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FERROCEMENT FOR HURRICANE – PRONE STATE OF FLORIDA

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ABSTRACT

Ferrocement could be beneficial in hurricane - prone areas like Florida because of its structural integrity and repairable characteristic. Hurricanes had caused notable damages to homes and buildings in Florida and repairing the damage had been usually the preferred option instead of replacement. This paper presents the results of an experimental study on the repairability of ferrocement. Specimens were designed and subjected to four types of loads, which were compression, tension, flexure, and shear, to determine the recovered strength after repair of a ferrocement component. The strength recovery factor cannot be generalized because of the effects of the type of loading, number of layers of mesh reinforcements, cement-sand-water mixture, and other related properties. However, experimental and analytical results indicated that an estimate of approximately 70% of the original strength for compression, 90% for tension and 100% for flexure could be recovered after repair. Shear strength recovery was not investigated. Ferrocement technology offers a better way to build structures to withstand a hurricane environment.

INTRODUCTION

Joseph Louis Lambot of France started it all in 1848 when he constructed several rowing boats, seats, plant pots, and other items from a material that he called "ferciment". Later, in the early 1940s, Professor Pier Luigi Nervi of Italy established some preliminary characteristics of ferrocement through a series of tests. He soon used this material to construct several roofs and small tonnage vessels. It was in this undertaking that Nervi did pioneering work on the use of ferrocement.

Figure 1. Sample ferrocement structure - solar home. (Courtesy: www.ferrocement.com)

Ferrocement is a type of thin wall reinforced concrete commonly constructed of hydraulic cement mortar strengthened with closely spaced layers of continuous and relatively small diameter wire mesh that may be of metallic or other suitable materials. It is a form of reinforced concrete and is also a composite material where the basic underlying concepts in the behavior and mechanics of materials apply (ACI Committee 540, 1982). A sample ferrocement structure designed by Milenko Milinkovic (www.ferrocement.com) is shown in Figure 1.

In the early 1960s, ferrocement construction became widely accepted in Australia, New Zealand, and the United Kingdom. From then on, thousands of ferrocement vessels and structures were built in quite a number of countries. Ferrocement houses utilizing local materials such as wood, bamboo or bush sticks as equivalent steel replacement have been constructed in Bangladesh, Indonesia and Papua New Guinea. Precast ferrocement elements have been used in India, the Philippines, Malaysia, Brazil, Papua New Guinea, Venezuela and the Pacific for roofs, wall panels and fences. In Sri Lanka, a ferrocement house resistant to cyclones has also been developed and constructed. In Israel, ferrocement is used to improve existing houses. Precast corrugated roof units reinforced with local fibers comparable to asbestos cement sheet and galvanized iron sheet are used in Singapore, India, Indonesia, Peru and Zimbabwe (Robles-Austriaco, L., 1992).

Ferrocement technology can be advantageous if used in hurricane - prone areas like Florida. Homes and buildings in Florida had incurred damage due to hurricanes. Repairing the damage had been usually the preferred option instead of replacement.

STATEMENT OF THE PROBLEM

Ferrocement cracks when damaged. The cracks often affect the structural action or the durability of ferrocement significantly. Some common types of damage that occur in ferrocement are delaminations, spalls, scaling, fire damage and local fractures. Delaminations occur when ferrocement splits between layers. A spall is a depression resulting when a fragment is detached from a larger mass by a blow, by the action of weather, by pressure, or by expansion within the mass. Scaling is a local flaking of material near the surface of the mortar. Fire damage releases the amount of chemically bound water in the cement, destroys the bond between the cement and the aggregate, and oxidizes the reinforcement. Local fractures are cracks in which displacement of the section has occurred as a result of impact.

Many types of cracking can be loosely grouped together as "non-structural", which may occur before or after the hardening of ferrocement. Non-structural cracking includes constructional movement of the formworks, settlement shrinkage that cause cracks around reinforcements and fine aggregates, setting shrinkage, internal heat of hydration, temperature stresses, differences in thermal properties of aggregates, and external temperature variations. Another major source of cracking is the chemical reaction in ferrocement constituents such as corrosion, carbonation, and reaction of foreign bodies and reactive aggregates. Cracking due to stress concentrations on the reinforcements, creeping, foundation settlement, member connections, fire, earthquake, overloading, fatigue and vibration can be classified as "structural".

Damage can only be minimized but cannot be totally eliminated. Repairs are thus only to be expected. At times the economic feasibility of repair needs to be considered but the more urgent concern is the comparative strength recovery of a ferrocement element after repair. This paper aims to provide relevant information on the recovered strength after the repair.

Figure 2. Tested specimens

EXPERIMENTAL INVESTIGATION

One hundred sixty two (162) ferrocement specimens shown in Figure 2 were subjected to three types of loading: compression, tension, and flexure; 54 each for compression, tension, and flexure. Eighteen shear specimens were tested but the results needed further investigation and were not included in this paper. In each of the three types of loading, the units were grouped according to cement-sand water

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content proportions by weight: 1:2 (cement-sand) 45% (water-cement ratio), 1:2.5 50%, and 1:3 55%. Three classifications were further formed for each mix proportion and those were according to number of layers of mesh reinforcements: two, four and five layers. A grand total of 27 subgroups were formed with six specimens per subgroup.

The matrix used consisted of mortar made with Portland cement, water and aggregate. Portland cement Type II, was used during the study. This type of cement gives low early and higher late strength. The aggregate consisted of well-graded fine aggregates (sand) that passed through ASTM No. 8 (2.36 mm) sieve. The result of grain size (sieve) analysis is shown in Table 1.

Table 1. Sieve analysis results.

Sieve size	% Passing by weight
No. 10 (2.000 mm)	100
No. 20 (0.850 mm)	81.15
No. 40 (0.425 mm)	54.37
No. 60 (0.250 mm)	19.95
No. 80 (0.180 mm)	6.83
No. 100 (0.150 mm)	3.56

The locally available galvanized hexagonal woven wire mesh, that was more commonly called the chicken wire mesh, was used as reinforcements. Chicken wire mesh was the most commonly used mesh and readily available in most countries. This mesh was fabricated from cold-drawn wire that was generally woven into hexagonal patterns. It could be woven at the site from coil straight wire, allowing the user greater opportunity to choose the mesh size and wire diameter appropriate to any given job. The selection of chicken wire mesh was based on results of tests on the strength and performance of different wire strands. Other types were welded square mesh and expanded aluminum mesh. Chicken wire mesh was the least expensive and had the highest tensile strength among the meshes. It exhibited the most brittle failure. The diameter of a chicken wire mesh strand used in this study was 0.67 mm. The mesh had an opening of approximately 19 mm. A stress – strain diagram for a strand of chicken wire mesh is shown in Figure 3.

Three proportions of cement–sand and water were selected based on the results of preliminary tests of wall panels for low-cost ferrocement housing. For 1:2 cement sand ratio by weight, 45% water-cement ratio by weight had the highest compressive strength. Similarly, the highest mortar strengths for 1:2.5 and 1:3 cement-sand ratios were obtained when the water-cement ratios were 50% and 55%, respectively. In this study, the average compressive strength obtained from cylinder (150 mm x 300 mm) tests was 288 kgf/cm^2 for a mixture of 1:2 45%, 222 kgf/cm^2 for 1:2.5 50%, and 211 kgf/cm^2 for 1:3 50%. The mortar casting procedure implemented was the so-called open-mold system. In this system, the mortar was applied from one side through layers of mesh attached to the open-mold.

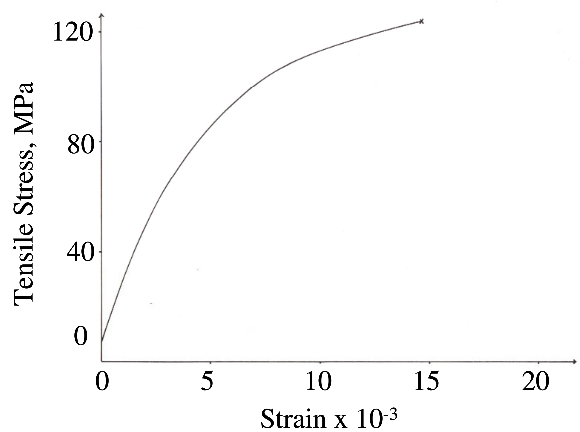


Figure 3. Stress – strain diagram of a strand of chicken wire mesh.

<u>Design of Specimens</u>

Figure 4 shows the dimensions of the specimens. Hollow cylindrical specimens with an outer diameter of 150 mm, a thickness of 32 mm, and a height of 200 mm were subjected to axial compression. The shape of compression specimen was patterned after the specimens used to study the stress-strain curve and the Poisson's ratio of ferrocement under axial compression (Rao et al., 1986). The mesh was rolled to obtain the required number of layers. Plastic molds for 150 mm x 300 mm (6" x 12") concrete test cylinders were modified to create hollow cylindrical compression specimens. As shown in Figure 5, filler made of a round wooden platform with a cylindrical pole at the middle was used to attain the required dimensions of a compression specimen.

In casting the tension, flexure, and shear specimens, layers of plywood were cut to form the shapes of the specimens as shown in Figure 5. The plywood pieces were secured to the platform using 5-mm diameter steel bolts spaced at 100 mm on center. Layers of wire mesh were placed in between the plywood pieces. Wire mesh strands were pulled outward slightly to minimize sagging of the mesh and to maintain the spacing between layers of mesh.

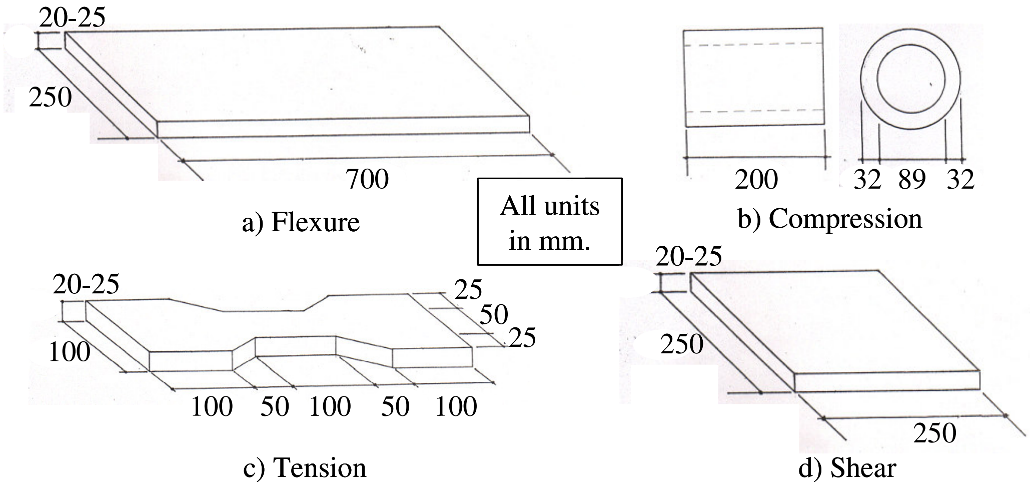
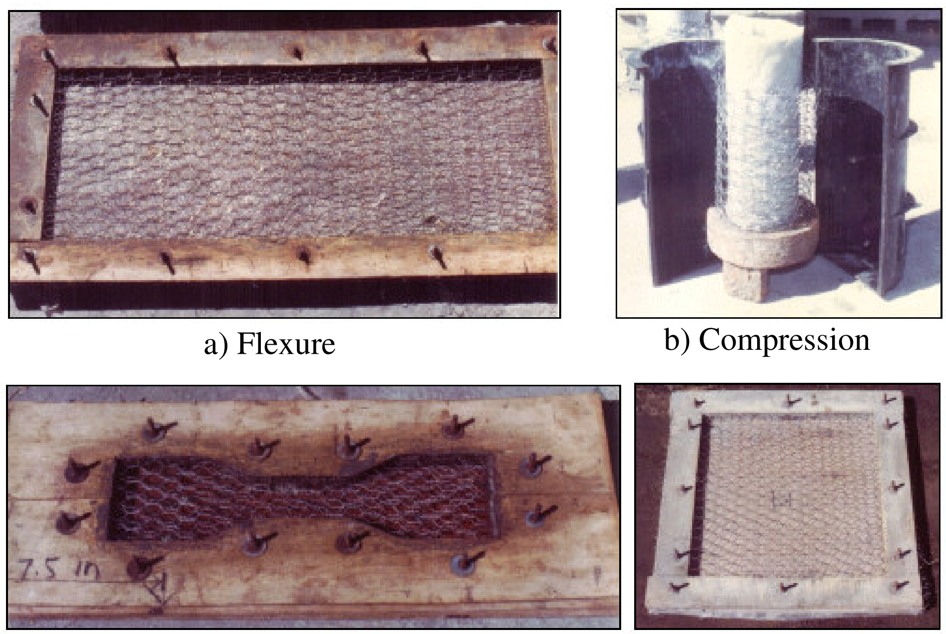


Figure 4. Dimensions of specimens.

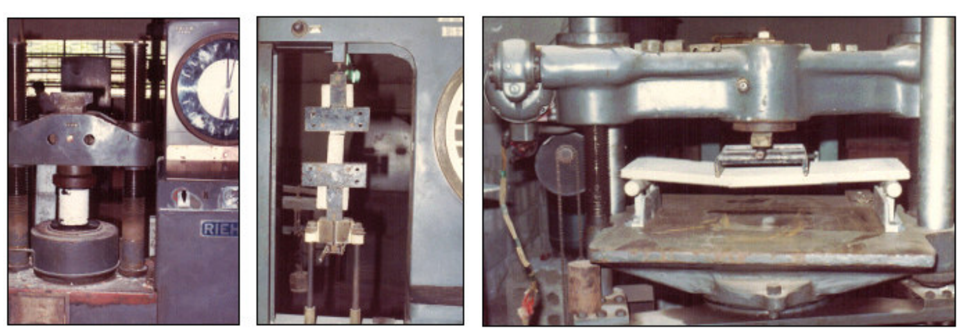


a) Flexure b) Compression

c) Tension d) Shear

Figure 5. Moldings for specimens

After pouring the mortar matrix, the molds were subjected to vibration for approximately 30 seconds. The mortar was then cured for one hour before applying the finishing touches on the surfaces of the specimens. All specimens were demolded after 24 hours and were submerged in a curing tank full of water for approximately five days in order to keep the water evaporation to an absolute minimum. On the seventh day, the specimens were dried and tested.



a) Compression b) Tension c) Flexure

Figure 6. Loading setup.

<u>Test Setup</u>

The experiment was part of research on the use of ferrocement for low-cost housing that was conducted at the University of the Philippines Building Research Service. Specimens were tested at the testing laboratory of the Civil Engineering Department, University of the Philippines. Each type of specimen was subjected to loading as shown in Figures 6.

As shown in Figure 6a, an axial load was induced on each specimen for compression. Tension specimens were held on the inclined edges as shown in Figure 6b and were pulled concentrically. Specimens for bending were subjected to a two-point loading on third points as shown in Figure 6c. A compressive load was applied along the diagonal of a square shear specimen (not shown). Shear tests were discontinued when the expected diagonal tension failure did not occur.

a) Flexure

b) Tension

c) Compression

Figure 7. Repair Method.

Repair Procedure

Tests to determine the original ultimate strengths of the specimens were done on the seventh day after casting. Each specimen was immediately repaired, cured for five days, and tested on the seventh day after the repair. The process of repair, which was accomplished by hand, varied for each type of specimen. For a flexure specimen, the mortar was chipped off using a hammer and a concrete nail up to a distance of approximately 50 mm from the last visible crack shown in Figure 7a. Wire meshes with strained or ruptured strands were repaired by placing new sets of wire mesh. The ends of the strands of the new sets of wire mesh were secured to the original mesh by creating hooks. As shown in Figure 7b, the same repair procedure was done on tension specimens but only 30 mm of mortar from the last visible crack was chipped off.

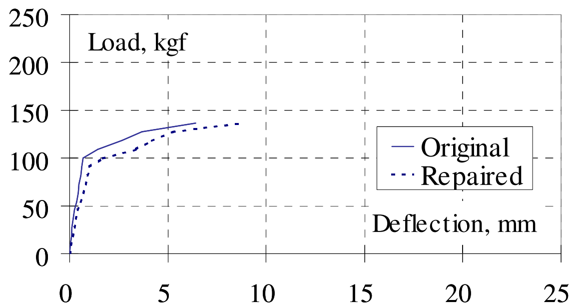
In repairing compression specimens, approximately 20 mm of mortar from both sides of visible cracks was removed from each original specimen. As illustrated in Figure 7c, the full depth of mortar was chipped off using a hammer and a concrete nail. Portions of the mesh layers that were exposed after chipping off were reinforced with additional layers of mesh. After removing the presumably damaged mortar, the same mortar mixture was prepared and plastered on each chipped specimen until the original shape was obtained. The plastering was done by pushing the fluid mortar on the mesh cage from one side until excess appeared on the opposite face. The excess was then pushed back and the surface was flush finished. A trowel was used to place a stiff mortar.

Table 2. Test Results - Hodges-Lehmann Location Estimates (Non-Parametric Statistical Method)

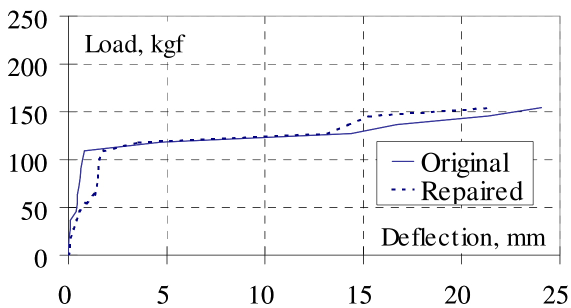
Mixture	Loading	No. of Layers	Recovery Factor (Median)
1:2 45%	Compression	2	0.6960
		4	0.6400
		5	0.7210
	Tension	2	1.1130
		4	1.0395
		5	1.0465
	Flexure	2	1.0485
		4	1.1200
		5	1.0735
1:2.5 50%	Compression	2	0.6360
		4	0.4700
		5	0.9975
	Tension	2	1.1760
		4	1.0075
		5	0.8170
	Flexure	2	0.8665
		4	1.1910
		5	1.0100
1:3 55%	Compression	2	0.8670
		4	0.9015
		5	0.8310
	Tension	2	1.0950
		4	0.7805
		5	0.4910
	Flexure	2	1.1830
		4	0.8530
		5	0.5930

Experimental Results and Discussion

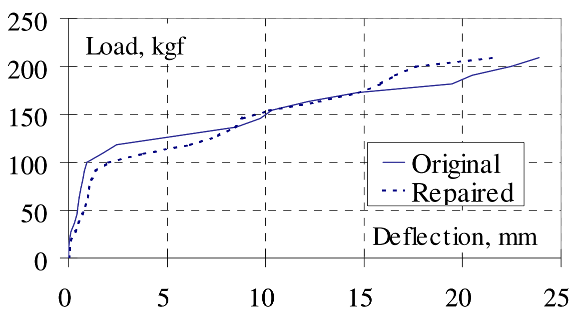
The strength recovery factors of 162 tested ferrocement elements were tabulated in Table 2. The location estimates in Table 2 were the medians of the strength recovery factors that were computed using the Hodges-Lehmann estimation procedure, a non-parametric statistical method of analysis. The results for the other 18 shear specimens were not included. Figure 8 shows typical graphs used to compare the load-displacement diagrams of the original and the repaired flexural specimens. Typical load-displacement relations for original and repaired compression elements were plotted in Figure 9. In each type of loading, when the water-cement ratio was varied, the load-displacement diagrams had no significant change in shape.



a) Two layers of wire mesh

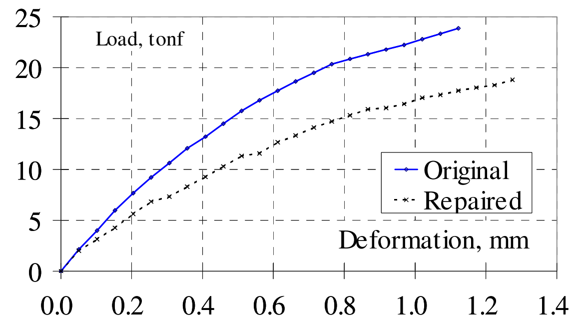


b) Four layers of wire mesh

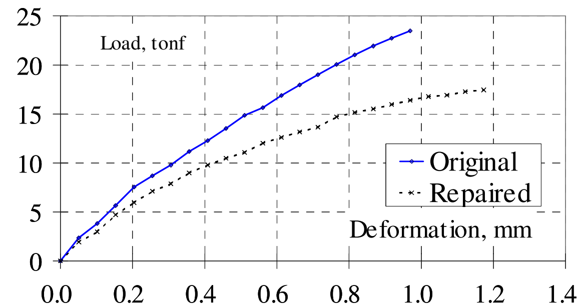


c) Five layers of wire mesh

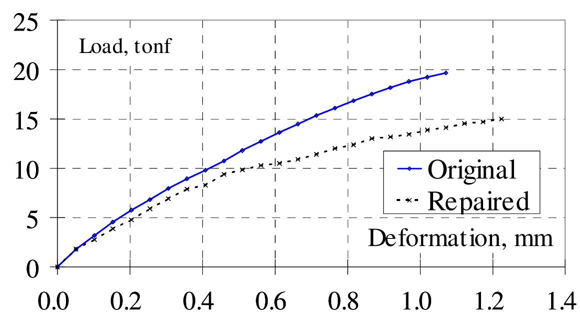
Figure 8. Typical load – deflection diagrams for flexural specimens



a) Two layers of wire mesh



b) Four layers of wire mesh



c) Five layers of wire mesh

Figure 9. Typical load – deflection diagrams for compression specimens

First, the results were analyzed using parametric statistical methods on the assumption that the population of independent samples followed a normal probability distribution. Second, another statistical analysis was performed using non-parametric methods (ANOVA - Analysis of Variance and Kruskal-Wallis) without the traditional assumption that the underlying populations were normal. No distributional assumption was needed for non-parametric statistical procedures. The application of both parametric and non-parametric methods deemed to provide more reliable results. Good agreement between the results of the parametric statistical analyses and the results of the non-parametric methods indicated that the data were reliable. Third, theoretical structural calculations were carried out to determine the functional relationship between the analytical strengths and the experimental results. The theoretical predictions were compared to the results of the experimental tests.

Theoretical Results and Discussion

Compression - The assumption was that the nominal resistance of ferrocement sections subjected to uniaxial compression was approximately equal to the load-carrying capacity of the unreinforced mortar (concrete) matrix assuming a uniform stress distribution of 0.85*f'c. If the cracked mortar, after subjecting the element to its ultimate load, was chipped off and was replaced with the same mortar mixture, its compressive strength could be theoretically equal to the original load-carrying capacity.

From the experimental results, the mean levels of the recovered strength for compression specimens ranged from approximately 50% to 100% of the original strength, with a predominant frequency of values falling below 100%. Based on the observations during experimentation, some reasons why the recovered strength tend to be lower than the original could be stated as follows:

- Not all cracks were visible, therefore only those parts with visible cracks were chipped off and replaced by new mortar. If a repaired compression unit were subjected to loading, the load resistance would be reduced due to unrepaired cracked portions. The evidence could be seen on some specimens that cracked within the portion of unreplaced mortar. The lost bonding between fine aggregates that controlled the splitting strength of the unrepaired portions was not recovered. Cold joints between new and existing mortar created planes of weakness.

Tension – The nominal resistance of cracked ferrocement elements subjected to pure tensile loading was approximately equal to the load-carrying capacity of the mesh reinforcement alone in the direction of loading. The nominal tensile capacity was directly proportional to the effective cross-sectional area of the mesh reinforcement.

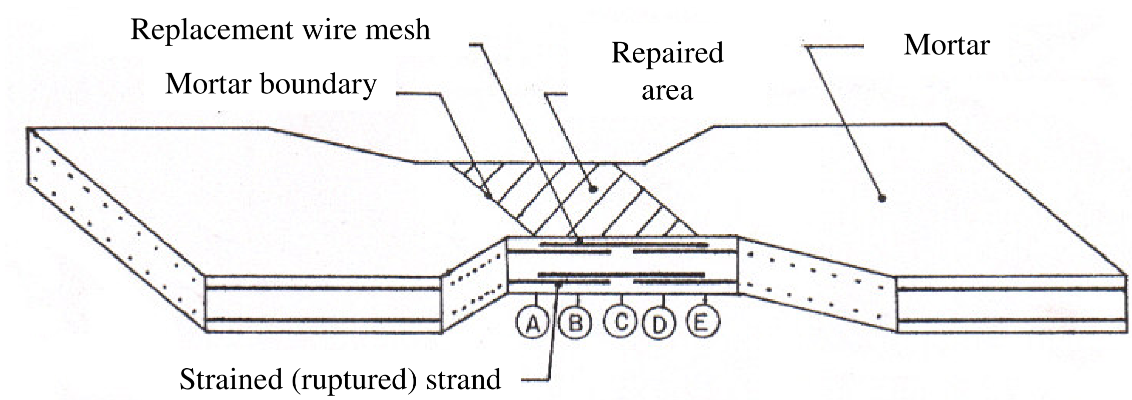


Figure 10. Illustration of sections after repair of tension specimen.

In the repair of tension specimens, same wire meshes were added to reinforce strained wires. The strained wires, whether ruptured or not, were retained so that the added wires could be anchored on them. The original wire mesh had remaining tensile strength that contributed to the tension resistance of a repaired specimen. As illustrated in Figure 10, tensile strengths at sections (A), (C) and (E) were considered the same as the original section because of the effective number of reinforcement layers. Sections (B) and (D) were presumably stronger due to doubled wire mesh layers. Theoretically, the expected recovered strength after repair was approximately 100% of the original tensile strength if the failure occurred at any of the sections (A), (C) or (E), and approximately 200% of the original strength if the failure was either at section (B) or at section (D). Experimental results showed that most of the recovered tensile strengths were more than 100% of the original strengths.

The following justifications can be derived from the observed behavior of the tension specimens:

- 100% of the original strength was recovered when the failure occurred somewhere at section (A), (C) or (E).
- In most cases, since the points of discontinuity or rupture of the strained layers of reinforcement did not lie on the same cross-sectional plane shown in Figure 10, the effective mesh reinforcement area on that particular transverse section was larger than the original, causing a strength recovery higher than 100% of the original strength.
- During the testing of tension specimens, failure occurred at the boundaries of the replacement mortar and the original mortar, where the ends of the replacement wires were laid. At the boundary section, the wires resisting the tensile load were the original wires that had been loaded during the testing of the original specimen. If these wires had already yielded, the resistance recovery will be lower than the original tensile capacity. If the wires had not yet failed and were still within the proportional limit there will be a 100% recovery of the strength.

Flexure – The load-carrying capacity of a ferrocement unit under flexural loading was not dependent on only one variable. Some factors affecting its resistance were: the number of layers of wire mesh, the volumetric fraction, the compressive strength of concrete, the yield strength of wire mesh, the orientation of mesh and the dimensions of the section.

A computer program to calculate the nominal moment capacity of a ferrocement cross-section was developed based on the specifications of ACI Structural Journal, Guide for the Design, Construction, and Repair of Ferrocement, reported by the ACI Committee 549. Calculation results showed that a repaired ferrocement unit could acquire a nominal moment capacity greater than the original flexural resistance. For 2 layers, 4 layers and 5 layers, a theoretical average of approximately 174%, 189% and 196% respectively, of the original moment capacity could be recovered. On the average, the experimental strength recovery factor ranged from approximately 70% to 120%, which were lower than the theoretical results. Some rationalizations behind the differences were as follows:

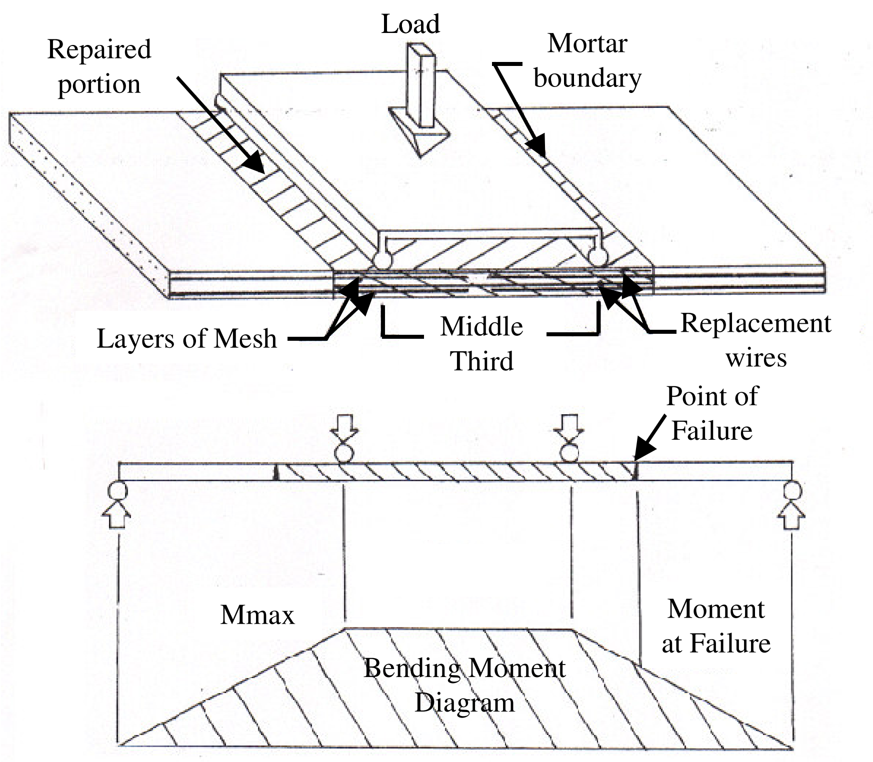


Figure 11. Load and bending moment diagram for flexural specimens

- Since the strained wires were not removed during the repair, the number of layers was considered doubled in the theoretical analysis. Since the original wires were not continuous anymore, the sections on the points of discontinuity had a lower flexural load resistance that governed the moment capacity of a specimen. If the points of discontinuity or rupture were lying on a transverse section, that section would have a recovered moment capacity approximately equal to the original moment resistance. In most cases the points of discontinuity were lying on different transverse sections, thus a recovery factor of a little more than 100% was obtained.

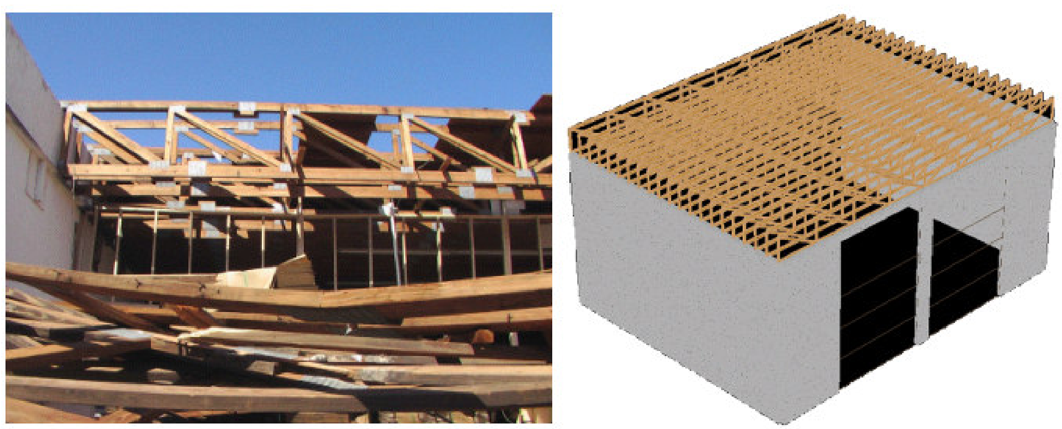
- Most of the failures occurred on the boundary of the replaced mortar and the old mortar. In most cases the boundaries were not within the middle third of the specimen, where the maximum moment occurred. Rather, they were located at a certain distance from the third points. The moment capacity at that point, which was lower than the moment capacity of any section within the middle third, could be obtained by doing an interpolation on the moment diagram of the flexural set-up as illustrated in Figure 11. Based on several interpolations, the governing bending moment on the boundaries came out to be more or less equal to the original load-carrying resistance (Adajar, J.C., 1989).

POTENTIAL APPLICATION

Hurricanes had caused significant damage to buildings, houses and other structures in Florida. Structural damage and non-structural damage were observed through forensic engineering investigations in the state. In this paper, distress on components that were part of the main wind force resisting system was classified as structural damage. Damage on components that were not part of the main wind force resisting system was considered non-structural. It was observed that non-structural damage was more common. Roofing was the problem in many cases. Although a roof structure was not compromised, the unsealed roofing allowed water intrusion that caused notable damage inside a building or a house.

There were few cases where non-structural destruction led to the collapse of the structural framing. An example was a warehouse type building shown in Figure 12 that used rectangular trusses as roof framing. The roof trusses had inadequate cross bracings. When the plywood roof deck was blown off by the wind, the roof trusses became unstable and half of the roof assembly collapsed. Part of the slender reinforced concrete-framed wall filled with concrete masonry units also collapsed due to loss of lateral bracing.

Detachments of built-up roofing, shingles, roof tiles, sheet metal roofing and wood shakes had been the typical hurricane damage to roofs. In some cases, cracking of the wall stucco and concrete masonry units was observed. Claims were innumerable but investigation results indicated that all damage had one thing in common: Each failure was governed by the lack of structural integrity of non-structural components.



a) Hurricane roof damage b) Structural model

Figure 12. Collapse of roof due to loss of roof deck lateral bracing.

Inadequate capacity of fasteners to resist wind uplift caused the detachment of built-up roofing. Shingles were torn due to inadequate flexural and tensile resistance. Insufficient fasteners and bond deterioration between the grout and the attached roof tiles allowed the wind to displace the tiles. Corrugated sheet metal roofing between fasteners had bent and created gaps for water intrusion.

Most of the roof assemblies consisted of separate pieces that were joined together using connectors and fasteners. Failure that usually occurred in the connections caused the problem. Connecting the component discontinuities was the concern that needed to be addressed.

Being able to identify the cause and origin of the problem was a step toward finding its solution. The use of ferrocement technology could be a further step to solve the problem. Roof assemblies that consisted of separate pieces of different types of materials could be replaced with one monolithic piece made of ferrocement. Minimal connections would provide more continuity in the load path. The required strength of the roof or wall component could be obtained by calculating and providing the necessary design thickness. Various shapes could be created without compromising the integrity of the ferrocement structure. In short, the use of ferrocement technology could alleviate some hurricane-related problems in Florida. However, corrosion of mesh and steel reinforcements as a result of exposure to salt water environment should be seriously taken into account because it could result to excessive cracking of mortar.

The construction method used in building a house using prefabricated ferrocement roof and wall panels shown in Figure 13 could be implemented in Florida. A recently developed precast concrete connection method (Imai, H. 1993) would be an appropriate way to connect prefabricated ferrocement panels. Results of experimental and analytical studies on precast concrete structural walls (Adajar, J.C. 1997) could be applied to design mesh and steel reinforcements of a ferrocement element.

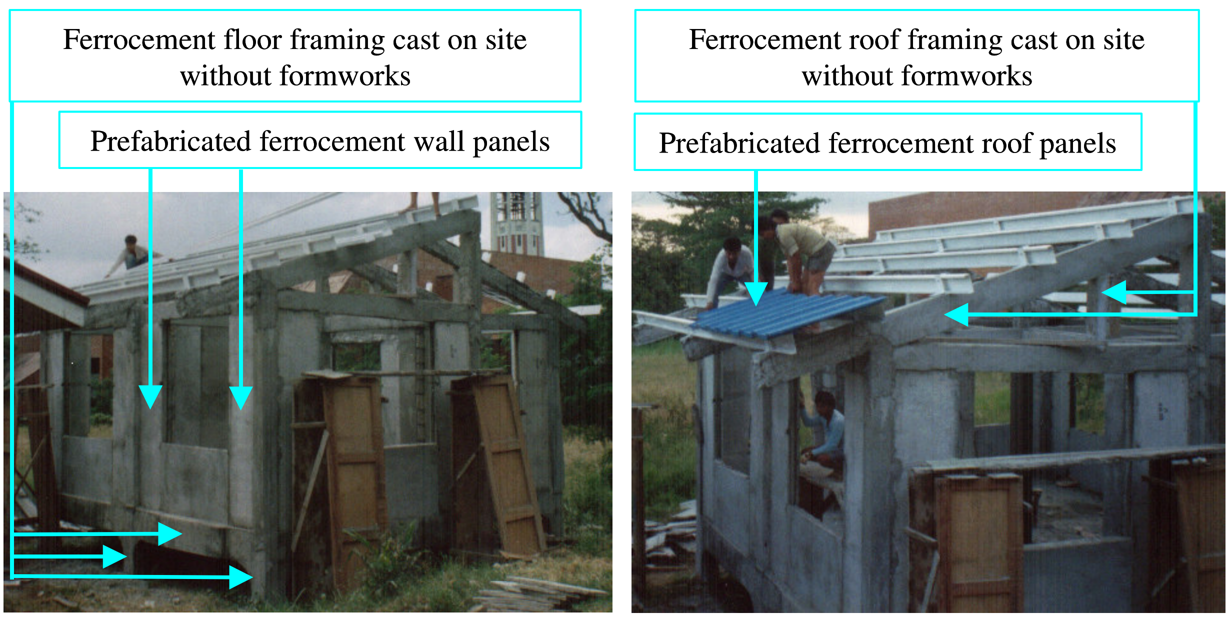


Figure 13. House made of prefabricated and cast-in-situ ferrocement components.

The Dome of a Home in Pensacola, Florida (www.domeofahome.com) shown in Figure 14 had illustrated the idea on how ferrocement technology could be beneficial. The owner, while making the repairs to his home after being severely devastated by Hurricanes Alberto, Erin and Opal, kept thinking, "There has to be a better way to build structures to withstand this environment." After researching building techniques that would alleviate extensive hurricane devastation, he came up with a dome-type monolithic structure. Withstanding 483+ kph (300+ mph) winds, storm surges, termites, rising energy costs, fires, and even earthquakes, air form concrete domes were claimed indestructible. The owner had proven that it was a true sanctuary, a place to come home to even after a hurricane.

CONCLUSIONS

The recovered strength after repair of a ferrocement element could not be generalized. The type of loading, number of mesh reinforcement layers, cement-sand water content mix, and other related properties had effects on the strength recovery factors. Experimental and analytical results indicated that the recovered strength could be roughly estimated to be approximately 70% of the original strength for compression, 90% for tension and 100% for flexure.

Because of the structural integrity and the repairability, ferrocement can be an appropriate substitute to commonly used construction materials in Florida that were less resistant to hurricane forces.

Figure 14. A potential structure for ferrocement technology.

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