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Plastic Hinge Length Requirements in Reinforced Concrete Couple Shear Wall Buildings for Seismic Reinforcement Detailing

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Abstract

Proper reinforcement detailing in plastic hinge regions is one of the important measures that could help damage control of structural walls subjected to any severe earthquake event. Inelastic curvatures are commonly assumed to be uniform over a height called plastic hinge length. Non-linear dynamic analyses are performed on a set of coupled shear wall buildings of simple configurations for different heights. Inelastic curvatures are calculated on numerous heights of all the buildings and plotted along with the height of the buildings. Plastic hinge lengths are estimated with the yield curvatures from analytical results. It becomes a common practice to estimate the plastic hinge length equal to 0.5 to 1.0 times the wall length, which basically were developed from experimental studies on beam and column elements. As per the Canadian standards CSA A23.3-04, the requirements to calculate plastic hinge lengths are identical for both cantilever and coupled shear walls, i.e., 1.5 times the wall length in the direction under consideration. Results from the present study show that inelastic curvatures are not uniform over the plastic hinge length and the Canadian requirement as per CSA A23.3-04 to calculate plastic hinge length is unconservative for couple shear walls and more critical for slender coupled shear walls. And the plastic hinge length calculation as per Canadian code CSA A23.3-14 (clause 21.5.2.1.2) is over conservative for coupled shear wall buildings and much more over conservative for slender coupled shear walls. A comparison studies with different researchers and building codes are made. A new multiplication factor is proposed for the safe estimation of plastic hinge length for couple shear walls of medium and high rise reinforced concrete buildings. Results indicate that it needs to consider 2.0 times wall length instead of 1.5 times wall length in the direction under consideration for the safe estimation of coupled shear wall plastic hinge length calculations.

Keywords: Reinforced concrete • Buildings • Couple shear wall • Non-linear dynamic analysis • Plastic hinge • Seismic design • Inelastic curvatures • Damage control

Introduction

In seismic design of concrete shear wall, the design philosophy is to ensure the flexural displacement capacity is greater than flexural displacement demand. The inelastic portion of the flexural displacement demand developed due to the concentration of the inelastic curvatures at the bottom potion of the cantilever wall. For simplicity, the inelastic curvatures are usually assumed to be uniform over a height and this height is called the plastic hinge length of the wall. It has been found by different researcher through experimental and analytical results that at the bottom portion of the walls, the inelastic curvatures are not uniform and vary linearly. The coupled shear wall system is one of the effective potential options in comparison with other Moment Resisting Frame (MRF) and shear wall combination system in earthquake resisting building design. Moment resisting frame and shear wall combination systems are governed by both shear and flexural behaviour, whereas the couple shear wall combination systems is usually governed by flexural behaviour. And the behaviour of conventional beam in MRF and shear wall combination systems is governed by flexural capacity and the behaviour of coupling beams in couple shear wall system is usually controlled by shear capacity. The energy dissipation happens through both inelastic yielding in beams and columns for MRF and shear wall frame combination systems; whereas energy dissipation happens through inelastic yielding in coupling beams and at the base of the wall in coupled shear wall systems during the earthquake. Hence, the amount of earthquake energy dissipation and ductility obtained from Moment Resisting Frame (MRF) and shear wall frame systems is less than that of coupled shear wall combination systems. A common practice is to assume that the plastic hinge length of a cantilever shear wall varies from 0.5 to 1.0 times the larger horizontal dimension of wall length. But this was established basically on beam and column test results [1,2].

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Research significant

Concrete shear wall is one of the vertical elements of seismic force resisting system and has been become a common practice to provide lateral strength and stiffness in mid-rise and high-rise buildings. Couple shear wall is one of the categories of concrete shear wall. These walls should have enough shear strength and flexural displacement capacity in order to achieve adequate seismic behaviour. And the flexural displacement capacity depends on different factors like the compression strain capacity of concrete, the neutral axis length and plastic hinge length. It becomes a common practice to assume the inelastic curvatures at the base of cantilever wall constant throughout a height but practically this plastic hinge length is not uniform throughout the height. Analytical studies done on plastic hinge length by different researcher and found that it depends on wall length, height, axial force etc. Different researchers worked on and developed empirical equations to estimate plastic hinge length.

Most of these models are calibrated to find out the real total displacement and real total rotation at failure. Not too many experimental data are available in this regard. Different researchers in the past studied on the influence of member dimensions (length, height), longitudinal reinforcement properties, effect of axial loads (tension and compression), strain hardening etc. Among these studies, very a smaller number of studies were done on plastic hinge length prediction of couple shear wall. Another important observation that most of the previous worked done on individual wall member, very less amount of work done on the effect of concrete wall systems on plastic hinge length and it is still not very clear the parameters that effect mostly for accurate prediction of plastic hinge length calculations in concrete shear wall buildings including couple shear wall buildings. This study concentrates only finding parameters those effects on couple shear walls only. In couple shear wall buildings, a system of couple shear walls are connected together by coupling beams. And the systems of high degree of coupling are subjected to lateral loading, the shear force in the coupling beams induce high degree of axial force in the walls. One wall will be subjected to tension and other will be subjected to compression. Geometry of the coupling beam, span, and the axial forces (tension and compression) are expected to have significant effect on plastic hinge prediction.

Objectives

The objectives of this study to investigate parameters that affect the plastic hinge length of couple shear walls, using non-linear finite element analysis. The analysis was performed using computer software SeismoStruct. It was modelled 08 storey, 12 storey, and 16 storey buildings of similar geometry with different heights. Dynamic time history analysis will be performed with Seismostruct. And it will be investigating the influencing parameters that effect on plastic hinge length calculation of couple shear wall. Mostly it will be investigated the influence of geometry of the wall, aspect ratio, geometry of coupling beams, depth span ratio of coupling beams, and the axial force.

Chan: Chan performed some experimental test on concrete column to find out the spread of plasticity and hence provided an equation. He also looked at how the lateral confinement affect in the strain capacity of Concrete. He considered the effect of strain hardening to define the plastic hinge length. And he considered linear relationship to formulate the length where the yield moment is exceeded. L_p/L_s = 1–My/Mu; Where, L_s or z= Distance from the maximum moment location to the zero-moment location. M_y= Yield moment and Mu= Ultimate moment; Chan also looked that with the changing of the steel ratio did not have that much affect on the plastic hinge length Lp and he considered an average value of 0.4z for the estimation of plastic hinge length, where z= shear span.

Baker: Baker investigated the plastic deformation of concrete members. He provided an estimate far safe calculating plastic hinge length of concrete members that is between 0.5h to h; [3].

Where

h= Length of the concrete wall.

Cohn and petcu: Cohn and Petcu investigated the factors affecting rotational capacity of plastic hinges of continuous concrete beams. He basically studied the effect of percentage of steel on plastic hinge calculations. He used the test results in order to calculate the bending moments at yielding and failure. He also considered linear variation of the bending moment diagram like Chan; and he also used the same equation that Chan utilized to calculate the plastic hinge length [4].

I.C.E. formula: I.C.E. committee proposed the following equation for calculation of plastic hinge length considering constant curvature over that length. $L_P = k_1k_2k_3\ (z/d)^{1/4}d$, $k_2 = 1+0.5(P/P_u)$, $k_3 = 0.9 - [(f_{cc}-13.8)/92]$, where k_1: Factor considered for the influence of tension rebar. It is 0.7 for mild steel and 0.9 for cold-formed steel. k_2: Factor considered for the influence of axial load. k_3: Factor considered for the influence of concrete strength. d: Effective depth of concrete member. P: Axial load. P_u: Axial compressive strength of concrete member with any eccentric loadings. f_{cc}:Concrete cube strength in MPa. And z: distance of the critical section to the point of contraflexure.

Baker and Amarakone: Baker and Amarakone developed an equation to predict the inelastic rotations. $\Theta_p = 0.8(\epsilon_{cu} - \epsilon_{ce})\ k_1k_3(L_{s/}d)$.

Where, ϵ_{cu} = Concrete strain fiber at ultimate curvature in extreme compression. ϵ_{ce} = Concrete strain at yield curvature in extreme compression fiber.

Sawyer: Sawyer developed a methodology to calculate the inelastic rotation for the reinforced concrete frame members. He considered bi-linear moment curvature relationship. He made some assumptions to do that. He assumes that the maximum moment at any section is equal to ultimate moment. Based on some experimental test he considers the ratio of yield moment and ultimate moment ($M_{y/}M_u$) is equal to 0.85. And the third assumption is that the plasticity spreads d/4 past the section, the bending moment in that section is equal to yield moment. He developed following expression to calculate the inelastic rotation. $\Theta_p = ½(0.15L_s)\ (\Phi_u–\Phi_y) = 0.075L_s\Phi_p$ but inelastic rotation can be developed by integrating inelastic curvatures, $\Theta_p=\Phi_pL_p$; Equating these two equations, he developed the following expressions, $L_p = 0.075L_s$; considering the third assumption, he developed: $L_p= 0.25d+0.075L_s$; In generalized form Sawer's equation can be written $L_p = \alpha d+\beta z$ [5].

Mattock: Mattock performed experimental studies on reinforced concrete beams, considering simply supported span loaded at the midspan to investigate the rotational capacity. He investigated the influence of following factors: concrete strength, effective depth, the distance from the section of maximum moment to the section of zero moment, the yield stress of reinforcement and the amount of reinforcement. He made a report for 37 beams. He used the test results and observed the spread of plasticity on each side of the beam. Mattock observed that inelastic deformation occurred beyond the half of the effective depth of the beams. And it depends on the distance from the section of maximum moment to the section of zero moment called shear span, the effective depth and amount of flexural reinforcement. He used the ratio of total inelastic rotation considering length L_s, Θ_p, to the inelastic rotation considering length d/2, $\Theta_{p,\ d/2}$, as a measurement of spread of plasticity. He calculated the total inelastic rotation for each beam from the plastic deformation at the midspan. He obtained the inelastic rotation at the length d/2 from the other measurement taken at the midspan. Mattock developed the following equation: $\Theta_{p/}\Theta_p$, d/2 = 1+(1.14√L_s/d–1) [1–{(ω–ω')/ω_b } √d/0.411], Where $\omega_b = \rho_b\ \{f_y/f_c'\}$

L_s= Distance in meter from the section of maximum moment to the section of zero moment (meter).

d= Effective depth of the concrete beam (meter).

ω_b= Tension reinforcement index considering balanced ultimate strength condition.

ρ_b= Reinforcement ratio without compression reinforcement considering balanced ultimate strength condition [6].

Mattock also observed that the length of the plastic hinge is proportional to the difference between the yield moment and ultimate moment and this difference strongly depends on the strain hardening in the tension reinforcement. And he observed the similar findings from his test results. He calculated the inelastic rotation at d/2 distance using compatibility and equilibrium equations. He proposed the following equation to estimate the plastic hinge length: $\Theta_{p,\ d/2}d/2 = \Theta_{p/}L_p$; with superimposing this equation to the above equation, Mattock proposed following equation to calculate plastic hinge length, L_p = d/2{1+(1.14*(√($L_{s/}$d)-1)*[1-(ω-ω')/ω_b)*√(d/0.411)], and finally, Mattock proposed a simplified form of this equation, L_p =0.5d+0.05z.

Corley: Corley performed experimental tests on reinforced concrete simply supported beams. He also considered concentrated load at midspan like Mattock. He investigated the effect of the confinement reinforcement in compression and also the effect of member size in their rotational capacity. He extended the work done by Mattock and Corley reported the results that obtained from 40 beam tests. He used the experimental tests results to determine the spread of plasticity of the beam at each side of the midspan. He also used the ratio of $\Theta_{p/}\Theta_p$ at midspan as a measurement of plasticity similar to the work performed by Mattock. Corley plotted the graph obtained from $\Theta_{p/}\Theta_p$ (at midspan) as a function of spread of plasticity and found the reinforcement did not have significant effect on the spread of plasticity. Corley observed that the least square fit was not appropriate because of significant scattering of the variables. He proposed following equation for the spreading of plasticity. $\Theta_{p/}\Theta_{p}$, d/2 = 1+(0.064/√d) ($L_{s/}$d), Where L_s and d are in meter. He compared the inelastic rotations obtained from the above equations and compared with the tests results found by Mattock and Corley finally proposed the following equation to calculate the plastic hinge length.

$L_p = 0.5d+0.032(L_{s/}\sqrt{d})$ [7].

ACI-ASCE committee 428: The ACI-ASCE Committee 428 Committee proposed following equation for the plastic hinge calculations on their progress report on code clauses. It showed the lower and upper bound limit for plastic hinge calculation on the following equation, ACI-ASCE Committee 428; Min $[R_\epsilon\ (d/4 +0.03L_sR_m),\ R_\epsilon d]<L_p<R_\epsilon\ (d/2 +0.10L_sR_m)$

Where, $R_\epsilon = (0.004-\epsilon_{ce})/(\epsilon cu - \epsilon_{ce})$, $R_m = (M_{max-}M_y)/(M_u–M_y)$

It was reported also the following equation to calculate the distance from the section of maximum moment to the section of zero moment subjected to uniform distributed load consideration.

$L_s = 4M_{max/}[4V_z+\sqrt{(w_zM_{max}R_m)}]$

Where,

V_z= The reaction at the section of maximum moment.

W_z= The uniformly distributed load at the section of maximum moment.

ASCE Committee 428 recommended α = 0.5 and β = 0.1 for the upper bound estimate of L_p.

Paulay and Uzumeri: Paulay and Uzumeri are the first researchers, who modified Sawyer's equation for applying in concrete shear walls by assuming d = $0.8l_w+\beta h_w$; Where lw is the wall length and hw is the wall height [8].

$L_p =\alpha*0.8l_w+\beta h_w$.

Paulay: Paulay developed the design procedures for the ductile reinforced concrete walls for the seismic loadings. He observed and proposed that the plastic hinge length of ductile reinforced concrete walls was primarily the function of wall lengths. He proposed that the plastic hinge length lies between $0.5L_w$ to L_w.

Priestly and Park: Priestly and Park performed experimental tests on reinforced concrete bridge column with different cross-section subjected to combined axial and bending effect in order to study their strength and ductility. They investigated the influence of following parameters on seismic behaviour of reinforced concrete bridge columns: the axial load, the yield strength of transverse reinforcement and their quantity and the aspect ratio. The experimental tests were done on different shapes, like square, octagon with circular reinforcement and Hollow Square. They performed two different tests, one for squat column with aspect ratio 2 and the other one was for slender column with aspect ratio 04; they consider square and octagonal section as squat columns. They followed the test procedure similar to the test procedure was used by Park and Potangaroa. These experimental studies were done to develop an analytical equation that can be used to calculate the plastic hinge length. And finally, they proposed the following expression to calculate the plastic hinge length.

$L_p = 0.08L_s+6d_b$

Where,

L_s = Distance in meter from the section of maximum moment to the section of zero moment.

d_b = Bar diameter of the tension reinforcements.

Paulay and Priestly: Paulay and Priestly proposed following equation to calculate the plastic hinge length for beams and columns, $L_p = 0.08L_s+0.022f_yd_b \geq 0.044f_yd_b$

Where, f_y = Yield strength in MPa unit. L_s = Distance in meter from the section of maximum moment to the section of zero moment. d_b = Bar diameter of the tension reinforcements. Paulay and Priestly also commented that the above equation gives approximately 0.5h values equivalent to the plastic hinge length for common dimensions of beams and columns.

Wallace and Moehle: Wallace and Moehle developed an analytical procedure to determine the confinement necessity of reinforced concrete walls subjected to seismic loadings. They also found that plastic hinge length lies between $0.5l_w$ to l_w.

Where, 1_w =length of the wall [9].

Paulay and Priestly: Paulay and Priestly performed experimental tests on ductile reinforced concrete walls to investigate the out of plane buckling and also the plastic hinge rotations subjected to seismic loadings. They considered rectangular shapes. They proposed the following equation to calculate plastic hinge length, Paulay and Priestly by considering α = 0.25 and β = 0.044 in Sawyer's equation.

$L_p= 0.25l_w+0.044h_w$; Where, h_w= Total height of the wall. L_w= Length of the wall.

They commented that the above equation gives conservative estimate of plastic hinge length. They also commented that the above equation gives a good approximation of the overall length along which the out of plane buckling occurs [10].

Mandis: Mandis performed experimental tests on simply supported beams subjected to concentrated load at midspan similar worked done by Mattock and Corley. He investigated the influence of the following parameters on plastic hinge length of reinforced concrete members: tension, compression, axial force, shear forces and transverse reinforcements. He investigated results obtained from 13 beams. His experimental results were compared with the values calculated from the formulas from different researchers. Mandis observed and concluded that the upper and lower values suggested by ACI committee 428 gives reliable estimates of plastic hinge length for both normal and higher strength concrete up to 80 MPa.

Bohl and Adebar: Bohl and Adebar investigated the profile of inelastic curvatures that should be used to estimate the flexural displacement capacity of high-rise concrete walls by using the methodology to use a non-linear finite element modeling concept. They used non-linear finite element program Vector 2 for modelling purpose and it uses the material model for cracked reinforced concrete subjected to shear combined with axial and bending moment. They also modelled the interconnecting walls that connects with different floor levels using truss bar elements with very high axial stiffness and strength. Bohl and Adebar made comparison with the wall test results to validate the analytical model. Bohl and Adebar found a very good agreement between the predicted and observed curvature distributions, both of which indicate that inelastic curvatures vary approximately linearly over approximately 2 m. Finally, they proposed following expression to calculate the maximum curvatures in the systems of walls with different plastic hinge length. Φ_{max}, 2 =Φ_y,1+(Φ_{max}, 1–Φ_y, 2)/(L_p,1/L_p,2), where Φ_{max} is the maximum curvature at the base of the wall, Φ_y is the yield curvature, and subscripts 1 and 2 refer to the longer and shorter walls respectively [11].

ACI 318: According to ACI 318-19 the reinforcement in the plastic hinge region for structural walls shall extend vertically above and below the critical section at least the greater of L_w and $M_u/3V_u$ (ACI 318-19, Clause # 18.10.2.4). Where L_w is the length of the wall (horizontal length).

Modeling in seismostruct: Geometric non-linearities are considered in seismostruct modelling. Large displacement/ rotations and large independent related to frame element chord (p-delta effect) are considered in the modelling through a total co-rotational formulation, which was developed by Correia and Virtuoso. Material non-linearity also considered in Seismostruct modelling. In seismostruct, fiber approach modelling is used to represent the cross-sectional behaviour, where each fiber is modelled with uniaxial stress-strain relationship. Then, the sectional stress-strain state of beam column element is calculated through the integration of the non-linear uniaxial stress strain response of the individual fiber in which the section is subdivided, i.e., the discretization of the reinforced concrete section. In seismostruct modelling, it has been considered the distributed inelasticity instead of lumped inelasticity. In this research modelling recent developed Forced Based (FB) elements are considered for distributed inelastic finite element modelling. In terms of material inelasticity, it does have one advantage over Displacement Based (DB) elements model. With a displacement-based model refined discretization (meshing) are required, on the other hand, in Forced Based (FB) model it does not require. FB formulation does not depend on sectional constitutive behaviour; hence the solution is always exact.

Materials and Methods

A set of buildings with a simple configurations and different heights are modelled in SeismoStruct. The seismic design parameters are considered for Vancouver location. It has been considered real seismic ground motions those are taken from PEER (Pacific Earthquake Research Institute). The selection of ground motion considers the ratio of peak acceleration (A) to peak velocity (V) close to 1, which represents the seismicity of Vancouver. Three reinforced concrete buildings with couple shear walls as a seismic force resisting systems of 08, 12, and 16 stories heights and 3 bays by 3 bays are modelled and performed analysis with SeismoStruct. The buildings are of heights equal to 30.4, 45.0, and 59.6 m respectively covering limits of the applicability of the ESL methods as permitted by (NBCC) National Buildings Construction Corporation. The design live load is equal to 2.4 kN/m^2 for all floors except the first storey which is 4.8 kN/m^2. The snow load is calculated to be 2.3 kN/m^2.

The dead load is 0.85 kPa exterior walls, 1.0 kPa for partition on floors, 0.5 kPa for ceiling and mechanical services for all floors and 0.5 kPa for roofing. The walls are considered ductile partially couple shear walls (NBCC) National Buildings Construction Corporation with R_d = 3.5 and R_o = 1.6 for the lateral force resisting systems, where R_o and R_d are over strength and ductility factors respectively. Those buildings (08, 12, and 16 storey) are modelled in SeismoStruct having a plan dimension 21 m × 21 m. The thickness of the couple shear wall are 400 mm, 650 mm, and 650 mm for 08, 12, and 16 storeys respectively. The wall thickness, the dimension of coupling beams is considered as per the minimum requirements of CSA A23.3-04. The longitudinal, transverse reinforcements in main couple shear walls and the diagonal reinforcements in coupling beams are also provided as per the code (CSA-A23.3-04) requirements.

In Seismostruct the concrete is modelled with Mander non-linear concrete model and reinforcements are modelled with Menegotto Pinto steel model (SeismoStruct). Dynamic time History analyses are performed with different wall length (L_w) 2 m, 2.5 m, and 3 m and design parameters for those buildings 08, 12, and 16 stories height. Inelastic curvatures variation along the height of the buildings are plotted in below Figures 1-8 and plastic hinge lengths (L_p) are calculated, those are shown in Tables 1-7 for 08, 12, and 16 stories buildings. Total 79 numbers of couple shear walls are analyzed with different aspect ratio, wall length, compressive loads, and coupling beam (d/L) ratios, those data are provided in Tables 2-7.

Yield curvature: Priestley and later Priestley provided a formula to calculate yield displacements for circular bridge columns considering into account shear deformation and strain penetration of longitudinal reinforcements into the foundation. They expressed effective yield curvature in terms of yield strain of the longitudinal reinforcement and diameter of the gross section.

$\Phi_y = 2.25^* \epsilon_{ys}/D$

In this research some expressions are used those are developed based on moment curvature analysis of a large number of column sections. (M. Neaz Sheikh). The estimated yield curvatures are used to calculate the plastic hinge length of couple shear walls of different buildings. In this procedure the yield displacements are calculated using simple expression that accounts flexural deformation of the columns. Those expressions are developed on the basis of displacement-based procedure on the yield curvature at the critical section. In displacement- based procedure, the ductility demands are calculated and that will be used to calculate the effective damping of the structure. After having the effective damping level, the inelastic displacement demand and effective natural period of the structures are calculated based on elastic displacement response spectrum methods E. Miranda. Later on, Montes and Aschleim proposed another equation to calculate effective yield curvature based on moment curvature analysis in terms of yield strain and diameter of the gross section similar to Priestley [12-19].

$\Phi_y = 2.4 * \epsilon_{ys} // D$ for f_y =400 MPa

$\Phi_y = 2.3 * \epsilon_{ys/} D$ for f_y=500 MPa

In this research yield curvatures are also calculated based on Priestley, Montes and Aschleim and Eurocode ($\phi_y = f_{y/}$[Es (1 - ϵ_y) d]) considering steel yielding, where f_y= yield strength, E_s = Modulus of Elasticity, ϵ_y = yield strain and d=depth of the section. It has been performed also the push over analysis (SeismoStruct) to calculate the yield curvature. The calculated yield curvatures are populated in below Table 1 and are used to calculate the plastic hinge length (L_p) from the analysis results as shown in below (Figures 1-8).

Calculation of plastic hinge length (L_p): The elastic and inelastic curvatures are calculated through the dynamic analysis using SeismoStruct 2020. The yield curvatures are calculated through push over analysis and some other methods as well. Then it has been plotted the elastic and inelastic curvatures of the compression and tension walls along the height of the

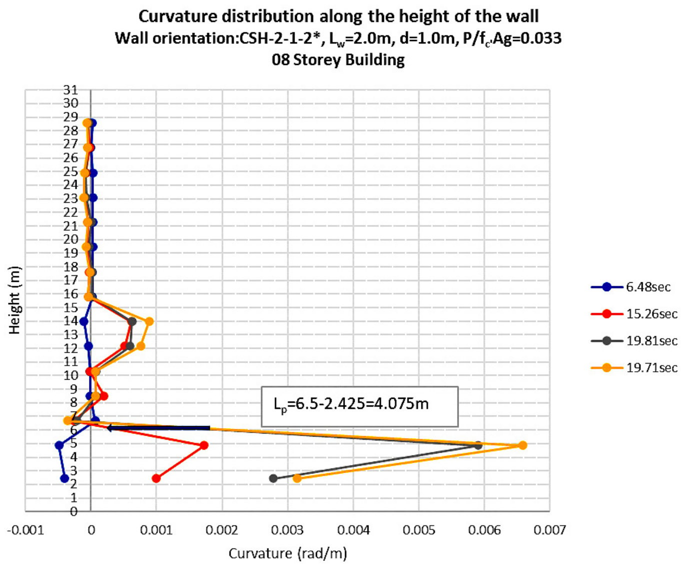

Figure 1. Curvature distribution along the height of the wall, CSH-2-1-2*, L_w= 2.0 m, d= 1.0 m, P/fc'Ag= 0.033, 08 storey.

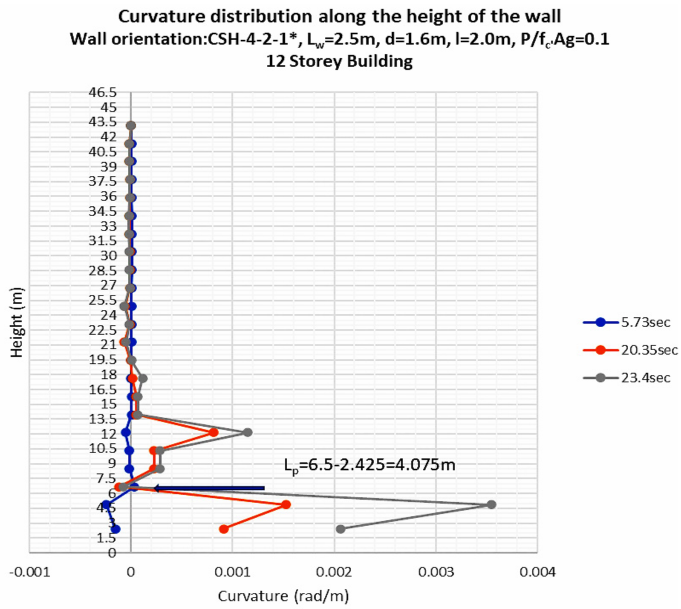

Figure 2. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 2.5 m, d= 1.6 m, P/fc'Ag= 0.1, 12 storey.

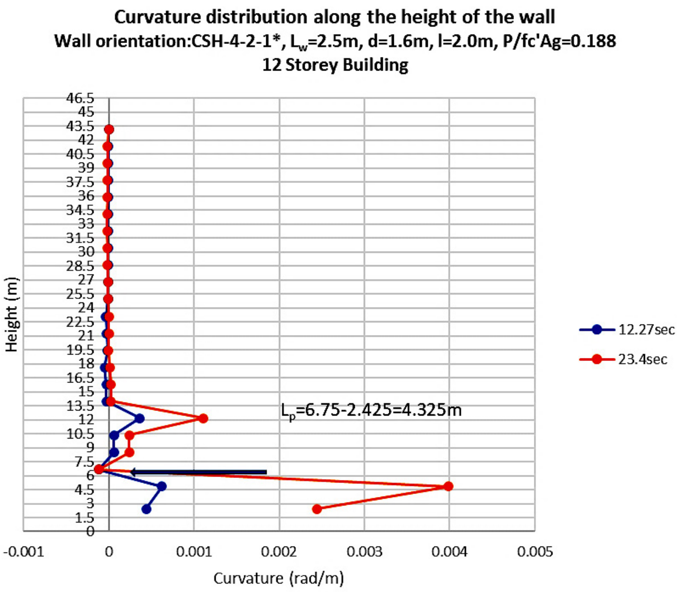

Figure 3. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 2.5 m, d= 1.6 m, P/fc'Ag= 0.188, 12 storey.

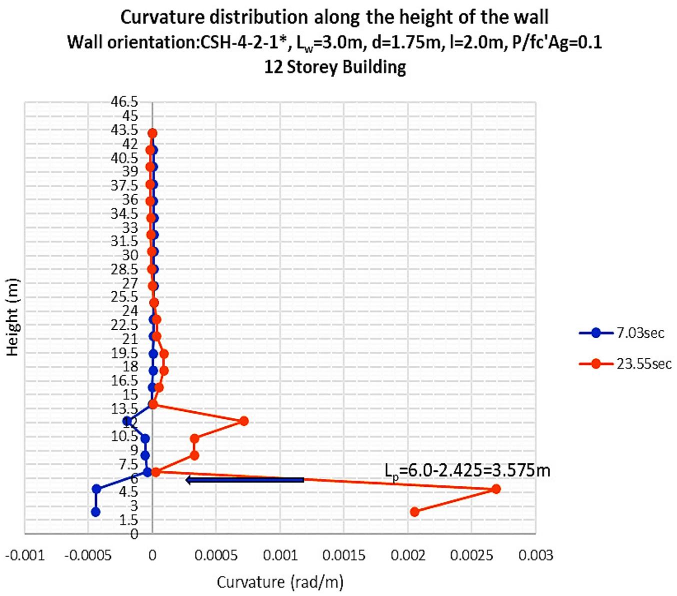

Figure 4. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 3.0 m, d= 1.75 m, P/fc'Ag= 0.1, 12 storey.

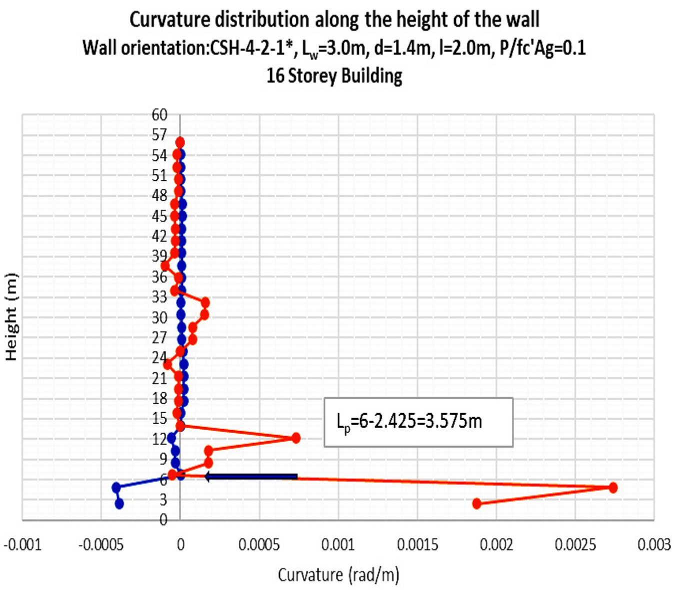

Figure 5. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 3.0 m, d= 1.4 m, P/fc'Ag= 0.1, 16 storey.

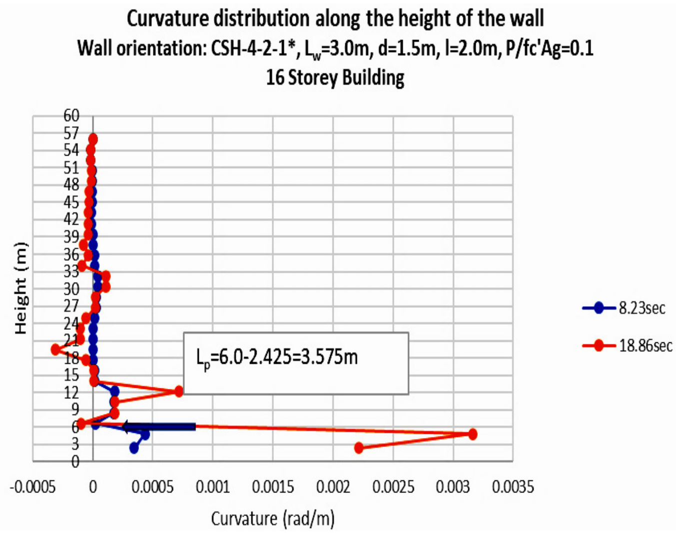

Figure 6. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 3.0 m, d= 1.5 m, P/fc'Ag= 0.1, 16 storey.

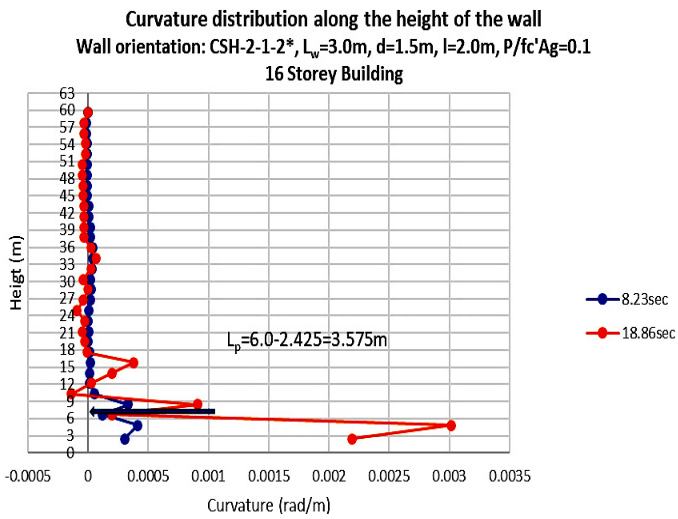

Figure 7. Curvature distribution along the height of the wall, CSH-2-1-2*, L_w= 3.0 m, d= 1.5 m, P/fc'Ag= 0.1, 16 storey.

corresponding walls. Then the plastic hinge lengths (L_p) are calculated from the inelastic portion of the graph in bottom of the buildings using the yield curvature. Reference Figure 1, the vertical coordinate of the integration section for the bottom element which represents the base of the building is 2.425 (m). Corresponding vertical co-ordinate based on the yield curvature (0.002, see Table 1) along the height (inelastic curvatures, SeismoStruct results is 6.5 (m).

Hence, L_p = 6.5 (m)–2.425 (m)= 4.075(m)

Analyses results: Inelastic curvatures are calculated on numerous heights of all the buildings (08, 12, and 16 stories buildings), those are plotted along the height of the couple shear walls. Some of them are shown in below Figures 1-8. Total 79 numbers of couple shear walls with different design parameters are analysed (Tables 2-7).

Discussion

The present study focuses on estimation of plastic hinge length for couple shear wall. Couple shear wall is one of the effective seismic forces resisting

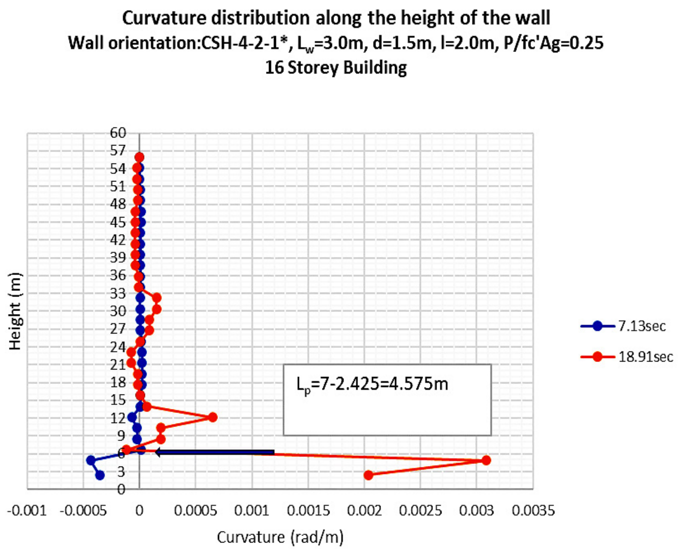

Figure 8. Curvature distribution along the height of the wall, CSH-4-2-1*, L_w= 3.0 m, d= 1.5 m, P/fc'Ag= 0.25, 16 storey.

Table 1. Yield curvature used in the calculations.

Methods of Calculation	Yield Curvature
Priestley, et al.	0.002 for 2.0 m width, 0.0018 for 2.5 m width,0.0015 for 3.0 m width
Montes and Aschleim	0.002 for 2.0 m width, 0.0019 for 2.5 m width, 0.0016 for 3.0 m width
Eurocodes	0.000879 for 2.0 m width, 0.000704 for 2.5 m width, 0.00586 for 3.0 m width
Push Over Analysis (SeismoStruct)	0.001 for 2.5 m width, 16 storey building

system that have been using all over the World for medium and high-rise buildings. Seismic reinforcement detailing over the hinge regions depends on accurate estimation of plastic hinge length. According to CSA-A23.3-04, the couple shear wall falls under the clause article 21.6.8 and are noted that for those walls, the height of the plastic hinge length shall be taken at least 1.5 times the longest individual element in the direction under consideration.

In general, it is a common practice to assume that the inelastic curvature in a cantilever shear wall is uniform over the plastic hinge length and can be taken from 0.5 to 1.0 times the wall length (L_w) for safe estimation. But it we looked the literature review mentioned in this research, most of the estimation formulas developed by different researchers are taken basically experimental studies done on beam or column members. The structural behaviour of shear wall, specifically couple shear wall cannot be even close to the column member actions. In Canadian code (CSA-A23.3-04), the requirements of plastic hinge length for cantilever shear wall and couple shear wall (clause 21.6.8) are identical, i.e., 1.5 times the wall length (L_w), where L_w is the length of the longest cantilever wall and for the couple shear wall L_w is the longest individual element in the direction under consideration. And, in Canadian Code (CSA A23.3-14), the equation to calculate the plastic hinge length for cantilever and couple shear wall is also identical, i.e. ($0.5L_w$ +0.1*H), where l_w is the length of longest cantilever shear wall and overall length of the coupled shear walls in the direction under consideration.

But structural action of cantilever wall and couple shear wall are different. In couple shear wall, one wall acts as tension and other one acts as compression wall. Whereas in cantilever wall, one side of the wall acts as tension and other side acts as compression. Couple shear walls are connected with coupling beams that makes mass and stiffness variation different throughout the height than the cantilever shear wall. And the systems of high degree of coupling are subjected to lateral loading, the shear force in the coupling beams induce high degree of axial force in the wall. In the couple shear wall, the lateral force distribution also different than the cantilever shear wall due to the connections of the coupling beams.

Table 2. Plastic hinge lengths (L_p) in meter for 08 storey buildings.

Wall Parameters								Non-linear Dynamic Analysis		CSA-A23.3.04	CSA-A23.3-14/ ACI-481 (1968)
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D / L Ratio	L_p (m)	L_p in terms of L_w	L_p = 1.5*Lw (m)	$L_p = 0.5*L_w$ +0.1*H (m)
CSH-4-2-2*	2.5	30.4	12.16	0.033	0.8	2	0.4	2.575	1.03	3.75	5.54
CSH-4-2-1*	2.5	30.4	12.16	0.033	0.8	2	0.4	2.575	1.03	3.75	5.54
CSH-2-1-2*	2.5	30.4	12.16	0.033	0.8	2	0.4	2.575	1.03	3.75	5.54
CSH-4-2-2*	2.5	30.4	12.16	0.033	1	2	0.5	2.575	1.03	3.75	5.54
CSH-4-2-1*	2.5	30.4	12.16	0.033	1	2	0.5	2.575	1.03	3.75	5.54
CSH-2-1-2*	2.5	30.4	12.16	0.033	1	2	0.5	2.575	1.03	3.75	5.54
CSH-4-2-2*	2.5	30.4	12.16	0.05	1	2	0.5	3.075	1.13	3.75	5.54
CSH-4-2-1*	2.5	30.4	12.16	0.05	1	2	0.5	2.575	1.03	3.75	5.54
CSH-2-1-2*	2.5	30.4	12.16	0.05	1	2	0.5	3.075	1.23	3.75	5.54
CSH-4-2-2*	2	30.4	15.2	0.033	0.8	2	0.4	3.075	1.5375	3	5.04
CSH-4-2-1*	2	30.4	15.2	0.04	0.8	2	0.4	2.575	1.28	3	5.04
CSH-2-1-2*	2	30.4	15.2	0.04	0.8	2	0.4	4.075	2.0375	3	5.04
CSH-4-2-2*	2	30.4	15.2	0.05	0.8	2	0.4	3.075	1.5375	3	5.04
CSH-4-2-1*	2	30.4	15.2	0.05	0.8	2	0.4	4.075	2.0375	3	5.04
CSH-2-1-2*	2	30.4	15.2	0.05	0.8	2	0.4	4.075	2.0375	3	5.04
CSH-4-2-2*	2	30.4	15.2	0.033	1	2	0.5	3.075	1.5375	3	5.04
CSH-4-2-1*	2	30.4	15.2	0.033	1	2	0.5	4.075	2.0375	3	5.04
CSH-2-1-2*	2	30.4	15.2	0.033	1	2	0.5	4.075	2.0375	3	5.04
CSH-4-2-2*	2	30.4	15.2	0.05	0.8	2	0.4	3.075	1.5375	3	5.04
CSH-4-2-1*	2	30.4	15.2	0.05	1	2	0.5	4.075	2.0375	3	5.04
CSH-2-1-2*	2	30.4	15.2	0.05	1	2	0.5	4.075	2.0375	3	5.04
CSH-4-2-1*	3	30.4	10.13	0.033	1	2	0.5	3.075	1.025	4.5	6.04
CSH-2-1-2*	3	30.4	10.13	0.033	1	2	0.5	2.575	0.858	4.5	6.04
CSH-4-2-2*	3	30.4	10.13	0.033	0.8	2	0.4	2.575	0.858	4.5	6.04
CSH-4-2-2*	3	30.4	10.13	0.05	0.8	2	0.4	4.075	1.358	4.5	6.04
CSH-4-2-1*	3	30.4	10.13	0.05	0.8	2	0.4	4.075	1.358	4.5	6.04
CSH-2-1-2*	3	30.4	10.13	0.05	0.8	2	0.4	2.575	0.858	4.5	6.04
CSH-4-2-2*	3	30.4	10.13	0.05	1	2	0.5	2.575	0.858	4.5	6.04
CSH-4-2-1*	3	30.4	10.13	0.05	1	2	0.5	2.575	0.858	4.5	6.04
CSH-2-1-2*	3	30.4	10.13	0.05	1	2	0.5	2.575	0.858	4.5	6.04

Note: For coupled shear wall, L_w is considered overall length of the coupled shear wall. (CSA A23.3-14, clause 21.5.2.1.2)

Dynamic time history analyses are done for total 79 numbers different walls having different wall aspect ratio, compressive loads, and different aspect ratio of coupling beams and tabulated in above Tables 2-7; It needs to be noted that CSH 2-1-2*, CSH-4-2-1*, and CSH-4-2-2* are the bottom level elements of the Couple shear wall of different buildings (08, 12, and 16 stories) along the direction of the earthquake loadings are applied. The maximum deflections are checked as per CSA-A23.3-04, at the top the buildings to ensure maximum

Table 3. Plastic hinge lengths (L_p) in meter for 08 storey buildings.

Wall Parameters								Non-linear Dynamic Analysis		Mattock	Paulay & Priestly
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D / L Ratio	L_p (m)	L_p in terms of L_w	$L_p = 0.5*d + 0.05z$ (m)	$L_p = 0.25*L_w + 0.044*h_w$ (m)
CSH-4-2-2*	2.5	30.4	12.16	0.033	0.8	2.0	0.4	2.575	1.03	2.77	1.96
CSH-4-2-1*	2.5	30.4	12.16	0.033	0.8	2.0	0.4	2.575	1.03	2.77	1.96
CSH-2-1-2*	2.5	30.4	12.16	0.033	0.8	2.0	0.4	2.575	1.03	2.77	1.96
CSH-4-2-2*	2.5	30.4	12.16	0.033	1.0	2.0	0.5	2.575	1.03	2.77	1.96
CSH-4-2-1*	2.5	30.4	12.16	0.033	1.0	2.0	0.5	2.575	1.03	2.77	1.96
CSH-2-1-2*	2.5	30.4	12.16	0.033	1.0	2.0	0.5	2.575	1.03	2.77	1.96
CSH-4-2-2*	2.5	30.4	12.16	0.05	1.0	2.0	0.5	3.075	1.13	2.77	1.96
CSH-4-2-1*	2.5	30.4	12.16	0.05	1.0	2.0	0.5	2.575	1.03	2.77	1.96
CSH-2-1-2*	2.5	30.4	12.16	0.05	1.0	2.0	0.5	3.075	1.23	2.77	1.96
CSH-4-2-2*	2.0	30.4	15.2	0.033	0.8	2.0	0.4	3.075	1.5375	2.52	1.83
CSH-4-2-1*	2.0	30.4	15.2	0.04	0.8	2.0	0.4	2.575	1.28	2.52	1.83
CSH-2-1-2*	2.0	30.4	15.2	0.04	0.8	2.0	0.4	4.075	2.0375	2.52	1.83
CSH-4-2-2*	2.0	30.4	15.2	0.05	0.8	2.0	0.4	3.075	1.5375	2.52	1.83
CSH-4-2-1*	2.0	30.4	15.2	0.05	0.8	2.0	0.4	4.075	2.0375	2.52	1.83
CSH-2-1-2*	2.0	30.4	15.2	0.05	0.8	2.0	0.4	4.075	2.0375	2.52	1.83
CSH-4-2-2*	2.0	30.4	15.2	0.033	1.0	2.0	0.5	3.075	1.5375	2.52	1.83
CSH-4-2-1*	2.0	30.4	15.2	0.033	1.0	2.0	0.5	4.075	2.0375	2.52	1.83
CSH-2-1-2*	2.0	30.4	15.2	0.033	1.0	2.0	0.5	4.075	2.0375	2.52	1.83
CSH-4-2-2*	2.0	30.4	15.2	0.05	0.8	2.0	0.4	3.075	1.5375	2.52	1.83
CSH-4-2-1*	2.0	30.4	15.2	0.05	1.0	2.0	0.5	4.075	2.0375	2.52	1.83
CSH-2-1-2*	2.0	30.4	15.2	0.05	1.0	2.0	0.5	4.075	2.0375	2.52	1.83
CSH-4-2-1*	3.0	30.4	10.13	0.033	1.0	2.0	0.5	3.075	1.025	3.02	2.09
CSH-2-1-2*	3.0	30.4	10.13	0.033	1.0	2.0	0.5	2.575	0.858	3.02	2.09
CSH-4-2-2*	3.0	30.4	10.13	0.033	0.8	2.0	0.4	2.575	0.858	3.02	2.09
CSH-4-2-2*	3.0	30.4	10.13	0.05	0.8	2.0	0.4	4.075	1.358	3.02	2.09
CSH-4-2-1*	3.0	30.4	10.13	0.05	0.8	2.0	0.4	4.075	1.358	3.02	2.09
CSH-2-1-2*	3.0	30.4	10.13	0.05	0.8	2.0	0.4	2.575	0.858	3.02	2.09
CSH-4-2-2*	3.0	30.4	10.13	0.05	1.0	2.0	0.5	2.575	0.858	3.02	2.09
CSH-4-2-1*	3.0	30.4	10.13	0.05	1.0	2.0	0.5	2.575	0.858	3.02	2.09
CSH-2-1-2*	3.0	30.4	10.13	0.05	1.0	2.0	0.5	2.575	0.858	3.02	2.09

Table 4. Plastic hinge lengths (L_p) in meter for 12 storey buildings.

Wall Parameters								Non-linear Dynamic Analysis		CSA-A23.3-04	CSA-A23.3-14/ ACI-481
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D/ L Ratio	L_p (m)	L_p in terms of L_w	$L_p = 1.5*L_w$ (m)	$L_p = 0.5*L_w + 0.1*H$ (m)
CSH-4-2-2*	2.5	45	18	0.1	1.6	2.0	0.8	3.575	1.43	3.75	7.0
CSH-4-2-1*	2.5	45	18	0.1	1.6	2.0	0.8	4.075	1.63	3.75	7.0
CSH-2-1-2*	2.5	45	18	0.1	1.6	2.0	0.8	4.075	1.63	3.75	7.0
CSH-4-2-2*	2.5	45	18	0.188	1.6	2.0	0.8	3.075	1.23	3.75	7.0
CSH-4-2-1*	2.5	45	18	0.188	1.6	2.0	0.8	4.325	1.73	3.75	7.0
CSH-2-1-2*	2.5	45	18	0.188	1.6	2.0	0.8	4.075	1.63	3.75	7.0
CSH-4-2-2*	2.5	45	18	0.075	1.75	2.0	0.875	3.075	1.23	3.75	7.0
CSH-4-2-1*	2.5	45	18	0.075	1.75	2.0	0.875	4.075	1.63	3.75	7.0
CSH-2-1-2*	2.5	45	18	0.075	1.75	2.0	0.875	7.075	2.83	3.75	7.0
CSH-4-2-2*	2.5	45	18	0.188	1.75	2.0	0.875	3.575	1.43	3.75	7.0
CSH-4-2-2*	2.0	45	22.5	0.1	1.75	1.6	0.875	3.575	1.78	3.0	6.5
CSH-4-2-2*	2.0	45	22.5	0.188	1.6	2.0	0.8	3.575	1.78	3.0	6.5
CSH-4-2-1*	2.0	45	22.5	0.188	1.6	2.0	0.8	3.825	1.91	3.0	6.5
CSH-2-1-2*	2.0	45	22.5	0.188	1.6	2.0	0.8	4.075	2.03	3.0	6.5
CSH-4-2-2*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.0	6.5
CSH-4-2-1*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.0	6.5
CSH-2-1-2*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.0	6.5
CSH-4-2-2*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.0	6.5
CSH-4-2-1*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.0	6.5
CSH-2-1-2*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.0	6.5
CSH-4-2-2*	3.0	45	15	0.075	1.6	2.0	0.8	3.325	1.1	4.5	7.5
CSH-4-2-1*	3.0	45	15	0.075	1.6	2.0	0.8	8.075	2.69	4.5	7.5

CSH-2-1-2*	3.0	45	15	0.075	1.6	2.0	0.8	3.575	1.19	4.5	7.5
CSH-4-2-2*	3.0	45	15	0.188	1.6	2.0	0.8	2.575	0.858	4.5	7.5
CSH-4-2-1*	3.0	45	15	0.188	1.6	2.0	0.8	4.075	1.358	4.5	7.5
CSH-2-1-2*	3.0	45	15	0.188	1.6	2.0	0.8	3.825	1.275	4.5	7.5
CSH-4-2-2*	3.0	45	15	0.075	1.75	2.0	0.875	1.575	0.525	4.5	7.5
CSH-4-2-1*	3.0	45	15	0.075	1.75	2.0	0.875	2.575	0.858	4.5	7.5
CSH-2-1-2*	3.0	45	15	0.075	1.75	2.0	0.875	8.075	2.69	4.5	7.5
CSH-4-2-2*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	4.5	7.5
CSH-4-2-1*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	4.5	7.5
CSH-2-1-2*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	4.5	7.5
CSH-4-2-2*	3.0	45	15	0.075	1.4	2.0	0.7	2.575	0.858	4.5	7.5

Note: For coupled shear wall, L_w is considered overall length of the coupled shear wall. (CSA A23.3-14, clause 21.5.2.1.2

Table 5. Plastic hinge lengths (L_p) in meter for 12 storey buildings.

Wall Parameters								Non-linear Dynamic Analysis		Mattock	Paulay & Priestly
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D / L Ratio	L_p (m)	L_p in terms of L_w	L_p = 0.5*d + 0.05z (m)	L_p = 0.25* L_w + 0.044* h_w (m)
CSH-4-2-2*	2.5	45	18	0.1	1.6	2.0	0.8	3.575	1.43	3.5	2.61
CSH-4-2-1*	2.5	45	18	0.1	1.6	2.0	0.8	4.075	1.63	3.5	2.61
CSH-2-1-2*	2.5	45	18	0.1	1.6	2.0	0.8	4.075	1.63	3.5	2.61
CSH-4-2-2*	2.5	45	18	0.188	1.6	2.0	0.8	3.075	1.23	3.5	2.61
CSH-4-2-1*	2.5	45	18	0.188	1.6	2.0	0.8	4.325	1.73	3.5	2.61
CSH-2-1-2*	2.5	45	18	0.188	1.6	2.0	0.8	4.075	1.63	3.5	2.61
CSH-4-2-2*	2.5	45	18	0.075	1.75	2.0	0.875	3.075	1.23	3.5	2.61
CSH-4-2-1*	2.5	45	18	0.075	1.75	2.0	0.875	4.075	1.63	3.5	2.61
CSH-2-1-2*	2.5	45	18	0.075	1.75	2.0	0.875	7.075	2.83	3.5	2.61
CSH-4-2-2*	2.5	45	18	0.188	1.75	2.0	0.875	3.575	1.43	3.25	2.61
CSH-4-2-2*	2.0	45	22.5	0.1	1.75	1.6	0.875	3.575	1.78	3.25	2.48
CSH-4-2-2*	2.0	45	22.5	0.188	1.6	2.0	0.8	3.575	1.78	3.25	2.48
CSH-4-2-1*	2.0	45	22.5	0.188	1.6	2.0	0.8	3.825	1.91	3.25	2.48
CSH-2-1-2*	2.0	45	22.5	0.188	1.6	2.0	0.8	4.075	2.03	3.25	2.48
CSH-4-2-2*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.25	2.48
CSH-4-2-1*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.25	2.48
CSH-2-1-2*	2.0	45	22.5	0.1	1.8	2.0	0.9	3.575	1.78	3.25	2.48
CSH-4-2-2*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.25	2.48
CSH-4-2-1*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.25	2.48
CSH-2-1-2*	2.0	45	22.5	0.188	1.8	2.0	0.9	4.075	2.03	3.25	2.48
CSH-4-2-2*	3.0	45	15	0.075	1.6	2.0	0.8	3.325	1.1	3.75	2.73
CSH-4-2-1*	3.0	45	15	0.075	1.6	2.0	0.8	8.075	2.69	3.75	2.73
CSH-2-1-2*	3.0	45	15	0.075	1.6	2.0	0.8	3.575	1.19	3.75	2.73
CSH-4-2-2*	3.0	45	15	0.188	1.6	2.0	0.8	2.575	0.858	3.75	2.73
CSH-4-2-1*	3.0	45	15	0.188	1.6	2.0	0.8	4.075	1.358	3.75	2.73
CSH-2-1-2*	3.0	45	15	0.188	1.6	2.0	0.8	3.825	1.275	3.75	2.73
CSH-4-2-2*	3.0	45	15	0.075	1.75	2.0	0.875	1.575	0.525	3.75	2.73
CSH-4-2-1*	3.0	45	15	0.075	1.75	2.0	0.875	2.575	0.858	3.75	2.73
CSH-2-1-2*	3.0	45	15	0.075	1.75	2.0	0.875	8.075	2.69	3.75	2.73
CSH-4-2-2*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	3.75	2.73
CSH-4-2-1*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	3.75	2.73
CSH-2-1-2*	3.0	45	15	0.188	1.75	2.0	0.875	3.575	1.19	3.75	2.73
CSH-4-2-2*	3.0	45	15	0.075	1.4	2.0	0.7	2.575	0.858	3.75	2.73

inelastic rotational demand is less than the maximum rotational capacity and to make sure ductility at the plastic hinge locations. Those are summarized in below (Table 8).

The maximum inelastic rotational demand of coupling beams as per CSA A23.3-04 (clause 21.6.8.4) are calculated also and found within the inelastic rotational capacity, those are summarized in below (Table 9).

The findings from the present studies are:

- The maximum inelastic curvatures are not uniform over the plastic hinge length.
- The plastic hinge length calculation as per Canadian code (CSA-A23.3-04) for couple shear wall (clause 21.6.8), i.e., 1.5 times the wall length (L_w) is not conservative and underestimation for couple shear wall seismic reinforcement detailing and more critical for slender couple shear walls. And the plastic hinge length calculation as per Canadian code CSA A23.3-14 (clause 21.5.2.1.2) is over conservative for coupled shear wall buildings and much more over conservative for slender coupled shear walls. The equation

Table 6. Plastic hinge lengths (L_p) in meter for 16 storey buildings.

	Wall Parameters							Non-linear Dynamic Analysis		CSA-A23.3-04	CSA A23.3-14/ACI-481
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D / L Ratio	$L_{p\,(m)}$	L_p in terms of L_w	$L_p = 1.5*L_{w\,(m)}$	$L_p = 0.5*L_w + 0.1*H$ (m)
CSH-2-1-2*	3	59.6	19.86	0.1	1.4	2	0.7	3.575	1.19	4.5	8.96
CSH-4-2-2*	3	59.6	19.86	0.1	1.5	2	0.75	1.575	0.525	4.5	8.96
CSH-4-2-1*	3	59.6	19.86	0.1	1.5	2	0.75	3.575	1.19	4.5	8.96
CSH-2-1-2*	3	59.6	19.86	0.1	1.5	2	0.75	3.575	1.19	4.5	8.96
CSH-4-2-2*	3	59.6	19.86	0.25	1.5	2	0.75	2.575	0.858	4.5	8.96
CSH-4-2-1*	3	59.6	19.86	0.25	1.5	2	0.75	4.575	1.53	4.5	8.96
CSH-2-1-2*	3	59.6	19.86	0.25	1.5	2	0.75	3.575	1.19	4.5	8.96
CSH-4-2-2*	3	59.6	19.86	0.25	1.35	2	0.675	2.575	0.858	4.5	8.96
CSH-4-2-1*	3	59.6	19.86	0.25	1.35	2	0.675	3.575	1.19	4.5	8.96
CSH-2-1-2*	3	59.6	19.86	0.25	1.35	2	0.675	4.575	1.53	4.5	8.96
CSH-4-2-2*	3	59.6	19.86	0.1	1.85	2	0.925	2.575	0.858	4.5	8.96
CSH-4-2-1*	3	59.6	19.86	0.1	1.85	2	0.925	3.575	1.19	4.5	8.96
CSH-2-1-2*	3	59.6	19.86	0.1	1.85	2	0.925	3.575	1.19	4.5	8.96
CSH-4-2-2*	3	59.6	19.86	0.25	1.85	2	0.925	2.575	0.858	4.5	8.96
CSH-4-2-1*	3	59.6	19.86	0.25	1.85	2	0.925	3.575	1.19	4.5	8.96
CSH-2-1-2*	3	59.6	19.86	0.25	1.85	2	0.925	4.575	1.53	4.5	8.96

Note: For coupled shear wall, L_w is considered overall length of the coupled shear wall. (CSA A23.3-14, clause 21.5.2.1.2)

Table 7. Plastic hinge lengths (L_p) in meter for 16 storey buildings.

	Wall Parameters							Non-linear Dynamic Analysis		Mattock	Paulay & Priestly
Wall Name	L_w	h_w	SL ratio	P / fc'Ag	d	L	D / L Ratio	$L_{p\,(m)}$	L_p in terms of L_w	L_p=0.5*d +0.05z (m)	$L_p = 0.25*L_w + 0.044*h_w$ (m)
CSH-2-1-2*	3	59.6	19.86	0.1	1.4	2	0.7	3.575	1.19	4.48	3.37
CSH-4-2-2*	3	59.6	19.86	0.1	1.5	2	0.75	1.575	0.525	4.48	3.37
CSH-4-2-1*	3	59.6	19.86	0.1	1.5	2	0.75	3.575	1.19	4.48	3.37
CSH-2-1-2*	3	59.6	19.86	0.1	1.5	2	0.75	3.575	1.19	4.48	3.37
CSH-4-2-2*	3	59.6	19.86	0.25	1.5	2	0.75	2.575	0.858	4.48	3.37
CSH-4-2-1*	3	59.6	19.86	0.25	1.5	2	0.75	4.575	1.53	4.48	3.37
CSH-2-1-2*	3	59.6	19.86	0.25	1.5	2	0.75	3.575	1.19	4.48	3.37
CSH-4-2-2*	3	59.6	19.86	0.25	1.35	2	0.675	2.575	0.858	4.48	3.37
CSH-4-2-1*	3	59.6	19.86	0.25	1.35	2	0.675	3.575	1.19	4.48	3.37
CSH-2-1-2*	3	59.6	19.86	0.25	1.35	2	0.675	4.575	1.53	4.48	3.37
CSH-4-2-2*	3	59.6	19.86	0.1	1.85	2	0.925	2.575	0.858	4.48	3.37
CSH-4-2-1*	3	59.6	19.86	0.1	1.85	2	0.925	3.575	1.19	4.48	3.37
CSH-2-1-2*	3	59.6	19.86	0.1	1.85	2	0.925	3.575	1.19	4.48	3.37
CSH-4-2-2*	3	59.6	19.86	0.25	1.85	2	0.925	2.575	0.858	4.48	3.37
CSH-4-2-1*	3	59.6	19.86	0.25	1.85	2	0.925	3.575	1.19	4.48	3.37
CSH-2-1-2*	3	59.6	19.86	0.25	1.85	2	0.925	4.575	1.53	4.48	3.37

Table 8. Max deflection at top of the buildings and inelastic rotational demand and capacity.

Building Description	Max Deflection at Top (mm)	Inelastic Rotational Demand	Inelastic Rotational Capacity
08 storey building	126	0.023	0.025
16 storey building	213	0.02	0.025

Table 9. Inelastic rotational demand and capacity of coupling beams.

Building Description	Wall Length (m)	Inelastic Rotational Demand	Inelastic Rotational Capacity with Diagonal Reinforcements
08 storey building	2	0.04	0.04
16 storey building	2.5	0.04	0.04

used to calculate the plastic hinge length as per CSA A23.3-14 is basically identical equation to ACI 481 recommendation, which is over conservative and using this equation in construction requirements is basically wasting of money. It was not even continued in recent US code also, (ACI 318-19).

- The plastic hinge length for couple shear wall calculation also mostly depends on wall length.
- Not significant impacts are found on axial load effect.
- Not significant impacts are found on yield strength of reinforcements.
- Minor effects are noted based on (d/L) ratio of coupling beam.
- A comparison studies with different researchers are shown in above Table 2- 7. After investigating different results of different researchers as well as Canadian codes, CSA A23.3-04 and CSA A23.3-14

and ACI 318-19, an expression is proposed for more accurate and safe estimation of plastic hinge length for couple shear wall, with a multiplication factor of 2.0 instead of 1.5 for wall length (L_w); Where L_w is considered as the longer wall length between tension wall and compression wall. Hence, for couple shear wall calculation, the proposed equation shall be 2.0 times the wall length in the direction under consideration.

- Similar observations in estimation of inelastic curvature are observed in experimental studies done by Adebar Figure 9 and author's findings Figure 10.

Results

The author's findings are compared with one of the experimental studies done by Adebar. In this Figure the curvatures of the wall were measured at numerous locations of the wall experimentally and plotted (rad/km), Figure 9. In this experiment 12.2 m high and 1.625 m long cantilever shear walls (flanged cross-section) were tested. To simulate location of the plastic hinge in the test, which is practically located immediately above the foundation base for cantilever shear wall, a construction joint was created in the test 426 mm (17 inch) up from the wall base block of the experiment (Adebar, Ibrahim). The vertical axis data of the Figure 9 were measured from this construction joint, which is basically very close to the foundation (Adebar, Ibrahim). Consequently, curvatures are calculated analytically (author's findings) at numerous points over the height and plotted (rad/km) for 12 storey buildings, Figure 10: The curvature at the 5 m height from the experimental results, done by Adebar, is approximately 1.5 rad/km and the curvature at level 05 m from analytical results (author's findings) is approximately 1.6 rad/km, which is very close (Figures 9 and 10).

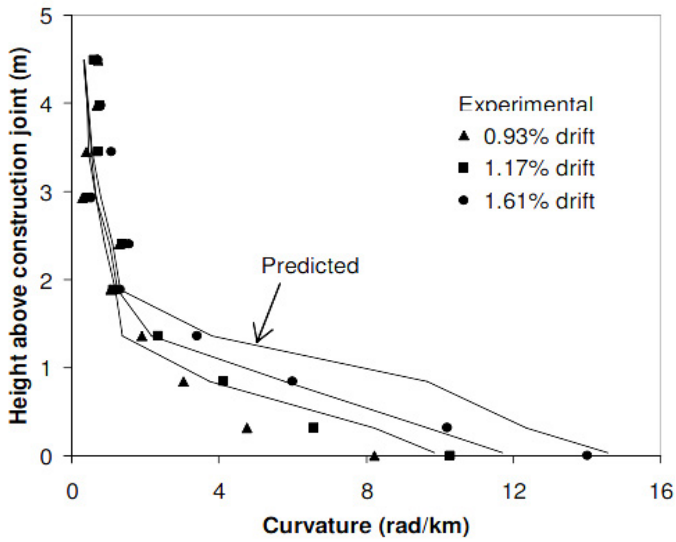

Figure 9. Calibration of analytical model: Comparison with results from a large-scale test (Adebar, et al.).

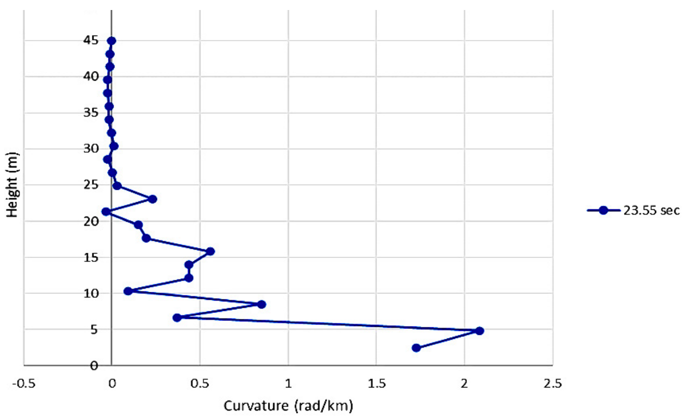

Figure 10. Curvature distribution along the height of the wall, L_w= 3.0 m, d= 1.6 m, P/ fc'Ag= 0.075, 12 storey wall.

Conclusion

Hence, it can be commended that using the proposed multiplication factor to calculate the plastic hinge length calculation will cover the safe estimation of plastic hinge length calculations. In this research, buildings with couple shear walls seismic force resisting systems are modelled with simple configurations and distribution of main reinforcements are considered uniform. But in real industry practice the gradual variation of reinforcement are considered due to gradual reduction of bending moment capacity along the height of the walls. The reduction of reinforcements can result further spreading the plasticity in critical regions. That phenomenon can be considered in future research. Along with this, the effect of reinforcement yield strength needs to be further studied with more detail data.

Acknowledgement

None.

Conflict of Interest

None.

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