STRENGTHENING OF RC CHIMNEYS WITH FRP COMPOSITES

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ABSTRACT: Chimneys are usually subject to severe environmental conditions such as chemical attacks and extreme heat or cold. In addition, they are often subjected to operating conditions that change over time. Chimneys begin to deteriorate from the moment they are built, even before being put into service. As a result, many existing chimneys may require repair and strengthening after few years of operation. This paper describes how externally bonded fiber-reinforced polymer (FRP) reinforcement was used to increase the seismic resistance of an existing 140 m tall reinforced concrete (RC) chimney located in a power plant in Arkansas, United States. Design of the FRP reinforcement for this project was based on combining the design principles of ACI 440.2R and ACI 307 to develop an iterative design procedure. The design method, utilizing the concepts of strain compatibility and force equilibrium, allows for the determining the flexural capacity of chimney with and without openings. Chimney strengthening was achieved using vertical carbon FRP strips bonded to the inside and outside faces of the chimney. At the base of the chimney, strengthening was achieved using a combination of externally bonded FRP and concrete section enlargement.

1. Introduction

Strengthening of concrete chimneys have been historically achieved using concrete enlargement or externally attached steel, as a mean of extending the service life of deficient and deteriorated chimneys (Pinfold and Scott, 1997). Steel sheathing has the advantage of lower additional weight in comparison to concrete enlargement. However, the high cost of labor associated with the installation of steel plates, difficulties with splicing and welding, and considerable maintenance costs to prevent corrosion have limited the applications of this repair option. In the mid-eighties, Ohbayashi company and Mitsubishi Kasei company developed in Japan the concept of strengthening and retrofitting existing RC chimneys using carbon fiber strands and tapes (Kobatake et al., 1988, ACI 440.R-07). In this repair method, carbon FRP tapes were glued first to the concrete in the longitudinal direction so that flexural strength is enhanced. Carbon FRP strands were then installed around the chimney surface for additional lateral reinforcement. Between 1987 and 1994, a total of 28 sites were reported by Mitsubishi Kasei where retrofit had been conducted using either the tow winding or manual lay-up application processes (Emmons, 2008). In general, FRP have relatively high strength, are lightweight, and have considerably lower installation and maintenance costs. Combining these factors with the relatively simple installation procedure and the immunity to corrosion makes FRP a better choice over steel sheathing for the repair of tall chimneys.

In the United States, Europe and Middle East, the use of composites for chimneys and smoke stacks upgrade is still very limited, although increasingly gaining momentum. The reason for the limited use, despite the significant construction savings in time and materials, is due in large part to the lack of design guidelines and the limited experience of chimney designers with FRP materials. This paper attempts to overcome these limitations by presenting a simple methodology for flexural strengthening of annular chimneys with FRP. A case study is used to demonstrate the full scale application of FRP for increasing the seismic resistance of an existing concrete chimney.

2. Conventional RC Chimneys

ACI 307 provides design requirements for reinforced concrete chimneys, in which the nominal flexural strength is determined based on equilibrium of forces, strain compatibility and the constitutive behavior of concrete and steel materials. This ultimate strength design approach requires that:

$$\phi R_n \geq \sum_{i=1}^{n} \alpha_i S_i = S_u \tag{1}$$

where

ϕ = Strength reduction factor;

R_n = Nominal strength;

S_i = Loads acting on the structure (e.g. Self weight, wind, earthquake, etc.)

α_i = Load factors accounting for the predictability of design loads.

ACI 307 requires that the nominal moment capacity be multiplied by a strength reduction factor ϕ equal to 0.8 for vertical strength and 0.90 for circumferential strength. The nominal moment strength of a circular chimney cross section is obtained based on the design assumptions of ACI 318 except that the maximum tensile strain in the steel is limited to 0.07. If the steel strain limit is reached first, then the maximum concrete strain computed from the linear strain diagram at failure is less or equal than 0.003. In addition, ACI 307 assumes rectangular concrete compressive stress block even when the maximum concrete compressive strain is less than 0.003. In these instances, the assumed uniform compressive stress is modified by a correction factor referred to as parameter Q (ACI 307, 2008).

Openings in the tension zone are usually ignored when calculating the flexural capacity, because any bars cut by an opening must be replaced with equal or larger amount of bars placed on the sides of the opening. However, the effects of opening on capacity must be considered when the opning side is in compression.

3. FRP Strengthening of RC Chimneys

For the current project, design of the FRP flexural reinforcement was achieved in accordance with ACI 440.2R. To avoid plastic deformations at service, the stress in the steel at service was limited to 80% of the yield stress. Similarly, to avoid failure of the FRP at service, the service stress in the FRP was maintained below its creep-rupture stress limit.

Design of FRP was based on principles of equilibrium and strain compatibility. For ultimate strength, ACI Committee 440 refers to the strength reduction factor in ACI 318 building code and introduces an additional strength reduction factor, Ψ_f, applied to the contribution of the FRP. The additional strength reduction factor, Ψ_f, addresses different reliability and failure modes observed in FRP strengthened members, as compared with steel reinforced members. While a constant flexural strength reduction factor is given in ACI 307, ACI 440.2R uses a flexural strength reduction factor that is a function of the ductility of the section, expressed in terms of the tensile steel strain at ultimate capacity. As such, the flexural strength reduction factor for FRP strengthened RC chimney can be expressed as:

$$\phi = \phi_{ACI-318} = \begin{cases} 0.9 \ for\ \varepsilon_t \geq 0.005 \\ 0.65 + \dfrac{0.25(\varepsilon_t - \varepsilon_{sy})}{0.005 - \varepsilon_{sy}} \ for\ \varepsilon_{sy} < \varepsilon_t < 0.005 \\ 0.65 \ for\ \varepsilon_t < \varepsilon_{sy} \end{cases} \tag{2}$$

where ε_t is the net strain in the extreme tension steel at nominal strength and ε_{sy} is the yielding strain of steel. Equation (2) sets a low strength reduction for ductile sections, and higher strength reduction for brittle sections where the steel does not yield, and provides a linear transition between the two extremes.

Figure 1 illustrates the stress and strain distribution for a circular RC chimney cross-section, strengthened with externally bonded FRP composites considering the case of an opening located in the compression zone of the section. However, the calculation of the nominal flexural capacity was based on the minimum value calculated for both the cases of opening in compression and tension.

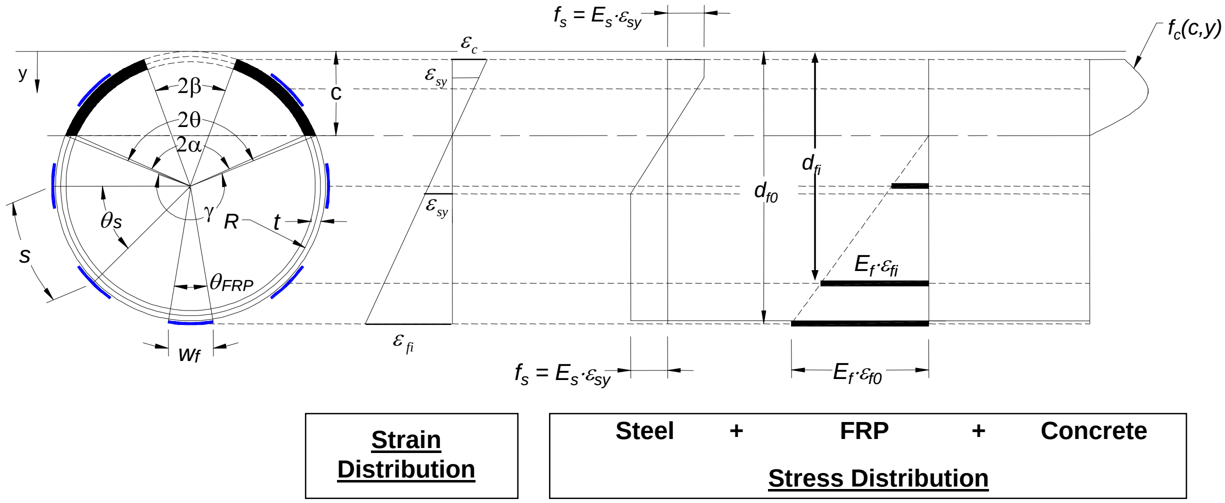


Fig.1- tress-Strain Distribution (Opening in Compression)

Using force equilibrium, the equation for the forces in the vertical direction can be written as:

$$P_u = C + S_3 + S_4 - S_1 - S_2 - nt_f w_f E_f \varepsilon_{f0} - \sum_{i=1}^{\frac{n_{st}-1}{2}} 2nt_f w_f E_f \varepsilon_{fi} \tag{3}$$

where:

C = total force in concrete compressive stress block;

P_u = factored vertical load acting on section;

S_1 = tensile force where steel stress is below yield point;

S_2 = tensile force where steel stress is at yield point;

S_3 = compressive force in steel where stress is below yield point;

S_4 = compressive force in steel where stress is at yield point;

n = number of FRP plies;

t_f = thickness of one FRP ply;

w_f = width of each FRP strip;

n_{st} = number of FRP strip. The total number of strips must be odd;

ε_{f0} = strain in the bottom FRP strip;

ε_{fi} = strain in the i^{th} FRP strip. Strain is taken as zero if the strip is in compression;

The maximum strain level that can be achieved in the FRP reinforcement is governed by either the strain level developed in the FRP at concrete crushing, steel strain of 0.007, or FRP debonding strain. These three conditions are given in Equations 4a, 4b, and 5, respectively.

$$\varepsilon_{f0} = \varepsilon_c \frac{d_{f0} - c}{c} - \varepsilon_{bi0} = \varepsilon_c \frac{2R - c}{c} - \varepsilon_{bi} \le \varepsilon_{fd} \tag{4a}$$

$$\varepsilon_{fi} = \varepsilon_c \frac{d_{fi} - c}{c} - \varepsilon_{bii} \tag{4a}$$

where ε_{bi0} and ε_{bii} represent the initial strain level in the concrete substrate and can be determined from elastic analysis of the existing member. The design strain ε_{fd}, represent the strain level at which debonding may occur and is given in ACI 440.2R-08 as follows:

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f'_c}{nE_f t_f}} \leq 0.9\varepsilon_{fu} \tag{5}$$

The nominal moment capacity of the section can then be determined from force equilibrium as follows:

$$M_n = P_u R\cos(\alpha) + C' + S'_1 + S'_2 + S'_3 + S'_4 + \Psi_f n t_f w_f E_f \varepsilon_{f0}\left(d_{f0} - c\right) + \Psi_f \sum_{i=1}^{\frac{n_{st}-1}{2}} 2n t_f w_f E_f \varepsilon_{fi}\left(d_{fi} - c\right) \tag{6}$$

where C', S_1', S_2', S_3', and S_4' represent the moments of C, S_1, S_2, S_3, and S_4 about the neutral axis, respectively. The amount of FRP reinforcement is determined through iteration until Equation (1) is satisfied.

4. Case Study: Power Plant Chimney, Arkansas, USA

The 140 m reinforced concrete chimney is located in a power plant in Arkansas. The sloped wall circular chimney has an average diameter of 18 m in the first 18 m of the chimney that then linearly decreases to 10 m at the top of the chimney. Similarly, the thickness of the chimney wall is constant for the first 18 m of the chimney and then decreases linearly along the height from 1200 mm at the base to 250 mm at the top. The chimney required flexural strengthening to address changes in the seismic design criteria. Analysis of the existing chimney indicated that the level of flexural deficiency varies with height, as shown in Figure 2.

Considering the complexity of this type of project, a specialty repair contractor with many years of experience with FRP applications and in-house engineering capacity, was selected to perform the work. Several options were considered by the specialty contractor for strengthening the chimney - including the installation of an internal reinforced concrete jacket or vertical steel plates spaced around the inside face of the wall and bonded to the concrete wall with epoxy adhesive and steel mechanical anchors. Although all of these options could increase the strength of the chimney, they were not economically viable solutions for the owner. A more cost-effective strengthening option was achieved by using a combination of externally bonded carbon FRP, concrete jacketing and steel plating. The main benefit of using FRP is that the additional moment capacity was provided to the chimney stack without adding significant weight. As such, no upgrade was necessary for the existing foundation. Detailed analyses were performed at various sections along the height of the chimney using the FRP design procedures described earlier. Because of the variable level of deficiency, the amount of FRP varied along the height of the chimney. While some sections required multiple plies of FRP on both sides of the chimney, other sections only required a single ply on one or both sides of the chimney wall. A schematic of the adopted strengthening solution is shown in Figure 3.

Because of the FRP debonding design limit, it was not possible to achieve the required strength increase in the bottom 3 m of the chimney. As such, the required strength increase at this location was achieved using reinforced concrete enlargement. For this, the concrete surface was roughened to minimum 6mm amplitude and steel dowels were installed to improve the horizontal sheer force transfer between the existing and the new concrete wall sections. Mild steel reinforcement was then installed and the concrete placed using the form-and-pump technique to ensure composite behavior of the new jacket with the existing chimney wall. Another critical location was a 4 m wide wall opening located from elevation 8 m to elevation 20 m. At this location, flexural strengthening of the chimney section also was not feasible using FRP alone. The required strength was achieved using a combination of steel plates around the opening and CFRP applied to the inside and outside faces of the chimney. The steel plates were mechanically anchored into the chimney wall through the FRP.

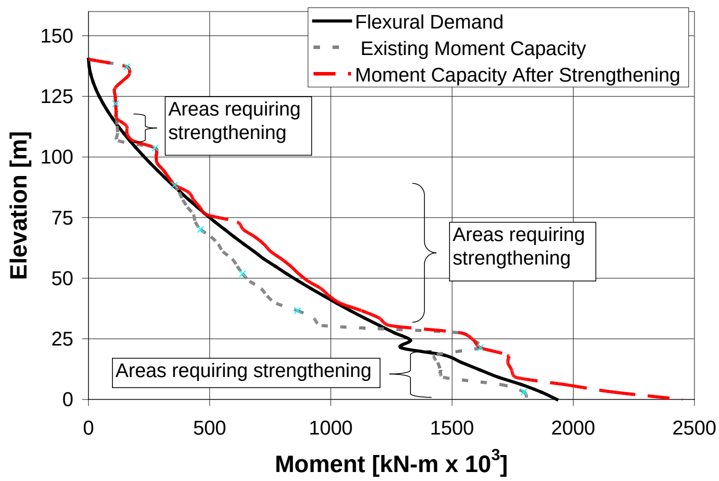
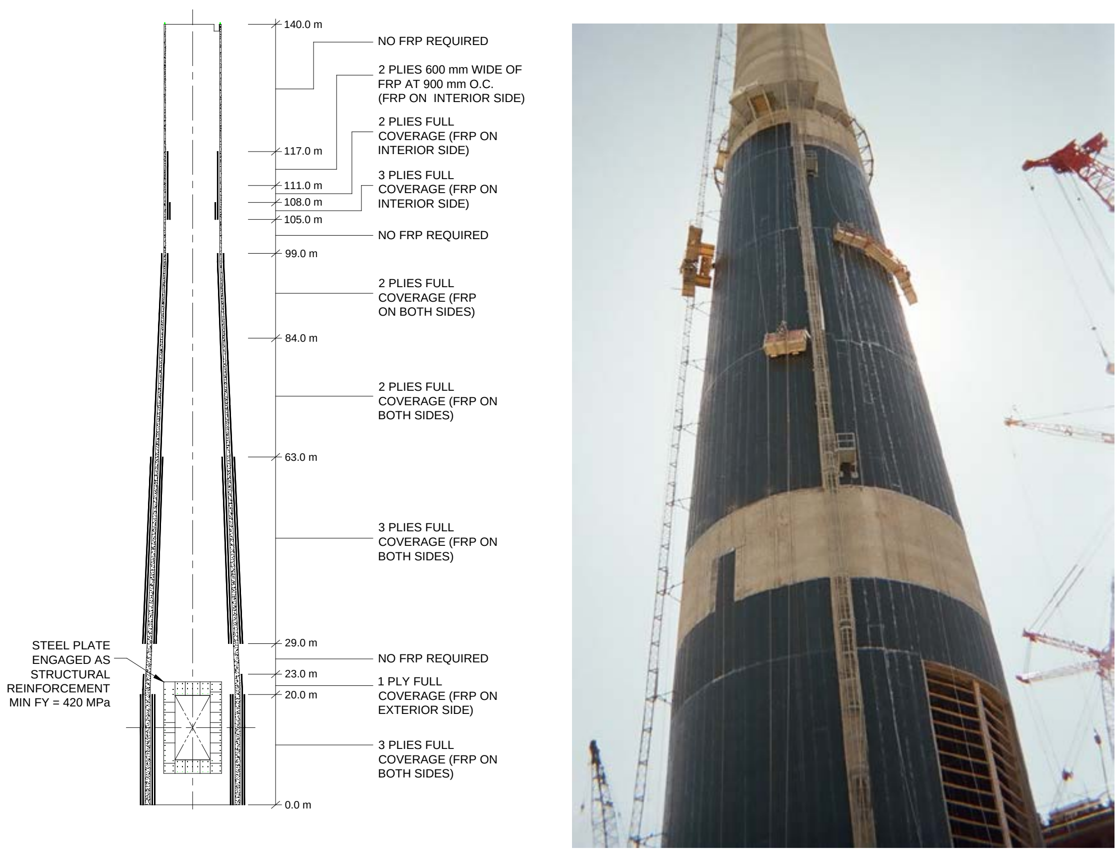


Fig. 2 – Demand vs. Capacity for Un-Strengthened and Strengthened Chimney



(a) FRP Layout (b) Installation

Fig. 3 – Chimney Strengthening

The FRP solution used to provide flexural strengthening was tailored to the deficiency shown in Figure 2. It is interesting to note that, for this particular chimney, the 0.8 ϕ - factor used by ACI 307 always governed the calculations. This fact implies that even after strengthening with FRP, the structure maintained a relatively ductile behavior. This condition, however, cannot be generalized to all chimney FRP strengthening applications since it depends on the section geometry, amount of existing steel reinforcement, and required strength increase.

5. Conclusions

This paper presents a design methodology for the strengthening of RC chimneys using externally bonded FRP composites. The FRP design approach that was based on the design requirements of ACI 307-08 for concrete chimney and design methodology of ACI 440.2R-08 allowed for the determination of the required amount of FRP reinforcement through simple iteration procedures that can be implemented in a spreadsheet or any other mathematical calculations software.

A case study was presented in the paper that demonstrated how the proposed design procedure was used to develop the required FRP reinforcement to strengthen an industrial chimney deficient in flexure. The flexural deficiency was caused by an increase in seismic load demand. The strengthening solution that consisted of a combination of externally bonded carbon FRP, concrete jacketing and steel plating provided the most cost-effective solution. Due to its light weight, FRP increased the bending capacity of the chimney without increasing its weight. As such, no upgrade was necessary for the existing foundation. The case study also emphasizes that condition assessment and design of strengthening solutions for existing chimney structures are infinitely more complex than new construction. Typically, challenges arise because of unknown factors associated with the structural state - such as in-place reinforcing steel amount and layout (as-built conditions), load path, and material properties. The degree to which the upgrade system and the existing structural elements share the loads must be evaluated and addressed properly in the upgrade design. Implementation procedures are very critical to ensure success of strengthening projects. Special attention should be paid to detailing due to its direct effect on the effectiveness and durability of structural upgrades.

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