Behavior and design of reinforced concrete walking columns

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Abstract: This research is focused on finite element analysis on reinforced concrete walking columns structure. Walking columns with three span-to-depth ratios (aspect ratios) are tested. The aspect ratio of 1:9 is taken to represent the slender walking columns. For a medium aspect ratio of 1:4 is used. Meanwhile, the aspect ratio of 1:1 is adopted for squat walking columns. All walking column models are designed in accordance with ACI 318-19 strut-and-tie method. This method is considered since it is widely used for deep beam design and a walking column serves a similar function as a deep beam as a shear transfer element. The finite element analysis shows that the strut-and-tie method provides conservative design results for all aspect ratios.

Keywords: *walking column, strut-and-tie, aspect ratio, axial capacity, finite element analysis*

1. Introduction

For architectural purposes, some columns are not continuously built throughout the building. For thus shifted part, a transfer structure is required to accommodate the high shear force in the shifted region. The walking column is one of the shear transfer elements. It is commonly built in a non-seismic region due to lesser limitations for irregularities. Nevertheless, the researches about the walking columns are very rare. Consequently, the behavior of the walking column is not yet understood. Moreover, there are no specific guidelines for walking column design.

This research's design method for walking columns employs the strut-and-tie method (STM) based on ACI 318-19 provision. This method is widely used in designing other shear transfer elements such as deep beams. This method is proven effective in predicting the shear strength compared to semi-empirical shear design equations [1]. The code provision [2] defines that for members that are loaded on one face and supported on the opposite face and satisfy (a) clear span does not exceed four times overall member depth and (b) concentrated loads exist within a distance of 2h from the face of the support can be categorized as deep beam and can be designed using STM.

However, walking columns are different from the deep beam because the span-to-depth ratio of walking columns commonly found in practice varies from 1:9 to 1:4. In practice, for an aspect ratio below 1:4, the deep beam is preferred since it is safer from overturning moment as it is supported on the two opposite faces. Since the common aspect ratio of the walking column is relatively small, for some slender cases, strut-like compression elements cannot form diagonally as it occurs on deep beam elements described in [2]. Therefore, it is also necessary to evaluate whether or not using STM to design walking columns is efficient in terms of strength and material usage for all aspect ratios, including the slender one.

Finite element analysis (FEA) is used to assess the performance of walking columns in this research. The FEA can be a powerful tool to analyze structural elements, especially those whose behavior is not well understood. The finite element model could accurately predict the shear transfer mechanism and the influence of cracks on several structural elements, as done by Chen et al. [3].

This research objective is to study the behavior of the walking columns better so that an appropriate design method can be determined. In this research, squat, medium, and slender walking column models are built in the ATENA 3D FEA software. Those models are analyzed using an incremental displacement test to obtain the reactions and downward displacements. Then the actual axial capacity obtained from FEA is then compared to those obtained from STM to study whether or not the STM can provide a safe design method for walking columns.

2. Research Significance

This research aims to investigate the behavior of walking column elements and their appropriate design method. It could provide tools for engineers in designing walking columns safely.

3. Finite Element Analysis

3.1 General

The behavior of the walking column is not yet well known. Therefore, it will be costly and risky to conduct full-scale specimen tests. For this reason, it will be much more effective, cost-friendly, and time-saving to use three-dimensional FEM analysis to model walking columns. This research uses ATENA 3D FEM analysis to observe the behavior of reinforced concrete walking columns under specific axial forces.

The FEM analysis in this research will be verified by test results of deep beam done by Aguilar et al. [4]. The deep beam is used for verification since previous research of walking columns behavior is not widely available. Therefore, the deep beam is taken since its closeness to a walking column in terms of function as a shear transfer element. Testing of the deep beam with an aspect ratio of 1:1 is used for squat aspect ratio verification.

3.2 Material Modeling

The concrete material is modeled using solid elements. The compressive stress-strain relationship is governed by the model proposed by Chen [5]. The peak compressive stress of the concrete, f_c', is 8 ksi. The limit compressive strain, ε_d at zero stress, depends on a plastic displacement and a band size. The latter is determined based on the crack band theory [6]. The post-cracking behavior of concrete follows the Modified Compression Field Theory [7]. After cracking, the concrete compressive strength is reduced in the direction parallel to the crack.

For the tension part, the tensile stress grows linearly until it reaches the tensile strength within the uncracked range. The linear slope is the same as the secant modulus at the peak stress in compression. The tensile strength will decrease as the crack width increases. The crack opening law follows the exponential crack opening law proposed by Hordijk [8]. The tensile stress will reduce until the stress is completely released.

The reinforcements are modeled as one-dimensional lines. The stress-strain relationship uses a bilinear model by European code, EN 1992-1-1. This code has three kinds of strength: characteristic strength (actual strength), design strength, and mean value of strength. In this research, characteristic strength is used since the strength reduction factor should be 1 to capture the actual behavior of the rebar. The tensile yield strength of the rebar is 60 ksi for all reinforcements, and the rebar ductility class is B. The stress-strain relationship of concrete and reinforcement rebar are shown in Figure 1 (a) and (b) respectively.

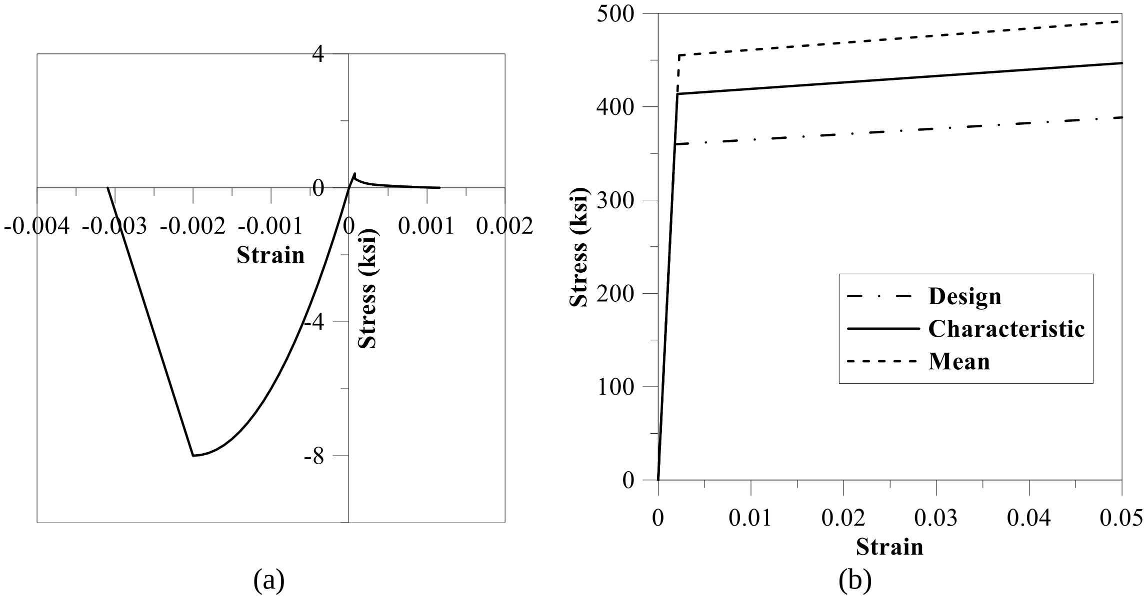


Figure 1. Material stress-strain relationships: (a) concrete, (b) reinforcement rebar

3.3 Model Verification

A deep beam specimen from Aguilar et al. [4] is taken as a reference for verification. The specimen is designed based on the ACI-318-99 Code (ACI-I). The aspect ratio of the specimen is 1:1. This research is force-controlled. The axial load is applied on the two load transfer plates until it reaches 600 kips for each plate. The concrete compressive strength was 4000 psi for design purposes, and the reinforcement yield strength was 60 ksi.

A finite element model of the deep beam is created in ATENA 3D as shown in Figure 2. The upper and lower column of the deep beams were modeled as force transfer plates using rigid material with an elastic modulus of 29000 ksi. While the concrete part of the deep beam was modeled using solid cementitious elements. The reinforcements were modeled using a one-dimensional line element and assumed to be fully bonded with the concrete. Thus, it is assumed no slip occurred on the bond before concrete cracked or crushed.

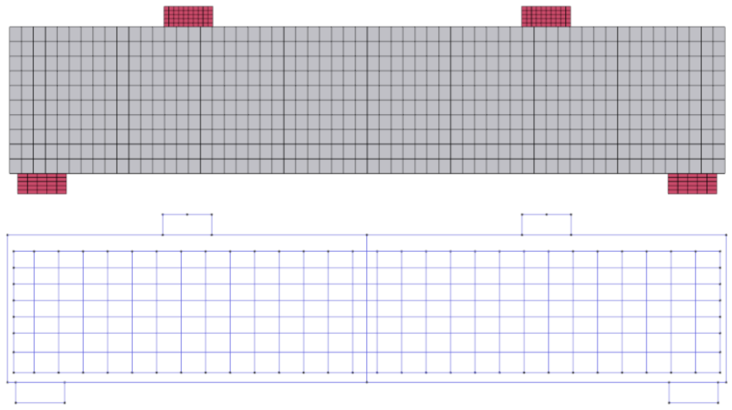


Figure 2. Finite element model of Aguilar et al. used as finite element model verification

Table 1. Comparison of peak displacement and peak applied load of experiment and finite element analysis of Aguilar et al.

Peak Displacement (in.)			Peak Applied Load (kips)		
Experiment (1)	FEA (2)	(2)/(1) (%)	Experiment (1)	FEA (2)	(2)/(1) (%)
1.24	1.29	104.03	305.0	300.0	98.36

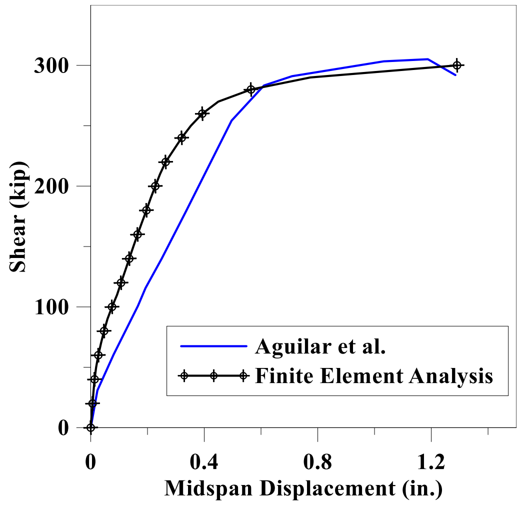


Figure 3. Comparison of finite element analysis results to experiment result

It can be observed from Table 1 and Figure 3 that the peak displacement and the peak applied load obtained from finite element analysis are close to experimental results by Aguilar et al. It is also evident that the FEA can capture the changes in stiffness of the deep beam which the experiment cannot clearly define. The stiffness for linear elastic part is slightly different between Aguilar et al. and finite element method since in finite element, assumptions from mathematical model are used while in laboratory test the condition of the specimen is not very similar.

4. Modeling of Walking Column

Three finite element models are built comprising three different aspect ratios. The aspect ratio of 1:1, 1:4, and 1:9 are taken to represent squat, medium, and slender walking columns, respectively. The reinforcement is designed using the strut-and-tie method by ACI 318-19. Figure 3 shows the application of strut-and-tie method in walking column analysis. In this research, the width is 12 in. for all aspect ratios. And the loading plate is assumed to have a dimension of 12 in. x 12 in.

The reinforcement yield strength is 60,000 psi, while the concrete compressive strength is 8000 psi. It is essential to ensure that the tie reinforcement possesses enough development length in the nodal zone to prevent the reinforcements pull out before reaching the yield strength. The slab thickness is 9 in. It becomes a constraint in tie reinforcement design because all tie reinforcement should be extended into the slab and should have enough development length measured from the critical section. Therefore, the tie height cannot be taken more than the slab thickness for the slab to accommodate all tie reinforcement.

The amount of tie reinforcement should not be less than the minimum amount of reinforcement calculated by the larger of equations (a) and (b) in the code section 9.6.1.2 of ACI 318-19. The cracking control reinforcement should also be designed as vertical and horizontal reinforcement based on the code section 23.5. This reinforcement localizes the cracks to prevent the diagonal cracks from propagating. The cracking control reinforcement is suggested to be closed to have a confinement effect on the concrete section. Both horizontal and vertical cracking control reinforcement should satisfy the minimum distributed reinforcement ratio of 0.0025 in each direction. The walking columns reinforcement design results are shown in Table 2. The reinforcement layout of the walking columns is illustrated in Figure 5.

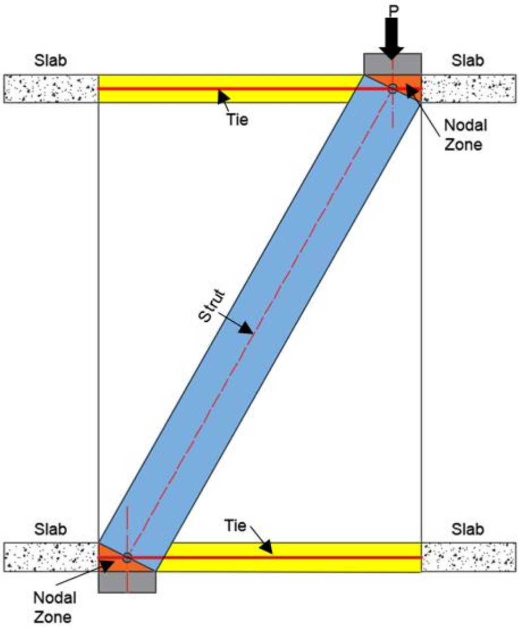


Figure 4. Strut-and-tie concept applied to the walking column

The concrete part is modeled using a solid element in finite element analysis. At the same time, the reinforcement bar is modeled as 1D line reinforcement. The displacement is incrementally applied at the loading plate above the walking column. This loading plate is modeled as linear elastic material with Young's modulus of 29,000 ksi in the FEA since it is not intended to deform under the axial load. Besides, the support and slab elements are also modeled using linear elastic material. The support is restrained in a translational direction. At the same time, the slab provides lateral constrained on the walking column element. It portrays the actual situation, where the surrounding slab laterally constrains the walking column element. Both reinforcement and concrete materials are modeled using the characteristic strength. It means that no reduction factor is used for the material strength.

Table 2. Reinforcement detailing of a walking column designed using STM

Aspect Ratio	Strut-and-tie Method		
	Tie Reinforcement	Vertical Reinforcement	Horizontal Reinforcement
1:9	1#3	#4@10 in. (0.33%) 4#4	#4@10 in. (0.33%)
1:4	3#7	#4@12 in. (0.28%) 6#4	#4@12 in. (0.28%)
1:1	12#8	#4@12 in. (0.28%) 20#4	#4@12 in. (0.28%)

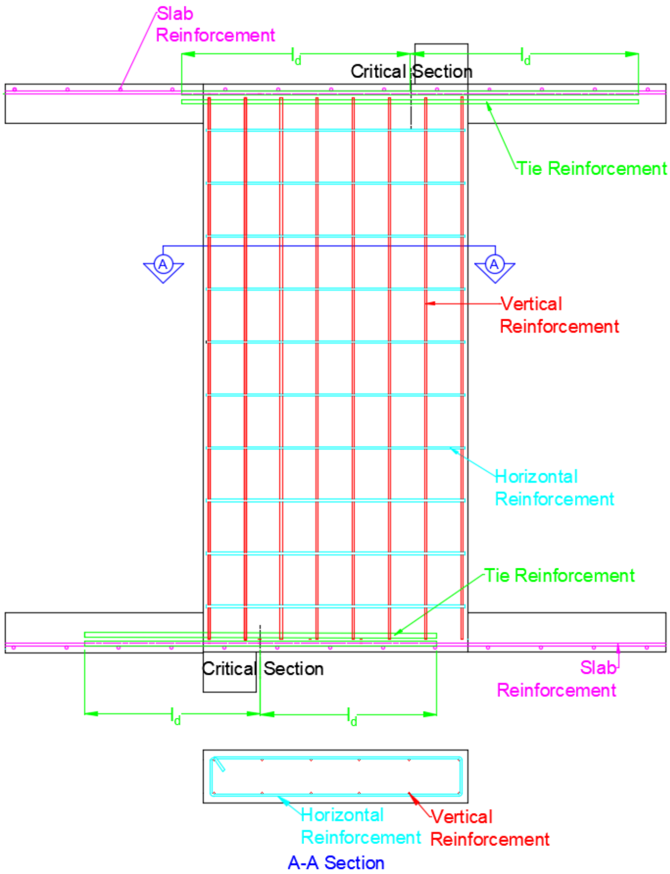


Figure 5. Reinforcement layout of a walking column designed using STM

Two parameters are measured in the FEA. The support reaction and the downward displacement. The support reaction is monitored at the base of the support, and the downward displacement is observed at the bottom of the walking column element. The boundary conditions of the finite element model are shown by Figure 6.

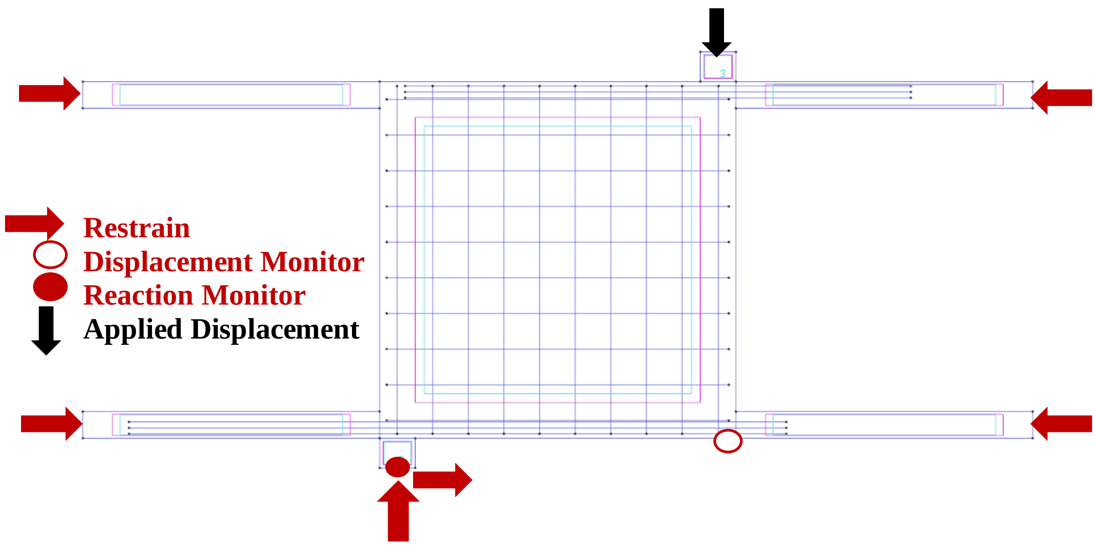


Figure 6. Walking column FEA model for aspect ratio of 1:1

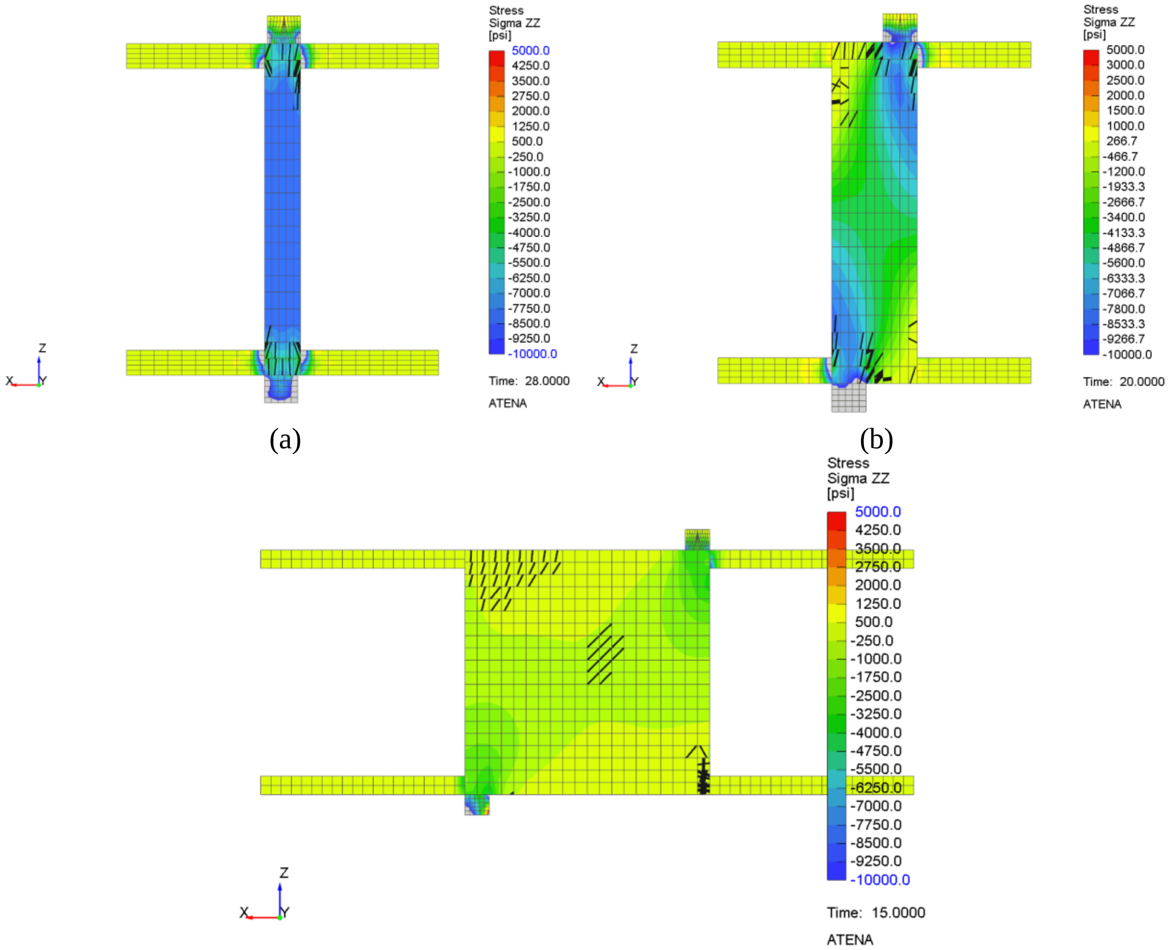
5. Results and Design Methodology

5.1 Results

The stress flow and cracking at the ultimate condition are observed. Figure 7 (a), (b), (c) shows the stress condition of walking columns in z-direction for aspect ratio of 1:9, 1:4 and 1:1 respectively. Obviously, in the case of the slender walking column with an aspect ratio of 1:9, the entire column is under high compressive stress. The crack also occurs vertically. It shows that the walking column with an aspect ratio 1:9 failed due to axial compression. This behavior is similar to column behavior.

Meanwhile, for an aspect ratio of 1:4, the compression strut can be seen. The stress propagates from the loading plate diagonally to the support. While zero stress apparently occurs at the area outside the diagonal strut. At the ultimate condition, the crack occurs at the upper surface outside the strut because the walking column displaces downward, and the upper surface is in tension due to bending.

For the squat case with an aspect ratio of 1:1, it can be learned that the compressive stress flows diagonally and forms a strut. Diagonal cracks start to appear within the compressive strut. While cracks also appear at the upper surface of the walking column due to bending as it is moving downward due to the applied displacement.



(c)

Figure 7. Finite element stress contour at ultimate condition for aspect ratio of (a) 1:9, (b) 1:4, (c) 1:1

5.2 Proposed Design Method

The axial capacity obtained from FEA is then compared to the axial capacity from STM as it is presented in Table 3 and Figure 8. The axial capacity of STM is defined as the lowest capacity of the concrete strut, nodal zone, and tensile tie, which would govern the failure mode. The axial strength of FEA for the slender walking column is much higher than the STM strength. Thus, the STM design result is very conservative for the slender case. It can be seen in Table 4 that for the slender case, the strut failure governs the failure mechanism. Thus, tie reinforcement is not necessary for the slender walking column. The stress propagation confirms at section 4 that the entire walking column is under uniform compressive stress for the slender case.

The aspect ratio of 1:4 shows quite different behavior. The failure mechanism is determined by tie failure. However, the axial capacity obtained from FEA increases to 1694.3 kips from 1362.5 kips for the slender case. The squat walking column with an aspect ratio of 1:1 also shows the same behavior. The failure mechanism is also governed by tie failure. However, the axial capacity from FEA decreases to 789.5 kips which are close to the axial capacity of the STM, 783 kips.

The FEA results are supposed to show the actual behavior of the walking columns. The FEA axial capacity shows an increasing trend as the aspect ratio increase from 1:9 to 1:4 and then decrease as it becomes squat with an aspect ratio of 1:1. At the same time, the STM results indicate that the axial strength is relatively constant in around 900 kips as the aspect ratio increase from 1:9 to 1:4 and then decrease as the aspect ratio goes up to 1:1, which is the same trend as the FEA.

Hence, the conclusion is that STM is a conservative method to design walking columns for all aspect ratios. However, it cannot accurately predict the slender to medium walking column behavior. The STM shows a similar behavior and axial capacity compared to FEA for the squat walking columns. The failure behavior of the slender to the medium walking column is not governed by diagonal compressive strut as it is observed using STM as it shows a different trend to the FEA results.

Table 3. The comparison of the axial capacity of walking columns obtained using FEA and STM

Aspect Ratio	Finite Element Analysis	ACI 318-19 Strut-and-tie
1:9	1362.5	981.9
1:4	1694.3	925.0
1:1	789.5	783.0

Table 4. Axial capacity and failure mode obtained from STM

Aspect Ratio	Axial Capacity (kips)		
	Strut	Nodal Zone	Tie
1:9	982.1	1044.5	4074.4
1:4	1010.3	1044.5	925.0
1:1	851.8	1047.8	783.0

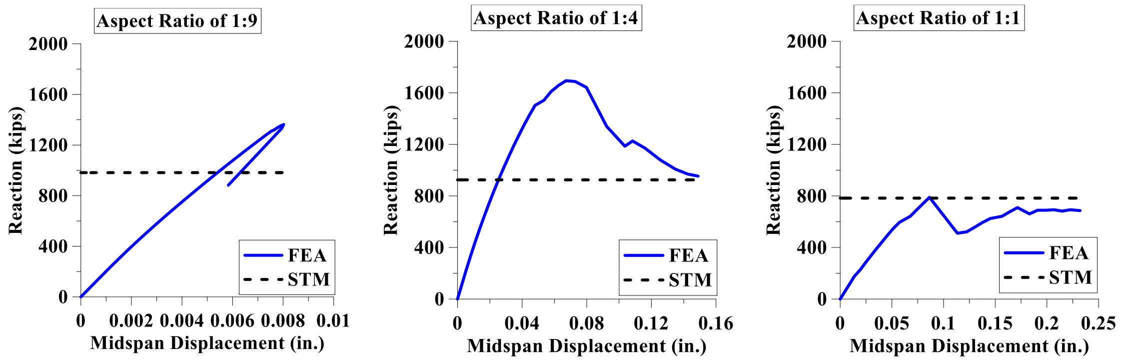


Figure 8. Comparison of FEA pushover curve to STM axial capacity

The slender walking column does not undergo sizeable downward displacement since it does not have a long free span. The brittle failure occurs on the slender aspect ratio, the reaction keeps increasing linearly until the peak strength and then suddenly fails. It indicates that the tie reinforcement does not pay a significant contribution to the strength and ductility of the slender walking column. However, for medium and squat walking columns, the strength decreases after a peak but still provides some strength. This is because the tie reinforcement still takes the load after the concrete crushing. This indicates that the tie determines the failure mechanism for medium and squat walking columns which satisfies the findings in Table 3.

6. Conclusions

The walking column behavior was observed using finite element analysis (FEA) in this research. Strut-and-tie method (STM) is used to design the reinforcement based on ACI 318-19. Then a comparison can be ruled out between the axial strength obtained using STM to the actual axial strength from the FEA. From this research, several conclusions can be drawn:

1. STM can be used as a safe and conservative design method for walking column with aspect ratio ranging from 1:9 to 1:1. The axial strength obtained from STM is lower than finite element strength. Therefore, it can be conservatively used as design method for walking columns. The axial strength from STM converges on the FEA axial strength for squat aspect ratio as the aspect ratio increases to 1:1.
2. The strut failure governs the failure of walking column with aspect ratio 1:9. While for aspect ratios of 1:4 and 1:1, tie failure determines the failure mode. This is because as the aspect ratio decreases the angle of strut is getting smaller, the tie force then is increasing cause the tie reinforcements to fail first.

7. References

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