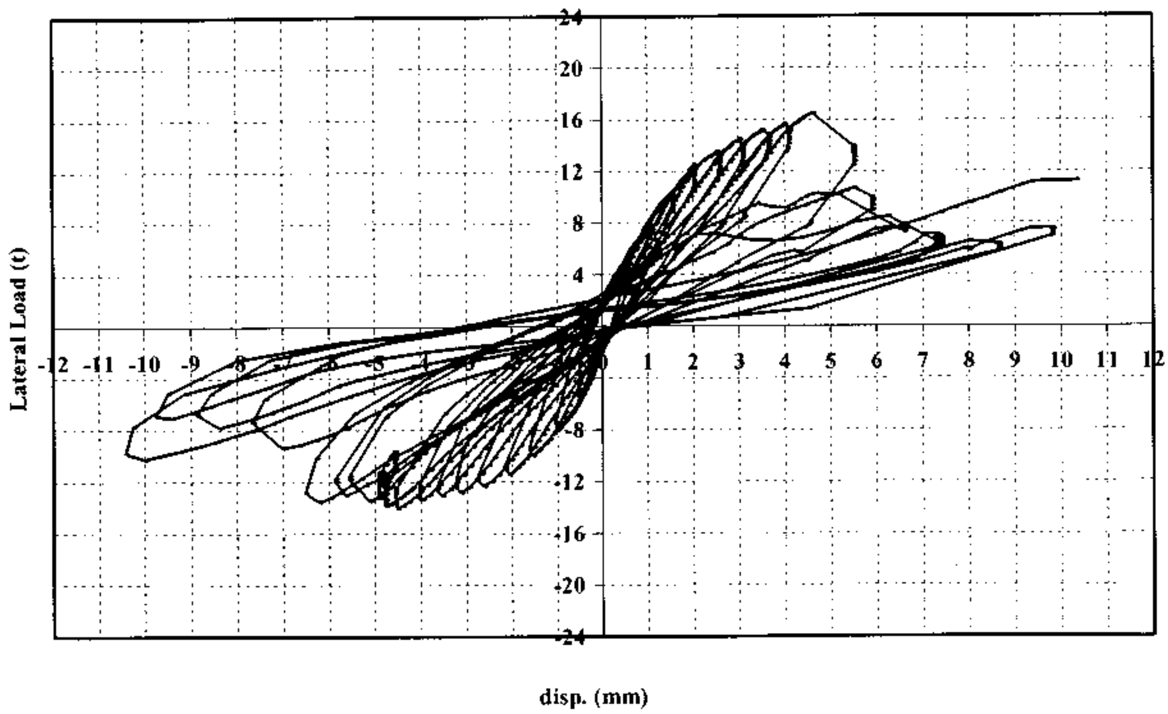

Figure 11. Lateral load- displacement hysteresis loops of wall (W4)

BASE ROTATION

The base rotation is computed as the sum of the vertical deformations measured at base ends divided by the base length. Figure 12 shows the envelope of base rotation for the tested specimens. The ultimate load occurred at base rotation less than 1 radian. Then, the base rotation increased progressively with load degradation.

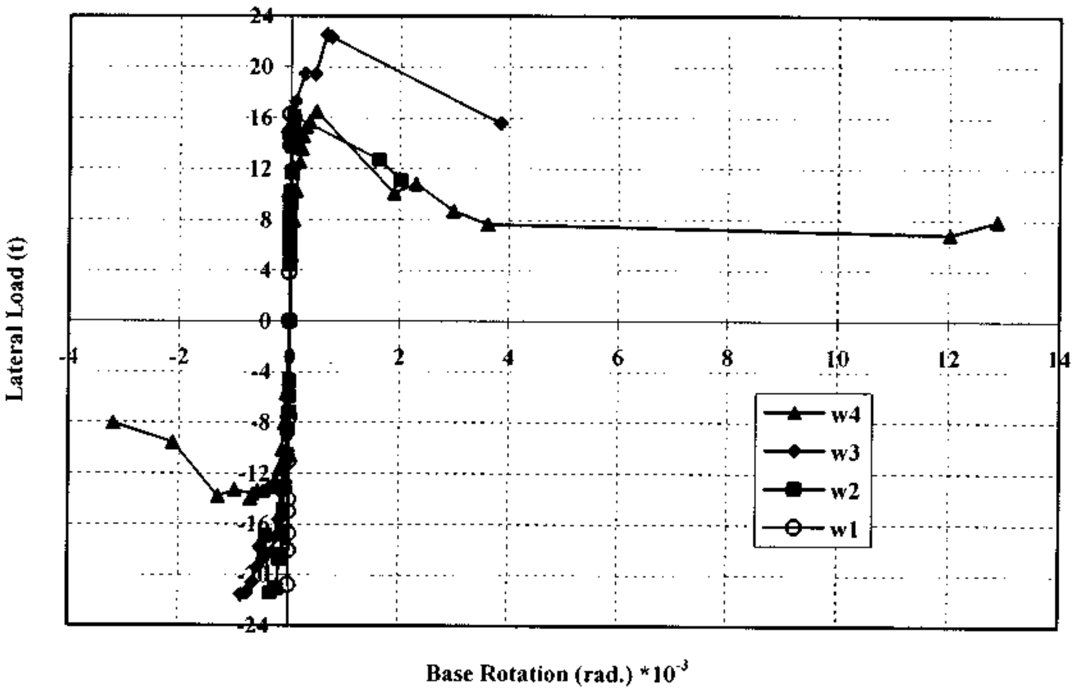

Figure 12. Lateral load – base rotation of tested specimens

DIAGONAL DEFORMATIONS

No diagonal cracks appeared in specimens W1, W2, and W4. In specimen W3 with window opening diagonal cracks appeared at cycle number ten started from the opening corners (drift ratio 0.31 %). However, these cracks did not govern the failure. Figure 13 shows the relation between diagonal tensile strain measured along the full length of the wall diagonal and the lateral load.

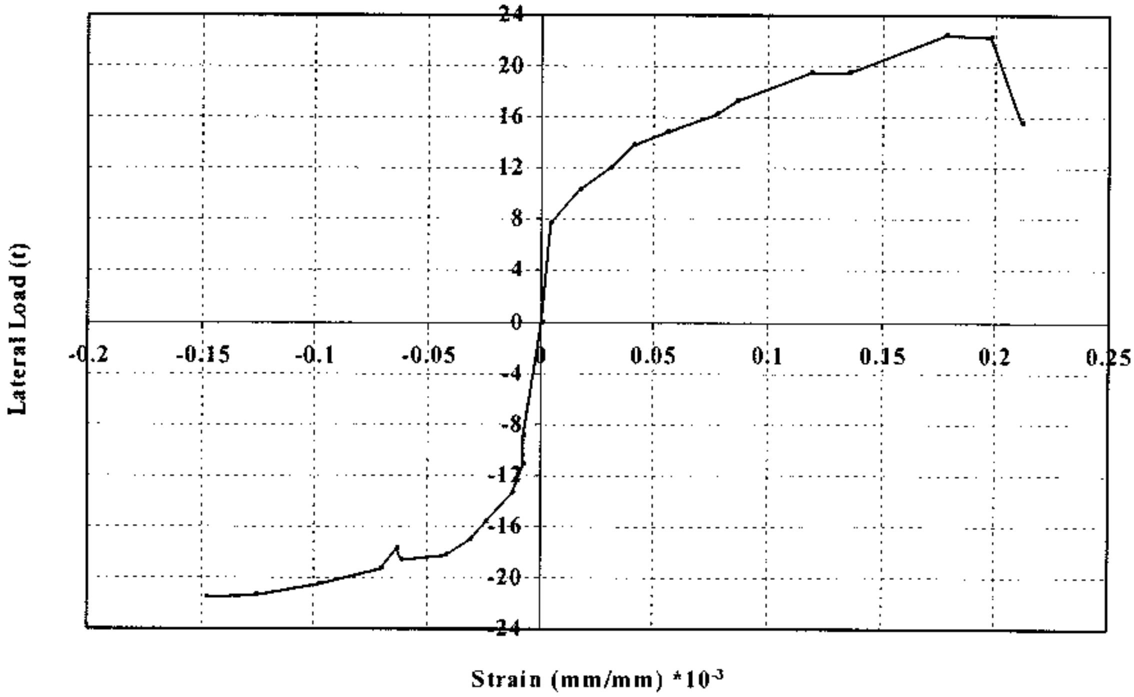

Figure 13. Lateral load – diagonal concrete strain for wall (W3)

EVALUATION OF TEST RESULTS

The ductility, strength envelope, stiffness degradation, and energy dissipation of the tested specimens are evaluated and discussed.

DUCTILITY

The drift ratio is defined as:

Drift ratio = lateral displacement at wall top/ wall height

The drift ratio is used to represent the specimen ductility. The drift ratios measured at the ultimate loads ranged between 0.25 % to 0.375 % (i.e. 1/400 to 1/266). Then the load degrades progressively. No ductile behavior was observed in load degradation.

STRENGTH ENVELOPE

Figure 14 shows the strength envelopes for the tested specimens. The ultimate load varied between 16 t to 20 t. After reached the ultimate value the load started to decrease steeply. This variation of ultimate load occurred due to test setup.

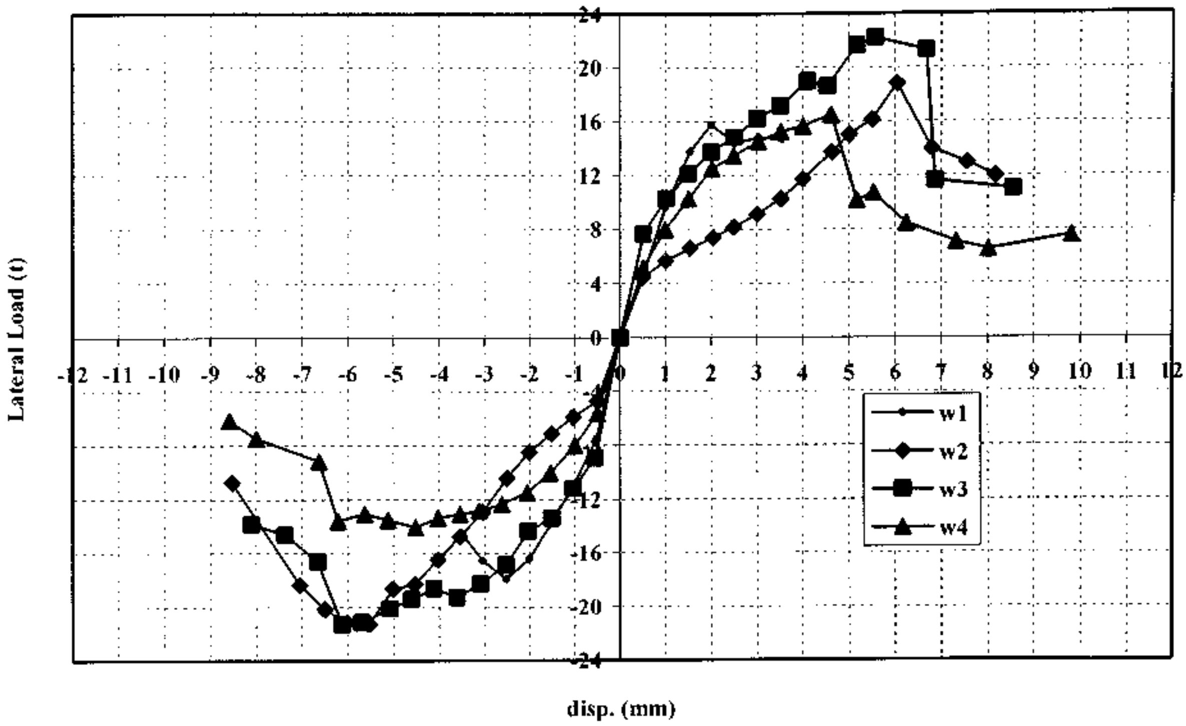

Figure 14. Strength envelopes of tested specimens

STIFFNESS DEGRADATION

The stiffness of each loading cycle is defined as:

Cycle stiffness = sum of maximum push and pull loads/ sum of maximum push and pull
displacement

Figure 15 shows the relationship between the cycle stiffness and the lateral drift ratio for the tested specimens.

ENERGY DISSIPATION

The dissipated energy at each cycle is computed as the area inside the lateral load-displacement loop. The relationship between the accumulated dissipated energy and the displacement is presented for the tested specimens in Figure 16.

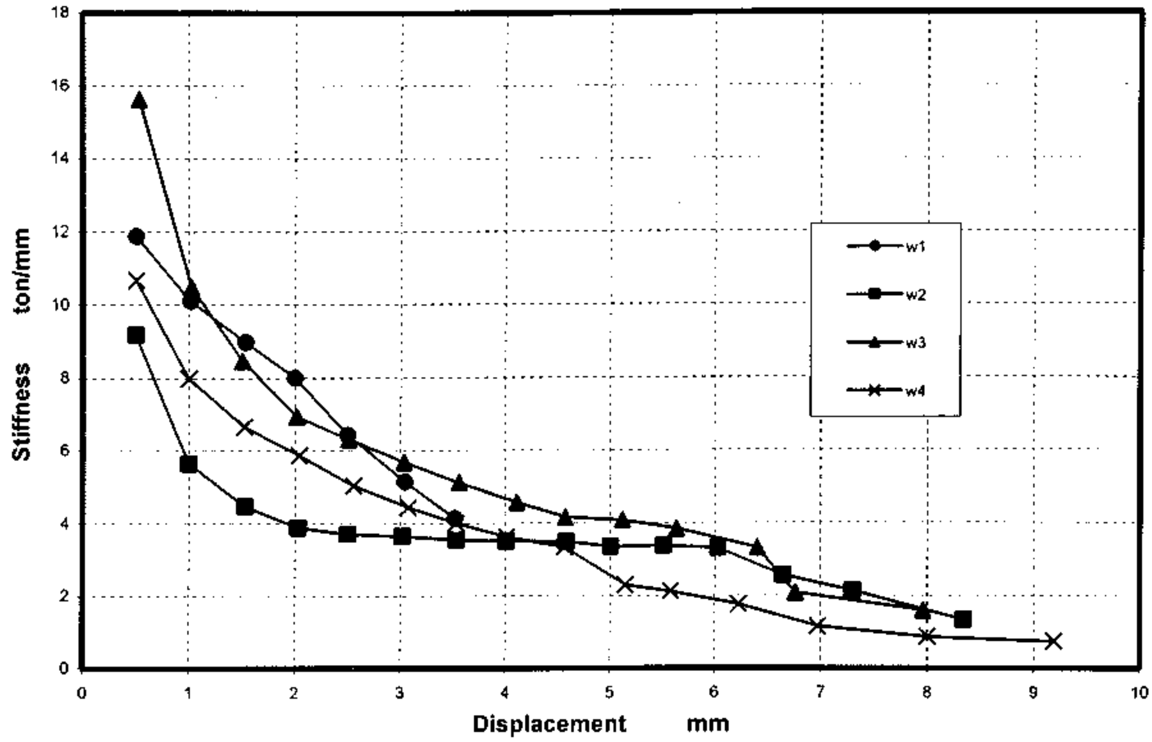

Figure 15. Stiffness degradation of tested specimens

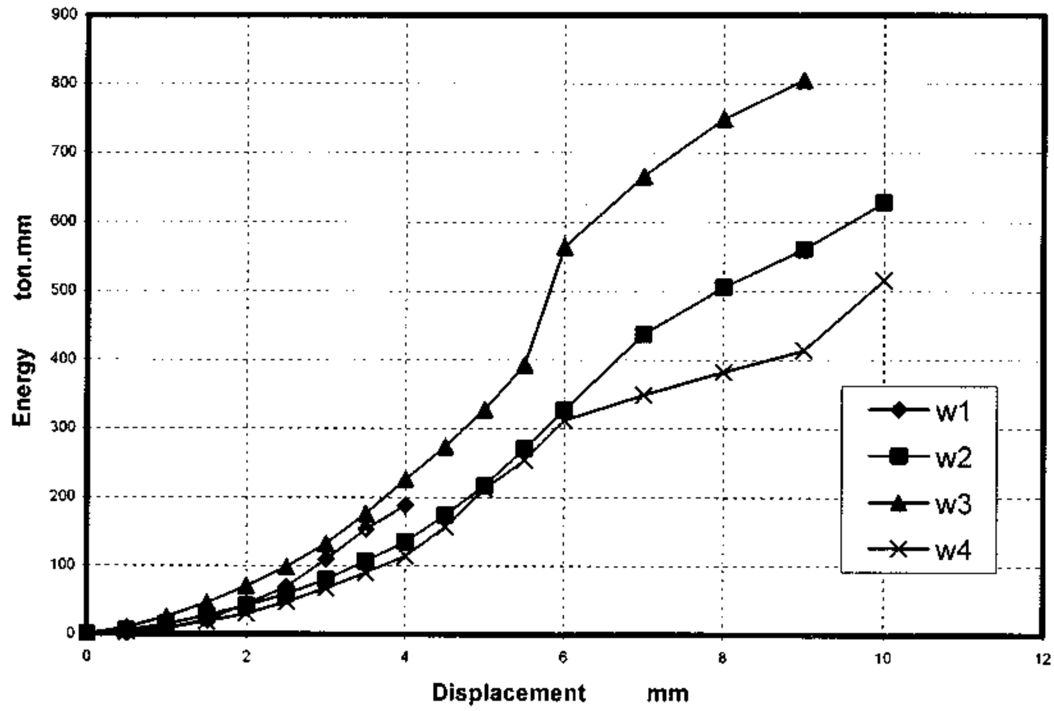

Figure 16. Energy dissipated of tested specimens

CONCLUSIONS

The following conclusions are derived:

(1) The 3-D wall system could resist lateral loads.
(2) The typical vertical splice details are sufficient to attach vertical wall panels.
(3) The typical horizontal splice details representing joint at floor level is sufficient to transmit lateral loads between successive floors.
(4) The details of reinforcement around the openings prevent the enlargement of diagonal cracks and strengthen the wall to resist lateral loads.
(5) Integrity of 3-D wall panel system is approved and the system can be used in multi storey buildings.

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VULNERABILITY OF HIGH-RISE BUILDINGS IN MEDIUM SEISMICITY REGIONS

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ABSTRACT: This study addresses the vulnerability of high-rise RC buildings in medium seismicity regions to all viable design earthquake scenarios at the construction site. These include severe earthquakes with a long epicentral distance and medium events with short source-to-site distance. A case study is investigated; a 54-story RC residential building in a region with medium seismic exposure. In the light of the fact that the three-dimensional inelastic time-history analysis is particularly demanding for this type of structures, a detailed 2D idealization capable of representing the spread of inelasticity during the entire multi-step analysis is adopted. Based on a rigorous hazard study, a set of seven natural and synthetic records was selected to represent the abovementioned seismic scenarios. A number of criteria are employed to define the response limit states on the member and the structure levels. Several types of analyses including inelastic pushover and response-history analyses are conducted to predict of the capacity and the seismic demand of the structure. The study indicated that the contribution of higher modes of vibration is more significant compared with that of the fundamental mode even under the effect of long period events. Contrary to the short site-to-source events, distant earthquakes produce the maximum demand and amplify, with various levels, vibrations in the fundamental mode of the structure. The study, which is feasible for several countries in the Middle East, emphasizes the need for considering all possible seismic scenarios for similar structures and sites.

Keywords: High-rise buildings, medium seismicity regions, finite element modeling, inelastic analysis.

INTRODUCTION

Although time-history analysis is not normally used in practical design of multi-story buildings, it may be a mandate for more important high-rise buildings requiring seismic design to locate structural weakness. The method is required by modern seismic codes such as IBC, FEMA and EC8[1,2,3] for buildings whose response is significantly affected by contributions from higher modes of vibration. This is deemed based on the regularity and the dynamic characteristics of the structure. Moreover, prediction of structural earthquake response by inelastic dynamic analysis provides vital information to structural designers on the effectiveness of their method of design. The accuracy of the analysis depends very much on the quality of the analytical tool and the mathematical model used. It is important to note that the inelastic time-history analysis should be properly validated with respect to the seismic input, the constitutive model used and the method of interpreting the results of the analysis[3]. Interpretation of the analysis output in the time and the frequency domains requires high experience and should be undertaken with great caution.

Medium seismicity regions around the world are not susceptible to powerful events. Nevertheless, the seismic design may be governed by frequent earthquakes occurring in neighboring countries. For such case, it is necessary to investigate the seismic response of structures, particularly those of vital importance, to all possible seismic scenarios. These include strong events with a relatively large epicentral distance and moderate earthquakes with an epicenter near to the construction site. The former scenario may have high amplification in the long period range. Hence it may coincide with the fundamental mode of high rise buildings. Moreover, the short source distance events may amplify the higher modes of vibration for such structures, hence may have higher impact compared with the first scenario. Accordingly, high attention should be paid to investigate the inelastic seismic response of high-rise buildings to the abovementioned scenarios which are feasible for several countries in the Middle East.

The objective of this study is to use a state of the art analytical tool and a thoroughly developed analytical model of a typical high rise building design to predict the seismic response to these classes of earthquakes. The case study is a 54-story RC residential building in a region with medium seismic exposure. A detailed 2D idealization capable of representing the spread of inelasticity along the member length during the entire analysis via the fibre approach is developed. Based on a rigorous hazard study for the construction site, a set of seven natural and synthetic records was selected to represent the abovementioned seismic scenarios. A set of parameters are employed to monitor the seismic response on the member and the structure levels. Several types of analyses are carried out to predict of the capacity and the seismic demand of the structure. The emphasis of the study is on the inelastic global and local response of the structure.

GROUND MOTIONS FOR DYNAMIC ANALYSIS

Estimating the probabilities and recurrence rates of strong events requires extensive analysis of many years of data. Recommendations of the hazard study carried out for the site concerned here is beyond the scope of the present study. Further information is found elsewhere[4]. The ground motions employed here were selected base on the latter hazard study to reliably represent the seismicity of the construction site. According to the NEHRP provisions[2], no fewer than three data sets of ground motions should be employed in time-history analysis. In this case the maximum value of each response parameter (e.g., force in a member, displacement at a specific level) should be used to determine design acceptability. If seven or more time history data sets are employed, the average value of each response parameter is permitted to determine design acceptability. Moreover, the adequacy of the results of inelastic analysis depends strongly on the adequacy of the employed set of input ground motions. Hence, employing an ensemble of carefully selected ground motions is necessary for effective and accurate assessment of the seismic response. It was therefore decided to employ seven input ground motions in the current study and take average of their results.

Five artificially generated (BEQTS03, BEQTS05, BEQTS07, SEQTS07 and SEQTS08) and two natural records (Emeryville, USA, 1989 and Hollister City Hall, USA, 1974) were selected. Based on recommendations of the hazard study of the site, the records were scaled to a Peak Ground Acceleration (PGA) of 0.16g before applying to the structure. The acceleration time-histories and the elastic spectra of the records are shown in Figures 1 and 2, respectively. The spectra shown in Figure 2 are compared with the uniform spectrum for 10% probability of exceedance recommended for the construction site[4]. It is clear that the records have high amplification at different frequency ranges, covering the whole range of structural periods. This ensures that the investigated structure is analyzed under input ground motions representing the all possible seismic scenarios of the site.

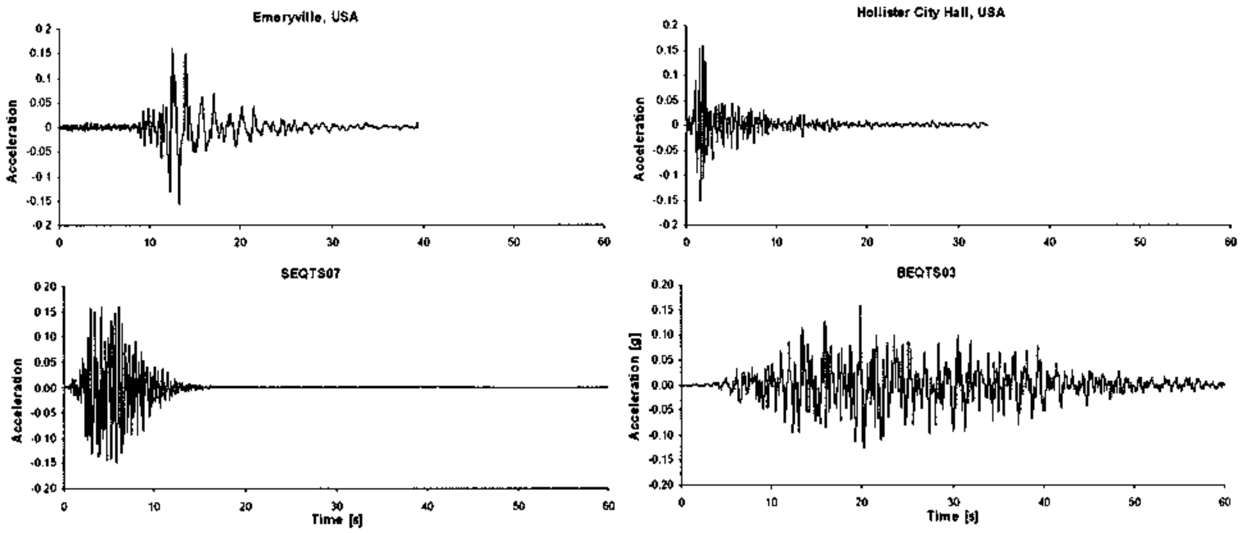

Figure 1. Acceleration time-histories employed in the present study (four of seven records)

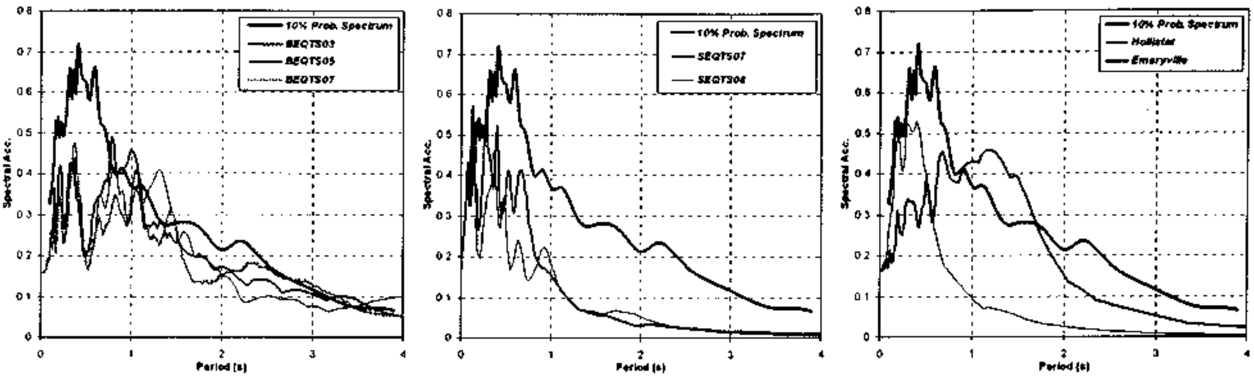

Figure 2. 5% response spectra of input ground motions

DESCRIPTION OF STRUCTURE

The case study investigated here is a 54-story RC residential building. The tower is constructed in a region with medium seismic exposure. The structural system comprises of a system of RC cores, shear walls and columns supporting a system of slabs and beams at the perimeter. The foundation system comprises of a system of piles supporting a rigid RC raft. Figure 3 shows the plane of the tower. It is clear that the main lateral force resisting systems in the transverse direction are Frame 2 and Frame 3. Frame 1 is assumed here not participating in resisting the lateral forces since it is stopped at the mid-height, as shown from Figure 4. In the longitudinal direction, Frame 4 represents the main lateral force resisting system, whilst exterior frames are also assumed resisting gravity loads only. Characteristics of these systems are summarized in Table 1 and Figures 3 and 4.

Table 1. Characteristics of the lateral force resisting systems

	Reference	Structural System	No. of Storeys	Total Height (m)	Total Width (m)
Trans.	*F1 (on Gridlines Fa - G)*	RC Frame	25	187	42.5
Trans.	*F2 (on Gridline Eb)*	RC Frame-Wall	54	187	42.5
Trans.	*F3 (on Gridlines Da - Ea)*	RC Frame-Wall	54	187	25.5
Long.	*F4 (on Gridlines 2a – 4a)*	RC Frame-Wall	54	187	51.0
Long.	*F5 (on Gridline 2)*	RC Frame-Wall	54	187	46.2

MODELING AND LIMIT STATES

The program used in this analysis is ZeusNL[5], which has been used in a large number of research projects and verified against test data from Imperial College, UK, University of Illinois at Urbana Champaign, USA, and elsewhere. Detailed fibre modeling approach is undertaken for the assessed structural system. Sections are discretized in steel, unconfined and confined concrete fibres. The stress-strain response at each fibre is monitored during the entire multi-step analysis. Each structural member is modeled using several cubic elasto-plastic elements capable of representing the distribution of inelasticity. RC column-section, flexural wall-section, rectangular hollow section and T-section are used for modeling of columns, shear walls, cores and beams, respectively.

A number of modeling approaches is investigated before executing the inelastic analysis. These include the rigid arms employed for modeling of cores and walls and the material modeling approach. It is confirmed that the stiffness of rigid arms plays a significant role in determining the lateral stiffness of the structure and hence its period of vibrations. The effect of the material strength (characteristic and mean) on the periods and modes of vibration is insignificant. For the sake of brevity, results of these analyses are not presented here.

Two particularly limit states in the response of the structure are essential in any assessment study to be defined; that at which significant yield occurs and that at which the first indication of failure is observed. Yielding is assumed on the member level when the strain in the main longitudinal tensile reinforcement exceeds the yield strain of steel. Three criteria are utilized to define significant failure on the structure level. These are an excessive drift ratio, formation of a column hinging mechanism or a significant drop in the overall lateral resistance.

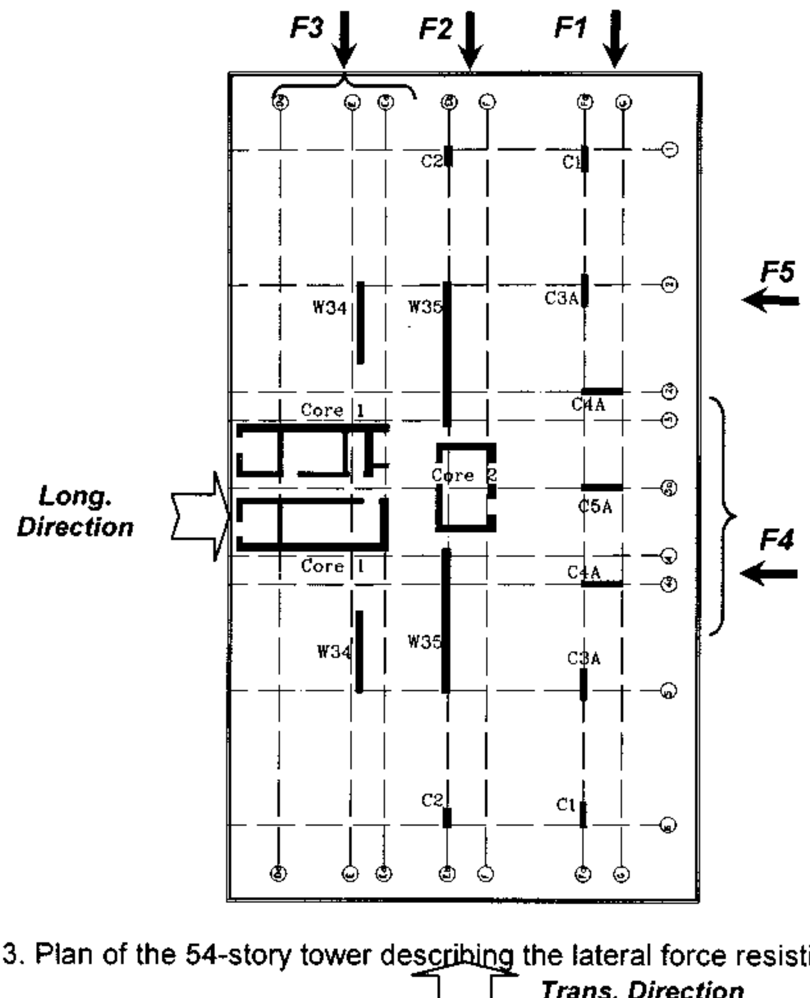

Figure 3. Plan of the 54-story tower describing the lateral force resisting systems

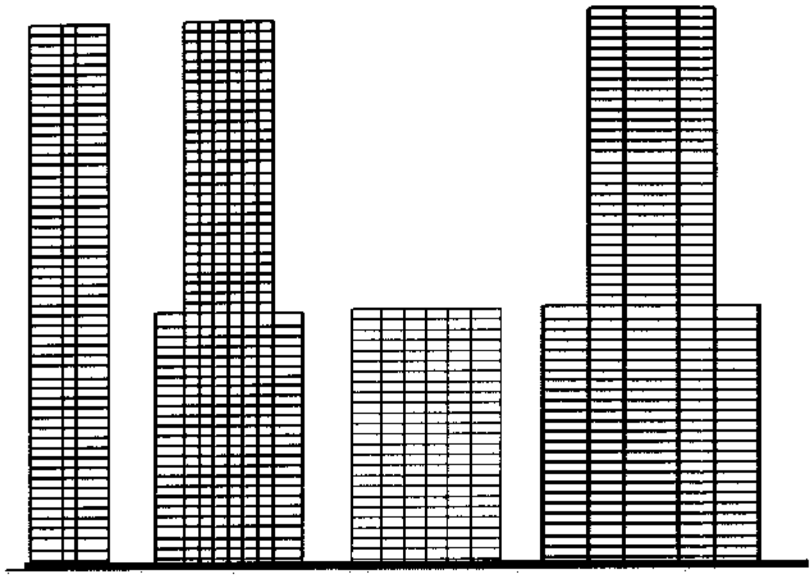

Figure 4. The frame systems in the longitudinal and transverse directions

STRUCTURAL CAPACITY

Based on the abovementioned discussion, the two systems resisting the lateral forces in the transverse direction are F2 and F3, whilst F4 is the main lateral force resisting system in the longitudinal direction. Inelastic pushover analysis is performed using ZeusNL to evaluate the overall capacity of the structure in the two directions, identify the spread of yielding and any formation of collapse mechanism. Two lateral load distributions are employed in this analysis, the uniform and the inverted triangular load patterns. This is undertaken to account for the effect of higher modes of vibration. Further information concerning the method may be found elsewhere[6]. The capacity envelops of the three systems are shown in Figures 5 to 7. The results are summarized in Table 2. These are shown for the longitudinal and the transverse systems at different drift limits. It is clear that the summation of the capacities in the transverse direction is almost equivalent to that of Frame 4, which is the main shear resisting system in the longitudinal direction. The comparison between the capacity values with the design seismic forces ($V/W = 0.0246$) shows the overstrength exhibited by each structural system.

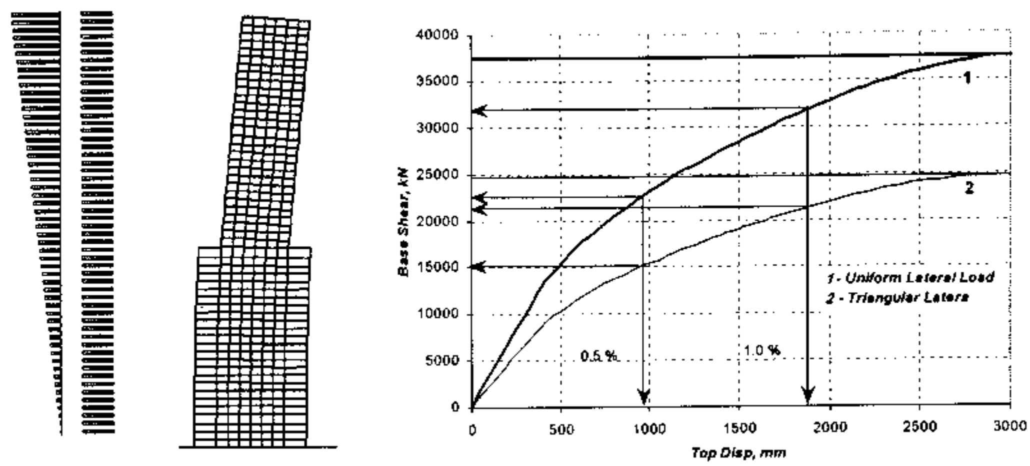

Figure 5. Capacity envelopes of frame F2

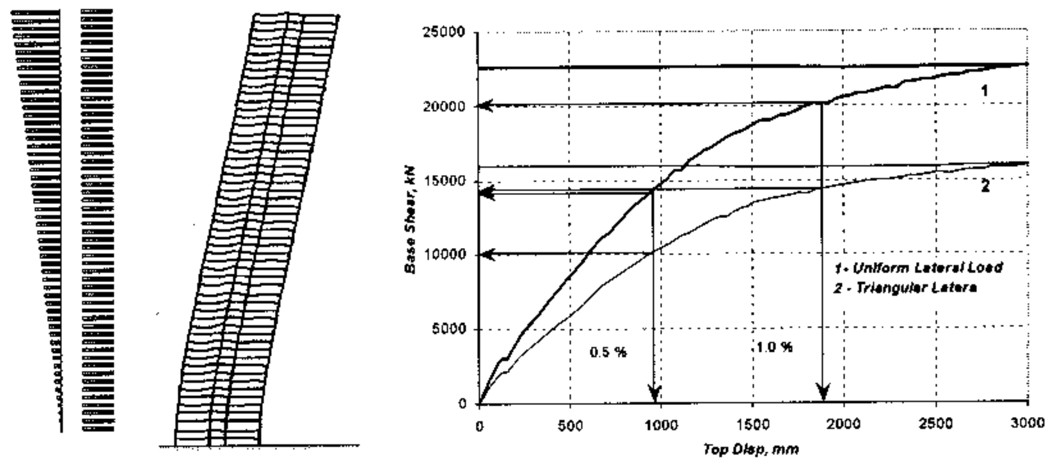

Figure 6. Capacity envelopes of frame F3

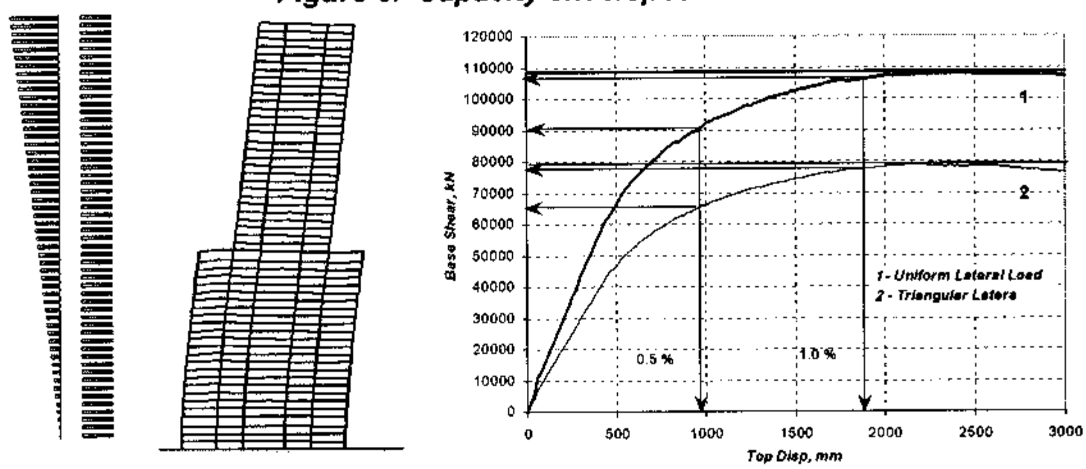

Figure 7. Capacity envelopes of frame F4

Table 2. Lateral capacity obtained from inelastic pushover analysis

	Lateral Load	At 0.50% Drift		At 1.00% Drift		Max. Capacity	
	Distributions	***V (kN)***	***V/W***	***V (kN)***	***V/W***	***V (kN)***	***V/W***
F2	triangular	15000	0.074	21100	0.105	24700	0.123
	Uniform	22500	0.112	32000	0.159	37500	0.186
F3	triangular	10160	0.048	14300	0.068	15900	0.075
	Uniform	13900	0.066	20000	0.095	22640	0.107
F4	triangular	65700	0.083	77400	0.097	78800	0.099
	Uniform	90900	0.114	106150	0.133	107750	0.135

SEISMIC DEMAND

Time-history analysis is performed to examine the response of the structures under the selected set of input ground motions. The principal outcomes from the analyses are global response parameters such as top displacement and base shear. Moreover, Due to the expected higher mode effects, formation of plastic hinges in different structural members and the inter-story drift are monitored during the analysis to provide a more meaningful measure to the level of damage in the structure.

Global Response

Sample roof displacement and base shear time-history results obtained from these extensive analyses are presented in Figures 8 to 10. These are given for four records from the seven ground motions employed in this study. The variability of the inelastic response is quite significant, where the big artificial earthquakes (BEQTS03, BEQTS05 and BEQTS07) produce the maximum drift demand. The Hollister record and the small synthetic earthquakes (SEQTS07 and SEQTS08) show the lowest deformation. The results obtained from the Emeryville record is between these two sets and close to the average response calculated from the seven records.

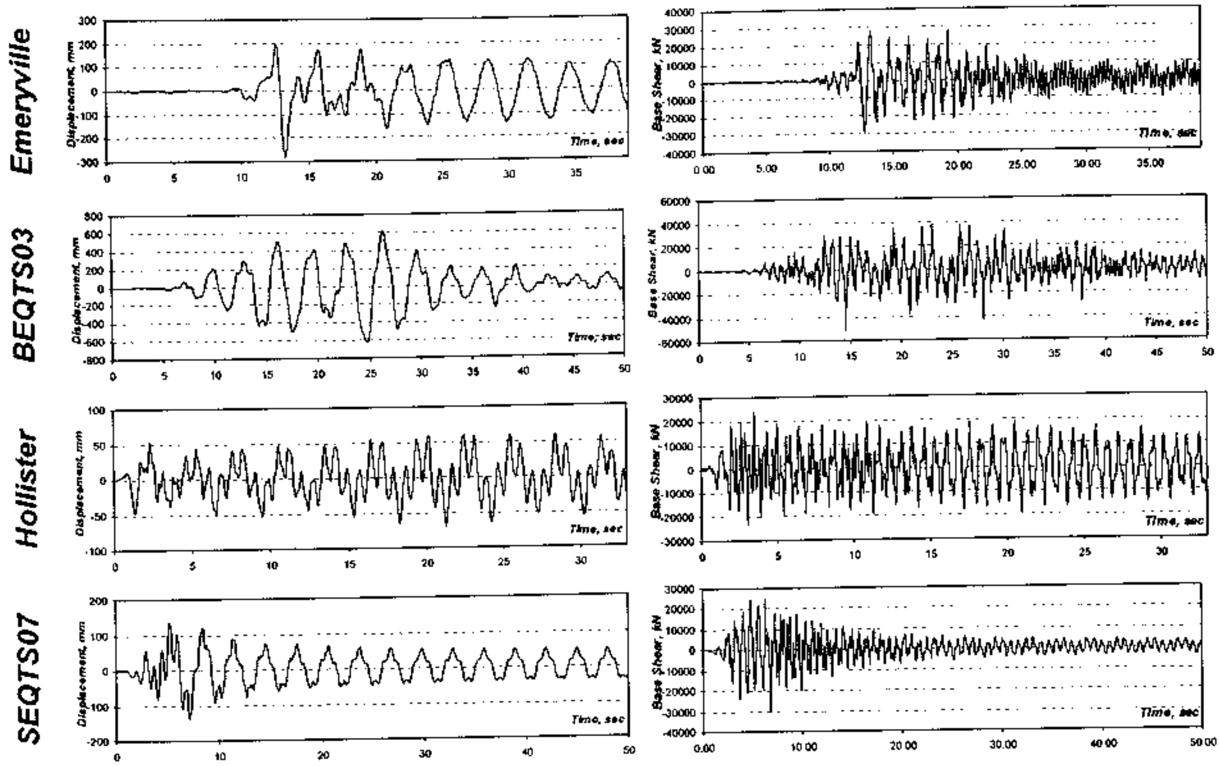

Figure 8. Displacement and base shear time-histories of frame F2

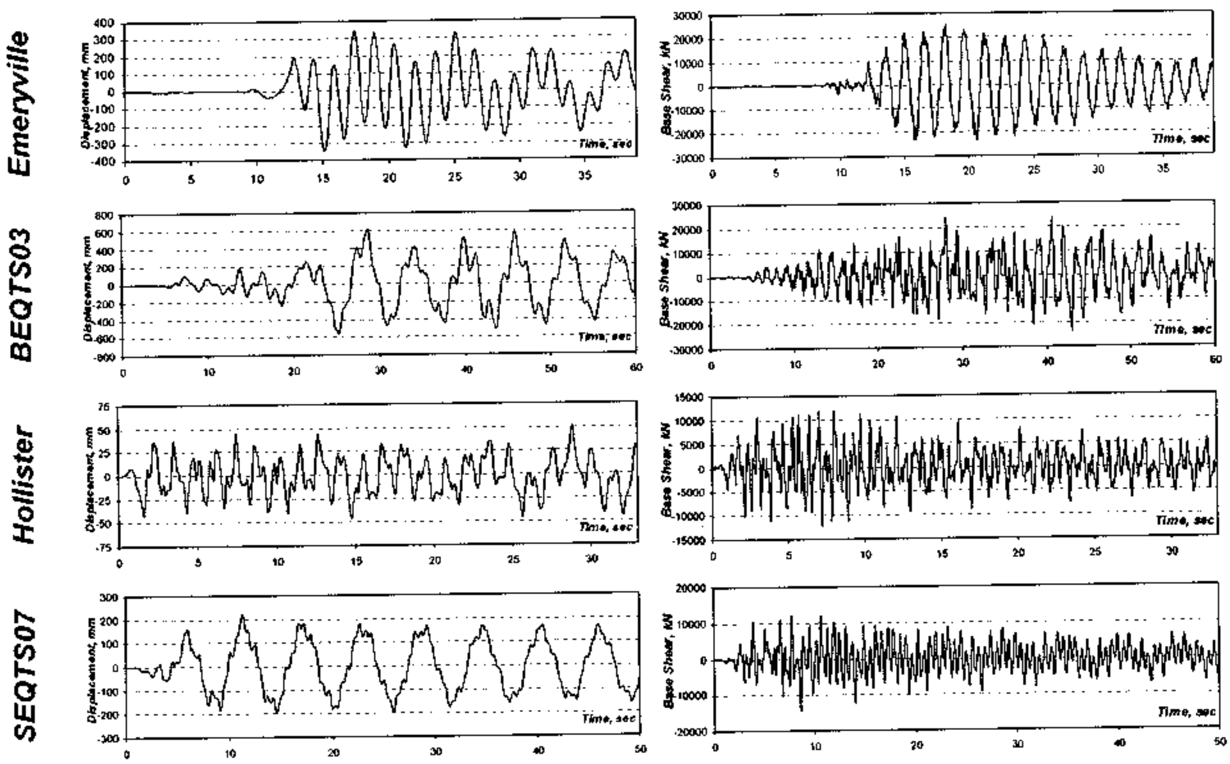

Figure 9. Displacement and base shear time-histories of frame F3

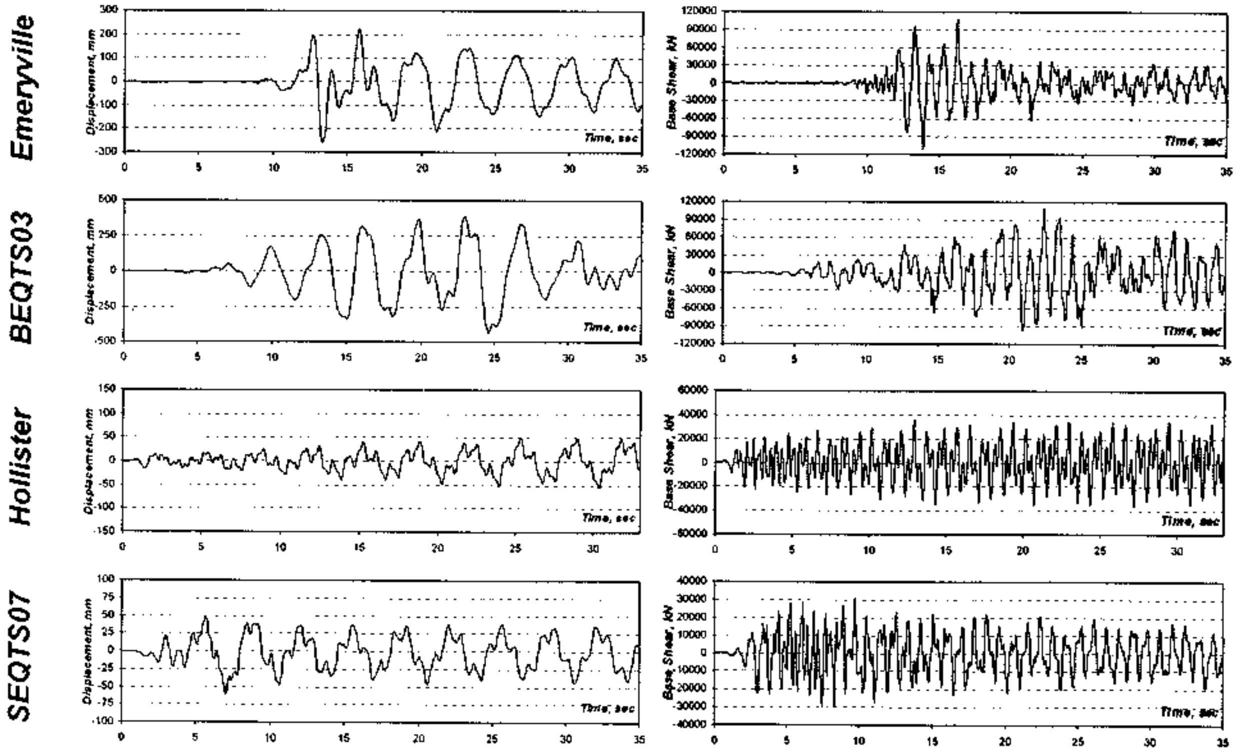

Figure 10. Displacement and base shear time-histories of frame F4

Summary of the maximum response from each earthquake and also the average demand from the seven records are presented in Table 3. It is clear that the Big and the Emeryville earthquakes produce the highest base shear in comparison with Hollister and the Small records. It is confirmed from the elastic spectra of the records that the Big and the Emeryville earthquakes have high amplifications in the period range above 1.0 sec. Therefore, the effect of the fundamental mode of vibration is observed only under the effect of this set of records. The results also reflect the significant contribution of higher modes to the response of the structure. This justifies the high base shear demand observed for values of total displacements smaller than those obtained from the inelastic pushover analysis. Therefore, higher attention is given to the story drift rather than the overall top displacement.

The distributions of the maximum interstory drift (ID) in the transverse and the longitudinal directions calculated from the Emeryville record are presented in Figure 11. The ID results of this record are only shown since it produces very close drift and base shear demand to the average response of the seven records used here. It is clear that the interstory drift demand is not significant except at the 36th story. This is due to the reduction in the cross-section and reinforcement of the walls and cores at this level. It is shown that the safety margins, defined as the ratio of the ID collapse prevention limit state (2%) to the observed ID demand, are reasonable. However the ID demand observed at higher stories suggests taking this into consideration during the final design stage.

Finally, it is also important to note that the results presented here are on the conservative side since only hysteretic damping (damping due to inelastic energy absorption) is taken into consideration.

Table 3. Maximum global demand obtained from inelastic time-history analyses

	Earthquake Record	*Top Drift Demand (mm) (Value)*	*(%)*	*Base Shear Demand (kN)*
Frame F2	*Emeryville*	281	0.15	29800
	Hollister	70	0.04	23650
	SEQTS07	138	0.07	30230
	SEQTS08	171	0.09	24780
	BEQTS03	604	0.32	51815
	BEQTS03	391	0.21	36500
	BEQTS03	607	0.32	51275
	AVERAGE	**323**	**0.17**	**35436**
Frame F3	*Emeryville*	340	0.18	25400
	Hollister	52	0.03	12190
	SEQTS07	218	0.12	14685
	SEQTS08	159	0.09	13433
	BEQTS03	612	0.33	24100
	BEQTS05	490	0.26	20300
	BEQTS07	461	0.25	22300
	AVERAGE	**333**	**0.18**	**18915**
Frame F4	*Emeryville*	258	0.13	112200
	Hollister	55	0.03	36400
	SEQTS07	60	0.03	30800
	SEQTS08	89	0.05	38000
	BEQTS03	431	0.23	110230
	BEQTS05	454	0.24	108000
	BEQTS07	430	0.23	105000
	AVERAGE	**254**	**0.134**	**77233**

Member Response

Summary of the observations of the local response from the time history analysis is given in the Table 4. It is clear that high ductility demands are generated in the short coupling beams, whilst the demand in vertical structural members are rather low, reflecting their low contribution in dissipating input energy. In considering the relative flexural stiffness of the coupling beams compared with adjacent walls and the sensitivity of these beams to changes in the wall curvature, significantly larger inelastic demands occur in such beams compared with walls that are coupled. It was confirmed[7] that the bidiagonal reinforcement typically used in coupling beams is a very efficient mechanism in dissipating energy and sustaining large deformations. After yielding of top and bottom reinforcements and crushing of concrete at the extreme fibre, the bidiagonal elements will continue to provide an efficient load transfer mechanism. This leads to a very efficient mechanism in dissipating energy and sustaining large deformations. Therefore the steel yielding of coupling beams, which are based on the strain of top or bottom longitudinal reinforcement, may not be considered as an indication of significant damage. The observed local response also suggests taking into consideration the yielding observed in the slabs connecting external walls to the cores.

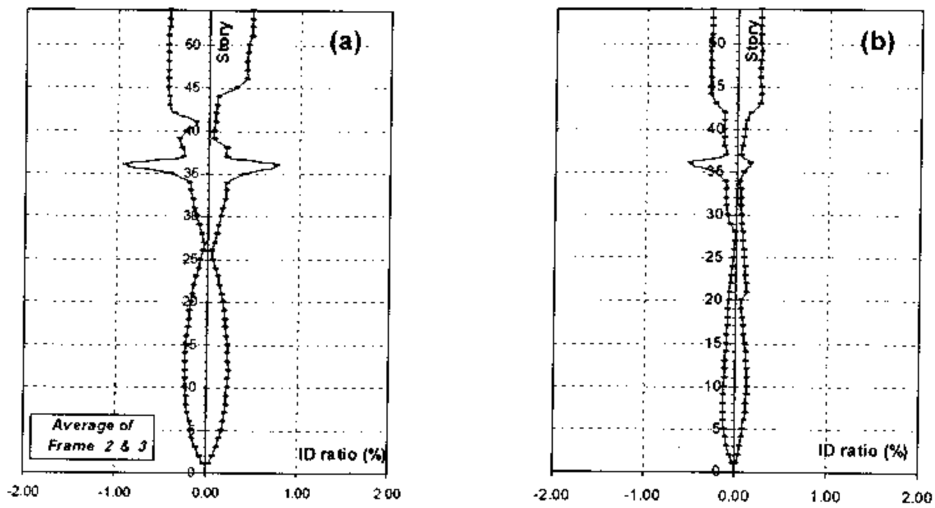

Figure 11. Maximum interstory drift distribution from Emeryville: (a) transverse direction, (b) longitudinal directions

Table 4. Local yielding observed from inelastic time-history analyses

Earthquake	***Lateral Force Resisting System***		
	Frame F2	*Frame F3*	*Frame F4*
Emeryville	-	Several connecting beams	Several connecting beams
Hollister	-	-	-
SEQTS07	-	Few connecting beams	-
SEQTS08	-	Few connecting beams	-
BEQTS03	Few connecting beams	Several connecting beams	Several connecting beams, slabs and ground story columns
BEQTS05	-	Several connecting beams	Several connecting beams and slabs
BEQTS07	Few connecting beams	Several connecting beams	Several connecting beams and slabs

CONCLUSIONS

Vulnerability of high-rise buildings in medium seismicity regions is investigated in this study. The case study is a 54-story RC building currently in a medium seismic exposure region. A rigorous hazard study for the construction site indicated that the structure may be subjected to severe earthquakes with a long epicentral distance and medium events with short source-to-site distance. Seven natural and synthetic records were selected to represent the seismicity of the site and possible seismic scenarios. A detailed fibre modeling approach was adopted and a set of limit state criteria were employed to monitor the response. Several types of analyses including inelastic pushover and time-history analyses were conducted to estimate the capacity and predict the seismic demand.

The study indicated that the sum of the capacities of the lateral force resisting systems in the transverse direction is equivalent to that in the longitudinal direction. This underpins the efficiency of the structural system suggested for resisting the lateral forces. The ductility demands of vertical structural members are rather low, whilst high demands are observed in the coupling beams indicating their significant role in dissipating energy. The shear walls and cores effectively control drift and provide a reasonable safety margin except at higher stories. This is the result of the reduction in the cross-sections and reinforcements at these levels. The consequences of the abrupt changes in stiffness even at higher stories are clearly confirmed by the presented results.

The seismic design of structures in medium seismicity regions may be affected by earthquakes occurring in neighboring countries. In such case, it is necessary to investigate the seismic response to strong earthquakes with a relatively large epicentral distance and moderate earthquakes with epicenters near to the construction site. Contrary to the near source events, distant earthquakes produce the maximum demand and amplify vibrations in the fundamental mode of the structure. Contribution of higher modes governs the response even under the effect of long period earthquake motion.. The results are on the conservative side since only hysteretic damping (damping due to inelastic energy absorption) is taken into consideration in the analysis. The study, which is feasible for several countries in the Middle East, emphasizes the need for considering both near- and far-source seismic scenarios for similar structures and sites.

ACKNOWLEDGMENTS

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PROGRESSIVE FAILURE ANALYSIS OF A COMPOSITE ARMY BRIDGE

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ABSTRACT: Military composite bridges offer many unique advantages for the army including its lightweight (high strength-to-weight ratio), as compared to current steel and aluminum bridges, as well as their superior corrosion and fatigue resistance properties. This paper presents the results of a part of a comprehensive research program sponsored by the US Army to develop innovative field repair techniques for military composite bridges. In this paper, results of virtual testing and progressive failure analysis (*PFA*) simulation conducted on a Composite Army Bridge (*CAB*) prototype are presented. The simulation was conducted using the state-of-the-art simulator, GENOA, developed jointly by Alpha Star Corp and NASA. The virtual tests were performed on the composite treadway under three different loading cases. The three cases used in this study are: i) maximum static shear loading case, ii) maximum static flexural loading case and iii) fatigue progressive failure analysis for the maximum moment case loading. The simulation results matched well the full-scale laboratory test results. For example, the variation between the maximum deflections predicted by the GENOA simulation for the maximum shear and those obtained from the full-scale tests was only 3.2%. In addition, the location and type of damages at the ultimate load were very close to those obtained from the full-scale laboratory tests.

INTRODUCTION

In the recent years, the use of advanced composites in building bridges and bridge components for both military and civilian applications became an attractive topic of research for many structural engineers *(DARPA (2000), Iyer (2002) and Mosallam and Haroun (2003))*. Composite bridges (*Figure 1*) provide several attractive features including its lightweight (*high specific stiffness and strength*), compared to current steel and aluminum bridges, as well as its superior corrosion resistance properties that is preferred in harsh environmental conditions. The lightweight features of composites are attractive and essential property in order to fulfill the goal of rapid operational mobility of army vehicles. However, during hostile battlefield conditions, damages are likely to occur due to a variety of reasons including punctures from rocks, impact loads from deployment/handling, dropping the structure from moderate heights and various battlefield threats (*Iyer (2002)*).

In order to develop an effective repair methodology for military composite bridges, and in order to verify its validity through virtual testing, one should identify i) the most potential loading scenarios that would cause damages to the bridge including extreme live loading conditions, and ii) loads that the "repaired" bridge should withstand after being repaired.

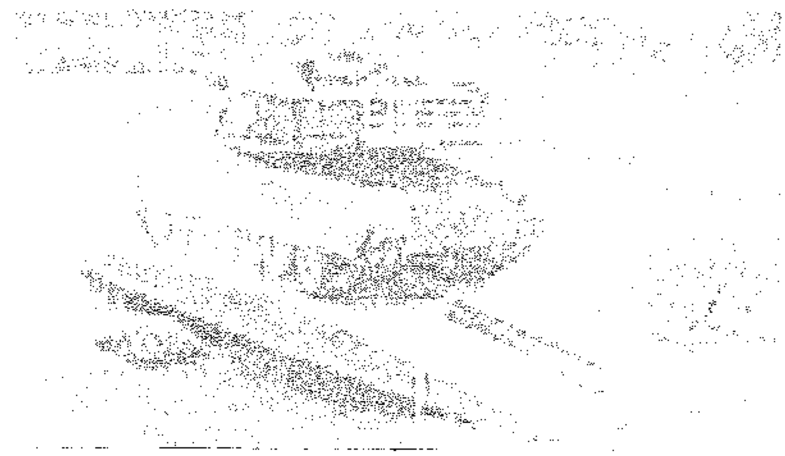


Figure 1. The Composite Army Bridge (DARPA (2000))

The first part of this study involved calibrating the materials constituent properties of the composite army bridge that will be used in demonstrating and virtually verifying the smart repair technology for military bridges. In the second part of the study, GENOA virtual testing and progressive failure simulations were performed on the composite treadway under three different loading cases.

The three cases used in this study are: i) Maximum static shear loading case (*Load Case 1*), ii) Maximum static flexural loading case (*Load Case 2*), and iii) fatigue progressive failure analysis for the maximum moment case loading (*Load Case 3*).

CALIBRATION OF MATERIALS CONSTITUENT PROPERTIES USING GENOA MCA & PFA CODES

Objective: The objective of this analysis is to calibrate the materials constituent properties of the CAB that are used in demonstrating and verifying the smart repair technology for army composite bridges. This calibration process is essential in developing both the NASTRAN finite element (FE) and the GENOA Progressive Failure Analytical (PFA) models of the composite army bridge before and after damage occurs as well as after performing the smart repair process.

In order to gain confidence in the material properties that are used in evaluating the field repair methodology and to ensure reliable results, calibration of materials constituent properties that include weighing the effect of the braided tri-axial properties was preformed

Background: Material calibration is a backbone process of constituent properties based on the laminate test data. The purpose of this calibration process is to build a databank to be used in for structural analysis of the bridge structure. In addition, material calibration is necessary because lamina and laminate manufacturing processes involve many unknowns and variables that influence product properties. The use of mechanical properties supplied by vendors usually results in overestimating both the lamina and laminate properties. The numerical material calibration is similar to the concept of coupon tests (*virtual testing*).

In this process, the GENOA code is used to calculate the mechanical properties of each ply using best available constituent data (*e.g. fibers, matrix, etc*). If the calculated ply properties do not match the experimental data, then the constituent properties are adjusted until the experiments and predictions are in agreement. Ply properties are calculated from algebraic equations that are functions of the constituent properties. For a single set of fiber and matrix properties, there are almost 50 different values that can be changed in order for each of the ply properties to match any available data. Fortunately, each of the ply properties only depends on about 2 to 10 different constituent properties, and is highly sensitive to only a few of those constituent properties. Therefore, in trying to match up each ply property at a time, only two or three variables are manipulated between each prediction. The calibration process is performed by calculating ply properties, comparing them to experimental results, manipulating the constituent properties as needed. This process is repeated until predictions match experimental results. First, it should be assumed that the experimental results are reliable and are measured from a ply whose constituent properties were not measured at the time of the experiment. The rest of the calibration process/prediction matching occurs in series. The experimental results are then set equal to the same results that would be obtained from a mathematical, mechanics of materials model of the ply of concern. Finally, the fiber and matrix properties that would give similar mathematical results are then sought. This becomes a root finding problem, not a curve fitting process, for each of the ply properties. At some point in the design space for each ply property, the prediction will match the experiment. Calculating ply properties using mathematical models of composite materials and these inputs and having the values match up to experiments is a good accomplishment. It states that a set of fiber and matrix properties, or design point, has been found that could possibly be the same set of properties that might have been measured from the plies tested experimentally.

Calibration Procedure: The following are the typical procedure used in the calibration process:

1. Prepare a databank with fiber/matrix properties as close as possible to the calibrated fiber/matrix properties. The closer, the faster the calibration process. Otherwise, the fiber/matrix properties provided by vendors are used.
2. Understand the dependency of lamina properties on constituent properties using the sensitivity module in GENOA-MUA (Materials Uncertainty Analyzer).
3. Using GENOA-MCA (Material Constituent Analyzer), adjust fiber/matrix moduli in the databank until the calibrated lamina moduli agree with the lamina test data.
4. Adjust fiber/matrix strengths in the databank until the calibrated lamina strengths agree with the lamina test data.
5. If the composite laminate coupon test data is available, simulation may be conducted for further confirmation of the calibrated databank using GENOA-PFA.

Steps 2 and 3 can be switched because the calibrations of lamina moduli and strengths are not obviously coupled. Figure (2) shows the flowchart of the GENOA calibration procedure.

Calibrated Materials: A total of four different composite materials were calibrated in this task that comprise the structure of the CAB (*refer to Figure 3*). Prior to performing the calibration process, original mechanical information were identified from DARPA (2000) report.

Calibration Results: The results of the calibration process identified several variations between the calculated properties and those obtained fro ASTM laboratory coupon tests, especially in the shear moduli, strength and Poisson's ratios. As expected, the original and the calibrated results for unidirectional laminates agreed well. Figures (4) through (7) present graphical presentation of the results obtained from the calibration analysis for different materials.

PROGRESSIVE FAILURE ANALYSIS

The maximum shear loading case (*Load C*
ase 1) and the maximum bending loading case for MLC 100 described in DARPA (2000) report were simulated using the GENOA code. In addition, a fatigue progressive failure analysis was conducted for the maximum flexural loading case (*Load Case 2*).

Load Case 1 (Maximum Shear): The NASTRAN FEM model of the composite bridge treadway was transformed into a GENOA numerical model. The model consisted of 14,514 mixed elements. A Progressive Failure Analysis (PFA) was performed on this model for Load Case 1 (Max Shear) shown in Figure (8).

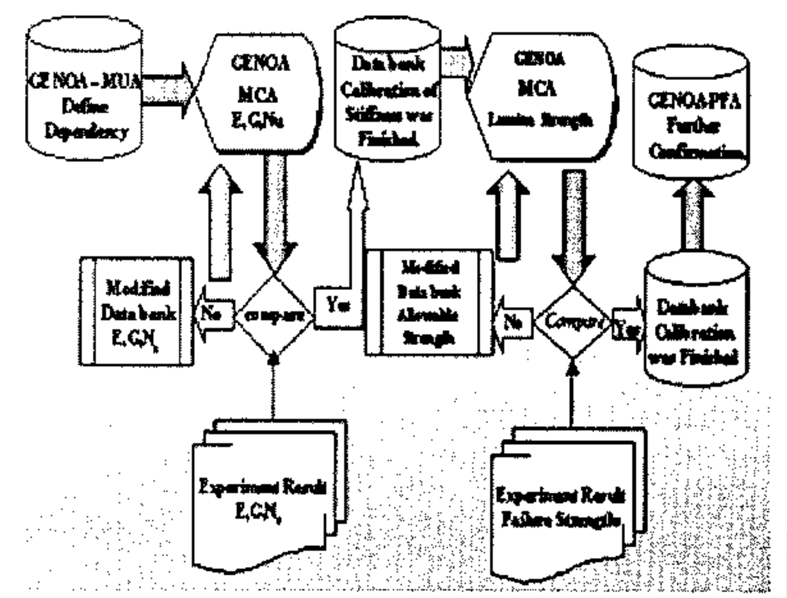


Figure 2. GENOA Calibration Procedure

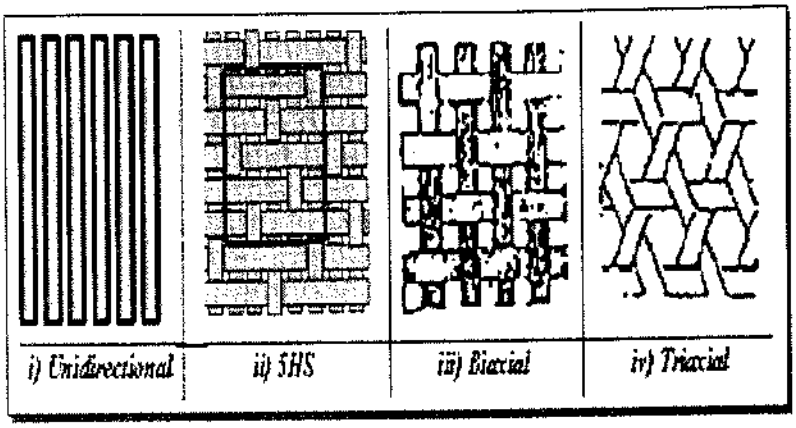
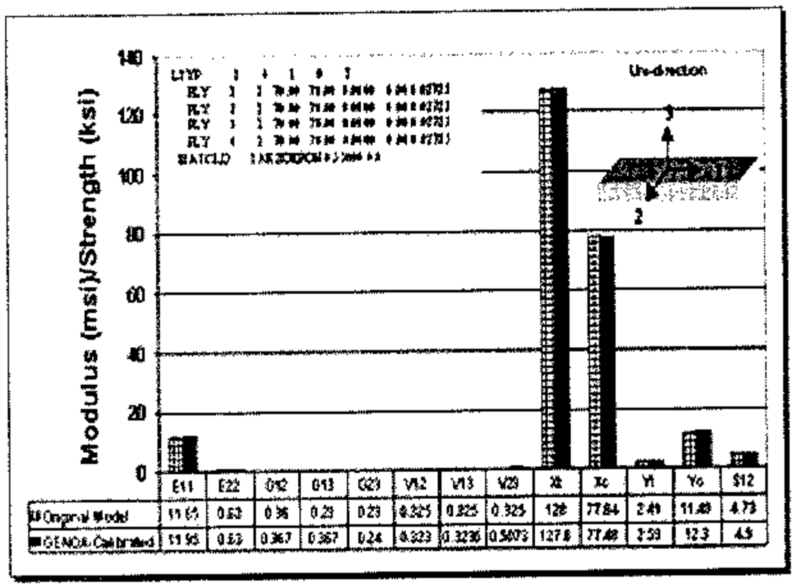


Figure 3. Types and Geometry of Fibers Used in the FEM Analysis

Unlike the NASTRAN FEA, the GENOA analysis is non-linear. Any damage or fracture calculated to have occurred in the structure causes the model stiffnesses to change and the internal loads to redistribute. Larger deflections would, therefore, result. In the case of the CAB model, some damage occurs at the working pressure level. In GENOA parlance, damage and fracture failure are distinctly different phenomena. Damage is defined as an intra-lamina event. One or more plies in a laminate have failed. The laminate still has a load bearing capacity albeit at a lower level. The laminate properties are reduced for subsequent GENOA analyses. Fracture occurs in GENOA when laminate level failure has been reached at a node. The GENOA model is modified at a fractured node by disconnecting it from adjacent, non-fractured elements. This also results in an altered load path. When the accumulation of damage and fracture causes the structure's load bearing capacity to drop to zero, it is considered to have failed.



	E11	E22	G12	G13	G23	V12	V13	V23	Xt	Xc	Yt	Yc	S12
Original Model	11.65	0.63	0.36	0.23	0.23	0.325	0.325	0.325	128	77.84	2.49	11.49	4.73
GENOA Calibrated	11.93	0.63	0.367	0.367	0.24	0.323	0.3236	0.5073	127.8	77.48	2.59	12.3	4.5

Figure 4. A Comparison between Calibrated and Original Mechanical Properties of Unidirectional Laminates

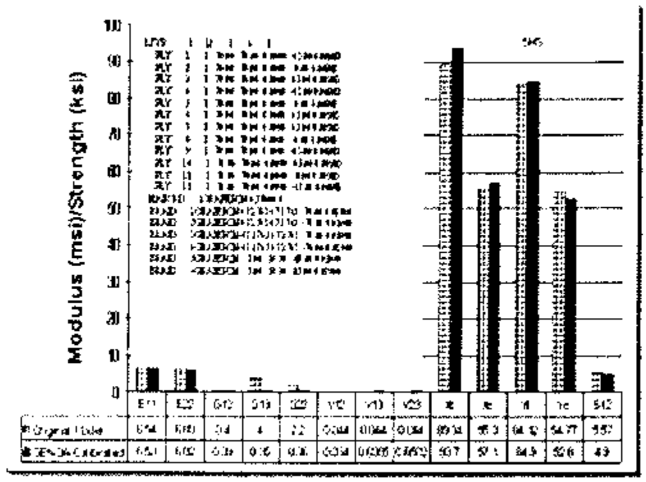


Figure 5. A Comparison between Calibrated and Original Mechanical Properties of 5HS Laminates

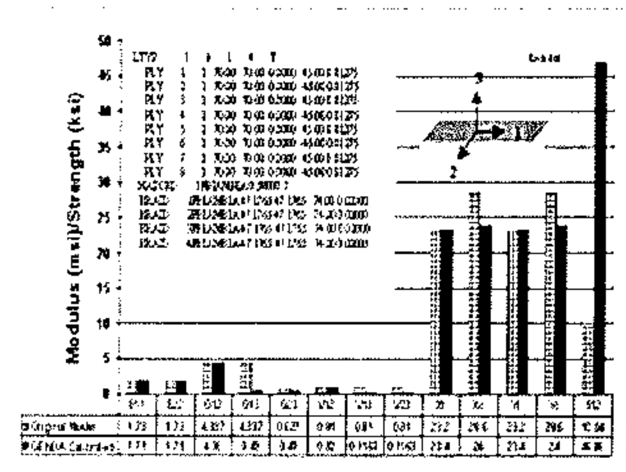


Figure 6. A Comparison between Calibrated and Original Mechanical Properties of the Biaxial Laminates

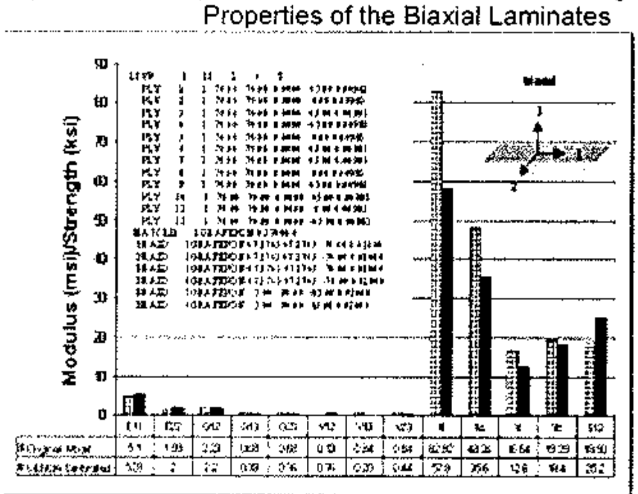


Figure 7. A Comparison between Calibrated and Original Mechanical Properties of the Triaxial Laminates

The maximum deflection at the working pressure was 3.31" (84 mm) as compared to 3.42" (86.86 mm) experimental value presented in DARPA (2000) report. The variation between the experimental and GENOA predicted maximum displacement was 3.4%. The ultimate failure was a combination of local damage of both the balsa wood and the composite sidewalls of the bridge (refer to Figures 9). A summary of both experimental and simulated results for the maximum shear loading case is shown in Table (1).

Case 2- Maximum Moment: Similar to Load Case Load 1, the NASTRAN FEM model was transformed into a GENOA numerical model. A progressive failure analysis (PFA) using GENOA simulation code was performed on the bridge model for Load Case 2 (*Max Flexural Loading)* shown in Figure (10) which is identical to the loading pattern used in the full-scale laboratory tests performed and described in DARPA (2000) report.

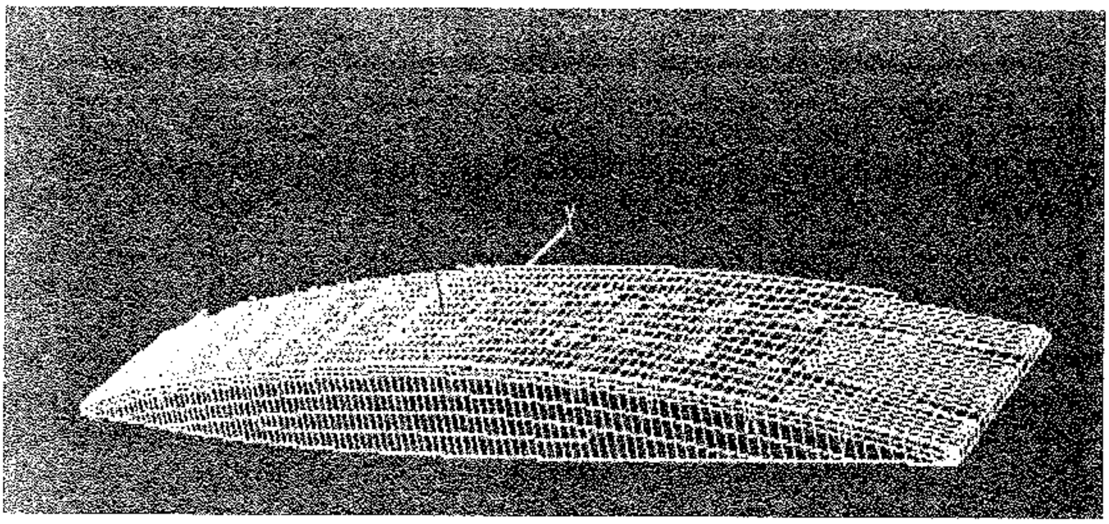


Figure 8. Static Shear Loading Pattern (Load Case 1)

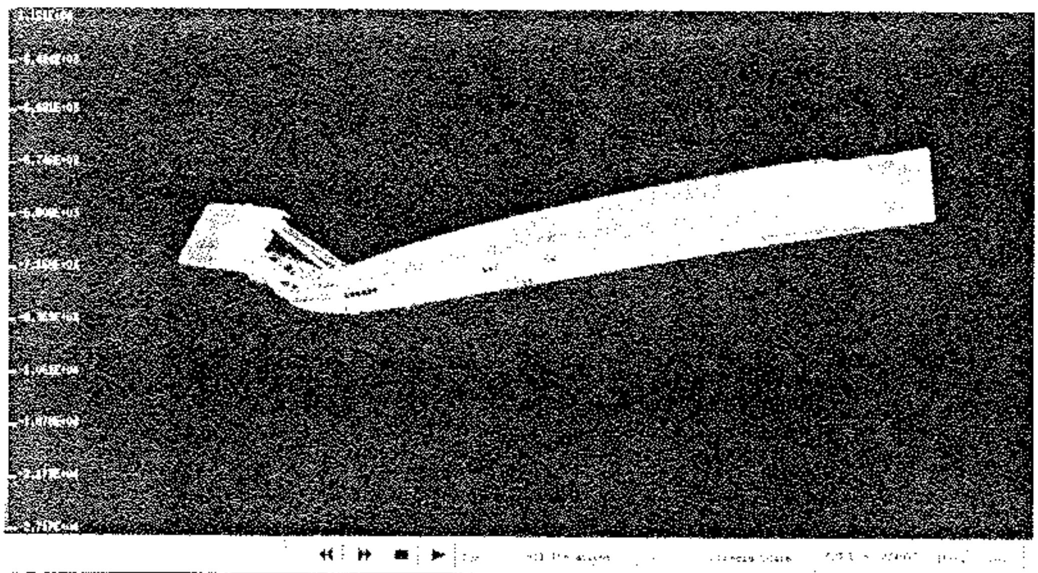


Figure 9. Generalized N_{xy} (lb-in) Shear Stress Distribution at the Ultimate Load *[280 kips (1,245.44 kN)]*

Table 1. Summary of Experimental and PFA Results for Load Case 1 (Max Shear)

Experimental Service Load	151 kips (672 kN)
GENOA Ultimate Load	280 kips (1,245.5 kN)
Experimental Ultimate Load	Not Available
Maximum Displacement at Service Load (GENOA)	3.31 inches (84.10 mm)
Maximum Displacement at Service Load (Experimental)	3.42 inches (86.87 mm)
Maximum Displacement at Ultimate Load (GENOA)	6.26 inches (159 mm)
Maximum Displacement at Ultimate Load (Experimental)	Not Available

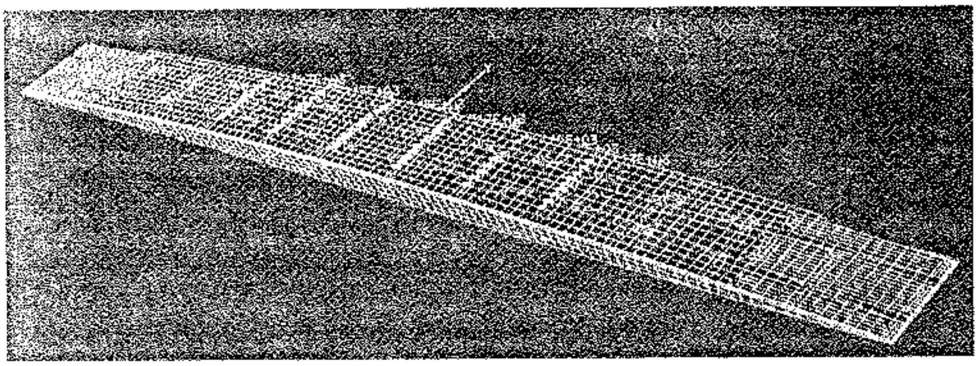


Figure 10. Static Flexural Loading Pattern (Load Case 2)

The GENOA-PFA simulation was initiated at a single actuator load level of 107 kips (*476 kN*). At this initial loading, no fractured nodes were formed. However, a total of 105 local damages were calculated. Although, no fractured nodes were observed, the number of local ply damages increased (*refer to Figure 11*).

The last equilibrium was achieved at a load level of 202 kips/898.50 kN per loading actuator which was set to be the ultimate failure load. At this load level, a total of 11 nodes were fractured resulting in ultimate failure of the bridge. The node damage distribution just before the ultimate failure is shown in Figure (11).

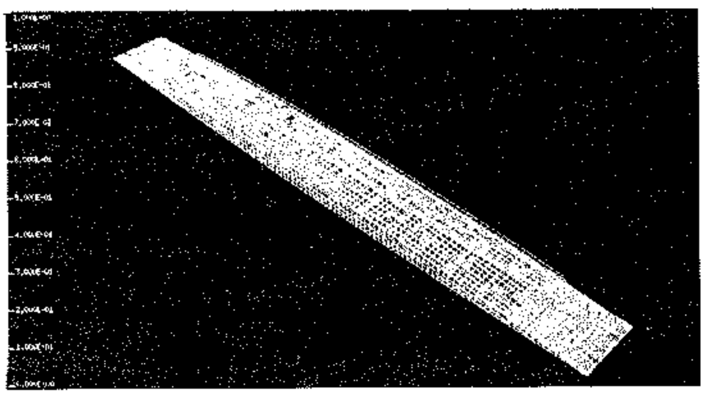


Figure 11. Nodes Damage Distribution at Initial Loading Level

As mentioned earlier, the majority of the local damages were concentrated at the span's mid-third in both the deck balsa wood core and the composite sidewalls/web as shown in Figure (12). The concentration of these localized damages led to the ultimate failure of the bridge treadway as shown in Figure (13). The location and mode of simulated failure were close to those observed in the laboratory test that were reported in DARPA (2000). The simulated maximum displacement at failure was 6.87" (*174.50 mm*) as compared to 6.80" (172.72 mm) as reported in Figure 142 of the DARPA (2000) report. A comparison between the simulated and experimental actuators loads are presented in Figure (14).

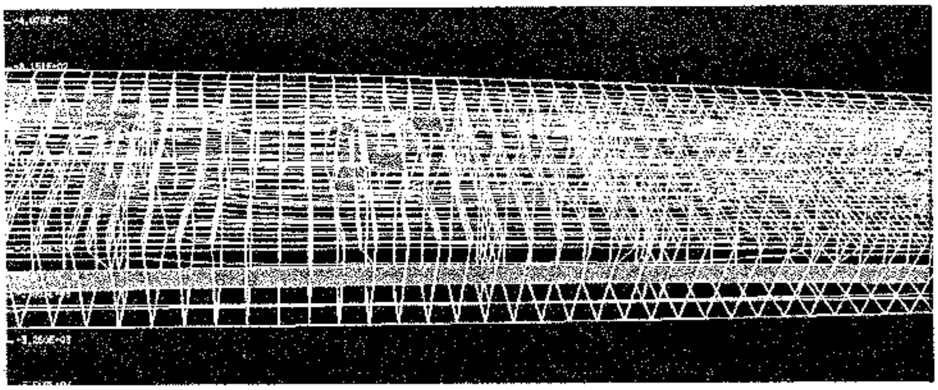


Figure 12. Zoom-In Top View of Damage Distribution Just before the Ultimate Failure

Progressive Fatigue Failure Analysis (Load Case 3): The GENOA computational simulation is implemented by the integration of three distinct computer codes that are used as the modules of a progressive fracture tracking code. These computational modules are: (1) composite mechanics, (2) finite element analysis, and (3) damage progression tracking. The overall evaluation of composite structural durability is carried out in the damage progression module that keeps track of composite degradation for the entire structure. The damage progression module relies on a composite mechanics code (*Nakazawa, et al. (1987)*) for composite micromechanics, macromechanics, laminate analysis, as well as cyclic loading durability analysis, and calls a finite element analysis module that uses anisotropic thick shell and 3-D solid elements to model laminated composites.

The composite bridge treadway was subjected to a cyclic loading up to failure. The fatigue analysis was performed for the maximum moment load case (Case 2) as shown in Figure (10). The GENOA-PFA simulation started at 5,000 cycles and was continued until ultimate failure occurred. In this analysis, a degradation factor of 0.1 was used to construct the assumed S-N relation with a stress ratio (R) equal to zero.

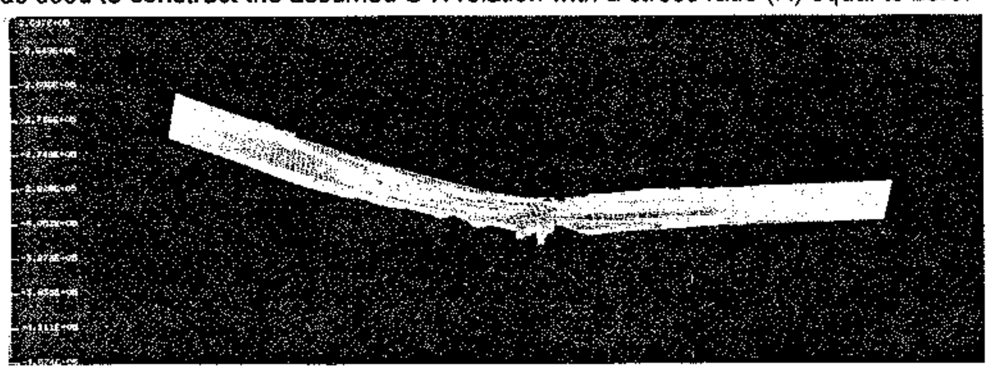


Figure 13. Ultimate Failure of the Composite Bridge Treadway

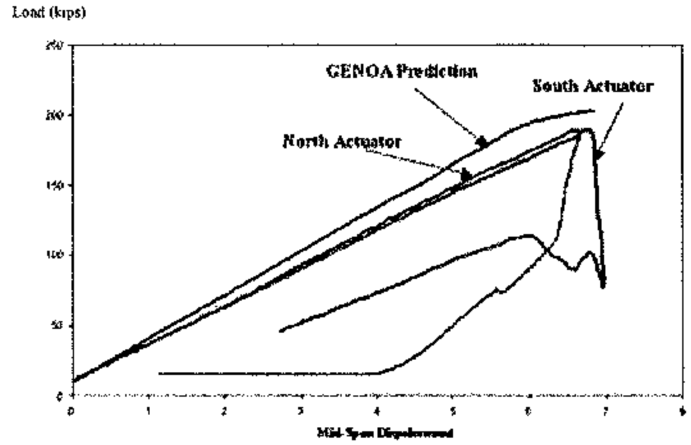


Figure 14. Comparison between Full-Scale Experimental and GENOA Predicted Results for Load Case 2 *(Max Flexural Loading Case)*

At the end of 11,250 cycles, the accumulative number of fractured nodes was five. As the number of cycles increased, the number of fractured increased. After 25,312 loading cycles, a total of 12 nodes were fractured. The last equilibrium prior to the ultimate failure was achieved after 128,140 cycles, which is the predicted fatigue life of the composite treadway. Figures (15) and (16) show the generalized N_x (lb-in) stress distributions after 128,140 cycles. The damage growth was monitored using the modified-distortion-energy-damage criteria.

The second part of the paper presented the numerical results that were accomplished using GENOA progressive failure simulation of the composite treadway under three different loading cases. The simulated results matched well the full-scale experimental results reported in DARPA (2000). For the fatigue case, a comprehensive study is now in progress that compares the simulated fatigue data with both the full-scale laboratory and field cyclic tests that were performed on both the treadway and a CAB prototype that was subjected to actual field loading conditions.

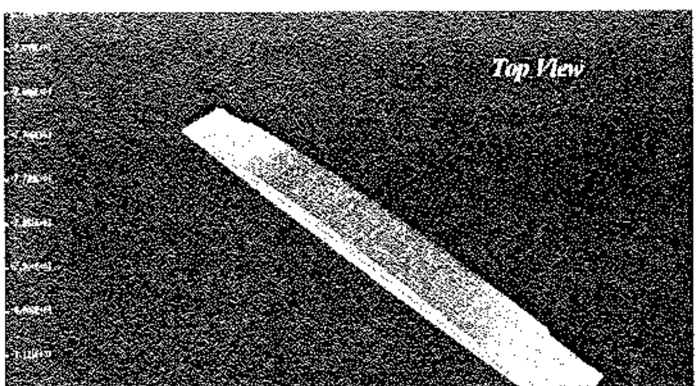


Figure 15. Generalized Normal Stress N_x Distribution after 128,140 Cycles *(Fatigue Life) – Top View*

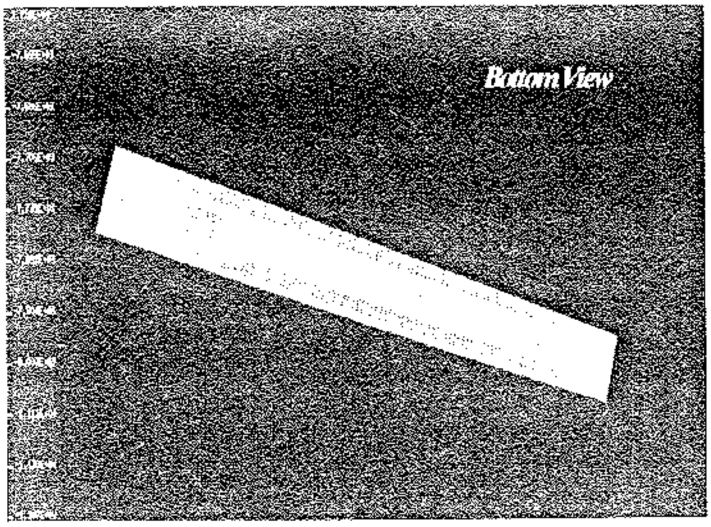


Figure 16. Generalized Normal Stress N_x Distribution after 128,140 Cycles *(Fatigue Life) – Bottom View*

SUMMARY & CONCLUSIONS

In this paper, the results of two studies were presented. The first part of this paper focused on calibrating the materials constituent properties of the composite army bridge that will be used in demonstrating and virtually verifying the smart repair technology for military bridges. This calibration process is essential in refining both the NASTRAN finite element (FE) and the GENOA Progressive Failure Analytical (PFA) models of the composite army bridge before and after damage as well as after performing the smart repair process. The results of the calibration process identified several variations, especially in the shear moduli, strength and Poisson's ratios. As expected, the original and the calibrated results for unidirectional laminates agreed well.

ACKNOWLEDGEMENTS

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ESTIMATING THE HYDRAULIC CONDUCTIVITY USING EMPIRICAL FORMULAE, FIELD PERMEABILITY AND PUMPING TESTS

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ABSTRACT: A series of pumping, in addition to field permeability, tests were conducted at four different sites, in Egypt, to determine the hydraulic conductivity of the water-bearing layer. These values were used in designing the during-construction groundwater control facilities at the four sites. In addition, at a series of laboratory tests, including grading analysis, for soil samples extracted during site investigation were performed. In this paper, results of these pumping and field-permeability tests are used to check the applicability of the indirect methods relating soil permeability to grading characteristics. Confined, unconfined and mixed aquifer conditions are characterized based on field measurements. Three different indirect methods are used, namely Hazen (1948), Shepherd (1989) and Elyamani and Sen (1993). Considering hydraulic conductivities extracted from pumping tests as reference values, it is found that it is more significant to estimate soil permeability using field technique rather than indirect methods. If indirect methods are only available, such as for tender preparation purposes, Hazen formula can give a good estimate.

Keywords: Aquifer, Hydraulic Conductivity, Groundwater, Permeability, Pumping Test.

INTRODUCTION

Estimating hydraulic conductivity of different types of soil is the most important step in designing any groundwater control system. So, different techniques were proposed to either directly measure or indirectly estimate its value. Direct methods, Fireman[1] (1944), are based on measuring the hydraulic conductivity of either undisturbed or disturbed soil samples using permeameter apparatus. Either constant or falling head tests can be performed, according to the type of tested soil. In addition, indirect methods, that are relating soil permeability to grading characteristics and physical properties, are widely used. Kozeny[2] (1927) presented the Kozeny-Carman equation to give the soil permeability or hydraulic conductivity, k, in terms of soil porosity, n, specific surface area, S_v, and the hydraulic gradient, i. Another similar attempts were made by Hazen, Terzaghi and Peck[3] (1962), Shepherd[4] (1989) and Alyamani and Sen[5] (1993). The applicability of these formulae depends on the type of soil for which hydraulic conductivity is to be estimated. Moreover, in-situ measurements of hydraulic conductivity attracted many investigators. Luthin[6] (1949), detailed the method of single auger hole to measure the hydraulic conductivity through pumping water out of a drilled borehole. Childs[7] (1950) proposed a similar technique, but adopting two-auger system, by pumping water from one to the other until reaching steady state conditions and measuring the amount of discharge, Q, hydraulic gradient, i, to obtain permeability, k. Furthermore, for the purposes of design finalization, full-scale pumping tests, using monitoring and pumping wells or even well points, can also be performed.

In this work, the applicability of the empirical formulae in addition to feasibility of using field permeability tests, such as pumping into boreholes, shall be investigated. Values of hydraulic conductivity obtained for four different sites, in Egypt, through performing full-scale pumping tests are analyzed and compared to those extracted from the use of either indirect methods or the field permeability tests.

GEOTECHNICAL AND HYDROLOGICAL CONDITIONS

Four different sites were studied. The first one, site (A), is located in the northern Egyptian zone, on the Mediterranean Sea cost, whereas the second site, (B), lies in Delta, where a major power plant is constructed. The third site, (C), is located in Sohag, in Upper Egypt, and the fourth one, (D), is in Giza governorate, where an elevated water tank is under construction. According to the performed soil investigations, in site (A), soil consists of fine beach sand extending from ground surface down to a depth of 13.40m followed by an approximately impervious layer of soft clay extends down to the end of boring, at a depth of 20.0m. Final groundwater table was observed at 0.95m from ground surface, NGS. In site (B), soil formation consists of surface fill layer of 1.0m thickness then a 5.5m thickness silty clay layer is encountered. Sand aquifer starts at 6.5m depth below ground surface and extends down to the end of boring, at 20.0m from natural ground. Groundwater is slightly under artesian pressure inside sand layer as the final piezometric surface was observed at 4.0m below ground surface while the top of water bearing sandy layer was at 6.5m. In site (C), a 4.0m thickness silty clay layer followed by sandy silt one of 6.0m thickness is found. Below sandy silt, fine sand is encountered, to the end of boring at 20.0m below ground surface. Final groundwater is measured at 3.8m below natural ground. In the last site, (D), soil consists of a surface layer of agricultural clay of 1.6m thickness followed by another medium sand layer of 3.4m thickness. Below the medium sand, a medium stiff to stiff clay is found to a depth of 20.0m, end of boring. Final groundwater table in boreholes is at 0.5m from ground surface. Geotechnical and hydrological sections representing the four sites are shown in Figure 1.

ESTIMATING HYDRAULIC CONDUCTIVITY FROM PUMPING TEST RESULTS

At steady state condition, the flow towards fully penetrating wells is governed by one of the two equations of both confined and unconfined aquifers, Batu[8] (1998). These equations can be written as:

$$H-h = (Q/2\pi kD) \ Ln \ (R_o/r) \qquad \text{(Confined Aquifer)} \tag{1}$$

$$H^2-h^2 = (Q/\pi k) \ Ln \ (R_o/r) \qquad \text{(Unconfined Aquifer)} \tag{2}$$

In both equations, H is the piezometric head before starting pumping; h is the steady-state head at a point located at a radial distance r from the center of the pumping well whose radius of influence is R_o and Q is the well discharge. D is the confined aquifer thickness. In fact, most of performed pumping tests are analyzed on the basis of one of the two equations given above. As a typical procedure, generally followed during performing all pumping tests at the four sites, well discharge, Q, and drawdown values, H-h, are measured during the test course. Hence, the radius of influence, R_o, and the hydraulic conductivity, K, can be obtained. Readings of drawdown are extracted at pre-scribed time increments until reaching constant value of drawdown, steady state conditions. At the end of test, dewatering well is shutdown and recovery readings are collected for possible time-dependant aquifer characteristics analysis.

Arrangement of Test Facilities at Different Sites

As mentioned above, full-scale pumping tests are performed to confirm the value of the hydraulic conductivity obtained using either field-permeability, falling-head type, or indirect methods. To perform these tests, a series of facilities such as pumping wells, PW, well points, WP, and piezometers, PZ, was arranged at each site. Figure 2 shows

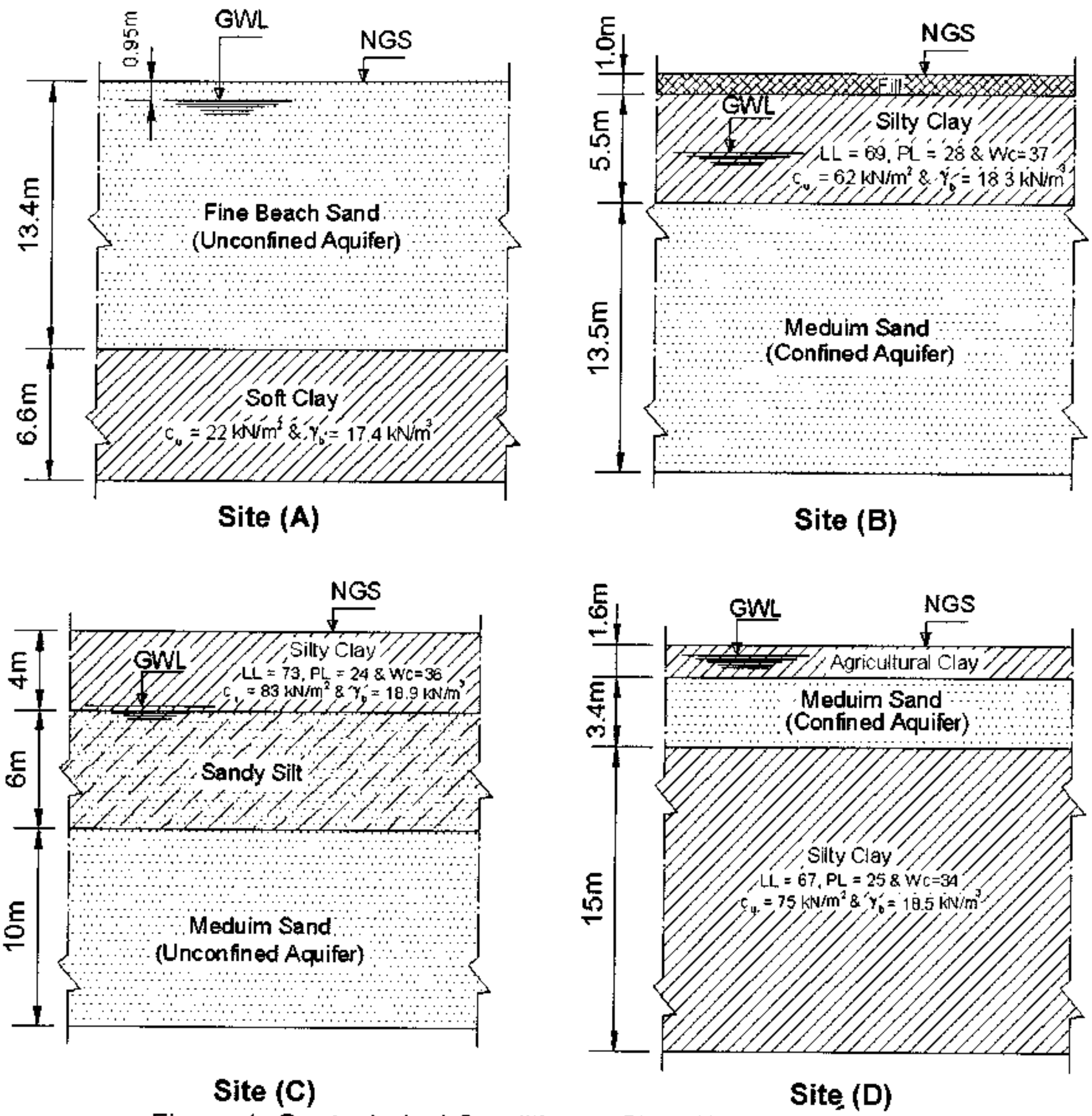


Figure 1. Geotechnical Condition at Sites (A), (B), (C) and (D)

the test arrangement adopted at site (A) in addition to the configuration of the used well and piezometers. The corresponding plot of site (D) is given in Figure 3. It may worth mentioning that the use of well points, as dewatering facilities, at site (D) was due to the limited thickness of the test aquifer. In the other sites, (B) and (C), a similar arrangement to that used in site (A) is adopted, but with different geometry, piezometer and well configurations.

Unconfined Aquifer Characterization, Site (A) and (C)

According to geotechnical and hydrological conditions presented in Figure 1, it can be easily noted that flow towards test well, in site (A), shall be characterized by the equation of the unconfined aquifer, Equation 2. It is foreseen that the 40cm diameter test well shall be operated to 20.0 m^3/hr. Consequently, when constructing the drawdown versus radial distance plot, the value of the radius of influence, R_0, is found

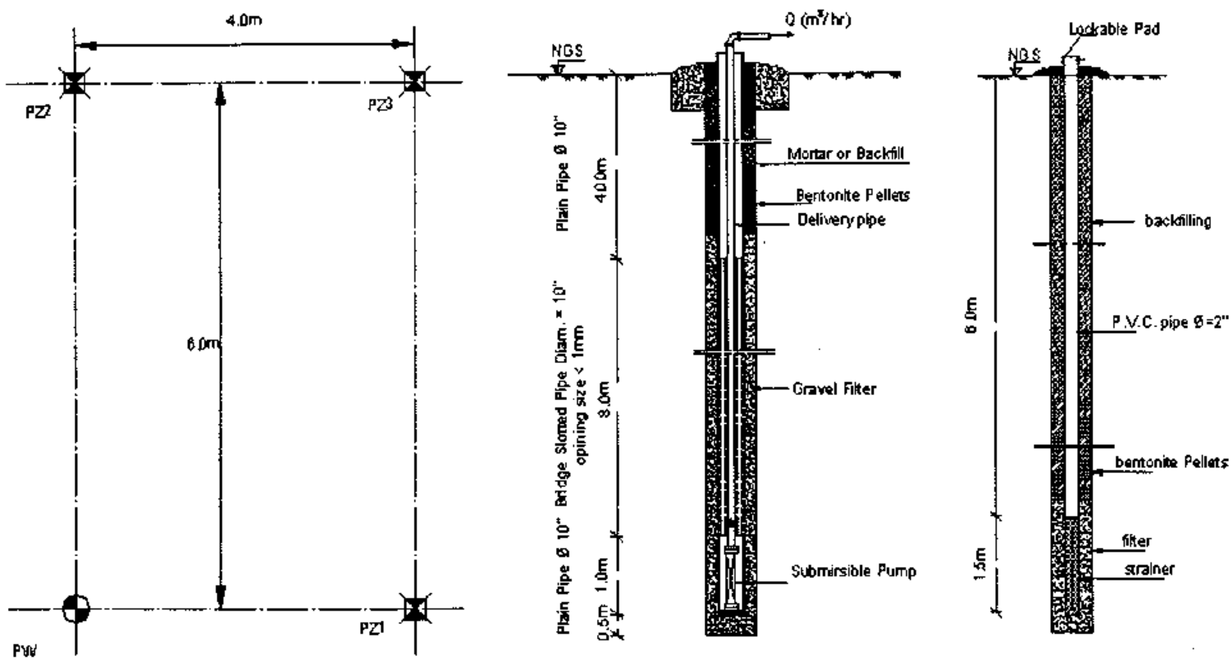


Figure 2. Site (A): Test Arrangement, Well and Piezometer Configurations

to be about 50m. Thus substituting in Equation (2), the hydraulic conductivity or permeability, *k* in m/s, can be obtained as:

$$k = [0.00556/(489-3.14h^2)\ Ln\ (50/r] \quad \text{"Site (A)"} \qquad (3)$$

In site (C), Q=30.0 m^3/hr, radius of influence, R_0, is estimated by 65.0m and H is assumed equal 20.0m. Similarly, the hydraulic conductivity, *k* in m/s, can be expressed as follows:

$$k = 0.0083/(60-3.14h^2)\ Ln\ (65/r) \quad \text{"Site (C)"} \qquad (4)$$

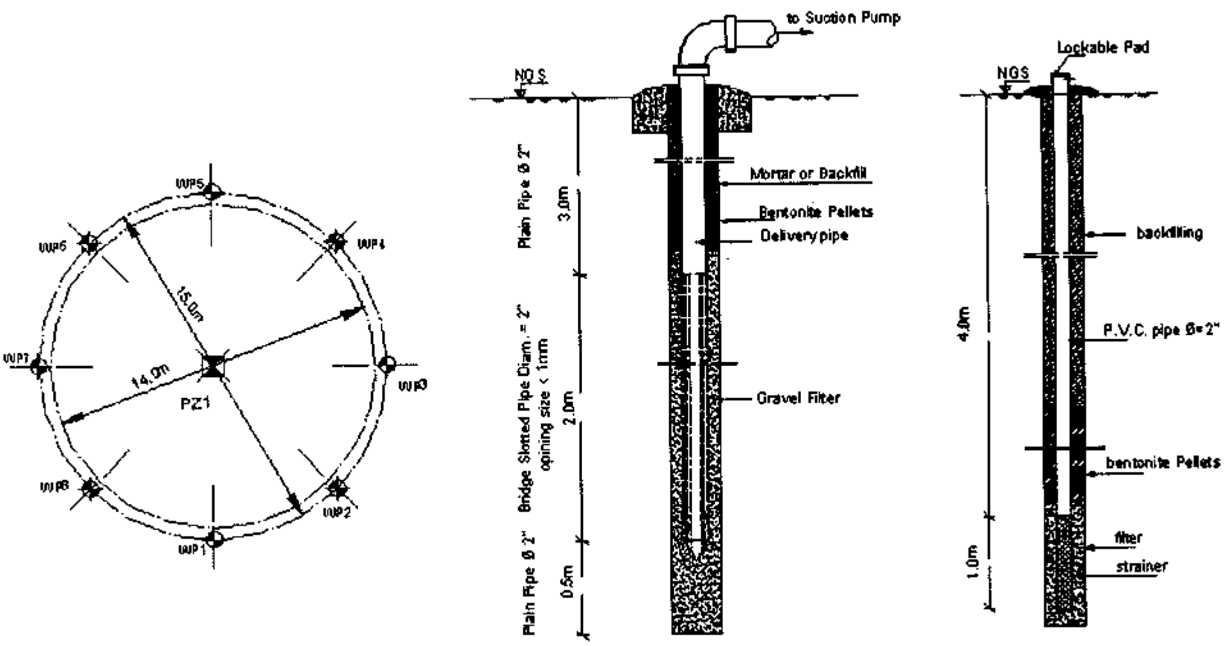
Confined Aquifer Characterization, Site (D)

It follows, from the inspection of the used well-point configuration, Figure 3, and the soil formation at site (D), Figure 1, that the flow is characterized by the equation of confined aquifer. Discharge, Q, for the well point is 3.0 m^3/hr and the aquifer depth, D, is 3.40m and H=4.5m. Radius of influence, R_0, is estimated by 20m. Thus, hydraulic conductivity, k in m/s, can be derived as:

$$k = 4x10^{-5}/(4.5-h)\ Ln\ (20/r) \qquad \text{"Site (D)"} \qquad (5)$$

Mixed Aquifer Characterization, Site (B)

The pumping test performed at site (B) represents the case that rarely occurs. This is due to the fact that before turning the discharging well on, the medium sand aquifer was under slight artesian effect, refer to Figure 1. When the test is started and steady state is reached, well discharge, Q, was about 70.0 m^3/hr and the drawdown values at the zone adjacent to the pumping well exceeded 2.5m. Thus, flow inside such zone becomes unconfined, approximately to a distance of 3.0m from centerline of pumping well. Next to that zone flow is confined as the observed free piezometric surface was higher than the bottom of clay layer, overlaying the test aquifer. Radius of influence, R_0, is found to be 120m. If H is assumed to be equal 20.0m, thus hydraulic conductivity, k in m/s, can be expressed as:



(A) (B) (C)

Figure 3. Site (D): Test Arrangement, Well and Piezometer Configurations

$$k = 0.006/(260-h^2)\ Ln\ (3.0/r) \qquad \text{"Site (B), } r<3.0m\text{"} \qquad (6)$$

$$k = 7.7x10^{-4}/(50-h)\ Ln\ (120/r) \qquad \text{"Site (B), } r>3.0m\text{"} \qquad (7)$$

ESTIMATED HYDRAULIC CONDUCTIVITY USING FIELD PERMEABILITY

Field permeability tests in boreholes, falling head type, are performed only at sites (A) and (B). Inflowing water is instantaneously recharged inside the boreholes drilled using temporary casing pipe of 8" diameter. After accurately measuring the groundwater table inside the borehole, test is started to estimate soil permeability at the level at which the casing pipe ends. Water level inside the casing is increased to an initial value Z_0. At any time t from test start, the acting head inside the casing, Z_t, is recorded. The value of (Z_t / Z_0) is then plotted versus t on a logarithmic plot. The time lag, T, is then obtained. The shape factor, F, is considered according to Hvorslev[9] (1951). Soil permeability, k, is then calculated as:

$$k = A/(FT) \tag{8}$$

A is the cross-sectional area of the casing pipe.

ESTIMATED HYDRAULIC CONDUCTIVITY FROM GRADING ANALYSIS

During conducting the boreholes at different sites, soil samples are extracted. After classifying these samples, a series of laboratory tests were carried out including grading analysis using standard sieves. The purpose of performing such test is to determine the grain size distribution curves of the test samples. These grading curves are given for the tested samples, extracted from different sites, in Figure 4.

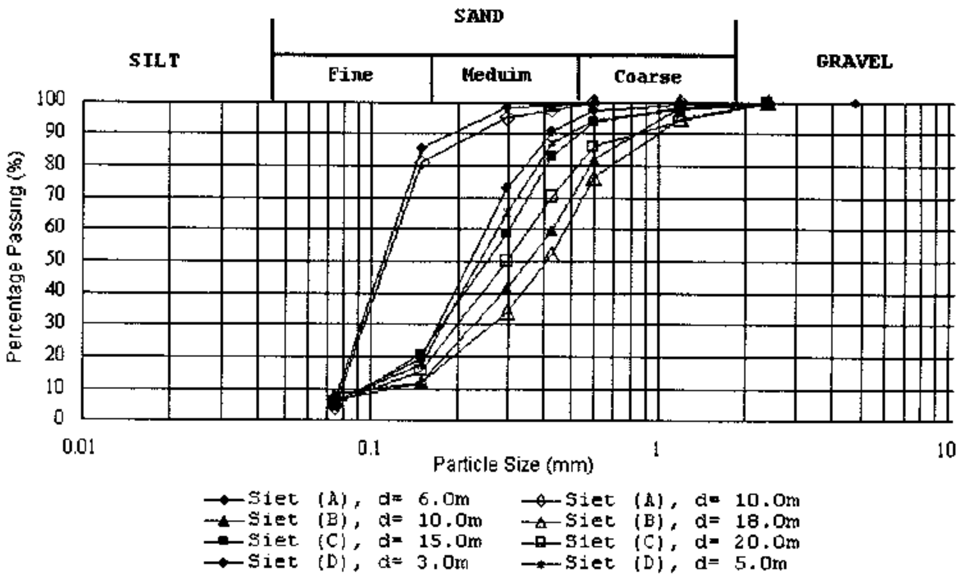


Figure 4. Grading Curves for Test Aquifers

In fact, hydraulic conductivity, k, can be estimated in terms of the characteristics of these curves. In this work, namely three methods are adopted. The first one is the traditional Hazen's method that directly relates the coefficient of permeability, k, to the effective diameter, D_{10}, which can be obtained from grading curves. According to Hazen, k is given by:

$$k = (D_{10})^2/100 \tag{9}$$

The second method used herein was presented by Shepherd (1989)[4] who may express the hydraulic conductivity, k, in terms of the effective grain size, D_{10}, as:

$$k = a\ (D_{10})^{b} \tag{10}$$

a and b are parameters depending on the nature of soil. Values of a and b used herein are 0.002 and 1.70, respectively. The last estimation of the permeability coefficient, k, was performed based on Alyamani and Sen (1993)[5] where k can be given by:

$$k = 0.015[I_0 + 0.025(D_{50}-D_{10})]^{2} \tag{11}$$

D_{50} is the grain size at 50% passing and I_0 is the intercept of the line formed by D_{50} and D_{10} with the grain size axis, i.e. has the same dimensions as D_{10} and D_{50}. It should be stated that in all above mentioned equations, D_{10} and D_{50} are in mm and the estimated value of k is in m/s.

RESULTS OF DIFFERENT APPROACHES

Hydraulic conductivity values are calculated using the indirect methods adopting the above mentioned equations, Equations 9 through 11. In addition, results of pumping and field permeability tests are analyzed to obtain permeability estimates, based on Equations 3 through 7. Since the analysis of pumping tests results in obtaining an equivalent value for the horizontal permeability of the aquifer, it is foreseen to obtain another estimate by averaging the values of field permeability test results over the depths. This is to obtain the equivalent average value over the aquifer depth. Results obtained for site (A) are given in Figure 5. It is clear that the permeability values reached for the site using either field permeability, averaged value, or the pumping tests do not differ by more than 8%. An excellent estimate can be obtained by Hazen formulae. Elyamani and Sen[5] (1993) overestimated the value of the hydraulic conductivity. This due to the fact that the aquifer soil is poorly-graded, fine beach, sand while Equation 11. is very sensitive to the slope of the D_{50}-D_{10} line, dispersion of the grain size, and its use may not be appropriate for such type of sand.

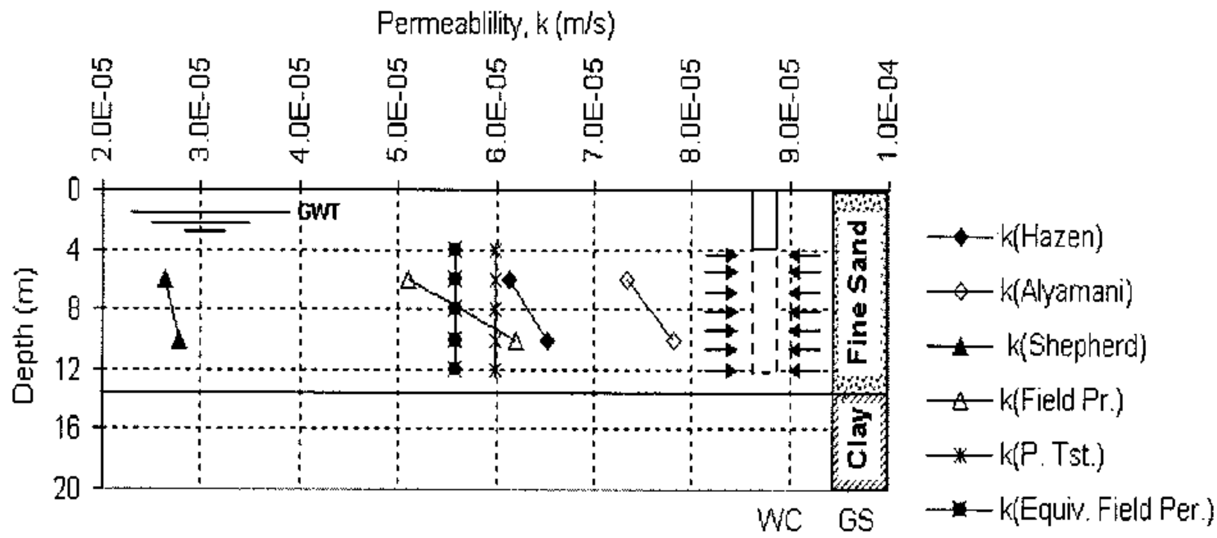


Figure 5. Site (A): Permeability Values obtained using Different Methods

The next results are presented for site (B), Figure 6. Similar to that obtained in site (A), good agreement, between pumping test and field permeability results, is obtained.

Hazen's formula underestimated the value by about 50% while the values resulted from the use of Shepherd[4] (1989) and Elyamani and Sen[5] (1993) are greatly underestimated. Thus, adopting such two methods for similar soil as that of site (B) can lead to insufficient dewatering system design. It could be generally noted that the most appropriate indirect method is Hazen's one since the other two indirect methods really misestimate the permeability value. This can be supported by the results of the other two sites, (C) and (D), Figures 7 and 8.

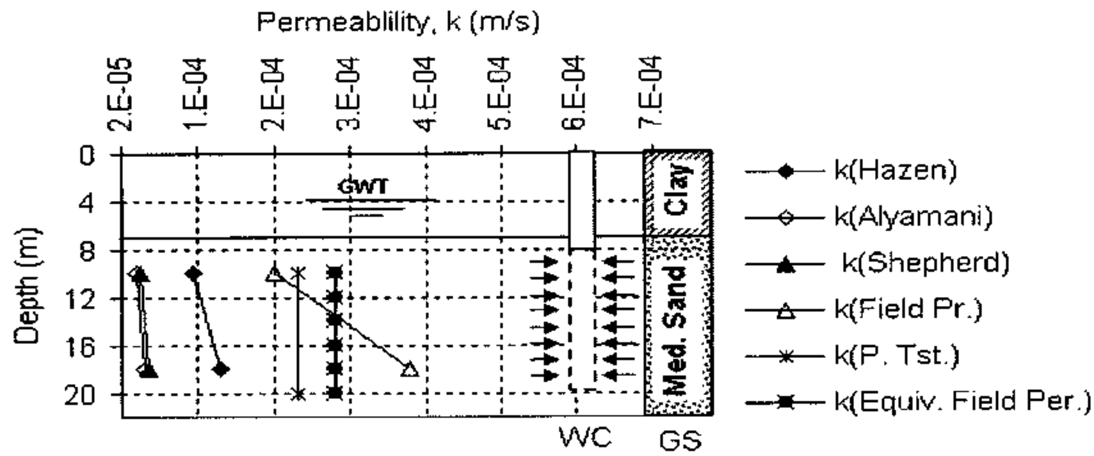


Figure 6. Site (B): Permeability Values obtained using Different Methods

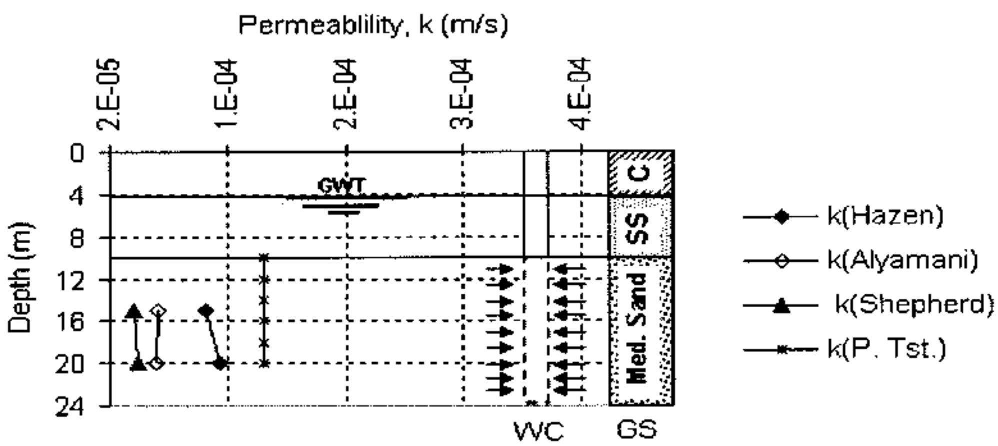


Figure 7. Site (C): Permeability Values obtained using Different Methods

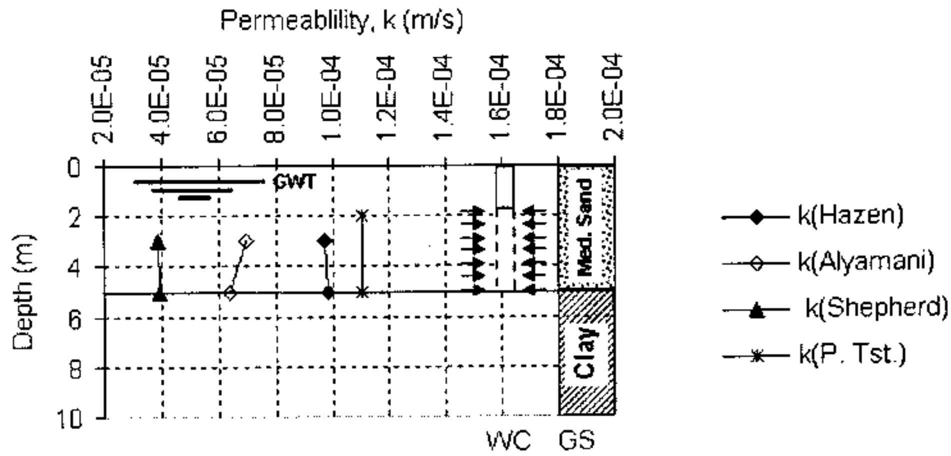


Figure 8. Site (D): Permeability Values obtained using Different Methods

CONCLUSIONS

Based on the aforementioned results and analysis, the following conclusions can be extracted:

(1) Estimating the hydraulic conductivity or soil permeability in terms of grading characteristics can relatively lead to under or overestimation unless the appropriate method is used.

(2) For the studied cases, and consequently may be for a wide range of soil type, the best estimation of permeability is reached based on Hazen's formula.

(3) The equation of Shepherd underestimates the hydraulic conductivity if compared to Hazen's one.

(4) Care should be taken when using the indirect methods to estimate the soil permeability as some equations are very sensitive to the shape of the grading curve such as that of Elyamani and Sen (1993).

(5) Values of field permeability are the most close to those of full-scale pumping test. This is provided that these values are averaged to get the equivalent permeability along the aquifer depth.

(6) Care should be taken when dealing with the pumping tests or system design in case of slightly confined aquifers. Unconfined zone can be generated and either test analysis of system design on the basis of totally confined aquifer may lead to inaccurate results.

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SAFETY AND COST CONSIDERATIONS IN SITE LAYOUT PLANNING

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ABSTRACT: Planning construction site layouts involves identifying, sizing and positioning of temporary facilities on site, and accordingly it has a significant impact on the safety and efficiency of construction operations. Although existing site layout planning models are capable of optimizing travel costs of resources on site, they do not consider safety as an important and independent objective in planning site layouts. This paper presents an enhanced site layout planning model that enables simultaneous optimization of construction safety and travel cost of resources. The model integrates two recently developed safety performance criteria that are designed to improve the safety of crane operations, and enhance the control of hazardous material on site. The optimization model is designed to consider all relevant decision variables, optimization objectives and practical constraints in this site layout problem, and is implemented using a multi-objective genetic algorithm. As such, the developed model enables construction planners to search for and identify optimal locations of all temporary facilities on site in order to minimize construction safety and cost simultaneously. These new capabilities should prove useful to construction planners and can lead to significant improvements in the safety of construction operations and the cost of constructed facilities.

Keywords: Site Layout Planning; Multi-Objective Optimization; Genetic Algorithms; Construction Optimization; Construction Safety; and Construction Planning.

INTRODUCTION

Construction site layout planning involves identifying, sizing, and on-site-positioning of temporary facilities which may include security fences, access roads, storage sheds, field offices, fabrication shops, sanitary facilities, electric power service, stockpiles of excavation, and batch plants. Developing and maintaining an effective site layout is a significant and critical task that should be properly performed and updated during the project planning and construction phases as it can lead to (1) reducing the costs of materials handling; (2) minimizing the travel times of labor, material and equipment on site; (3) improving construction productivity; and (4) promoting construction safety and quality[1, 2].

A number of studies were conducted in order to improve site layout planning in construction projects. These studies adopted a wide range of methodologies and development tools including neural networks[3], knowledge-based systems[4, 5, 6, 7], heuristics[8, 9], simulation[10, 11], and genetic algorithms (GAs)[12, 13, 14, 15, 16, 17, 18, 19]. Despite the contributions and practical features of available site layout planning models, they all focused on providing a solution that seeks to optimize the single objective of reducing travel distances of resources. In many real-world projects, this is often considered inadequate as other objectives such as improving safety may prove to be equally if not more significant. Construction safety is one of the important but least considered objectives in site layout planning and design[2]. This is particularly true in the construction industry which suffers from more accidents of greater severity than other industrial sectors[20]. The National Institute for Occupational Safety and Health (NIOSH) ranks the construction industry as the first in causing non-fatal injuries at a rate of 9.3 injuries per 100 full-time workers in 1997[21]. Moreover, the Bureau of Labor Statistics (BLS) ranks the construction industry among the top three industries causing fatal injuries in the United States in 2002 with a rate of 12.2 fatalities per 100,000 workers[22]. The total cost of accidents is significant and has been reported to reach up to 15% of the total costs of new construction[23]. These figures and statistics highlight the pressing need for further research efforts to develop advanced and expanded site layout planning models that are capable of considering and improving safety while seeking to minimize the travel cost of resources on construction site.

The main objective of this paper is to introduce new and innovative measures that can be utilized to quantify and maximize safety during the design of construction site layouts. The paper presents (1) relevant safety considerations and measures that affect the planning of construction site layouts; (2) applicable cost considerations that influence resources operation and materials handling on site; and (3) an optimization model to maximize construction operations safety and simultaneously minimize travel cost of resources on site.

SAFETY CONSIDERATIONS IN SITE LAYOUT PLANNING

Safety of construction operations is usually affected by many factors which include the site layout design, safety planning, personnel practices, and level of personnel training, among others. This study focuses on site layout design aspects that affect the safety of construction operations. A comprehensive literature review[2, 18, 24, 25] and several field studies were conducted in order to explore and identify relevant and important practical considerations that can enhance the safety of construction operations. This investigation led to identifying the following two key measures: (1) proper positioning of temporary facilities to improve crane operations safety and minimize accidents caused by falling objects; and (2) control of hazardous material and equipment on site. The following sections provide a brief discussion of these two measures, and the newly developed performance criteria to quantify their impact on construction safety.

Safety of Crane Operations

Statistics indicate that cranes and falling objects are among the major causes of construction accidents[2, 21, 26, 27]. The Occupational Safety and Health Administration (OSHA) reports that an average of 71 fatalities occur each year in the United States due to crane accidents[28]. To minimize the risk of such crane accidents, OSHA requires contractors to provide protection and safety measures against falling objects, especially below steel erection operations[25, 29]. In order to comply with these safety requirements, construction planners often seek to locate site offices and high occupancy facilities outside the reach of crane operations whenever possible. To facilitate the implementation of this site layout planning measure and the quantification of its impact on construction safety, this study presents a newly developed performance metric named crane safety criterion (CSC). This new performance metric (CSC) is designed to enable planners to measure and quantify the degree of safety due to positioning facilities in the neighborhood of cranes as a function of (1) the sensitivity (V_i) of each temporary facility i to potential falling objects from cranes, which can be used to represent the potential risk of injuries and/or fatalities if such an incident occurs; and (2) the distance (d_{ik}) between facility i and crane k, as shown in Figure 1 and Eqs. (1) and (2). First, the sensitivity of temporary facilities (V_i) can be specified by construction planners by selecting from three categories of low, medium and high sensitivity. Second, the distances (d_{ik}) between temporary facilities and cranes on site are automatically calculated and considered by the model for each generated site layout plan.

The sensitivity (V_i) of each facility and its proximity (d_{ik}) are the main variables that influence the crane safety indicator (CS_{ik}) due to positioning facility i in the neighborhood of crane k, as indicated in Eq. (2). This indicator classifies the space around the crane into three zones, as shown Figure 1. Zone 1 is the area covered by the crane jib ($d_{ik} < J+M/2$), and it represents the highest risk zone due to its vulnerability to falling objects from the crane during its operations. Zone 2 is located between zones 1 and 3 ($J+M/2 \le d_{ik} < R+J+M/2$), and it represents an intermediate level of risk due to its minor vulnerability to low probability crane accidents such as the tilting and/or collapse of the crane. Zone 3 lies outside the crane risk areas ($d_{ik} \ge R+J+M/2$), and therefore its safety is unaffected by crane operations (i.e. CS_{ik} = 100%). Based on the sensitivity of each facility (V_i) and its location (d_{ik}) within these three zones, each temporary facility i is assigned a crane safety indicator (CS_{ik}) due to its proximity to crane k, as shown in Eq. (2). The summation of all calculated indicators (CS_{ik}) for each combination of facility i and crane k is then averaged to identify the crane safety criterion (CSC) for the overall site layout plan, as shown in Eq. (1). It should be noted that the crane safety criterion (CSC) is formulated in a practical way that enables its measurement on a performance scale that ranges from 0 to 100%.

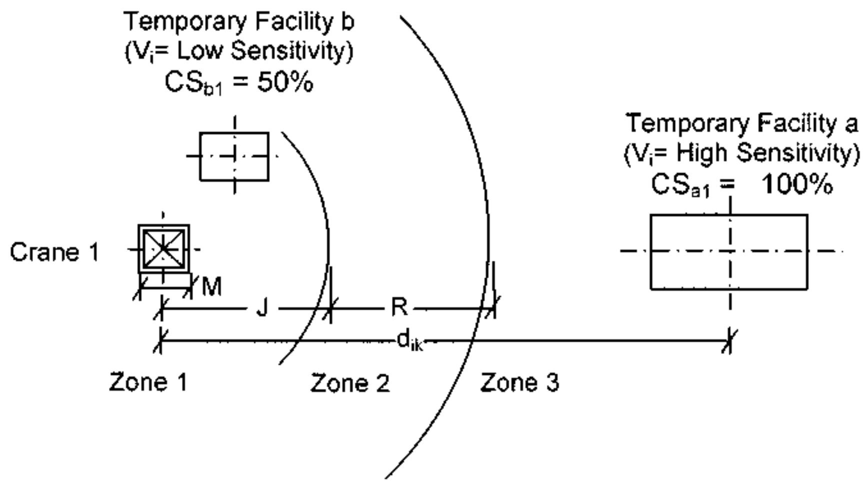

Figure 1. Evaluating Crane Safety Performance of Temporary Facilities

$$\text{Crane Safety Criterion (CSC)} = \frac{\sum_{k=1}^{K} \frac{\sum_{i=1}^{I} (CS_{ik})}{I}}{K} \tag{1}$$

$$CS_{ik} = \begin{cases} CS^1_{ik} = \begin{cases} 0\% \ (V_i = \text{High}) \\ 25\% \ (V_i = \text{Medium}) \\ 50\% \ (V_i = \text{Low}) \end{cases} & (\text{zone 1: } d_{ik} < J+M/2) \\ (100 - CS^1_{ik})(R+J+M/2 - d_{ik})/R + CS^1_{ik} & (\text{zone 2: } J+M/2 \leq d_{ik} < R+J+M/2) \\ 100\% \ (V_i = \text{High, Medium or Low}) & (\text{zone 3: } d_{ik} \geq R+J+M/2) \end{cases} \tag{2}$$

where

CS_{ik} crane safety performance of temporary facility i due to its proximity to crane k,
CS^1_{ik} crane safety performance of temporary facility i due to its proximity to crane k in zone 1,
V_i sensitivity of facility i to falling objects,
d_{ik} distance between facility i and crane k,
I total number of facilities on site,
K total number of cranes on site,
J length of the crane jib,
M width of the crane mast, and
R reach of the crane.

For example, Figure 2 shows two site layout scenarios to illustrate the use of the crane safety criterion (CSC) metric. As shown in Table 1, the CSC of site layout 2 is higher than that of site layout 1, reflecting that site layout 2 is safer than site layout 1. To ensure the validity of the proposed model, the safety performance metrics were also tested collectively in larger case studies, taking into account cost considerations[30]. The results of all this analysis confirmed the validity and the practically of the presented concepts[30].

Table 1. Crane safety criterion for layouts 1 and 2

Facility	Vi	CS^1_{ik}	Layout1				Layout1			
			d_{i1}	CS_{i1}	d_{i2}	CS_{i2}	d_{i1}	CS_{i1}	d_{i2}	CS_{i2}
1	High	50	44.13	0.00	158.49	100.00	121.15	100	162.66	100
2	Medium	25	89.07	81.08	118.19	100.00	146.92	100	171.93	100
3	Low	0	103.35	27.17	99.69	39.37	129.78	100	130.21	100
4	Medium	25	117.63	100.00	85.18	90.80	165.65	100	145.42	100
5	Medium	25	149.95	100.00	50.61	25.00	161.88	100	118.67	100
ΣCS_{ik}				308.24		355.17		500		500
$\Sigma CS_{ik}/I$				61.65		71.03		100		100
$\Sigma(\Sigma CS_{ik}/I)$						132.68				200
Crane Safety Criterion (CSC)						66.34%				100.00%

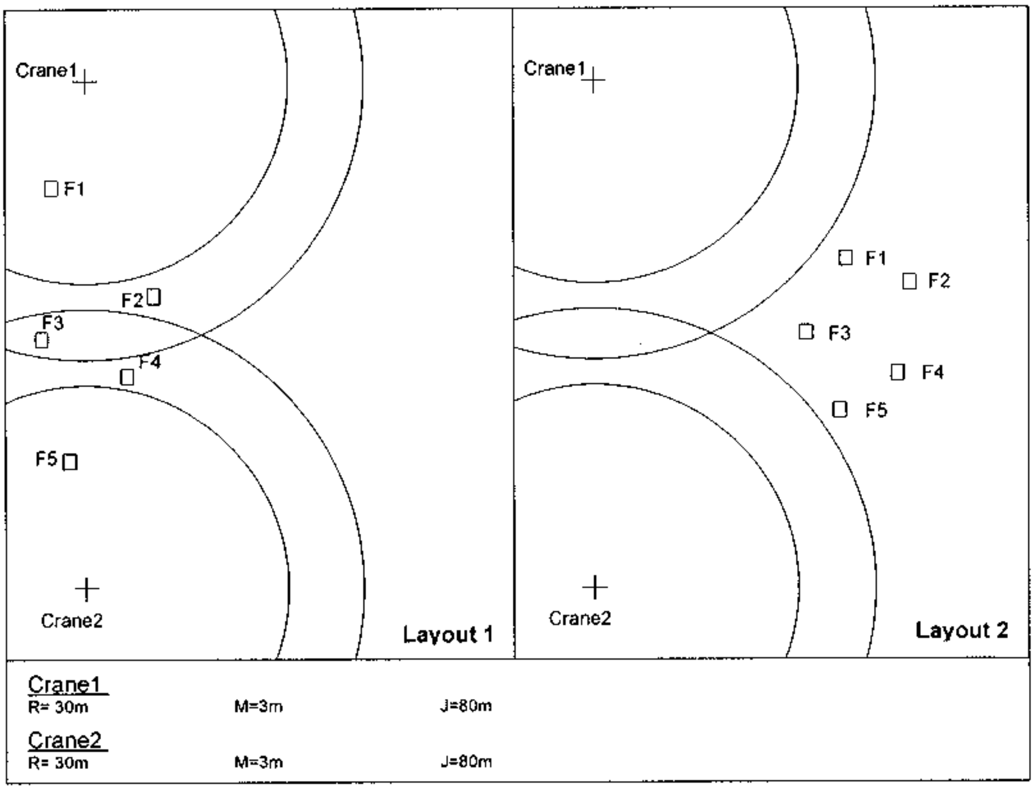

Figure 2. Evaluating Crane Safety Performance of Temporary Facilities

Control of Hazardous Material

Hazardous material and equipment are often utilized and located on construction sites, exposing construction workers and engineers to safety risks[29]. Hazardous material include (1) explosives and blasting devices used in rock excavation; (2) flammable material such as fuel used by construction equipment; (3) toxic substances such as asbestos, coal tar pitch volatiles, cadmium, benzene, formaldehyde, methyl chloride among other materials including 13 carcinogens identified by OSHA[25]; and (4) sources of harmful radiation and high electric voltage. These hazardous material and equipment need to be properly stored and adequately separated to minimize the risk of accidents on site. For example, OSHA standard 1926.407 recommends storage facilities of electrical equipment and possible sources of sparks be located far away from flammable material[25].

In order to improve safety on construction sites, planners need to comply with OSHA standards and identify proper storage locations for all hazardous material on site. These locations should be selected to ensure that there is adequate separation between (1) specific combinations of material and/or equipment that can create hazardous conditions on site (e.g., explosives and blasting devices); and (2) hazardous material and workers. In order to support planners in this vital site layout planning task, this study proposes using a new performance metric named hazards control criterion (HCC). This new performance metric is designed to enable planners to measure and quantify the degree of hazard control on site as a function of (1) the hazard control weight (HCW_{ij}) of facilities i and j which represents the degree of hazard that can be encountered on site if the two facilities are not adequately separated; and (2) the separation distance (d_{ij}) between facilities i and j, as shown in Eq. (3). The value of hazard control weight between facilities i and j is to be assigned in the range from 0% to 100% for combinations of facilities that create no hazards to those that pose the highest level of hazard if they are not separated on site.

$$\text{Hazards Control Criterion (HCC)} = \sum_{i=1}^{I-1} \sum_{j=i+1}^{I} \left(HCW_{ij} \times d_{i,j} \right) \tag{3}$$

$$d_{ij} = \sqrt{(X_i \quad X_j)^2 + (Y_i \quad Y_j)^2} \tag{4}$$

where

HCW_{ij} hazard control weight that represents the risk of accidents that can be encountered on site if facilities i and j are not adequately separated,

d_{ij} separation distance between facilities i and j,

X_i, Y_i coordinates of center of gravity of facility I,

X_j, Y_j coordinates of center of gravity of facility j, and

I total number of facilities on site.

It should be noted that unlike the earlier described crane safety criterion (CSC), the present hazards control criterion (HCC) values do not necessarily fall within a performance range of 0% to 100%. In order to ensure consistency in performance measurement among all the safety criteria in the present model, each calculated HCC value in Eq. (3) is normalized to generate a normalized performance metric (NHCC) that ranges from 0% to 100%, as shown in Eq. (5). NHCC is calculated for each HCC value, using the maximum (HCC_{max}) and minimum (HCC_{min}) values of all generated site layouts.

$$\text{Normalized Hazards Control Criterion (NHCC)} = \frac{HCC - HCC_{min}}{HCC_{max} - HCC_{min}} \tag{5}$$

where

HCC calculated hazards control criterion, using Eq. (3),

HCC_{min} minimum HCC value obtained from all generated site layouts, and

HCC_{max} maximum HCC value obtained from all generated site layouts.

Overall Construction Safety

The present model is designed to aggregate the above described two safety criteria into a single objective function to facilitate the evaluation of the overall safety performance for each possible site layout plan. To this end, the present model incorporates a weighted average formula that depicts the overall safety performance on site, as shown in Eq. (6). In this formula, the relative weight/significance (w_1 and w_2) of the two safety criteria can be specified by construction planners according to the special conditions of the project being considered. They can also be determined by analyzing historical records to compare the rate of accidents attributed to the inefficiency of enforcing these two safety measures. For example, the accidents data collected by BLS during the period from 1992 till 2001[22] can be used to assign a relative weight of 70% and 30% for the crane safety criterion and the hazard control criterion, respectively.

$$\text{Maximize Construction Safety} = \text{Maximize}\ [\,w_1 \times CSC + w_2 \times NHCC\,] \qquad (6)$$

where

w_1 relative weight or scaling constant of crane safety criterion; and

w_2 relative weight or scaling constant of hazards control criterion.

COST CONSIDERATIONS IN SITE LAYOUT PLANNING

The cost of construction operations is affected by the location of temporary facilities and the travel cost of resources (e.g. materials, equipment and labor) on site. In order to enable the search for and identification of the location of temporary facilities that minimize this type of construction costs, the present model utilizes an objective function that seeks to quantify and minimize the travel costs of resources on construction sites, as follows:

$$\text{Minimize Travel Cost of Resources} = \text{Minimize} \sum_{i=1}^{I-1} \sum_{j=i+1}^{I} (C_{i,j} \times d_{i,j}) \qquad (7)$$

$$d_{ij} = \sqrt{(X_i - X_j)^2 + (Y_i - Y_j)^2} \qquad (8)$$

where

$C_{i,j}$ travel cost rate in \$/meter of distance traveled between facilities i and j.

dij distance in meters between facilities i and j;

X_i, Y_i coordinates of center of gravity of facility i;

X_j, Y_j coordinates of center of gravity of facility j; and

I total number of facilities on site.

OPTIMIZATION MODEL

An optimization model is developed to support construction planners in identifying near optimal locations for all temporary facilities on construction sites such as storage areas of material and equipment, stockpiles of excavation, site offices, fabrication shops, and batch plants. The model is developed using a multi-objective genetic algorithm to enable the simultaneous maximization of construction safety on site and minimization of travel cost of resources[30]. As such, the considered decision variables in the present model are the coordinates (X_i,Y_i) of the center of gravity of each temporary facility (i = 1 to I). In the present model, these variables are represented by an artificial genetic chromosome that depicts the coordinates (X_i, Y_i) of each temporary facility (i) on site and they are among the main output data of the model. The designed chromosome enables the present model to evaluate the impact of various site layout solutions (S_1 to S_N) on safety and travel costs of resources. The planner needs to provide the dimensions of temporary facilities, while their near optimal locations are identified by the model, as shown in Figure 3. The optimization model starts its operations with initialization of random site layout solutions and then runs a series of evaluation, modification and combination of good solutions, attempting to reach optimal solution that maximize safety of construction operations and minimize travel cost of resources on site[30]. The model was tested on a number of application examples to validate its use. The results of the analysis highlight the new and unique capabilities of the developed site layout planning model and its usefulness in generating optimal tradeoffs between these two objectives[30].

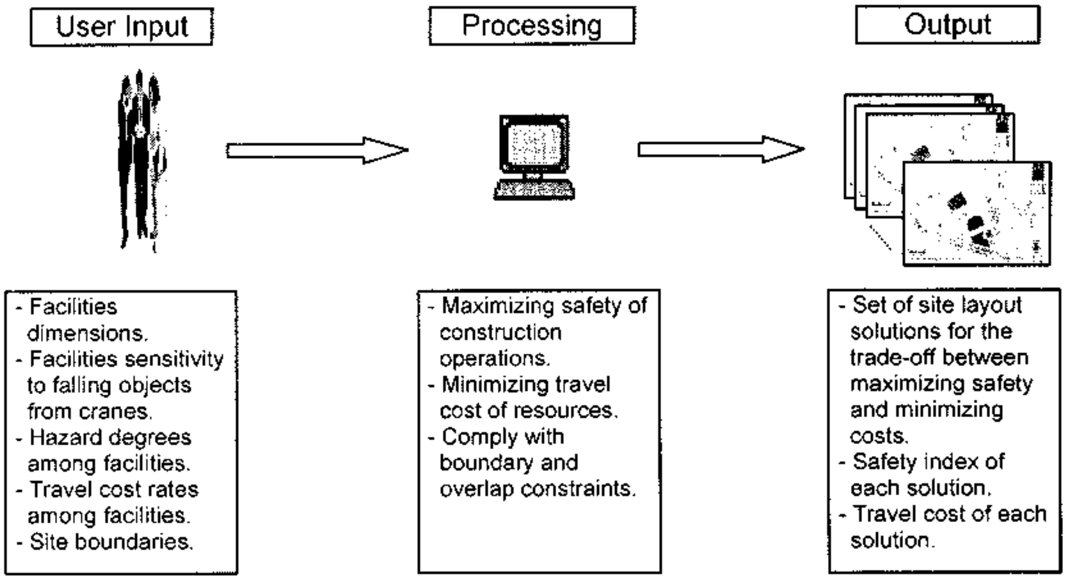

Figure 3. Data Input and Output of the Optimization Model

Furthermore, the developed optimization model is capable of considering all practical site layout constraints, including (1) boundary constraints; and (2) overlap constraints. The purpose of boundary constraints is to ensure that temporary facilities are located within the site boundaries, while overlap constraints are required to avoid the overlap of facilities on site[30]. This illustrates the practicality of the developed model and proves its usefulness in designing appropriate site layouts.

SUMMARY AND CONCLUSIONS

This paper discussed significant safety and cost considerations in site layout design. Two performance metrics were newly developed in order to (1) improve the safety of crane operations; and (2) enhance the control of hazardous material on site. The first metric is designed to search for safe locations of temporary facilities in an attempt to minimize the risk of crane accidents and falling objects. The second metric is designed to ensure that there is adequate separation between specific combinations of material, workers and/or equipment that can create hazardous conditions on site. These metrics were integrated with a third metric that quantifies the impact of various site layout plans on construction cost, using a multi-objective optimization model. The model can be used to search for and generate optimal arrangements of temporary facilities that provide optimal tradeoffs between safety of construction operations and travel cost of resources on site, while satisfying all practical constraints in this construction problem. This should prove useful to construction planners and can lead to significant improvements in the safety and cost of construction operations.

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SURVEYING AND MANAGING ACCESSIBILITY TO PUBLIC SPACE IN GREAT URBAN AREAS

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ABSTRACT: Pedestrian accessibility of urban public space in cities is a branch of the wider discussion about life quality and, more specifically, urban quality. This paper describes a system to support the management of urban public space. A specific field of application is assumed: it concerns the problems of accessibility of the public spaces by "weak" users and in particular by disabled persons. The system is a part of a wider project, promoted by Rome's public transport board, within the framework targeted to the enhancement of citizens and tourists mobility. Aims of the project are: to define a methodology to survey and to analyse the accessibility of urban pedestrian routes; to provide information on the accessibility of pedestrian routes for users with impaired mobility and/or sensory disabilities (user-oriented product); to produce an informative system with a geographic component, able to support the modification process (transformation - re-qualification) of the city public space. Analysis and evaluation of accessibility level of urban public space are based on GIS technology. In the paper, we firstly outline the assumed management framework; then a short description of survey methodology is given., including a synthetic specification of the gathered geographical data base; lastly we discuss methodologies and techniques to measure accessibility and to evaluate usability of public space, developed in GIS environment.
Project was tested on sidewalks within a distance of 150 meters from tramway in Rome city centre.

Keywords: accessibility, management of public space, GIS technology

OBJECTIVES AND BACKGROUND PROBLEMS

The aims pursued in the research on pedestrian public space, promoted by Rome's Transport Board, were quite broad; they included:

- The study of a comprehensive methodology finalised to the knowledge of the urban pedestrian public space, aiming to:
  - support the making of urban policies and intervention programs oriented to the improvement of pedestrian accessibility

- support the programming of interventions in defined urban areas
- support the making of projects for accessibility improvement
- distribute information to users on public space accessibility conditions
- The operational development of methodology, including:
  - the development of tools for geographical management of information on public pedestrian space;
  - the actual application of methodologies and tools on a wide urban area, in the city of Rome.

The setting of the overall methodology was focused on a range of relevant themes, that can be summarised as follows:

- The dimension of the problem

The present lack of urban quality depends on absence of planning actions specifically applied to pedestrian space and extended on the whole urban area. Thus, an improvement strategy in the urban environment has to comprehend extensive interventions on public spaces, while only efforts to ameliorate single sites in the cities have been performed so far. Instead, the methodology had to deal with i) medium-large sized cities, ii) the whole urban area (not only historical centre or residential or business districts ...), iii) a medium-long time for the application of improvement policies, iv) pedestrian spaces accessibility in bad repair (due to geometrical characteristics and/or current use).

- The process

The public pedestrian space is the site where many subjects act, simultaneously and/or in different times, each one having their own functional objectives to reach. Thus, the physical making of the public space is a typical multi-actor - multi-objective process, occurring over the time. If co-ordination among actors is not provided, then the product will be the present space, resulting in a casual pattern of objects, coming up from the independent (i.e. not mutually verified, but cumulative) actions of all the operating subjects. The methodology has to provide a common basis for the needed co-ordination, that is for the compatibility evaluation both of new interventions, as of the current condition.

- The "rules"

The few available design "rules" for pedestrian public space seem not to be the proper ones, not even to ensure access to disabled people. Nor it is reasonable to draw rigid and/or complicated new rules to solve the problem. Thus, as a minimal characteristic for quality, the "usability" of public space has been assumed; and afterwards the "accessibility" is defined as a representative performance indicator. Such measurement can be built in a quite simple way, when having the suitable information on the state of public space and on the request for new utilisation.

THE MANAGEMENT REFERENCE PROCESS

The aim of a very extensive improvement of the pedestrian urban space implies necessarily a long time and, consequently, a "stepped" and spatially distributed strategy. Firstly, the stepping of strategy is discussed, differentiating what has to happen for the re-establishment of the minimum level of "usability" (pedestrian accessibility), from what has to be managed in the following "routine" phases. The strategy will then be referred to the problems arising from the articulated geography of urban areas, mainly with the purpose of setting tools aiding the decision on spatial application of policies.

Let now sketch the actions to carry out at the beginning of the process of improvement; we will refer to generic urban area, for which it is already assumed the decision to intervene.
As a starting point, the current condition of public space is considered; it is the result of past actions performed by many actors, without any co-ordination or explicit and specific planning. In Figure 1. it is the period (t_0). The actions in this period are those directed to the re-establishment of the minimum level of "usability", after which a "routine" management should begin, in case with further improvement actions (period t_1). The foreseeable actions in the two different periods are like, with some meaningful difference.
The operative actions in the period t_0 should be:

- Survey; usually it is necessary for the lack of a detailed and updated cartography at a proper scale. We must underline that survey technicalities have to ensure an information output both efficient (remember the extent we aim to cover) and effective (as a good basis for the subsequent operations)

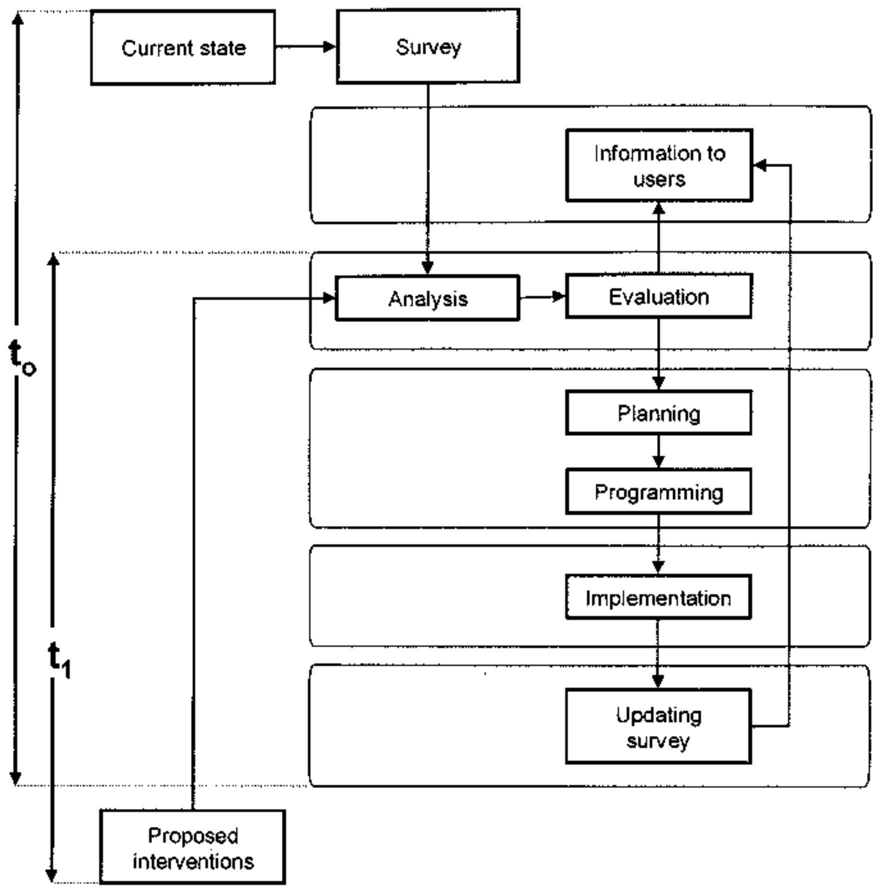

Figure 1 - Activities framework

- Analysis; a classification has to be done, in which each elementary module of urban pedestrian space (the front of buildings) will be marked depending by its complexity, both physical-technological and managerial (number of subjects using the space). This classification will show the technical difficulty to intervene on the space modules.

- Evaluation; the theme being the accessibility, firstly a measurement of that has to be taken (with a geometrical and topological analysis); then an assessment of the actual condition will be performable, considering all kind of obstacles in the existent path. This will produce a new classification of pedestrian space modules, that will show the user's difficulty.

From the evaluation block the actions flow towards two different branches; the first one leads to an activity of information transfer to users.

Along the second flow branching out from the block we find the actions directed to the functional improvement; they are:

- Planning; we assume the planning activity as merely including the only functional contents, with no regard for aesthetics; that is to take into account exclusively the accessibility problem. The plan will provide only interventions needed to reach the threshold of the minimum accessibility.;
- Programming; the given plan has now to be implemented in a given "action area": that area is a "living environment", whose life must be troubled as less as possible. Thus, the programming activity has to define a good sequence in time and space for the interventions foreseen;
- Implementation; the physical realisation of the interventions will be driven by the output of the programming activity. A reasonable time-space works sequence has to take into account both the impact on the urban life and the technical and economic logic of the contractors;
- Survey updating; this activity can be performed on the basis of plans, but not forgetting that many changes may occur during the implementation. Objects displaced, removed, changed or newly inserted must be updated on the whole layout;
- Information to users; a new version must be generated, updating the previous one.

After the first period, which is dedicated to the rehabilitation of pedestrian space, it has to be expected that new needs of change will arise. So the problem in the subsequent periods is to control the sustainability of transformation proposals coming up from subjects using or willing to use the space for their objectives.

The operative actions are quite similar to the previous ones, but in this case they are applied to the proposal of changing the public space utilization, coming from both public and private subjects.

Also at this moment we need to analyze and evaluate the compatibility or the interference among such proposal and the existing objects. A new connotation we have to associate to the planning, programming and implementing activities, due the needing of coordination between the subjects acting in space modification. However, after the implementation, we need to update the current configuration of public space, and to regenerate the information for the pedestrian users.

Levels

As we told, the outlined process is referred to a single generic urban area. Though a "low profile" goal is been assumed for the improvement of public space (The Minimum Level Of "Usability"), it is obvious that no authority in an only middle-sized city could perform simultaneously those activities (period t_0) over the whole urban area (and we have to recall that our reference extension IS the whole urban area). Thus, a permanent policy, to stand firm over a medium-long time, and its spatial break down are needed.

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SAFETY AND COST CONSIDERATIONS IN SITE LAYOUT PLANNING

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ABSTRACT: Planning construction site layouts involves identifying, sizing and positioning of temporary facilities on site, and accordingly it has a significant impact on the safety and efficiency of construction operations. Although existing site layout planning models are capable of optimizing travel costs of resources on site, they do not consider safety as an important and independent objective in planning site layouts. This paper presents an enhanced site layout planning model that enables simultaneous optimization of construction safety and travel cost of resources. The model integrates two recently developed safety performance criteria that are designed to improve the safety of crane operations, and enhance the control of hazardous material on site. The optimization model is designed to consider all relevant decision variables, optimization objectives and practical constraints in this site layout problem, and is implemented using a multi-objective genetic algorithm. As such, the developed model enables construction planners to search for and identify optimal locations of all temporary facilities on site in order to minimize construction safety and cost simultaneously. These new capabilities should prove useful to construction planners and can lead to significant improvements in the safety of construction operations and the cost of constructed facilities.

Keywords: Site Layout Planning; Multi-Objective Optimization; Genetic Algorithms; Construction Optimization; Construction Safety; and Construction Planning.

INTRODUCTION

Construction site layout planning involves identifying, sizing, and on-site-positioning of temporary facilities which may include security fences, access roads, storage sheds, field offices, fabrication shops, sanitary facilities, electric power service, stockpiles of excavation, and batch plants. Developing and maintaining an effective site layout is a significant and critical task that should be properly performed and updated during the project planning and construction phases as it can lead to (1) reducing the costs of materials handling; (2) minimizing the travel times of labor, material and equipment on site; (3) improving construction productivity; and (4) promoting construction safety and quality[1, 2].

A number of studies were conducted in order to improve site layout planning in construction projects. These studies adopted a wide range of methodologies and development tools including neural networks[3], knowledge-based systems[4, 5, 6, 7], heuristics[8, 9], simulation[10, 11], and genetic algorithms (GAs)[12, 13, 14, 15, 16, 17, 18, 19]. Despite the contributions and practical features of available site layout planning models, they all focused on providing a solution that seeks to optimize the single objective of reducing travel distances of resources. In many real-world projects, this is often considered inadequate as other objectives such as improving safety may prove to be equally if not more significant. Construction safety is one of the important but least considered objectives in site layout planning and design[2]. This is particularly true in the construction industry which suffers from more accidents of greater severity than other industrial sectors[20]. The National Institute for Occupational Safety and Health (NIOSH) ranks the construction industry as the first in causing non-fatal injuries at a rate of 9.3 injuries per 100 full-time workers in 1997[21]. Moreover, the Bureau of Labor Statistics (BLS) ranks the construction industry among the top three industries causing fatal injuries in the United States in 2002 with a rate of 12.2 fatalities per 100,000 workers[22]. The total cost of accidents is significant and has been reported to reach up to 15% of the total costs of new construction[23]. These figures and statistics highlight the pressing need for further research efforts to develop advanced and expanded site layout planning models that are capable of considering and improving safety while seeking to minimize the travel cost of resources on construction site.

The main objective of this paper is to introduce new and innovative measures that can be utilized to quantify and maximize safety during the design of construction site layouts. The paper presents (1) relevant safety considerations and measures that affect the planning of construction site layouts; (2) applicable cost considerations that influence resources operation and materials handling on site; and (3) an optimization model to maximize construction operations safety and simultaneously minimize travel cost of resources on site.

SAFETY CONSIDERATIONS IN SITE LAYOUT PLANNING

Safety of construction operations is usually affected by many factors which include the site layout design, safety planning, personnel practices, and level of personnel training, among others. This study focuses on site layout design aspects that affect the safety of construction operations. A comprehensive literature review[2, 18, 24, 25] and several field studies were conducted in order to explore and identify relevant and important practical considerations that can enhance the safety of construction operations. This investigation led to identifying the following two key measures: (1) proper positioning of temporary facilities to improve crane operations safety and minimize accidents caused by falling objects; and (2) control of hazardous material and equipment on site. The following sections provide a brief discussion of these two measures, and the newly developed performance criteria to quantify their impact on construction safety.

Safety of Crane Operations

Statistics indicate that cranes and falling objects are among the major causes of construction accidents[2, 21, 26, 27]. The Occupational Safety and Health Administration (OSHA) reports that an average of 71 fatalities occur each year in the United States due to crane accidents[28]. To minimize the risk of such crane accidents, OSHA requires contractors to provide protection and safety measures against falling objects, especially below steel erection operations[25, 29]. In order to comply with these safety requirements, construction planners often seek to locate site offices and high occupancy facilities outside the reach of crane operations whenever possible. To facilitate the implementation of this site layout planning measure and the quantification of its impact on construction safety, this study presents a newly developed performance metric named crane safety criterion (CSC). This new performance metric (CSC) is designed to enable planners to measure and quantify the degree of safety due to positioning facilities in the neighborhood of cranes as a function of (1) the sensitivity (V_i) of each temporary facility i to potential falling objects from cranes, which can be used to represent the potential risk of injuries and/or fatalities if such an incident occurs; and (2) the distance (d_{ik}) between facility i and crane k, as shown in Figure 1 and Eqs. (1) and (2). First, the sensitivity of temporary facilities (V_i) can be specified by construction planners by selecting from three categories of low, medium and high sensitivity. Second, the distances (d_{ik}) between temporary facilities and cranes on site are automatically calculated and considered by the model for each generated site layout plan.

The sensitivity (V_i) of each facility and its proximity (d_{ik}) are the main variables that influence the crane safety indicator (CS_{ik}) due to positioning facility i in the neighborhood of crane k, as indicated in Eq. (2). This indicator classifies the space around the crane into three zones, as shown Figure 1. Zone 1 is the area covered by the crane jib ($d_{ik} < J+M/2$), and it represents the highest risk zone due to its vulnerability to falling objects from the crane during its operations. Zone 2 is located between zones 1 and 3 ($J+M/2 \leq d_{ik} < R+J+M/2$), and it represents an intermediate level of risk due to its minor vulnerability to low probability crane accidents such as the tilting and/or collapse of the crane. Zone 3 lies outside the crane risk areas ($d_{ik} \geq R+J+M/2$), and therefore its safety is unaffected by crane operations (i.e. CS_{ik} = 100%). Based on the sensitivity of each facility (V_i) and its location (d_{ik}) within these three zones, each temporary facility i is assigned a crane safety indicator (CS_{ik}) due to its proximity to crane k, as shown in Eq. (2). The summation of all calculated indicators (CS_{ik}) for each combination of facility i and crane k is then averaged to identify the crane safety criterion (CSC) for the overall site layout plan, as shown in Eq. (1). It should be noted that the crane safety criterion (CSC) is formulated in a practical way that enables its measurement on a performance scale that ranges from 0 to 100%.

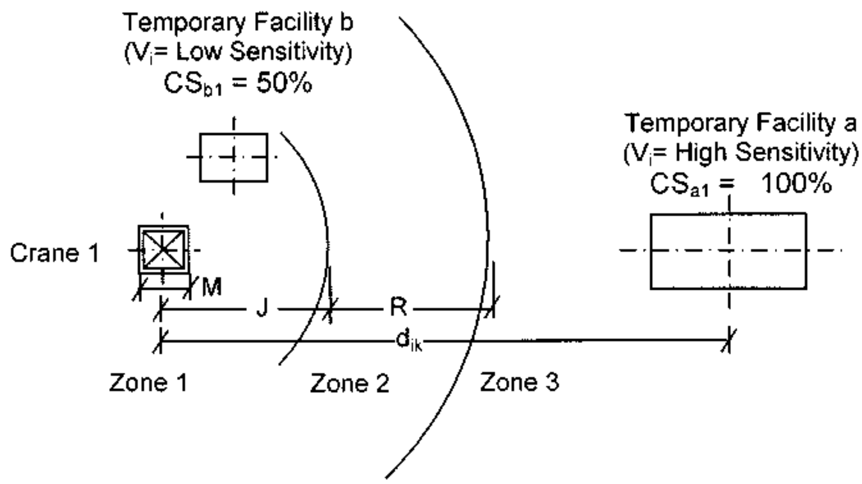

Figure 1. Evaluating Crane Safety Performance of Temporary Facilities

$$\text{Crane Safety Criterion (CSC)} = \frac{\sum_{k=1}^{K} \frac{\sum_{i=1}^{I}(CS_{ik})}{I}}{K} \qquad (1)$$

$$CS_{ik} = \begin{cases} CS^1_{ik} = \begin{cases} 0\% \ (V_i = \text{High}) \\ 25\% \ (V_i = \text{Medium}) \\ 50\% \ (V_i = \text{Low}) \end{cases} & (\text{zone 1: } d_{ik} < J+M/2) \\ (100 - CS^1_{ik})(R+J+M/2 - d_{ik})/R + CS^1_{ik} & (\text{zone 2: } J+M/2 \le d_{ik} < R+J+M/2) \\ 100\% \ (V_i = \text{High, Medium or Low}) & (\text{zone 3: } d_{ik} \ge R+J+M/2) \end{cases} \qquad (2)$$

where

CS_{ik} crane safety performance of temporary facility i due to its proximity to crane k,
CS^1_{ik} crane safety performance of temporary facility i due to its proximity to crane k in zone 1,
V_i sensitivity of facility i to falling objects,
d_{ik} distance between facility i and crane k,
I total number of facilities on site,
K total number of cranes on site,
J length of the crane jib,
M width of the crane mast, and
R reach of the crane.

For example, Figure 2 shows two site layout scenarios to illustrate the use of the crane safety criterion (CSC) metric. As shown in Table 1, the CSC of site layout 2 is higher than that of site layout 1, reflecting that site layout 2 is safer than site layout 1. To ensure the validity of the proposed model, the safety performance metrics were also tested collectively in larger case studies, taking into account cost considerations[30]. The results of all this analysis confirmed the validity and the practically of the presented concepts[30].

Table 1. Crane safety criterion for layouts 1 and 2

	Facility	Vi	CS^1_{ik}	Layout1				Layout1			
				d_{i1}	CS_{i1}	d_{i2}	CS_{i2}	d_{i1}	CS_{i1}	d_{i2}	CS_{i2}
	1	High	50	44.13	0.00	158.49	100.00	121.15	100	162.66	100
	2	Medium	25	89.07	81.08	118.19	100.00	146.92	100	171.93	100
	3	Low	0	103.35	27.17	99.69	39.37	129.78	100	130.21	100
	4	Medium	25	117.63	100.00	85.18	90.80	165.65	100	145.42	100
	5	Medium	25	149.95	100.00	50.61	25.00	161.88	100	118.67	100
ΣCS_{ik}					308.24		355.17		500		500
$\Sigma CS_{ik}/I$					61.65		71.03		100		100
$\Sigma(\Sigma CS_{ik}/I)$							132.68				200
Crane Safety Criterion (CSC)							66.34%				100.00%

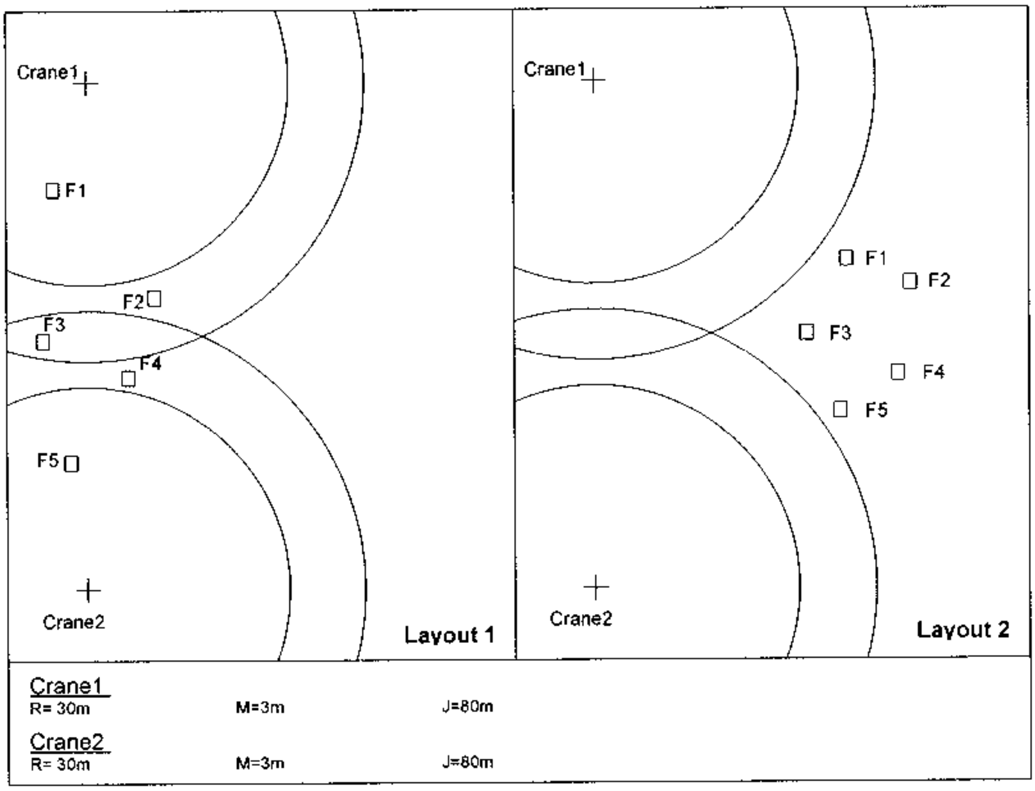

Figure 2. Evaluating Crane Safety Performance of Temporary Facilities

Control of Hazardous Material

Hazardous material and equipment are often utilized and located on construction sites, exposing construction workers and engineers to safety risks[29]. Hazardous material include (1) explosives and blasting devices used in rock excavation; (2) flammable material such as fuel used by construction equipment; (3) toxic substances such as asbestos, coal tar pitch volatiles, cadmium, benzene, formaldehyde, methyl chloride among other materials including 13 carcinogens identified by OSHA[25]; and (4) sources of harmful radiation and high electric voltage. These hazardous material and equipment need to be properly stored and adequately separated to minimize the risk of accidents on site. For example, OSHA standard 1926.407 recommends storage facilities of electrical equipment and possible sources of sparks be located far away from flammable material[25].

In order to improve safety on construction sites, planners need to comply with OSHA standards and identify proper storage locations for all hazardous material on site. These locations should be selected to ensure that there is adequate separation between (1) specific combinations of material and/or equipment that can create hazardous conditions on site (e.g., explosives and blasting devices); and (2) hazardous material and workers. In order to support planners in this vital site layout planning task, this study proposes using a new performance metric named hazards control criterion (HCC). This new performance metric is designed to enable planners to measure and quantify the degree of hazard control on site as a function of (1) the hazard control weight (HCW_{ij}) of facilities i and j which represents the degree of hazard that can be encountered on site if the two facilities are not adequately separated; and (2) the separation distance (d_{ij}) between facilities i and j, as shown in Eq. (3). The value of hazard control weight between facilities i and j is to be assigned in the range from 0% to 100% for combinations of facilities that create no hazards to those that pose the highest level of hazard if they are not separated on site.

$$\text{Hazards Control Criterion (HCC)} = \sum_{i=1}^{I-1} \sum_{j=i+1}^{I} \left(HCW_{ij} \times d_{i,j} \right) \tag{3}$$

$$d_{ij} = \sqrt{(X_i \quad X_j)^2 + (Y_i \quad Y_j)^2} \tag{4}$$

where

HCW_{ij}	hazard control weight that represents the risk of accidents that can be encountered on site if facilities i and j are not adequately separated,
d_{ij}	separation distance between facilities i and j,
X_i, Y_i	coordinates of center of gravity of facility I,
X_j, Y_j	coordinates of center of gravity of facility j, and
I	total number of facilities on site.

It should be noted that unlike the earlier described crane safety criterion (CSC), the present hazards control criterion (HCC) values do not necessarily fall within a performance range of 0% to 100%. In order to ensure consistency in performance measurement among all the safety criteria in the present model, each calculated HCC value in Eq. (3) is normalized to generate a normalized performance metric (NHCC) that ranges from 0% to 100%, as shown in Eq. (5). NHCC is calculated for each HCC value, using the maximum (HCC_{max}) and minimum (HCC_{min}) values of all generated site layouts.

$$\text{Normalized Hazards Control Criterion (NHCC)} = \frac{HCC - HCC_{min}}{HCC_{max} - HCC_{min}} \tag{5}$$

where

HCC	calculated hazards control criterion, using Eq. (3),
HCC_{min}	minimum HCC value obtained from all generated site layouts, and
HCC_{max}	maximum HCC value obtained from all generated site layouts.

Overall Construction Safety

The present model is designed to aggregate the above described two safety criteria into a single objective function to facilitate the evaluation of the overall safety performance for each possible site layout plan. To this end, the present model incorporates a weighted average formula that depicts the overall safety performance on site, as shown in Eq. (6). In this formula, the relative weight/significance (w_1 and w_2) of the two safety criteria can be specified by construction planners according to the special conditions of the project being considered. They can also be determined by analyzing historical records to compare the rate of accidents attributed to the inefficiency of enforcing these two safety measures. For example, the accidents data collected by BLS during the period from 1992 till 2001[22] can be used to assign a relative weight of 70% and 30% for the crane safety criterion and the hazard control criterion, respectively.

$$\text{Maximize Construction Safety} = \text{Maximize}\,[\,w_1 \times CSC + w_2 \times NHCC\,] \qquad (6)$$

where

w_1 relative weight or scaling constant of crane safety criterion; and

w_2 relative weight or scaling constant of hazards control criterion.

COST CONSIDERATIONS IN SITE LAYOUT PLANNING

The cost of construction operations is affected by the location of temporary facilities and the travel cost of resources (e.g. materials, equipment and labor) on site. In order to enable the search for and identification of the location of temporary facilities that minimize this type of construction costs, the present model utilizes an objective function that seeks to quantify and minimize the travel costs of resources on construction sites, as follows:

$$\text{Minimize Travel Cost of Resources} = \text{Minimize} \sum_{i=1}^{I-1} \sum_{j=i+1}^{I} (C_{i,j} \times d_{i,j}) \qquad (7)$$

$$d_{ij} = \sqrt{(X_i - X_j)^2 + (Y_i - Y_j)^2} \qquad (8)$$

where

$C_{i,j}$ travel cost rate in \$/meter of distance traveled between facilities i and j.

dij distance in meters between facilities i and j;

X_i, Y_i coordinates of center of gravity of facility i;

X_j, Y_j coordinates of center of gravity of facility j; and

I total number of facilities on site.

OPTIMIZATION MODEL

An optimization model is developed to support construction planners in identifying near optimal locations for all temporary facilities on construction sites such as storage areas of material and equipment, stockpiles of excavation, site offices, fabrication shops, and batch plants. The model is developed using a multi-objective genetic algorithm to enable the simultaneous maximization of construction safety on site and minimization of travel cost of resources[30]. As such, the considered decision variables in the present model are the coordinates (X_i,Y_i) of the center of gravity of each temporary facility (i = 1 to I). In the present model, these variables are represented by an artificial genetic chromosome that depicts the coordinates (X_i, Y_i) of each temporary facility (i) on site and they are among the main output data of the model. The designed chromosome enables the present model to evaluate the impact of various site layout solutions (S_1 to S_N) on safety and travel costs of resources. The planner needs to provide the dimensions of temporary facilities, while their near optimal locations are identified by the model, as shown in Figure 3. The optimization model starts its operations with initialization of random site layout solutions and then runs a series of evaluation, modification and combination of good solutions, attempting to reach optimal solution that maximize safety of construction operations and minimize travel cost of resources on site[30]. The model was tested on a number of application examples to validate its use. The results of the analysis highlight the new and unique capabilities of the developed site layout planning model and its usefulness in generating optimal tradeoffs between these two objectives[30].

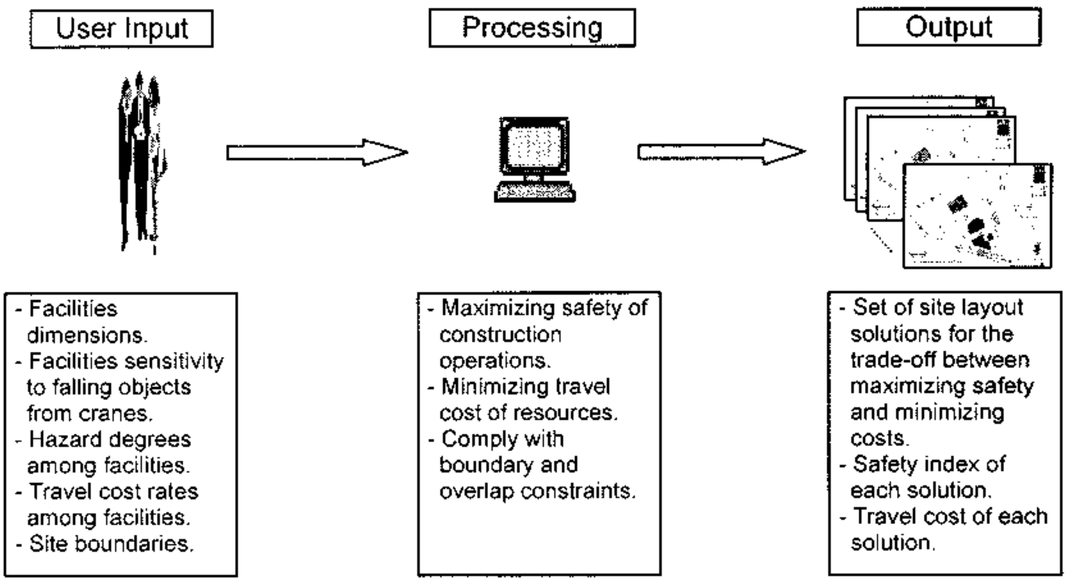

Figure 3. Data Input and Output of the Optimization Model

Furthermore, the developed optimization model is capable of considering all practical site layout constraints, including (1) boundary constraints; and (2) overlap constraints. The purpose of boundary constraints is to ensure that temporary facilities are located within the site boundaries, while overlap constraints are required to avoid the overlap of facilities on site[30]. This illustrates the practicality of the developed model and proves its usefulness in designing appropriate site layouts.

SUMMARY AND CONCLUSIONS

This paper discussed significant safety and cost considerations in site layout design. Two performance metrics were newly developed in order to (1) improve the safety of crane operations; and (2) enhance the control of hazardous material on site. The first metric is designed to search for safe locations of temporary facilities in an attempt to minimize the risk of crane accidents and falling objects. The second metric is designed to ensure that there is adequate separation between specific combinations of material, workers and/or equipment that can create hazardous conditions on site. These metrics were integrated with a third metric that quantifies the impact of various site layout plans on construction cost, using a multi-objective optimization model. The model can be used to search for and generate optimal arrangements of temporary facilities that provide optimal tradeoffs between safety of construction operations and travel cost of resources on site, while satisfying all practical constraints in this construction problem. This should prove useful to construction planners and can lead to significant improvements in the safety and cost of construction operations.

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SURVEYING AND MANAGING ACCESSIBILITY TO PUBLIC SPACE IN GREAT URBAN AREAS

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ABSTRACT: Pedestrian accessibility of urban public space in cities is a branch of the wider discussion about life quality and, more specifically, urban quality. This paper describes a system to support the management of urban public space. A specific field of application is assumed: it concerns the problems of accessibility of the public spaces by "weak" users and in particular by disabled persons. The system is a part of a wider project, promoted by Rome's public transport board, within the framework targeted to the enhancement of citizens and tourists mobility. Aims of the project are: to define a methodology to survey and to analyse the accessibility of urban pedestrian routes; to provide information on the accessibility of pedestrian routes for users with impaired mobility and/or sensory disabilities (user-oriented product); to produce an informative system with a geographic component, able to support the modification process (transformation - re-qualification) of the city public space. Analysis and evaluation of accessibility level of urban public space are based on GIS technology. In the paper, we firstly outline the assumed management framework; then a short description of survey methodology is given., including a synthetic specification of the gathered geographical data base; lastly we discuss methodologies and techniques to measure accessibility and to evaluate usability of public space, developed in GIS environment.
Project was tested on sidewalks within a distance of 150 meters from tramway in Rome city centre.

Keywords: accessibility, management of public space, GIS technology

OBJECTIVES AND BACKGROUND PROBLEMS

The aims pursued in the research on pedestrian public space, promoted by Rome's Transport Board, were quite broad; they included:

- The study of a comprehensive methodology finalised to the knowledge of the urban pedestrian public space, aiming to:
  - support the making of urban policies and intervention programs oriented to the improvement of pedestrian accessibility

- support the programming of interventions in defined urban areas
- support the making of projects for accessibility improvement
- distribute information to users on public space accessibility conditions

* The operational development of methodology, including:
  - the development of tools for geographical management of information on public pedestrian space;
  - the actual application of methodologies and tools on a wide urban area, in the city of Rome.

The setting of the overall methodology was focused on a range of relevant themes, that can be summarised as follows:

* The dimension of the problem

The present lack of urban quality depends on absence of planning actions specifically applied to pedestrian space and extended on the whole urban area. Thus, an improvement strategy in the urban environment has to comprehend extensive interventions on public spaces, while only efforts to ameliorate single sites in the cities have been performed so far. Instead, the methodology had to deal with i) medium-large sized cities, ii) the whole urban area (not only historical centre or residential or business districts ...), iii) a medium-long time for the application of improvement policies, iv) pedestrian spaces accessibility in bad repair (due to geometrical characteristics and/or current use).

* The process

The public pedestrian space is the site where many subjects act, simultaneously and/or in different times, each one having their own functional objectives to reach. Thus, the physical making of the public space is a typical multi-actor - multi-objective process, occurring over the time. If co-ordination among actors is not provided, then the product will be the present space, resulting in a casual pattern of objects, coming up from the independent (i.e. not mutually verified, but cumulative) actions of all the operating subjects. The methodology has to provide a common basis for the needed co-ordination, that is for the compatibility evaluation both of new interventions, as of the current condition.

* The "rules"

The few available design "rules" for pedestrian public space seem not to be the proper ones, not even to ensure access to disabled people. Nor it is reasonable to draw rigid and/or complicated new rules to solve the problem. Thus, as a minimal characteristic for quality, the "usability" of public space has been assumed; and afterwards the "accessibility" is defined as a representative performance indicator. Such measurement can be built in a quite simple way, when having the suitable information on the state of public space and on the request for new utilisation.

THE MANAGEMENT REFERENCE PROCESS

The aim of a very extensive improvement of the pedestrian urban space implies necessarily a long time and, consequently, a "stepped" and spatially distributed strategy. Firstly, the stepping of strategy is discussed, differentiating what has to happen for the re-establishment of the minimum level of "usability" (pedestrian accessibility), from what has to be managed in the following "routine" phases. The strategy will then be referred to the problems arising from the articulated geography of urban areas, mainly with the purpose of setting tools aiding the decision on spatial application of policies.

Let now sketch the actions to carry out at the beginning of the process of improvement; we will refer to generic urban area, for which it is already assumed the decision to intervene.
As a starting point, the current condition of public space is considered; it is the result of past actions performed by many actors, without any co-ordination or explicit and specific planning. In Figure 1. it is the period (t_0). The actions in this period are those directed to the re-establishment of the minimum level of "usability", after which a "routine" management should begin, in case with further improvement actions (period t_1). The foreseeable actions in the two different periods are like, with some meaningful difference.
The operative actions in the period t_0 should be:

- Survey; usually it is necessary for the lack of a detailed and updated cartography at a proper scale. We must underline that survey technicalities have to ensure an information output both efficient (remember the extent we aim to cover) and effective (as a good basis for the subsequent operations)

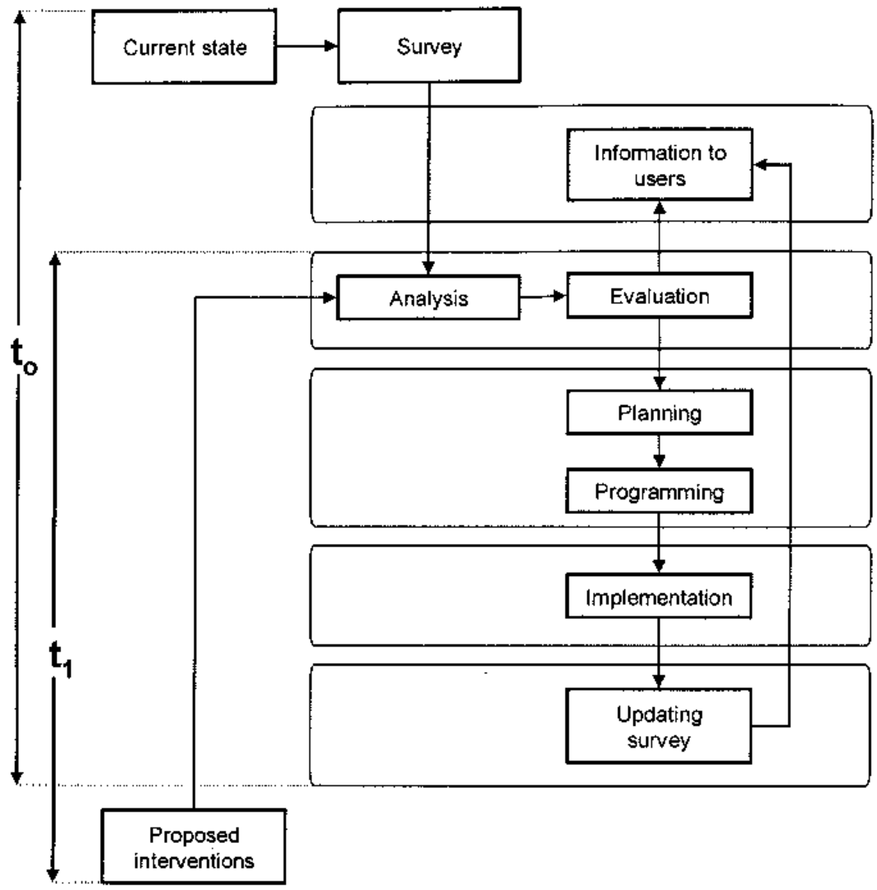

Figure 1 - Activities framework

- Analysis; a classification has to be done, in which each elementary module of urban pedestrian space (the front of buildings) will be marked depending by its complexity, both physical-technological and managerial (number of subjects using the space). This classification will show the technical difficulty to intervene on the space modules.

- Evaluation; the theme being the accessibility, firstly a measurement of that has to be taken (with a geometrical and topological analysis); then an assessment of the actual condition will be performable, considering all kind of obstacles in the existent path. This will produce a new classification of pedestrian space modules, that will show the user's difficulty.

From the evaluation block the actions flow towards two different branches; the first one leads to an activity of information transfer to users.
Along the second flow branching out from the block we find the actions directed to the functional improvement; they are:

- Planning; we assume the planning activity as merely including the only functional contents, with no regard for aesthetics; that is to take into account exclusively the accessibility problem. The plan will provide only interventions needed to reach the threshold of the minimum accessibility.;
- Programming; the given plan has now to be implemented in a given "action area": that area is a "living environment", whose life must be troubled as less as possible. Thus, the programming activity has to define a good sequence in time and space for the interventions foreseen;
- Implementation; the physical realisation of the interventions will be driven by the output of the programming activity. A reasonable time-space works sequence has to take into account both the impact on the urban life and the technical and economic logic of the contractors;
- Survey updating; this activity can be performed on the basis of plans, but not forgetting that many changes may occur during the implementation. Objects displaced, removed, changed or newly inserted must be updated on the whole layout;
- Information to users; a new version must be generated, updating the previous one.

After the first period, which is dedicated to the rehabilitation of pedestrian space, it has to be expected that new needs of change will arise. So the problem in the subsequent periods is to control the sustainability of transformation proposals coming up from subjects using or willing to use the space for their objectives.
The operative actions are quite similar to the previous ones, but in this case they are applied to the proposal of changing the public space utilization, coming from both public and private subjects.
Also at this moment we need to analyze and evaluate the compatibility or the interference among such proposal and the existing objects. A new connotation we have to associate to the planning, programming and implementing activities, due the needing of coordination between the subjects acting in space modification. However, after the implementation, we need to update the current configuration of public space, and to regenerate the information for the pedestrian users.

Levels

As we told, the outlined process is referred to a single generic urban area. Though a "low profile" goal is been assumed for the improvement of public space (The Minimum Level Of "Usability"), it is obvious that no authority in an only middle-sized city could perform simultaneously those activities (period t_0) over the whole urban area (and we have to recall that our reference extension IS the whole urban area). Thus, a permanent policy, to stand firm over a medium-long time, and its spatial break down are needed.

From the methodological and operational points of view, the problem then is to individuate an activity flow and the proper tools to support knowledge and decision about the urban areas where (and when) to apply the improvement process. The methodological scheme we propose is quite traditional, including three levels:

- Whole Urban Area extent
- Single Urban Area extent (a part of the Whole urban area)
- Urban Action Area extent (a part of a Single Urban Area)

To the " Single Urban Area" and to the " Urban Action Area " are applied the activities described in Figure 1. For the "Whole Urban Area" extent we must now define the activities satisfying the support needed to breakdown the overall policy into an operative strategy.

Those activities are oriented to gather and manipulate information on: i) the pedestrian space condition, and ii) the user's demand. A hierarchy among Urban Areas should be done comparing the space condition (good - bad) and the user's demand (high - low), with regard to i) an absolute lower bound in space condition, and ii) the intensity - and the specificity too - of user's demand.

Techniques useful to quickly measure the current "load" on public space (i.e. space occupied by objects) have been successfully tested, while the user's demand has been mapped starting from highly frequented places and demographic users profiles. Furthermore, a reasonable draw of public space improvement strategies over the city has to consider other concurrent objectives, normally expressed by authorities.

Levels And Activities

The description of the methodological framework being completed, is now useful to observe synoptically all the items intervening in the process flow, that is by now distributed by levels. Table 1 shows:

- Scalar levels (strategic, planning, programming);
- Objectives (knowledge and decision);
- Activities/tools (methodologies, techniques and tools for knowledge and decision).

Table 1. Levels, objectives, activities/tools

LEVELS	OBJECTIVES	ACTIVITIES/TOOLS
WHOLE URBAN AREA	◆ Synthetic knowledge of: i) public space condition ii) social-functional relevance of single urban areas	◆ Quick accessibility survey ◆ Multivariate geographical analysis
	◈ Determination of "Single Urban Areas" and definition of time priorities	◈ Multicriteria geographical analysis
SINGLE URBAN AREA	◆ Analytical knowledge of public space (physical state, running urban functions)	◆ **Analytical public space survey**
	◆ Analytical detection of local criticalities (workload, accessibility, overcrowding)	◆ **Spatial analysis**
	◈ Determination of "Urban Action Areas"	◈ **Multicriteria geographical analysis** ◈

URBAN ACTION AREA	◈ Definition of physical interventions	♦ Geographical-statistical analysis ◈ **Geographical location of local criticalities to remove**
	◈ Definition of implementing modalities (space, time, operating subjects)	◈ **Time-space analysis of temporal compatibility**

♦ Knowledge objectives, activities and tools
◈ Decision objectives, activities and tools

Only items in bold will be further discussed in this paper. That is because of the relevant usefulness of the tools there developed for a quick public space assessment and minimal improvement actions.

ANALYTICAL PUBLIC SPACE SURVEY

Conceptual problems

Conceptual problems dealing with the analysis are:

A. How to formalise the accessibility concept, to make it measurable

For this purpose, we take into account the different categories of users of the public space: from the healthy user to the disabled persons. We assume that accessibility corresponds with the availability of clear public space, along with users can walk. We fixed this measure in 90 x 90 cm, that is the smallest corridor that a disabled person, using a wheelchair, needs to move lengthwise. We also consider another measure (150 x 150 cm), that is the minimum area needed by the same user to make a rotation and to change direction.

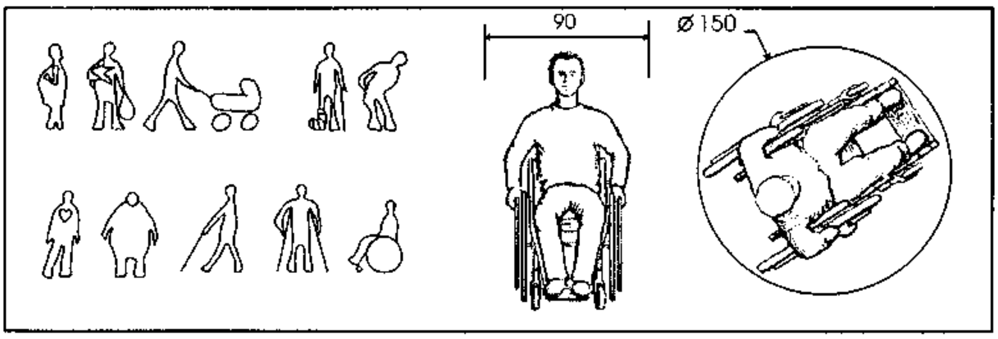

Figure 2. Accessibility measures

B. What kind of linear elements are to be used to represent the accessibility conditions, which are measured and evaluated on the basis of an areal space

We choose to use a network, which components are: crossing lines and front lines (links), connecting nodes. These components, in all, schematise the pedestrian network. The network is built up in GIS environment, with the support of an automated tool developed in ARC/INFO Macro language (AML)

C. In which way to split up the areal space (public space), to carry out measurements

Referring to the previous assumption, surroundings of each links are the basic units to be analysed. A specific tool, developed in AML, provides to select each link of the network, to built up a buffer around it (which width is referred to the sidewalk dimension), to split up the surveyed space enclosed into the buffer.

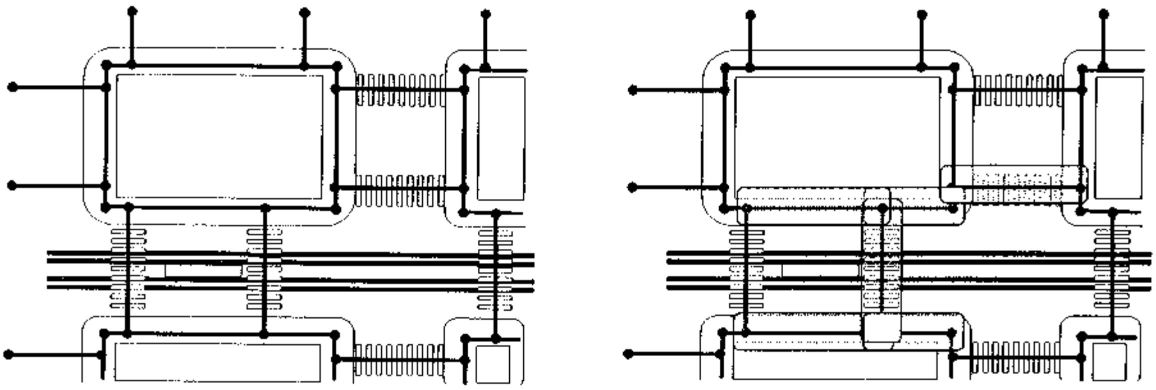

Figure 3. Pedestrian network and Basic Units of analysis

D. which tools and techniques are to be used to measure accessibility conditions
We decide to exploit GIS' resources and particularly the GRID environment combined with specific tools developed in VB environment, to identify accessible corridors and to evaluate the public space usability.

THE SURVEY AND THE GEOGRAPHICAL DATA BASE

Project constraints

(A) Level of precision and detail for a DB supporting accessibility information providing
While an architectonic barrier is quantitatively measurable, it cannot, however, be regarded as purely objective because its importance varies in relation to the physical condition of the person who meets it.
For this, our first goal was to provide the user with the possibility of putting in relationship her/his subjective physical capacity with the real qualitative and quantitative characteristics of the architectonic barriers along a route.
If the user is to enjoy this freedom of choice, the methodology adopted in the survey must allow for recognition of all the items identifiable as architectonic barriers as well as the description of the quantitative and qualitative characteristics of such items.
(B): Level of precision and detail for a DB supporting urban planning
In the case of the database oriented towards planning, the only useful tool for the technician, called upon to plan the elimination of architectonic barriers in an urban context, is an architectonic survey comprising both the location and the qualitative and quantitative characteristics of all the items present along a route.
As a result, it was decided that the set of items to be surveyed should include the universe of objects encountered along the pedestrian route and in its vicinity, and that it should be possible to represent such objects in proportion to their actual size in a format directly usable by the planner.
(C) Easy survey method to apply
Due the extension to survey, the activities mustn't involve the large amount of time and professional skills required to carry out a topographic survey. Moreover, we must

consider that survey teams were made up of two technicians, one of whom is a "user-operator" (i.e. a person with restricted mobility making use of aids like wheelchair, crutches, etc.).

(D) Reproducibility in other contexts

This constraint implies that basic informative elements, technological instruments and tools have to be widely available and not produced ad hoc.

Methodology definition

A methodology was developed of surveying and representation possessing the characteristics and degree of precision of an authentic architectonic survey, but at the same time proving easy and quick to apply .

The mechanism identified is based on:

a) the classification of all the items to be surveyed by means of an alphanumerical code;
b) the spatial location of each individual item by means of two coordinates referred to an element that can be regarded as spatially and quantitatively constant over time (i.e. the point where buildings meet the ground as shown in the relevant cadastral maps).

Having codified all the objects, the survey personnel go on to complete a table bearing the code of the object encountered together with its distance from the "foot" of the relevant building (1st coordinate), its longitudinal distance from the point at which the building commences (2nd coordinate) and, eventually, its rotation angle.

To build up the base elements for the survey, 110 cadastral maps have been elaborated, to reduce the amount of thematic information (streets, parcels, buildings etc) included in this kind of cartography. According to the survey needs, only the information about building's boundaries (that are the base element to place surveyed information), have been extracted.

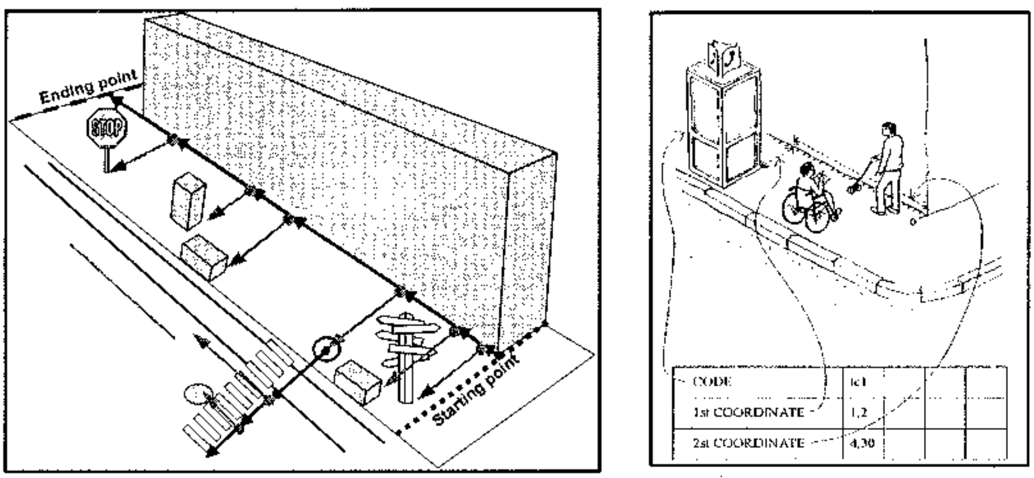

Figure 4. (a) Object's spatial location (b) The survey

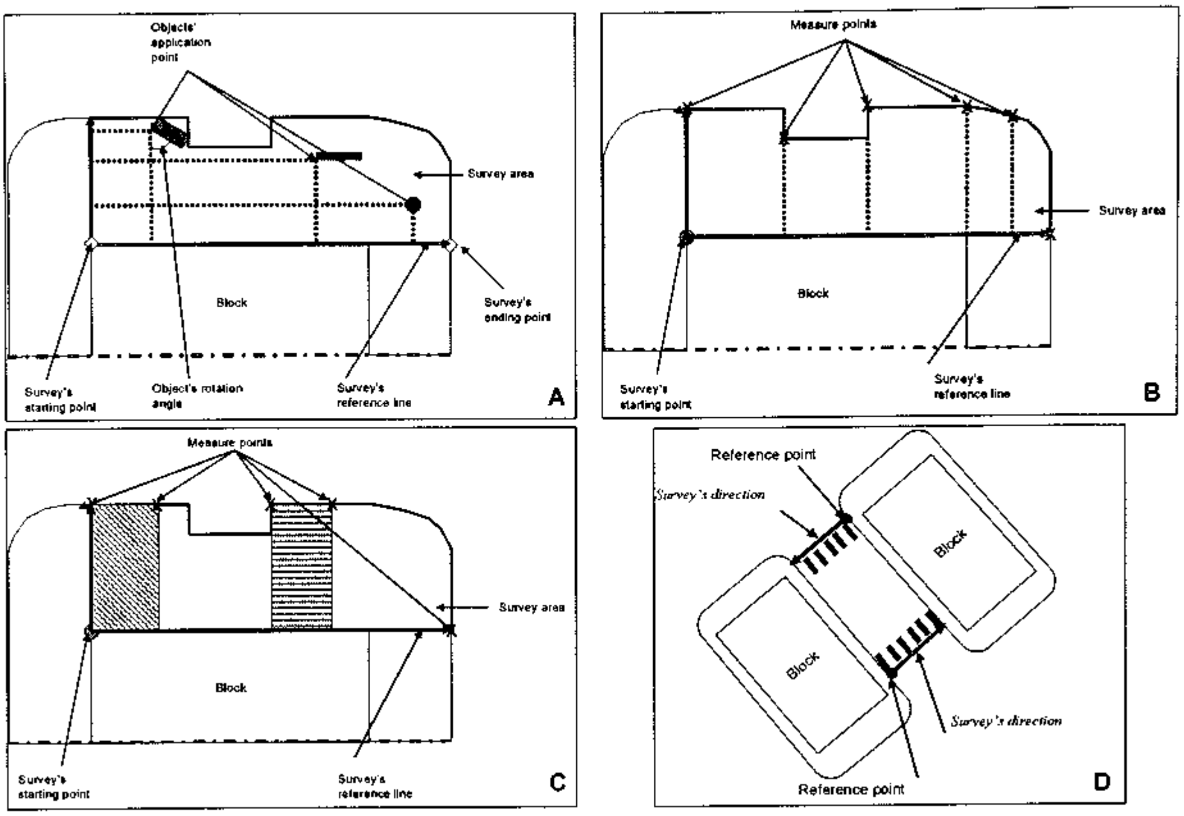

Figure 5. Survey's rules

In the Figure 5, the rules to make survey are shown, concerning: A) Objects, B) Sidewalk profile, C) Paving materials and maintenance condition, D) Pedestrian crossing

Building up the geo database

The phase of on-site surveying is followed by representation in CAD format. By means of an application developed by "Tor Vergata" University, by using a CAD block's table (Figure 6 A) and the building's front line (Figure 6. B), the alphanumerical codes present in the table (Figure 6. C) are transformed into graphic representations located in space in accordance with the real metrical references collected.
During the operations of surveying and representation, information is also gathered on the state of maintenance of the paving and the real measurements of the pedestrian route. Produced CAD file includes: pavement's outline, front, objects, and information about paving and maintenance (Figure 6. D).
A couple of codes (typological and numerical) are written for each one of these elements; codes are stored like labels near the linear and punctual elements or inside polygonal objects. DWG files are then converted in DXF format.
These files are then imported in GIS environment, with the support of a fully automated tool developed in ARC/INFO Macro language (Figure 6.E).
The numbers of the survey campaign, achieved by two survey's teams in six months of activities are: 102 blocks, 340 crossings, 40 km of survey area, 150.000 mq of pavements, and 14.478 objects.

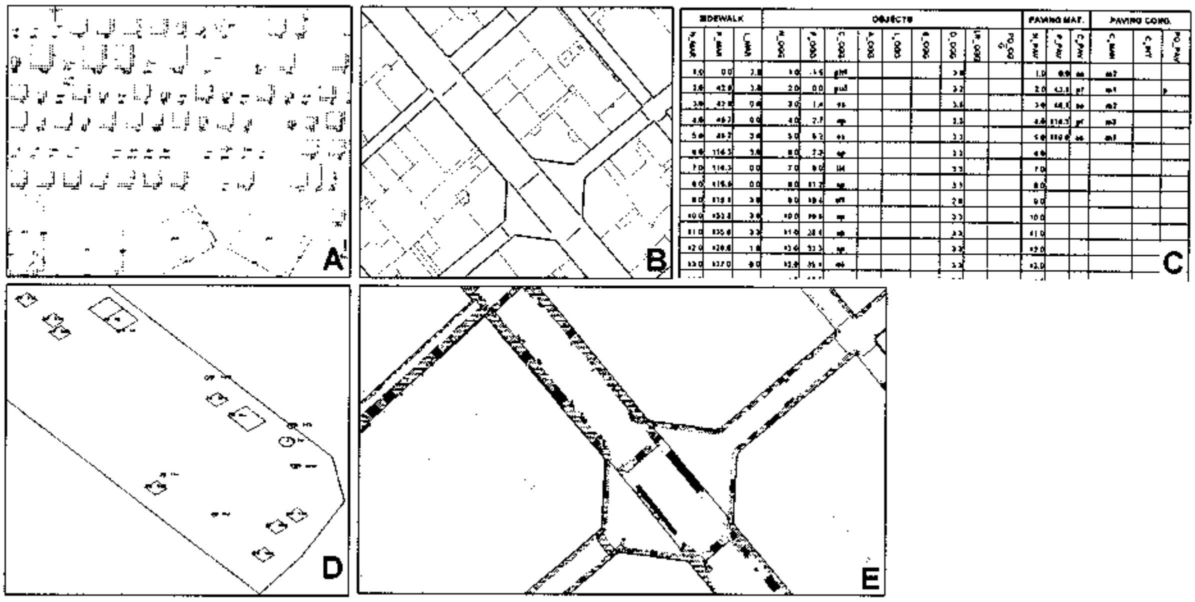

Figure 6. Building up the geo database

SPATIAL ANALYSIS TO FIND PEDESTRIAN CORRIDORS

The analysis is carried out for each one of the basic units described above (Figure 3.). The entire analysis is performed using a collection of tools, partly developed in GIS environment and partly in VB environment, as shown below.

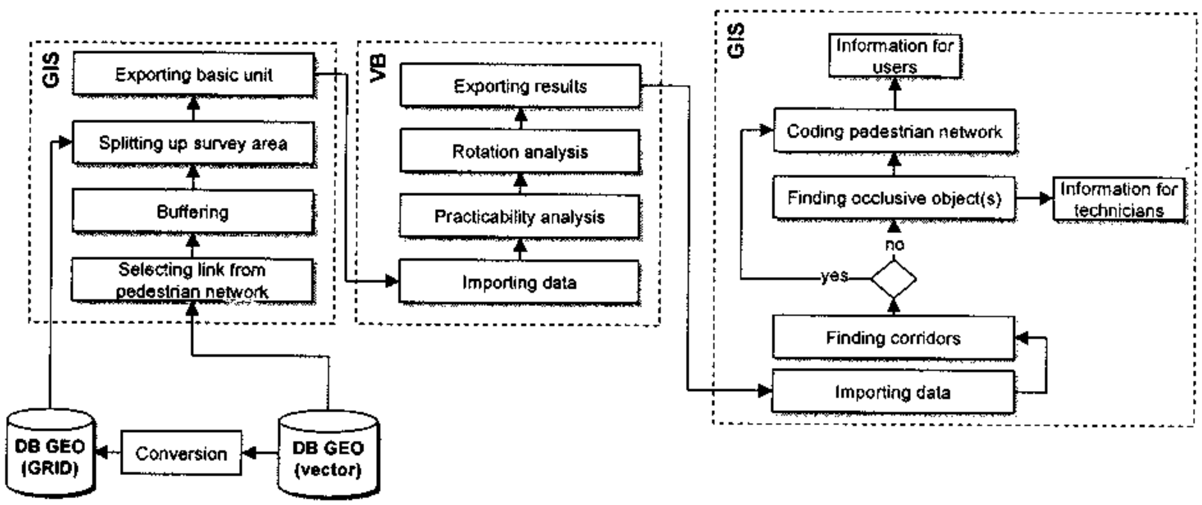

Figure 7. Elaboration flow

In GIS environment, the survey data base is converted from vectorial to grid format. Pixels' dimension is 10 x 10 cm; each pixel is coded according to the following contents: a) sidewalk area, b) occlusive objects, c) ramps (if ramps are accessible, they are coded as sidewalk area; otherwise they area coded as occlusive objects), d) carriage entrances (if entrances have a step, they are coded as occlusive objects). Each basic unit is exported in ASCII format and then imported into a specific tool, developed in VB environment. This tool analyses the surface, using a couple of moving window (simulating a wheelchair moving lengthwise or making a rotation) which dimensions are, respectively, 9 x 9 pixels and 15 x 15 pixels.
The output consists in a new grid, coded with 3 different values which meaning is: accessible area (this code is assigned to each pixel upon which the centre of wheelchair can stay), usable area (each pixel upon which wheelchair can pass, but its centre), occlusive area (Figure 8.).

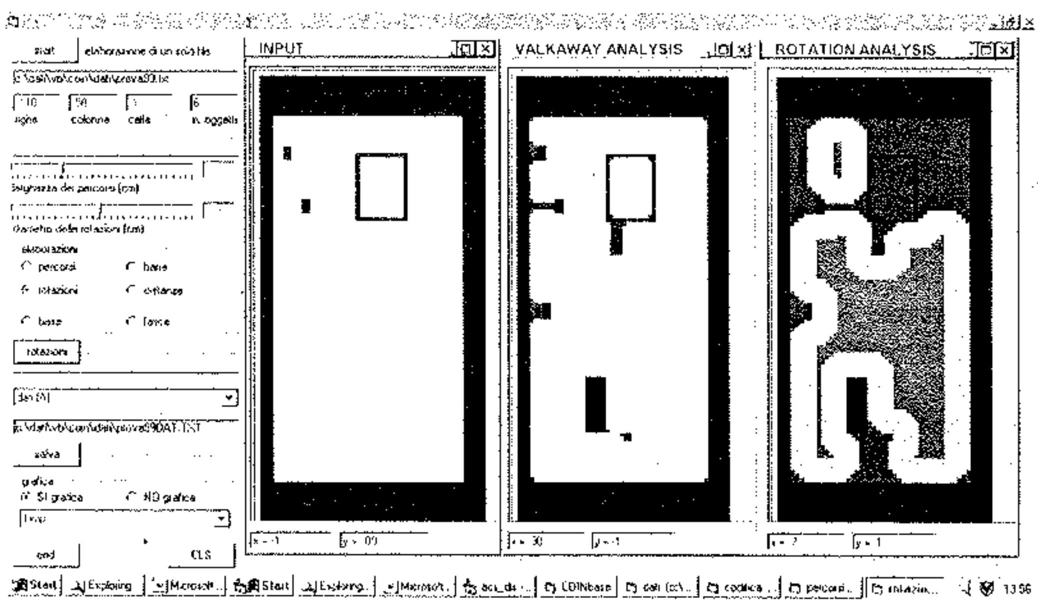

Figure 8. Moving window analysis

The classified grid is then imported in GIS environment and analysed to find the existence of a clear corridor.
For this purpose a "costdistance" surface is built up and then a "costpath" analysis is applied, to find the shortest way connecting link's nodes. If the corridor doesn't exist, due the presence of occlusive elements, a second step of investigation is performed, to identify the specific object(s) to be removed.

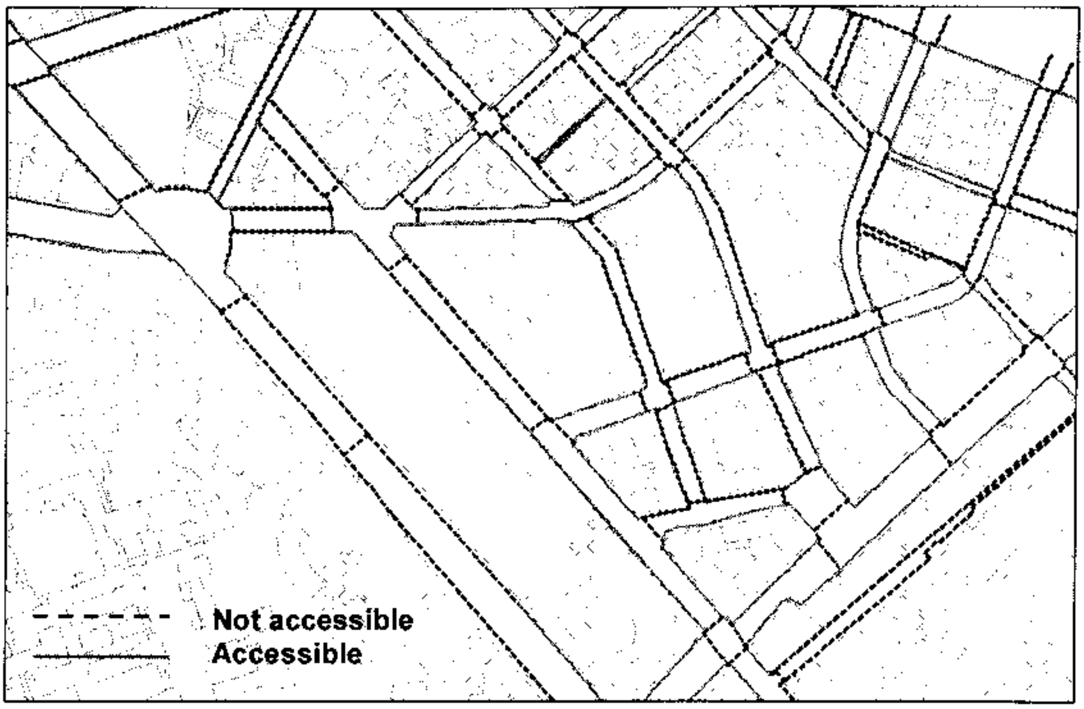

Figure 9. Accessibility of pedestrian network

The output of such analysis is then joined to the specific link of the pedestrian network. The whole process is iterated for each one of the analysed basic unit, corresponding to each link of the pedestrian network. In such way, all the links are qualified in terms of existence / absence of accessible corridors, both for walkway and for rotate, that is the information needed by pedestrian users (Figure 9.).

DEFINING INTERVENTIONS AREAS

To identify the interventions areas, a geographical analysis is performed, based on the survey output. The aim of the analysis is the qualification of the public space supporting the formulation of intervention policies to remove the inaccessibility conditions.
To this purpose, we perform three steps of analysis.

A. Typological analysis and classification of the objects, referred to their i) geometric dimension, ii) technological complexity, iii) building complexity
B. Analysis of dimensional ratio between public space and objects (for each areal unit of analysis)
C. Synthetic qualification and classification of public space units, by combining the previous outputs. The output of this step is the formulation, from the technical point of view, of the intervention policies typologies needed to remove the inaccessibility condition in each unit of analysis. These policies are grouped in three classes, low, medium and heavy, related to the complexity (in time, technological condition and costs) of interventions.

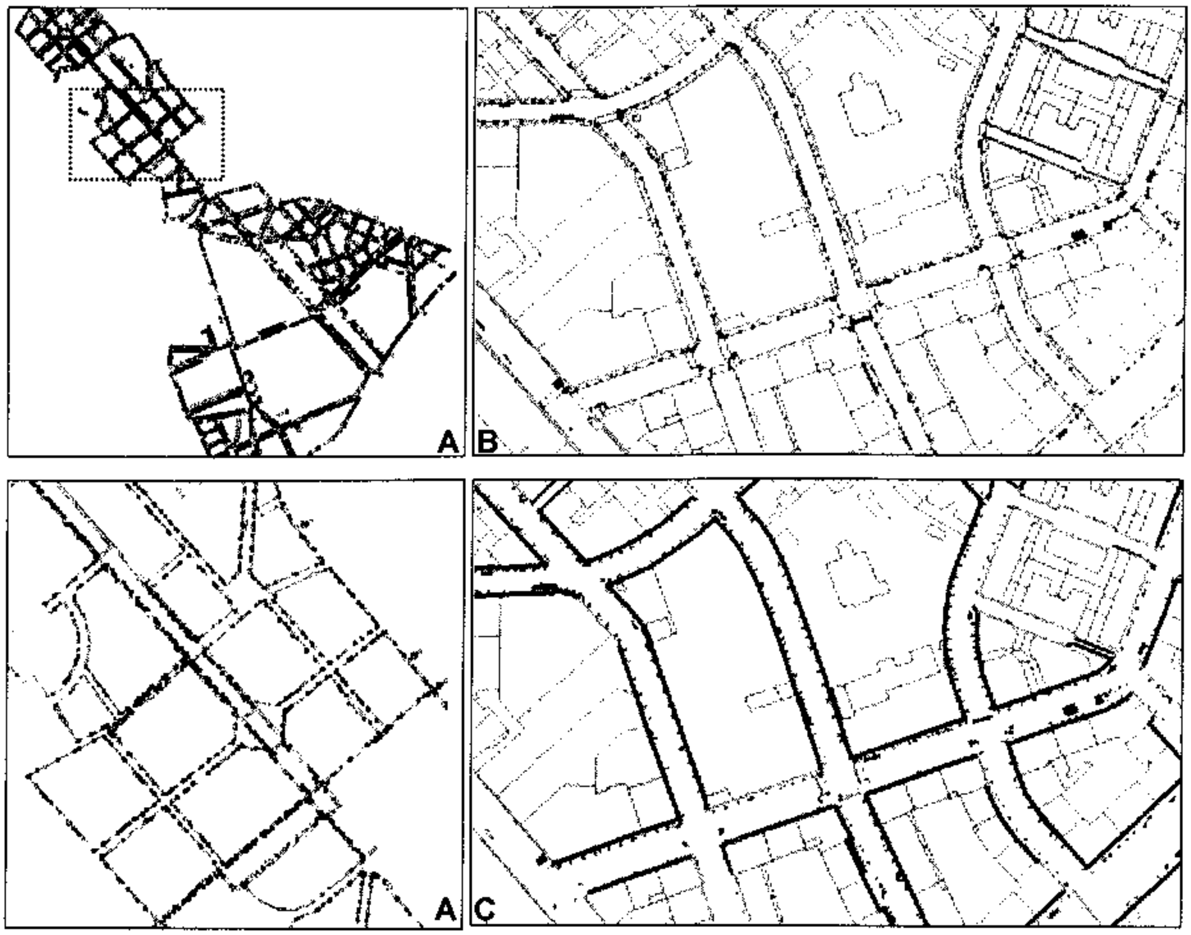

Figure 10. Defining policies

Finally, the intervention areas are identified by combing three elements: i) the accessibility analysis output, ii) the object qualification, iii) the policies typologies. To do this, we implemented a multicriterial analysis, by which all the elements contributing to the area definition are weighted under different hypothesis, mainly related to the policy options. We produced several scenarios, one of which is shown in Figure 11.

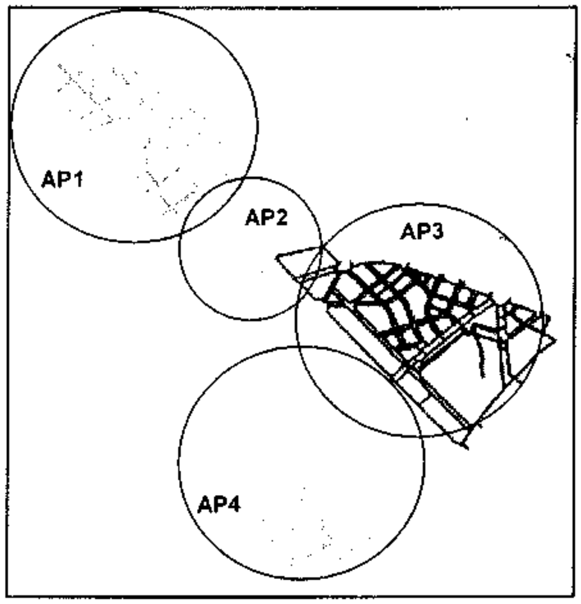

Figure 11. The intervention areas

CONCLUSIONS

The whole system of activities, instruments and techniques, described in the paper, is experimented, with different results:

- methodology: it seems to us that this kind of methodology assures all the expected performance: effectiveness (in time and cost), easiness and reproducibility, even if it requires a great coordination activity among all the involved working groups.
- instruments developed to support strategies outlining, directly referred to decision makers: this kind of instrument supply quantitative information, but their are used to support and drive a decision that is essentially political
- instruments developed to measure the accessibility demand, developed in GIS environment: this kind of instrument are certainly mature, and their application and results are broadly tested and verified
- integrating technical instruments and tools: to achieve the described results we used a wide range of heterogeneous technical instruments (CAD, GIS; Spreadsheet, VB) and this is the most relevant question to manage in implementing the methodology. In fact, also at this time, commercial instruments don't assure a good mutual compatibility; as a consequence, we must ensure a fully normalization of each step of the activity.

ACKNOWLEDGEMENT

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INFORMALISATION OF A PLANNED NEIGHBOURHOOD IN NAIROBI.

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ABSTRACT: Nairobi was established more than 100 years ago as a transit point for the Uganda Railway. This railway was developed by the British Colonial Administration, to link Mombasa on the Indian Ocean coast with Lake Victoria in the interior of the East African colonies. Over the years, Nairobi has grown into a major communications hub for the East and Central African region. It is home for approximately 3 million people and the political and economic capital of Kenya. Nairobi's annual population growth rate of about 4.5% is fairly high. If this growth rate is maintained, the city's population will double in almost 20 years time. This growth rate by itself puts pressure on the type of legislative and development control measures the city authorities adapt. One peculiar characteristic of many post-colonial cities, is that their production of urban space is increasingly being informalised. This is partly because the post-colonial administration opted for continuity rather than change the colonial development control structures. On the attainment of independence, the restriction on the movement of Africans was relaxed, creating a large influx of people from the rural areas into the city. The development control measures which were able to function in a regime based on restrictions were unable to cope with the new changed socio-economic/political circumstances. This paper examines the Buru Buru middle income neighbourhood, designed and executed from the early 1970s to early 1980s. Over the last 20 years or so, this neighbourhood has been informalised. People are deliberately not complying with the laid down statutes, by manipulating space within the limits of their property boundaries. The paper argues that the legislative and development control measures for Buru Buru are not sustainable, and that the city authorities need to dialogue with the property owners and arrive at an acceptable and workable development control system.

Keywords: Post-colonial, Informal, Urban Majority, Urbanisation, Transformation, Segregation.

THE URBANISATION PROCESS IN NAIROBI

The Pre-colonial Colonial Period

During the pre-mercantile period, the caravan route which linked the East African interior to the Indian Ocean passed on the eastern side of Nairobi through the present day Kariokor (a place where carrier corps were recruited). At this time there was a thriving trade in ivory tusks, bee-wax, hides and skins etc., before the advent of the East African slave trade (Amin, S. 1976). Before the arrival of the railway line to the present site of Nairobi, there was no major concentration of Africans. However the sire was used as a trading point among the Kikuyu and the Masai, who also used the area for grazing their cattle. In fact the name Nairobi is derived from the 'Enkare Nyaribe' (meaning a place of cold water) in the Masai language (Zwanenburg and King, 1975).

The Colonial Period

In 1895 a decision by the British government to declare the East African Protectorate, led to the building of the Uganda railway starting in 1896 (Hill, 1949). The laying of the railway from the Indian Ocean at Mombasa to Lake Victoria in the hinterland was to facilitate a communications corridor for exploiting the resources of the East African region. The labour for constructing the railway was imported from the British crown colony of India, as the African peasants and nomads were not available as labour. Concurrently the British, introduced wage labour through a monetary system, which over time replaced the local barter economy. The money economy led to the introduction of hut tax, income tax, poll tax, and at the same time, the blocking of other means of income apart from that earned as a wage labourer for a settler (Emig and Ismail, 1980). The establishment of a physical network – a railway and the beginnings of urban structures, must be understood as a part of a comprehensive project master minded and brought into effect by the ruling classes of Great Britain in order to gain political control in East Africa

Before the railhead actually reached Nairobi in 1899, there was a need in 1898, for a plan for a railway town. The choice of the site of Nairobi for this town, was based on its topographical surroundings, spacious flat terrain suitable for laying out of shunting areas, depots, workshops for European staff, commerce etc. One other major reason, rarely raised by many scholars, was the proximity of the site to the densely populated area around Kikuyu, as this was going to be a major source of labour for the future town. The Railway Town Plan's point of departure was the railway line, and the railway employees and thus the name, Plan for a Railway Town.

The plan only took into consideration the European employees of the railway and the European and Asian traders, who were expected to come just as they had done at other stations between Mombasa and Nairobi. The plan completely neglected the Asian labourers or coolies and the Africans. Nairobi was going to be a railway town for Europeans with a mixed European and Asian trading post. The town's functions were directly expressed by notions of segregation, including segregation by class and race. The population and size of the town grew dramatically during the colonial period between 1900 and 1963. The land area of the town was approximately 18km² in 1900, 25km² in 1920 and 83km² in 1927. Further expansion at independence in 1963 brought the total land area to the current 690km² (Morgan, (Ed) 1970; Amis, 1983; Obudho and Aduwo, 1992). Similarly the population grew from 11,512 people in 1906 to 118,976 people in 1948 and rising to 266,795 people in 1962, just before independence (Obudho and Aduwo, 1992).

Because of the above demographic and physical changes in the town, the colonial administration made another two plans for the town. The two were; the 1927 Plan for a Settler Capital, and the 1948 Master Plan for a Colonial Capital. The main contents of the 1927 plan was extensive traffic regularization to match the increased land area, drainage and swamp clearance, building and density regulation and attempts to furnish Nairobi with a monumental administrative centre. The most remarkable aspect of this plan was the inflated land prices in the Asian and African residential areas. The plan was also based on racial and class segregation, and was in complete agreement with the interests of the settler class (Emig and Ismail, 1980).

The 1948 Master Plan for a Colonial Capital was also based on segregation by race and class. The main aim of the Master Plan was to make Nairobi attractive for industrial investments. Nairobi was not only to be the capital of Kenya, but of East Africa. "As the capital of East Africa it is the natural centre for tertiary and quaternary industries. Almost all the head offices serving business, government, communication systems, are located here and as more and more services are unified for whole East Africa region, Nairobi will increase in value" (Thornton White, L. W. et al. 1948:40). The main spatial structure of the plan was to establish neighbourhood units for the working classes, which in a way was segregation for purposes of surveillance and dominance (Mitchell, 1991).

The 1927 and 1948 plans were however never fully realised, as the amount of capital outlay that was required for their implementation was never allocated. This in turn marginalised the African urban majority, and propagated informal urbanisation on the town's periphery (Thornton White, L. W. et al. 1948:43). At independence in 1963, the new ruling elite chose to adapt continuity rather than change in the legal and governance structures. The adapted dual structures of Provincial Administration (previously 'native authorities') and Local Government Administration (previously 'townships') quite often had overlapping roles. This dualism allowed the ruling elite to manipulate the system for their own individual gain at the expense of the populace (Home, 1983).

The Post-colonial Period

Following the colonial trend the new African city government, proposed a plan for a metropolis in 1973, named the Metropolitan Growth Strategy (MGS).This MGS was worked out by the Nairobi Urban Study Group (NUSG), and was funded by the Nairobi City Council, the Kenya Government, the World Bank and the United Nations. In general the MSG was a tool for state intervention and it supported the interests of the hegemonic class alliance of the local bourgeoisie and the Multi National Corporations (MNCs). The interests of the urban majority were neglected, in a similar way to the three preceding colonial plans. Segregation was used based on economic and class lines as opposed to racial and class lines.

The1948 Master Plan used the formal concept of functionalism; similarly the MGS used the formal concept of decentralisation a principle, of functionalism in articulating its aims. The strategy of the NUSG was to apply a technical approach to planning, by relying heavily on statistics (with 'reasonable' social explanations). These statistics remove the physical approach to urban planning and gives it instead a mathematical and scholarly approach (Emig and Ismail, 1980: 81). The MGS was published in two volumes. The first volume apart from discussing the aims and reasons for the preparation of the MGS dealt with "Demographic and Social issues", "Potential for physical Growth", "The recommended strategy" and "Implementation". The purpose of this strategic plan was not to present the various sectoral parts in detail, but to fit them into an overall structure over time. The second volume contained "Background data and Procedures". The appendices of this volume have the following headlines: "Population", "The economic outlook", "The use of land", "Transportation"; Housing Policy and Programming" and Nairobi City Council's Revenue and Expenditure 1964-1985"

The appendices give various scenarios and strategies for the various parameters (variables), for the urban growth of Nairobi up to the year 2000. For example appendix one dealt with population statistics and projections. It stipulated an aggregate population of Nairobi of between 3 million and 4.2 million people by the year 2000, however the reality shows a population slightly less than 3 million people. The MGS was never realised as it was meant to be financed by private capital which could not be guaranteed or co-ordinated. In any case the MGS was basically a political document which was using a spatial intervention as a means of political control. In the process the urban majority were marginalised further and informalisation thrived since the late 1970s to date.

There has been no major attempt at laying down in concrete terms the strategies for Nairobi's future urbanisation, since the MGS of 1973. The 1993 Nairobi City Convention on Actions towards a Better Nairobi, is the nearest the city authorities came close to addressing the urban question of Nairobi. In the mean time urban development has been ad-hoc and clientelistic, creating room for corrupt practices. The mushrooming informal settlements currently house more than 60% of the city's residents, and the high poverty levels continue to impact negatively on the urban majority. With the current population of approximately 3 million people, the informal settlements dwellers number approximately 2 million people. This trend will only magnify urbanisation by decree, where informal areas are politically declared formal (Castillo, 2000)

LEGAL AND DEVELOPMENT CONTROL MEASURES

Legal Practices and Development Control Measures

The colonial logic of indirect rule left a legacy of multiple legal and development control measures in the post-colonial city. Several legal practices applied in the colonial state, these were inherited at independence and actively maintained by the new power elites. There were basically four legal practices in operation; the Christian, Islamic, Customary and Statutory. The statutory law which comprised the written law adopted mostly from English common law and Roman practices was supreme with regard to matters beyond personal law. The dual Provincial and Local Administrative structures seem to duplicate roles, e.g. County Council and District legal structures. There is definitely a need for merging and harmonising these structures, if a more responsible governance structure has to emerge (Home, 1983).

Several development control measures were in force in both the colonial and the post colonial state with minor modifications. Some of these were; the public health act, Adoptive Building by-laws, land control act, land planning act, land acquisition act and even the trespass act. These control measures impacted directly on the urbanisation process. One of the most influential sources of colonial legislation was the British Town and Country Planning Act of 1932. This was the source of urban planning legislation in Kenya, Tanzania, Nigeria and the Rhodesias (UNCHS)-Habitat, 1996:88). Public health also played a major control mechanism, and its enforcement, including the use of the grid-iron pattern of settlement layout ensured the ease of surveillance and domination by the ruling classes. In addition to the above measures, the inherited British ideology of impartial officials guided by notions of technical rationality, advising elected politicians who viewed the exercise of power as a moral non-political activity – was inappropriate in a post-independence situation (Rakodi, 1997).

Master Planning and Failure of Development Control Measures

Master planning was the main tool used in the development control of urban areas. As was shown earlier all the development control measures for Nairobi in its urbanisation process used the Master Plan as a development control tool. Zoning, density control, height restrictions etc., are used within the Master Plan to parcel certain sections of the city for certain functions. The 1948 Master Plan adopted functionalism as the guiding principle, while the 1973, Metropolitan Growth Strategy adopted decentralisation as the main principle. It should however be noted that rarely have Master Plans been tools of inquiry or documents for the development of strategies. Political decisions have preceded plans, rather than the other way round. Planning within a system of monopolistic power serves to reinforce hegemony rather than operate as regulatory framework (Castillo, 2000:21).

Most African countries (Kenya included) inherited and have kept a legal framework for urban development that was designed to contain settlement rather than to deal with rapid growth. These regulations restrict the provision of housing and services affordable to the poor and the not-so-poor. The result is the emergence of unauthorised or illegal settlements (Hansen and Vaa, 2004). Because of the above rigid legal and development control measures, the urban majority resort to informal urban practices. Informality (Illegality) is therefore occurring by default.

Burra, 2004:143, posits that informal planning is the popular practice of self-help based activities by which individuals, groups of people or local communities provide land rights, undertake spatial structuring or land subdivision, land transfers and service provision. They do this without referring to the administrative and legal state structures, i.e. outside the conventional public sector planning and land development control mechanisms (Burra, in Hansen and Vaa, 2004: 143-157). By marginalising the urban majority, the legitimacy of the adopted legal framework and development control measures, begins to be doubted. Legality is a social construct and can vary from time to time and from place to place. Legality needs to accommodate the urban majority for it to be legitimate.

DEVELOPMENT OF RESIDENTIAL NEIGHBOURHOODS

Rental Housing

After the Second World War, and the preparation of the 1948 Master Plan for a colonial capital, the city authorities embarked on an exercise of building council rental housing for the African urban residents. At this time it was possible to control both the supply and demand because of the restriction of the movement of Africans. The post-independence regime continued with this trend of urban development; however the supply could not cope with the high demand. The relaxation of the movement of Africans created an influx of people into the city, and in any case, even if the supply of housing was substantial, the majority of the urban unemployed could not afford the rental payments to the council.

This type of housing delivery was based on the urban policies, derived from modernisation theory with its strong Western bias. It was believed that strong regulatory intervention (based on Western minimum standards and solutions), was necessary in order to control, direct and rationalise urban growth (Burgess, et al. 1997). This method of housing delivery was not sustainable and was subsequently stopped. The last council rental houses were built in 1978.

Tenant Purchase/Site and Service Schemes

In the late 1960s and early 1970s the government changed the housing delivery strategy and adopted the tenant purchase and site and service methods for housing provision for the low income groups. The 1970s policies that emerged were consolidated at the Habitat 1 conference held in Vancouver in 1976. The new policies that emerged out of consensus included: site and services and self-help housing projects; core housing; slum and squatter settlement upgrading etc. This delivery method could still not satisfy the demand as the resources invested were inadequate. This meant that the low income target group could not access this type of housing as it was taken by middle income groups. The low income groups were therefore pushed out of the formal system into the informal realm. Over the years the government has been unable to intervene in the provision of housing and urban services for the poor and the not-so-poor. This has led to the proliferation of informal activities across all spheres of urban life.

Mortgage Housing

The mortgage housing specifically targeted the middle income groups as the high income groups could fend for themselves without state intervention. This housing delivery system was anchored in the neo-liberalists theories "affordability – cost recovery – replicability" formula, and primacy of individual private home ownership. Several housing schemes on this basis were realised, and it is through this system that the Buru Buru neighbourhood was developed.

The mortgage housing schemes adopted in Nairobi laid emphasis on individual title ownership, and were therefore fairly rigid. It is only fairly recently that the development of condominium properties has been tried under the 'Sectional Properties Act'. This mode of housing development tends to be more flexible and can allow individuals to own flats in walk up apartments. The Buru Buru example is however, was tied to the 15 year or 20 year mortgage, under which the title to each lot was held by the financier/developer until the mortgage was fully amortised. The informalisation that has occurred in Buru Buru, began to happen after the expiry of this 15 to 20 year period. The property owners got possession of their individual title deeds, and were free to mortgage them as they pleased.

INFORMALISATION OF BURU BURU NEIGHBOURHOOD

The State and MNCs Symbiosis

Buru Buru is one of the residential neighbourhoods developed in the 1970s as an owner occupier middle income scheme. The scheme was a joint venture between the Nairobi City Council, the Kenya Government and the Commonwealth Development Corporation (formerly Colonial Development Corporation). It is a good example of the State-MNC symbiosis, for capital accumulation, which establishes MNC economic privileges, whish in turn are shared with an emerging, state-dependent African bourgeoisie (Emig and Ismail, 1980). The basis of this symbiosis rests on the technology transfer capacity of the MNC, and on the Kenyan state's regulatory ability to shape domestic economic conditions so as to assure the profitability of such transfers. In order to achieve the above conditions, the selection process of potential house owners ensured that there would be minimum risk in owners defaulting on mortgage repayments.

The Central Spine

The Mumias South Road corridor was designed as the central circulation spine of the Buru Buru neighbourhood, it was meant to have commercial facilities and community amenities along it. During the implementation period only a few shops and one restaurant were built, while the provision of amenities such as cultural centres, markets, library were never realised. This omission facilitated for an ad-hoc development process on this corridor, and clientelism and manipulation by the power elite as the main principle of urban space procurement. Fig.1 shows a part of the built fabric along this central spine resulting from this informal development process.

It is evident that notions of containerisation and the dual concepts of inside/outside were used in organising the neighbourhood layout. All the lots adjoining the central spine cannot be accessed directly from this spine, making the spine an outside. These lots can only be accessed from the internal courtyards away from the central spine. There is a 9m building line along this spine, which over the years has been ignored and manipulated by individual plot owners, to develop additional commercial space, opening onto the spine and maximising on the economic potential of the lot. Fig.2 shows a part layout of the Buru Buru neighbourhood, the primacy of the central spine is quite distinct.

Courtyard Typology

The general organising principle of the whole neighbourhood was to create introverted courtyards from which all houses were to be accessed. These courtyards were interconnected by footpaths for the free movement of people; they were also linked to the central spine by foot paths. Over the years for security reasons and control, these courtyards have gradually been closed off using turnpikes or gates, creating a sort-off gated community arrangement. The walk through thoroughfares that linked the courts to the central spine have also been sealed, such that courtyards have only one entry/exit point.

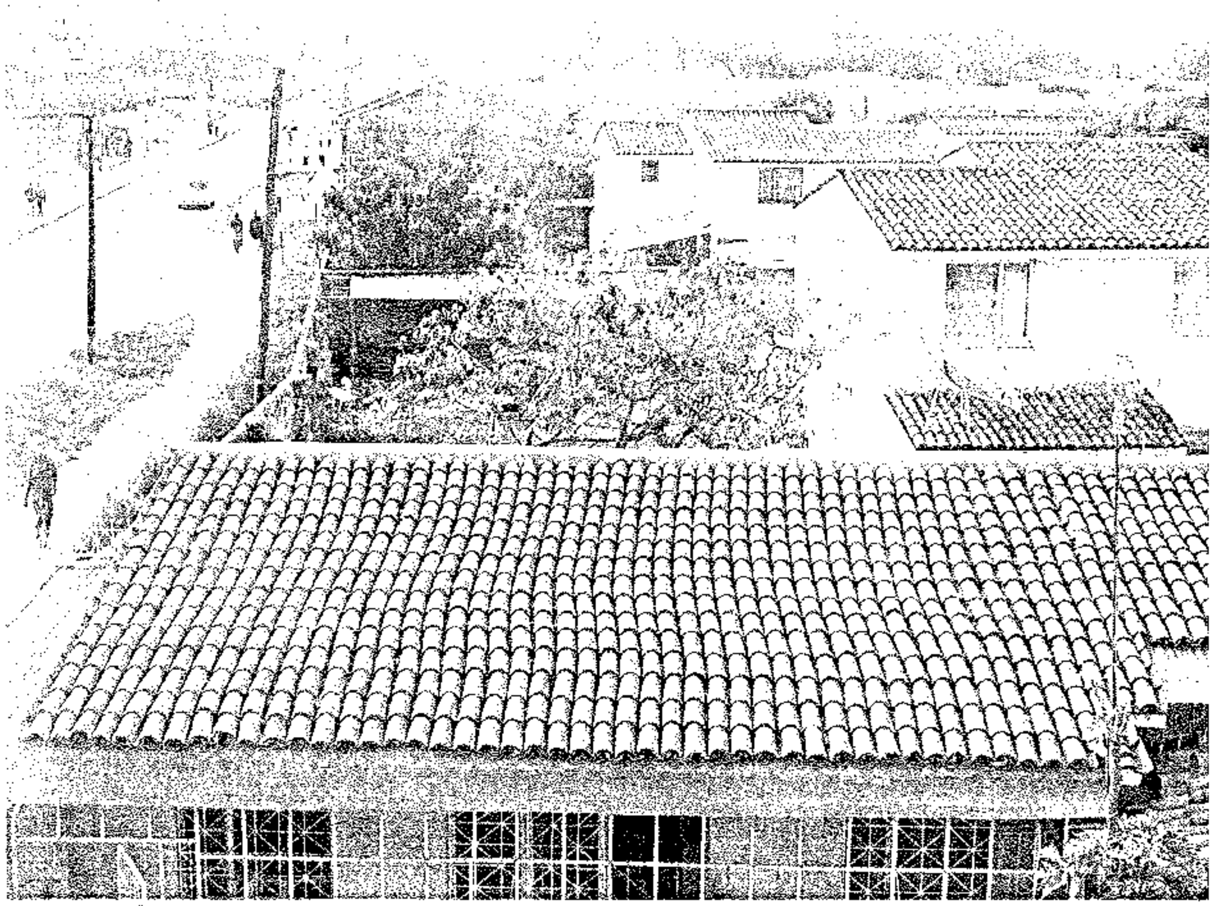

Figure1. Shows the ad-hoc development along the central spine, property owners build extensions up to the plot boundary ignoring the building line as seen above.

Fig.2. Part Layout of the Buru Buru Neighbourhood, showing the primacy of the central spine and the individual lot, where the individual lot dominates the 'whole'.

Transformations on Individual Lots

Fig.3, 4 and 5 show an example of how individuals are able to create additional commercial space within their lots. They basically flout nearly all the by-law requirements on plot ratio, plot coverage, building setbacks and in some cases they even build on service lines. The reasons for these transformations are varied, but basically revolve around the need to generate additional family income. These extensions/alterations are used for all sorts of activities such as; rental housing, home based enterprises or commercial outlets, in some cases there is just a need for additional spaces for enlarged family use.

The property owner on lot 437, after building the extensions, increased the plinth area on the plot from the original 84m² to 184m². This changed the plot ratio from 19.5% to 59%, while the plot coverage changed from 10.5% to 34%. It is important to note that these new higher density figures are still within the allowable design plot ratio of 75% and plot coverage of 35% for the Buru Buru area. This disparity can partly be attributed to the fact that plots adjoining the central spine measure approximately 400m², whereas the average plot measures about 150m². The other reason could also be that, property owners are willing to increase densities by building extensions, but within the council's by-laws without building on public land. In effect they are making a direct critique of the validity of the 9m building along the central spine.

It cost approximately K.Shs 1.7 million (USD $ 22,670) to build the extensions on lot 437, which is quite a high investment relative to Nairobi standards in 1997 when this work was carried out. The rental units and the hair saloon fronting the central spine, are accessed directly from this spine giving a sense of privacy to the main household. These two extensions generate a monthly rental income of K.Shs. 37,000 (USD $494) the main household. This kind of income ensures a guaranteed source of money for the household in a system where social security is weak and retirement benefits are not assured, these transformations act as strategy for future economic survival. They also generate employment opportunities for the informal and small scale entrepreneurs. However the net effect of these transformations is that the carrying capacity of the infrastructural services is increasingly stretched. There is therefore a need for some control in order to have a balanced and sustainable urbanisation process. Finally because of these transformations, there emerges a new hybrid built form typology for each transformed lot.

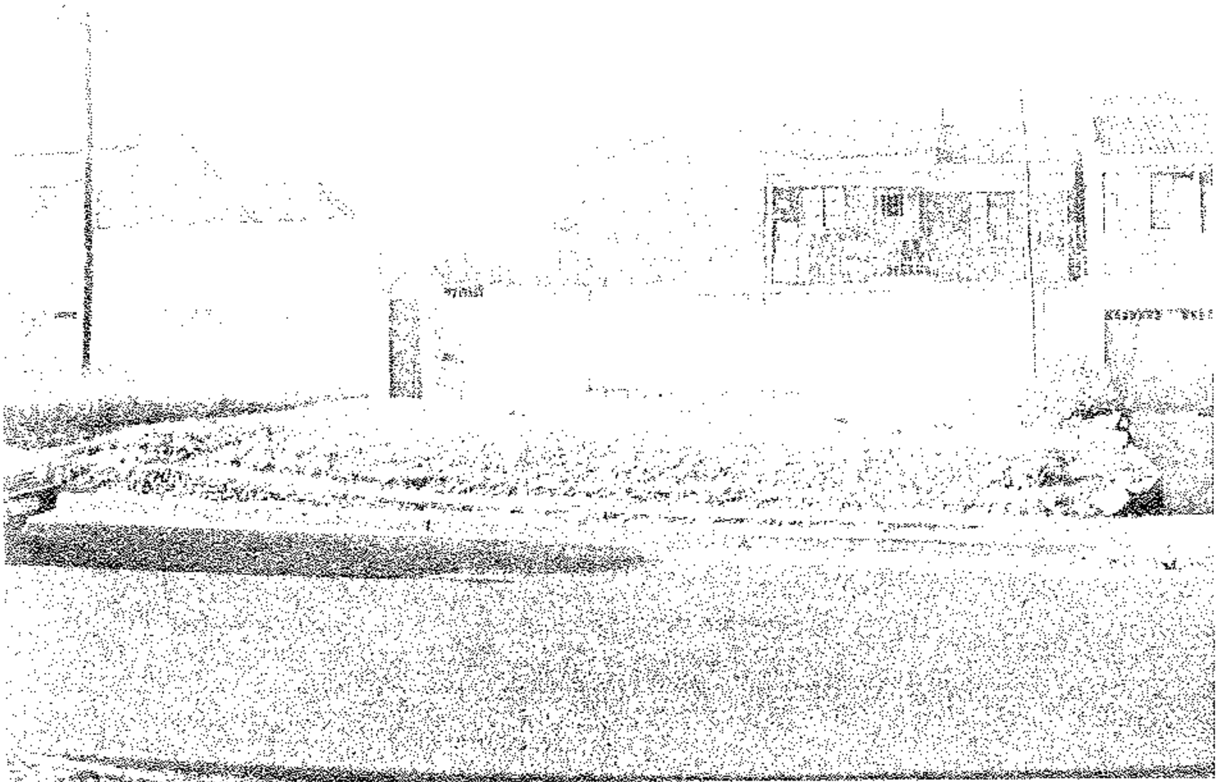

Fig.3 shows the extension lot 437, opening directly to the central spine, creating a porous urban edge and defying the inside/outside divide.

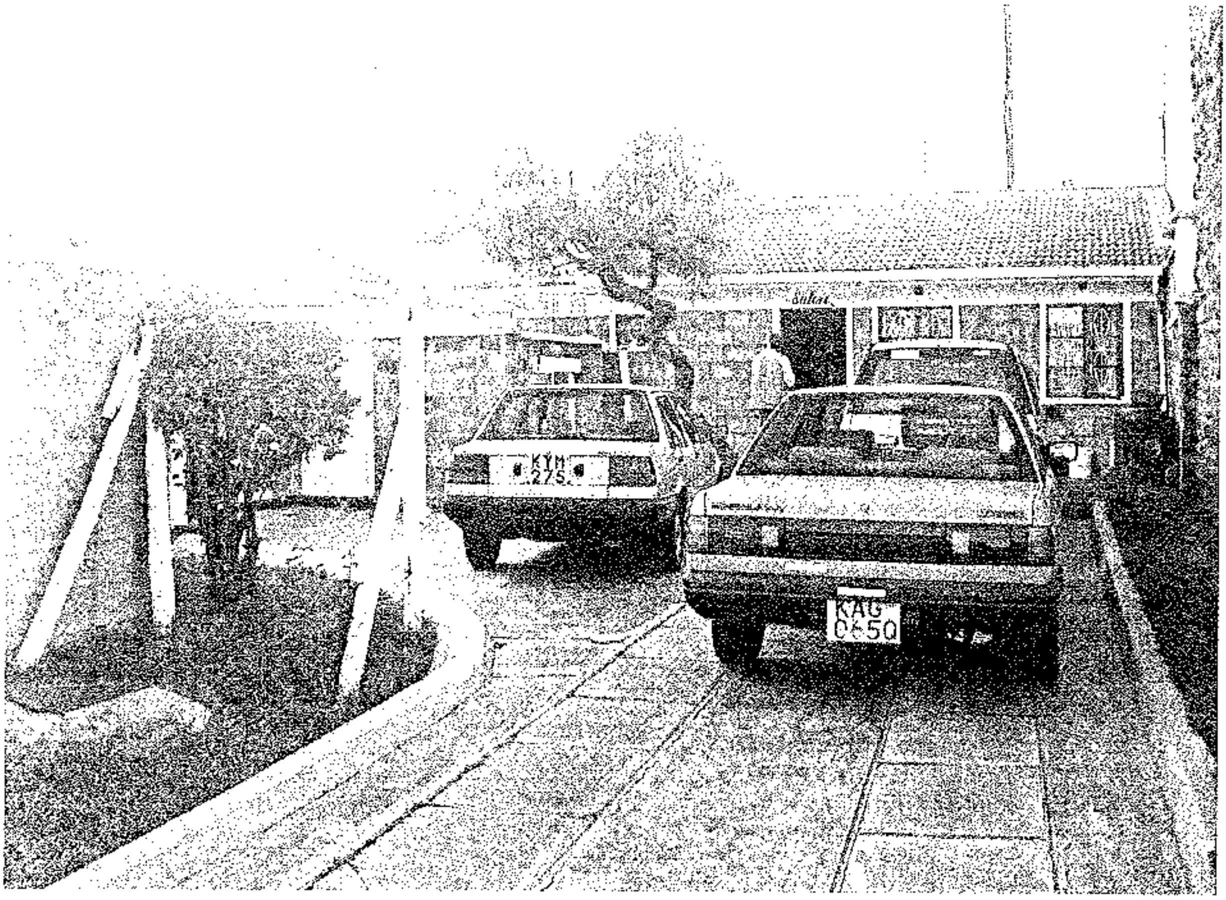

Fig.4 shows the parking lot on lot 437, the hair saloon is to the right and the rental units are to the back. The screen wall ensures privacy between the tenants and landlord.

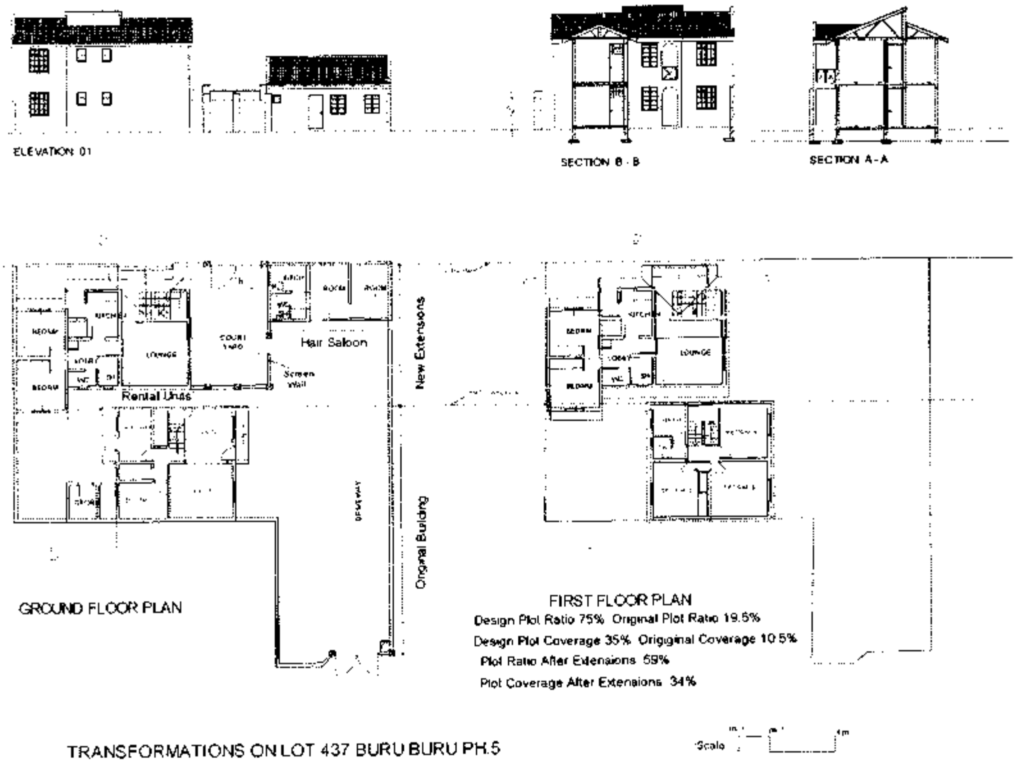

Fig. 5 Shows the transformations on lot 437, creating a higher density but playing within the City Council's design plot ratio of 75% and plot coverage of 35%

Fig.6.Transformers created two storey commercial outlets, with the first floor balconies cantilevering beyond the plot boundary as required by law, an interface of the formal and informal.

CONCLUSION

The urbanisation process of the city of Nairobi is such that many residents don't have much choice in terms of access to housing and urban services. The Buru Buru neighbourhood has shown that middle income people are able and willing to invest in additional urban built forms. However due to lack of affordable serviced land, they are transforming their properties to maximise on the property's economic potential. In the process informalising what was originally formal, 'ex-formal' as Soliman would put it. They are opening out and re-framing the containerised solution they adopted in the 1970s, and are re-defining the urban edge by making it porous.

They also seem to have agreed among themselves that no new structure will be more than two stories high, one owner who tried building beyond two stories, had his structure demolished. Fig.6 shows how some transformers have re-defined the urban edge with commercial outlets. They maintain the two storey typology, and seem to follow the Nairobi by-laws for commercial buildings quite closely, with the balconies cantilevering beyond the plot boundaries as is required by the law. Although the procurement of these buildings is informal many formal mechanisms are engaged concurrently. In general these transformations may not be entirely negative, but they require a certain level of co-ordination and control. The public and the individual good may not always be observed when people transform their lots in an ad-hoc manner. The council therefore needs to dialogue with these transformers, in order to put in place some control mechanisms. As a concept, formalism is a shifting phenomenon, and what the Buru Buru transformers are showing is that people are willing to endure higher physical densities at the expense of some monetary gain.

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EFFCTIVE PARTNERSHIPS AS A WAY TOWARDS ACHIEVING SUSTAINABLE URBAN DEVELOPMENT
CRITIQUE TO SOME EGYPTIAN EXPERIENCES

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ABSTRACT: Urban development is a complex set of actions which require a holistic and agreed set of responsibilities amongst the various actors involved. The past 10-15 years has brought considerable innovation in that respect in which governments in both developing and developed countries couldn't achieve, solely, much without effective partnerships with the private sector, the civil society organizations and other stakeholders. It is a crucial process if the Egyptian government is decisive in enabling other actors in order to achieve sustainable urban development.
This paper is mainly evaluating a number of Egyptian initiatives in managing urban development through innovative approaches. The paper will discuss and analyze some theoretical background behind the necessity of securing a number of elements in order to achieve sustainable urban development. It will critically assess and evaluate the benefits of effective partnerships with the private, the civil society and other stakeholders in the Egyptian perspective. Through the cases of Qena and Fayoum, the paper will critically assess the advantages and disadvantages of effective partnerships in the field of urban development. Finally, the paper will draw some guidelines and main conclusions from the different case studies in order to assess replicability of these assessed case studies in other Egyptian governorates.

KEYWORDS: Urban Management, Local Development, Sustaianble Urban Development

INTRODUCTION

Egypt is passing towards a new era where the Egyptian government is keen in enabling the spectrum of stakeholders in order to achieve sustainable urban development. Baring with this challenge in mind, this paper is evaluating a number of Egyptian initiatives in managing urban development through innovative approaches. The paper is composed of four main sections; the first section will build a practically and internationally driven framework to evaluate the strategies which aims to achieve sustainable urban development. The second section will describe and analyze the two case studies (Qena and Fayoum), where urban development strategies were articulate using different approaches. The third section of the paper will critically assess the two case studies using the framework built in the first section and finally the last section will summarize the main lessons and the final conclusions of the paper.

FRAMEWORK FOR DIAGNOSING THE EGYPTIAN CASE STUDIES

Before moving into description and critical assessment of both case studies that will be discussed in this paper, it is important to build-up a framework for analysing the two case studies. This framework is based on identifying different elements that important for implementing any urban management activities sustainablly. In this section we will illustrate these elements such as:

- Importance of setting priorities prior to implementation
- Identifying specific actions considering the policies, programs and projects which follow from them
- Profiling actors (Who will carry what).
- Mobilising these actors
- Assuring the participation of the different actors
- Strengthening their capacities to perform.
- Securing co-ordination.
- Mobilising the required resources
- Obtaining political support
- Mobilising/providing leadership

These elements were set by practitioners [4,7 & 8] and it will be used to analyse and assess the innovative approaches applied in the two case studies;

- ***Setting priorities:*** a prime function of urban development planning is to set priorities. Priorities will vary from place to place with the consequences that an essential function of managing an urban area is to establish and maintain a sense of what are the priorities are, whose priorities they are, who defines them and how they are defined. City's most pressing problems must be identified and prioritised by public, private and popular sector representatives[11]
- ***Identifying specific strategies***: this is the second step in implementing the urban development plans. It is important to apply a participatory approach when identifying specific strategies for actions [11]. The strategies are materialised into actions through programs, policies and urban development projects. This step is crucial in any urban management framework. Strategies must be defined within a dynamic process in which the means and resources are assessed as well as the necessary steps and decisions about who will carry out what and when. A series of "mini consultation" has to be performed for the prioritised problematic areas such as solid waste improvement, servicing city expansion, managing surface water, conserving green areas, employment generation, etc. In this stage, the different

stakeholders should be involved in order to prioritise the most pressing problems, agree on strategies, set immediate and medium term actions, formulate, mobilise and launch detailed cross-sectoral and multi-institutional working groups to prepare detailed spatial, financial and institutional action plans for each strategy component and finally action plans have to be prepared and implemented in partnership with other public, private and popular sector institutions that will trigger demonstration projects in selected areas of the city.

- ***Identifying a profile of the different actors:*** There are three main actors in any urban development projects. Firstly, the public sector; among this sector there are different tiers of governmental officials including central government ministries, public service authorities, and in some cases an intermediate level of provincial government. Secondly, this is made of a broad spectrum of organisations which construct, operate and maintain urban development projects; henceforth their participation in setting priorities, identifying strategies and projects is crucial. Finally, the popular sector; this sector can perform significant urban management actions through a variety of voluntary and co-operative organisations. Understanding the profile and intrinsic motivations and mission of these actors will lead to better co-ordination in setting priorities, identifying problems and formulating strategies. However, Governments at all levels tend to view the private and popular sectors as having little or no role in implementing urban development strategies. The gaps in these relationships hamper communication to a degree which greatly reduces the possibilities for concerted implementation of management actions. Agreements on strategies among several actors is rare; the elements of a single strategies/projects are often decided by a single agency- usually in central government for urban development projects- with minimum involvement of others [7].
- ***Motivating actors:*** New urban management policies/strategies must take account of the motivations of those administrators, politicians, community leaders or whoever is likely to be involved in the implementation. By doing so resources will be more efficiently used [3]. The community could be motivated through being efficiently involved in the different stages of planning and implementing urban development plans. Supporting partnerships with the community can lead to better delivery and maintenance of infrastructure as documented by several authors i.e. delivering these strategies into actions [8]. The government actions required to support partnerships with the communities in the provision of useable information for more

transparency in local government actions. Through these actions, the government will initiate a sense of commitment and hence decrease the negative impact of rapid urbanisation on the environment.

- ***Participation of the different actors;*** Participation of the target population in the planning and implementation process of urban development is crucial. Municipal policy is predominantly political in character and will usually contain general objectives. The translation of policies into explicit programme contains well-articulated targets, budgets and implementation time frames that are essential function of municipal management. However, this can't be accomplished without the participation of the target group. It is crucial to know what is the form of participation when and how can their involvement be achieved for motivating these actors. Thus supporting the implementation of action plans outcoming from the planning process. Yet the formulation of objectives can not be separated from the identification of strategies used to achieve them. This is where the enabling approach to development comes into play [11].
- ***Strengthening their capacities to perform:*** The sustainable urban development approach must evolve the enablement of both the private and the community sectors in the formulation and implementation of urban development strategies and plans. This requires that local government staff must have greater capacity to stimulate, control, guide and negotiate those strategies and the consequent prioritised actions/projects of urban development. Strengthening their capacity is not only about training but also about inventing/changing the institutional setting to allow direct involvement in the planning process.
- ***Securing co-ordination*:** Securing co-ordination will minimise competition and open gates for collaboration. This means articulating and coordinating all tasks and processes, carried out by several agencies in order to accomplish cohesion and maximization of program objectives, resources and budgets. It links various organisations in defining overall priority needs and opportunities, in deciding what is to be done and undertaking actions in the right sequence where different stakeholders are involved. Appropriate co-ordination needs to be envisaged in the planning process, because the most effective and cost efficient programmes frequently require attention to several separated actions. In reality actors are placed in competition with each other for scarce funds and limited skilled manpower and equipments [7].
- ***Mobilising the required resources:*** Proper resource allocation is essential to sustain actions that improve living conditions. The major

resources required are human, financial, material, institutional and informational resources. The skills and capacities of human resources are found not only in the public sector but also in the private and the popular sectors as well. Hence mobilising these resources is essential for implementing of urban development plans. Financial resources are found in the three sectors as well; land is a fundamental resource that must be secured. Institutional resources include the framework of formal relationships and legislation which grants mandates and authority to organisations. Information resources are type of resources to be mobilised vis-à-vis the knowledge and expertise that are fundamental for strategic planning formulation [7].

- ***Obtaining political support:*** Some authors argued that this element is very important driving force. There is a tendency to regard political will as an independent variable "An act of God". If is there, things work; if it is absent, everything fails [4]. Political support is required for attaining support and institutional collaboration to strategic actions at the governorate level. It is crucial in supporting the less powerful groups in the different stages of formulating strategic planning i.e. setting priorities, identifying strategies and actions. It is crucial as well to motivate the different actors and to mobilise the different resources shown above. Over arching co-ordination between different actors is key role to be strengthened by executive and political leaders.
- ***Providing leadership:*** A central need is political leadership, vision and commitment by the city authority to address the challenges of urban development. Political authority needs the support of strong administrative leadership. This authority is needed to solve problems at the local level; the authority of administrative leadership is needed to deter political leaders from excessive intrusion in managerial functions. It is needed to co-ordinate actions and visions of the different actors involved in the preparation of a strategic plan [4].

The elements defined above are crucial in assessing the innovation in any urban management activities. In the next section of the paper will critically assess the availability of those elements in a number of Egyptian cases. Some of them were already known for the Egyptian while others are not with the same profile in terms of their publicity amongst the Egyptian experts and urban development professionals.

THE EGYPTIAN CONTEXT OF EFFECTIVE PARTNERSHIP FOR URBAN DEVELOPMENT: A RAPID ASSESSMENT

Egypt has passed through generation of complete centralised management style applied in the local government system. However, since the beginning of the 90s the practise has changed towards a mix of both centralised and decentralised systems. Both systems acknowledge that effective partnership is an effective tool towards sustainable urban development. The central Egyptian government and the governors believed that centralisation will never achieve the goals of sustainable development, without enabling the other actors most importantly the civil society organisation and the private sector, innovative urban development solutions will never be achieved costs. This section will describe and assess two case studies where urban development strategies was developed using different approaches

The case of Qena

The experience of urban management in Qena governorate showed an integrated approach for the regeneration of Qena which not only innovative in the approach which will be discussed in the next passages but also innovative in the way that urban development took place in a balanced approach between innovation and the regulatory framework which guide local government in Egypt.

Qena is one of the poorest governorates in southern parts of Egypt. Due to the bad living conditions, it has become one of the main sources for terrorism in Egypt. The governorate stretch over almost 11 thousand km2, only 16 % of this area is inhabited with a total population of 2.7 million inhabitants and more than 78 % of this population inhabits rural areas. The governorate is equipped with a variety of resources that could generate the economic and social gentrification of the governorate in addition to agriculture, mining, tourism and most importantly human resources [13].

Prior to 1999, the governorate was suffering several types of pollution arising from different mining industries in addition to sugar cane factories and aluminum factory, the governorate was mainly suffering lack of proper management of its arid land in any economic activity, also the governorate was suffering high percentage of unemployment reaching more than 50 % of the total populations, moreover, the governorate was suffering illiteracy reaching more than 50 % amongst adults. Physically, the governorate was suffering lack of proper water and sanitation services amongst the different parts of the city, huge number of informal settlements and building violations on public and private property, lack of environmentally accepted solid waste management collection system due to the lack of proper equipments in addition to the lack of final disposal system and finally lack of recreational areas [15]

The governor of Qena realized all these challenges in addition to highlighting the possible advantages of the governorate. The governorate mobilized all the scientific efforts of the south valley university and the information centre affiliated to the governorate in addition to the efforts of other internationally funded programs most importantly SEAM program. The philosophy of the governorate was based on a number of strategic goals including:

- Linking the economic, social and environmental planning into one integrated planning approach.
- Full implementation and respect to the existing regulatory framework without violating urban development laws
- Full participation of the community especially in generating new financial resources apart from the traditional centrally located resources.
- Leadership where the governor plays the role of the community leader in order to avoid any breaking of the general rules
- Proper identification of problems through scientific research, weekly meetings and facilitating mini-consultation groups in different themes such as agriculture, water resources, solid waste, industrial pollution and health.
- Mobilisation of the university and the scientific centres and the use of the local artistic in the beautification of the city.

The governorate applied the participatory planning approach in cooperation with a number of donor funded projects including SEAM program (British funding), Life program, UNICEF and Egyptian Swiss Development Fund (ESDF). This approach was reflected in the following strategies and the main programs of actions in various fields including the following;

1) Cleaning and beautification strategy. The strategy began with Qena city and stretching over other areas in Qena governorate. This strategy was mainly targeting to increase the liveability of the city and to increase its economic base in order to attract both local citizens and tourists and their involvement in the city regeneration.
2) Revitalising the industry at the governorate level as one of the main vehicles for city regeneration. This strategy was materialised in a number of activities including, building new industrial areas to accommodate the polluting industries in addition to renovating Small and Medium Enterprises (SME's) as one important feature of the governorate transformation.
3) Renovating the agricultural base of the governorate. The agricultural land in Qena is only 333 thousand feddan, the governorate, through applying technology based tools such as the GIS, will be reclaiming another 352 thousand feddan in the different markaz. The plan was mainly to target those unemployed especially the youth to increase the participation of the youth in the regeneration of the governorate and to include them in the working force of the governorate.
4) Tourism revitalisation; this strategy was aiming to build a number of Nile docs to accommodate the Touristic Nile Cruisers, renovating the

main Nile Road (Corniche), building a number of hotels, sound and light museum and beautification of the highway linking Qena to Luxor and Red Sea governorates.

5) The final strategy aimed to develop all the basic social services in the governorate including educational, health, recreational and infrastructure. This development happened in an integrated way rather than piecemeal approach and in cooperation with the community [14]

Although, the implementation of these plans and strategies faced shortage of financial resources, however, the executive authorities at the governorate headed by the governor mobilised the local popular council in order to raise local financial resources either added to the electricity bill or building and driving licences.

In the next paragraphs, the paper will highlight some of the main features of the different action plans and projects implemented to achieve the above strategic goals.

- As mentioned before, The regeneration initiatives began in Qena city in order to give priority for capital and to generate financial resources from the attraction of tourists and to use these resources later for the sake of the different poorer areas.
- Covering canals and drains inside the urban fabric which was one of the reasons behind the low health standards at the governorate.
- Paving the main and the secondary roads, expanding the road and making a number of service roads
- Relocating 680 polluting workshops outside the governorate to a 50 feddan piece of land allocated for vocational activities in addition to relocating the non aesthetic minibus stations to another lot of land allocated for this activity outside the urban fabric of the city, in addition to enforcing a certain route for these type of transportation service to avoid any traffic problems in the future.
- Adding several landscaping elements throughout the city to feel like a city for people, including the addition of green areas inside the city, changing most of the outdated light poles and adding new artistically designed once instead, renovating a number of squares by changing the pavement and using common colours in the pavement, sidewalks, street facades and unifying the design of signs.
- The physical projects were coupled with others to raise the income of the community and hence increasing their responsiveness including increasing the investment allocated for building industrial establishments. More than 30 establishments were built to add up more than 1500 job

opportunity in a total investment of 790 million pounds, financing 6000 new job opportunities in the field of livestock production through mobilising a new local fund raised to support the employment generation without waiting for the central budget [15].

The case of Fayoum

The second case study will describe and analyse the process of building an innovative approach to guide urban development which started in 1992 in Fayoum governorate. The process was based on applying participatory planning and building effective partnerships with these stakeholders in order to secure the implementation of urban development plans. The process was mainly based on mobilising the research centres and the university in setting down and prioritising strategies for the urban development planning at the governorate level.

Fayoum is a natural depression of some 12,000 Km2 encompassing three separate sub-basins; Fayoum depression, Wadi El-Aryan and Muwellih. It is laid in the western desert of Egypt in the south west of Cairo Governorate and largely surrounded by uninhabited desert uplands and has a direct access to the Nile valley and a direct supply of Nile water. It has an attractive rural setting, lakes and historical sites and includes a number of wetlands of great economic and ecological importance (Qaroun Lake, Wadi El-Rayan) [4]. Only 22.5% of Fayoum population lives in the urban areas, the majority 77.5% inhabit smaller towns, villages and hamlets of the rural areas. Population density is 1191 person/km2 in urban areas [6].

The largest proportion of employment in Fayoum is engaged in agriculture (including hunting and fishing). The next largest sector covers government and public services (electricity, gas, and water, social and personal services). Industry (mining, manufacturing and construction), trade (commerce, hotels, and restaurants), private services (banking, insurance, real state, etc.) and other activities represent much smaller employment proportions [4].

The above natural characteristics, population distribution, and the status of economic forces in Fayoum Governorate contributed to fostering the change in the planning types practised into an innovative strategic planning approach.

- The above environmental resources is an asset which require great emphasis when dealing with urban development projects
- The demographic balance between the city and the village areas requires certain balance in between the urban development and the rural development proposals through emphasising upon the improvement of living conditions of the inhabitants of the different villages.
- The concentration of employment in some sectors mainly, agriculture, hunting and fishing lead to the overemphasising strategically on these sectors which have certain implication on the urban development prioritisation.

The above opportunities together with the great support of the two consecutive governors lead to the formulation of its innovative approach for implementing urban development plans. Fayoum, which encloses the Fayoum depression and Wadi El-Rayan sub-basins, has some unique features but also some unique environmental and natural resource use challenges:

- Every gram of water, salt or pollutant that enters the basin accumulated there, thus demanding careful attention for water quality and water balances.
- A density settled and growing human population of 1.99 million (1996) cultivates a limited land Resource of fertile soils (342.55 Feddan) requires careful management of the limited land resources.
- An uninhabited sub-basin (Wadi El-Rayan) with two largely unused freshwater lakes represents a development potential which requires a careful development planning [4].
- The demands of the high density of the population in the urban areas regarding the limited resources, the restricted possibilities to both urban and agricultural expansion and the existing problems in Fayoum with water and sanitation and other infrastructure and urban services especially in the villages threatened the health, the environment and the urban productivity .

This case study will articulate the content of the urban development approach built in this governorate through discussing the outcomes of two main reports. The first report for Development Horizons was prepared when the ex-governor of Fayoum (Dr. Abd El-Rehim Shehata) built a strong relationship between the Governorate and the Academy of Scientific Research and Technology (ASRT). The governor believed that research bodies are capable to draw scientific and rational strategic views of urban development and link these views to the national development plans (five years plan). Through the support of the ex-governor the governorate started with the technical assistance of (ASRT) to formulate its urban development strategy till the year 2000 and was presented in the conference (Development Horizons in Fayoum:1) which was held in 1990 and published to the different actors involved in the development process[10]. In 1997, the successive governor in Fayoum (Mr. Mohamed Hasan Tantawy) supported the formulation of the urban development strategy development strategy in the next twenty years (till 2017) in cooperation with Cairo University (Fayoum Branch). The university together with the governorate staff and information centre started to prepare the development horizons of Fayoum and presented this strategy in a documented report in 1998 (Development Horizons in Fayoum -2) [1].

The Governorate and the Academy for Scientific Research and Technology (ASRT) organised working groups formed of professionals and experts from different universities and research centres and through a series of working sessions. They prepared their views and discussions on the governorate different themes of development as a basis to articulate the conflicting themes with the participation of representatives from the following stakeholders :-

1. Governorate Staff
2. Members of Local People's Councils
3. Members of Executive Councils
4. Central Government

These mini consultation groups gathered data and information about challenges facing urban development in Fayoum, projects achieved, problems and proposals to overcome these problems, recommendations for each sector for development, and finally the required action plans for each sector.

The above approach was prepared, documented and shown in the first conference at 1990. It tackled 10 fields of development that the Governorate considered this initiative to be of great importance for achieving urban development objectives, and published these studies to the different actors involved the development process .These fields are:- Flora wealth - Fauna wealth - Fish wealth – Tourism - Social development - Small industries and cultivating based industries - mining and industry – Energy – infrastructure - environment and population [10].

The second report for Development Horizons was prepared by Cairo University (Fayoum Branch) with close co-operation with the Governorate and its information centre through the backing from the governor. The report illustrated the present status in the Governorate in every field using the published data gained form the information centre of the governorate. The strategic aims of development were briefed in the report as follows;

- Improving the living environment for the citizen in Fayoum.
- Improving of Services in general.
- Conservation of the Environmental assets and bio-diversity.
- Creating employment opportunities.
- Development of some economic sectors such as industry and Tourism.

Five workshops were held in Fayoum Governorate; stakeholders were identified and participated on the preparation of the report and in the consultation workshops and working groups including;

- The Governor
- Vice Head of the University in Fayoum
- Deans of the Faculties
- Representatives of the Administrative Departments in Fayoum Governorate
- Chiefs of the People's Councils
- Experts and lecturers from the University [1].

The two consultative approaches identified the important role of the research bodies in formulating the strategic thinking of the governorate. However, it is not enough to have the research bodies involved, there must be a great role played by the target groups including the private and the community sectors. This experience faced some challenges that couldn't cope with innovatively including securing financial resources to implement the basic strategies and actions plans. The approach used in this experience showed that strategic thinking requires some tools to secure its implementation, participation of the different stakeholders and securing local financial resources were amongst these tools. The case study showed that stakeholders workshops and large gatherings was used as a means to facilitate coordination between the different stakeholders besides the traditional two weeks meetings held for the executive councils at the governorate level [12]

CRITICAL ASSESSMENT OF THE TWO CASE STUDIES

To analyse the two case studies, it is worth applying the framework discussed at the beginning of the paper. The second case study represented the shortcomings facing the implementation of strategic urban development when the main elements of innovation in implementing urban management actions are not found at the governorate level. The financial challenges will be and always the main challenge required implementing the innovative strategies and its action plans.

For setting the strategies, which was discussed before several mini consultation were set in both case studies. In Qena, the governor led all the different actors and stakeholders to participate while in Fayoum case only two main parties; the governorate departments and the university or any other research bodies were participating, which did not represent the diverse needs of the communities. In the second consultative approach (Development horizon part 2) of Fayoum case, the needs were prioritised through the mini consultation groups of the university members only.

Considering the different actors, there is no evidence that the private sector (formal and informal) was actively involved in setting the strategies or the priorities of the strategic urban development approach especially in the case of Fayoum. The informal and formal private sector were participating in the setting down the strategic goals and agreeing about them and hence both actors were actively involved in the implementation of those strategies. in the mini consultative approach in the first and the 2nd development horizon initiatives of Fayoum case, the community role was not identified as well in any step of this strategic approach especially those undertaken by the governorate. In the case of Qena, the role of the community was much more evident, the active participation of the community was the corner stone in revitalising the city. The popular council representing the communities played another important role and their cooperation with the executive council headed by the governor was, by no means, an important step that secured the cost recovery of several projects and hence its sustainability.

Considering the capacity building component, in both cases, budgetary allocation for training is very limited and couldn't meet the diverse training needs of the different stakeholders with few financial resources provided from the social fund for development (SFD) who invested heavily in various fields of urban development. In the case of Qena, capacity building and with special reference to training was mainly achieved through the cooperation of several donor funded projects especially SEAM program and Egyptian Swiss Development Fund (ESDF).

Securing co-ordination mechanism amongst the different actors, the paper identified the traditional mechanisms for supporting co-ordination especially on the level of the executive authority at the governorate together with the social fund, NGO's and Shourouk Program in the two cases. Both cases gained their strategic and integrated approach through mobilising the political support and the backing of the governors. In relation to the second case, stakeholders and large gathering meetings was used as a vehicle to bridge any conflicting gaols or interests, the university and the research centres played an important role in mobilising the governors top secure those coordination mechanisms strategic approach we can not identify any kind of co-ordination between the different actors. The case of Qena illustrated good coordination mechanisms through securing the direct communication between the different officials as a tool to eradicate the bureaucratic paper work. Moreover, mini consultation and working groups were enacted as a clear coordination mechanism in Qena.

Mobilising resources. In relation to the fact of resources' scarcity in Fayoum, although, the different conferences highlighted before acknowledged the scarcity of developable land as one of the main constraint at the governorate level, none of the two conferences came up with a practical action plans to deal with this challenge, with the exception that some unique projects, would require the communities to donate portion of their lands for return to some community projects. In the contrary, there was evidence that information resources was heavily mobilised. The governor in Qena secured the importance of land resources either through land reclamation projects that are implemented or through successful partnerships built to attract SMEs in order to invest in the industrial zone allocated. Institutional and human resources mobilisation is one of the positive aspects in Fayoum where the two governorates successfully built effective partnerships with both the research bodies and universities in both governorates. Furthermore, Qena showed that investment in human resources was one of the main features of the integrated development approach

Finally, the paper, through evaluating the two cases stressed the importance of mobilising the political will of the different governors in mobilising the above efforts. Fayoum is a case where we can identify both political leadership and administrative leaderships merged together. The same could be visualised in Qena governorate where prior to 1999, the governorate was suffering like other governorates while after 1999, the political will of the governor mobilised all the different stakeholders especially the community leaders, community mobilisers and the local popular council.

FINDINGS AND RECOMMENDATIONS

The paper discussed and highlighted the different approaches for implementing urban development strategies and plans. It showed the impact of securing the different elements of the framework on the implantation of these strategies and plans.

The case of Fayoum showed and discussed the implementation of innovative approach for urban development plan through mobilising the role of scientific centres and universities. The role of the two successive governors was decisive and the innovation of securing innovative coordination mechanism was showed as well. However the outcomes were not as great as the process pf building the planning approach. Securing innovative mechanism to generate financial resources required for the implementation was missing, the participation of a wide spectrum of stakeholders was also missing. The case showed that these two elements were decisive for the success of the planning approach.

The case of Qena showed and discussed a different and more integrated approach towards the formulation and implementation of sustainable urban development. The role of the governor in Qena was much more evident and central. He mediated between the strong regulatory framework which guides local government at the national and governorate levels and between the actual situation in Qena, he mediated to guide, mobilise and lead the different stakeholders including the public, private, civil society organisations and donor funded projects. He initialized different actions to secure some coordination mechanisms at the governorate level and he formulated and agreed upon with the popular council about certain mechanisms for the financial involvement of the different stakeholders. This approach was realised through innovative projects through which integrated approach of development was implemented

In essence, this paper is confirming the importance of securing innovative mechanisms for the successful implementation of the sustainable urban development

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ENERGY PERFORMANCE OF BUILDINGS DIRECTIVE IN EGYPT A NEW DIRECTION

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ABSTRACT: The Egyptian community in its path for rapid development is endeavouring to make all necessary and appropriate measures to enhance the efficiency of energy utilisation and increase the beneficiation of the energy resources. The energy production, transmission, distribution and utilisation efficiency becomes a vital factor and measure of national development. Governmental organisations were established earlier to be responsible for energy planning and efficient utilisation, information dissemination and capacity building as well as devising the necessary codes and standards. Throughout the Nation Energy resources are widely used and consumption rates are in general exceeding the International accepted values. Energy rationalisation and audit exercises were developed and monitored by OEP, Universities and Research centres through the past two decades with a definitive positive energy reduction and beneficiation. The development of the relevant codes for Residential and Commercial Energy Efficiency in Building is underway. The Housing & Building Research Center HBRC is a governmental Body responsible for the research and development in the building Technology sector and is the umbrella under which the Egyptian National Codes are developed and issued. A proposed new Energy Performance in Buildings Directive would fulfil the following main targets of Energy Performance Directive:

1. "Legestilative authorities shall ensure that, when buildings are constructed, sold or rented out, an energy performance certificate is made available to the owner or by the owner to the prospective buyer or tenant, as the case might be. ...
2. The energy performance certificate for buildings shall include reference values such as currant legal standards and benchmarks in order to make it possible for consumers to compare and assess the energy performance of the building. The certificate shall be accompanied by recommendations for cost-effective improvement of the energy performance..." The following steps are required for the energy certification:

1. Develop methodologies for energy declaration of the buildings
2. Develop reference values (key numbers) and /or systems for benchmarking
3. Provide a labelling system for selected buildings

Keywords: energy performance, standards, calculations and modelling

ENERGY DECLARATION OF BUILDINGS

The primary use of energy declarations is, among others, to:

1. Create consciousness of energy efficiency in buildings and also improve the knowledge of energy use in buildings;
2. Use the information to determine if the building works as well as possible with regard to its technical design;
3. Use the information for benchmarking;
4. Use the information for proposing measures and recommendations for energy use reduction;
5. Provide necessary information to make calculations of the environmental impact due to the energy use, e.g. CO_2 emission;
6. Describe selected energy properties of the building;
7. Give the basis for a common energy performance certification of a building

It is believed that depending on the purpose of the energy declaration, different procedures can be of interest. Different actors need different information[1-8]. For giving relevant advice to the property owner which measures are cost-effective a very careful examination and calculation of the building's energy balance is necessary. A careful analysis is also necessary to give relevant information to the users how they can decrease their energy use without decreasing, under an acceptable level, the indoor air quality and thermal comfort. The main purpose of the new energy directive is to reduce the energy consumption in the building sector and consequently carbon dioxide emissions. Therefore, priority should perhaps be given to the buildings with the highest energy consumption. One way to proceed is to carry out the energy calculation in different steps for existing buildings[7]:

- The first step is to collect measured energy-use, e.g. from energy bills, and make a benchmarking to decide if the actual building is better or worse compared to similar buildings. If the energy-use seems to be higher than the average for a comparable grouping of buildings.
- A second step is to make a careful energy calculation that can be compared to the measured energy use. This has to be done for identifying what kind of measures can be recommended in order to reduce the energy use in the building.
- Some important aspects necessary to take into consideration when developing a common tool for energy declaration of buildings are discussed. The discussion is here focused on residential buildings but similar principles are relevant for other types of buildings.

1. Requirements on an energy declaration of a Residential Building

To determine how much energy is needed annually during normal weather conditions in a specific building in order to meet user requirements on thermal comfort and indoor air quality. The energy declaration should also supply information on the potentials for energy reduction. The energy declaration must include information on the total energy use in the building, e.g. energy for heating, ventilation, cooling and domestic hot water as well as electricity for running the building and for use in the household. Thermal comfort and indoor air quality levels can be defined by ISO or equivalent national standards or by using well developed and evaluated questionnaires to be answered by the tenants, indicating their satisfaction. In order to get an acceptable thermal comfort, the indoor air temperature may be different depending on the thermal performance of the building envelope and on different living patterns in different regions. To get an acceptable thermal comfort in a badly insulated and leaky building the temperature must be higher, compared to a well-insulated building.

2. Parameters affecting the energy use in a Residential Building are:

1. The thermal performance of the building envelope, the efficiency of the ventilation system, the heating system and other equipment installed. This also includes the possibility of the building to utilise passive solar heat gain.
2. The user behaviour such as use of domestic hot water and use of household electricity; also the users' choice when purchasing different electric appliances and how these are used.
3. How the building and its installations are operated; how well the heating (cooling) control system works. If the heating control system does not work properly and/or the ventilation system does not provide acceptable indoor air quality, the users have to make corrections like opening windows. In most existing buildings the thermal performance of the building and how the building services are managed have a major impact on the total energy use. The user behaviour and how well the technology interacts with users' requirements become more and more important the more energy efficient the buildings are. In conclusion the energy use in a building is a complex mixture between the building technology, HVAC-systems, the maintenance of the building and the behaviour of the users. This has to be considered in an energy declaration. A common, well-accepted and standardised system for energy declaration of buildings is strongly recommended.

3. Energy Declaration of Existing Buildings – A Possible Procedure

For most existing buildings the energy use usually is well known via the energy bills. The construction details are on the other hand not very well documented. Calculations of the energy use will thus in most cases be very uncertain and difficulties will occur when giving relevant recommendations of cost-effective energy conservation measures. The energy declaration needs a combination of tools for calculations based on the information from the energy bills. For existing buildings the energy declarations can be based on both measurements and calculations in order to fulfil the purposes mentioned above. Normally the heating bills are based on measured heat delivered to the building. In most cases, the energy for domestic hot water is included in the heating bill. To get the total energy use in a building, the electricity to run the building and household electricity have to be added. In electrically heated houses, common in e.g. Sweden, the house-owner just got one bill covering the total energy use. To make the measured values objective and comparable several corrections and calculations are necessary:

- it is necessary to check that the indoor thermal comfort and air quality meet agreed requirements
- corrections of the heat use to normal outdoor climate – primarily outdoor temperature (maybe also solar heat gain)
- necessary corrections for internal heat gains, e.g. differences in household electricity presentation of the energy use in comparable units e.g. based on areas

and/or volumes which have to be defined (in a standard), so called key values the energy use has to be split up in parts referring to the performance of the building, the installations incl. operation, and user behaviour.

The energy has also to be divided into electricity and heat. In order to determine if the building works as well as possible, with regard to its building and installation technologies, theoretical calculations of the energy use are necessary. There exists a whole set of useful methods presented as ISO and CEN standards. To meet the requirements of an energy declaration, ISO and CEN should be requested to modify these methods to fit more current practices. The difference between the corrected measured values and the theoretical calculated values would depend on bad operation of the building services or wrongly inputted data in the calculation models or bad technical performance of the building. This information can be used as a basis for decisions on necessary improvements of the building. Theoretical calculations shall be used to predict the energy reductions due to the different

4. Energy Declaration of New Buildings – A Possible Procedure

The proposed energy declaration of a new building has to be based primarily on calculations. In comparison with existing buildings, the knowledge of the building construction is very good. The most critical part for the outcome of calculations is the choice of input-data. To achieve good comparability, a common procedure to determine input data like energy for domestic hot water, household electricity or electricity for the activity in the building, choice of indoor temperature, electricity for operating the building, energy for lighting etc has to be developed. In many countries many new buildings have very low energy demand for heating, particularly in Middle East countries. The solar heat gain, internal gains and energy losses from equipment etc cover the major part of the heating demand (or should be utilized for that effect). In several buildings the internal gains etc... are so large that cooling is necessary even in temperate climate. For buildings with large glassed areas and/or large internal gains, basic energy calculations need to be performed on hourly basis. Many modern residential apartment blocks, offices, education buildings and restaurants need very little heat supply from the heating system and need very often air-conditioning to get an acceptable thermal comfort.

RELEVANT STANDING COMMITTEES

The Technical Committees of EOS that should be involved in the preparation of the standards comprise:

- Thermal performance of buildings and building components
- Ventilation for buildings
- Light and lighting
- Heating systems in buildings
- Building automation, controls and building management
-

1. Overview of the calculation process

The calculation is based on the characteristics of the building and its installed equipment, and are structured and listed hereafter in three levels:

- Calculation of the building net energy (energy requirements for heating and cooling);
- Calculation of the building delivered energy;
- Calculation of the overall energy performance indicators (primary energy, CO_2 emissions, etc.).

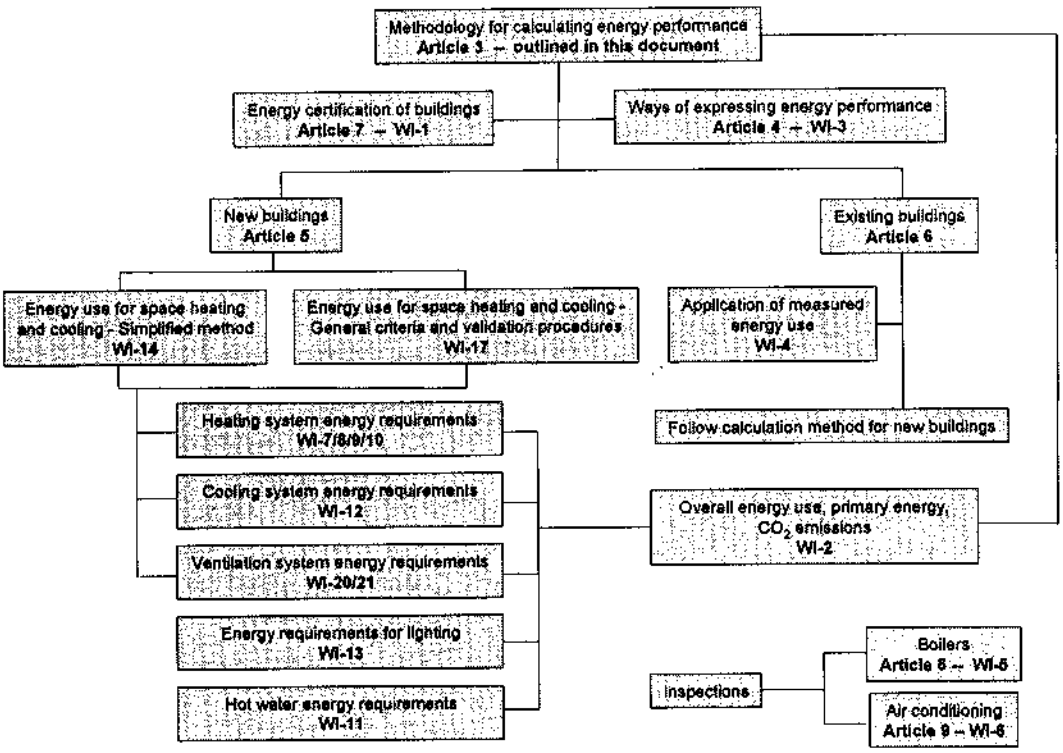

Figure 1: Energy Performance of Building (EPD)[1]

The calculation sequence is:

1) Calculate the building net energy, considering the building properties and not those of the heating/cooling system and results in the net energy use (energy to be given out by heat emitters or to be extracted from the conditioned space, in order to maintain the specified internal temperature). The existing standard EN ISO 13790 covers the calculation of the heating requirement, and the methodology will be extended to also include cooling.

2) Take account of the characteristics of the heating, cooling, domestic hot water and lighting systems, inclusive of controls and building automation, to calculate the delivered energy, using standards. At this stage, energy used for different purposes and by different fuels is recorded separately. The calculations take account of heat emission, distribution, storage and generation, and include the auxiliary energy needed for fans, pump etc.

3) Combine the results from 2) for different purposes and from different fuels to obtain the overall energy use and associated performance indicators, using standards in shown here.

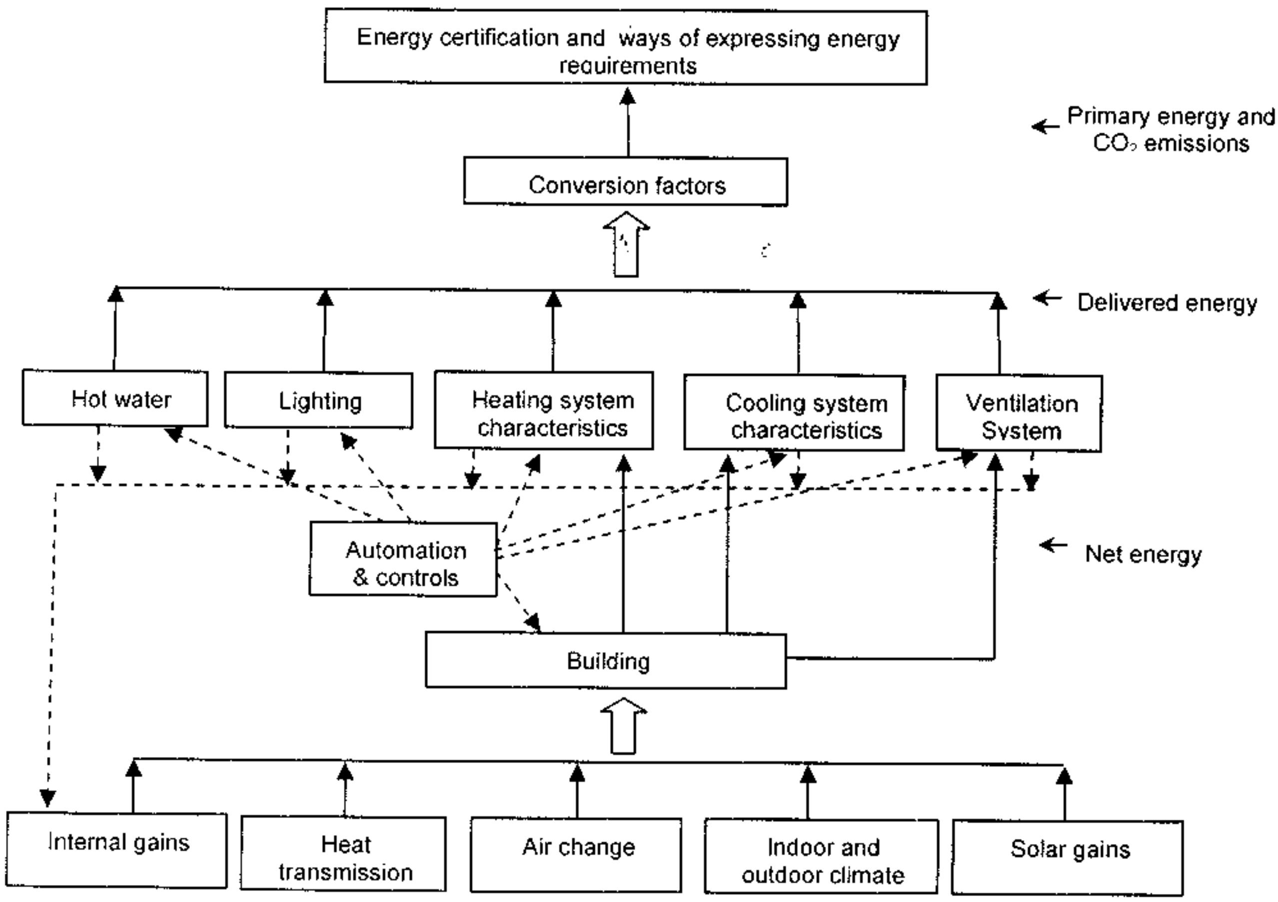

Figure 2: Energy Flow Chart [2]

2. *Summary of Relevant standards*

This describes the role of the main available International standards in Energy performance directive build up.

- ***Standards concerned with the calculation of delivered energy***

Standards in this section provide the link between net energy and delivered energy for space heating and cooling, and also the energy requirements for hot water and lighting. The uses of energy are calculated separately:

1. Space heating –Aspects of heating control systems will generally be incorporated into these standards.
2. Space cooling –Aspects of air-conditioning control systems will generally be incorporated into this standard.
3. Domestic hot water – including both the specification of hot water requirements for different types of building, and the calculation of the energy needed to provide it.
4. Lighting – based on installed lighting power and annualised usage according to building type, occupancy and lighting controls.
5. Integrated building automation and controls – accounting for additional energy optimisation based on interdisciplinary control functions and applications for heating, ventilation, cooling, domestic hot water and lighting.

- ***Standards concerned with calculation of net energy for heating and cooling***

There are two routes to the calculation of the net energy for heating and cooling:

1. Simplified methods based on monthly calculations and simplified description of the building (in terms of element U-values, etc).

2. Detailed numerical calculations. The precise calculation procedure is not specified in the available standards:

These calculations take account of control aspects that affect building gains and losses, such as control of internal temperature, ventilation and solar protection.

- ***Supporting standards***

These standards provide the input data for the calculation of net energy use as follows:

1. Thermal performance of building components

Standards for the calculation of the thermal performance of building components are discussed here. The overall transmission heat loss coefficient is obtained by EN ISO 13789, which refers to other standards for the calculation of U-values. The standards for U-values fall into two groups:

1. simplified methods (EN ISO 6946, EN ISO 13370, EN ISO 10077-1, prEN 13947), which can be used for components within the scope of those standards; and

2. Detailed methods (EN ISO 10077-2, EN ISO 10211-1, EN ISO 10077-2), which can be used as an alternative, or for cases for which there is not an applicable simplified method.

Thermal bridges (at junctions between elements, etc) are covered in EN ISO 10211-1; EN ISO 10211-2 and EN ISO 14683.The standards in this group also include those for obtaining thermal values (EN ISO 10456 and EN 12524).

2. Ventilation and air infiltration

Methods for calculation air flow rates to enable the calculation of heat losses due to air exchange. Standard EN 13779 covers mechanically ventilated buildings (including those with air conditioning).

3. Overheating and solar protection

Including standards for estimating internal temperatures without air-conditioning, and to calculate the effect of solar protection devices. These calculations can be used to determine whether there is a need to consider air conditioning.

4. Indoor conditions and external climate

Containing standards related to indoor conditions and specifications for the calculation and presentation of climatic data. [Note: The parts of prEN ISO 15927 do not contain climatic data, but rather a specification for it, so that data in conformance with this standard is on a known basis and a known format.]

5. Definitions and terminology

EN ISO 7345, EN ISO 9288, EN ISO 9251 and EN 12792 contain definitions of terms and quantities used by other standards.

- ***Standards concerned with monitoring and verification of energy performance***

These standards include the determination of air leakage rates and infrared Thermography, which can be used in the verification of the performance of buildings.

3. Scopes of the standards arranged by standard number

There are three tables[3, 7 & 8]:

EN standards, EN-ISO and New Work items

- **EN standards**

EN	Title and Scope
CR 1752	**Design criteria and the indoor environment** **SCOPE:** This Technical Report specifies the requirements for, and the methods for expressing the quality of the indoor environment for the design, commissioning, operation and control of ventilation and air-conditioning systems.

EN	Title and Scope
EN 12524	**Building materials and products – Hygrothermal properties – Tabulated design values** **SCOPE:** This standard gives design data in tabular form for heat and moisture transfer calculations, for thermally homogeneous materials and products commonly used in building construction.
EN 12599	**Ventilation for buildings – Test procedures and measuring methods for handing over installed ventilation and air conditioning systems** **SCOPE:** This European Standard specifies checks, test methods and measuring instruments in order to verify the fitness for purpose of the installed systems at the stage of handing over.
EN 12792	**Ventilation for buildings – Symbols, terminology and graphical symbols** **SCOPE:** This European Standard comprises the symbols and terminology included in the European standards covering 'Ventilation for buildings' produced by CEN/ TC156.
EN 13187	**Thermal performance of buildings – Qualitative detection of thermal irregularities in building envelopes – Infrared method** **SCOPE:** This standard specifies a qualitative method, by thermographic examination, for detecting thermal irregularities in building envelopes.
EN 13363-1	**Solar protection devices combined with glazing – Calculation of solar and light transmittance - Part 1: Simplified method** **SCOPE:** This standard specifies a simplified method based on the usually known data of the glazing, the U- and g-value and the transmittance and reflectance of the blind.
prEN 13363-2	**Solar protection devices combined with glazing – Calculation of solar and light transmittance - Part 2: Detailed calculation method** **SCOPE:** This standard specifies a detailed method, based on the spectral transmission data of the materials comprising the solar protection devices and the glazing, to determine the total solar energy transmittance and other relevant solar-optical data of the combination.
prEN 13465	**Ventilation for buildings – Calculation methods for the determination of air flow rates in dwellings** **SCOPE:** This European Standard specifies methods to calculate air flow rates for single family houses and individual apartments up to the size of 1000 m^3.

EN	Title and Scope
prEN 13779	**Ventilation for non residential buildings - Performance requirements for ventilation and room conditioning systems** **SCOPE**: This European Standard applies to the design of ventilation and room conditioning systems for non-residential buildings subject to human occupancy. It focuses on the definitions of the various parameters that are relevant for such systems.
prEN 13791	**Thermal performance of buildings – Calculation of internal temperatures of a room in summer without mechanical cooling – General criteria and validation procedures** **SCOPE:** This standard specifies the assumptions, boundary conditions, equations and validation tests for a calculation procedure, under transient hourly conditions, of the internal temperatures (air and operative) during the warm period, of a single room without any cooling/heating equipment in operation.
prEN 13792	**Thermal performance of buildings – Calculation of internal temperatures of a room in summer without mechanical cooling – Simplified methods** **SCOPE:** This standard specifies the required input data for simplified calculation methods for determining the maximum, average and minimum daily values of the operative temperature of a room in the warm period
prEN 13779	**Ventilation for non residential buildings-performance requirements for ventilation and room conditioning systems** **SCOPE:** This document applies to the design of ventilation, air-conditioning and room-conditioning for non-residential buildings subject to human occupancy.
EN 13829	**Thermal performance of buildings – Determination of air permeability of buildings – Fan pressurization method (ISO 9972:1996, modified)** **SCOPE:** This standard is intended for the measurement of the air permeability of buildings or parts of buildings in the field.
prEN 13947	**Thermal performance of curtain walling – Calculation of thermal transmittance – Simplified method** **SCOPE:** This standard specifies a method for calculation of the thermal transmittance of a curtain walling that consists of glazed and/or opaque panels fitted in frames or sashes, without shutters.

EN	Title and Scope
prEN 14335	**Heating systems in buildings - Method for calculation of system energy requirements and system efficiencies – Part 1: General** **SCOPE:** This standard outlines the method and structure for calculation of the energy requirements of the heating system to meet a certain building heating energy demand.

- ## EN ISO standards

EN ISO	Title and Scope
EN ISO 6946	**Building components and building elements – Thermal resistance and thermal transmittance – Calculation method** **SCOPE:** This standard gives the method of calculation of the thermal resistance and thermal transmittance of building components and building elements, excluding doors, windows and other glazed units, components which involve heat transfer to the ground, and components through which air is designed to permeate.
EN ISO 7345	**Thermal insulation – Physical quantities and definitions** **SCOPE:** This International defines physical quantities used in the field of thermal insulation, and gives the corresponding symbols and units.
EN ISO 9288	**Thermal insulation – Heat transfer by radiation – Physical quantities and definitions** **SCOPE:** This international standard defines physical quantities and other terms in the filed of thermal insulation relating to heat transfer by radiation.
EN ISO 9251	**Thermal insulation – Heat transfer conditions and properties of materials – Vocabulary** **SCOPE:** This international standard defines terms used in the field of thermal insulation to describe heat transfer conditions and properties of materials.
EN ISO 10077-1	**Thermal performance of windows, doors and shutters – Calculation of thermal transmittance – Part 1: Simplified method** **SCOPE:** This standard specifies methods for the calculation of the thermal transmittance of windows and doors consisting of glazed or opaque panels fitted in a frame, with and without shutters.

EN ISO	Title and Scope
EN ISO 10077-2	**Thermal performance of windows, doors and shutters – Calculation of thermal transmittance – Part 2: Numerical method for frames** **SCOPE:** This standard specifies a method and gives the material data required for the calculation of the thermal transmittance of vertical frame profiles, and the linear thermal transmittance.
EN ISO 10211-1	**Thermal bridges in building construction – Calculation of heat flows and surface temperatures – Part 1: General methods** **SCOPE:** Part 1 of this standard sets out the specifications on a 3-D and 2-D geometrical model of a thermal bridge for the numerical calculation of: - heat flows in order to assess the overall heat loss from a building; - minimum surface temperatures in order to assess the risk of surface condensation.
EN ISO 10211-2	**Thermal bridges in building construction – Calculation of heat flows and surface temperatures – Part 2: Linear thermal bridges** **SCOPE:** This part 2 of the standard gives the specifications for a two-dimensional geometrical model of a linear thermal bridge for the numerical calculation of: - the linear thermal transmittance of the linear thermal bridge; - the lower limit of the minimum surface temperatures.
EN ISO 10456	**Building materials and products – Procedures for determining declared and design thermal values** **SCOPE:** This International Standard specifies methods for the determination of declared and design thermal values for thermally homogeneous building materials and products.
EN ISO 12569	**Thermal performance of buildings – Determination of air change in buildings – Tracer gas dilution method (ISO 12569:2000)** **SCOPE:** This International Standard describes the use of tracer gas dilution for determining the air change in a single zone as induced by weather conditions or mechanical ventilation.
EN ISO 13370	**Thermal performance of buildings – Heat transfer via the ground – Calculation methods** **SCOPE:** This standard gives methods of calculation of heat transfer coefficients and heat flow rates, for building elements in thermal contact with the ground, including slab-on-ground floors, suspended floors and basements.

EN ISO	Title and Scope
EN ISO 13786	**Thermal performance of building components – Dynamic thermal characteristics – Calculation methods** **SCOPE:** This standard specifies the characteristics related to dynamic thermal behaviour of complete building components and gives methods for their calculation.
EN ISO 13789	**Thermal performance of buildings – Transmission heat loss coefficient – Calculation method** **SCOPE:** This standard specifies a method and provides conventions for the calculation of the steady-state transmission and ventilation heat transfer coefficients of whole buildings and parts of buildings.
EN ISO 13790	**Thermal performance of buildings – Calculation of energy use for space heating** **SCOPE:** This standard gives a simplified calculation method for assessment of the annual energy use for space heating of a residential or a non-residential building, or a part of it, which will be referred to as "the building".
EN ISO 14683	**Thermal bridges in building construction – Linear thermal transmittance – Simplified methods and default values** **SCOPE:** This standard deals with simplified methods for determining heat flows through linear thermal bridges which occur at junctions of building elements
prEN ISO 15927-1	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 1: Monthly and annual means of single meteorological elements** **SCOPE:** This standard specifies procedures for calculating and presenting the monthly means of those parameters of climatic data needed to assess some aspects of the thermal and moisture performance of buildings.
prEN ISO 15927-2	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 2: Data for design cooling loads and risk of overheating** **SCOPE:** This standard gives the definition and specifies methods of calculation and presentation of the monthly external design climate to be used in determining the design cooling load of buildings.

EN ISO	Title and Scope
prEN ISO 15927-3	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 3: Calculation of a driving rain index for vertical surfaces from hourly wind and rain data** **SCOPE:** This standard specifies a procedure for analysing hourly rainfall and wind data derived from meteorological observations so as to provide an estimate of the quantity of water likely to impact on a wall of any given orientation.
prEN ISO 15927-4	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 4: Data for assessing the annual energy for heating and cooling** **SCOPE:** This standard specifies a method for constructing a reference year of hourly values of appropriate meteorological data suitable for assessing the average annual energy for cooling and heating.
prEN ISO 15927-5	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 5: Winter external design air temperatures and related wind data** **SCOPE:** This standard specifies the definition, method of calculation and method of presentation of the climatic data to be used in determining the design heat load for space heating in buildings.
prEN ISO 15927-6	**Hygrothermal performance of buildings – Calculation and presentation of climatic data – Part 6: Accumulated temperature differences (degree days) for assessing energy use in space heating** **SCOPE:** This standard specifies the definition, method of computation and method of presentation of data on accumulated temperature differences, used for assessing the energy used for space heating in buildings.

- ## New Work items

Note: The scopes given in this set indicate the intended scope of the work item. Scopes for the actual standards will be developed as part of the drafting process.

Mandate reference	Title
1	**Energy performance of buildings – Energy certification of buildings**
2	**Energy performance of buildings - Overall energy use and primary energy and CO_2 emissions**
3	**Energy performance of buildings – Ways of expressing**

Mandate reference	Title
	energy performance of buildings
4	Energy performance of buildings – Application of calculation of energy use for space heating and cooling to existing buildings using metered energy
5	Energy performance of buildings – Systems and methods for the inspection of boilers and heating systems
6	Energy performance of buildings – Guidelines for the inspection of air-conditioning systems
8	Heating systems in buildings - Method for calculation of system energy requirements and system efficiencies – Part 2.1: Space heating emission systems
9	Heating systems in buildings - Method for calculation of system energy requirements and system efficiencies – Part 2.2: Space heating generation systems
10	Heating systems in buildings - Method for calculation of system energy requirements and system efficiencies – Part 2.3: Space heating distribution systems
11	Heating systems in buildings – Method for calculation of system energy requirements and system efficiencies – Part 3.1 Domestic hot water requirements Part 3.2: Domestic hot water systems
12	Calculation of room temperatures and of load and energy for buildings with room conditioning systems
13	Energy performance of buildings – Energy requirements for lighting (including delighting)
14	Thermal performance of buildings – Calculation of energy use for space heating and cooling – Simplified method
16	Thermal performance of buildings – Sensible room cooling load calculation – General criteria and validation procedures
17	Energy performance of buildings – Calculation of energy use for space heating and cooling – General criteria and validation procedures
19	Ventilation for buildings - Calculation methods for the determination of air flow rates in buildings including infiltration
20	Ventilation for buildings - Calculation methods for energy requirements due to ventilation systems in buildings

Mandate reference	Title
21	**Ventilation for buildings - Calculation methods for energy losses due to ventilation and infiltration in dwellings**
22	**Calculation methods for energy efficiency improvements by the application of integrated building automation systems**
26	**Design of embedded water based surface heating and cooling systems, to facilitate renewable low temperature heating and high temperature cooling**
29	**Data requirements for standard economic evaluation procedures, including for renewable energy sources**
30	**Energy performance of buildings – Guidelines for the inspection of ventilation systems**
31	**Specification of criteria for the internal environment (thermal, lighting, indoor air quality)**

CONCLUSIONS

From the above analyses, one may conclude the importance of incorporating an energy performance directive as a Standard in Egypt such a goal will aid energy savings in large buildings and set regulations to energy efficient designs that are based on Standard calculation methods. The proposed Egyptian Standard would be largely based on International Standards and appropriately modified to suit local practices .The proposal is basically to:

1. Develop standardised tools for the calculation of the energy performance of buildings
2. Define system boundaries for the different building categories and different heating systems
3. Prepare models for expressing requirements on indoor air quality, thermal comfort in winter and when appropriate in summer, visual comfort, etc.
4. Develop transparent systems to determine necessary input data for the calculations, incl. default values on internal gains
5. Provide transparent information regarding output data (reference values, benchmarks, etc.)
6. Define comparable energy related key values (kWh/m^2, kWh per person, kWh per apartment, kWh per produced unit etc.) The areas/volumes need to be defined.
7. Develop a method to translate net energy, used in the building, to primary energy and CO_2 emissions
8. Develop a common procedure for an "energy performance certificate"
9. Develop and compile relevant standards applicable for each individual building category

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5. prEN 13363-1, Solar protection devices combined with glazing – Calculation of solar energy and light transmittance – Part 1: Simplified method.
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ENERGY PERFORMANCE OF BUILDING ENVELOPE IN EGYPT

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ABSTRACT: The incremental growth in the demand for energy consumption, especially in buildings, leads to a considerable problem that most of developing countries have faced during the last few years. This study investigates theoretically and experimentally the effect of modern technology used by the building envelope on energy conservation in the building sector under the effect of 3 different climatic conditions in Egypt: Alexandria, Cairo and Aswan. These represent the climates of the Mediterranean Sea, the hot-dry and the very hot-dry regions respectively.
The development of building technology during the19th and 20th centuries is discussed to determine the effect of external envelope technology on energy conservation in buildings. Principles of energy conservation and different theoretical methods for evaluation of thermal performance of building envelope were examined through this study. The effects of orientation, building shape, window to wall ratio, glazing type and building materials on thermal performance were discussed. Also the study takes into consideration the different strategies used in different climatic zones, especially the hot-dry zone. A new technique called overall thermal transfer value OTTV was developed and used to evaluate the efficiency of the building envelope in the case of conditioned buildings.
The results of this investigation show that for the development of building energy technology and the emerging new trends, it is essentiality to take into consideration thermal performance efficiency and the compatibility of the building with the environment. Our study showed that from the calculation of the overall thermal transfer value we can evaluate the energy performance of buildings. Thermal factors requirements for designing and constructing the external envelope for conditioned and unconditioned buildings are orientation, R-value, shading factor and solar heat gain.

INTRODUCTION

The industrial revolution during the early of 19th century in Europe brought about the advent of new building materials and new technology. All these developments were accompanied by the appearance of new styles in architecture and new architecture schools.

The new materials such as steel, concrete and new types of glass that appeared during this period all contributed to an increase in energy consumption in the building sector [1].

The fuel crises of 1973 make the world dwell on what international architecture were producing and its impact on energy consumption and on the environment. New groups of architects called for the essentiality of considering local and environmental aspects. During the late 1980s and early 1990s, major advancements in building automation opened the way to construct new buildings more responsive to their environment [2].

The use of new building technologies leds to an increase in the rates of energy consumption in building and this was due to excess use of glass and steel; reduction in the use of insulation materials in hot arid areas, unplanned use of air conditioning systems and the indiscriminate transfer of western technologies without their being localized [1].

Building can consume a large percentage of the total energy used in a country (e.g. more than 20 % in Egypt) owing to increasing energy costs and the decline in the design of energy efficient buildings. The ultimate energy efficient building is a completely passive building where an acceptable indoor environment is achieved by natural ventilation, natural lighting and a good passive thermal performance. Many authors have studied the effect of new envelope building technology on the energy performance of buildings. Glazing and shading provide alternate means of regulating the amount of passive gain in buildings, thus helping to reduce the consumption of conventional energy for heating, cooling, ventilation and lighting. Thermal insulation technology of walls and roofs has improved the thermal performance of the indoor climate, thus helping also to reduce the consumption of conventional energy for HVAC. The thermal design of buildings depends mainly on the (i) outdoor climate, (ii) indoor thermal conditions and (iii) design of building elements such as walls, roofs, etc, for a given climatic conditions [3].

Thermal performance characteristics and energy conservation strategies in hot-dry region

The main thermal performance characteristics of building in hot-dry region is affected by the rate of indoor heating during daytime in summer, increasing the rate of indoor cooling in summer evenings, minimizing dust penetration, good ventilation in the summer evenings and a higher indoor temperature, relative to outdoors, in winter.

One of the problems faced is that some of these requirements are conflicting, such as the desire to have indoor temperatures lower than the outdoor average in summer and higher than the outdoors' in winter, The different performance characteristics also call for conflicting architectural and structural details of the building .

Energy conservation for summer cooling and winter heating can be achieved in desert regions by combinations of the following strategies: lowering the indoor temperature, natural ventilation, minimizing heat gain and loss when air conditioning is unavoidable, and utilization of natural energy for heating and cooling [4].

THEORETICAL BACK GROUND OF THE PROBLEM

Thermal characteristics of walls and roofs

First, the issue of orientation in hot-dry regions is concerned mainly with the windows and other glazed areas. The heating effect of solar radiation impinging on walls can easily be minimized by choosing reflective (very light) colors for the walls. Specific issues of window design in hot-dry regions are proposed. Small windows are more suitable than larges ones for deserts. With special design, however, large windows can be provided in these regions with advantages from the thermal point of view. When highly-insulated shutters are added to large, operable windows, the thermal effect can be adjusted to varying needs, both diurnally and annually. In summer, the shutters can be closed during the hot hours, admitting light into the house. In the evening, the shutters and windows can be opened, to increasie the rate of cooling of the interior. Recommended thermal resistance (R_{req}) of building materials for walls and roofs is calculated according the following equations [4].

i) for walls

$$R_{req} = 0.05\,(T_{(ao)max} - 27) + 0.002\,(\alpha\, I_{max}) \qquad (1)$$

ii) for roofs

$$R_{req} = 0.05\,(T_{(ao)max} - 27) + 0.003\,(\alpha\, I_{max}) \qquad (2)$$

Where: $T_{(ao)max}$ is the maximum outdoor air temperature during the summer season, α is absorptivity of surface and I_{max} is the maximum solar energy on the surface. Also, the heat capacity (Q_{req}) of building can be specified by the following approximate formula

i) for walls

$$Q_{req} = 2.5\,(T_{aomax} - T_{aomin}) + 0.1\,(\alpha\, I_{max}) \qquad (3)$$

ii) for roofs

$$Q_{req} = 2.5\,(T_{aomax} - T_{aomin}) + 0.15\,(\alpha\, I_{max}) \qquad (4)$$

Overall thermal transfer value

Energy conservation for summer cooling in conditioning building is affected by the overall thermal transfer value (OTTV) of building envelope. This value consists of three parts, the heat transfer by conduction through the opaque part of envelope, heat transfer by conduction through transparent part, and the heat transfer by solar radiation penetrated through the transparent part. The total OTTV of walls or roofs is given by [5]:

$$OTTV = [\alpha A*(1-WWR)*U_w*T_{deq}+ A*WWR*U_F\,\Delta T+A*SF*CF(1-SGR)*(1-WWR)*SC/A)] \qquad (5)$$

where A is the area of wall or roof, α is absorptiveity, WWR is the window to wall ratio, T_{deq} is the equivalent temperature, U_W and U_F are the thermal transmittance of opaque and transparent parts respectively, SF is the solar factor, SGR is shading opening ratio and SC is the shading coefficient of window or sky light.

Simulation program energy performance programs

The choice of building sections from the point of view thermal consideration is based on a rational parameter. The parameters in vogue are (i) overall heat transmission coefficient, U, (ii) thermal time constant Q/U and damping D, where Q is the thermal storage. (iii) M-factor and (iv) dimensionless parameter $\omega L^2/k$ where ω is frequency (rad/s), L the thickness (m) and k the thermal diffusivity (m^2/s) [6, 7].

In hot arid zones, the thermal problem becomes more complicated due to the large diurnal swings of temperature and high intensity of solar radiation. Under these conditions the steady state property viz. and , overall heat transmission coefficient U, alone cannot be, satisfactory basis. The storage effects of thick building sections are significant and hence cannot be ignored. The thermal time constant Q/U, and damping factor D are based on empirical relationships and fined limited applications. These parameters though take into account thermal capacity, through simulation programs.

Building energy performance simulation programs such as pss-6, Energy + and DOE-2 neglect thermal comfort [7]. The simulation programs assume that the HVAC system provides comfort as long as the air temperature is maintained as specified by the program user. However, it is well known that comfort depends on other variables that are not taken into account by simulation programs. The factors that affect thermal comfort such as mean radiant temperature and wind speed are not considered in calculations. They are used successively in the case of comparison between energy performance rather than energy predictions. The energy performance of building was modeled hourly using visual Doe-2 building simulation program. Parametric runs were performed to examine the effects of orientation, building materials, window to wall ratio and glass type [8, 9].

Although building energy performance simulation programs are used all over the world, there is still a need for the development of such programs in Egypt.

RESULTS AND DISCUSSION

Examples of new technology

Example 1: Renewable Energy Institute

Figure 1-shows the southern facade of the renewable energy institute building. To minimize the amount of solar energy reached to southern facade during summer season. The designers made a recess in the building lines from top to bottom to let the sun's rays penetrate inside in winter and prevent it in summer time. In addition to the recessed building lines, In the western façade the designer used vertical and horizontal shading devices around the openings to minimize solar energy and maximize shading area. Also, cooling ventilation system is used through the openings made at the top and bottom of windows to let air flow over the exposed glass to increase the external surface heat transfer coefficient, and thus improve the external loss from the windows. To minimize the heat stress gained from roofs, a ventilated double ceiling was constructed [1].

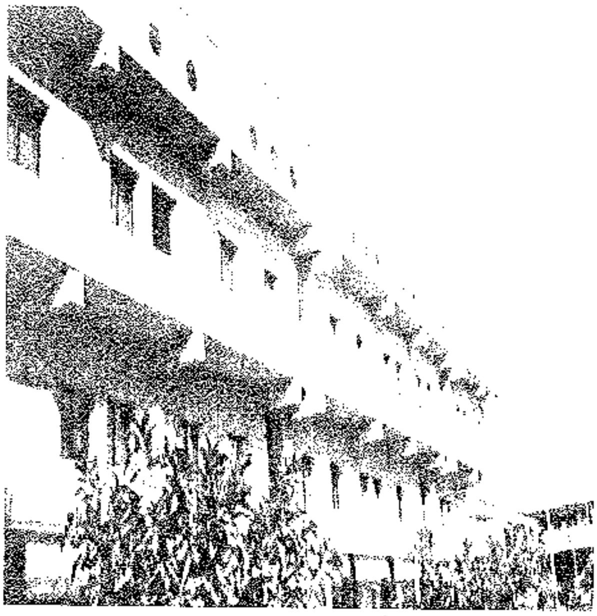
Example 2 : Nile City Tower

Nile City Tower is one of the modern buildings located in Cairo Egypt facing Nile River. New building technologies were used in the envelope to control the effect of external climatic conditions on indoor climate. Thick walls of about 38.5 cm of hollow cement brick, with a 20 cm air cavity were constructed. The thermal conductivity of this brick is 1.7 W/m k. Thermal insulation was added to the walls so that thermal resistance reaches 2 m^2k/W. The double selective glazing used has thermal transmittance (U-value) of about 2.2 W/m^2k and shading coefficient of about 0.2 (Figure 2) [1].

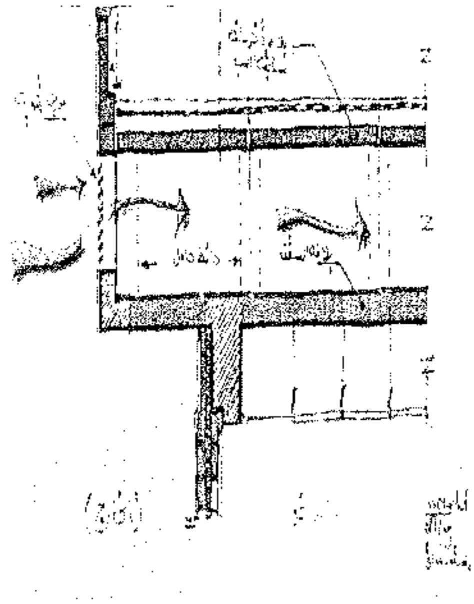
Example 3 Toshky Testing Model by HBRC

The observations and results of the Housing Building Research center team emerged from many attempts which were carried out to built a building suitable for the climates of Toshky region. All these attempts used the sandstone and non – burning bricks as the bearing walls. Domes and vaults used in the roof. The studies of this building show that, uncomfortable indoor climate conditions observed during the night hours in summer season areobserved. The Team work studied the characteristics of available local building materials in Toshky region. The study concluded that thermal insulation must be added to the local building materials to improve the physical thermal properties of the materials. The team is developed new type of lika brick which has a U-value less than 0.3 W/m^2k. New design is proposed and developed for the test building under consideration using some desert construction elements such as air catch with wet surface and fogger air, courts, domes and vaults. A photograph of the final view of the test building in Toshky region is shown in Figure 3.a. The diurnal variation of outdoor and indoor air temperatures from 18 to 21 July 2002 is shown in Figure 3.b. The results of this investigation show that wet surface and fogger air catch helps building to be in the comfortable zone during day and night time [1].

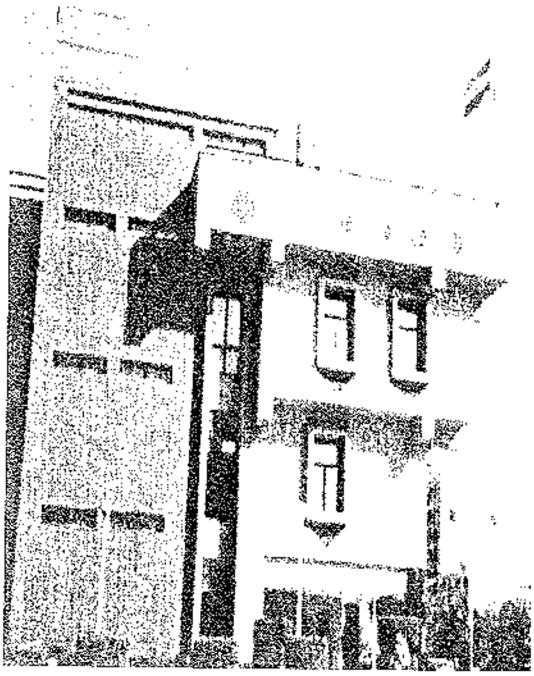
Discussions of the three examples show that with technology transfer there is a need to develop and to adapt the new technology according to the local conditions. In addition, the investigation of the above examples demonstrated the need to find tools to examine the suitability of new technology to the climate.



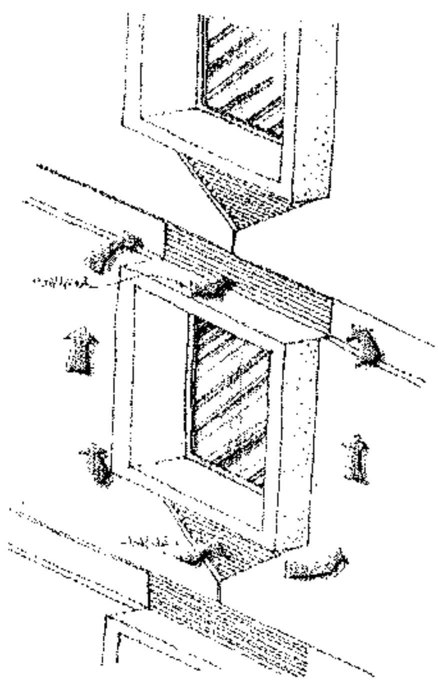
a) Southern facade



b) Roof

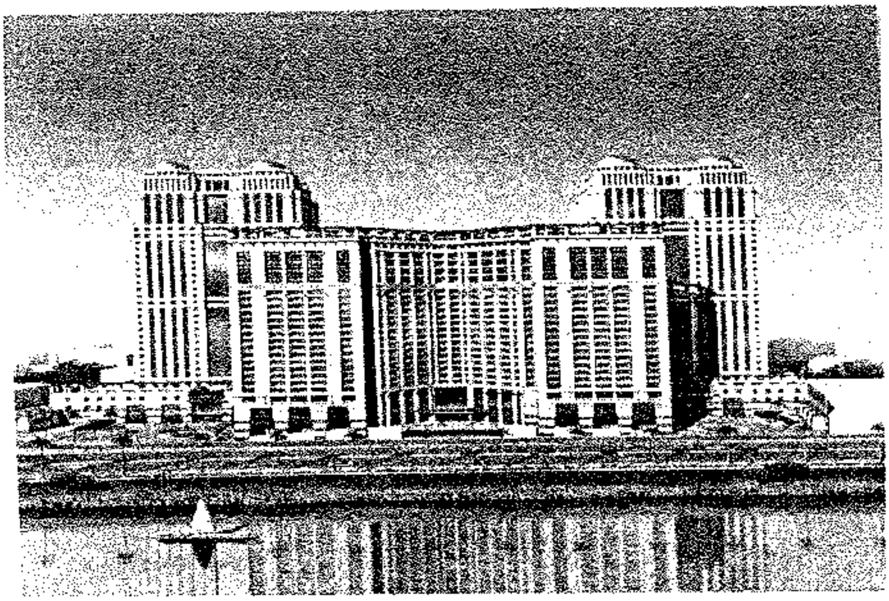


c) Western façade

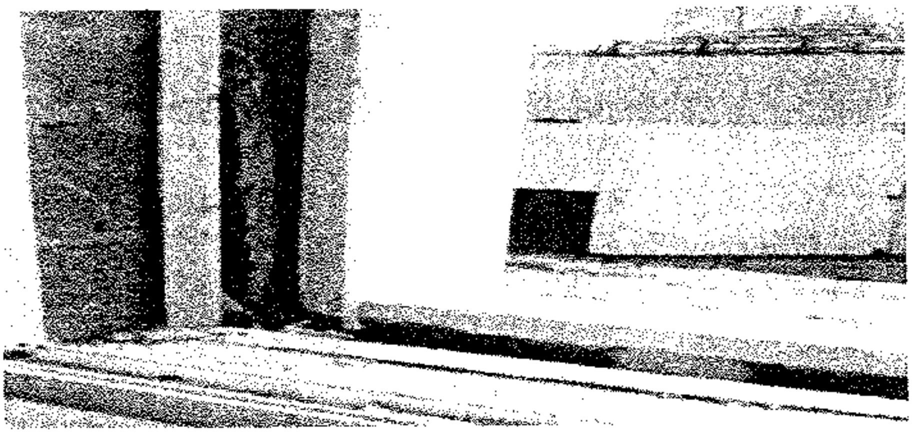


d) Window

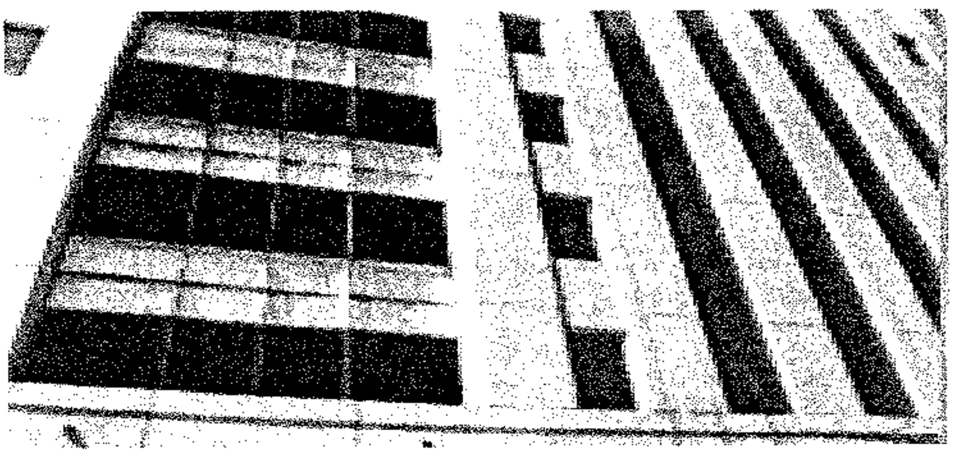
Figure 1. Different Facades Of The Renewable Energy Institute.



a) Main elevation



b) Windows



c) Reflected insulation materials on the window
Figure 2 : Nile City Tower

Figure 3a. Test House Built By HBRC In Toshky Region

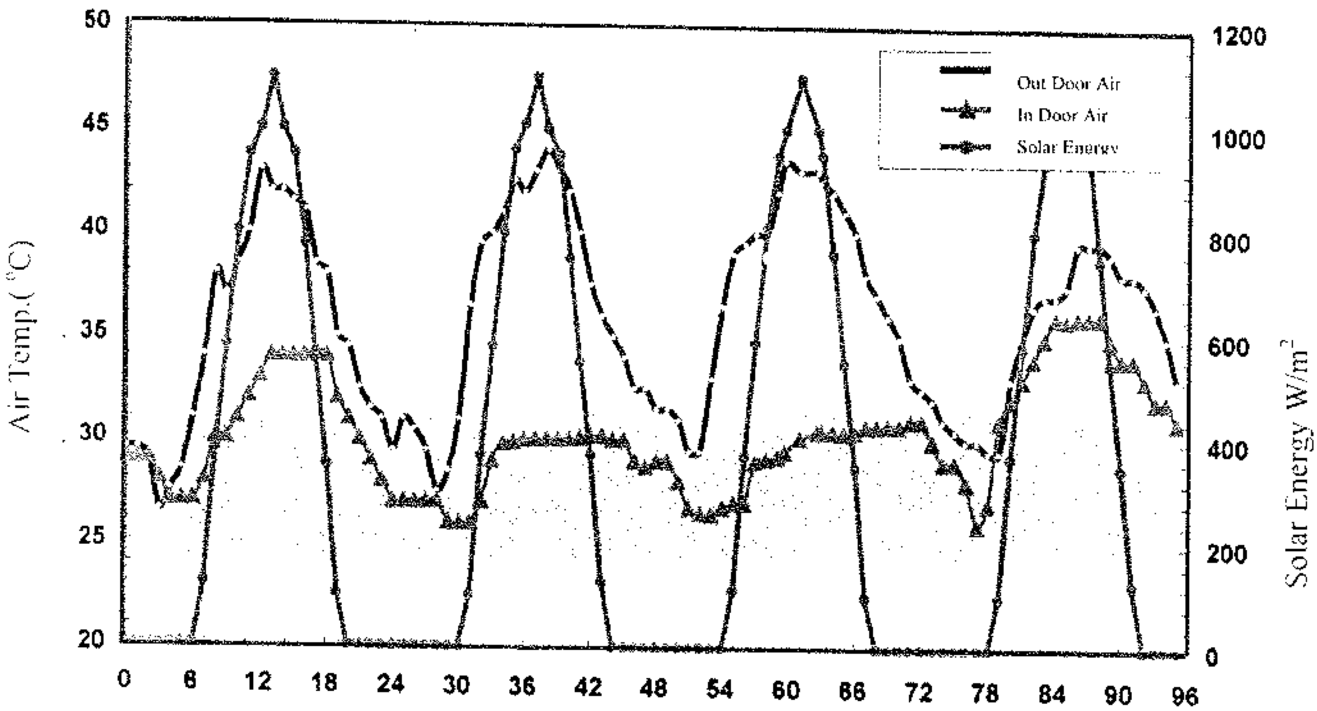


Figure 3b. Diurnal Variation Of Indoor And Outdoor Air Temperature Of Test House Under The External Climatic Conditions During July 2002

Thermal characteristics of envelope

Table 1 shows the external climatic conditions and the thermal resistance required for six climatic zone covering Egypt. The table also illustrates the required thermal resistance of the walls in different orientations increases as the latitude decreases.
Also, Alexandria, Cairo and Aswan cities have a required thermal resistance of 0.44, 0.87 and 1.2 m^2K/W respectively for north orientation when the external color in between light and dark. The colors of external surfaces of walls and roofs play a significant role in decreasing the thermal resistance of construction elements. Also required thermal resistance of the external roofs increased as the latitude decrease. This increasing in the required thermal resistance means that, thermal insulation must be added to the external roofs for the region from Cairo to Aswan. Also the table illustrated that, East and west orientations have a large required thermal resistance compared with the other orientations.
Table 2 illustrates the OTTV for different opening ratio, different shading ratio and different shading coefficient factor under the eternal climatic conditions of three cities in Egypt, namely Alexandria, Cairo and Aswan respectively.

CONCLUSIONS

The above discussion is mainly concerned with unconditioned and conditioned buildings. The following conclusions are listed.

1) Most of the required thermal resistance of building envelope elements tabulated in Table 1 is recommended by the Egyptian Energy Code.
2) In Aswan city the required thermal resistance of the building envelope is not enough and this value need modification.
3) The OTTV method is considered a reasonable method to evaluate the thermal energy performance of conditioned building.

Table 1.Climatic Conditions And Thermal Resistance Required For Six Climatic Zones Covered Egypt.

Region	City & latitude	Max. air temp.	Maximum solar energy						Absorbtivity index	Thermal resistance required (m^2k/W)					
			E.	N. E.	E. W.	S.W.S .N.	S	Roof		N	N. E.	E. W	S.W. .S. N.	S.	Roof
1	Saint Catherine 28° 36`	30	154	532	645	640	623	950	0.38	0.36	0.56	0.74	0.74	0.73	1.4
									0.5	0.4	0.78	0.89	0.89	0.88	1.5
									0.7	0.47	0.99	0.9	1.15	0.89	2.4
2	Alexandria 31°12`	30.6	155	529	645	639	641	950	0.38	0.39	0.68	0.77	0.77	0.77	1.45
									0.5	0.44	0.8	0.93	0.93	0.92	0.82
									0.7	0.5	1	1.2	1.18	1.18	2.4
3	Hurghada 27°12`	35	167	538	645	631	620	1028	0.38	0.63	0.91	0.99	0.98	0.97	1.6
									0.5	0.67	1	1.15	1.13	1.12	2
									0.7	0.73	1.3	1.4	1.38	1.38	2.7
4	Cairo 29°52`	35	155	531	645	635	640	1000	0.38	0.62	0.9	0.99	0.98	0.9	1.7
									0.5	0.66	1	1.14	1.13	1.14	2
									0.7	0.72	1.2	1.4	0.89	1.4	2.6
5	Assuit 27° 9`	40	164	536	645	632	625	1026	0.38	0.87	1.2	1.2	1.2	1.2	1.8
									0.5	1.1	1.3	1.4	1.4	1.4	2.2
									0.7	1.3	1.5	1.7	1.6	1.6	2.8
6	Aswan 24° 6`	45	170	544	6454	470	600	1030	0.38	1.2	1.4	1.5	1.4	1.5	2.2
									0.5	1.1	1.5	1.6	1.5	1.6	2.5
									0.7	1.2	1.8	1.9	1.7	1.8	3.16

* The column No 10 refers to absorptivity index of the external surfaces and it is taken as 0.38 for light color, 0.5 for middle colors and 0.7 for dark colors.
* The six climatic zones are classified according to the climatic considerations and the effect of these conditions on comfort.
* Saint Catherine and Hurghada, have specific external climatic conditions due to the height and nearness to the sea compared with other climatic zone.

Table 2. The OTTV For Different Orientation And Different Opening Ratio (WWR), Different Shading Configuration Shading Coefficient Factor Under The External Climatic Conditions Of Three Cities In Egypt: Alexandria, Cairo And Aswan

Alexandria

ORIENT.	SF	WWR	10%				20%				30%				40%			
			SHADING COEFFICIENT FACTOR															
		0.2	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8
N	148	0.2	13	16	18	20	22	26	31	36	30	37	44	51	38	47	57	66
		0.4	13	14	16	18	20	24	27	31	28	33	39	44	36	43	50	57
		0.8	11	12	13	13	18	19	20	22	24	26	28	30	31	33	36	38
N-E N-W	539	0.2	20	28	37	45	34	51	69	86	48	74	100	126	63	97	132	166
		0.4	17	24	30	37	30	43	56	69	42	61	81	100	54	80	106	132
		0.8	13	15	17	20	21	25	30	34	29	36	42	48	37	46	54	63
E-W	676	0.2	22	32	43	54	38	60	81	103	55	87	119	152	72	115	158	201
		0.4	19	27	35	43	33	49	65	81	47	71	95	119	61	93	125	158
		0.8	14	16	19	22	22	28	33	38	31	39	47	55	39	50	61	72
S-E S-W	440	0.2	18	25	32	39	31	45	59	73	44	65	86	107	57	85	113	141
		0.4	16	21	27	32	27	38	48	59	38	54	70	86	50	71	92	113
		0.8	13	14	16	18	20	24	27	31	28	33	38	44	36	43	50	57
S	191	0.2	14	17	20	23	23	29	35	41	32	41	50	59	41	53	65	77
		0.4	13	15	18	20	21	26	31	35	29	36	43	50	38	47	56	65
		0.8	12	12	13	14	18	20	21	23	25	27	29	32	32	35	38	41

Table 2. (Cont.)

Cairo

ORIENT.	SF	WWR	10%				20%				30%				40%			
			SHADING COEFFICIENT FACTOR															
			0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8
N	162	0.2	21	24	27	29	36	41	47	53	51	60	74	82	67	78	89	100
		0.4	20	22	25	27	34	39	43	47	49	55	68	74	64	72	81	89
		0.8	19	20	20	21	32	33	34	36	45	47	55	57	58	61	64	67
N-E N-W	560	0.2	27	37	46	55	49	67	86	104	71	98	132	160	93	130	167	204
		0.4	25	32	39	46	44	58	72	86	64	85	111	132	83	111	139	167
		0.8	21	23	25	27	35	39	44	49	50	57	70	77	65	74	83	93
E-W	672	0.2	29	40	50	61	52	73	94	116	75	107	145	177	98	141	184	226
		0.4	26	34	42	50	46	62	78	94	67	91	121	145	87	120	152	184
		0.8	21	24	26	29	36	41	46	52	51	59	73	81	66	77	87	98
S-E S-W	405	0.2	20	36	42	27	32	54	66	78	61	78	102	120	79	103	126	150
		0.4	23	27	32	36	39	48	57	66	78	61	78	102	73	91	109	126
		0.8	20	21	23	24	33	36	39	42	47	52	62	67	61	67	73	79
S	164	0.2	20	23	25	27	35	39	43	48	49	56	68	75	64	73	82	90
		0.4	20	21	23	25	33	37	40	43	48	53	63	68	62	68	75	82
		0.8	19	19	20	20	31	32	33	35	44	46	54	55	58	60	62	64

Table 2. (Cont.)

Aswan

ORIENT.	SF	WWR	10%				20%				30%				40%			
			SHADING COEFFICIENT FACTOR															
		0.2	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8	0.2	0.4	0.6	0.8
N	175	0.2	21	24	27	29	36	41	47	53	51	60	74	82	67	78	89	100
		0.4	20	22	25	27	34	39	43	47	49	55	68	74	64	72	81	89
		0.8	19	20	20	21	32	33	34	36	45	47	55	57	58	61	64	67
N-E N-W	580	0.2	27	37	46	55	49	67	86	104	71	98	132	160	93	130	167	204
		0.4	25	32	39	46	44	58	72	86	64	85	111	132	83	111	139	167
		0.8	21	23	25	27	35	39	44	49	50	57	70	77	65	74	83	93
E-W	668	0.2	29	40	50	61	52	73	94	116	75	107	145	177	98	141	184	226
		0.4	26	34	42	50	46	62	78	94	67	91	121	145	87	120	152	184
		0.8	21	24	26	29	36	41	46	52	51	59	73	81	66	77	87	98
S-E S-W	370	0.2	24	30	36	42	32	54	66	78	61	78	102	120	79	103	126	150
		0.4	23	27	32	36	39	48	57	66	56	69	89	102	73	91	109	126
		0.8	20	21	23	24	33	36	39	42	47	52	62	67	61	67	73	79
S	136	0.2	20	23	25	27	35	39	43	48	49	56	68	75	64	73	82	90
		0.4	20	21	23	25	33	37	40	43	48	53	63	68	62	68	75	82
		0.8	19	19	20	20	31	32	33	35	44	46	54	55	58	60	62	64

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ENVIRONMENTAL PROTECTION OF COASTAL URBAN AREAS IN THE EUROPEAN UNION

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ABSTRACT: The growth of cities in the coastal space constitutes a practice that will continue to extend over time. The concentration of cities in the coasts constitutes an age-old phenomenon, since the adjacency with the marine element provided appreciable advantages in all the sectors. However, the coastal space constitutes a separate territorial unit and has a lot of particularities, which differentiate it from all other spaces. It is particularly sensitive and vulnerable, since it is - mainly today - subject to multiple negative anthropogenic interventions with all the repercussions that this can have on its more general environment. The problems of coastal space, on the one hand, are not coincidental in nature, while on the other, can by no means be solved without essential interventions, which will be common for all the countries so as to be effective. All these assessments have matured progressively and have led to the establishment of policies and to the taking of measures by the European Union so as to effective by face the permanently expanding problems of its coastal space. At the same time, it is absolutely natural that any action and intervention carried out with the intention of management of coastal areas in the European Union will subsequent by influence the urban space where these areas of the environment are found. The more general policies of the Union and the programmes that are applied in the coastal regions, aim at achieving a viable development of coastal regions and, in the long run, setting a balance and dealing with the environmental, economic and social issues of coastal cities. The objective of the present paper is to investigate:

- The intensity and the extent of the repercussions of the policies and actions of the European Union in the environment of urban regions and

The extent to which these policies and actions can contribute to the resolution of environmental problems in coastal urban areas.

Keywords: Coastal, Integrated Coastal Zone Management, ICZM

INTRODUCTION

For the last two decades cities in the European Union have been facing new variety of challenges. These challenges are connected with changes associated with the whole spectrum of human activities: from economy and development to culture. Taking for granted that the economic reformation of European cities is associated very closely to intense social and also environmental changes, it is expected that problems will appear - new or old ones in a new form - which are called to be solved. The confrontation of these problems requires coordinated and total actions in order for the results to be positive in all the sectors. In different case, taking measures to confront certain issues may have a negative (or contrary) result on others. For example the development process is often contrary to the protection of the environment. In the work that follows are investigated the repercussions of the policies and actions implemented by the European Union regarding the environment of coastal urban regions and how much these policies and actions can contribute to the resolution of the problems in these regions. As it has already been said, these subjects are faced concertedly, as it is a known fact that interdependence between urban problems exists.

The concerted confrontation of problems in the European Union (EU) constitutes a very important challenge for the European cities: for the citizens of these cities/the local societies, city authorities, but also for the regional and national authorities. Their joint confrontation in the Community level constitutes, also, an important challenge. The bodies of the EU shape and apply suitable, for this aim, directions and policies. Policies regarding coastal cities are basically a subset of policies regarding the coastal zone as a whole[1].

The European Union's interest in the confrontation of problems regarding the coastal zone has come up over the last two decades and it had initially appeared as a series of faint concerns. As years went by, it was made clear that the coastal space constitutes a separate territorial unit and has a lot of particularities that differentiate it from all remainder. This space is particularly sensitive and corruptible, since it is subject to multiple human interventions and all the unfavourable repercussions of these in its general environment. The problems that it faces, on the one hand are not of coincidental nature and on the other, in no case can they be solved without essential interventions, which must be coordinated between all the member states, so that they can also be effective. This ascertainment has matured progressively and has led to the establishment of policies and to the taking of measures by the European Union in order to face the permanently expanding problems of coastal space.

The need for taking measures, as well as, the existence of special planning, progressively led to a specialised European policy for the coastal zone. At an initial stage, most measures of the Community concerned the natural environment alone. These measures were not always strictly oriented in protecting the coastal space, but expanded throughout the whole national territory. However, in many cases their application had important (positive) effects in the coastal zone, since in most cases these areas faced the largest number of environmental problems[1].

Taking for grated that Europe is surrounded by a coastline of approximately 100.000 kilometres and the corresponding continental shelf, we can conceive that in relation with the remainder continents, coastal regions have a particular importance in the life and development of the EU. This observation becomes more interesting, if we contemplate that these coastal zones present a big variety with regard to the natural environment that is found in them and in relation to the stay of people and human activities that are found there. Consequently the interest of bodies and services of European Union turned to the coastal areas of its territory[2,3,4,5].

An important percentage of European citizens, which in certain countries constitutes the majority of their inhabitants, live in coastal regions. Thus, for example in coastal areas in Spain, 30% of all municipalities of the country are found while 54% of the total population of the country also live there. The corresponding percentages for France are 9% and 24%, for Portugal 44% and 73%, for Holland 21% and 75%, for Italy 30% and 60%, Sweden 50% and 78%, Finland 36% and 59%, Ireland 51% and 86%, in Denmark the 70% and the absolute total of population (100%), in the United Kingdom the 14% and 73%. and finally in Greece we observe that the 33% of the country 's population live in coastal Municipalities and Communities, and 85% of the total population live in a coastal zone expanding 50 km from the coast. Of course, the European Committee has characterized these countries - members as coastal or inland depending on the percentage of population that live in coastal municipalities or communities[6].

It is necessary in this point to clarify the way of defining a region as a coastal zone. A coastal zone is defined as a strip of sea and land with a width that varies according to the local topography and management needs. The coastal zone rarely corresponds to already existing administrative or planning subdivisions. Usually the coastal zone in its sea section is limited within a countries territorial waters of 12 miles from the coastline as set by the treaty on the law of the sea, whether they belong to certain biological units, or sufficiently defined management units, or not. The coastal zone within the boundaries of which human activities are connected to the exploitation of coastal resources can be extend farther than the territorial waters at 200 miles from the coastline (Area of exclusive economic exploitation)[7], while the land section of the coastal zone is a fixed strip extending for 50 km, from the coastline, so as to cover the sum of the effects on the environment and on human activities which are related directly or indirectly to the coastal zone[8].

Subjects and reflections relative with the coastal zones

Economics and social data

As it has already been mentioned above, the European coastal area is particularly over-populated, while a lot of and important activities are carried out inside it. The European Committee has stressed its big economic importance for the southern countries - members of Union. Specifically in Greece, most of the big cities are assembled in a narrow coastal band, which maintains above the 60% of the population of the country, while 70% of the industrial activity and the 90% of tourist investments is located within the country's coastal zone. The bodies of European Union have recorded the basic economic issues that appear to constitute a common place in the coastal regions of Union:

- The rapid growth at a higher pace than the remainder regions, which is a cause of problems in the infrastructure and services.

- Similar problems are created by the continuous increase of settling of permanent residents and tourist movement.
- In industrial regions the need for reformation of economic activities is presented, in order to face the decline of traditional industries.
- Delay of results is often noticed in the coastal regions of countries, which are part of the programs for economic and social cohesion.
- Interventions are needed in regions where mass tourism has downgraded the natural environment to an important degree.
- In the same regions there is need for taking measures, which will shield them economically, and socially from the ever-changing character of mass tourism.

The above issues create a mesh of joined problems, which put a burden on the social and economic climate of coastal regions. For example tourism, which is expressed in a mass scale and has financial importance, causes problems that are hard to solve. Thus, job opportunities created are in their majority seasonal, require little or no qualifications and offer respectively low wages[9].

Natural Environmental Issues

Natural Environmental issues regarding the coastal area of the European Union are many and important:

- Large areas of biotopes have been polluted, and wetlands have bee drained in order to be used in agriculture.
- Big parts of coastal areas have been used for accommodation, industrial purposes and tourist development[10].
- The loss of biotopes brings about the reduction of biodiversity.
- The rise of the sea level constitutes a threat to both the biotopes along the coastline and the continuous growth of the mainland[11].
- Because of fires and the spread of urban regions a loss of 200.000 hectares of forest in coastal regions is noticed each year. According to calculations one-quarter of forests located within the boundaries of coastal zones will have been lost by the year 2025.
- The increase of population and prosperity had as a result the reduction of water reserves and the deterioration of water quality. In a lot of coastal regions in the Mediterranean we have observed phenomena of overexploitation of the water resources and salt-water infiltration problems. The problem is also made worse by frequent periods of insistent drought, which are observed in the Mediterranean.
  The situation is also deteriorated by the pollution of coastal waters by land sources (from spilling of nutritious substances (nitrogen and phosphorus) which come from unrefined industrial waste, urban sewages, and agricultural cultures) and marine sources (oil slicks). It is estimated that 80 - 85% of the total level of pollution in the coastal area of the Mediterranean comes from the land[12].
- About 30% of the beaches in the EU undergo erosion (approximately 5.500 kilometres of coasts). In the natural process that takes place via the movement of sediments, the human element interferes unpredictably. Navigation channels, drained wetlands and the constructions of dams in rivers or docks in harbours disturb the normal flow of water and sediments.

- Loss of sources of sand and gravel from excavations of the sea bed, with destructive consequences in the fish habitats or the bio-societies found on the seabed.
- Passage of navigation lines near ecologically sensitive regions.
- Lack of communication and co-ordination between authorities responsible for the coastal regions, resulting in the development and management for each region to constitute a separate case[13].
- Although tourism constitutes a lever of development of coastal zones, it is still a threat on their natural environment because of the housing construction, which continues in order to meet tourist needs in accommodation.

European Union and Coastal Zones

The International Framework - Programs

As early as the 70's and the 80's the European Union had published directives for the protection of the environment and the rare biotopes and the quality of waters (85/337/EEC, 79/409/EEC). In 1992 the Council of Ministers of the Environment requested that the Union prepare a completed strategy for management of coastal zones. This strategy would include the current legislation of the EU, a key point of which were the Directives for the estimation of environmental consequences (85/337 EC). At the same time, it would increase homogeneity in a line of Committee and bodies.

In the following paragraph the evolution of the basic policies of the Union, which contributed to creating the strategy for aggregated management of coastal zones, is being described. As mentioned before, many of these policies have an indirect affect on coastal zones. However their affects were in some cases a determinant factor for the configuration of current policy. Specifically, the European Committee after the approval the map of the European Coasts and on request by the European Parliament (resolution 73424,1981) and Council, pointed out the need for the continuous application of measures for the coastal zones management[18]. Following that, in 1992 the Council suggested that an aggregated community strategy for coastal zones management (resolution 25.02.1992, EE C 59, 06.03.1992) be developed. This strategy will include the current legislation of the European Union in which was contributed by the Directives for the estimate of environmental sequences (85/337 EC). In order to promote this strategy, networks of co-operation aiming at information and knowledge exchange were created (EURISLES, COAST, MEDPLAN). The RESTORE network while dealing with the restorations of coastal cities and regions drew its funding funds from the RECITE programs (regions and cities in Europe) [19]. The motion of the Council for the development community strategy resulted in the fifth community action program for the environment and viable development to call for the undertaking of relative initiatives regarding the coastal region management[20].

It will be an important omission if at this point there was no reference to prior community initiatives, and the report of the Europe 2000 and Europe 2000+ Committees. The report of the Europe 2000 Committee[21] underlined the important spatial discrepancies in the European Union, which refer to many levels such as the demographic, infrastructure, development, and environment etc., things that define many areas, being at a disadvantage. In the report it is clearly shown that coastal areas are disadvantageous and geographically sensitive areas that face important problems, mainly of an environmental nature.

The report of the Europe 2000+ Committee[22] is similar to the previous one but more complete. It refers to the whole of the Community space, but a lot of directives issued regard the coastal areas. Specifically, in this report there is a mention about environmental protection and especially protection of ecologically sensitive areas from over exploitation (the coastal zone being one of those areas). Apart from general directions given that deal with the coastal zone, there are also specific references in special coastal regions such as Islands. This report refers to the need for development of inter- European networks of infrastructure and transportation, which will contribute in enforcing the competitiveness of the Union's island areas. The need for development of communication and information networks so as to attract investments in island areas, which it will aid in their development, is also pointed out. The need for development of natural gas and electric power networks was also mentioned. Alongside with the development of the transportation networks already mentioned the need for large-scale environmental works, combined with and accompanying those network infrastructure works, are also underlined.

The Demonstration Model

The European Union, wishing to maximize the effectiveness of coastal zone management, suggested in 1995 the materialization of a "demonstration model" in order to obtain the necessary experience in these types of issues[23]. The objective of the demonstration model was to describe the current situation and already existing problems, the creation of a model of co-operation, coordination and programming as well as, the spread of new technology. The demonstration model was developed through 35 demonstration projects and 6 thematic studies, which are considered to be node points for the successful management of coastal areas. Already, the characteristics that define the works for the integrated Coastal Zone Management (ICZM)[24] have been pointed out.

The demonstration programme for the ICZM, in a practical level of projects and actions, as well as the final evaluations and conclusions which came up from its implementation was completed in the beginning of the 1999. In the following paragraph, we present a synopsis of projects that were materialized with regard to the financial programs they were included in.

There are financial projects from the LIFE programme for viable tourist development, viable management of coast and dunes, Integrated Coastal Zone Management (Island, Bays, beaches, rivers) in many countries of the European Union[ANPPEDIX 1]. In addition to those, there are also projects from the Terra programme for coastal zone areas and coastal cities [APPENDIX 2] and projects funded by the PHARE programme for the ICZM of coasts in Latvia and Lithuania, as well as a financial project financed by the Norwegian Government and other local management programmes of the Norwegian coasts[25].

The strategy aimed at promoting the objectives of the European Treaty for viable development and the incorporation of environmental aspects in all other policies by means of efficient and coordinated exploitation of existing community means, as well as, through governing according to the European objectives for the time period 2000 - 2005.

The demonstration programme resulted in understanding that integrated solutions for the confrontation of the problems can be determined and materialized only in local and regional level, but the integration of sectorial policies in local and regional level is feasible when the highest levels of administration provide a complete legal and statutory framework and take measures in order to encourage local and regional actions[27].

The basic orientations of Community Policy for coastal areas

The general conclusion reached by the responsible bodies and committees of the EU foresaw that the in depth review of the problems of the European coasts, including the demographic explosion, the development of tourism and the loss of virgin regions and biotopes with high ecological importance, indicates the necessity to set a new course that will lead us to a viable development of European coastal areas[14]. In this scheme the European Committee has included as particularly useful the following factors:

- The need for an aggregated approach in the management of each coastal zone.
- Short-term, as much as, long-range tourism development planning.
- The estimate of the environmental consequences of each action or work that will be located in coastal area.
- The increase, and the level, of information dealing with the coastal areas.
- The characterization of areas as protected regions by legislation.
- The co-operation between the government or community institutions with the mechanisms of the free market.
- The consolidation of land purchase systems.
- The development of multiple networking activities regarding issues related to the coastal zones.
- The principle of collectiveness and understanding in a European level.

It is pointed out that each factor mentioned above plays its role in the management of coasts. It is likely that some factors are more important than others but not one of them can offer complete solutions by itself.

The result was to create an administrative, research and control authority in I European Union level: the Network for Management and Planning of Coastal Regions [15]. According to this authority the term 'management of coastal zones' is defined as the dynamic process during which a pre-determined and coordinated strategy is applied with the purpose of distribution of environmental, social - cultural and legislation resources in order to achieve the conservation and the sustainable multiple use of coastal zones. The conditions for the practice of any effective political action are:

- The existence of an integrated approach towards planning and management.
- The practice for control and interventions on behalf of public authorities.
- The enactment of special authorities in order to face the issues of coastal zone.
- The unhindered co- operation on the international, national and local levels, with a flow of planning from the first two levels to the third [16].

Integrated Coastal Zones Management (ICZM)

In 2000, the European Committee announced to The Council and the European Parliament a strategy on Integrated Coastal Zone Management of the European Union[26]. The results and the recommendations which constituted this final strategy, come as a result of the demonstrate programme, which set a final timetable of up to ten years for its full implementation[33].

The strategy of the European Union on the Integrated Coastal Zone Management includes many separate actions of varying importance, without being simply a list of alternative solutions, but one cohesive beam of measures. The implementation requires the participation and cooperation of partners and different services in the European Union. The programming actions of the European Committee in order to achieve its goals are:

- Promotion of ICZM activities in member states and in the regional sea level.
- Harmonization of sectorial legislation and European Union policies with ICZM.
- Promotion of dialogue among Europeans who are interested in coastal areas.
- Development of the means for the best possible practice of the ICZM.
- Collection of information and knowledge about coastal zone.
- Spread of information and increase in public awareness.

The European Committee committed itself on implementing the strategy regarding the ICZM as soon as possible. The continuous change of the conditions of the coastal area requires continuous adaptation of approximation, which is adopted by European Union for the coastal areas. The Committee will re- examines this strategy, after three years, which in the future will be reviewed according to the evaluation of the condition of the European Environment, which will be held by the European Environmental Organization. In particular, this evaluation includes three stages. First of all, the re – evaluation off all the stages of the implementation of actions and measures, secondly the evaluation of the effects of different problems, and thirdly the progress' analysis of the eradication of human and natural problems.

The objectives of the European authorities on Coastal Zone Management are:

- The wide spherical prospect (thematic and geographic)
- The long-term prospect
- Adaptive management within the frame of one graduated procedure.
- Pointing out local particularity.
- Function in connection with natural activities.
- Participial planning.
- Support and the participation of all responsible administrative services.
- Utilization of a combination of means.

6th Action Programme for the Environment

Based on the results of the European Parliament and Council[27] about the implementation of the Integrated Coastal Zone Management in Europe, for the enactment of the 6th community action programme for the environment 2001 - 2010, the strategy of the coastal zone management relied on:

- The protection of coastal environment based on the approach per ecosystem method, which retains its function and integrity, and on viable natural management for the coastal zone resources.
- Recognition of the dangers in coastal areas from climatic changes, the rise of sea level, the frequency and violence of storms.
- Measures for the protection of coasts, coastal area settlements, and their cultural heritage.
- Providing financial opportunities and the ability of choice of employment.

- The creation of a functional social - cultural system of the local communities.
- Providing areas to the public for entertainment and aesthetic enjoyment.
- The maintenance and promotion of the cohesion between long - distance coastal area.
- The improvement of the co-ordination of actions taken by different authorities in the sea and land aiming at sustaining the interaction between land and sea.

ICZM and Urban Environment

The interaction between coastal zone and city

The presence of urban environment at an important degree in coastal zones of the European Union has been pointed out in the introduction. Therefore, each study on coastal zones conducted, should consider the element of succession of natural and urban environment in coastal zones. The concentration of cities near the sea is a phenomenon dating back to ancient times, since adjacency to the sea provided important advantages. It is not accidental that the total number of cities where important cultural events took place in antiquity were seaside cities (Rome, Athens, Our, Alexandria). Today, large metropolis in North and South Europe, are found near the coast. Athens, Thessalonica, Venice, Naples, Rome, Marseille, Barcelona, Antwerp, Rotterdam, Hamburg, Copenhagen, Gutenberg, Oslo, London, Liverpool and Glasgow are typical examples of cities near the coast. Of course, their location in coastal zones means that any policy and action of the European Union, which is related to these zones, has important effects on the environment of these cities.

On the other side, the European Union includes important islands (Sardinia, Corsica, and Crete, Sicily) and famous group of islands (the Aegean islands, the Ionian Islands, Balearic Islands, islands of Denmark archipelago, Faeroe Islands etc.) In these islands there are residential areas of exceptional interest (big and small cities, villages and settlements), not only the environment but also the image the life style and the organization of which, depends on their geographic coastal location to a large extend.

It is not only the location of coastal cities in their specific geographic space, but also a series of events that play a decisive role in the fashioning of their environment, their organization and in their life. Their cultural and historic past has been connected with their coastal position. Consequently, a series of monuments or cultural activities, customs and traditions present a natural extent of their seaside character. In addition to that, traditional business activities are connected with the sea or the coast. It is not coincidental that the majority of the above mentioned European cities have been imprinted in the minds of people as ports, even if some of them make use of seaports (Athens - Piraeus, Rome - Ostia, London).

It is absolutely natural, that any action and intervention, which takes place aiming at the management of the coastal zones of the European Union, affects respectively the urban environment in these zones.

Coastal Character: Characteristic elements of cities

In the previous unit, we reported indicative elements, which prove the interaction between coast and city. If we want to examine the influence of the coastal type of one city in its general environment, organization, life and character, we will conclude that indeed these are determined up to a particularly important degree by the geographic coastal position[28]. Specifically, the location of a city within the coastal zone is related to almost the whole of activities which take place in the city, as well as, and the shaping of its specific and general morphology:

- The accumulation of population. A leaping increase of the permanent population in the coastal areas in relation to other areas in the Union is being observed.
- The composition of the population. There is not only an inrush of residents from the mainland, but also and increasing settlement of foreign immigrants.
- Transports. The whole of seaside cities have harbours, either by themselves or by the use of seaports, so that trading or mass transportation of persons takes place in great numbers.
- The cultural and historical past. The whole of European coastal cities have a long history indissolubly connected to the sea element. They have a lot of historical monuments and references from their sea – related past.
- Tourism. They take in a large number of tourists each year, who visit them in order to admire their cultural and historical monuments or the nature environment which is found in their periphery, or because they operate as transportation hubs.
- Architectural style. The fact that the seaside cities are near the sea, has played an important role in the configuration of their architectural design as, structural problems related to the location and the climate had to be resolved in addition to achieving satisfactory aesthetic results.
- The general land - urban organization. Streets and squares have clear an orientation to the sea, which is the natural border of city.
- The atmosphere and habits of residents.
- Professional occupations of residents.
- Economic activities.
- Priority given to the manufacture of works and infrastructures (for example Venice, Rotterdam)[29].

The effects of European Policy on coastal cities

We notice that one of the basic elements of the city, the natural environment, can be protected by Integrated Coastal Zone Management programmes. In the area of environmental protection there is a series of directives of the European Community, which contribute to the conservation of fauna and flora, to a point that even urban and suburban regions cal be influenced in a positive way (92/43/EOK, network NATURA 2000). Particular emphasis is given the preservation of the natural habitat of various species of the fauna and the characteristic elements of the flora. The directives 76/160/EOK and 79/923/EOK for the quality of water also have a supplemental effect to the above mentioned actions. Important emphasis is given to the treatment of urban sewages (directives 91/271/EOK, 91/676/EOK), which are channelled into streams and subsequently into coastal waters. This action is supplemented through predictions about agricultural pollutants, as well pollutants emitted to the atmosphere or the incineration of dangerous industrial waste and urban sewages near coastal regions (91/629/EOK, 89/369/EOK). Finally, the directive 85/337/EOK about the estimation of the environment effects of some works contributes to the preservation of the natural environment. The ACNAT - MEDSPA programmes through the financial support of the LIFE programme, move forward to the construction of works aiming at the protection and the management of coastal areas, many of which are located within urban space. Moreover, a European land use digital database has been developed through the CORINE programme so that the Environmental European Organization can evaluate and utilize this data[32]. In addition to that, significant funds were spent on reducing coastal zone pollution under the auspices of the ENVIREG programme[31].

The tourist activity is one of the most important parameters of an urban coastal region, provided that it has a positive affect on its environment and that the ICZM programmes face the issue effectively. Thus, a group of actions which includes the following is proposed:

- Control of the increase of number of visitors based on environmental restrictions and available Potential.
- Involvement of the local society in the process of planning and decision - making.
- Protection of biotopes, ecosystems and quality of waters.
- Constant investments in the infrastructure.
- Land use planning.
- Establishment of channels of information on the developments in the tourist sector.
- Discouragement of tourist activities, which can cause negative social effects.
- Encouragement of the creation of quality tourist products, so as to make the habits and cultural identity of local populations understandable by visitors.

We can examine at this point the directives of the UNEP study on a viable tourism development system in the Island of Rhodes.

- Reduction of the rate of lodging construction, and formation of a strategy for controlled development.
- Definition of "development regions" and conservation regions.
- Imposition of models of land uses regarding density via application of Coastal Zone Management.
- Constant development of employment policies.
- Motives of differentiation regarding tourism [34].

We observe that these interventions and actions provide the capability to the city, not only to preserve its economic tourist activities, but to set the basis for long-term development regarding these activities. On the other side, urban units with traditional colour (island), protect their particularities and educate their visitors. Thus, the visitors are in the position to understand and respect the traditional coastal urban environment. The restoration of coastal and entertainment areas which had been destroyed by the development and land planning in the first post-war decades is now feasible. Given the opportunity presented with the organisation of major events (Olympics Games of Barcelona, Olympics Games of Athens) there can be a reconnection between the city and sea with its elements. The RESTORE programme contributes to the reformation of the seaside urban centres and the re-establishment of their environmental quality. We can move towards protecting or restoring traditional resorts in a coastal urban environment because of changes in the tourist market or residential downgrading. Specifically, this programme focuses on intermediate and small size businesses, contributing thus in the simultaneous re-establishment of the particular characteristics, of a social and economic nature, of the area.

CONCLUSIONS

To sum up, we are led to the conclusion that the general policy of the European Union and the programmes that it applies in coastal regions, among others, attempt to counterbalance the environmental, economical and social issues of the coastal areas, restore the quality of the environment of the city and use different sectorial policies in the most effective way. Consequently, they contribute to upgrading and restoring the environment of coastal cities, while at the same time are used as a database and a source of experience in the elaboration of integrated management of urban environment programmes. In order to achieve more effective coastal zone management, including urban areas, (apart from the actions of the European Union and sectorial policies) the re-evaluation of the implemented strategy in relation to the course of developments and the particularities of each area, as well as the continuous involvement of citizens on a local level, are extremely important.

APPENDIX

1. 96/DK/012/PAZ -Integrated co-operation for viable tourist development and entertainment uses in the Wadden sea region,96F/386/PAZ - Integrated settlement and management of the Vresti bay and hydrologic basin, 96/F/386/PAZ - Agreement, co-ordination, Opale coast (C.O.4), 96/FIN/071/PAZ - Planning of coastal zones in the Gulf of Finland, 96/GR/537/PAZ - Integrated Coastal Zone Management programme in Cyclades (Picamcy), 96/GR/564/PAZ - Coordinated actions for Coastal Zones Management in Strimona area, 96/GR/580/PAZ - Information, Co ordinations – Requirements for viable coastal development, 96/P/601/LBL - Integrated estuaries management programme in Aveiro - Maria, 96/UK/401/PAZ - Coastal Zones management: Strategy development for open coastal areas, 96/UK/404/LBL- Applications of alternative strategies for the beaches and zones of Ireland. Community participation in viable Coastal development. A demonstration programme for viable management of coasts and dunes, 96/UK/406/PAZ - The forth-scientific forum on rivers and gulfs. A demonstration of substantial Integrated Coastal Zones Management, 96/UK/425/PAZ - The Demonstration programme for Integrated Coastal Zones Management, 97/IT/072/PAZ "RICAMA", 97/IRL/209/LBL"Bantry Gulf".
2. Coastal ink (13) (DENMARK), Coastal ink - Down District (UK), Coastal ink - ANAS (SPAIN), Coastalink - Cornwall (UK), Coastalink - Devon (UK), Coastalink - Ipiros (GR), Coastalink - Kent (UK), CZM - Alarve (PORTUGAL), CZM - kavala (GREECE), CZM - west FLANDRA (BELGIUM), POSIDONIA - Napoli (ITALY), POSIDONIA - Palermo (ITALY), POSIDONIA - Taranto (ITALY), POSIDONIA - Barcelona (SPAIN), POSIDONIA - Athens (GREECE), CONCERCOST -La Costera - Canal, Concercost - La Gironde (FRANCE), Concercost - Vale do Lima (PORTUGAL).

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15. See 8
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25. European Commission, "Report for the development of demonstration program relative to the Integrated Coastal Zone Management", Brussels 12.1.98.
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