II

RESEARCH ARTICLES

FLEXURAL PERFORMANCE OF RUBBERIZED CONCRETE PANELS REINFORCED WITH POLYMER GRID

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ABSTRACT

The disposal of discarded tires is a problem of significant proportion. In the present experimental study, rubber produced from the granulation of discarded tires was used as an additive to replace certain portions of mineral aggregates in concrete. This rubberized concrete was used in making thin panels. A layer of polymer grid was used to reinforce the rubberized concrete panels. These panels were developed to study their performance in applications where the concrete could be subjected to flexure. Buildings constructed in areas with extreme wind pressures resulting from hurricanes or tornadoes are examples of structures that require concrete that can handle considerable deformation without failing catastrophically. Three different panel thicknesses, three different water-cement ratios, and three different rubber contents were the parameters evaluated in this study. All panels were loaded in bending with two equal loads applied at two equal distances from the supports. Test results showed that the flexural resistance of the panel increases with an increase in the thickness of the section, and with a reduction in the water-cement ratio of the concrete. The panels behaved in a ductile manner and there were no signs of brittle failure. Considerable deformation was measured during load application where loaded panels fractured but remained intact relying on the elongating polymer reinforcement. In addition to the lightweight properties, it was concluded that rubber concrete and polymer grid could be used as effective tools to impart ductility to the concrete and to control the mode and nature of the brittle failure of conventional concrete.

KEYWORDS

concrete durability, discarded tires, flexural behavior, granulated rubber, polymer-reinforced panels, recycling, rubberized concrete

INTRODUCTION

Recycling is one of the most efficient ways to conserve natural resources. Many materials are being routinely recycled and reused in many applications. Metals, plastics, and glass are probably the most obvious examples of recyclable materials. Ghaly and Gill (2004) reported a study on the use of post consumer plastics in concrete products. They showed that concrete behavior could be altered to meet certain environmental and loading conditions with the use of a percentage of plastic powder in the mixture. There are presently about two billion scrap tires stockpiled across the United States, and at current rates, that number is increasing by about 200–250 million per year (Chung and Hong, 1999). Current trends show that of the 250 million tires scrapped annually, only 18% are being recycled as products, 42% are burned for energy, and about 5% are exported for use in developing countries (Chung and Hong, 1999). That leaves an estimated 35% that end up in landfills. A better use is needed for this waste stream.

While rubberized asphalt has been in use for many years in highway applications, rubberized concrete is only beginning to gain acceptance in the engineering world. If rubberized concrete could be used for large-scale market applications, the benefits to the environment would be substantial. Currently, used rubber tires take up large space in landfills. The chemical processes that are required in initial tire production make it difficult to recycle these tires into new tires. Tires create many problems in landfills and require a substantial amount of energy to compact. They are very resilient, and the voids in their centers are often a wasted space. Also, tires tend to "float" to the top of a trash pile, and can possibly disrupt

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geosynthetic capping systems upon the closure of landfills. Exposed tire piles in landfills also tend to collect and hold rainwater, which makes them a very favorable medium for rat and mosquito breeding grounds. Also, on several occasions, large tire fires have erupted where large quantities of scrap tires were stored, producing thick plumes of air-polluting toxic black smoke and creating water contamination from residue runoff.

Tire shreds have already been used as a component in highway embankments over soft soils and for backfill and drainage layers. Crumb rubber, a very finely ground substance created from scrap tires, is already used in large quantities for asphalt mixes, serving as a noise reducer and durability enhancer in areas where freeze-thaw effects are severe. This same crumb rubber material is only beginning to be used in lightweight concrete mixtures. A limited number of actual projects have been constructed with rubberized concrete. In the absence of a performance record, many builders and municipalities are skeptical about using it in their design and construction. However, as knowledge about the material grows, and more projects withstand the tests of weather and time, rubberized concrete may become an acceptable construction material. This experimental study examines the flexural properties of rubberized concrete in the form of small, lightweight panels of varying thickness, and reinforced with a polymer grid. Such panels could be used for buildings constructed in areas with very high wind pressures resulting from severe storms. The relationships between deformation, compressive and flexural strengths, water/cement ratio, panel thickness, and rubber content in the mix are the main focus of this paper.

HISTORICAL BACKGROUND

Although the concept of rubberized concrete has been around for a few decades, it was not until the early 1990s that extensive research articles were published on the subject. Eldin and Senouci (1993) reported observations on rubberized concrete behavior. They indicated that rubberized concrete did not demonstrate a brittle failure, but rather a ductile, plastic one. They also concluded that the rubberized concrete had the ability to absorb a large amount of plastic energy under compressive and tensile loads. Topçu (1995, 1997) and Topçu and Avcular (1997) reported the results of three studies conducted to study the properties of rubberized concretes. They postulated that rubberized concrete was a potential material for construction applications that are subjected to impact effects such as crash barriers, bridges and roads. They observed that the plastic energy capacity increases when the high elastic energy capacity of normal concrete is reduced by adding rubber to the mixture. Toutanji (1996) used rubber tire particles in concrete to replace mineral aggregates. He concluded that the incorporation of rubber tire chips in concrete exhibited a reduction in compressive and flexural strengths. He reported that the reduction in compressive strength was approximately twice the reduction of the flexural strength. Ahmad et al. (1997) investigated the freeze-thaw durability of rubberized concrete. They concluded that, due to the ductile nature of rubberized concrete, it has greater ability to withstand many cycles of freeze and thaw without exhibiting apparent damage.

Bayomy and Khatib (1999) studied the performance of rubberized Portland cement concrete. Due to significant compressive strength reduction of rubberized concrete, they suggested that the rubber content in the mix should not exceed 20% of the total aggregate volume. They also indicated that rubberized concrete might be more suitable for nonstructural purposes such as lightweight concrete walls, building facades and architectural units, or as cement aggregate bases under flexible pavements. Ghaly (2004) conducted an experimental study on the performance of rubberized concrete under moderate freeze-thaw conditions. He added rubber content of 5, 10, and 15% by volume of fine aggregate to the concrete mix. The specimens made were tested under normal no-freeze, freeze in air, freeze in water, and freeze-thaw cycles in water. Ghaly observed that the addition of rubber in concrete reduces the workability of the fresh mix and its 28-day strength. He concluded that freezing the specimens in air results in little or no effect on the strength of the concrete as compared with that of the specimens tested without freezing. Freezing the specimens in water results in a very slight, insignificant reduction in the strength of concrete. Subjecting the concrete specimens to freeze-thaw cycles in water results in a small loss of strength. He also concluded that rubberized concrete demonstrated greater ability to deform under the application of compressive forces, and that the failure of rubberized concrete is appreciably less brittle than that of non-rubberized concrete.

RESEARCH PROGRAM

This study examined the flexural performance of rubberized concrete panels of various thicknesses, at ages of 3, 7, and 28 days. The goal was to monitor the panel behavior with time. The tested parameters were:

1. Water/cement (W/C) ratios of 0.47, 0.54, and 0.61 with 0% rubber.
2. W/C ratios of 0.47, 0.54, and 0.61 with 25% rubber by volume of total concrete mix.
3. W/C ratios of 0.50, 0.54, and 0.61 with 50% percent rubber by volume of total concrete mix.

A total of 48 panels (Table 1) were made for testing in flexure. The panels were 2.5, 3.75, and 5.0 cm thick (Table 2). In addition to the granulated rubber that replaced a portion of the mineral aggregates in the concrete mixtures, fine aggregate (sand), small coarse aggregate (crushed stone), and large coarse aggregate (crushed stone) were used in the concrete mixtures. The fine aggregate, small coarse aggregate, and large coarse aggregate are referred to as FA, CA1, and CA2, respectively in Table 2. For each of these thicknesses, 6 concrete mixtures were designed (Table 3).

Rubber Particles and Aggregate Specifications

From the manufacturer's information, it was known that the largest dimension of the granulated rubber particles used in the mixtures was 1.4 mm. For the sand, the vast majority of particles was in the range 0.15–1.18 mm, for small coarse aggregate, the vast majority of sizes was in the range 1.75–3.35 mm, and for coarse aggregate, the vast majority of sizes was in the range of 3.35–12.7 mm. These results were obtained from mechanical sieve analysis tests.

Design of Rubberized Concrete Mixtures

No standardized procedure exists to design a rubberized concrete mix. The mix designs were experimental, and were largely based on the experience of past projects (Ghaly, 2004, and Ghaly and Gill 2004). The concrete mixtures were designed by volume, but the ingredients were proportioned by weight. The volume method was used in the design due to the fact that the unit weight of rubber is considerably low when compared with the unit weights of the mineral aggregates used in the mix. The rubber content in the prepared concrete mixtures is represented by the percent of rubber by volume of the total mix, and this replaced an equivalent volume of the mineral aggregate.

TABLE 1. Thickness, W/C ratio, and rubber content of panels used in testing.

2.5 cm thick panels						
Panel number	1, 2, 3	4, 5, 6	7, 8, 9	10	11	12
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber %	25	25	25	0	0	0
3.75 cm thick panels						
Panel number	13, 14, 15	16, 17, 18	19, 20, 21	22	23	24
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber %	25	25	25	0	0	0
5.0 cm thick panels						
Panel number	25, 26, 27	28, 29, 30	31, 32, 33	34	35	36
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber %	25	25	25	0	0	0
2.5 cm thick panels (no large coarse aggregate)						
Panel number	37, 38, 39	40, 41, 42	46, 47, 48	43	44	45
W/C	0.50	0.54	0.61	0.50	0.54	0.61
Rubber %	50	50	50	0	0	0

TABLE 2. Mass (in kg) of ingredients used in making panels.

Panel #	W/C	Water	Cement	CA1	CA2	FA	Rubber
2.5 cm thick panels							
1	0.47	1.63	3.46	4.11	4.11	4.32	1.44
2	0.47	1.54	3.11	3.70	3.70	3.89	1.29
3	0.47	1.54	3.11	3.70	3.70	3.89	1.29
4, 5, 6	0.54	5.12	9.48	12.94	12.94	19.34	4.83
7, 8, 9	0.61	4.47	7.32	11.29	11.29	17.66	4.42
10	0.47	1.48	3.14	3.00	3.00	5.82	0
11	0.54	1.48	2.73	3.00	3.00	6.20	0
12	0.61	1.48	2.73	3.00	3.00	6.43	0
13, 14, 15	0.47	6.58	14.02	16.64	16.64	17.50	5.83
16, 17, 18	0.54	6.14	11.38	15.53	15.53	23.21	5.80
19, 20, 21	0.61	6.39	10.46	16.13	16.13	25.23	6.31
3.75 cm thick panels							
22	0.47	2.25	4.77	4.56	4.56	8.85	0
23	0.54	2.25	4.15	4.56	4.56	9.42	0
24	0.61	2.25	4.15	4.56	4.56	9.77	0
5.0 cm thick panels							
25, 26, 27	0.47	8.52	13.95	21.51	21.51	33.64	8.41
28, 29, 30	0.54	8.17	15.14	20.65	20.65	30.87	7.71
31, 32, 33	0.61	8.50	13.91	21.45	21.45	33.56	8.39
34	0.47	3.02	6.41	6.12	6.12	11.93	0
35	0.54	3.02	5.57	6.12	6.12	12.65	0
36	0.61	3.02	5.57	6.12	6.12	13.12	0
2.5 cm thick panels							
37, 38, 39	0.50	6.00	12.00	0	6.00	18.00	12.00
40, 41, 42	0.54	5.40	10.00	0	5.70	17.11	11.40
43	0.50	1.50	3.00	3.00	3.00	5.82	0
44	0.54	1.48	2.73	3.00	3.00	6.20	0
45	0.61	1.48	2.73	3.00	3.00	6.43	0
46, 47, 48	0.61	5.40	8.90	0	5.40	16.10	10.80

CA1, particles predominantly 1.75–3.35 mm in size.
CA2, particles predominantly 3.35–12.7 mm in size.
FA, particles predominantly 0.15–1.18 mm in size.
Rubber, particles predominantly 1.4 mm in size.

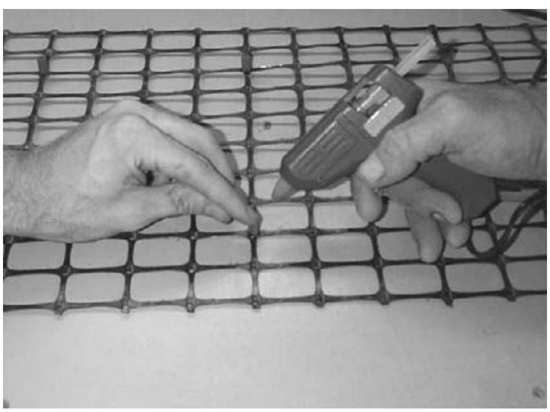
Preparation of Polymer-Reinforced Concrete Panels

All panels tested in this investigation were 20 × 90 cm in dimensions. Polymer grids used in reinforcing the panels were prepared to dimensions slightly less than those of the panels, to allow the grids some clearance from the walls of the molds. Figure 1 shows 1.25 cm plastic spacers being attached to one side of a polymer grid. This was the distance between the grid layer and the bottom of the concrete panel. This distance was maintained constant for all panels tested, regardless of their thickness. Figure 2 shows the polymer grid inside the form where the concrete was poured to mold the required panel. Concrete mixtures were placed into the forms and vibrated on a flat bed shaker (Figure 3). Using the same concrete mixture, 50 mm cubic molds were also filled and vibrated. These cubes were tested in compression to determine the compressive stress of concrete at 3 and 7 days, and the compressive strength at 28 days of pouring and curing (Table 3).

TABLE 3. Concrete cubes prepared for testing in compression and values of compressive stress.

Property	Panel 1, 2, 3			Panel 4, 5, 6			Panel 7, 8, 9		
Cubes from mixes used in 2.5 cm thick panels									
Cube no.	1, 2, 3	4, 5, 6	7, 8, 9	10, 11, 12	13, 14, 15	16, 17, 18	19, 20, 21	22, 23, 24	25, 26, 27
Age (days)	3	7	28	3	7	28	3	7	28
Stress (MPa)	8.33	13.45	13.96	7.94	13.40	10.63	6.10	9.14	9.16
W/C	0.47	0.47	0.47	0.54	0.54	0.54	0.61	0.61	0.61
Cubes from mixes used in 3.75 cm thick panels									
Property	Panels 13, 14, 15			Panels 16, 17, 18			Panels 19, 20, 21		
Cube no.	28, 29, 30	31, 32, 33	34, 35, 36	37, 38, 39	40, 41, 42	43, 44, 45	46, 47, 48	49, 50, 51	52, 53, 54
Age (days)	3	7	28	3	7	28	3	7	28
Stress (MPa)	10.41	15.27	15.33	10.16	11.89	10.67	8.73	10.02	9.40
W/C	0.47	0.47	0.47	0.54	0.54	0.54	0.61	0.61	0.61
Cubes from mixes used in 5.0 cm thick panels									
Property	Panels 25, 26, 27			Panels 28, 29, 30			Panels 31, 32, 33		
Cube no.	55, 56, 57	58, 59, 60	61, 62, 63	64, 65, 66	67, 68, 69	70, 71, 72	73, 74, 75	76, 77, 78	79, 80, 81
Age (days)	3	7	28	3	7	28	3	7	28
Stress (MPa)	10.94	14.72	12.80	9.41	10.45	11.06	8.69	8.23	9.03
W/C	0.47	0.47	0.47	0.54	0.54	0.54	0.61	0.61	0.61
Cubes from mixes used in 2.5 cm thick panels (no large-coarse aggregate)									
Property	Panel 37, 38, 39			Panels 40, 41, 42			Panels 46, 47, 48		
Cube no.	100, 101, 102	103, 104, 105	106, 107, 108	109, 110, 111	112, 113, 114	115, 116, 117	118, 119, 120	121, 122, 123	124, 125, 126
Age (days)	3	7	28	3	7	28	3	7	28
Stress (MPa)	3.45	3.73	4.38	1.59	2.92	3.09	2.93	3.03	3.48
W/C	0.5	0.5	0.5	0.54	0.54	0.54	0.61	0.61	0.61

FIGURE 1. Spacers being attached to polymer grid.



Polymer Grid Properties

The polymer grid used in reinforcing the panels made in the present study is similar to that used as earth reinforcement in geotechnical applications. Since these panels are only intended for light applications, polymer grid reinforcement, rather than steel reinforcement, was deemed sufficient for the task at hand. The polymer grid has square openings 39 × 39 mm dimensions (Figure 2). Both the transverse and the longitudinal ultimate strengths are 20 kN/m. Minimum carbon black content is 2% and the unit weight is 0.22 kg/m^2.

Compression Testing

Each of the 126 cubes that were made was tested in compression at a predetermined age after pouring and curing. For each panel that contained rubber, a corresponding set of 9 cubes was tested in compression. Table 3 shows the results of these tests. To ensure accuracy and consistency of the results, the stress

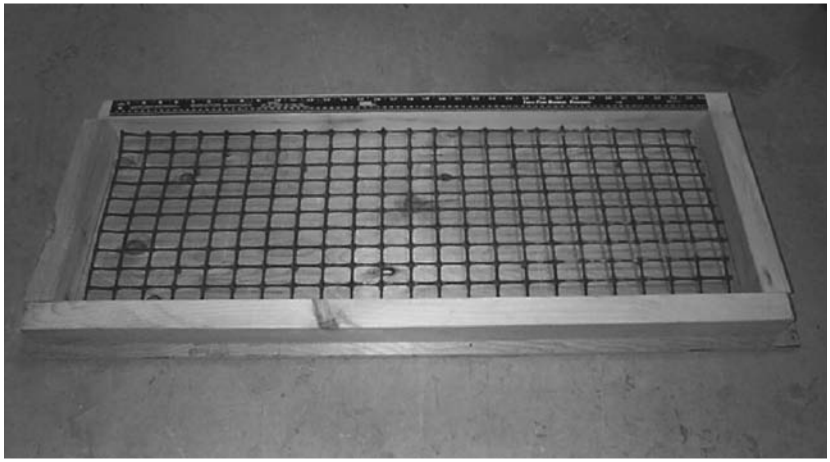


FIGURE 2. Polymer grid in form before pouring concrete.

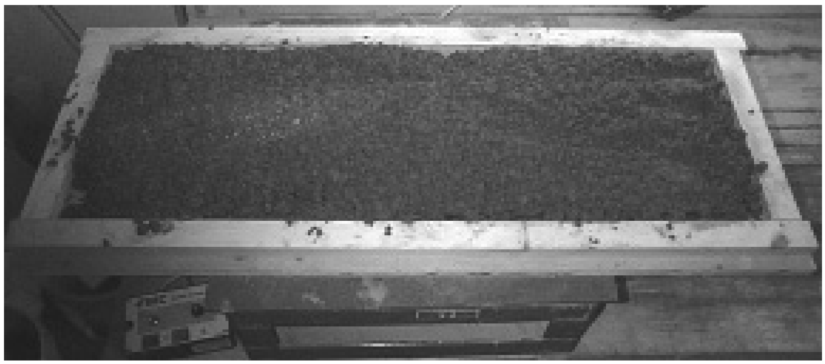


FIGURE 3. Panel on vibrating table.

values reported in this table are the calculated average values for the three cubes in each set.

Flexure Testing of Panels

All the panels were tested in pure bending. Each panel was placed on the loading device, with the reinforced side on the bottom, with supports at equal distances of 15 cm from the outer edges (Figure 4). This left 60 cm of simply supported span. The load was applied, using a Universal Testing Machine, operated at a constant loading rate. The load from the machine was applied at two contact points (Figure 5). This effectively divided the span to three equal distances. This is the loading pattern that allows pure bending to develop in the middle third of the loaded span where no shear occurs (ASTM C78). The bearing blocks used were only 15 cm in width, so the width of the beam was effectively reduced to 15 cm. Although the total width of the tested panels was 20 cm, only the middle 15 cm of that width was loaded because the polymer grid tended to curl at the edges during pouring of concrete (Figure 6). This was done to ensure the loaded section of the panel is fully reinforced with the polymer grid, and is at a constant distance from the bottom of the panel. By loading the internal section, these effects were minimized.

For each panel, load readings and corresponding displacement values were recorded automatically, using a computer and data acquisition system, until failure. The point of failure was defined as the point at which the panel physically breaks, or the point where the applied load remains constant, or decreases, while displacement continues to increase. Several of the tested panels did not fail during the experiment, and the loading was stopped at a deformation of 50 mm measured at the midspan of the panel (Figure 5).

FIGURE 4. Schematic of panel loading in pure bending.

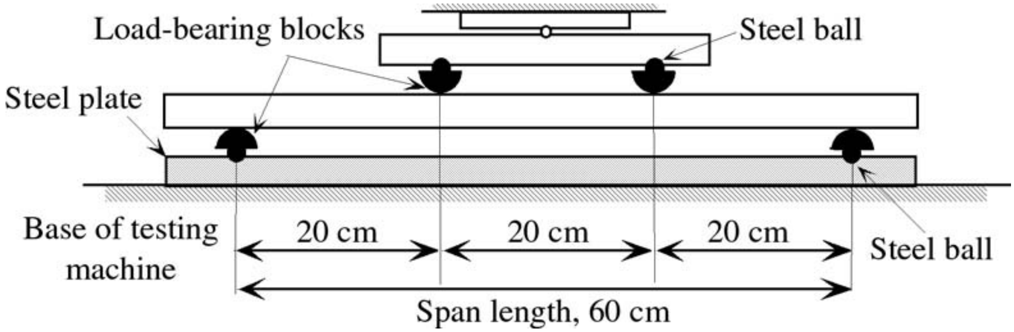


FIGURE 5. Loading on panel in progress.

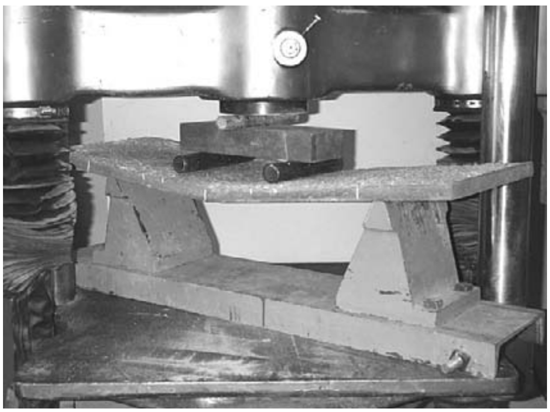
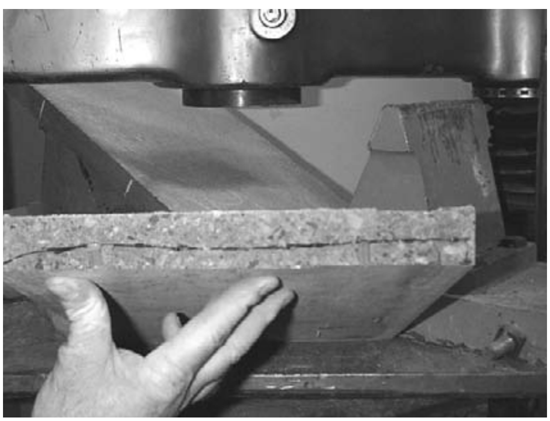


FIGURE 6. Section showing panel after failure.



TEST RESULTS

Figure 7 shows the relationship between the compressive stress of concrete with 25% rubber content and age. Figure 8 shows a similar relationship for concrete with 50% rubber content. In both graphs it can be seen that there is a significant increase in the stress during the first 7 days of concrete age, which is a behavior similar to that of non-rubberized conventional concrete. In concrete with 25% rubber content, there is a slight reduction in stress with time between 7 and 28 days; however, the concrete with the lowest W/C ratio achieved the highest stress. For concrete with 50% rubber content, there is a slight gain in stress between 7 and 28 days. The concrete with the lowest W/C ratio still achieved the highest compressive stress, however the results from the 0.54 and 0.61 W/C ratio concrete were almost identical.

For each panel in this study, the relationship between the applied load and the associated deformation was plotted. The plotted curves were used to determine a number of important values related to the performance of the panels under the applied flexure. These values were the load at failure, associated deformation at failure, load at proportional limit (defined as the load at the end of the initial linear elastic phase of loading), and displacement at proportional limit (defined as the displacement associated with the load at proportional limit). The average results of each three-panel set were used in plotting a number of relationships as will be shown below. For a given panel set, the average is calculated based on the results of three panels in the set. If any of the panels

FIGURE 7. Compressive stress versus time relationship for concrete with 25% rubber content.

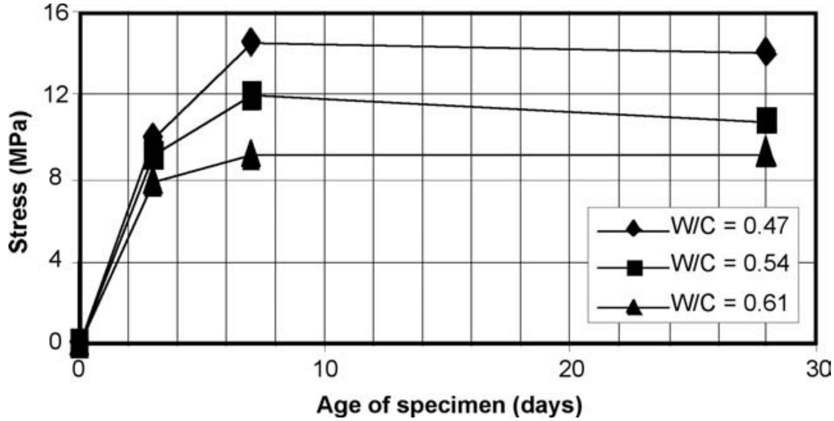
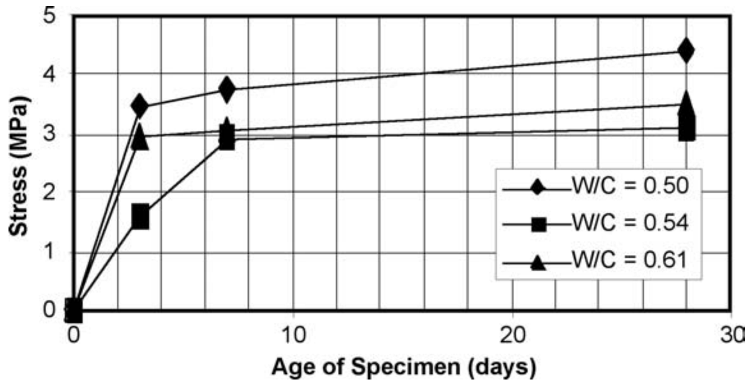


FIGURE 8. Compressive stress versus time relationship for concrete with 50% rubber content.



differed by more that 15% of the average, this panel was discarded. According to this criterion, only three of the panels tested in the present study were discarded. These panels were number 5, 8, and 41.

The shape of the load-deformation relationship was closely monitored. It was noticed that panels made with rubberized concrete and reinforced with polymer grid behave in a remarkable manner as the applied load increases. The applied load increases with deformation up to a point and an initial failure seemed to occur, often quickly, and the load decreases as the deformation continues to increase. The load would rebound and, in several tests exceeded its initial peak as the panel continued to deform. The cycle described above occurred a number of times in some of the samples, resulting in a load-deformation graph that resembled saw teeth (Figure 9). Some panels never showed signs of complete failure, even at 50 mm of deformation, which was the deformation value at which loading was terminated. Relative to conventional concrete panels reinforced with steel, it can be seen that rubberized or non-rubberized concrete panels reinforced with polymer grid behaved in a ductile nature where they were capable of sustaining large plastic deformations without fracture. Table 4 reports the deformations measured at failure of all panels tested in this study. Using the loads deduced from the load-deformation relationship of each panel, the values of the modulus of rupture (S_R), fiber stress at proportional limit (S_f), and shear stress (τ_m) were calculated using equations 1, 2, and 3 (ASTM C78-02). These values are reported in Table 5. For a panel set (such as 1, 2, and 3) made using the same concrete mix and having the same thickness, the values listed in Table 5 are averages of the results of the three panel set. It should be noted that, although ASTM C78 is a standard test method for beams subjected to flex-

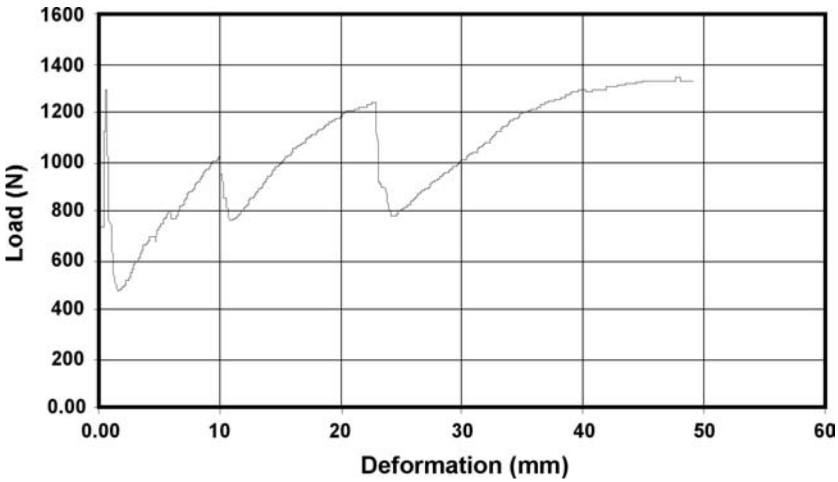


FIGURE 9. Typical relationship of load versus panel deformation (results of Panel # 17).

TABLE 4. Deformation at failure of panels tested in this study.

W/C	Rubber %	Deformation at failure (mm)
2.5 cm thick panels		
0.47	25	6.76
0.54	25	33.40
0.61	25	36.10
0.47	0	14.00
0.54	0	17.50
0.61	0	38.30
3.75 cm thick panels		
0.47	25	11.64
0.54	25	16.30
0.61	25	18.89
0.47	0	21.40
0.54	0	0.66
0.61	0	18.90
5.0 cm thick panels		
0.47	25	6.50
0.54	25	31.53
0.61	25	39.07
0.47	0	0.63
0.54	0	0.34
0.61	0	0.41
2.5 cm thick panels (no large coarse aggregate)		
0.50	50	20.51
0.54	50	42.90
0.61	50	25.64
0.50	0	50.00
0.54	0	0.36
0.61	0	0.51

ure, the procedures of this standard are recommended for the testing of flexural strength of panels (ASTM C1140), and for the testing and determination of flexural toughness of fiber-reinforced concrete (ASTM C1018).

The modulus of rupture is representative of the initial failure, no matter how much additional load is supported by the panel in later readings. For this reason, the modulus of rupture was calculated using the initial peak value of the applied load, no matter how many cycles the load might have rebounded. The following expression was used to compute the values of the modulus of rupture:

$$S_R = PL / bd^2 \tag{1}$$

Where

S_R = Modulus of Rupture
P = Initial peak load
L = Span of panel
b = Width of panel
d = Depth of panel

The fiber stress at proportional limit is reliant on the load at the end of the initial linear elastic phase of loading. If the load-deformation relationship showed a number of cycles, then the load required for the calculation of the fiber stress is determined from the initial linear elastic portion of loading within the first cycle. For some of the panels, this load was identical to the one used in the calculation of the modulus of rupture because all deformation up to the initial failure was proportional. The fiber

TABLE 5. Calculated properties and test results of panels loaded in flexure.

			2.5 cm thick panels			
Property/panel no.	1, 2, 3	4, 5, 6	7, 8, 9	10	11	12
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber	25%	25%	25%	0	0	0
S_R (kPa)	1907	549	849	4790	2423	1056
S_f (kPa)	1406	546	821	623	512	818
τ_m (kPa)	70	48	81	150	76	39
			3.75 cm thick panels			
Property/panel no.	13, 14, 15	16, 17, 18	19, 20, 21	22	23	24
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber	25%	25%	25%	0	0	0
S_R (kPa)	1178	1570	942	248	5931	178
S_f (kPa)	1073	1163	947	248	5931	178
τ_m (kPa)	75	99	63	90	278	136
			5.0 cm thick panels			
Property/panel no.	25, 26, 27	28, 29, 30	31, 32, 33	34	35	36
W/C	0.47	0.54	0.61	0.47	0.54	0.61
Rubber	25%	25%	25%	0	0	0
S_R (kPa)	2512	1901	1244	5650	3383	5057
S_f (kPa)	2512	1901	1244	5650	3383	5057
τ_m (kPa)	198	143	142	353	211	316
			2.5 cm thick panels (no large coarse aggregate)			
Property/panel no.	37, 38, 39	40, 41, 42	46, 47, 48	43	44	45
W/C	0.50	0.54	0.61	0.50	0.54	0.61
Rubber	50%	50%	50%	0	0	0
S_R (kPa)	753	226	206	1009	3162	5162
S_f (kPa)	594	193	175	1009	3162	5162
τ_m (kPa)	31	161	26	14	86	99

stress at proportional limit is determined using the following expression:

$$S_f = P'L / bd^2 \quad (2)$$

Where

S_f = Fiber stress at proportional limit

P' = Load on the panel at the proportional limit for the first increase

The shear stress resisted by the panels was calculated using the maximum load prior to failure. If the load-deformation relationship of a panel showed a number of cycles, the absolute maximum load was determined and used in this calculation. It is worth noting that, for different panels, this load did not necessarily occur in the same cycle. Thus, the deformation associated with this load could vary significantly, however, this deformation is not used in any of the calculations reported in Table 4. The shear stress is calculated using the following expression:

$$\tau_m = 3P_{max} / 4bd \quad (3)$$

Where

τ_m = Shear stress

P_{max} = Maximum load achieved during entire loading process

Figures 10, 11, 12, and 13 are all plotted for panels made with concrete containing 25% rubber. Figure 10 shows the relationship between the modulus of rupture and W/C ratio for panels 2.5, 3.75, and 5 cm thick. The general trend that can be seen is that, for a given W/C ratio, the modulus of rupture increases with the thickness of the panel. It can also be seen that, for a given panel thickness, the modulus of rupture generally decreases with an increase of the W/C

FIGURE 10. Modulus of rupture vs. W/C ratio, 25% rubber.

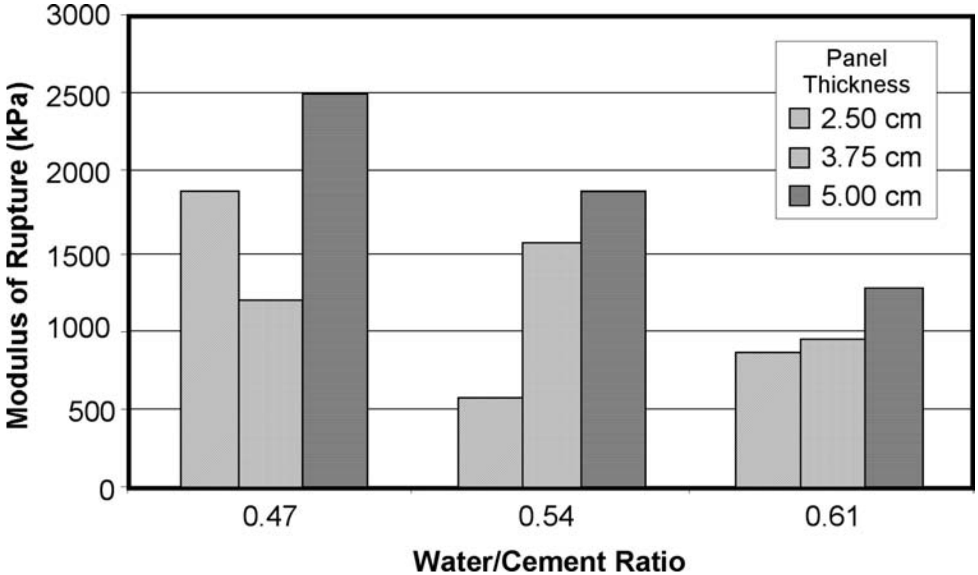
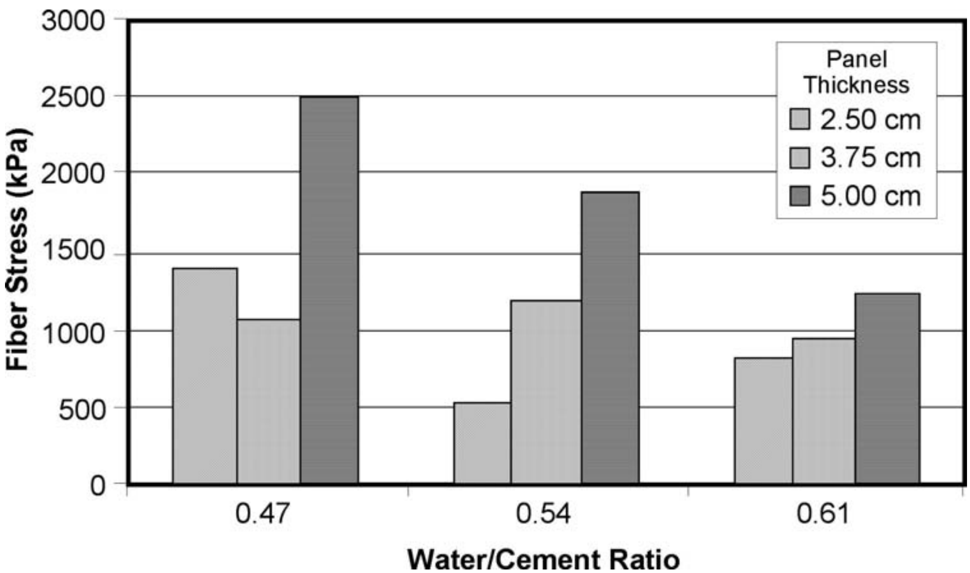


FIGURE 11. Fiber stress vs. W/C ratio, 25% rubber.



ratio. Figure 11 shows the relationship between the fiber stress and W/C ratio for panels 2.5, 3.75, and 5 cm thick. It can be seen that, for a given W/C ratio, the fiber stress generally increases with the thickness of the panel. Figure 12 shows the relationship between the shear stress and W/C ratio for panels 2.5, 3.75, and 5 cm thick. The figure shows that, for a given W/C ratio, the shear stress increases with the thickness of the panel. Figure 13 shows the relationship between deformation at failure and W/C

FIGURE 12. Shear stress vs. W/C ratio, 25% rubber.

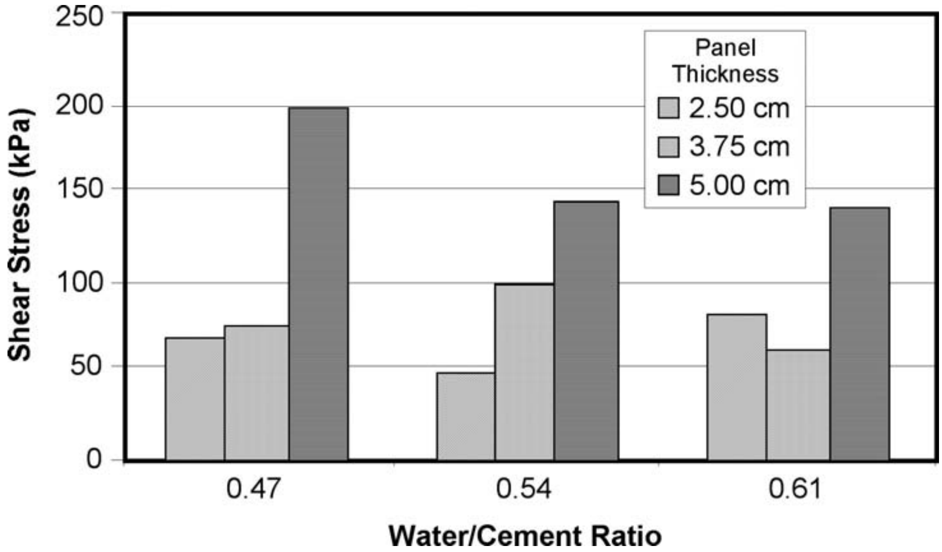
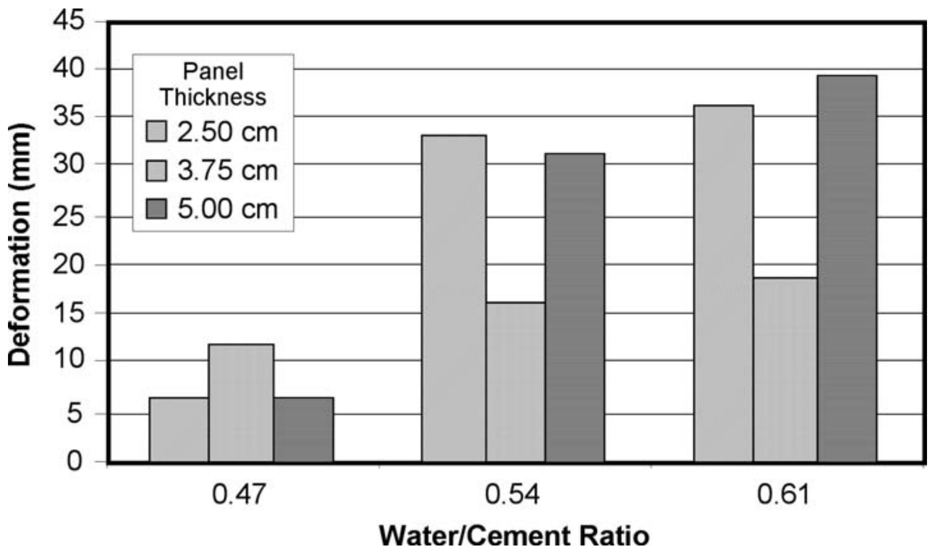


FIGURE 13. Deformation vs. W/C ratio, 25% rubber.



ratio for panels 2.5, 3.75, and 5 cm thick. Although no specific trend can be found for a given W/C ratio, it seems that for a given panel thickness, deformation generally increases with higher W/C ratio.

Figures 14, 15, 16, and 17 are all plotted for panels made with concrete containing 0% rubber and with thickness of 2.5, 3.75, and 5 cm. There seems to be no specific general or clear trends. It appears, however, that a thicker panel results in higher values of modulus of rupture, fiber stress, and shear stress. It also appears that the lower the W/C ratio, the higher the resistance of the panel,

FIGURE 14. Modulus of rupture vs. W/C ratio, 0% rubber.

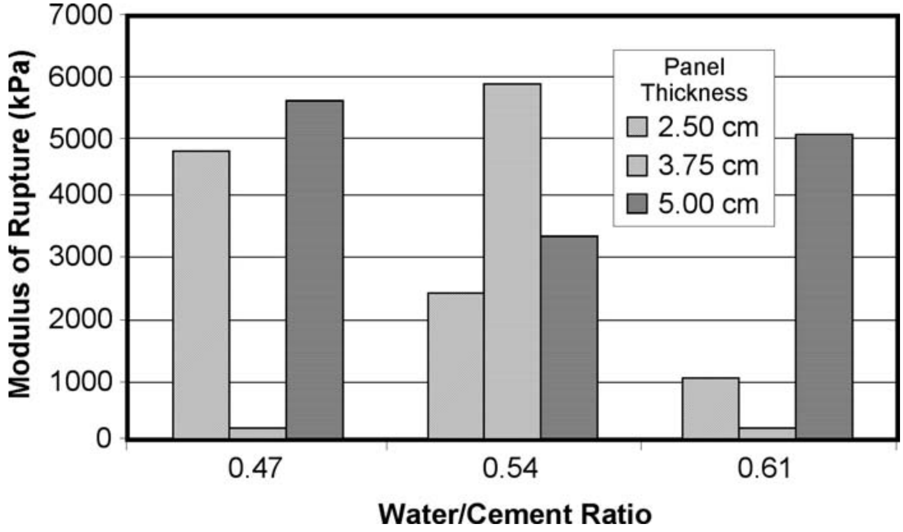
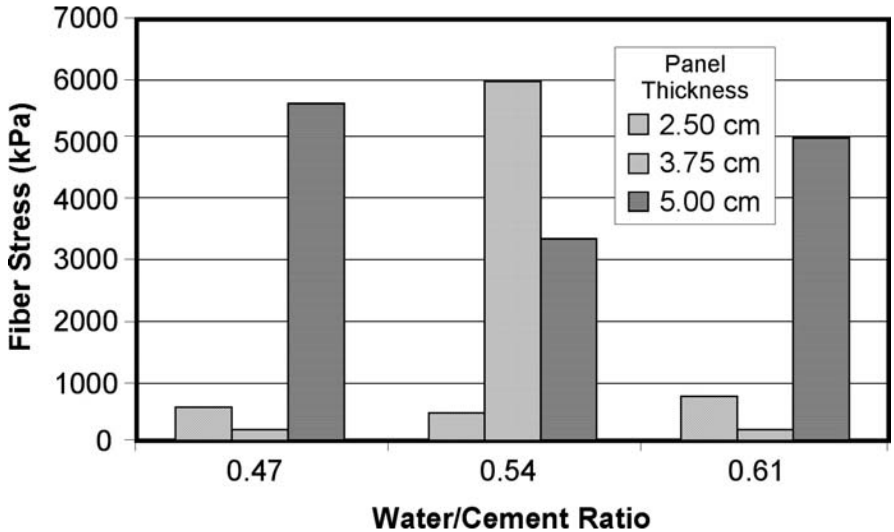


FIGURE 15. Fiber stress vs. W/C ratio, 0% rubber.



which is expected due to higher strength of concrete. The relationship between deformation and W/C ratio shown in Figure 17 depicts a general trend that, for concrete with 0% rubber content, the thicker the panel the less deformation it experiences at failure. The 5 cm thick panels failed without significant deformation in what appeared to be a sudden, brittle breakage with little or no measurable deformation.

Figures 18, 19, 20, and 21 are all plotted for 2.5 cm panels made with concrete containing 0%, 25%, and 50% rubber. It should be noted that the panels with

FIGURE 16. Shear stress vs. W/C ratio, 25% rubber.

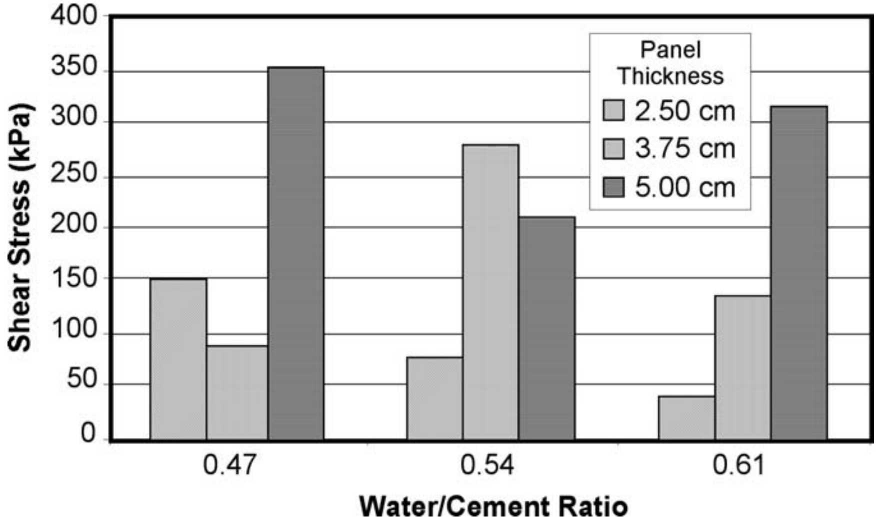
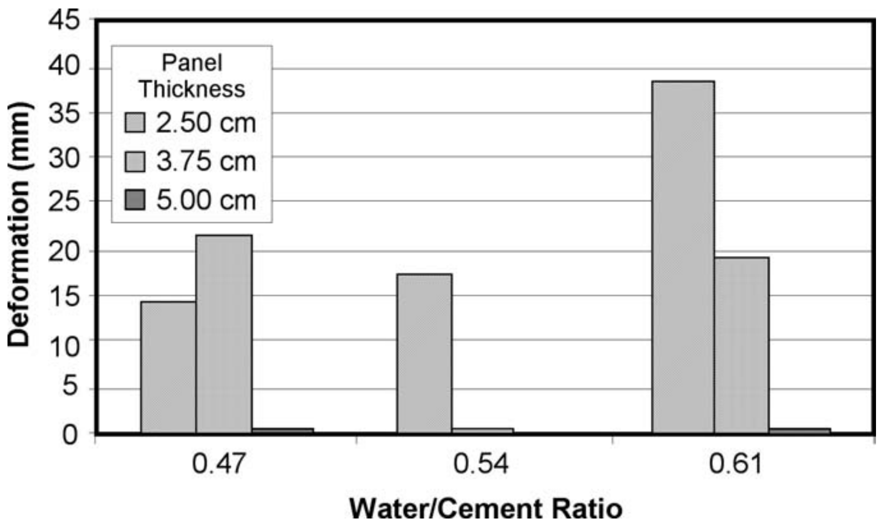


FIGURE 17. Deformation vs. W/C ratio, 25% rubber.



50% rubber content were made with W/C = 0.50, however they are clustered with the W/C = 0.47 for the purpose of comparison as given in Figures 18, 19, 20, and 21. Figure 18 shows the relationship between the modulus of rupture and W/C ratio. This figure shows a clear trend that, for a given W/C ratio the modulus of rupture decreases with the increase of rubber content in the mix. For a given rubber content, the modulus of rupture decreases with the increase of the W/C ratio. Figure 19 shows the relationship between the fiber stress and W/C ratio for panels with 0%, 25%, and 50% rubber content. The

FIGURE 18. Modulus of rupture vs. W/C ratio.

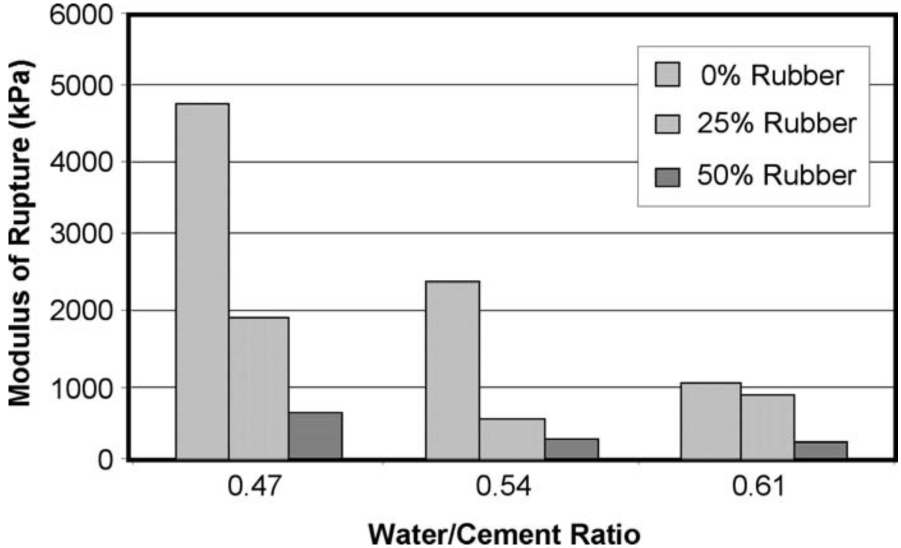


FIGURE 19. Fiber stress vs. W/C ratio.

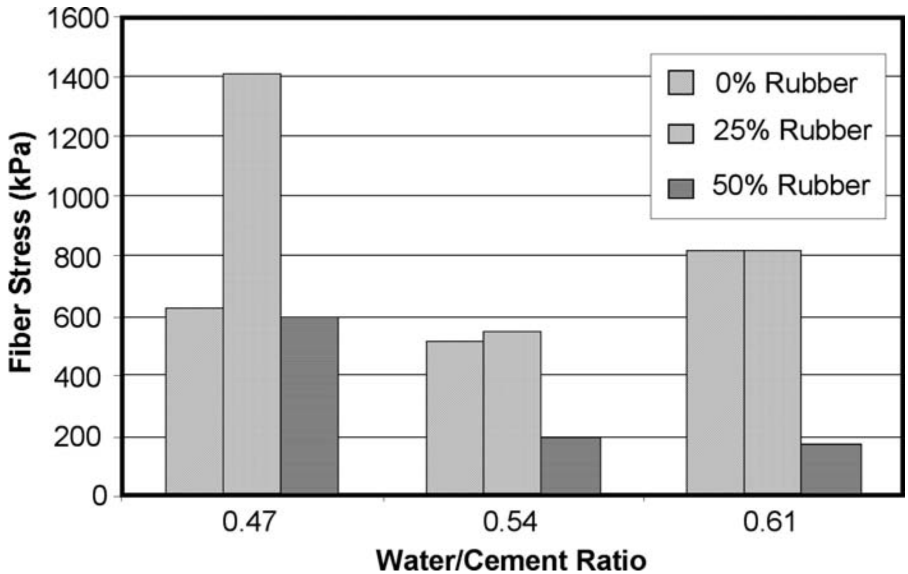


figure shows that the fiber stress decreases with the increase of the W/C ratio. This trend appears clearly in panels with higher rubber content (50%). Figure 20 shows the relationship between shear stress and W/C ratio. With a few exceptions, the overall trend is that, for a given rubber content, the shear stress decreases with the increase of the W/C ratio. Figure 21 shows the relationship between deformation at failure and the W/C ratio. No clear trend can be depicted from this figure. This can be attributed to the fact that

FIGURE 20. Shear stress vs. W/C ratio.

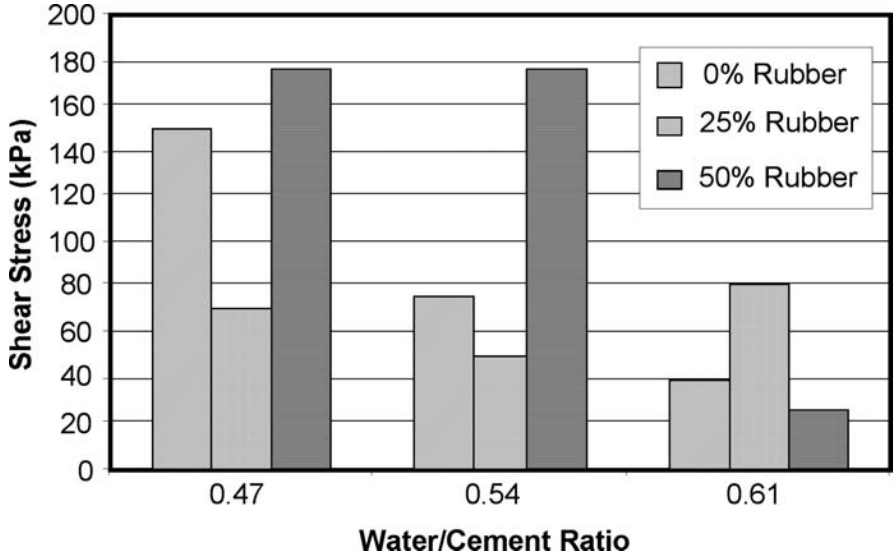
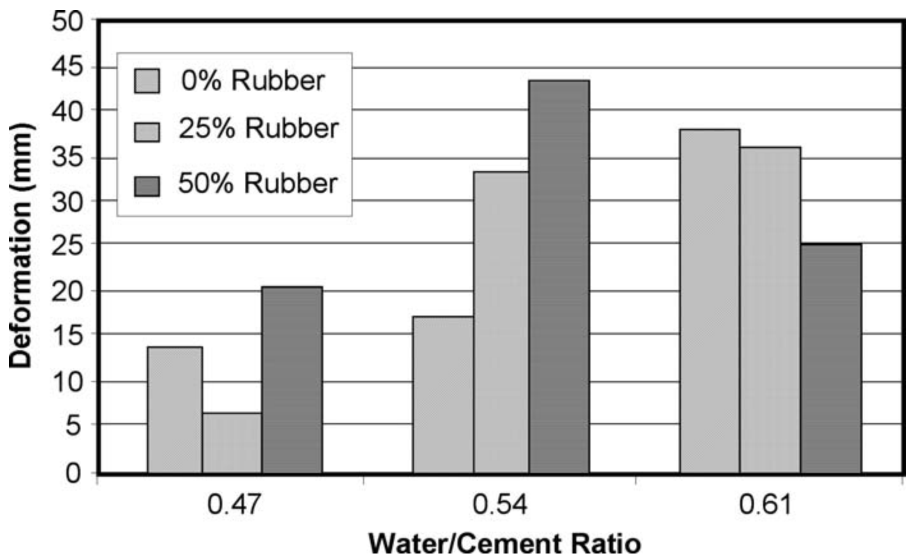


FIGURE 21. Deformation vs. W/C ratio.



deformation at failure was almost sudden and brittle in panels without rubber content, whereas panels with high rubber content experienced considerable deformation through a number of cycles in which the load peaked and declined several times.

Graphs similar to those shown in Figures 18, 19, 20, and 21 can be plotted for the 3.75 and 5 cm thick panels. However, these graphs can be plotted only for 0% and 25% rubber content because no 3.75 and 5 cm panels were made with 50% rubber.

CONCLUSIONS

Based on the results of the present investigation, the following conclusions can be drawn:

- The flexural resistance of concrete panels increases with an increase in the thickness of the section, and with a reduction in the water-cement ratio of the concrete.
- Panels made with rubberized concrete and reinforced with polymer grid behaved in a ductile manner where there were no signs of brittle failure.
- Considerable deformation was measured during load application where loaded panels fractured, but the panels remained intact as a result of the elongating polymer reinforcement.
- In addition to the lightweight benefit of rubberized concrete, it is concluded that the inherent ductility gained by using granulated rubber as one of the ingredients in concrete, combined with the use of polymer grid reinforcement, can be an effective tool to control the mode and nature of the brittle failure of conventional concrete. This seems to be a desired property of concrete serving in areas subjected to extreme loading conditions.
- Further research may be made on other important properties of the developed panels. These properties include sound absorption, heat conductivity, performance under freeze and thaw conditions, and fire resistance. These properties are of great importance in construction applications.

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