Engineering Structures 136 (2017) 245–260

Contents lists available at ScienceDirect

Engineering Structures

journal homepage: www.elsevier.com/locate/engstruct

In-plane flexural strength of unbonded post-tensioned concrete masonry walls

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ARTICLE INFO

Article history:
Received 7 January 2016
Revised 3 January 2017
Accepted 5 January 2017

Keywords:
Masonry
MSJC2013
Shear strength
Flexural strength
Unbonded post-tensioning bars
In-plane behavior, post-tensioned wall

ABSTRACT

The target of this paper is to develop a design equation to predict the in-plane flexural strength of unbonded post-tensioned masonry walls (PT-MWs). Using validated finite element models, a parametric study was performed to investigate the effect of different parameters on the wall rotation and compression zone length. Multivariate regression analysis was performed to develop an equation to estimate the rotation of the unbonded PT-MWs at peak strength. Using the drift capacity of the walls and the proposed equation, a design expression and a related step-by-step design method have been developed to estimate the flexural strength of unbonded PT-MWs. The accuracy of the procedure was examined using experimental and finite element model results. Ignoring the elongation of the PT bars in the strength prediction resulted in an underestimation of about 40%, while using the proposed approach the prediction improved significantly. It was found that the wall length and axial stress ratio are the most influential factors contributing to the rotation and compression zone length of unbonded PT-MWs. According to the results presented in this study it is recommended to limit the axial stress ratio to a value of 0.15. In addition, limiting the maximum spacing between PT bars to a distance of six times the wall thickness is recommended to prevent local shear failure and vertical splitting cracking.

1. Introduction

Incorporating post-tensioning into masonry offers a simple and potentially cost-effective structural system. The post-tensioning techniques can be applied to different types of masonry members as either bonded or unbonded reinforcement [1,2]. Post-tensioned (PT) walls can be ungrouted, partially grouted, or fully-grouted. In unbonded PT masonry walls, where grout is used in the cells containing the post-tensioning (PT) bars, the PT bar is enclosed in a PVC tube and hence is not embedded in or bonded to the grout. An unbonded PT bar is designed to provide a restoring force to return the wall to its original vertical alignment, therefore reducing residual drifts after a seismic event. The reduced residual drifts in unbonded post-tensioned systems result in significant reductions in repair costs and downtime, and also allow the structure to remain operational following an earthquake. The system can be used for new construction as well as repair for existing structures.

For members having unbonded PT bars, the theoretical evaluation of the stress developed in the PT bars is challenging, as it depends on the elongation of the unbonded PT bar, which in turn depends on the structural element rotations. To predict the behavior of unbonded walls, empirical expressions based on finite element analyses of unbonded post-tensioned masonry walls (PT-MWs) under out-of-plane and in-plane loads, were proposed [1,2]. However, the developed finite element models were not able to predict the post-peak performance of the investigated walls. Moreover, the expressions are not able to accurately predict the strength of PT-MWs having large spacing between tendons [3].

In this study, well-calibrated finite element models are used to predict the rotation of the unbonded PT-MWs at peak strength. A parametric study of theoretical walls analyzed using the finite element model was carried out in two stages. In the first stage (set I), the effects of total applied axial stress, initial stress ratio of the PT bars, height of the walls, length of the walls and spacing between PT bars were evaluated on 25 walls. During stage-two (set II), analyses of 45 additional walls were carried out to determine a relationship between the wall rotation and the compression zone length at peak strength, and other parameters including the wall configuration and the level of applied axial stress. An expression was developed based on multivariate regression analysis to predict the rotation and compression zone length at peak strength, which was integrated into a proposed equation to predict the strength of

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http://dx.doi.org/10.1016/j.engstruct.2017.01.016
0141-0296/

unbonded PT-MWs. The predicted flexural strength using the proposed design equation and the approach in which the elongation of PT bars is ignored were then compared with the values obtained from the experimental results available in the literature.

2. Stress in unbonded PT bars

When an unbonded cantilever wall with a dominant flexural deformation is subjected to lateral load (Fig. 1(a)), the normal stresses on wall's heels reduce gradually with increasing applied lateral load. If the tensile strength of masonry is ignored, the wall's heel starts to open when the normal stress in the heel reaches zero i.e. at the decompression point. Decompression at the wall-footing interface occurs when the axial stress due to post-tensioning and gravity loads is reduced to zero at the edge of the wall base by the overturning moment resulting from the lateral loads. The PT bars display an increase in their initial post-tensioning stresses when the wall-footing interface joint opens. The elongation in PT bar i, is:

$$\Delta_i = (\theta_m - \theta_0)(d_i - c) \tag{1}$$

where θ_m is the wall rotation at peak strength and θ_0 is the rotation corresponding to the decompression point, c is the compression zone length (neutral axis depth), and d_i is the distance from the extreme compression fiber to the ith PT bar. Before the decompression point, the PT force remains constant, hence:

$$f_{psi} = f_{se\,i} + (\theta_m - \theta_0)\frac{E_{ps}}{L_{ps}}(d_i - c) \leqslant f_{py} \tag{2}$$

where L_{ps} is the unbonded length of the PT bar, E_{ps} is the Young's modulus of the PT bar, f_{py} is the yield stress of the PT bars and $f_{se\,i}$ is the effective stress in the ith PT bar after stress losses. (All the short term losses including anchorage slip, friction and elastic shortening, and long term losses including creep, moisture movement and steel relaxation were supposed to be considered in calculation of $f_{se\,i}$.)

Considering a linear stress-strain relationship in the masonry, assuming plane sections remain plane and ignoring the elongation of the PT bars before the decompression point, according to Fig. 1 (b), the absolute maximum masonry compressive strain corresponding to the decompression point, ε_0, is:

$$\varepsilon_0 = \frac{2\left(\sum f_{se\,i}A_{ps\,i} + N\right)}{L_w t_w E_m} \tag{3}$$

where A_{ps} is the area of the PT bar(s), E_m is the elastic modulus of concrete masonry = 900 f'_m [4] (f'_m: the compressive strength of masonry), t_w and L_w is the thickness and length of the wall, and N is the gravity load. The axial stress, f_m, is defined as:

$$f_m = \frac{\sum f_{se\,i}A_{ps\,i} + N}{L_w t_w} \tag{4}$$

It has been recommended to limit f_m/f'_m to 0.15 [5,6]. Considering a maximum value of 0.15 for f_m/f'_m, will mean that the stress in the masonry will approach the decompression point at 0.3 f'_m. Therefore, considering a linear stress-strain relationship in the masonry at the decompression point is a reasonable assumption.

The lateral displacement at the top of the wall corresponding to the decompression state, Δ_0, is

$$\Delta_0 = \frac{\varphi_0 h_w^2}{3} \tag{5}$$

where φ_0 is the maximum value of the curvature at the decompression point = ε_0/L_w, also $\theta_0 = \Delta_0/h_w$, hence, for concrete masonry (blocks):

$$\theta_0 = \left(\frac{1}{1350}\right)\frac{f_m}{f'_m}\frac{h_w}{L_w} \tag{6}$$

Using equilibrium, the compression zone length, c, can be expressed as:

$$c = \frac{\sum f_{se\,i}A_{ps\,i} + N}{\alpha\beta f'_m t_w} \tag{7}$$

where α and β are the stress block parameters which are provided by different building codes (e.g. in MSJC 2013: $\alpha = \beta = 0.8$).

In order to determine the stress in the PT bar using Eq. (2), the values of rotation at peak strength, θ_m, and compression zone length at peak strength, c, need to be determined. In this study finite element models were used to determine the relationship between c and θ_m and other parameters of the wall.

3. Finite element model

Ryu et al. [3] developed finite element models for unbonded PT-MWs. However, the model was calibrated on the seismic responses of cavity walls built using clay units. In the current work the model was calibrated and validated against single-leaf PT-MWs built using concrete masonry units (CMUs). In this study, three dimensional smeared-crack models of masonry walls were developed using LS-DYNA software, which is a general purpose finite element code. As the global behavior of the fully grouted walls was of prime importance for this study, a smeared-crack approach was applied and concrete masonry was modelled as a homogenous isotropic material using a nonlinear material model.

An eight-node brick solid element was used to model the masonry components. This element includes a smeared crack for crushing in compression and cracking in tension. The material model assigned to the masonry elements[1] is capable of plastic deformation, cracking in three orthogonal directions and crushing. The masonry constitutive properties are presented and discussed in the subsequent section. The concrete material for the footing and bond beam was modelled as a linear-elastic material. To simulate the PT bars, two-node beam elements with 2 × 2 Gauss quadrature and truss formulation were used.

A bi-linear material model with linear kinematic hardening was assigned to PT bar beam elements.[2] The material properties of the PT bar considered in this study were: Young's modulus of 190 GPa, tangent modulus of 2.5 GPa, Poisson's ratio of 0.3 and tensile yield strength of 970 MPa.

Contact elements were used to model the interface between the wall and footing as well as between the wall and bond beam.[3] The default contact parameters of LS-DYNA were adopted; however, as is explained below, the soft constraint option was modified to improve the reliability of the model to account for the relative stiffness of the contact elements. The assigned contact between wall and footing was capable of simulating interaction between contact interfaces of the two discrete components, which is a significant factor in capturing rocking and sliding behavior. For the stiffness of the surface to surface contact interface, a penalty-based approach was used in which the size of the contact segment and its material properties are used to determine the contact spring stiffness. This method was considered as the material stiffness parameters between the contacting surfaces were in the same range. Node-to-surface contact elements[4] were also incorporated to model the interface between the PT bars and elements of the footing and bond beam. This contact element prevents the PT bars from penetrating into the solid

[1] MAT_CONCRETE_DAMAGE_REL3.
[2] MAT_PLASTIC_KINEMATIC.
[3] AUTOMATIC_SURFACE_TO_SURFACE.
[4] AUTOMATIC_NODE_TO_SURFACE.