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Strengthening of RC Chimneys with FRP Composites

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ABSTRACT: Chimneys begin to deteriorate exponentially 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. In addition, many chimneys that were constructed for compliance with the governing code may now require upgrade to satisfy the seismic performance requirements of a newer, and usually stricter, code. One example for the latter case is the reinforced concrete (RC) chimney described in this paper.

The design of chimneys in the United States is typically achieved in accordance with ACI 307, which offers a simple design procedure for RC annular sections under the assumption that the steel reinforcement is uniformly distributed around the section. When fiber reinforced polymer (FRP) laminates are added to the cross-section, the bending behavior and failure is mostly governed by the properties and the linear tensile behavior of the FRP. Capacity of the FRP upgraded section of the chimney should therefore be determined using integration that accounts for the strain variation in the FRP placed around the chimney section. Since bending due to lateral forces can occur in any direction, the design of sections with openings should account for the worse condition.

This paper describes how externally bonded FRP reinforcement was used to increase the seismic resistance of an existing 140 m RC chimney located in a power plant in the United States. The design approach used for this project incorporated the design principles of ACI 440.2R into ACI 307 to develop an iterative FRP design procedure. The design method, utilizing the concepts of strain compatibility and force equilibrium, allows for the determination of the flexural capacity of a circular chimney with and without openings. The carbon FRP strips were bonded to the inside and outside faces of the chimney - serving as vertical tension reinforcement. Since carbon FRP added minimal weight to the structure, it did not increase the seismic demand. At lower chimney sections, strengthening was achieved using a combination of externally bonded carbon-based FRP and concrete section enlargement.

1 BACKGROUND

Repairs to concrete chimneys by concrete enlargement or steel sheathing have been carried out for many years 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 to install the usually heavy steel plates, difficulties with splicing, and considerable maintenance costs to prevent corrosion have limited the applications of this repair option. Fibers reinforced polymers (FRP), on the other hand, have relatively high strength, are lightweight, and have a consistently lower cost. Combining these factors with the relatively simple installation procedure and the immunity to corrosion makes FRP the material of choice over steel sheathing for the repair of tall chimneys.

In the mid-eighties, Ohbayashi Co. and Mitsubishi Kasei Co. in Japan developed the concept of strengthening and retrofitting existing RC chimneys using carbon fiber reinforced polymer (CFRP) strands and tapes (Katsumata et al., 1990, ACI 440.R-96). In this repair method, CFRP tapes were glued first to the concrete in the longitudinal direction to enhance flexural strength. CFRP strands were then impregnated with resins and spiral-wound around the surface for additional lateral reinforcement. The primary function of the spiral wound strand is to improve shear capacity and ductility of the chimney (ACI 440.R-96). FRP installation using automated wet winding drew considerable attention to the potential use of composites for the retrofit of civil infrastructure and several chimney upgrade projects were completed using this technology. A few years later, the automated wet winding method was completely replaced by the use of carbon fiber sheet material, which is applied externally to the concrete using a manual lay-up process. Between 1987 and 1994, Mitsubishi Kasei reported a total of 28 sites where retrofit had been performed using either the tow winding or manual lay-up application processes (Emmons, 2008).

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 tries to overcome these limitations by a presenting a simple methodology for the 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 DESIGN PHILOSOPHY

2.1 Conventional Reinforced Concrete Chimneys

In the United States, ACI 307 provides design requirements for reinforced concrete chimneys. ACI 307 follows the ultimate strength design (USD) approach in which the nominal strength (M_n, P_n, V_n), is determined based on static equilibrium, strain compatibility and the constitutive behavior of concrete and steel materials. The USD approach requires that:

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

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 specifies a constant ϕ –factor for combined flexural and axial loads on the chimney. In particular, ACI 307 requires that the nominal moment should 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. The concrete strain at crushing is limited to 0.003. If the steel fracture limit is reached first, the maximum concrete strain computed from the linear strain diagram 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, however, the assumed uniform compressive stress is modified by a correction factor referred to as parameter Q (ACI 307, 2008).

A simple iterative procedure that allows for the determination of the nominal flexural capacity of the reinforced concrete chimney with and without openings is also given in ACI 307. Openings in the tension zone are usually ignored when calculating the flexural capacity, because the tensile strength of the concrete is neglected, and any bars cut by an opening must replaced at the sides of the opening. Similarly, openings in the compression zone are ignored in calculations of the forces in the compression reinforcement also because the cut bars must be replaced at the sides of the openings. However, the effect of the opening on the section of concrete in compression must be considered. These assumptions also are reasonable when the flexural capacity is supplemented with FRP.

2.2 FRP Strengthening of Reinforced Concrete Chimneys

2.2.1 General concepts

The FRP design approach presented in this paper is based on ACI 440.2R, which provides guidance for the selection, design, and installation of FRP systems for external strengthening of concrete structures (ACI 440.R, 2008).

When designing FRP strengthening, the first check to evaluate FRP as an option is typically to verify the minimum existing strength limit. The member considered for strengthening must be able to support at least 1.1 times the design dead loads and 0.75 times the sustained design live loads. This limit is to ensure that in case the FRP is lost due to damage, fire or others, the member will still maintain sufficient structural capacity until the damaged FRP has been repaired.

For both flexural and shear strengthening, to prevent debonding the strain level in the FRP reinforcement at the ultimate-limit is limited to an upper value. This requirement recognizes that laminates with greater stiffness or thickness are more prone to delamination. In addition, to avoid plastic deformations at service, the stress in the steel at service must be limited to 80% of the yield stress. Similarly, to avoid failure of the FRP at service, the service stress in the FRP must be maintained below its creep-rupture stress limit.

2.2.2 Strength reduction factors

When designing FRP strengthening for RC chimneys, it is possible to develop equations for strength and resistance using the principles of equilibrium and strain compatibility. Knowing that FRP has different reliability as a material and as an externally bonded element than concrete and steel, the designer must decide how to implement separate ϕ factors for concrete/steel and FRP into the overall strength formulation. These formulations can be based on either a suitable overall composite ϕ factor that addresses the reliability of all the components, or a set of individual ϕ factors that apply separately to the strength contribution of each material (Kelley et al. 2000). ACI Committee 440 uses the strength reduction factor of the ACI 318 building code and introduces an additional strength reduction factor, Ψ_f, to the contribution of the FRP. The additional strength reduction factor, Ψ_f, accounts for the different reliability and modes of failure observed in FRP strengthened members (ACI 440.2R, 2008). ACI 440.2R is also recommended for chimney applications.

In addition, while a constant flexural strength reduction factor is given by ACI 307, ACI 440.2R uses a variable flexural strength reduction factor that accounts for the ductility of the section, as expressed in terms of the tensile steel strain at ultimate. The approach recommended by the authors is to use the strength reduction factor of ACI 440.2R, with ACI 307 as an upper boundary. As such, the flexural strength reduction factor for an FRP strengthened RC chimney can be expressed as:

$$\phi = \min\left(\phi_{ACI318} = \begin{cases} 0.9 \text{ for } \varepsilon_t \geq 0.005 \\ 0.65 + \dfrac{0.25\left(\varepsilon_t - \varepsilon_{sy}\right)}{0.005 - \varepsilon_{sy}} \text{ for } \varepsilon_{sy} < \varepsilon_t < 0.005, \phi_{ACI307} \\ 0.65 \text{ for } \varepsilon_t < \varepsilon_{sy} \end{cases} \right) \quad (2)$$

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.

2.2.3 Nominal flexural capacity

Figure 1 illustrates the stress and strain distribution for a circular RC chimney cross-section, strengthened with externally bonded FRP composites and refers to the case of opening in compression. The case of opening in tension is not presented in this paper for brevity, since it is virtually identical to the case of opening in compression. However, the calculation of the nominal flexural capacity should be based on the minimum value calculated for both the cases of opening in compression and tension.

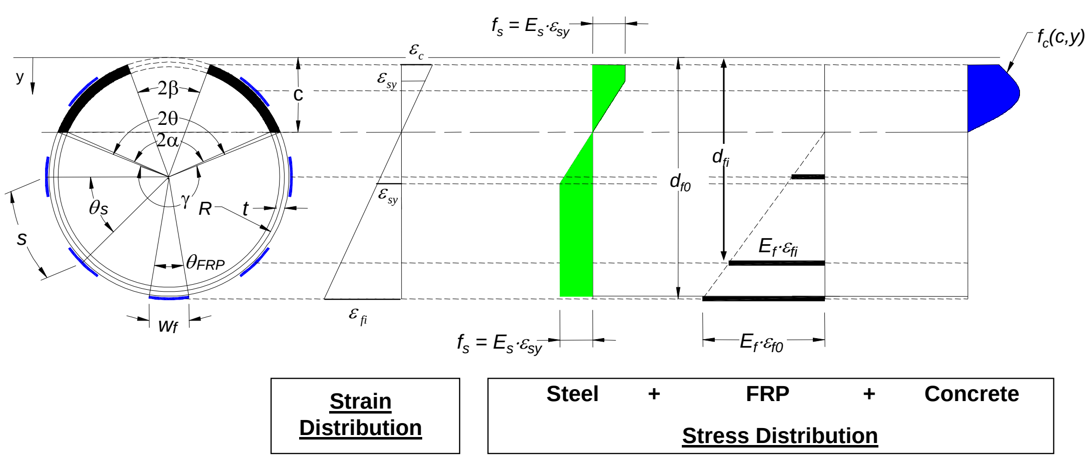


Figure 1. Stress-Strain Distribution (Opening in Compression)

The following assumptions are made in calculating the flexural resistance of a section strengthened with an externally applied FRP system:

- The strains in the steel reinforcement and concrete are directly proportional to the distance from the neutral axis (i.e. plane section before loading remains plane after loading);
- There is perfect bond between FRP and concrete;
- The maximum usable compressive strain in the concrete is 0.003;
- The maximum tensile stress in the steel is 0.007;

– The tensile strength of concrete is neglected; and
– The FRP reinforcement has a linear elastic stress-strain relationship to failure.

Using these assumptions, the equation of equilibrium 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 resultants in the vertical steel in tension and compression can be calculated assuming uniform distribution of vertical reinforcing steel around the circumference. Closed form equations that are functions of the location of the neutral axis c for C, S_1, S_2, S_3, and S_4 can be found in Appendix A of ACI 307 (ACI 307, 2008).

The strain level in the FRP reinforcement at the ultimate limit state dictates the level of stress developed in the FRP and, therefore, the strength contribution by FRP. 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.

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

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

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, considering all the loads acting on the member during the installation of the FRP system. The design strain ε_{fd}, represent the strain level at which debonding may occur and is given by ACI 440.2R-08 by:

$$\varepsilon_{fd} = 0.41\sqrt{\frac{f_c^{'}}{nE_f t_f}} \le 0.9\varepsilon_{fu} \tag{5}$$

in which f'_c is the concrete strength and ε_{fu} is the ultimate tensile strength of the FRP material determined using the characteristic design values and environmental reduction factors given in ACI 440.2R (ACI 440.2R, 008).

Equation (3) is solved by iteration assuming the location of the neutral axis, c, and calculating the force distribution in concrete steel and FRP. The nominal moment capacity of the section is then determined by equilibrium as follows:

$$M_n = P_u\, R\, \cos(\alpha) + C' + S_1' + S_2' + S_3' + S_4' + \Psi_f n t_f w_f E_f \varepsilon_{f0} \left(d_{f0} - c\right) + \\ + \Psi_f \sum_{i=1}^{\frac{n_{st}-1}{2}} 2 n t_f w_f E_f \varepsilon_{fi} \left(d_{fi} - c\right) \tag{6}$$

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 neutral axis, respectively. Close forms equations as functions of the location of the neutral axis c for C', S_1', S_2', S_3', and S_4' can be found in Appendix A of ACI 307 (ACI 307, 2008).

The amount of FRP reinforcement is then determined by iteration until Equation (1) is satisfied.

3 CASE STUDY: PLUM POINT CHIMNEY OSCEOLA, ARKANSAS

The 140 m reinforced concrete chimney is located in a power plant in Osceola, 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 in 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. Due to increased seismic loads, the chimney required flexural strengthening. 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, Structural Preservation Systems (SPS), 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 SPS 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 major 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 outlined in this paper. Because of the variable level of deficiency, the amount of FRP varies 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 by concrete enlargement. For the enlargement, the concrete surface was roughened and steel dowels were installed to supplement 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 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.

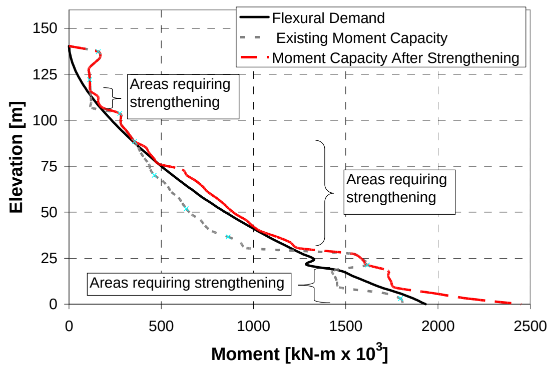
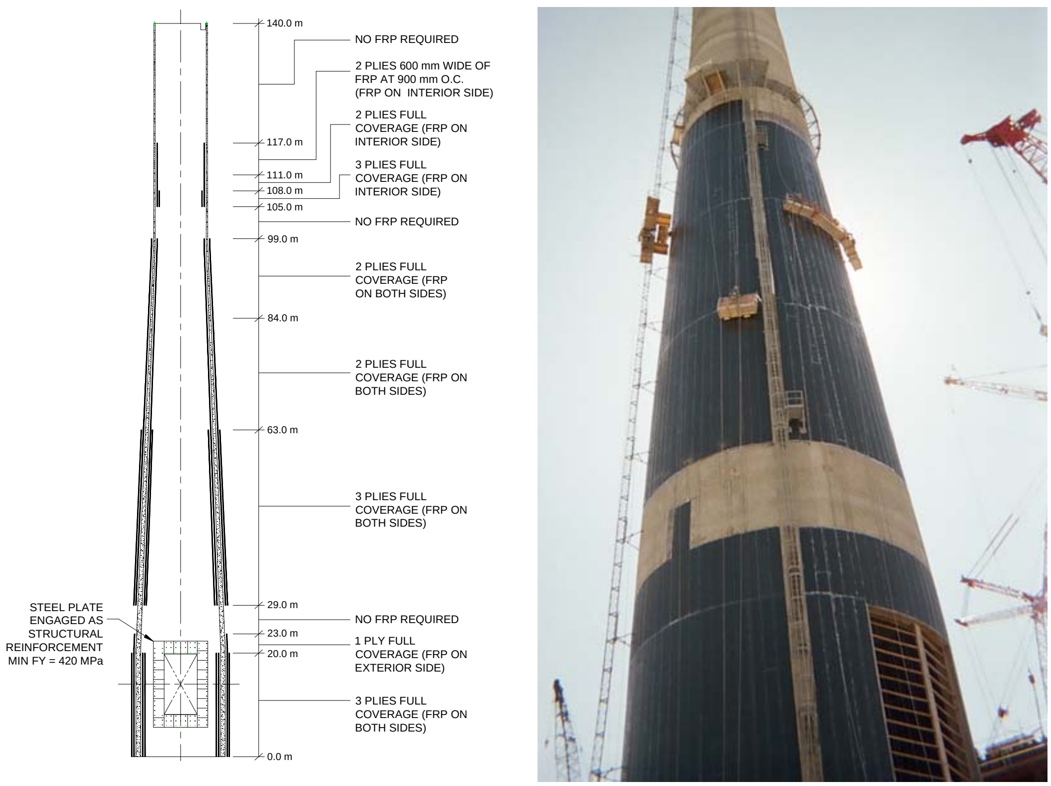


Figure 2. Demand vs. Capacity for Un-Strengthened and Strengthened Chimney



(a) FRP Layout (b) Installation

Figure 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 chimneys since it depends on section geometry, amount of existing steel reinforcement, and required strength increase.

4 CONCLUSIONS

The paper presents a design methodology for the strengthening of RC chimneys using externally bonded FRP composites. The proposed design approach, based on ACI 307-08 and ACI 440.2R-08, allows for the determination of the flexural capacity through simple iteration procedures that can be implemented in a spreadsheet.

A case study was presented in which the proposed design procedure was used in the strengthening of an industrial chimney deficient in flexure due to seismic loads. The case study also emphasizes that strengthening, assessment and design of strengthening solutions are infinitely more complex than new construction. Typically, challenges arise because of unknown factors associated with the structural state - such as condition, load path, and material properties. The degree to which the upgrade system and the existing structural elements share the loads also must be evaluated and addressed properly in the upgrade design, detailing and implementation procedure. The importance of detailing and its direct effect on the effectiveness and durability of structural upgrades is crucial.

5 REFERENCES

ACI Committee 318, 2005. Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI 318R-05). American Concrete Institute, Farmington Hills, Michigan.

ACI Committee 307, 2008. Design and Construction of Reinforced Concrete Chimneys. ACI 437-08, American Concrete Institute, Farmington Hills, Michigan.

ACI Committee 440, 1996. Report on Fiber Reinforced Plastic (FRP) Reinforcement for Concrete Structures. ACI 440.R-96, American Concrete Institute, Farmington Hills, Michigan.

ACI Committee 440, 2008. Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures. ACI 440.2R-08, American Concrete Institute, Farmington Hills, Michigan.

Emmons, P., 2008. Strengthening Using FRP Composite in the United States: from Emerging to a Mainstream Technology. 5th Middle East Symposium on Structural Composites for Infrastructure Applications, Hurghada, Egypt, 52-65

Ilki A., 2008. Existing Building Stock, Seismicity and FRP Activities in Turkey. IIFC FRP International, Volume 5 Issue 2, pp. 8-9

Kelley P. L., Brainerd M. L., and Vatovec M., 2000. Design Philosophy for Structural Strengthening with FRP. Concrete International, Volume 22, Issue 2, p. 72-82

Kobatake, Y.; Katsumata, H.; and Okajima, T., 1990. Retrofitting with Carbon Fiber for Seismic Capacity of Existing Reinforced Concrete Bridge Columns. Proceedings, 45th Annual Conference of the Japan Society of Civil Engineering (in Japanese).

Meier,H., Hess P., 2000. Verstärkung eines Hochkamins ohne Betriebsunterbruch (Strengthening of a chimney without interruption of operation).Schweizer BaublattNo.39 / Bauwerksanierung No.5.

Pallarés F. J., Ivorra S., Pallarés L. And Adam J. M. 2009. Seismic Assessment of a CFRP-Strengthened Masonry Chimney. Proceedings of the ICE - Structures and Buildings, Volume 162, Issue 5, p. 291-299.

Pinfold G M, Scott S., 1997. Repair of a Damaged Concrete Chimney Using a Steel Sheath" CICIND 47th Meeting — Penang, Malaysia, Volume 13, No. 2, p 43-45