

Capacity Analysis of Precast Piles Connection with Cast in Situ Pillars

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Abstract: The dominant axial force on reinforced concrete bridge columns may affect the ability of pillars to respond to various lateral excitations. This study aimed to analyze the strength of concrete, tensile behavior, and shear strength in the pillar structure of a bridge. The method used to identify the behavior of pillars is to separately analyze the internal capacity of the pillars according to the characteristics of concrete and steel quality, cross-sectional shape, and the configuration of the reinforcement installed in three sections along the pillar height. By checking the adequacy of the lap joints and the length of reinforcement distribution, we assume that the dominant pillar is in tension or compression. We also assume that the dominance of tension in the reinforcement joints is in anticipation of a tensile response in the reinforcement joints when bending behavior is dominant (for tall pillars). *Curvature:* Analyzing the relationship of force-lateral displacement by simulating the height of the pillars from 1 to 5 m and comparing it with the shear capacity of the pillars so that a confining reinforcement spacing of 50 mm is obtained both on the precast pile and the additional cast in situ pier has been increasing the compressive strength and strain of the pillars so that with a factored axial compressive force of 730 kN, it produces compressive stress at the effective cross-section of the pillar of 13 MPa, which means that the strength of the concrete on the pillars is still sufficient. The shear capacity of the pillar obtained by factored loading is from 17 to 86 kN, so the pillar is still safe from shear failure along its height.

Keywords: bridge, capacity, connection, precast pillar.

現澆柱預製樁連接承載力分析

摘要：鋼筋混凝土橋樑柱上的主要軸向力可能會影響柱響應各種橫向激勵的能力。本研究旨在分析橋樑支柱結構中混凝土的強度、拉伸性能和剪切強度。識別支柱性能的方法是根據混凝土和鋼材的質量特性、截面形狀以及沿支柱高度分佈在三個截面上的鋼筋配置，分別分析支柱的內部承載力。透過檢查搭接接頭的適當性和鋼筋分佈的長度，我們假設主導支柱處於拉伸或壓縮狀態。我們也假設，當彎曲行為占主導地位時（對於高柱），鋼筋接頭中張力的主導地位是預期鋼筋接頭中的拉伸反應。曲率：透過模擬 1~5 米的樁高，與樁的抗剪承載力比較，分析力與側向位移的關係，得到預製樁與預製樁的圍護鋼筋間距均為 50 毫米。增設現澆墩柱，提高了柱體的抗壓強度和應變，當分解軸向壓力為 730 千牛時，在柱體有效截面處產生的壓應力為 13 兆帕，這意味著強度柱子上的混凝土仍然足夠。透過因子負荷獲得的支柱的抗剪承載力為 17 至 86 千牛，因此支柱沿其高度仍然不會發生剪切破壞。

关键词：桥梁、容量、连接、预制支柱。

1. Introduction

Developing traffic infrastructure, including roads and bridges, significantly improves regional socio-cultural and economic development [1, 2]. The primary function of bridges is to provide transportation to support the smooth flow of goods and services and community activities across rivers or other obstacles [3]. The ability of a bridge for optimal traffic services also closely relates to its shape or dimensions [4]. Meanwhile, another factor needed for a bridge to provide maximum service is the strength of the bridge construction itself. These factors are the basis for consideration of constructing appropriate and tough structural elements under existing conditions in the field. Hence, the resulting structural elements are safer, more comfortable, and more economical [5]. For example, the construction of a bridge in Kalimantan, Indonesia, that crosses river and peat land barriers. Bridge construction must consider the details of the bridge's structural elements. Elements that need more attention are the pillars or columns.

Reinforced concrete columns on bridges should meet the requirements for detailed reinforcement, both the need for longitudinal and transversal reinforcement and the interaction of the composite with the concrete [6]. There must be enough confinement reinforcement to stop the internal breaking pressures that act laterally on the concrete core if bridge piers are to have a high compressive axial capacity. In addition, the concrete, transverse reinforcement, and compressive force effects must provide sufficient shear strength to prevent rapid shear collapse of the pillars. For the proper work of pillars, all of this requires detailing probable plastification sites and outside plastic joints.

The application of additional pillars cast directly in the field (cast in situ) with the dimensions as the embedded precast piles may reinforce the configuration. The arrangement might be strengthened by adding more pillars formed in situ—directly in the ground—and having the same proportions as the embedded precast piles. Achieving the characteristics of the concrete material and reinforcing the new pillars that correspond with those of the old pillars is the main goal while managing the addition of new pillar connections (from precast pillars to cast-in-place pillars). If this is possible, the deformation and rotation behavior can occur that is at least somewhat similar to that of the two connected reinforced concrete pieces. This behavior must be treated in the connecting area to ensure that it continues unaltered along the pillar's height. This study analyzes the strength of concrete, tensile behavior, and shear strength in the pillar structure of a bridge.

2. Materials and Method

The research was carried out using field testing. The object of this research was the overpass Column in Kutai Kertanegara Regency, East Kalimantan. This bridge construction used a type of friction pile because land comprises peat soil whose friction pile relies on the transfer of load from the structure to the ground with the friction force between the surface of the pile and the soil around the pile [7, 8]. The entire surface of the pile transfers the load from the structure to the ground (Fig. 1).

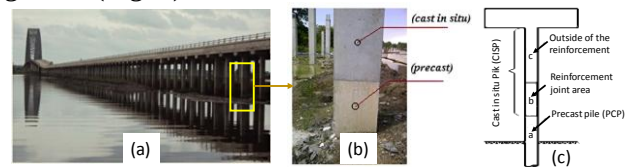


Fig. 1 Study site in (a) Bridge column in Kutai Kertanegara Regency, East Kalimantan, (b) Precast and cast in situ position, (c) Diagram of column structure (Developed by the author)

In this construction, problems were identified in connection with technical measures in the form of adding/connecting new non-stressed pillars, so it was suspected that there would be a change in the pillar's capacity to carry loads, which was identified as follows:

(1) There is a possibility that the flexural capacity decreases in the connection area between the strands in the form of wire from the pile and reinforcement from additional pillars as a result of a) The length of reinforcement distribution required to anticipate tensile behavior due to dominant bending and to anticipate dominant axial compression conditions is inadequate, b) The woven strands are loose and the bond to the new reinforcement is inadequate;

(2) It is possible that the average concrete strength of the piles and additional pillars is not the same, and the meeting of the two surfaces cannot be guaranteed as a monolith;

(3) The short shear capacity of the pillars may be vulnerable to the potential for dominant shear failure.

The method used to identify pillar behavior is to analyze the internal capacity of the pillar according to the characteristics of concrete quality, steel quality, cross-sectional shape, and configuration of reinforcement installed in three sections along the height of the pillar separately. For this reason, the following analysis is carried out by a) checking the adequacy of lap connections and the length of reinforcement distribution, assuming that the dominant pillar is experiencing tension or compression. The assumption of tensile dominance in reinforcement

connections is to anticipate the occurrence of a tensile response in reinforcement connections when bending behavior is dominant (for tall pillars); b) analyze the increase in concrete compressive strength in the compressive stress-strain relationship due to confinement configurations on piles and additional pillars; c) analyze the moment-curvature relationship of the pillar in three cross-sections in different segments along the height of the pillar (segments a, b and c) as in Fig. 1c; d) predict the position of the maximum moment and the potential for bending response; e) analyze the force-lateral displacement relationship by simulating the pillar height from 1 to 5 m, and compare it with the shear capacity of the pillar (Fig. 2).

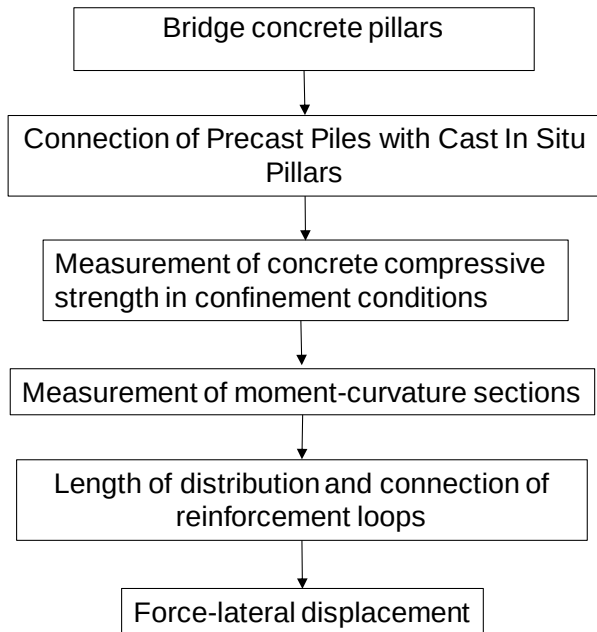


Fig. 2 Conceptual framework of the study (Developed by the author)

The results of the analysis will provide a qualitative picture of the piles, additional pillars, and connection areas so that it can be concluded that the pillar capacity is still adequate for loading. The analysis of pillar cross-sections, apart from producing moment-curvature relationships, also identifies the behavior of strains and internal forces in the cross-section according to the external axial forces applied. The parameters that influence the analysis are concrete quality and steel quality, including the stress-strain relationship model of the two materials as well as the configuration of longitudinal and transverse reinforcement.

Area A in Fig. 3 is a cross-section modeled according to the condition of the precast pile, whereas in area C, the cross-section is modeled according to additional pillars made by casting in situ. In the connection area (area B), the cross-section and configuration of the pillar reinforcement are modeled according to area A but using the quality of concrete as in area C. Cross-section analysis with Response 2000

Software was developed using Modified Compression Field Theory [9].

Connecting piles with new pillars is done by removing the remaining concrete at the end of the pile, connecting the reinforcement rods for new concrete pillars to a strand of piles with a distribution length of 50 cm, bending the strand in such a way that it can adjust from a circular longitudinal reinforcement configuration to a square longitudinal configuration, and creating the same distance from the transverse reinforcement in the connection area to the top end of the new pillar, namely 10 cm.

After handling the connection of additional reinforced concrete pillars in this project, several things have been identified that require special attention in the form of unusual conditions, and their feasibility has not been regulated in the existing regulations. Therefore, a review is needed that provides a second opinion that can compare existing design results with actual practical handling in the field. This review was not carried out comprehensively but only focused on the case of handling pillar connections due to the height of the piles not reaching the planned elevation. This analysis uses Response 2000 v 1.0.5 software.

The basic formula for concrete confined under monotonic loads was obtained by adopting the compressive stress-strain relationship [10]. An illustration of the compressive stress-strain relationship of confined concrete is shown in Fig. 3.

$$f_c = \frac{f_{cc}' x^r}{r - 1 + x^r} \quad (1)$$

$$x = \varepsilon_{co} / \varepsilon_{cc} \quad (2)$$

f_{cc}' - compressive strength of the confined concrete (to be determined later).

f_{co}' - unconfined compressive strength of concrete.

ε_{co} - longitudinal compressive strain of unconfined concrete, 0.002.

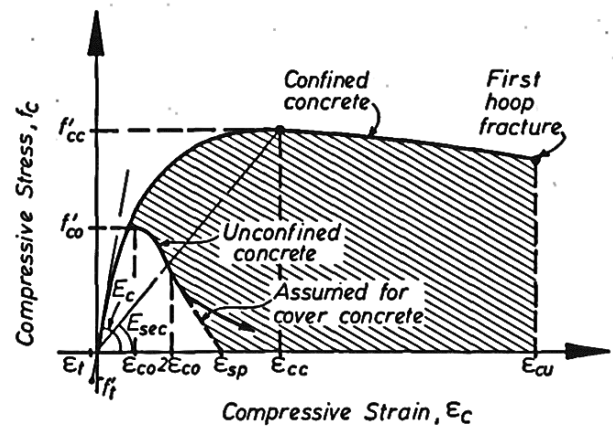


Fig. 3 Model of the compressive stress-strain relationship for concrete with monotonic loads between confined and unconfined [11-13]

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad (3)$$

$$r = \frac{E_c}{E_c - E_{sec}} \quad (4)$$

where r is the elasticity of concrete, $E_c = 4700\sqrt{f'_c}$ (adapted to SNI 1726 2012, (2012) and E_{sec} will be determined later according to the secant slope shown in Fig. 1 or with the following formula:

$$E_{sec} = \frac{f'_{cc}}{\varepsilon_{cc}} \quad (5)$$

2.1. Effective Restrained Lateral Pressure and Effective Restraint Coefficient

The maximum transverse pressure of the reinforcing steel restraining the concrete core can only be used effectively in the part of the concrete core where the restraining stress operates fully between the arching actions. Fig. 4 shows the arching action assumed to occur between the transverse reinforcement. The area enclosed by the center lines of the transverse spiral or hoop reinforcement is confined, A_{cc} , while the area enclosed by the arching action is an effectively confined area with a smaller area so that $A_e < A_{cc}$.

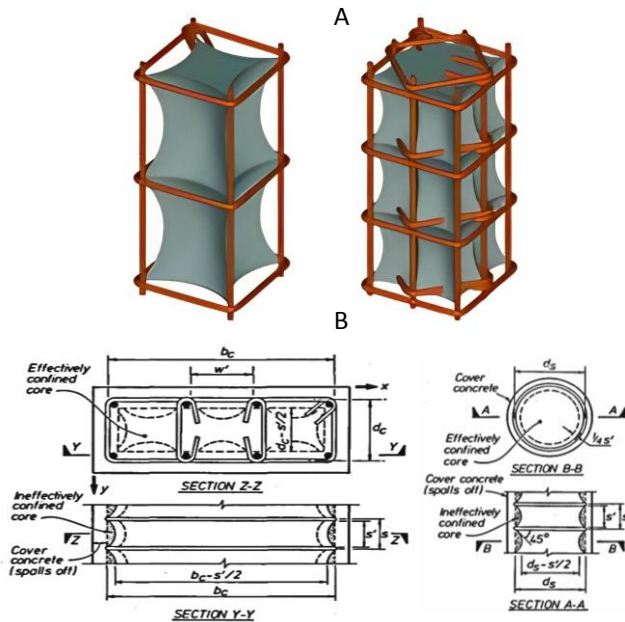


Fig. 4 Model of effective restraint area in a concrete core: a) arching model, b) square and circular stirrup reinforcement [13]

The effective lateral pressure on the concrete core is given by Eq. (7). Columns with a square cross-section

$$f'_{lx} = f'_{ly} = f'_l \quad (6)$$

$$f'_l = k_e f_l \quad (7)$$

$$k_e = \frac{A_e}{A_{cc}} \quad (8)$$

$$A_{cc} = A_c (1 - \rho_{cc}) \quad (9)$$

$$\rho_{cc} = \frac{A_s}{A_c} \quad (10)$$

The area of the concrete core bounded by the center line of the perimeter of the transverse reinforcement is A_c ($A_c = d_c b_c$ for square stirrups and $A_c = \frac{1}{4} \pi d_s^2$ for circular stirrups), whereas ρ_{cc} is the ratio of the total longitudinal reinforcement area (A_s) to the concrete cross-sectional area A_c . The lateral resistance pressure contributed by the transverse reinforcement is f_l as shown in Eq. (11).

$$f_l = \rho_{sh} f_{yh} \quad (11)$$

$$\rho_{sh} = \frac{A_{sh}}{s b_c} \quad (12)$$

The transverse reinforcement ratio for a square column is ρ_{sh} and f_{yh} is the quality or yield stress of the transverse reinforcing steel. The total area of hoop reinforcement is the number of legs of the transverse reinforcement toward the confinement under consideration expressed as A_{sh} , while s is the spacing of the transverse reinforcement from as to as of reinforcement. The outer dimensions of the concrete core from the center line of the transverse reinforcement are symbolized as b_c for length and d_c for width so that $b_c > d_c$ for rectangular columns and $b_c = d_c$ for square columns.

For restraints with circular or spiral transverse reinforcement, the balance between uniform lateral pressure and tensile stress of the reinforcement is as follows:

$$s d_s f_l = 2 A_{sp} f_{yh} \quad (13)$$

$$\rho_{sp} = \frac{4 A_{sp}}{s d_s} \quad (14)$$

$$f_l = \frac{1}{2} \rho_{sp} f_{yh} \quad (15)$$

where A_{sp} is the cross-sectional area of the spiral or circular reinforcement.

The effective confined area (A_e) for square columns via transverse hoop reinforcement with or without cross ties determined by assuming that the arching action is a parabola of degree two with a tangent angle of 45° . The arching action, which is formed vertically between the transverse hoops and horizontally between the longitudinal reinforcement, is an ineffective area. The effective unconfined area is given by Eq. (16), where w'_i is the clear distance of longitudinal reinforcement and n is the number of longitudinal reinforcements.

$$A_i = \sum_{i=1}^n \frac{(w'_i)^2}{6} \quad (16)$$

The value of the area that is effectively confined is the area of the concrete core that is confined minus the area of the part that is not effectively confined, so that it becomes as in Eq. (17).

$$A_e = \left(b_c d_c - \sum_{i=1}^n \frac{w_i^2}{6} \right) \left(1 - \frac{s'}{2b_c} \right) \left(1 - \frac{s'}{2d_c} \right) \quad (17)$$

$$A_e = \frac{\pi}{4} d_s^2 \left(1 - \frac{s'}{2d_s} \right)^2 \quad (18)$$

When the s' clear spacing of transverse reinforcement is measured from the outside. Referring to Eq. (8), the effective restraint coefficient is described as in Eq. (18) for square pillars and Eq. (19) for circular pillar sections.

$$k_e = \frac{\left(1 - \frac{1}{b_c d_c} \sum_{i=1}^n \frac{w_i^2}{6} \right) \left(1 - \frac{s'}{2b_c} \right) \left(1 - \frac{s'}{2d_c} \right)}{1 - \rho_{cc}} \quad (19)$$

$$k_e = \frac{\left(1 - \frac{s'}{2d_s} \right)^2}{1 - \rho_{cc}} \quad (20)$$

The compressive stress of confined concrete based on triaxial pressure can be expressed using the formula proposed by [11].

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad (21)$$

2.2. Determination of the Moment–Curvature Relationship of the Column Sections

The moment–curvature relationship of the pillar cross-section provides an idea of the inelastic behavior of the structural element. The characteristics of the concrete and reinforcing steel materials greatly influence this behavior. The concrete constitutive model in the cross-section consists of a confined concrete constitutive model as in Eq. (1) and a concrete constitutive model without restraint, namely by assuming $f'_l = 0$. If the concrete compression block in the column cross-section analysis is modeled on the basis of the stress–strain relationship conditions provided in Eq. (1), then the determination of the area of the compressed block in concrete is illustrated as in Fig. 5 for a strain value of ϵ_{cm} from the concrete stress–strain curve.

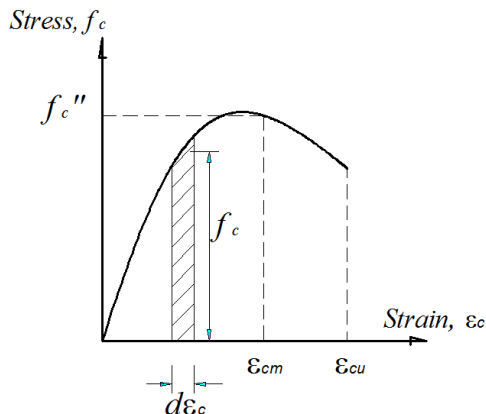


Fig. 5 Compressive stress block area from the actual concrete stress–strain curve (Developed by the author)

The area under the curve in the form of a multiplier was calculated according to equations

$$\int_0^{\epsilon_{cm}} f_c d\epsilon_c = \alpha f'_c \epsilon_{cm} \quad (22)$$

$$\alpha = \frac{\int_0^{\epsilon_{cm}} f_c d\epsilon_c}{f'_c \epsilon_{cm}} \quad (23)$$

The central factor of the concrete compressive force (γ) on the cross-section is determined by the principle of the moment of the area under the concrete stress–strain curve according to Eq. (22).

$$\gamma = 1 - \frac{\int_0^{\epsilon_{cm}} f_c \epsilon_c d\epsilon_c}{\epsilon_{cm} \int_0^{\epsilon_{cm}} f_c d\epsilon_c} \quad (24)$$

The compressive axial force under balanced conditions for longitudinal reinforcement spread over all four sides of the cross-section is expressed as follows:

$$P_r = \alpha f'_c b_{co} c_b + \sum_{i=1}^n A_{si}' f_{si}' + \sum_{i=1}^n A_{si} f_{si} \quad (25)$$

The moment about the neutral line is expressed according to Eq. (24).

$$M_r = (\alpha f'_c b_{co} c_b) \left(\frac{h_c}{2} - \gamma c_b \right) + \sum_{i=1}^n A_{si}' f_{si}' \left(\frac{h_c}{2} - d_i \right) + \sum_{i=1}^n A_{si} f_{si} \left(\frac{h_c}{2} - d_i \right) \quad (26)$$

where C_s and T_s are the compressive and tensile forces of the reinforcement, respectively, A_{si}' and A_{si} are the cross-sectional areas of the i row of reinforcement in the tension and compression areas, respectively [15]. For every increase in compressive strain ϵ_{cm} it will result in a change in the height of the neutral line kd or c and a change in the strain of the reinforcing steel in the section area experiencing tension [16].

The stress–strain behavior of the reinforcing steel is modeled using the following equations [16]: If it is still in an elastic condition, that is if $0 \leq \epsilon_s \leq f_y$, so:

$$f_s = E_s \epsilon_s \quad (28)$$

When the first melting has been exceeded and $f_y \leq \epsilon_s \leq \epsilon_{sh}$, so:

$$f_s = f_y \quad (29)$$

When strain hardening is exceeded, $\epsilon_{sh} \leq \epsilon_s \leq f_{su}$

$$f_s = p f_y \quad (30)$$

where p is the coefficient defined as follows:

$$p = \left[\frac{m(\epsilon_s - \epsilon_{sh}) + 2}{60(\epsilon_s - \epsilon_{sh}) + 2} + \frac{(\epsilon_s - \epsilon_{sh})(60 - m)}{2(30q + 1)^2} \right] \quad (31)$$

$$m = \frac{\left(\frac{f_{su}}{f_y} \right) (30q + 1)^2 - 60q - 1}{15q^2} \quad (32)$$

$$q = f_{su} - \epsilon_{sh} \quad (33)$$

Furthermore, analysis of the curvature moment relationship was performed using Response 2000 software. The cross-sectional analysis assumption in Response 2000 does not differentiate the behavior of

the compressive stress-strain relationship between concrete in stressed areas and those without constraints (concrete blankets).

2.3. Bracing Reinforcement Requirements

The minimum stirrup reinforcement requirements for square restraints in plastic hinge areas are set out in AASHTO-LRFD Bridge Design Specifications (2010) Sec. 5.10.11.4.1d, which is no less than the largest in Eqs. (29) and (30).

$$A_{sh} = 0.3sh_c \left(\frac{f'_c}{f_{yh}} \right) \left(\frac{A_g}{A_{ch}} - 1 \right) \quad (34)$$

$$A_{sh} = 0.12sh_c \left(\frac{f'_c}{f_{yh}} \right) \quad (35)$$

For columns with spiral or circular reinforcing reinforcement, this requirement is expressed in the reinforcement ratio ρ_{sp} .

$$\rho_{sp} = 0.12 \left(\frac{f'_c}{f_{yh}} \right) \quad (36)$$

These three equations are also recommended by ACI 343R-95, Analysis and Design of Reinforced Concrete Bridge Structures Sec. 11.6.4.6, as well as within ACI-318M-11 (2011) Sec. 21.6.4.4(b).

The minimum tie bar usage requirements for square columns based on CALTRANS Bridge Design Specifications Sec. 8.18.2.3.1 [16] shall not be less than the greatest of Eqs. (32) and (33).

$$A_{sh} = 0.3sh_c \left(\frac{f'_c}{f_{yh}} \right) \left(\frac{A_g}{A_{ch}} - 1 \right) \left(0.5 + \frac{1.25P_u}{A_g f'_c} \right) \quad (37)$$

$$A_{sh} = 0.12sh_c \left(\frac{f'_c}{f_{yh}} \right) \left(0.5 + \frac{1.25P_u}{A_g f'_c} \right) \quad (38)$$

For columns with spiral or circular confining reinforcement, this requirement is expressed as the reinforcement ratio ρ_{sp} . For columns with spiral or circular confining reinforcement, this requirement is expressed as the reinforcement ratio.

$$\rho_{sp} = 0.12 \left(\frac{f'_c}{f_{yh}} \right) \quad (39)$$

$$\rho_{sp} = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \left(\frac{f'_c}{f_{yh}} \right) \quad (40)$$

If the compressive axial force is considered,

$$\rho_{sp} = 0.45 \left(\frac{A_g}{A_{ch}} - 1 \right) \left(\frac{f'_c}{f_{yh}} \right) \left(0.5 + \frac{1.25P_u}{A_g f'_c} \right) \quad (41)$$

$$\rho_{sp} = 0.12 \left(\frac{f'_c}{f_{yh}} \right) \left(0.5 + \frac{1.25P_u}{A_g f'_c} \right) \quad (42)$$

2.4. Column Shear Capacity

The shear capacity regulations ACI 434R-95 (1995)

and ACI 318M-11 (2011) refer to the principle of the shear capacity of structural elements as a contribution from the shear capacity of concrete and transverse reinforcement $V_n = V_c + V_s$. The shear force contributed by the concrete is given by Eq. (38).

$$V_c = 0.17 \left(1 + \frac{N_u}{14A_g} \right) \lambda \sqrt{f'_c} b_w d \quad (\text{MPa}) \quad (43)$$

In ACI 343R-95 (1995), all the V_c formulations do not consider the influence of the factor $\lambda = 0.7$; therefore, in all cases, the value from ACI 318M-11 (2011) is always slightly lower.

For detailed calculations of the shear strength of structural elements that bear axial compression, bending, and shear forces, you may use ACI 318M-11 (2011) sec. 11.2.2.1 sec. 11.2.2.2 as expressed in Eq. (39).

$$V_c = \left(0.16\lambda \sqrt{f'_c} + 17\rho_w \frac{V_u d}{M_u} \right) b_w d \quad (\text{MPa}) \quad (44)$$

where $\rho_w = A_s/b_w d$, and the value M_m can replace M_u , which is expressed as

$$M_m = M_u - N_u \frac{(4h - d)}{8} \quad (45)$$

but the value of Eq. (39) cannot be greater than that given in Eq. (41).

$$V_c = 0.29\lambda \sqrt{f'_c} b_w d \sqrt{1 + \frac{0.29N_u}{A_g}} \quad (\text{MPa}) \quad (46)$$

For pillars with the combined influence of bending, shear and axial compression forces must be met $V_u d/M_u < 1.0$. However, if the value M_m replaces M_u , then it is permissible $V_u d/M_u > 1.0$. Component $\lambda \sqrt{f'_c}$ in the equations above is the tensile stress of concrete, with values $\sqrt{f'_c}$ may be taken to exceed 8.3 MPa.

Requirements for shear resistance contributed by transverse reinforcement perpendicular to the axial axis of the structural element for square stirrups according to ACI 318M-11 (2011).

$$V_s = A_v f_{yh} d/s \quad (47)$$

For circular transverse reinforcement, it is as follows:

$$V_s = \frac{\pi}{2} \frac{A_v f_{yh} d_s}{s} \quad (48)$$

2.5. Length of the Reinforcement Distribution

It is assumed that the length of the strand prestress deliberately created at the end of the pile by removing part of the concrete before adding the new pillar is the length of the distribution of the l_d strand reinforcement that enters the new concrete. Checking the adequacy of the distribution length refers to SNI 03-2847-2002, Sec. 14.2.

$$\frac{l_d}{d_b} = \frac{12 f_y \alpha \beta \lambda}{25 \sqrt{f_c'}} \quad (49)$$

or

$$\frac{l_d}{d_b} = \frac{9 f_y}{10 \sqrt{f_c'}} \frac{\alpha \beta \gamma \lambda}{c + K_{tr}} \quad (50)$$

where $(c + K_{tr})/d_b > 2.5$, $K_{tr} = \frac{A_{tr} f_{yt}}{10sn}$, $\sqrt{f_c'} < 25/3$ and l_d not less than 300 mm. The α factor is the reinforcement location factor, β is the coating factor, γ is the reinforcement rod size factor and λ is the lightweight aggregate concrete factor, all of which are regulated in SNI 03-2847-2002. The value of $\alpha\beta$ must not be less than 1.7, where c is the dimensional spacing of the concrete blanket and d_b is the diameter of the reinforcement. The A_{tr} value is the cross-sectional area of all transverse reinforcement in the spacing range s . The required yield strength for transverse reinforcement is f_{yt} , and ns is the number of rods or wires distributed along the split plane [16].

The distribution length for threaded rods under compression is

$$l_{db} = \frac{d_b f_y}{4 \sqrt{f_c'}} > 0.04 d_b f_y \quad (51)$$

2.6. Pass Connection

The minimum length of a tensile lap connection must be taken on the basis of the class requirements as regulated in SNI 03-2847-2002, Sec. 14.15, i.e. $1.0 l_d$ for class A connections and $1.3 l_d$ for class B connections, but not less than 300 mm.

The minimum lap connection (l_d) for reinforcement under compression is checked in accordance with the general provisions in SNI 03-2847-2002, Sec. 14.16, as well as in CALTRANS Bridge Design Specifications, Sec. 8.32 [17] as follows:

$$l_d = 0.07 d_b f_y \text{ for } f_y \leq 400 \text{ MPa}$$

$$l_d = (0.13 f_y - 24) d_b \text{ for } f_y > 400 \text{ MPa}$$

For f_c' less than 20 MPa, the length of the pass must be multiplied by 1.3. If the gap exceeds $0.0015hs$ in the ties along the joint area, multiplying by 0.83 may reduce the penetration joint.

Analysis of the lateral force–displacement relationship and some descriptions of the characteristics of pillar structural elements can also be obtained from the software after simulating several lengths of pillar elements.

3. Results and Discussion

3.1. Effect of Restraint (Confinement)

The increase in the compressive capacity of concrete pillars can be identified from the stress–strain

relationship between confined concrete (CC) and unconfined concrete (UCC), as shown in Fig. 6. The abbreviation PCP is a precast pile, and CISP is the abbreviation for cast in situ pile. The compressive strength of concrete on piles and the stress–strain relationship characteristics appear to be higher than those provided with additional pillars in unconfined and confined conditions [18].

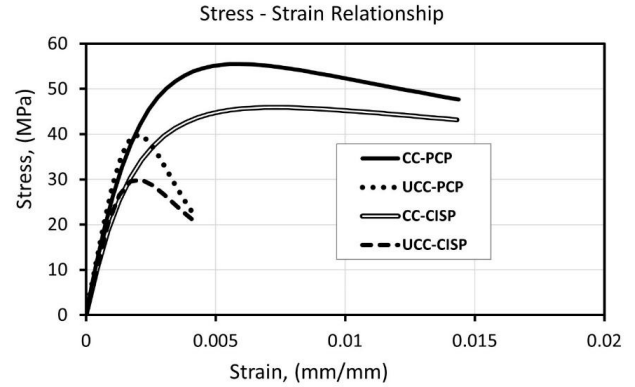


Fig. 6 Concrete stress-strain relationship (Developed by the author)

With a factored design compressive axial force of 730 kN, it produces a compressive stress of 13 MPa on the effective cross-section of the pillar. This means that the strength of the concrete in the pillars is still sufficient.

3.2. Moment–Curvature Relationships of the Cross Sections

The results of the analysis of the moment–curvature relationship in the three pillar cross-section conditions are shown in Fig. 7. The cross-sections with codes PCP 40×40 (area A), connection (area B), and CISP 40×40 (area C) are each assumed if the maximum moment or plastification occurs at that part.

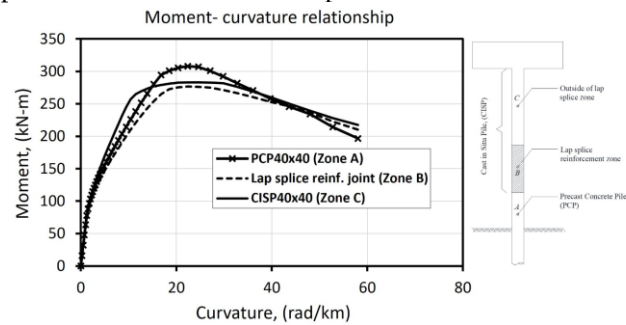


Fig. 7 Moment–curvature relationship of the pillar cross-section (Developed by the author)

The moment–curvature behavior of the pillars in the three cross-sections is considered still coincide when they are still in a linear condition or in the condition before cracking. Likewise, the moment before the first melting occurred did not experience a significant difference. Table 1 shows these results.

Table 1 Moment identification results under several cross-sectional conditions (Developed by the author)

Section	Unit	1 st crack	1 st yield	Peak	Ultimate
PCP 40×40 (area A)	kN-m	78	280	301	196
Connection (area B)	kN-m	71	265	276	210
CISP 40×40 (area C)	kN-m	76	257	282	218

Clarke et al. [18] provided an illustration of the potential for maximum moments in embedded pillars as shown in Fig. 8, where the potential for plastification in pillars occurs in the range of 2D to 5D below the original ground level [19]. From this figure, the conditions that allow this to occur are as follows: 1) if the connection is carried out on the original ground surface and the behavior of the pillar is as a single bending, then the maximum moment can be ensured not to occur in the connection area, or the spread of energy through damage will not occur at the connection; 2) if the connection is in the maximum moment area, this means that plastification will occur in that connection area, so it is necessary to check the pillar capacity by continuing to use precast cross-sections and reinforcement configurations but using concrete quality such as cast-in-place concrete quality (curvature in area B); and 3) if the two previous conditions are safe and acceptable then further checking of shear is necessary.

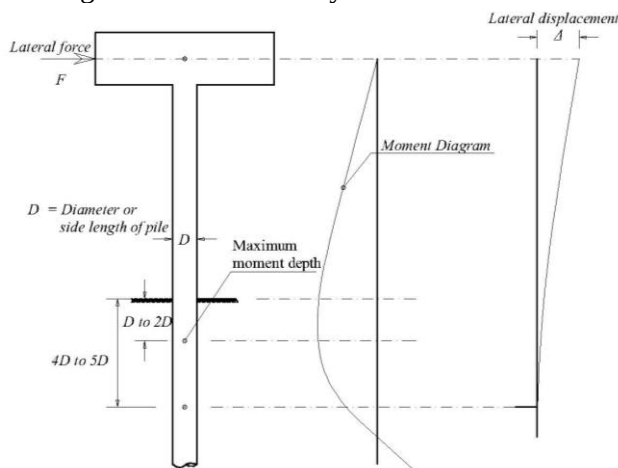


Fig. 8 Areas with the greatest potential to experience maximum moment and plasticization [5]

Looking at the conditions in the field where all connections occur at elevations above ground level, the potential for plastic joints to form will fall on the cross-section in area A or on precast piles. With this approach, the curvature moment relationship that best represents the actual condition of the pillar is the curvature moment relationship on the PCP 40×40 cross-section (area A) in Fig. 8.

If the factored moment in the design is 88 kN-m compared with the results of the cross-sectional analysis, the pillar capacity is still sufficient and with a factored moment of that size, the pillar reinforcement

has not experienced the first yield.

3.3. Lateral Force-Displacement Relationships

The lateral displacement force relationship was obtained from further development of the cross-sectional analysis in Response 2000 by adding an element length component to the analysis. These results provide a better picture of the structural behavior of pillar elements, which can be used as a basis for checking the strength and ability of pillar structures to undergo inelastic deformation. The lateral force-displacement relationship was also used to check that shear along the height of the pillar provided a uniform response (Fig. 9).

Image of the analysis results provides variations in the span length of the cantilever pillar from 1 to 5 m with a compressive axial force of 730 kN. If the length of the pillar is reduced, the shear strength of the pillar increases; however, this increase is accompanied by a decrease in the lateral inelastic displacement deformation of the pillar. The characteristics of the lateral displacement force relationship of the pillars also do not appear to show significant differences between those modeled as PCP, connection conditions, or CISP. The largest shear response is experienced by pillars with a height of only 1 m, and in this condition, the pillars are very brittle and less capable of inelastic deformation.

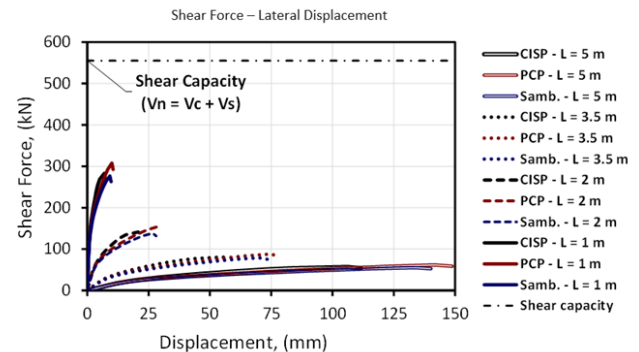


Fig. 9 Pillar lateral force-displacement relationship (Developed by the author)

The pillar shear capacity obtained from the contribution of concrete and transverse reinforcement also appears to be quite high compared with the shear response of each pillar element. If the factored shear force of the pillar works between $F_u = 17$ and 86 kN, then the pillar is still safe from shear failure along its height.

3.4. Length of Distribution and Connection of Reinforcement Loops

The requirements for the length of strand wire distribution into additional concrete pillars and the length of the longitudinal reinforcement passing connections are shown in Table 2. The two connection lengths considered under tension and compression conditions are then compared with actual conditions. The calculation results show that the lap connections

and actual distribution lengths in the pillar structure meet the requirements.

Table 2 Requirements for the distribution length and length of reinforcing loop connections (Developed by the author)

Condition	Delivery length, mm	Late connection (mm)	Actual length (mm)
Tensile	466.22	389.1	500
Strength	303.53	465.5	500

4. Conclusion

The confining reinforcement spacing of 50 mm in the PCP and additional PICP has increased the compressive strength and strain of the pillar so that with a factored design axial compressive force of 730 kN, it produces a compressive stress of 13 MPa in the effective cross-section of the pillar, which means that the strength of the concrete in the pillars is still sufficient. Compared with previous research using pin or socket connections, the addition of PICP was quite successful in increasing compressive and connection strengths. Considering that all connection conditions occur above ground level, the possibility of a maximum moment forming in the potential connection area is small. If the maximum moment is formed in the pile area (PCP) or the additional pillar area (CISP), then based on analysis of the moment-curvature behavior of the cross-section, the pillar can still be ductile under all these conditions. The shear capacity of the pillar obtained from the contribution of concrete and transverse reinforcement is still high compared to the shear response of each pillar element when experiencing a factored load of $F_u = 17$ to 86 kN so that the pillar is still safe from shear failure along its height. The lap connections and actual length of distribution in the pillar structure meet the requirements so that the strength of the connection area is sufficient to develop maximum reinforcement strain.

This study recommends further ensuring that the behavior of pillars in the form of connections between precast piles and cast in situ pillars can be more reliable by performing experimental tests on models of pillar elements connected with a combination of compressive axial forces and lateral forces. Another alternative to ensure that the behavior of pillars is by predictions is through non-linear finite element model analysis, where the concrete elements are applied as solid, and the reinforcement is modeled as link elements.

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