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Minimum splice length for fiber reinforced polymer (FRP) reinforcement in concrete applications

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Abstract:

Given the complexity of engineering and the need for inexpensive solutions in modern structures it is necessary to lay out a system of codes and regulations which govern the design and construction of these structures. For concrete applications the ACI codebook is used by engineers and contractors to efficiently design concrete structures that are both safe and economical. This codebook is by far the most popular design aid and it is constantly being updated and improved to meet the growing needs of our modern society.

In the past decade FRP applications in concrete have increased in popularity and demand due to the increased performance and decreased cost over steel reinforcement. However, much of the ACI codebook has very little to say about FRP and it is in need of being updated to take into consideration the application of this new material. My specific research is concerned with developing an ACI code detailing the minimum splice length necessary to adequately transfer the load from one FRP bar to another in a given situation. This code has been developed for steel, but not for FRP.

Introduction:

When reinforcement is spliced together within concrete it is necessary to overlap the bars enough for stress in one bar to be fully transferred to the other bar without a pullout failure or a shear failure in the concrete occurring. The load transfer occurs between the FRP fibers and the bond between the reinforcement and the concrete. The ACI 440 Commentary explains it this way: "In a reinforced concrete flexural member, the tension force carried by the reinforcement balances the compression force in the concrete. The tension force is transferred to the reinforcement through the bond between the reinforcement and the surrounding concrete. Bond stresses exist whenever the force in the tensile reinforcement changes. Bond between FRP reinforcement and concrete is developed through a mechanism similar to that of steel reinforcement and depends on FRP type, elastic modulus, surface deformation, and the shape of the FRP bar."[1] This minimum length is called the development length of the bar and is computed by Equation 1.

$$\ell_{bf} = \frac{d_b f_{fu}}{2700}$$

Equation 1 – Development length of a straight FRP bar[2]

In equation 1, l_{bf} is the development length, d_b is the diameter of one FRP bar and f_{fu} is the manufacturer ultimate stress value (or grade) for the bar. For this experiement, the development length of a FRP bar was computed to be 30 inches.
For steel reinforcement, there are two classes of splices given in the ACI code. The first, Class A, is equal to 1.0 times the development length (l_d) and the second, Class B, is equal to 1.3 times the development length. The following table shows when to use each case:

			Maximum percentage of A_f spliced within required lap length	
$A_{f,provided}$/ $A_{f,required}$	Equivalent to	f_f/f_{fu}	50%	100%
2 or more		0.5 or less	Class A	Class B
Less than 2		More than 0.5	Class B	Class B

Table 1 – Type of tension lap splice required[3]

This table is based on tests using steel reinforcement. Limited data are available for the minimum lap splice length for FRP applications. Available research[4] has indicated that a development length of $1.6l_d$ is necessary to reach ultimate capacity (Class B). ACI Committee 440 assumed that a value of $1.3l_d$ would be sufficient for a Class A splice[5] using FRP. Since the stress level for Class A splices, they reasoned, is not to exceed 50% of the tensile strength of the bar, using a value of $1.3l_d$ should be conservative[6]. The ACI Committee 440 acknowledges that more research is required in this area but recommends the values of $1.3l_d$ and $1.6l_d$ for Class A and B splices (respectively) in the mean time.

The research detailed below provides the empirically data desired by the ACI Committee 440 in the area FRP tension lap splices for Class A applications. The problem has been addressed by means of using FRP reinforced concrete beams in flexure. The four-point loading schemed created an area of constant moment over the full distance of the splice. This allowed for an experiment in which the stress transfer from one bar to the other in the splice could be analyzed. Strain gages where attached at critical locations on the bars to record needed stress levels. Four tests where done in total and a minimum splice length was determined using three different splice lengths ($0.75l_d$, $1.0l_d$, and $1.3l_d$) and a control beam with no splice. The adequacy of the ACI 440 recommendation was also verified.

Description of the experimental design:

The beam consists of 5000psi concrete with two #8 GFRP bars both of which are spliced at varying lengths at the center of the beam. #4 (grade 60) steel stirrups were used throughout the beam to keep shear failure from occurring. Two #6 (grade 60) steel rebar were used for compression reinforcement. The point loads were placed 44" apart to allow for a constant moment area over the entire splice length in every test. Each beam spanned 12 feet, was 18 inches tall, and 16 inches wide. The load was applied by a hydraulic jack pushing down on a steel beam. The steel beam transferred the force equally to the concrete beam through pin connections placed on wooden strips which kept concentrated loads from prematurely crushing the concrete by spreading the load out evenly. Figure 1 below shows the beam design used:

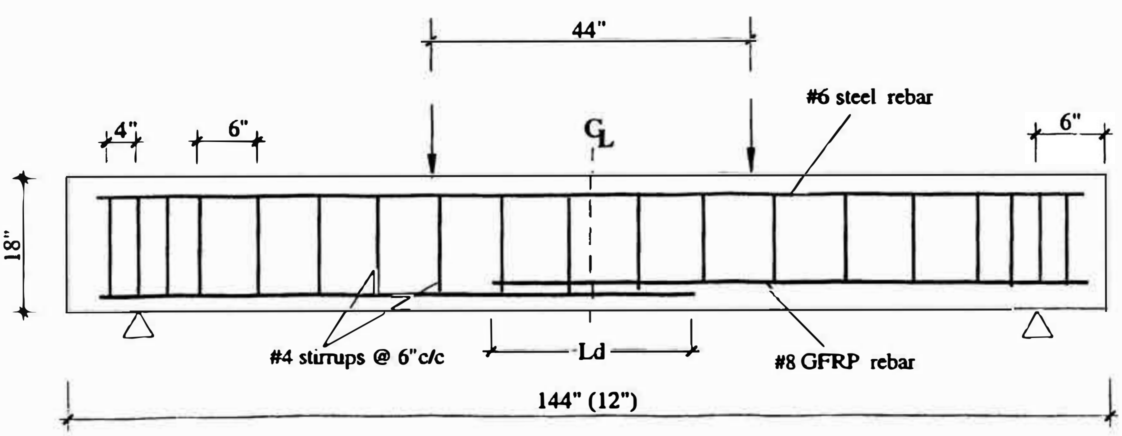

Figure 1 - Splice Design

Figure 2 below shows a cross section of the design. A cover of 1.5 inches was used and a distance of 2.5 inches was kept between the interior splices as shown.

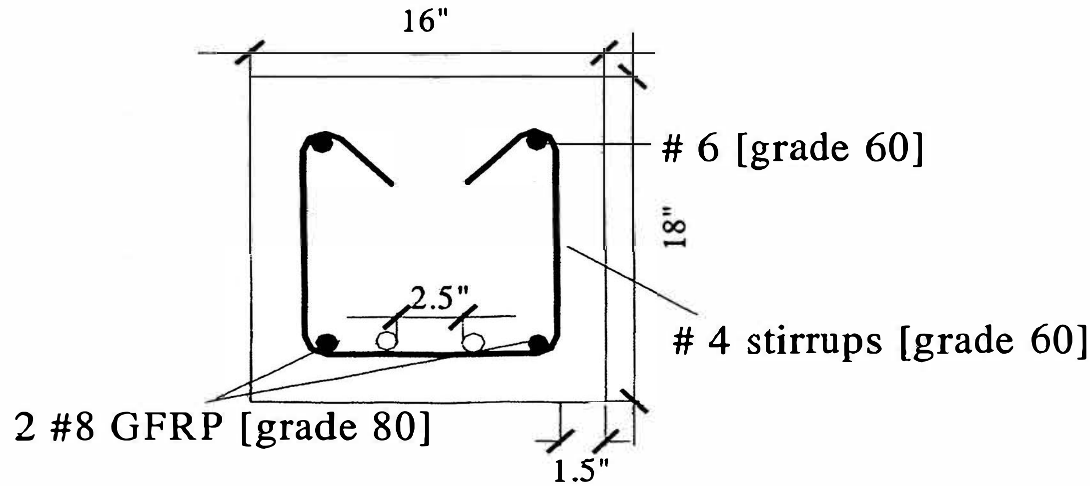

Figure 2 - Cross-section

Figure 3 below shows the actual test setup:

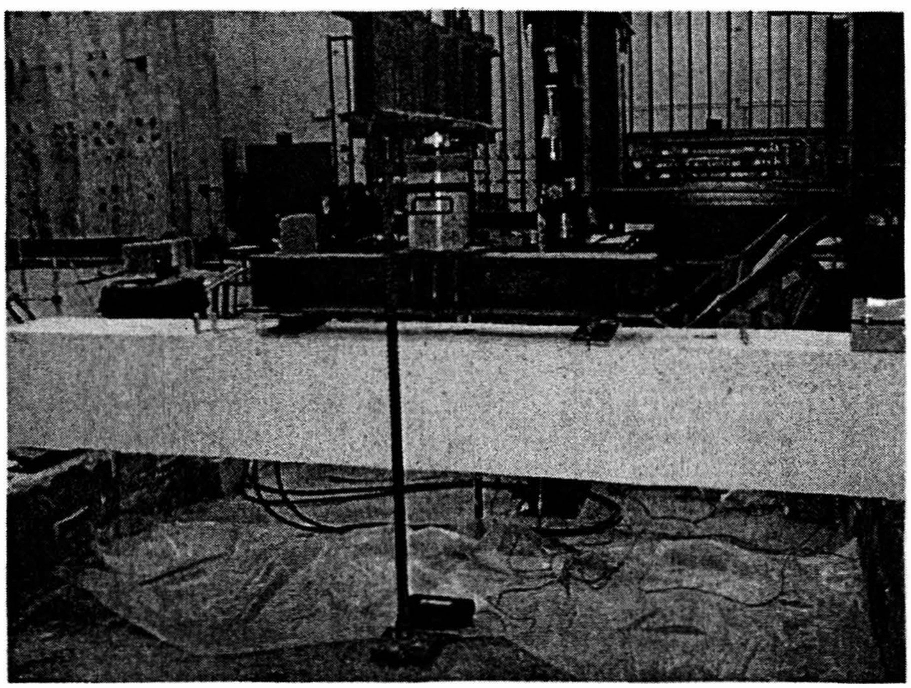

Figure 3 – Test Setup

Each specimen along with its modification factor (l_d) and resulting splice length is shown in Table 1 below:

Specimen	Modification Factor	Splice Length
So	o	No Splice
S1.0	$1.0L_d$	30 in
S1.3	$1.3L_d$	39 in
S0.75	$0.75L_d$	22.5 in

Table 2 -Test Matrix

<u>The Basis of the Design</u>

A 12 foot long beam design was chosen because it is a closer model to real world construction. Also, the large splice length required necessitated a long beam. The spacing between interior splices provided a minimum beam width for the design. Once it was determined that #8 GFRP bars would be tested and that a constant moment was needed over the entire splice length, the rest of the design followed naturally. The final formwork design is shown below:

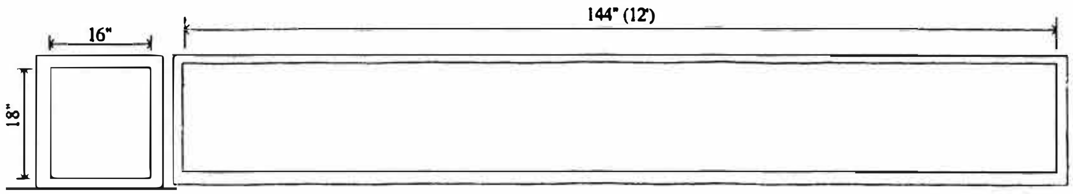

Figure 4 - Formwork

<u>Data Recording Method</u>

Four different types of data recording instruments were used in this experiment, Strain Gages, Linear Voltage Displacement Transducers (LVDTs), a Load Cell, and an Extensometer. The strain gages recorded the strain at different locations on the GFRP bars as well as the strain existing on the top of the beam at midspan. Figures 3-6 below show the locations of the interior strain gages for each beam:

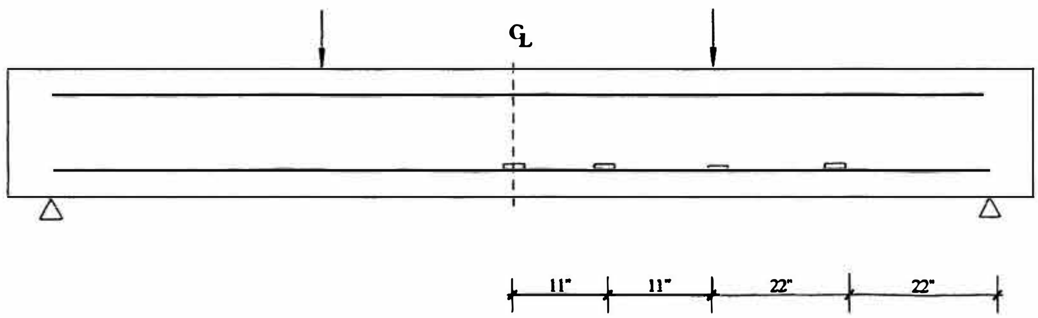

Figure 5 - Specimen S0

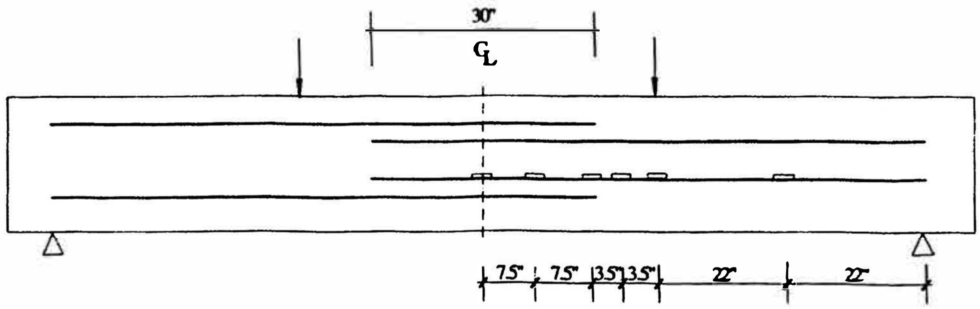

Figure 6 - Specimen S1.0

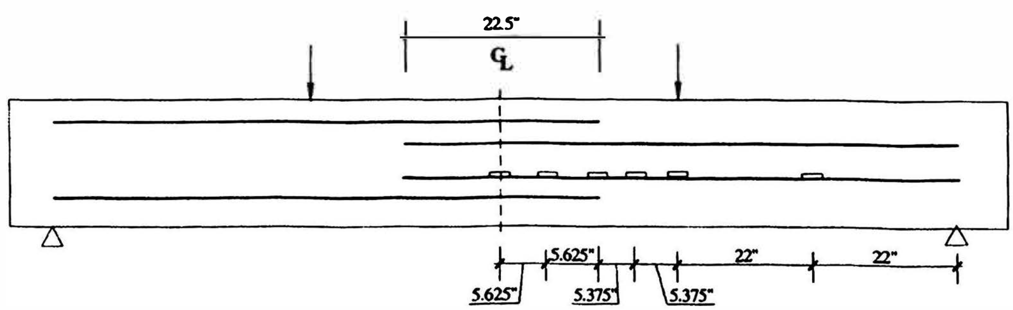

Figure 7 - Specimen S0.75

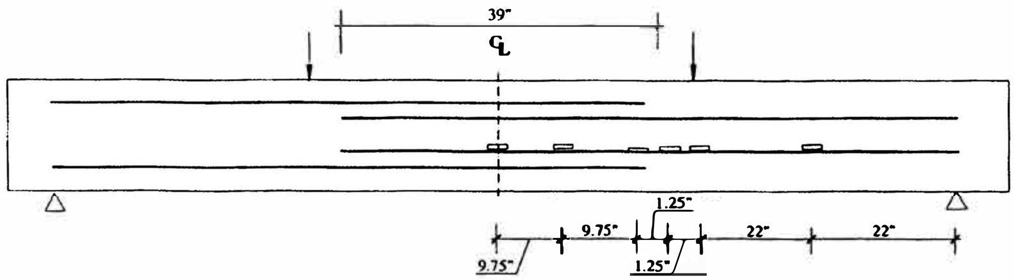

Figure 8 - Specimen S1.3

Each beam also had a strain gage installed on the compression concrete located at the midspan on the top of the beam as shown in the image below:

Figure 9 – Compression Strain Gage

LVDTs were installed at either end of each beam on the top and at the midspan of the beam on the bottom. Three LVDTs were used to get an accurate measurement of deflection which accounted for deflection of the supports during loading. One of the end LVDTs is shown below. The midspan LVDT is not shown but it was mounted near the bottom of the beam and was set to measure deflection from a plate that was securely attached to the bottom of the beam.

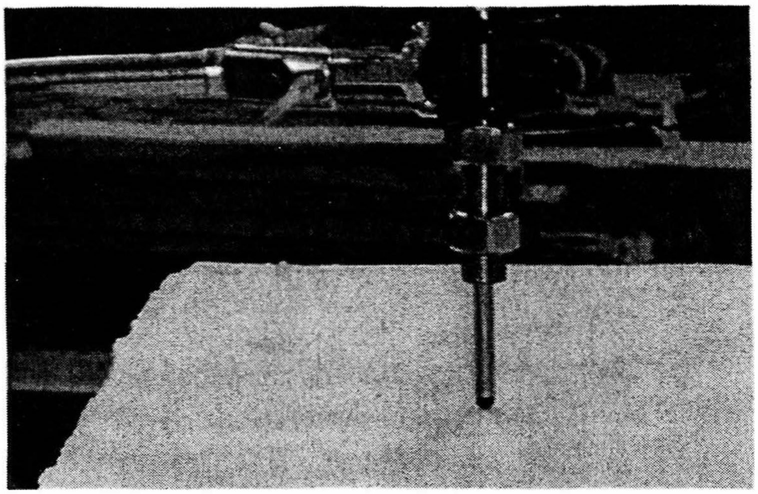

Figure 10 - LVDT

The Load Cell was placed between two steel plates which were themselves placed between the hydraulic jack and a steel reaction beam as shown. Since the hydraulic jack distributed its' force to the two point loads the reading on the load cell is twice the force found at the concentrated point load.

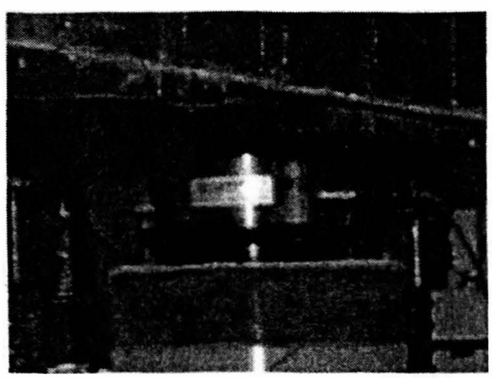

Figure 11 – Load Cell

To measure the Crack Width, an extensometer was installed on the first crack during testing. Two brackets were securely attached to the beam on either side of the crack and the extensometer was hung from these brackets as shown.

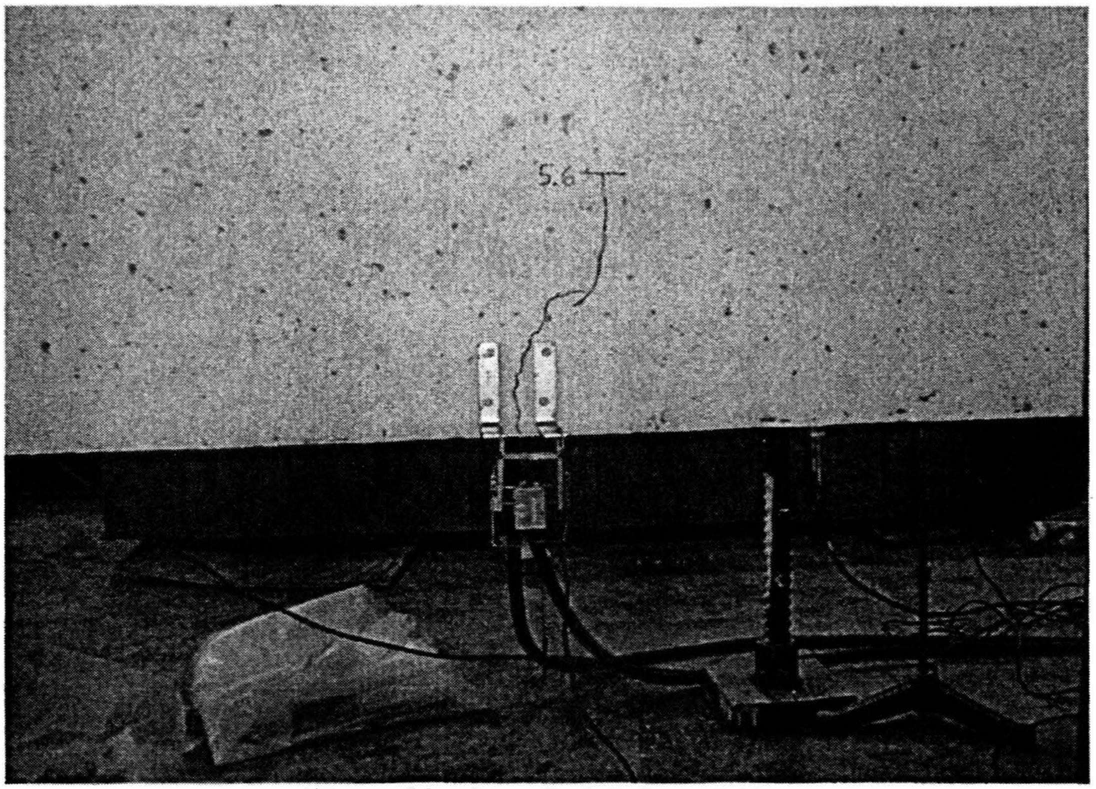

Figure 12 – Installed Extensometer

<u>Research Outcomes</u>

Each of the GFRP bars were rated to 80ksi ultimate stress (f^*_{fu}) by the manufacturer. The ultimate stress for design (f_{fu}) must be reduced by two factors, the environmental reduction factor (C_E) and the Creep rupture stress limit ($F_{f,s}$). The environmental reduction factor for Glass FRP (GFRP) is given as 0.8 from the environmental reduction factor table in the ACI 440 guide for design and construction of concrete reinforced with FRP shown below:

Exposure condition	Fiber type	Environmental reduction factor C_E
Concrete not exposed to earth and weather	Carbon	1.0
	Glass	0.8
	Aramid	0.9
Concrete exposed to earth and weather	Carbon	0.9
	Glass	0.7
	Aramid	0.8

Table 3 - Environmental reduction factor for various fibers and exposure conditions[7]

The design strength of FRP, environmental reduction factor, and manufacturer ultimate stress level is related by the equation:[8]

$$f_{fu} = C_E f^*_{fu}$$

where

f_{fu} = design tensile strength of FRP, considering reductions for service environment, psi;

C_E = environmental reduction factor, given in Table 7.1 for various fiber type and exposure conditions; and

f^*_{fu} = guaranteed tensile strength of an FRP bar defined as the mean tensile strength of a sample of test specimens minus three times the standard deviation ($f^*_{fu} = f_{u,ave} - 3\sigma$), psi.

Equation 1 – Environmental Reduction Factor

Therefore, the maximum design strength for grade 80 GFRP in our case is 0.8*80ksi which equals 64ksi. This value, however, does not take into account the loss in capacity due to creep rupture. Creep rupture occurs as FRP bars are subjected to a constant load over an extended period of time. They slowly fatigue and the resulting capacity decreases. The creep rupture stress for GFRP is given as 0.20 in the following ACI 440 table:

Fiber type	GFRP	AFRP	CFRP
Creep rupture stress limit $F_{f,s}$	$0.20f_{fu}$	$0.30f_{fu}$	$0.55f_{fu}$

Table ~~4 - Creep rupture stress limits in FRP reinforcement~~[9]

Therefore the maximum design capacity of a grade 80 GFRP bar after reducing it with respect to the environmental reduction factor and the creep rupture reduction factor is 0.8*0.2*80ksi which is equal to 12.8ksi. The modulus of elasticity for GFP is given as E5.7 by the following table:

	Modulus grade, × 10^3 ksi (GPa)
GFRP bars	E5.7 (39.3)
AFRP bars	E10.0 (68.9)
CFRP bars	E16.0 (110.3)

Table 5 - Minimum modulus of elasticity, by fiber type, for reinforcing bars[10]

Therefore the ultimate design strain in our case is given by the relationship e=f/E where e is the strain, f is the ultimate design stress, and E is the modulus of elasticity for GFRP. The ultimate design strain, therefore, is 12.8ksi/E5.7ksi which equals 0.00225in/in or 2250 micro-strains (με). Since the actual strain at failure was much higher than the maximum strain allowed in design it can be clearly seen in the graph below that all three splice schemes are adequate under normal design considerations.

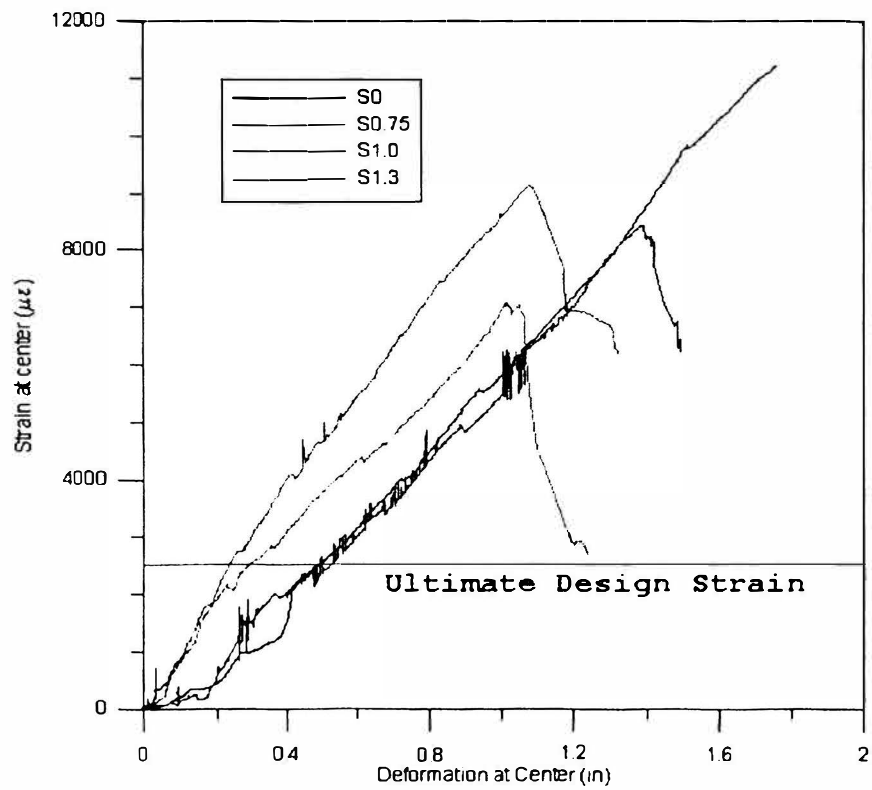

Figure 13 – Strain Vs. Deformation Graph

A beam analysis (shown below) shows that the maximum load capacity following ACI 440 design recommendations for the beams in under our analysis should be 12.8 kips as shown:

Flexure Analysis

Material Properties

f'c	4645	psi
fys	60000	psi
Es	29000	ksi
fyf	80000	psi
Efrp	5700	ksi
Ce	0.8	
Cr	0.2	

computed

eys	0.0021	in/in	eyf	0.0140	in/in
B1	0.82				
b	16	in			
h	18	in			

Beam	Afrp (in^2)	Diameter (in)	A's (in^2)	Diameter (in)	d (in)	d' (in)
1	1.58	1	0.88	0.75	15.25	2.375

Computed Values from above information

Beam	a (in)	efrp (in/in)	e's (in/in)	Mn (kip-in)
1	1.165	0.0047	-0.002	281.5

Point Load (from Mn=Pa) **a** 44 in

6.4 kips

Maximum applied design load seen by load cell is 6.4*2 = 12.8kips

were

f'c	strength of concrete from cylinder break after 30 days
fys	yield strength of streel
Es	modulus of steel
fyf	manufacturer FRP capacity
Efrp	modulus of GFRP
eys	yield strain of steel
B1	reduction factor which reduces "c" to an equivalent constant pressure compression zone termed "a"
b	width of compression face of member
h	height of member
eyf	manufacturer allowed strain capacity
Ce	Environmental reduction factor
Cr	Creep Rupture reduction factor

Figure 14 – Flexural Analysis Spreadsheet

The actual load capacity seen by the load cell is plotted below with respect to deflection:

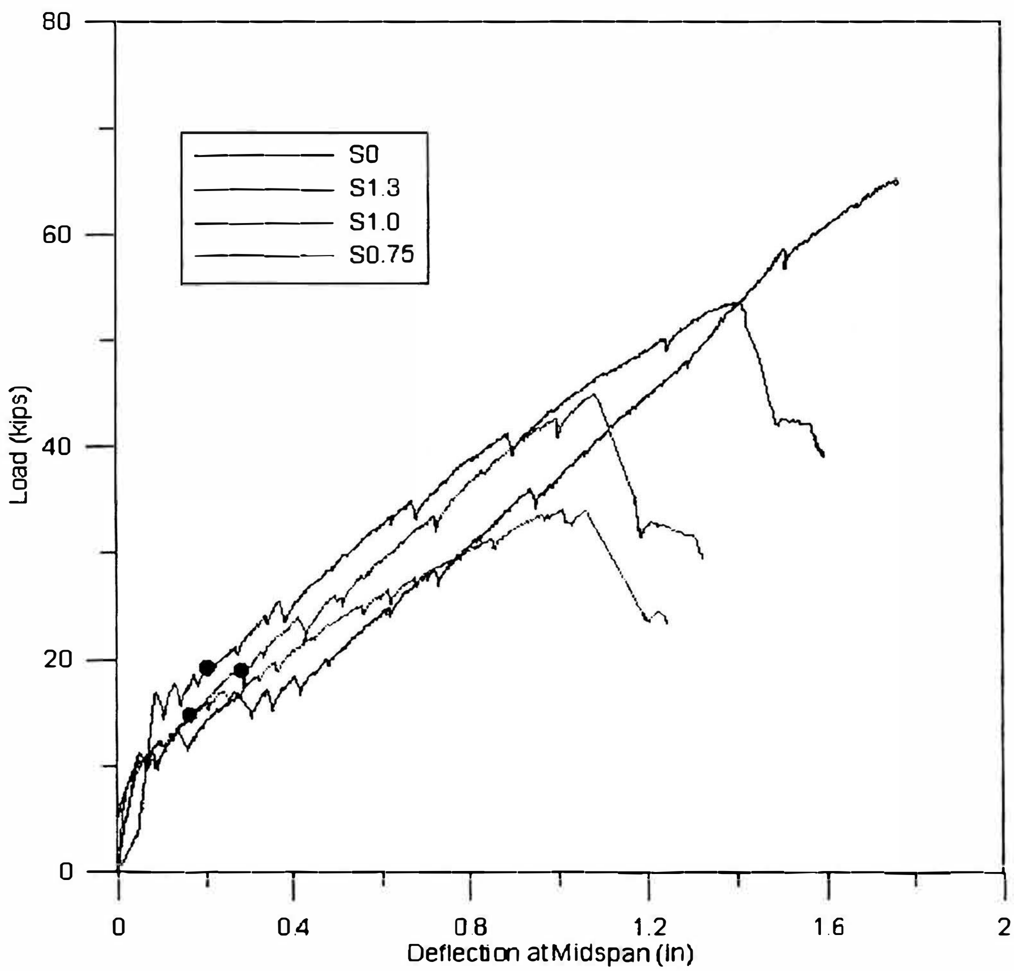

Figure 15 – Load vs Deflection Graph

All the beams performed well above the design capacity of 12.8 kips. However, the S0.75 beam experienced its first longitudinal crack (plotted as large dots on the graph above) shortly thereafter at 14 kips.

A look at the strain profiles for each beam also verifies above data. It is clear from these graphs that each beam was able to withstand strains much greater than the 2250 micro-strains derived from ACI 440 design considerations. Each graph shows strain values in the GFRP bars at a given load value up to capacity. The x-axis shows the locations of each strain gage with respect to the center of the beam.

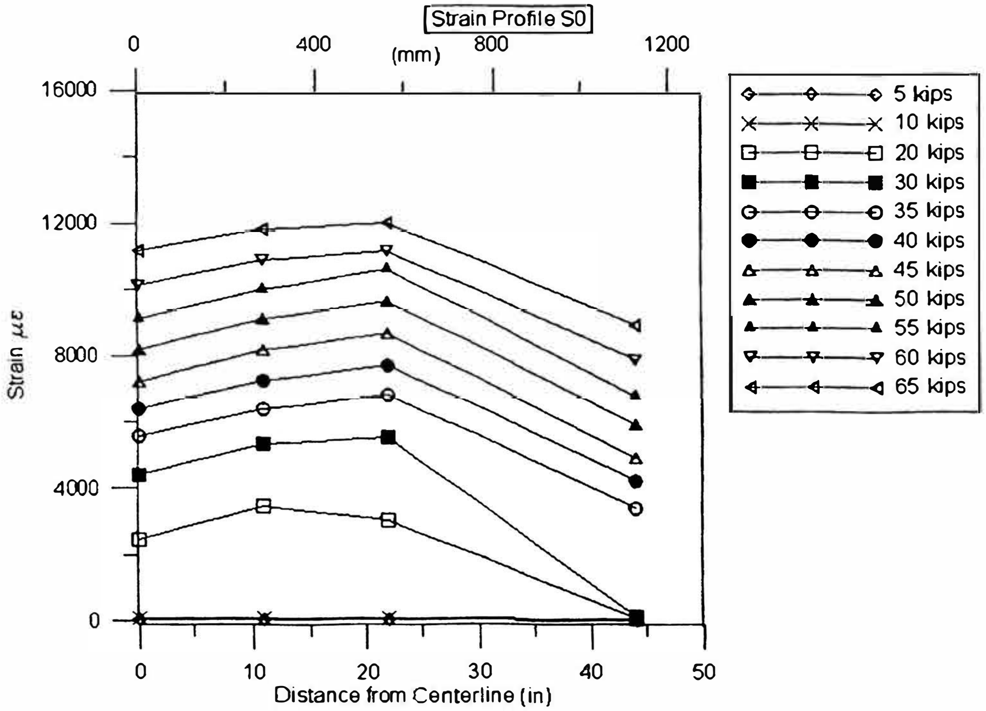

Figure 16 - Strain Profile for beam S0

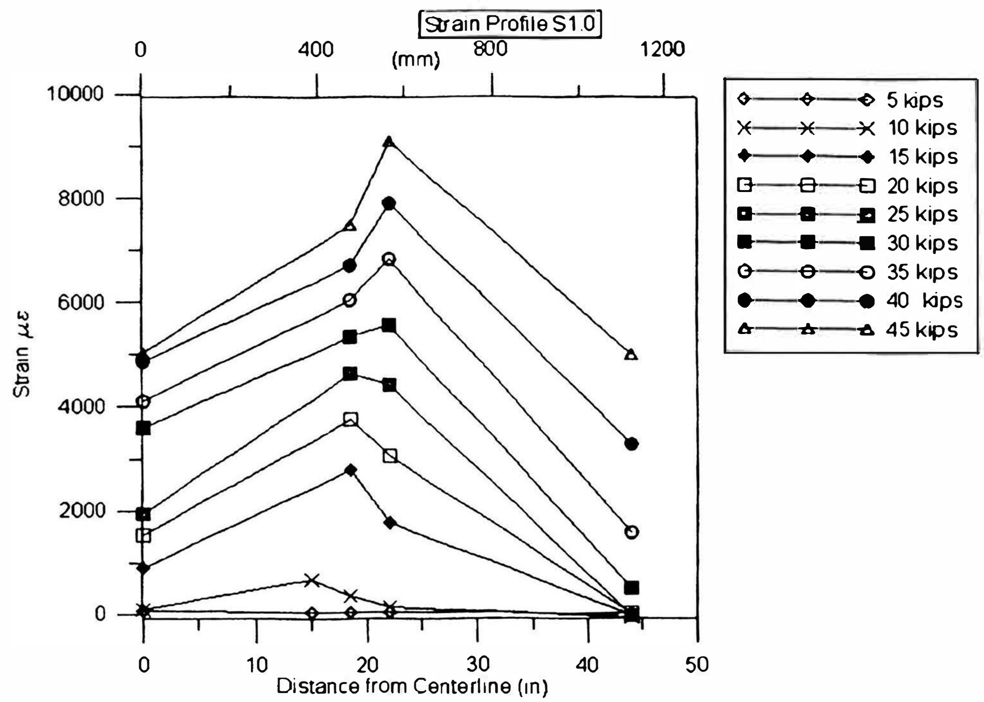

Figure 17 - Strain Profile for beam S1.0

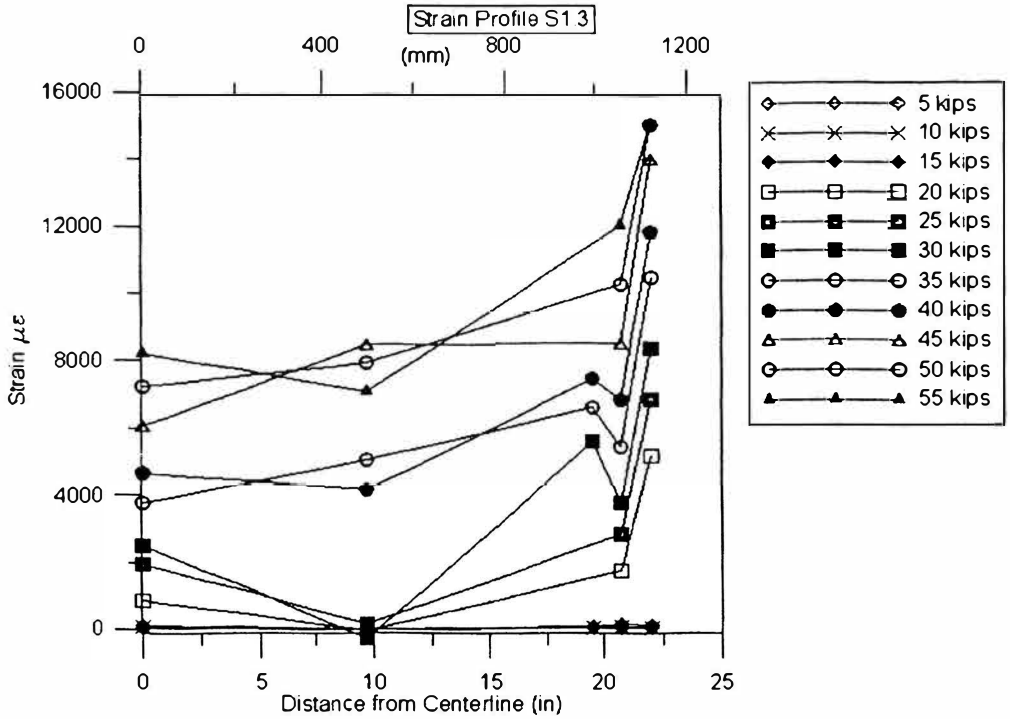

Figure 18 - Strain Profile for beam S1.3

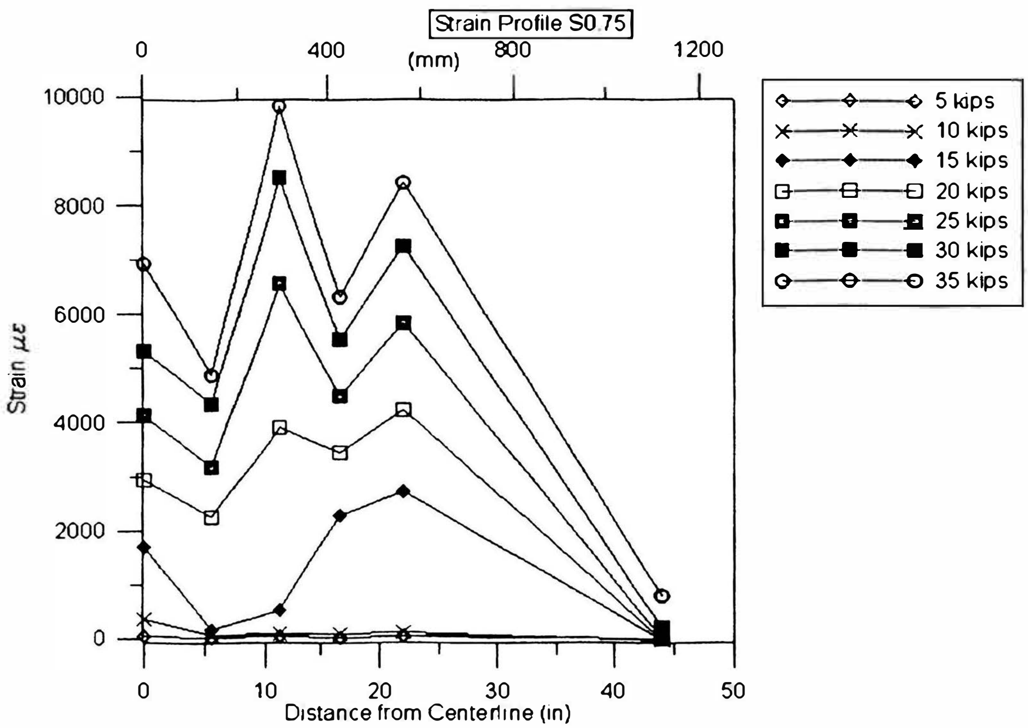

Figure 19 - Strain Profile for beam S0.75

Discussion of Results and Conclusions:

The above data verifies that the ACI 440 recommendation of $1.3l_d$ for a Class A splice was indeed conservative. In fact, a splice as small as $0.75l_d$ would be adequate. This short splice, however, is not recommended since the beam experienced longitudinal cracks shortly thereafter. Although more tests should be done to verify to data shown in this report it can be seen that a splice length of $1.0l_d$ is adequate for a Class A splice using FRP. This splice length is the same as that which is used for steel.

The strain gage data also verified the findings. However, this report has only considered #8 GFRP bars. It is recommended that similar experiments be conducted using varying sizes of GFRP bars to verify the results.

Since all beams met the required design capacity at varying splice lengths, it is impossible to determine an accurate minimum splice length for #8 FRP bars used in flexure. However, it has been shown that the ACI 440 recommendation of 1.3 times the development length was conservative and future designs could use a splice length of 1.0 times the development length according to the findings in this report.

It must be acknowledged that the longitudinal cracks shown in figure 15 above are approximate values. The actual crack occurred at + or – 3 kips from the recorded value. The inaccuracy in this value is due to the inability of the human observes to find the cracks the moment they occurred. Future researches should take this into account and seek solutions to this problem. One possible solution might simply be securing more observers during test times.

These findings have not been based on exhaustive research. More research must be done in this area to further verify and substantiate these findings.

Acknowledgements:

Thanks to Professor Pedro Silva P.E. for consultation and advising

Thanks to Michael Lubiewski for help with tests and analysis.

References:

[1] "Guide for the Design and Construction of Concrete Reinforced with FRP Bars", Reported by ACI Committee 440; ACI 440.1R-25;

[2] Ibid. ACI 440.1R-26; Equation 11-7

[3] "Guide for the Design and Construction of Concrete Reinforced with FRP Bars", Reported by ACI Committee 440; ACI 440.1R-27; Table 11.1.

[4] Benmokrane, B., 1997, "Bond Strength of FRP Rebar Splices," *Proceedings of the Third International Symposium on Non-Metallic (FRP) Reinforcement for Concrete Structures (FRPRCS-3),* Japan Concrete Institute, Sapporo, Japan, V.2, pp.405-412

[5] "Guide for the Design and Construction of Concrete Reinforced with FRP Bars", Reported by ACI Committee 440; ACI 440.1R-27; Chapter 11-3-Tension lap splice.

[6] Ibid.

[7] Ibid. ACI 440.1R-17; Table 7.1.

[8] Ibid. ACI 440.1R-16; Chapter 7.2 – Design material properties; Equation 7-1.

[9] Ibid. ACI 440.1R-23; Table 8.3.

[10] Ibid. ACI 440.1R-14; Table 5.1.