An Approach for Plastic Hinge Length of Reinforced Concrete Columns

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**Abstract.** Plastic hinge length is the crucial parameter in predicting the nonlinear response of structural elements. Due to the complexity of material nonlinearity, the accurate estimation of the plastic hinge length has been facing many difficulties. In past decades, some definitions and models were proposed for predicting this length, however, the results had a large discrepancy. Therefore, the paper presents a general approach of plastic hinge length that based on some criteria, i.e., strain profiles of rebar, peak strain of concrete cover and core, curvature profiles, and physical observation. In addition, a series of four full-scale reinforced concrete columns were implemented and tested under high axial load ratio and different transverse reinforcement details. The tested results implied that it is necessary to separate the plastic hinge lengths based on different criteria due to the large discrepancy. The high compression load could enhance the plastic hinge lengths based on compressive yield strain of rebar, peak strain of concrete, and curvature profiles. In contrast, it had a minor effect on plastic hinge length based on tensile yield strain. Besides, the amount of transverse reinforcement had an insignificant effect on plastic hinge lengths. Finally, some discussions on plastic hinge lengths of tested columns were established in this study.

**Keywords:** plastic hinge, plastic hinge length, plastic hinge region, reinforced concrete column, concrete structure.

1. INTRODUCTION

The structure seismic design is complemented successfully if the earthquake energy could be dissipated in the best way. There were many mechanisms to do this, one effective mechanism is to permit informing the plastic hinges on the structures. These plastic hinges would be informed at the ends of reinforced concrete (RC) beam or column. In the plastic region length, the RC member behavior was inelastic. So far, there were some definition of plastic hinge length (PL), such as Yuan et al. [1], Megalooikonomou et al. [2], Pam et al. [3], etc.… According to Yuan et al. [1], the PL was known as the length where RC members experience substantial plastic deformation and severe damage under extreme loads. Besides, Megalooikonomou et al. [2] defined the PL was as the length over a seismically swaying column where flexural moments exceed the yielding capacity. This length, measured from the critical section towards the shear span, signifies the region where intense inelasticity occurs during the earthquake, and is determined in design codes through calibrated empirical relationships that account primarily for the length of the shear span and the diameter of primary reinforcing bars. Yet, Pam et al. [3] defined the length of the critical region is one of the essential parameters in designing earthquake resistant where adequate transverse steel needs to be provided to confine the concrete core and subsequently to avert brittle failure under large inelastic deformation or curvature. Furthermore, the PL is the most important parameter for both seismic retrofitting of old structures and designing of new structures. Retrofitting of old structures need to know the PL to determine the extent of retrofitting, and also need it to calculate the ultimate displacement and ductility in design. However, due to the high nonlinearity of materials and complicated interaction between constituent materials, most researchers studied the PL through the experiments. Numerous models of PL have been proposed for RC columns as presented in Table 1. It is worth that the result of these models had a large difference, and causes the difficulty in choosing which suitable equation to apply for estimating the PL.

In addition, some researchers [4–6] pointed out that the PL depended on many factors, such as moment gradient, level shear stress in plastic hinge region that known as tension shift effect, mechanical properties of longitudinal bars that known such as the strain penetration effect, mechanical properties of transverse reinforcement, level of axial load, concrete and steel strength, level of confinement. Many experiments were implemented to observe PL, however, there is the lack of test on full-scale, double curvature, high confinement level under high axial load ratio (ALR). Therefore, four full-scale RC columns with high confinement level under high ALR were conducted to investigate the seismic behavior and PL. Through tested results and test database, a general approach for PL was proposed that based on the peak strain of concrete core and cover, the strain profiles of rebar, the curvature profiles, and physical observation. Yet, a revised equation is also established to estimate the PL with an acceptable accuracy.

1. SOME EXISTING MODELS FOR PLASTIC HINGE LENGTH

Corley [7] and Mattock [8] were the early researchers that tested forty simply RC beams subjected singe point loads with the confinement and size effect variables. They reported that the PL was a function of geometry, and the size of beams did not have a significant influence on the rotation capacity. They also proposed a PL equation as presented in Table 1. Besides, from previous achievements Park et al. [9] conducted four columns with dimension of (550×550×3300)mm, and the axial load ratio (ALR) ranged from 0.2 to 0.6. Park et al. showed that the calculated equivalent PL were insensitive to axial load ratio. Due to limiting the number of test columns, Park et al. also concluded that the PL should be taken as ( is the cross-section height) for simple and safe approximation. In addition, Mander [10] tested four hollow columns and concluded that the contributions of plastic deformation were from two factors, i.e., the spread of plasticity along the member length due to the moment gradient, and the yield penetration of the longitudinal. Mander also proposed an equation to estimate the PL as illustrated in Table 1. While Priestley et al. [11] reported that an elasto-plastic approximation should consider a PL proportional to the column height, and proposed the equation to estimate the PL as presented in Table 1. However, their experimental data did not show any relationship between PL and axial load ratio, longitudinal reinforcement ratio, or longitudinal reinforcement strength. In contrast, Sakai and Sheikh [12] concluded that the amount of transverse reinforcement, axial load level, and aspect ratio had an influence on the PL. The PL generally increased with increasing values of each parameter. Furthermore, Tanaka et al. [13] conducted two series of column tests with the dimensions of (400×400×1800)mm and (550×550×1650)mm, the shear span-to-depth ratios of 3 and 4, and the axial load ratio ranging from 0.1 to 0.3. They observed that when the axial load ratio increased, the equivalent PL increased. Besides, Paulay et al. [14] conducted an experiment and recommended that the result in values of PL was close to ( is the effective depth). Watson et al. [15] tested five square columns (400×400×1600)mm and two octagonal columns under moderate to high ALRs that range from 0.1 to 0.7. They observed that the length of potential plastic hinge regions increased as the axial load ratio increased. The other parameters, such as the aspect ratio and the section type of the columns, were found not to have a significant effect. Bayrak [16] constructed and tested twenty-four square and rectangular concrete column specimens with concrete strength ranged between 72MPa and 112MPa, with cross-sections of 305mm square, rectangular dimensions, and 1841mm in height. The shear span-to-depth ratios were 6, 7.4 and 5.3. The ALR ranged from 0.3 to 0.5. The simpler expression was proposed to estimate the PL such as , where x ranges from 0.9 to 1. Panagiotakos and Fardis [17] used a database of more than 1000 tests to develop expressions for the deformations of RC members at yielding or failure. An empirical expression has been also developed for estimating the PL as illustrated in Table 1. Bae [4] implemented four test specimens with a cross-section of , and a height of 2630mm. The ALR ranged from 0.2 to 0.5. The PL obtained from the analysis of compressive strains are approximate results. Berry et al. [18] conducted thirty-seven spiral reinforced column, and used the data from the tests of large-scale circular bridge columns to evaluate the models for performance-based earthquake engineering requirements for bridge columns, including a new expression for plastic hinge length. Bae et al. [19] observed PL ratio based on experimental observation, where the compressive strain profile was computed by using a developed analytical procedure for each specimen. Ning et al. [20] based on 133- column database, a probabilistic PL model is proposed by considering the effect of axial load ratio, rebar diameter, tension shift effect.

**Table 1.** Some existing plastic hinge length models.

|  |  |  |
| --- | --- | --- |
| Authors | Expression |  |
| Mattock [8] |  | (1) |
| Corley [7] |  | (2) |
| Mander [10] |  | (3) |
| Priestley et al. [21] |  | (4) |
| Paulay et al. [22] |  | (5) |
| Watson et al. [23] | : strength reduction factor; *Ag*: gross area of column; | (6) |
| Panagiotakos et al. [17] | the fixed-end rotation due to slippage; | (7) |
| Lu et al. [24] |  | (8) |
| Berry et al. [25] |  | (9) |
| Bae et al. [4] |  | (10) |
| Ning et al. [20] |  | (11) |

1. EXPERIMENTAL PROGRAM
   1. Specimen design

Four columns named T1-0.5P, T1S-0.5P, T2-0.5P, T2-0.1P that were designed with the cross section of , clear height of 3000mm. The design compressive strength of concrete was 40MPa, and the strength of longitudinal bars and transverse reinforcement were 420MPa. The detail information about the actual strength of used material is presented on Table 2. And, the high axial ratio of 0.5 was applied for T1-0.5P, T1S-0.5P and T2-0.5P specimens. The low design axial ratio of 0.1 was applied for T2-0.1P specimens. The longitudinal bars were used #6 and transverse reinforcement used number 4 with 100mm spacing. The transverse reinforcement was used closed-hooks, cross-ties. Furthermore, to anchor the longitudinal bars to top and bottom beam, using the anchor head SD420. The single cross-ties with one end is 90-degree hook, and 135-degree for the other end. The detail of tested columns was presented on Figure 1. To ensure the columns will be fail on the flexure failure mode, the ratio of nominal shear strength () and the nominal flexure strength () was controlled so that it would be less than 0.6 [26–28]. The nominal shear strength, was calculated according to ACI318-19 code [29] as presented in Eqs. (12-14). And, the nominal flexure strength was determined by the fiber-based cross-sectional analysis via OpenSees platform [30].

**Table 2.** Column design parameters.

|  |  |  |  |  |  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- | --- |
| Specimen | Concrete | Longitudinal reinforcement | | | Transverse Reinforcement | | | | Axial load (kN) | Axial load ratio |  |
| (MPa) | (MPa) | Nos | (%) | Nos | s (mm) | (MPa) | (%) |
| T1-0.5P | 49.7 | 485 | 12#6 | 0.021 | 4 | 100 | 470 | 0.013 | 3978 | 0.5 | 0.27 |
| T1S-0.5P | 49.7 | 485 | 4 | 100 | 470 | 0.013 | 3978 | 0.5 | 0.27 |
| T2-0.5P | 49.7 | 485 | 4 | 100 | 470 | 0.001 | 3978 | 0.5 | 0.32 |
| T2-0.1P | 51.9 | 485 | 4 | 100 | 470 | 0.001 | 830 | 0.1 | 0.27 |

* 1. Test setup and instrumentation

The columns were tested by Multi Axial Test System (MATS) with using double curvature, and lateral cyclic loading under constant axial load in National Center for Research on Earthquake Engineering (NCREE), Taiwan. The layout of MATS was present in Figure 1(a). The MATS have 6-DOF (Degree of Freedom) loading system for advanced seismic testing of structural components. And, it has two sets of vertical and horizontal actuators with capacities of 30MN and 4.5MN, respectively.

The columns were installed at the test system by fixing their top and bottom RC blocks to the steel plates of the MATS through multiple pre-stressed high-strength steel rods. A predetermined axial load was applied to the columns followed by the application of the prescribed horizontal displacement reversals on the foundation of the columns. During the test, the columns would deform in a double curvature configuration due to the boundary constraints applied by the MATS. The applied axial load was controlled to be constant during the entire test. The displacement reversals were applied using a displacement-controlled loading protocol with a targeted displacement history conforming to FEMA461 [31] shown in Figure 1(c). Each level was repeated twice to observe the stiffness and strength degradation.

Specimens were applied the high and low axial compression load with the axial ratio of 0.5 and 0.1, it was presented in Table 2. The top and bottom base was fixed by the strong platen by the high strength rods. Thirty markers were used to record the displacement of column, top and bottom beam. The position of markers was arranged so that there were more markers at near the top and bottom beam, and there was a marker layer on the top and bottom beam to record the displacement and rotation of top and bottom beam. The position of marker was presented in Figure 1(b). To record the strain in longitudinal bars and transverse reinforcement, twenty-four strain gauges were attached on the longitudinal bars, and twenty-one strain gauges were attached on the transverse reinforcement as Figure 1(a). Besides, two LVDTs (linear variable differential transformer) and two incline meters installed at the top and bottom base to measure the horizontal displacement and rotation of bottom and top base, respectively.

  (a) Specimen design and position of strain gauges (b) Position of markers



(c) Loading protocol

**Fig. 1.** Specimen design, strain gauge and marker position, and loading protocol.

1. DISCUSSIONS
   1. Damage pattern of columns

Figure 2 shows the damage patterns of the tested columns. At the early drifts before the peak load, high ALR columns (T1-0.5P, T1S-0.5P, and T2-0.5P) exhibited similar crack patterns. Due to the high compression load, the cracks did not appear at the initial drifts in those columns. The flexure cracks appeared in high ALR columns at the drift of 0.5%, while earlier flexural cracking occurred in the low ALR column at the drift of 0.25%, Furthermore, when the drift of high ALR columns reached 0.75%, vertical splitting cracks appeared due to the high compressive stress. Meanwhile, substantially more visible and horizontally-propagating cracks could be observed in T2-0.1P as compared to the high ALR columns. Some minor shear cracks occurred when the drift exceeded 0.375% for all columns. Similar drifts at the peak loads of 1.0%, 0.75%, and 1.0% for high ALR column T1-0.5P, T1S-0.5P, and T2-0.5P, respectively were obtained. The drift at the peak load was 2.0% for column T2-0.1P that is double the drifts for high ALR columns. Subjecting to increased ALR, the drift at peak load decreased and a more brittle failure of the columns occurred.

As shown in Figure 2, plastic hinge formations at the column ends could be observed in all specimens and the ultimate failure was due to the concrete crushing. The high compression load delayed the yielding of the longitudinal rebar and increased the yield moment. As a result, the high ALR columns had longer plastic hinges than that of the low ALR column. Furthermore, the longitudinal rebar buckled for all columns, except for T1-0.5P. This demonstrated that the closed hooks used in T1-0.5P can effectively enhance the confinement of the concrete core and restraint the longitudinal rebar buckling. The 90-degree hooks in T1S-0.5P and T2-0.5P series were opened, while the 90-degree hooks in T2-0.1P and all 135-degree hooks generally remained intact in all columns. No damage or fracture was observed for the transverse reinforcement.

From the cyclic load-displacement responses of the columns presented in Figure 2, it can be seen that all columns have reached their flexural capacity before failure. The lateral strength begins to drop at a drift demand of 1.0%, 0.75%, 1.0%, and 2.0% for T1-0.5P, T1S-0.5P, T2-0.5P and T2-0.1P, respectively. According to the observed failure patterns and load-displacement responses, all columns successfully displayed the expected flexural failure patterns. The drift capacity was determined at the point that the lateral strength decrease down to 85% of maximum lateral strength. The ALR effect on the drift capacity of the column was well observed by comparing T2-0.5P and T2-0.1P. The drift capacity decreased considerably from 5.0% to 1.5% when the ALR increased from 10% to 50%.

|  |  |
| --- | --- |
|  |  |
| a. T1-0.5P | b. T1S-0.5P |
|  |  |
| c. T2-0.5P | d. T2-0.1P |

**Fig. 2.** Damage pattern of columns.

* 1. The components of plastic hinge length

### Moment gradient

When the column reaches the plastic moment (), the column begins to exhibit plasticity response. It will lead to the information of plastic hinge, and causes the spread of plasticity along the column. Assuming that the moment diagram is linear, the PL due to moment gradient () is determined by Eq. (12) [2]. Where the plastic moment () is determined by the moment at the onset of first yield in longitudinal reinforcement; the ultimate moment is taken as the maximum moment; the shear span is gotten by a half of column height with double curvature.

|  |  |
| --- | --- |
|  | (12) |

### Tension shift

The tension shift effect is known as the effect of diagonal shear cracks (due to shear force) in the plastic hinge region. The plastic region would be extent under the shear effect. Although it was quite complicated to determine this component, few researchers investigated it through experiment and statistic such as, Ning et al. [20], Park et al. [32] etc. Park et al. [32] proposed a simple equation expressed as (where is the inclination of the diagonal cracks; φ is the inclination of the transverse reinforcement; and is the ratio of the shear strength to the shear demand). While Ning et al. [20] proposed to get by ( is the cross section height of column) through the experimental databases. Therefore, the PL which is directly relevant to the flexural deformation composed of two parts as presented by Eq. (13).

|  |  |
| --- | --- |
|  | (13) |

### Strain penetration

The strain penetration represents the gradual transferring of the forces in longitudinal reinforcement to the surrounding concrete material at the column supports. The result is the longitudinal reinforcement slips along a partial anchoring length. It would make increasing the plastic rotations of column ends. According to Megalooikonomou et al. [2] the PL due to the strain penetration effect () will depend on slip rotation due to strain penetration effect , yield and ultimate curvature, as illustrated by Eq. (14). While the yield curvature was determined by the curvature at the first yield point of longitudinal reinforcement, the ultimate curvature was taken as the curvature at the onset that lateral strength decreases reaching 85% of maximum lateral strength.

|  |  |
| --- | --- |
|  | (14) |

Therefore, the plastic hinge length can be expressed as Eq. (15).

|  |  |
| --- | --- |
|  | (15) |

### Evaluation of plastic hinge length

The PL could be approached by two methods, i.e., the indirect and direct methods. In the indirect method, the PL was calculated by the relation between displacement and curvature. In contrast, the direct method is defined by the physical observation or the measured curvature profiles along the height of columns in the direct method. Both methods enable to consider the effect of bar slip, shear, and flexural behavior.

***Indirect method***

In the indirect method, the PL would be determined by empirical equations. For instance, Park et al. [9] proposed Eq. (16) to calculate the yield displacement. Assuming that the plastic is concentrated at the center of the plastic hinge, and decomposing the total displacement, , into two components and as illustrated in Eqs. (17-18). From Eqs. (16-18), the PL can be established.

|  |  |
| --- | --- |
|  | (16) |
|  | (17) |
|  | (18) |

Where are yield and maximum curvature at column end; is the length from base to contra-flexure point (it is taken a half of column height for double curvature testing).

***Direct method***

The direct methods are the physical observation and measured curvature profile methods. Depend on the definition of plastic hinge length, it can classify three different regions, such as the rebar yielding zone (compression and tension), concrete crushing zone (cover or core), or curvature localization zone.

### The plastic hinge length based on yielding strain of rebar

The PL based on yielding strain of rebar (or strain profiles) is the region where the strain of rebar reaches or exceeds its yield strain. It can be classified by the tensile yield strain of rebar (), and the compressive yield strain of rebar (). The PLs based on the strain profiles were the lengths that were calculated from the critical sections (always located at the base-column interface) to the intersection point of the strain profiles and the vertical line having a value of tensile yield or compressive yield strain. In this experimental, the yield strain of longitudinal reinforcement is equal to 0.0023. However the strain gauges could not capture the bar slip effect, it only measured the PL due to moment gradient , and tension shift . Therefore, the total PL () should be added the PL due to strain penetration . The measured PL due to slip penetration was calculated by Eq. (14), and presented in Table 3. The PLs based on tensile and compressive strain of rebar were presented in Figure 3.

**Table 3.** PL due to strain penetration

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Parameter | T1-0.5P | T1S-0.5P | T2-0.5P | T2-0.1P |
|  |  |  |  |  |
|  |  |  |  |  |
|  |  |  |  |  |
|  | 120 | 107 | 93 | 107 |

### Method using peak strain of concrete

Similarly, the PL based on the peak strain of concrete was determined by the length from the critical sections to the intersection point of peak concrete strain. The concrete cover and core begin crushing and spalling when they reach the peak strain of unconfinement and confinement respectively. The PLs due to concrete cover and core were determined through the peak strain of confined and unconfined concrete. In this study, the peak strain of unconfined concrete was taken as . And, the peak strain of confined concrete is determined by Mander’s model [33] as illustrated in Eq. (19) (where are the peak strain and stress of confined concrete, respectively). Table 4 presents the peak strain of unconfined and confined concrete of all tested columns. The PL based on peak strain of concrete is presented in Figure 3 and Table 5.

|  |  |
| --- | --- |
|  | (19) |

**Table 4.** Peak strain of concrete

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Strain of concrete | T1-0.5P | T1S-0.5P | T2-0.5P | T2-0.1P |
|  | 0.0020 | 0.0020 | 0.0020 | 0.0020 |
|  | 0.0078 | 0.0078 | 0.0053 | 0.0053 |

### Method using measured curvature profiles

The PL based on curvature profiles () is the region length that had a curvature be larger than the yield curvature. The curvature was calculated by using Eq. (12). The yield curvature is determined by the average curvature of two column ends at the onset of first yield of the longitudinal reinforcement. The PL based on curvature profiles is established in Figure 4 with two critical curvatures presented.

### Method using observed damage of concrete

The PL based on physical observation method () is the region that has suffered the following damage, such as (1) spalling and crushing of concrete cover; (2) penetration of spalling from concrete cover into concrete core region, (3) local buckling or yielding of longitudinal rebars; (4) yielding of transverse reinforcement. The PLs of tested columns based on observing concrete damage were presented in Figure 5 and Table 5.

### Discussions on PLs

All PLs based on different criteria were calculated and presented in Table 5. It can be seen that there is a large discrepancy between them. The maximum difference between the observed PL and counterparts is 135% for tested columns. Therefore, it is necessary to separate the PLs based on the different criteria so that they could be satisfied the design requirements.

While the PL based on cover crushing had the greatest value to compare with counterparts, the PL based on observing gave the smallest value due to the strictly condition for crushing and spalling concrete cover and core. Similarly, the PL based on compressive strain was greater than that of tensile strain due to the effect of high axial compression load. The combination of compressive stress due to bending and axial compression load caused the greater compressive stress in longitudinal steel. Besides, the PL based on curvature profiles matched well with the PL based on observing cover damage with the average ratio of being equal to 1.07.

The tested results implied that the PLs could be enhanced by high ALR. Increasing ALR from 0.1 to 0.5, the PLs increased with the maximum value of 41%. Furthermore, it is worth noting that the PLs based on tensile strain were nearly the same for all tested columns regardless the effect of ALR.

In addition, the higher confinement led to the longer PL. However, this effect could be minor. Increasing 25% for amount of transverse reinforcement, the PLs only increases with the maximum value of 10%.

**Table 5.** The plastic hinge length

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| PL (mm) | T1-0.5P | T1S-0.5P | T2-0.5P | T2-0.1P |
|  | 775 (1.94h) | 768 (1.92h) | 737 (1.84h) | 594 (1.49h) |
|  | 1160 (2.90h) | 1059 (2.65h) | 1124 (2.81h) | 882 (2.21h) |
|  | 1056 (2.64h) | 986 (2.47h) | 966 (2.42h) | 806 (2.02h) |
|  | 765 (1.91h) | 890 (2.23h) | 837 (2.09h) | 860 (2.15h) |
|  | 576 (1.44h) | 579 (1.45h) | 552 (1.38h) | 342 (0.86h) |
|  | 400 (1.00h) | 400 (1.00h) | 380 (0.95h) | 320 (0.8h) |
|  | 550 (1.38h) | 550 (1.38h) | 590 (1.48h) | 450 (1.13h) |

*Note: the PL is taken the average of top and bottom measured PL*

|  |  |
| --- | --- |
| a. T1-0.5P | b. T1S-0.5P |
| c. T2-0.5P | d. T2-0.1P |

*Note: LS1, LS2 are the strain gauge names as presented in Figure 1a.*

**Fig. 3.** Plastic hinge length calculation

|  |  |
| --- | --- |
| a. T1-0.5P | b. T1S-0.5P |
| c. T2-0.5P | d. T2-0.1P |

**Fig. 4.** Curvature profiles

|  |  |  |  |
| --- | --- | --- | --- |
| a. T1-0.5P | b. T1S-0.5P | c. T2-0.5P | d. T2-0.1P |

**Fig. 5**. Measured plastic hinge lengths

1. Conclusions

Based on tested results, the length of plastic hinge was observed through some criteria such as strain profiles of rebar, peak strain of concrete core and cover, curvature profiles, and damage observation. The main conclusions are summarized below:

1. The failure patterns of tested columns were significantly affected by the ALR. Increasing ALR led to more brittle failure and make longer PLs, but decrease the drift capacity.
2. Due to the large discrepancy, the separation of PLs based on concrete strain, rebar strain, curvature profiles, and damage observation, are needed to exactly estimate the PL so that it would be satisfied the different design requirements.
3. The PLs are significantly affected by axial load ratio except for PL based on tensile strain of rebar. While the effect of ALR on the PL based on tensile strain of rebar is minor, the counterparts increase about when increasing ALR from 0.1 to 0.5.
4. The confinement had an insignificant effect on the length of plastic hinge. Increasing 25% amount of transverse reinforcement, the PLs only increased with maximum value of 10%.
5. The discrepancy between damage observation method and counterparts of tested columns ranges from 25% to 135%.

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