Comparison on reinforced concrete bar strengthening using CFRP method, the IWF bar support addition, and bar dimension enlargement

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Abstract. Structure failures often happen whether caused by bad implementation, especially at reinforced concrete bar. Failures on building structure elements might be caused by several factors such as planned age of use, transformation on function, and even improper construction procedure. Thus, the reinforced concrete bar strengthening is required especially on a 10 m bar. This study focus is about comparison on strengthening the original structure of the building using the CFRP method which overlaying the bar surface, adding the IWF bar as a cantilever from below of the desired structure, and enlarging the bar dimensions. The result of the study shows that using 5 m$^2$ CFRP overlaid on flexible reinforcement and 2.5 m$^2$ CFRP overlaid in shear reinforcement get the results in $M_n = 207.82$ KNm and $V_n = 146.33$ Kn. Furthermore, additional 400, 400, 13, 31 IWF bar for the strengthening result is in $M_n = 1045.32$ KNm and $V_n = 809.79$ Kn. Dimension enlargement to 650 mm x 350 mm of 4D16 reinforced concrete and P10-250 mm reinforced cross-bar resulting in $M_n = 529.23$ KNm and $V_n = 78.56$ Kn. Effectiveness analysis is done to select the most effective method considering its strength gained and the cost that required.

1. Introduction
The development of reinforced concrete technology as construction material in Indonesia has entered a new phase. Given that, there will be a maintenance work on the construction after physical construction. Each construction sometimes experiences structural failure caused by the improper procedure such as the age of the plan, changes in the function of the building and implementation.

Furthermore, the building structure will experience structural failure both in light and heavy conditions on reinforced concrete beams. This is the basis for structural improvements by providing reinforcement to the structure. In this study, the damage assumption occurs in a square cross-section of beam that has a span of 10 m.

Thus, the strengthening of the reinforced beam structural elements using IWF steel as a support on the bottom of the concrete beam is performed with Carbon Fibre Reinforced Polymer (CFRP) and by adding dimensions (grouting) and reinforcement on the side of the beam for withstanding a bigger moment.

This study is comparing the strength and the cost of Carbon Fibre Reinforced Polymer (CFRP) and Grouting of the three methods for strengthening the structural elements of concrete beams using the IWF profile steel method.
2. Literature review
The reinforcement methods and results are analysed to provide the solutions for repairing the occurred damage of reinforced concrete beams to get effective results. In the strengthening analysis of IWF Steel using SNI [2] Steel Regulation, the method with CFRP refers to ACI 440.2R-08, and the method with the addition of concrete dimensions are refers to the concrete regulations of SNI 2013. [1]

2.1 Comparison reinforced concrete beams strength using steel plates and Fiber Reinforced Polymer (FRP).
Fibre Reinforced Polymer (FRP), is a type of thin plate/sheet in which there were carbon fibres, glass, aramid, and fibre, was applied. [3] The three principles of using FRP in reinforcing structures increased the bending moment capacity, shear, and axial load on each beam, which had similarities between one and another. Moreover, factors such as the type of structure, available cost, planned load, and environmental conditions determined the type of fibre for strengthening the structure.

The research carried out to use a beam with 15 x 25 cm measure, with a length of 320 cm. Based on the analysis of the results, the increase in beam strength with FRP was 1.991 times from the initial strength, while the steel plate was 1.64 times from the initial strength. The test result showed the increase in beam strength with FRP of 1.44 times from the initial strength, while the steel plate was 1.056 times from the initial strength. Thus, the use of FRP in the tensile area was proven to be able to withstand greater strength than the steel plate, and to inhibit the initial crack.

2.2 Structural reinforcement design at Universitas Andalas Law Faculty multipurpose building using the bracing method by adding steel profile WF 200,100.5,5.8. [4]
In 2016, David [20] examined structural improvements at the multipurpose building of Universitas Andalas which was a composite concrete-steel structure that retrofitted using the bracing method by adding a WF 200, 100.5, 5.8 profile. The result of reinforcement of this bracing force was acted on the experienced beam and it could be reduced. The moment force that worked on the beam was decreased by 69.41% and the shear force acting was reduced by 45.45%. Moreover, due to the addition of bracing, an increase in beam cross-section capacity + 100% of the initial cross-sectional capacity was gained.

2.3 Case study of the method of repairing the basement structure of the PLTGU cooling water pump at PT. PLN Persero. [5]
According to research conducted by Wawansyah and Emilia [5] the method of repairing the rusted Ironing Cooling Water Pump basement structure was cut and replaced with a new reinforcement with the attention of the new bone joint extension. Furthermore, long reinforcement length exceeded the length of the new reinforcement with a length equal to diameter x 40. If the diameter of the reinforcement was 12 mm, then the length of the overlapping reinforcement was approximately 480 mm. Then, the connection was tied with wire or welding, a D13 shear connector was added with a depth of 110 mm, and each 150 mm shear connector was spaced, the new concrete did not slip and old concrete with new concrete became one unit.

3. Methodology
This study mainly focuses were calculating and comparing the strength and the cost of the three methods which are the IWF profile steel method, the CFRP method, and the grouting method on reinforced concrete beams. Then we analysed the comparison to determine the best method used in terms of effectiveness.

3.1 Analyzed data information
This study was conducted to use the three methods which every method had a specific specification. The IWF 400.400.21.13 steel addition method was used to support the load received on the beam. An additional Voute 400 x 600 was used as a flow of load from the beam to the column (see Figure 1). The CFRP method used additional 1 mm thickness CFRP that was coated on three sides on the beam (see...
While the dimension on grouting method had an additional 75 mm of concrete blanket and given a diameter reinforcement of 22 mm.

3.2 The research stages
This study was conducted in three main stages; the first stage was the calculation of the moment strength and the shear force of the three methods, the second stage was cost calculation of the three methods, and the third stage was comparison of the strength and cost of the three methods.

4. Analysis and discussion
Structural reinforcement was carried out using the reinforced concrete beam with specification of length (L) = 10 m, Concrete quality (Fc) = 30 Mpa, beam dimensions = 500 mm x 250 mm, column dimensions = 400 mm x 400 mm, and column height = 3.5 m. While the assessment on the structural reinforcement
was done by calculating the loads on a reinforced concrete beam which are plate = 0,1 m x 2400 kg/m³ = 240 kg/m², space = 0,02 m x 2100 kg/m³ = 42 kg/m², ceiling = 11 kg/m² + 7 Kg/m² = 18 kg/m² + Ins. electric = 25 kg/m², and concrete beam load = 0,5 m x 0,25 m x 2400 kg/m³ = 300 kg/m. While SAP2000 analysis was obtained Mu- Negative = 116,28 KNm, Mu+ positive = 62,17 KNm, and Vu = 61,08 KN.

4.1 CFRP analysis

![Figure 4. CFRP flexible reinforcement.](image)

![Figure 5. CFRP reinforcement geometry](image)

The reinforcement method in CFRP was divided into two parts, that two parts are flexible reinforcement and shear reinforcement (see Figure 4 and Figure 5). The ultimate tensile strength (Fu) that was used as a plant multiplied by the reduction factor which the value was influenced by the environmental conditions of the place. The exposure conditions of FRP in the room caused the value of Ce = 0.95, thus the value of stress and strain can be calculated as follows.

\[
f_{fu} = C_{E} f_{u} = 0.95 \times 621 = 589.95 \text{ Mpa}
\]

\[
\varepsilon_{fu} = C_{E} \varepsilon_{u} = 0.95 \times 0.015 = 0.01425 \text{ mm/mm}
\]

The component of the flexural strength of steel contribution to bending was determined as follows.

\[
M_{ns} = A_{s} f_{s} (d - \frac{\beta c}{2}) = 803.84 \times 414 (450 - \frac{0.496 \times 97.1}{2}) = 142.41 \text{ KNm}
\]

FRP’s contribution to bending

\[
M_{nf} = A_{f} f_{t} (d - \frac{\beta c}{2}) = 500 \times 305 (500 - \frac{0.496 \times 97.1}{2}) = 72.68 \text{ KNm.}
\]
While the check on crack limit on FRP can be calculated as follows.

\[
f_{fs} = F_{s,s} \left( \frac{E_f}{E_s} \right) \left( \frac{d_f - k_d}{d - k_d} \right) - E_{bl} E_f f_{fs} = 0.015 \left( \frac{37}{200} \right)^{\frac{500-178.6}{450-178.6}} - 0.00089 x 37 = 0.01783 \text{ Kn} \]

\[
F_{fs} = 17.83 \text{ F}_{fs} (17.839) \leq 0.55 f_{fu} (324.47) \tag{6}
\]

We determined the voltage level in FRP was in the recommended cyclic voltage. While the effective strain level in the shear amplifier was effective.

\[
L_e = \frac{23300}{(nt_f f)^0.33} = \frac{23300}{(2 x 1 x 37000)^0.33} = 7.252 \text{ mm}
\]

\[
K_1 = \left( \frac{F_{ve}}{27} \right)^{2/3} = \left( \frac{30}{27} \right)^{2/3} = 1.065
\]

\[
K_2 = \left( \frac{d_f - L_e}{d_{fv}} \right) = \left( \frac{250 - 7.252}{250} \right) = 0.985
\]

\[
K_v = \frac{K_1 k_2 L_e}{11910 f_{fu}} = \frac{1.065 x 0.06 x 34.93}{11910 x 0.01425} = 9.11 x 10^{-6}
\]

Effective voltage was good since it was below 0.004 and it was calculated as follows.

\[
\varepsilon_{fe} = K_v \varepsilon_{fu} = 3.83 x 10^{-5} x 0.01425 = 5.45 x 10^{-7}
\]

\[
\varepsilon_{fe} \leq 0.004
\]

\[
1.298 x 10^{-7} \leq 0.004
\]

The contributions of FRP reinforcement to shear strength were the slide reinforcement area \((A_{fv})\), the effective pressure in FRP \((F_{fe})\), and the shear contribution on FRP \((V_f)\). Those were calculated as follows.

\[
f_{fv} = \varepsilon_{fe} E_{fv} = 1.298 x 10^{-7} x 222530 = 2.019 x 10^{-3} \text{ KN/mm}^2
\]

\[
V_f = \frac{A_{fw} f_{e} (\sin \alpha + \cos \alpha) d_{fv}}{s_f} = \frac{2500 x 2.019 x 10^{-2} x 1 x 500}{270} = 133.73 \text{ KN}
\]

Furthermore, the FRP shear strength was calculated as follows.

\[
\omega V_n = \omega (V_c + V_s + \Psi V_d) = 0.75 (56.706 + 24.731 + (0.85 x 133.73)) = 146.33 \text{ Kn}
\]

\[
\omega V_n > V_n 146.33 \text{ Kn} > 76.53 \text{ Kn}
\]

Figure 6 shows the value of the force on concrete \((a)\), distance from the neutral line \((c)\), \(C_c\) distance against \(T_s\) \((Jd)\), and \(C_c\) distance against \(T_f\) \((Jdf)\) that calculated as follows.

Figure 7 shows the value of the force on concrete \((a)\), distance from the neutral line \((c)\), \(C_c\) distance against \(T_s\) \((Jd)\), and \(C_c\) distance against \(T_f\) \((Jdf)\) that calculated as follows.
Kd = c = 179.38 mm, so a = β1 x c = 0.85 x 179.38 = 152.47 mm

Tf = Asf x Fyf = 2 x 1 x 250 x 414 = 207000 N => 207 Kn

Ts = As x Fy = 4 (1/4 π d2) x 400 = 4 (0.25 x 3.14 x 162) x 400 = 321200 N => 321.2 Kn

Cc = Ts + Tf

0.85 x F’c x a x b = As.Fy + Asf.Fyf

0.85 x 30 x a x 250 = 321.2 + 207

a = 528.3/6.375

a = 82.87 mm

C = a/β1 = 82.87/0.85 = 97.49 mm

Jd = d – a/2

Jd = (h – ds -1/2 ds) – 82.87/2

Jd = (500 – 40 – ½ 16) – 82.87/2

Jd = 452 – 41.43

Jd = 410.57 mm

Jdf = h - a/2

Jdf = 500 - 82.87/2

Jdf = 458.57 mm

Mn = As.Fy.jd + Asf.fyf.jdf = 4 (1/4 π d2) x 400 x 410.57 + 2 x 1 x 250 x 414 x 458.57

= 147,293 + 94,923 = 242,216 Knm

\[ \begin{align*}
J_d &= d - \frac{a}{2} \\
J_d &= (h - d_s - \frac{1}{2} d_s) - \frac{82.87}{2} \\
J_d &= (500 - 40 - \frac{1}{2} 16) - \frac{82.87}{2} \\
J_d &= 452 - 41.43 \\
J_d &= 410.57 \text{ mm}
\end{align*} \]

Compact check based on Local Buckling Flange showed it was strong based on these calculation.

\[ 
\begin{align*}
\lambda &= \frac{b}{400/2} = 9.523 \\
\lambda_p &= \frac{170}{\sqrt{F_y}} = 9.983 \\
\lambda_r &= \frac{370}{\sqrt{F_y - F_r}} = 29.945, \lambda < \lambda_p
\end{align*} \]  

Compact check based on Local Buckling Flange showed it was strong based on these calculation.

\[ 
\begin{align*}
M_n &= M_p = 290 \times 3604584.3 \times 10^{-6} = 1045,329 \text{ Knm} \\
\Omega M_n &= 0.9 \times 1045.3 = 940.792 \text{ Knm} \\
FLB \Omega M_n &= 940,796 \text{ Knm} > M_u = 583 \text{ Knm}
\end{align*} \]
Because simple beam was used \( C_b = 1.14 \), it was determined that the nominal flexural strength meets (safe).

\[
M_r = (F_Y - F_r) S_x = (290 - 70) \times 3330000 = 732600000 \text{ Nmm} = 732,6 \text{ Knm}
\]

\[ (15) \]

\[
M_n = C_b \left[ M_r + \left( M_p - M_r \right) \frac{L_r - L}{L_r - L_p} \right] \leq M_p
\]

\[ M_n = 2.381 \left[ 732.6 + \left( 1045.329 - 732.6 \right) \frac{15.4 - 10}{15.4 - 4.78} \right] \leq 1045,329 \]

\[ (16) \]

\[
M_n = 1016,473 \text{ Knm} \leq 1045,329 \text{ Knm}
\]

\[ \Omega M_n = 0.9 \times 1016,473 = 914,836 \text{ Knm} > 583,333 \text{ Knm} \]

\[ (17) \]

While deflection of WF profile 400x400x21x13 50 steel quality was safe. See the calculation as follows.

\[
\delta = \frac{5}{384} \frac{W L^4}{E I} = \frac{5}{384} \frac{\left(32.5 + 10 \times 10^0\right) \times 10^6}{210000 \times 666 \times 10^6} = 39.567 \text{ mm}
\]

\[ (18) \]

\[
\delta_{\text{maks}} = \frac{L}{240} = \frac{10000}{240} = 41.667
\]

\[ \delta_{\text{maks}} = 41.667 > \delta \text{ nominal} = 39.567 \]

\[ (19) \]

Determination on the distances and forces that work was calculated as follows.

\[
A_c = b \times t_b = 250 \times 500 = 125000 \text{ mm}^2
\]

\[ (20) \]

\[
T = A_s \times F_Y = 21870 \times 240 = 5248800 \text{ N}
\]

\[ (21) \]

\[
C_c = 0.85 \times F'_c \times A_c = 0.85 \times 30 \times 125000 = 3187500 \text{ N}
\]

\[ (22) \]

\[
a = \frac{T}{C_c} = \frac{5248800}{3187500} = 411 \text{ mm}
\]

\[ (23) \]

because the value of \( T > C_c \), the neutral line was still in the steel profile (see Figure 8).

The \( C_c = T \) balance requirement, \( C_c = 3187500 \text{ N} \)

\[
d_1 = t_b - a/2 = 500 - 823/2 = 89 \text{ mm}
\]

\[ (24) \]

\[
d_2 = 0
\]

\[ (25) \]

\[
d_3 = d/2 = 400/2 = 200 \text{ mm}
\]

\[ (26) \]

\[ \text{Figure 8. Composite steel diagram} \]
4.3 Analysis of grouting method

Addition of concrete dimensions to existing concrete beams and reinforcement was done to minimize the load on reinforced concrete beams. The flexible reinforcement planning was calculated as follows.

\[ C_b = \frac{600}{600 + F_y} \times d = \frac{600}{600 + 400} \times 600 = 360 \text{ mm} \]  \hspace{1cm} (27)

\[ C_{b\text{max}} = 0.75 \times C_b = 0.75 \times 360 = 270 \text{ mm} \]  \hspace{1cm} (28)

\[ A_{b\text{max}} = 0.85 \times 270 = 229.5 \text{ mm} \]  \hspace{1cm} (29)

\[ C_{b} = \frac{0.85 \times A_b \times F_c}{0.85 \times 229.5 \times 350 \times 30} = 2048287.5 \text{ N} \]  \hspace{1cm} (30)

\[ M_n = C_{b} \times (d – ab/2) + C_s \times (d-d') = 643,072 \times (592 – 72,05/2) + 321,536 (592-58) = 529231 \text{ Nmm} \]

\[ = 529,231 \text{ Knm} \]  \hspace{1cm} (31)

\[ M_r = 0.8 \times M_n = 0.8 \times 529,231 = 423,285 \text{ Knm} \]  \hspace{1cm} (32)

While control was proven to be good and was calculated as follows.

\[ M_r > M_n \]

\[ 423,29 \text{ Knm} > 116,28 \text{ Knm} \]  \hspace{1cm} (33)

Strain stress distribution on reinforcement was calculated as follows.

\[ T_s = A_s \times f_y = 1607,68 \times 400 = 643072 \text{ Nmm} \]  \hspace{1cm} (34)

\[ C_c = T_s \]

\[ 0.85 \times F_c^' \times a \times b = T_s = 643072 \]  \hspace{1cm} (35)

\[ a = 643072/(0.85 \times F_c^' \times 30) = 72,052 \text{ mm} \]  \hspace{1cm} (36)

\[ c = a/\beta = 84,768 \text{ mm} \]  \hspace{1cm} (37)

The press reinforcement for the re-strain control was calculated as follows.

\[ \epsilon' = (c-ds) \times 0.003 = (84,77-58)/350 \times 0.003 = 0.0095 \]  \hspace{1cm} (38)

Which tensile pull was calculated as follows.

\[ \epsilon_s = (d-c) \times 0.003 = (592-84,77)/350 \times 0.003 = 0.0179 \]  \hspace{1cm} (39)

\[ F_y/\epsilon_s = 400/0.0179 = 0.002 \]  \hspace{1cm} (40)

Based on the assumption, the ductile reinforcement could be continued with a ductile reinforcement and was calculated as follows.

\[ a < a_{\text{max}}, 72,05 < 226,44 \]  \hspace{1cm} (41)

Then the assumption of the second value that the melting press was correct

\[ C_c = 0.85 \times a \times b \times F_c^' = 0.85 \times 72,05 \times 350 \times 30 = 643072 \text{ N} \]  \hspace{1cm} (42)

\[ C_s = A_{s} \times F_y = 321536 \text{ N} \]  \hspace{1cm} (43)

Value of nominal moment (M_n) and moment of plan (M_r) visible beam were good and were calculated as follows.

\[ M_n = C_c \times (d – ab/2) + C_s \times (d-d') = 643,072 (592 – 72,05/2) + 321,536 (592-58) = 529,231 \text{ Knm} \]

\[ = 529,231 \text{ Knm} \]  \hspace{1cm} (44)

\[ M_r = 0.8 \times M_n = 0.8 \times 529,231 = 423,285 \text{ Knm} \]  \hspace{1cm} (45)

\[ M_r > M_u \]

\[ 423,29 > 116,28 \]  \hspace{1cm} (46)

The support reinforcement on the beam was selected that the lower bond = 8D 16 and upper balance = 4D 16 (see Figure 9).

**Figure 9.** The 10 strain stress diagram on beam
The calculation on shear reinforcement indicated that the shear reinforcement was needed. See the calculation as follows.

\[ \Omega V_c = \Omega (0.17 \sqrt{f'c}) bw d = 0.75 \times 0.17 \times 1 \times \sqrt{30} \times 350 \times 582 = 56706 \text{ N} \]  
\[ \frac{1}{2} \Omega V_c = 0.5 (56706) = 28,35 \text{ Kn} \]  
\[ Vc1 = \frac{1}{3} \sqrt{(f'c) bw d} = \frac{1}{3} \sqrt{30 \times 350 \times 232} = 146767 \text{ N} \]  
\[ Vc2 = \frac{2}{3} \sqrt{(f'c) bw d} = \frac{2}{3} \sqrt{30 \times 350 \times 232} = 293535 \text{ N} \]  
\[ Vu > Vc \]

Next, we determined the area where the distance of the crossing was determined based on the maximum distance requirements between partners.

\[ Vs = \frac{(A_s F_yt d)}{S} = \frac{(157 \times 240 \times 232)}{300} = 29139 \text{ N} \]  
\[ \Omega Vs = 0.75 \times 29139 = 21854 \text{ N} \]  
\[ \Omega Vc + \Omega Vs = 56706 + 21854 = 78560 \text{ N} = 78,56 \text{ Kn} \]

The location where the stirrup could be placed with a maximum distance of 300 mm was sought from a triangle comparison.

The reinforcement against torque was proven to be needed torque reinforcement and was calculated as follows.

\[ Acp = 350 \times 650 = 227500 \text{ mm}^2 \]  
\[ PcP = 2 \times 1000 = 2000 \text{ mm} \]  
\[ Tn > \Omega x 0.083 x \lambda x \sqrt{f'c} x \left( \frac{[AcP]^2}{PcP} \right) \]

4.4 *The internal force of the three method*

The nominal moment results obtained from three methods were the reinforcement with 400,400,13.21 IWF steel beams had nominal moment of, \( Mn = 1045, 329 \text{ Knm} \) and nominal shear force, \( Vn = 809,793 \text{ Kn} \). While the strengthening with CFRP had the nominal moment, \( Mn = 207.82 \text{ Knm} \) and nominal shear
force, $V_n = 146.33$ Kn. Finally, the strengthening with the addition of concrete dimensions had the support nominal moment of, $M_n = 529,231$ Knm, the nominal moment of the field is equal to, $M_n = 529,231$ Knm, and the nominal shear force, $V_n = 78.56$ Kn (see Figure 10 and Figure 11).

![Figure 10. The comparison of the moment values of the three methods.](image)

![Figure 11. The comparison of shear force values of the three methods.](image)

4.5 Analysis on costs of the three methods

![Figure 12. The comparison of costs of the three methods](image)

Figure 12 shows that the grouting method is the most effective method related to its moment created and its cost.
5. **Conclusion and suggestion**

The results of this study indicated that reinforcement in reinforced concrete beams using IWF 400,400.13.21 steel had been able to carry the burden found on reinforced concrete beams. The calculation showed that the area covered with CFRP was 5 m², and CFRP coated area was 2.5 m². While the calculation of reinforcement on reinforced concrete beams showed cross-sectional area of the concrete was 650 mm x 350 mm.

The strengthening method with IWF Steel 400,400.13.21 cost Rp. 100,530,485,-, the strengthening method with concrete dimension addition cost Rp. 13,724,444,-, and the strengthening method with CFRP cost Rp. 12,693,665,-. The retrofitting method with the addition of concrete dimensions is the most effective method because the value of effectiveness is smaller than the three methods.

Finally, we strongly suggest it is best to investigate the damage to the structure to be reviewed to determine the type of damage before conducting research on structural reinforcement. Then, the structural reinforcement should be carried out experimentally directly in order to obtain good results in structural reinforcement studies.

6. **References**

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