Repairing of one-way reinforced concrete slab using externally CNC perforated steel plate. Theoretical study.

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Abstract

This research includes the design steps for using solid and perforated steel plate to repair an under reinforcement one-way concrete slab in flexure. All slabs were assumed to be subjected to 85 percent of ultimate load in flexure, therefore the section of the slab to be repairing assumed to be cracked, and the additional external reinforcement represented by the steel plate will be designed according to this assumption and according to the limitations of ACI 318 code specifications. Results of the theoretical calculation indicated that increasing the thickness of the solid plate causes increasing the interface stresses by (72.6%) when replacing the dimensions (width x thickness) of the plate section from (300mm×1mm) to (200mm×1.5mm), while increasing the width of the plate reduces the interface stresses by (48%) when changing dimensions of the plate section to (400mm×0.75mm). Using perforated plate will increase the ductility and reduce the interface shear stresses. Using the plate width to thickness ratio \(\{b_p/d_p\}=300\) will reduce these stresses by (14.8%) when using perforated plate instead of solid one. Increasing the ratio of \(b_p/d_p\) lead to reduce the interface shear stresses, as well as reduce the difference between the effect of using solid or perforated steel plates on interface stresses, also when reducing ratio of \(b_p/d_p\) the interface shear stress will be increase. Changing the number and diameters of the holes from (10 # 10 mm) to (2 # 50 mm) with keeping the cross-section area of the steel plate constant for both cases increased the resistance of the epoxy nails to the shear stresses that transmitted to them from the axial stresses in the plate by (400%). A new expression is presented in study to calculation the interface shear stress.

1. Introduction

One of the most important problems faced by reinforced concrete installations is the occurrence of failure in some elements of the structure, which facilitate the complete collapse if it left without repairing. Therefore, researchers making an effort to study types of failure that occur in the facilities to develop appropriate solutions and treating the weak structural elements that cause collapse. Development an economic and appropriate scientific technical method according to the nature of the structural element is a goal that most researchers seek in this field. Repairing of the damaged structures to restore structural performance or strengthen of undamaged structures to improve structural behavior with respect to surface and ultimate load often is for economic reasons [1]. The structure may be damage due to earthquakes, fire, over loading, blast loading, corrosion of reinforcement, any errors in mix design, and design flaws. Such reasons mentioned above may cause unsatisfactory performance with respect to ultimate strength and serviceability [2]. The plate bonding technique was introduced as anew repairing and strengthen technique when the discovery of epoxy as an adhesive. This technique is economical, easy, and fast, so it can apply while structure keep using the height of room in minimum reduction. This method used to strengthen many structures in site. Many buildings and bridges in different countries have been strengthening and repairing
by this technique. Investigations about these techniques showed its effectiveness of perpetuating old and distressed reinforced concrete structures [2]. There are two method for repairing members the Near Surface Mounting (NSM) and Externally Bonded Reinforcement (EBR). Using (NSM) method for repairing or strengthening the structural elements against bending loads may cause risks to the structural element, due to the drilling process in the concrete cover to make grooves which may cause significant damage to the bars of tension and shear reinforcement especially since the repair process is to be performed dearly for old or damaged sections where the concrete cover layer is in poor and weak condition. While (EBR) technique depend on using epoxy adhesive to fixed external plate with concrete slab without any change in the original section. Depending on the presented above, the externally bonded strengthening (EBR) technique has the safe and better practical application than the (NSM) technique [2] and [3]. However, several investigations worked on the use of steel plates in repairing strengthen to increase the flexural strengthen of reinforced concrete elements, using External Bonded Reinforcement (EBR) [2]. There are rare studies about using a perforated steel plates for external repairing of concrete sections. In the present theoretical study an attempt is made to cover the issue of the use of perforated steel plates to repair the damaged one way concrete slabs taking into account different parameters such as the thickness of the plate, the width of the plate, diameter of the holes and their number as well as adding the effect of the epoxy nails formed from filling the holes with epoxy and their role in increasing the fixation and resistance to the interface shear stresses between steel plate and concrete.

2. Literature Review
Several studies have been conducted to find out the mechanism of using external reinforcement using steel plate, especially for repairing or strengthen beams. Deric John and John Paul [4] shows that the peeling strength depends on the flexural rigidity of the cracked plated section, the tensile strength of the concrete, and the thickness of plate, and it was independent on the pervious loading history of the beam, on the initial curvature of the beam and the method of clamping the plate to the beam on gluing. Ziraba et al [5]. established a design procedure for externally plated RC beams considering the plate debonding and shear failure modes. Hussain et al [1] showed that externally plated beams are prone to diagonal cracking at load levels below shear capacities calculated from ACI 318 M American concrete institute [6]. Sabahattin Aykac et al [3] showed that the bolt anchorage of thin bottom plate to concrete beam does not have a significant contribution to the ductility and load carrying capacity of the beam. This contribution increases of the plate thickness increase, the use of perforated plates in replacement of solid ones improved the ductility of an externally plated beam despite decreasing its ultimate moment-capacity attributable to the reduction in the sectional area of the plate from perforations [7].

3. Theoretical study
The purpose of using steel plate is to repair or strengthen the slab by increasing the bending resistance of the section when the deflection and cracks at the tensile section exceeds the design limits, which may lead to failure of the structural component. The design procedures which adopted in current study are based on theoretical calculations, so it has been assumed that the concrete portion of the slab is cracked and the samples were subjected to a load of 85 percent of the ultimate load. Based on that the required section of the steel plate is calculated. Also, the stresses generated at the end of the plate will lead to sudden failure if it exceeds the permissible limits which were formed as a result of the influence of the bending moment and deflection of the plate as well as on the specifications, dimensions of the plate used and the specifications of the used glue to fix the steel plate to the concrete surface. Therefore, the distance between the end of the plate and the support must be determined and the design limitations not to be exceeded so the plate design requires several steps as follows:
3.1. Design the steel plate for flexural

3.1.1. Find plate thickness

According to the design of slabs by ultimate design method and Zirab et al [2] as shown in Figure 1 and 2:

Figure 1. Cross-section in the slab with epoxy layer.

Figure 2. Forces and strains in the section of slab

\[ C = T_s + T_p \quad \text{(From equilibrium)} \]

\[ a = \frac{T_s + T_p}{0.85 f_c' b_c} = \frac{A_x f_{yx} + A_p f_{yp}}{0.85 f_c' b_c} = \frac{A_x f_{yx} + b_p f_{yp}}{0.85 f_c' b_c} \quad \text{[6]} \]

\[ \frac{M_u}{\phi} = T_s \left( h_s - \frac{a}{2} \right) + T_p \left( h_p - \frac{a}{2} \right) \quad \text{(2)} \]

Substituting 1 in 2:

\[ \frac{b_p f_{yp}}{2} \left[ 1 - \frac{b_p f_{yp}}{0.85 f_c' b_c} \right] d_p^2 + b_p f_{yp} \left[ h_c + d_a - \frac{A_s f_{ys}}{0.85 f_c' b_p} \right] d_p + \left[ A_s f_{ys} \left( h_s - \frac{A_s f_{ys}}{2 \left( 0.85 f_c' b_c \right)} \right) - \frac{M_u}{\phi} \right] = 0 \quad \text{(3)} \]

From above relation we can find \( d_p \)

3.1.2. Check the value of \( d_p \) with \( d_p \) balance or \( t_p b \)

\[ t_p b = S \left( 0.85 f_c' b_c \right) - A_s f_{ys} \quad \text{[4]} \]

\[ C = T_s + T_p \; ; \; \; \; d_p = d_p b = t_p b \quad \text{(At balance condition)} \]

We can find \( x_b \) from strain diagram assumed that the internal steel reinforcement at tension zone yield and the strain of concrete at compression zone = 0.003

\[ x_p = \frac{\varepsilon_c h_s}{\varepsilon_s + \varepsilon_c} \quad \text{[5]} \]

Where:

- \( x_b \) = depth of natural axis from top of section,
- \( \varepsilon_c \) = concrete strain at top of compression section,
- \( \varepsilon_s \) = internal reinforcement strain.

Then find \( d_p b = \) (depth of plate at balanced condition)

\( d_p \) must be \( \leq d_p b \)
3.1.3. Check interface stresses

The interface stresses $\tau_o$, $\sigma_0$ represent the peak shear stress and peeling stress respectively, and we can find the values of $\tau_o$, $\sigma_0$ as follow [5]:

$$\tau_o = \alpha_1 f'_t \left( \frac{C_{R_1} V_o}{f_c'} \right)^{\frac{5}{4}}$$

$$\sigma_0 = \alpha_2 C_{R_2} \tau_o$$

$$C_{R_1} = \left[ 1 + \left( \frac{K_s}{E_p b_p d_p} \right)^{\frac{1}{2}} a' \right] \frac{b_p d_p}{I b_a} (h_p - h)$$

$$C_{R_2} = d_p \left( \frac{K_n}{4 E_p I_p} \right)^{\frac{1}{4}}$$

$$\tau_o + \sigma_0 \tan 2\theta^\circ \leq C_{all}$$

Also, the distance may be found from the end of plate to the support depend on the equations 6, 7, and 8 which limited the value of this distance ($a_0$).

3.1.4. Check shear capacity

For slabs the shear strength usually larger than shear stress from loads, therefore not need shear reinforcement [6].

$$V_c = \frac{1}{6} \left( \sqrt{f_c'} + 100 \rho_w \frac{V_{ud}}{M_u} \right) b_w d$$

Where:

$V_c$ = shear strength of concrete section,

Check if: $\frac{1}{2} \emptyset V_c > V_{ud}$

A new expression is derived in present study to calculate the shear stress in the interface layer to facilitate the process of calculation for these stresses by considering all the variables as fixed except for the value of the width ($b_p$) and thickness ($d_p$) of the plate and the assumption of a fixed section area of the plate and this was based on the research submitted by Ziraba et. Al [5]. The equation:

$$\tau_o + \sigma_0 \tan 2\theta^\circ = C_3 \left( \frac{1}{b_p} \right)^{\frac{5}{4}} + 0.585 C_4 \left( \frac{d_p}{b_p^{\frac{1}{4}}} \right)^{\frac{1}{4}}$$

where:

$$C_1 = \alpha_2 f'_t \left( \frac{C_{R_1} V_o}{f_c'} \right)^{\frac{5}{4}}$$

$$C_2 = \left[ 1 + \left( \frac{K_s}{E_p A_p} \right)^{\frac{1}{2}} (h_p - h) \right] \frac{b_p d_p}{I b_a}$$

$$C_3 = C_1 \times C_2^{\frac{5}{4}}$$

$$C_4 = \left( \frac{3 K_n}{E_p} \right)^{\frac{1}{4}}$$

The new expression can be used especially for thin plate. Furthermore, it can be used for a thick plate but with less accurate results due to slight change in value of ($h_p$) and moment of inertia because of the change in the value of ($d_p$) may significantly affect the values of ($h_p$) and I for the section. The values $C_1$, $C_2$, $C_3$, $C_4$.
and $C_4$ will be constant for any design if only $(d_p)$ and $(b_P)$ will be changed to find the best dimensions of design, and $(b_p \times d_p = A_P)$ constant value and $(b_o = b_p)$.

4. Theoretical calculation:

To determine the effect of the steel plate dimensions, using the perforated plate with circular holes and the diameter of the holes on the bending resistance of the slab section several calculations were made using the design procedures adopted by Ziraba et. al [5] and the new derived expression. these calculations conducted to find the required dimensions of the steel plate used for repairing, the specifications of composite slab section and the maximum shear stresses and the bending resistance as shown in the tables below. Table 1 shows the effect of the different dimensions of solid and perforated steel plate on the values of ultimate moment capacity, interface shear stresses and normal stress at the calculated distance $(a_o=70 \text{ mm})$ from the end of steel plate to the support.

**Table 1. Values of the ultimate moment capacity and the interface stresses in solid and perforated steel plate.**

| Property                                      | Solid steel plate | Perforated plate with (100 mm²) circular holes in the section |
|-----------------------------------------------|-------------------|---------------------------------------------------------------|
|                                               | Reference plate   |                                                         |
|                                               | $b_p \times d_p$  | $b_p \times d_p$                                            |
|                                               | $300 \times 1.0$  | $300 \times 1.0$                                            |
|                                               | (mm)              | (mm)                                                        |
| $A_{Pcr}$ (mm²)                               | 300               | 300                                                         |
| $I_{Pcr}$ (mm⁴)                               | 25.000            | 9.000                                                       |
| $I_{comp_{cr}}$ (mm⁴)                         | 9809167           | 9794191                                                    |
| $M_o$ (N.mm)                                  | $30 \times 10^6$  | $29.955 \times 10^6$                                        |
| $\tau_o$ (MPa)                                | 2.673             | 1.41                                                        |
| $\sigma_o$ (MPa)                              | 1.182             | 0.484                                                       |
| $\tau_o + \sigma_o \tan(2\beta)$            | 3.302             | 1.671                                                       |
| $< C_{all}$                                   |                   |                                                             |

Where:

$A_{Pcr}$ = critical section area of plate,

$I_{Pcr}$ = moment of inertia of critical section of plate,

$I_{comp_{cr}}$ = moment of inertia of critical composite section of slab.

These values are calculated at any section of solid plate or at the section passing the centers of holes for perforated plate.

The relation between $(b_p/d_p)$ and shear stress of interface layer are shown in Tables 2 and 3 and Figure 3.

**Table 2. The relation between $(b_p/d_p)$ and shear stress of interface layer for solid plate.**

| $b_p$ (mm) | $d_p$ (mm) | $b_p/d_p$ | $\tau_o + \sigma_o \tan(2\beta)$ (MPa) |
|------------|------------|-----------|----------------------------------------|
| 200        | 1.50       | 133.333   | 5.700                                  |
| 300        | 1.00       | 300.000   | 3.302                                  |
Table 3. The relation between \((b_p / d_p)\) and shear stress of interface layer for perforated plate.

\[
\begin{array}{cccc}
 b_p \text{ (mm)} & d_p \text{ (mm)} & b_p / d_p & \tau_o + \sigma_o \tan(2\theta) \text{ (MPa)} \\
 200 & 2.00 & 100.00 & 5.818 \\
 300 & 1.33 & 200.00 & 3.353 \\
 400 & 1.00 & 400.00 & 2.276 \\
 500 & 0.80 & 625.00 & 1.690 \\
\end{array}
\]

4.1. Shear strength of epoxy nails

When using the perforated plates with circular holes epoxy will fill these holes creating what looks like nails. These epoxy nails increase fixing the steel plate with the epoxy layer, in addition these nails will play a role by resisting the stresses on the plate. The Epoxy nails will be subjected to shear stresses that transferred to them from the steel plate when exposed to axial stresses generated by the bending. Fig. 4 shows the details of these nails. The effect of epoxy nails is significant on the shear strength between the surface of steel plate and the layer of epoxy as it explained in Table 4. While this effect is very small when calculating the values of \(A_{Fcr}, I_{Fcr}, I_{comp. cr}\), and ultimate moment. Therefore, it be neglected when calculate these values.
The Industrial Fastener Institute (Inch Fastener Standards, 7th ed. 2003. B-8) [8] states that shear strength is approximately 60% of the minimum tensile strength.

Tension strength of epoxy = 12 MPa
Shear strength = 0.6 × 12 = 7.2 MPa

4.1.1. For 10 mm diameter of nail

\[ A_{\text{nail}} = \frac{\pi \times 10^2}{4} = 78.54 \text{ mm}^2 \]
Shear force in one nail = \( 7.2 \times 78.54 = 565.5 \text{ kN} \)
For 10 nails, shear force = \( 10 \times 565.5 = 5655 \text{ kN} \)

4.1.2. For 25 mm diameter of nails

\[ A_{\text{nail}} = \frac{\pi \times 25^2}{4} = 490.87 \text{ mm}^2 \]
Shear force in one nail = \( 7.2 \times 490.87 = 3534.3 \text{ kN} \)
For 4 nails, shear force = \( 4 \times 3534.3 = 14137.17 \text{ kN} \)

4.1.3. For 50 mm diameter of nails

\[ A_{\text{nail}} = \frac{\pi \times 50^2}{4} = 1963.5 \text{ mm}^2 \]
Shear force in one nail = \( 7.2 \times 1963.5 = 14137.17 \text{ kN} \)
For 2 nails, shear force = \( 2 \times 14137.17 = 28274.33 \text{ kN} \)

| Property | Perforated steel plate |
|----------|------------------------|
| Diameter of hole (mm) | with deferent holes diameters | with deferent holes diameters |
| 10 | 200 | 200 | 200 | 300 | 300 | 300 |
| 25 | 16.5 | 16.5 | 16.5 | 25 | 25 | 25 |
| 50 | 8465174 | 8465174 | 8465174 | 9809167 | 9809167 | 9809167 |
| \( A_{P,cr} \) | \( 26 \times 10^6 \) | \( 26 \times 10^6 \) | \( 26 \times 10^6 \) | \( 30 \times 10^6 \) | \( 30 \times 10^6 \) | \( 30 \times 10^6 \) |
| \( I_{P,cr} \) | \( 8465174 \) | \( 8465174 \) | \( 8465174 \) | \( 9809167 \) | \( 9809167 \) | \( 9809167 \) |
| Ultimate moment strength at critical section (N.mm) | \( 5655.00 \) | 14137.17 | 28274.20 | \( 5655.00 \) | 14137.17 | 28274.20 |
| Shear strength from nails of epoxy (N) | \( 5655.00 \) | 14137.17 | 28274.20 | \( 5655.00 \) | 14137.17 | 28274.20 |

4.2. Technical example

4.2.1. Design of one-way slab full scale
Dimensions of the slab as shown in Figure 5
\( b_c = 500 \text{ mm}, \text{ Length of slab} = 3250 \text{ mm} \)

Min. thickness of slab = \( \frac{L}{20} \) (ACI318-code (9.5 a))

![Figure 5. Cross-section in the slab.](image)
\[ h_c = \frac{L}{20} = \frac{3250}{20} = 162.6 \text{ mm} \approx 170 \text{ mm} \]

∴ use 170 mm thickness of the slab

∴ dimension of slab = 3250 × 500 × 170 mm

\[ \rho_b = 0.85 \beta_1 \frac{f'_c}{600} \frac{600}{f_y 600 + f_y} = 0.85 \times 0.85 \times \frac{25}{500} \times 600 = 0.0197 \]

Use 4Ø10 as tension reinforcement for slab = 36% for \( A_s \) balance.

4Ø10 = 314.16 mm²

Distance between bar reinforcement center to center:

\[ S = \frac{500 - 2 \times 20}{3} = 153.33 \text{ mm}^2 \]

\[ \rho_{\text{min}} = \rho_{\text{shrinkage}} = \frac{0.0018 \times 420}{500} = 0.00151 \]

\[ A_{s,\text{min}} = \rho_{\text{min}} \times b \times h = 0.00151 \times 500 \times 170 = 128.35 \text{ mm}^2 < 314.16 \text{ mm}^2 \quad \therefore \text{O.K.} \]

Maximum spacing

\[ S_{\text{max}} = 3h = 3 \times 170 = 510 \text{ mm} \quad \text{or} \quad S_{\text{max}} = 450 \text{ mm} \]

\[
S = 153.33 \text{ mm} < 450 \text{ mm} \quad \therefore \text{O.K.}
\]

Check shear capacity:

\[ V_c = \frac{1}{6} \sqrt{f'_c} b d = \frac{1}{6} \sqrt{25} \times 500 \times 145 = 60.416 \text{ kN} \]

Ultimate strength of slab section:

\[ a = \frac{A_s f_y}{0.85 f'_c b_c} = \frac{314.16 \times 500}{0.85 \times 25 \times 500} = 14.784 \text{ mm} \]

\[ M_u = \emptyset A_s f_y \left( d - \frac{a}{2} \right) = 0.9 \times 314.16 \times 500 \left( 145 - \frac{14.784}{2} \right) = 19.45 \text{ kN.m} \]

The load which subjected to the slab = 85% from \( M_u \)

\[
0.85M_u = 0.85 \times 19.45 = 16.533 \text{ kN.m}
\]

\[ 1.5 \times P = 16.533 \times 10^3 \rightarrow P = 11.022 \text{ kN} \]

∴ For 85% of ultimate load, use 2 point load \( P = 11.022 \text{ kN} \) for every point.

4.2.2 Design of steel plate section

When we loaded the composite section of slab and plate use two-point load \( P = 20 \text{ kN} \) for every point as shown in Figure 6.

![Figure 6. Shear force and bending moment diagrams.](image-url)
\( V_{u\ max} = 20\ kN \) and \( M_{u\ max} = 30\ kN.m \)

4.3.2.1 Design steel plate for flexural

Use: \( b_p = 300\ mm \)

\[
A_1 = \frac{b_p f_{yp}}{2} \left( 1 - \frac{b_p f_{yp}}{0.85 f'_c b_c} \right) = -26825.294
\]

\[
A_2 = b_p f_{yp} \left( h_c + d_a - \frac{A_S f_{ys}}{0.85 f'_c b_c} \right) = 12815496
\]

\[
A_3 = \left( A_S f_{ys} \left( d_s - \frac{A_S f_{ys}}{2 \times 0.85 f'_c b_c} \right) - \frac{M_u}{\phi} \right) = -11717868.0
\]

\[
\therefore \ d_{p_1} = 46.84\ mm; \ d_{p_2} = 0.937\ mm \rightarrow \ d_p = 1.0\ mm
\]

4.2.3 Finding \( t_p_b \)

(Balanced thickness of steel plate)

At balanced condition inertial steel reaches yield concurrently which extreme concrete fiber reaching crushing strain.

\[
\frac{f_y}{E_S} = \frac{500}{200000} = 2.5 \times 10^{-3}
\]

\[
\varepsilon_c \ at \ crushing = 0.003
\]

\[
\frac{X_b}{h_s - X_b} = \frac{X_b}{0.003} = \frac{2.5 \times 10^{-3}}{145 - X_b} \rightarrow \ X_b = 79.091\ mm
\]

\[
t_p_b = \frac{\beta_1 X_b (0.85) f'_c b_c - A_S f_y}{b_p f_{yp}} = \frac{300 \times 270}{6.88} = 6.88\ mm
\]

\[
t_b = 6.88\ mm > d_p = 1.0\ mm \quad \therefore \ O.K
\]

4.2.4. Check interface stresses

Find \( h \) = [Depth of the center of area of the cracked composite (repaired) section to the extreme of compression zone]

\[
h = \frac{-B \pm \sqrt{B^2 + 4AC}}{2A}
\]

\[
A = \frac{E_c \times b_c}{2 E_p} = \frac{4700 \sqrt{25 \times 500}}{2 \times 200000} = 29.375
\]

\[
B = A_S + b_p \times d_p = 314.16 + 300 \times 1.0 = 614.16
\]

\[
C = h_S \times A_S + h_p \times b_p \times d_p = 145 \times 314.16 + 173 \times 300 \times 1.0 = 97453.2
\]

\[
h = 48.1\ mm
\]

\[
l_p = \frac{300 \times 1.0^3}{12} = 25\ mm^4
\]

\[
l_{comp} = \frac{E_c b_c h^3}{3 E_p} + A_S (h_S - h)^2 + b_p d_p (h_p - h)^2 = 9809167.1\ mm^4
\]

Use \( E_a = 5200\ MPa;\ v = 0.28 \)

\[
G_a = \frac{E_a}{2(1 + v)} = \frac{5200}{2(1 + 0.28)} = 2031.25\ MPa
\]
\[ K_S = G_a \frac{b_a}{d_a} = 2031.25 \times \frac{300}{3} = 203125 \]

\[ K_n = E_a \frac{b_a}{d_a} = 5200 \times \frac{300}{3} = 520000 \]

\[ C_{all} = 3.5 \text{ MPa}; \quad P_S = 40 \text{ kN}; \quad M_u = 30 \text{ kN.m}; \quad \alpha_1 = 35; \quad \alpha_2 = 1.1 \]

\[ a_0 = \text{distance from end of plate to support.} \]

\[ C_{R_2} = d_p \left( \frac{K_n}{4E_p l_p} \right)^{\frac{1}{4}} = 0.402 \]

4.3.2.5 Check the interface stresses at end of plate

\[ C_{R_1} = \left[ 1 + \left( \frac{K_S}{E_p b_p d_p} \right)^{\frac{1}{2}} a^* \right] b_p d_p \left( h_p - h \right) = 2.468 \times 10^{-4} \]

\[ a^* = \frac{M_o}{V_o} = \frac{632 \times 10^6}{20 \times 10^3} = 316 \text{ mm} \quad \text{(at end of plate)} \]

\[ M_o = \frac{70 \times 30 \times 10^6}{1500} = 1.4 \times 10^6 \text{ N.mm} \]

\[ V_o = 20000 \text{ N} \]

\[ V_o = 20000 \text{ N} \]

\[ C_{R_1} = 6.459 \times 10^{-5} \]

\[ \tau_o = \alpha_1 f'c_1 \left( \frac{C_{R_1} V_o}{f_c} \right)^{\frac{5}{4}} = 14.283 \text{ MPa} > C_{all} = 3.5 \text{ MPa} \quad \therefore \text{not O.K.} \]

The value of \( a_0 \) must be decreased to arrive that: \( \tau_o + \sigma_o \tan(28) < C_{all} \)

Use \( a_0 = 70 \text{ mm} \)

\[ V_o = \frac{70 \times 30 \times 10^6}{1500} = 1.4 \times 10^6 \text{ N.mm} \]

\[ V_o = 20000 \text{ N} \]

\[ \frac{M_o}{V_o} = \frac{1.4 \times 10^6}{20000} = 70 \]

\[ C_{R_1} = 6.459 \times 10^{-5} \]

\[ \tau_o = 2.673 \text{ MPa} \]

\[ \sigma_o = \tau_o \frac{C_{R_2} \alpha_2}{2.673 \times 0.402 \times 1.1} = 1.182 \text{ MPa} \]

4.3.2.6 Check shear capacity of composite section

\[ V_{up} = V_c + V_s \]

\[ V_s = 0 \quad \text{(no shear reinforcement)} \]

\[ V_{up} = V_c = \frac{1}{6} \left( \sqrt{\frac{f'}{c_1} + 100 \rho_w M_{ud}} \right) b_p d \]

\[ \rho_w = 0.0043; \quad V_{ud} = 20000 \text{ N}; \quad M_{ud} = 2.9 \times 10^6 \text{ N.mm} \]

\[ V_c = \frac{1}{6} \left( \sqrt{25 + 100 \times 0.0043 \times \frac{20000}{2.9 \times 10^6}} \right) \times 500 \times 145 \times 10^{-3} = 60.452 \text{ kN} > 20 \text{ kN} \quad \text{ok} \]
5. Discussion of results
Present study assumed increases the width of the external steel plate with decreasing its thickness to maintain the same critical section area (As shown in Table 1), this may lead to increase the interface shear resistance due to increase the surface area of the adherence. Furthermore, the reduction of the thickness may reduce the internal stresses resulting from the bending loads. The failure expected to occur is limited due to the bending, and vice versa, when increasing the thickness and decreasing the width of the plate, also this will lead to decrease the shear strength and the possibility of a shear failure before the occurrence of bending failure which cause to decrease the expected maximum load. In the case of using the perforated plate with keeping the same dimensions of the solid one (as shown in table 1 and 4), this will reduce the area of the critical section which led to reduce the value of the ultimate moment strength. At the same time, the holes will increase the surface area of contact with epoxy. Furthermore, the epoxy material filling the holes will act as nails and increasing the shear resistance against the adhesive ripping. However, when using the perforated plate and adding the area of the holes to the critical section by increasing the width of the plate to maintain the value of the ultimate moment strength (as shown in table 1 and 4), the presence of the holes will lead to increase the surface of the interface area as well as the presence of the epoxy nails formed inside the holes will increases the shear resistance. This method is considered the best way of increasing the capacity of the slab sections to the flexural loads and ensures that there is no sudden failure due to interface stresses. It is noticed in Figure 3 that when using a perforated plate instead of solid one the interface shear stresses will be decrease by (14.8%) if the ratio of \((b_p/d_p) = 300\),and if the ratio of \((b_p/d_p)\) increasing this will lead to a decrease in the interface shear stresses, as well as decrease in the difference between the effect of using the solid and perforated steel plates on interface stresses, also when reducing ratio of \((b_p/d_p)\) the interface shear stress will be increase. It is possible to increase the shear strength by specifying a suitable diameter of the holes as well as their number in the section and along the length of the plate (as shown in table 4). The change in the number and diameters of the holes in the external steel plate has an opposite effect on the resistance of the composite section, on the one hand, reducing the diameter of the holes and increasing their number leads to an increase in the interlock between the plate and the epoxy, thus reducing the peeling failure of the plate. On the other hand, increasing the diameter of the holes and reducing their number will lead to increase the cross-section area of the epoxy nails, thus increase the resistance to the shear stresses between the plate and the epoxy and increasing the total strength of the composite section as a whole. However, the present theoretical study showed that changing the number and diameters of the holes from (10 # 10 mm) to (2 # 50 mm) with keeping the cross-section area of the steel plate constant for both cases increased the resistance of the epoxy nails to the shear stresses that transmitted to them from the axial stresses in the plate by (400%).

6. Conclusion
The main objective of the research is to study the effect of changing the dimensions of the plate section as well as the effect of using perforated plate for the bending resistance when repairing one-way slab in this method, as the concrete panels were loaded with 85 percent of the ultimate load and then reinforced by external plate. The theoretical calculations show the following:

1. Increasing thickness of the solid plate led to increase the interface shear stresses by (72.6%) when changing dimensions of the plate section from (300mm×1mm) to (200mm×1.5mm). Furthermore, increasing width of the solid plate led to decrease the interface shear stresses by (48%) when changing the plate dimensions (300mm×1mm) to (400mm×0.75mm).
2. When using perforated plate instead of solid one this may reduce the interface shear stresses. It is noticed that when using a perforated plate section with \((b_p/d_p) = 300\) educes the shear stresses by (14.8%) in compare with the case of using the solid plate. Increasing \((b_p/d_p)\) ratio may lead to decrease the interface
shear stresses, as well as, decreasing the difference between the effect of using the solid and perforated steel plates on interface stresses. Furthermore, when reducing ratio of \((b_p/ \bar{d}_p)\) the interface shear stress will be increase.

4. using perforated plate increases the surface area of the contact between plate and epoxy, as well as, the epoxy which filling the holes will act as a nail to increase the plate fixity and the resistance to shear stress in the interface layer.

5. Changing the number and diameters of the holes from (10 # 10 mm) to (2 # 50 mm) with keeping the cross-section area of the steel plate constant for both cases increased the resistance of the epoxy nails to the shear stresses that transmitted to them from the axial stresses in the plate by (400%).

6. The new expression which presented in current study to calculate the interface shear stress to play a role to facilitate the process of calculating these stresses. However, results shows that expression give a good prediction for the shear stress in the interface layer.

**Notation**

- \(a\) = depth of compression block,
- \(A_p, A_s\) = area of plate and internal reinforcement respectively,
- \(b_c\) = width of concrete section,
- \(b_p, \bar{d}_p\) = width and thickness of plate respectively,
- \(C\) = compression force,
- \(f'_c\) = compression strength of concrete,
- \(f_{yp}, f_{ys}\) = yield stress of plate and internal reinforcement respectively,
- \(h\) = distance from center of area of composed section to the top of concrete section,
- \(h_c\) = depth of concrete section,
- \(M_{ul}\) = maximum ultimate moment,
- \(T_p\) = tension force of external plate,
- \(T_s\) = tension force of internal reinforcement.
- \(x_0\) = depth of natural axis from top of section,
- \(\varepsilon_c\) = concrete strain at top of compression section,
- \(\varepsilon_s\) = internal reinforcement strain.
- \(a^* = M_{o}/\nu_o\) (at end of plate zone),
- \(b_{a}, d_{a}\) = width and depth of epoxy layer,
- \(C_{all}\) = allowable coefficient of cohesion of the steel-glue-concrete interface (3.5 MPa),
- \(E_s, E_p, E_a\) = modulus of elasticity of internal steel, plate, and epoxy respectively,
- \(G_o\) = shear modulus of epoxy,
- \(I\) = moment of inertia of composite section,
- \(I_p\) = moment of inertia of plate,
- \(K_n = E_a b_o \bar{d}_a\) = normal stiffness of interface layer,
- \(K_s = G_a b_o \bar{d}_a\) = shear stiffness of interface layer,
- \(M_o, V_o\) = the ultimate moment and shear force at end of,
- \(\tau_o\) = peak shear stress,
- \(\sigma_o\) = peak peeling stress,
- \(\alpha_1, \alpha_2 = 35, 1.1\) = experimental regression coefficients [4]
- \(A_{P,cr}\) = critical section area of plate,
$I_{pcr} =$ moment of inertia of critical section of plate,

$I_{comp. cc} =$ moment of inertia of critical composite section of slab,

$a_0 =$ Distance from the end of plate to the support,

$t_{pb} =$ plate thickness at balanced load condition,

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