Non-dimensional design charts for unbonded, post-tensioned, split precast concrete walls

Elias I. Saqan and Rami A. Hawileh

Editor's quick points

- The authors investigate design procedures for the hybrid wall system proposed by the Precast Seismic Structural Systems (PRESSS) research program.
- A research goal was to develop design charts for the hybrid wall system that would eliminate the need for lengthy calculations.
- A design example is presented that compares the PRESSS (dimensional) and developed (nondimensional) design-chart procedures.

The Precast Seismic Structural Systems (PRESSS) research proposed five different seismic structural systems made from precast concrete elements.[1,2] These systems formed various parts of the structural framing in the PRESSS phase III building that was tested between June and September 1999 at the University of California at San Diego.[2,3]

Based on the PRESSS evaluation, one of the structural systems that has the potential to eliminate residual drift after an earthquake was the unbonded, post-tensioned, split precast concrete wall system (hereafter called the hybrid wall system). The PRESSS report[1] outlines a dimensional design procedure for the hybrid walls. This procedure is iterative, requiring lengthy calculations to achieve an optimum design. This study developed a set of new non-dimensional parameters and procedures for the design of such walls. The goal of this research was to develop a set of nondimensional design charts that require no iterations. Such charts are based on an optimum design of zero residual drift while the moment capacity of the wall is equal to the applied design moment.

Hybrid walls: concept description

The hybrid wall system contains two or more precast concrete wall panels that are connected to the foundation by unbonded post-tensioning reinforcement. Wall panels are connected with special shear connectors. The PRESSS report[1] recommends using U-shaped flexural plates (UFPs) as shear connectors because they performed well in the PRESSS phase III test building. Other comparable connector systems may be used as well. Both the unbonded post-tensioning reinforcement and the shear connectors contribute to the overall moment resistance.

For a given total moment strength, a higher proportion of shear connector areas would lead to more apparent damping, lower peak drift, and larger residual drift. An interesting feature of this hybrid wall system is the combination of shear connectors and post-tensioning reinforcement. The shear connectors dissipate energy by yielding, while the post-tensioning force along with the weight of the panel and gravity loads supported by the wall restore the wall to its original vertical position after an earthquake.

The general design philosophy of the hybrid wall system, as presented by Stanton and Nakaki,[1] is to minimize peak drift during a seismic event while maintaining zero residual drift. **Figure 1** shows the relationship between peak and residual drifts. This design process involves finding the optimum ratio of moment resisted by the shear connectors (dampers) to the total moment resisted by the wall.

The current design procedure for the hybrid walls is based on dimensional parameters.[1] Because the post-tensioning tendons are unbonded to the concrete, there is no strain compatibility in the system. Thus, the design procedure is an iterative process and becomes lengthy when seeking an optimal design.

Therefore, the current design procedure is not likely to be performed by hand. To reduce the overall design process, a design procedure that incorporates simplified hand calculations is essential. Such a design procedure can be implemented in a parametric study utilizing nondimensional parameters, which is the subject of this paper.

Objectives

Hawileh et al.[4] previously developed nondimensional design procedures for other precast, prestressed concrete hybrid frames. This article proposes similar tasks based on nondimensional procedures for the design of hybrid walls. The major objectives of this study were the following:

- Develop a set of dimensionless parameters and equations to replace the current dimensional equations in the calculation procedure.
- Perform parametric studies based on the nondimensional formulation of the PRESSS design procedures. These studies involve optimizing a large number of design conditions to minimize overall drift while maintaining zero residual drift.
- Generate sample nondimensional design charts for the design of hybrid walls. The charts will enable designers to perform the design in fewer steps and visually show how changing one or more parameters would affect the design.
- Prepare a design example that includes comparisons of PRESSS (dimensional) and the developed nondimensional design chart procedures.

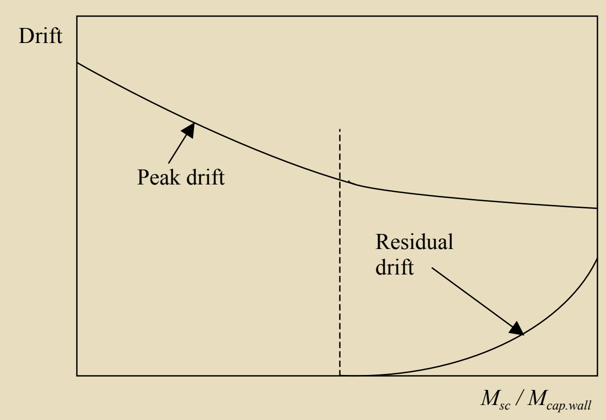

Figure 1. This graph plots the drift versus relative strength of the resisting elements. Source: Stanton and Nakaki 2002. Note: $M_{cap,wall}$ = total moment capacity of wall; M_{sc} = total moment capacity of all U-shaped flexural plate connectors.

Hybrid walls: PRESSS design procedure

The PRESSS report[1] proposes design equations based on the following assumptions:

- Design forces and drift limits are known. Forces may be obtained either by force-based design (FBD) or by displacement-based design (DBD).[5] Interface rotations are obtained from the drift ratio, using the geometry of the system.
- The overall dimensions of the wall are known from preliminary calculations.
- All wall panels have the same size and constant thickness.
- At each panel, the centroid of the post-tensioning tendon is located at the middle of the wall panel.
- The post-tensioning tendon remains within the elastic range at the design drift.
- Shear connectors are treated as rigid plastic.
- Material properties are known.

The PRESSS design equations also use deformation

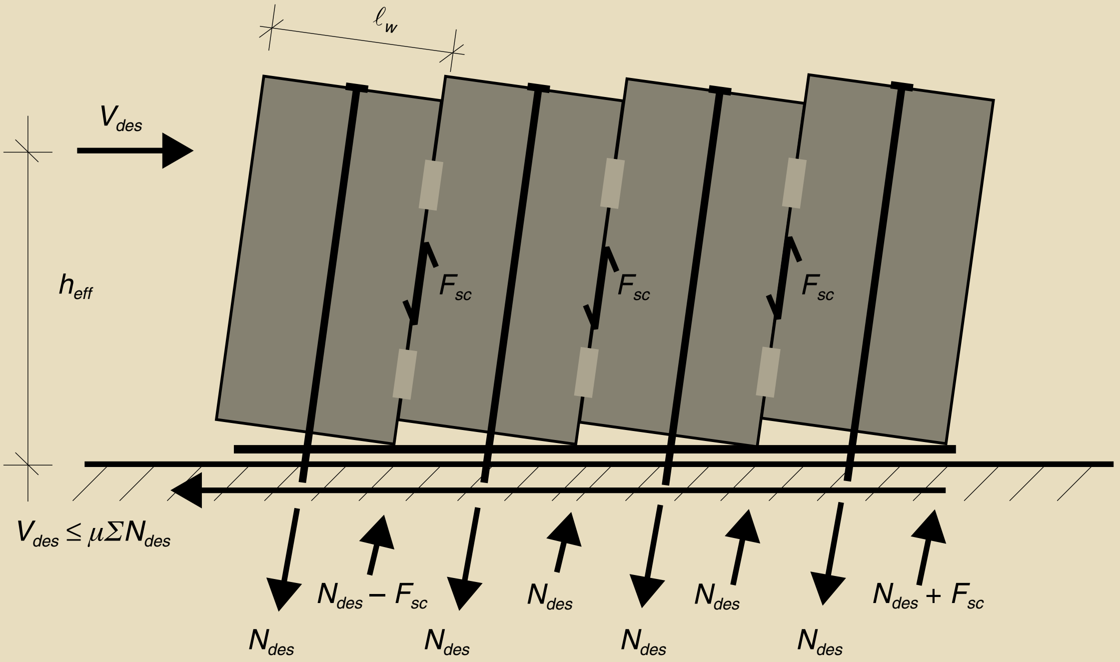

Figure 2. This figure shows the panels and their deformed shapes at design drift. Source: Stanton and Nakaki 2002. Note: F_{sc} = total yield force of all shear connectors in one vertical joint; h_{eff} = height above foundation of lateral load resultant on wall; ℓ_w = horizontal length of one wall panel; N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift; V_{des} = design base shear; μ = coefficient of friction against concrete.

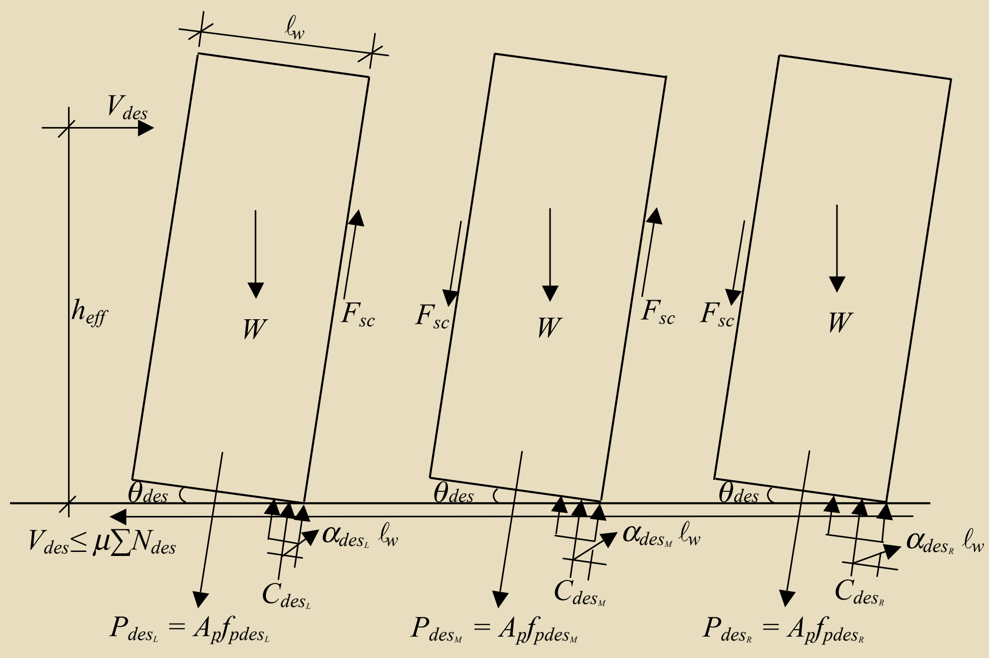

Figure 3. This figure displays the loads and forces acting on the seismic system at design drift. Note: A_p = area of post-tensioning tendons; C_{des_L} = compressive reaction on left wall panel at design drift; C_{des_M} = compressive reaction on interior wall panel at design drift; C_{des_R} = compressive reaction on right wall panel at design drift; f_{p,des_L} = stress in post-tensioning reinforcement of left wall panel at the design drift; f_{p,des_M} = stress in post-tensioning reinforcement of an interior wall panel at the design drift; f_{p,des_R} = stress in post-tensioning reinforcement of right wall panel at the design drift; F_{sc} = total yield force of all shear connectors in one vertical joint; h_{eff} = height above foundation of lateral load resultant on wall; l_w = horizontal length of one wall panel; N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift; P_{des_L} = force in the post-tensioning reinforcement of left wall panel at the design drift; P_{des_M} = force in the post-tensioning reinforcement of an interior wall panel at the design drift; P_{des_R} = force in the post-tensioning reinforcement of right wall panel at the design drift; V_{des} = design base shear; W = total gravity load from all floors on one wall panel including panel self-weight; α_{des_L} = distance from center of compressive reaction to edge of left wall panel divided by wall depth, at design drift; α_{des_M} = distance from center of compressive reaction to edge of interior wall panel divided by wall depth, at design drift; α_{des_R} = distance from center of compressive reaction to edge of right wall panel divided by wall depth, at design drift; θ_{des} = interface rotation at design limit state; μ = coefficient of friction against concrete.

compatibility and equilibrium to calculate the forces and moments. During a seismic event, the panels are assumed to move as rigid bodies and rock about their individual bases as shown in **Fig. 2** and **3**. The PRESSS procedure requires 11 steps to design the post-tensioning reinforcement and the UFP shear connectors. This procedure is iterative and often lengthy, utilizing numerous dimensional design parameters.

Nondimensional formulation

A simplified design procedure is instrumental in promoting and using this beneficial concept for precast concrete, seismic-resistant construction. Nondimensional parameters were varied in an extensive parametric study involving an optimization process. The objective of the optimization process was to achieve zero residual drift while keeping the moment capacity of the wall equal to the applied design moment. The resulting nondimensional parameters and equations helped generate user-friendly design charts, applicable to most cases.

Before deriving the nondimensional design equations, some necessary dimensional design equations will be presented to clarify the design procedure. These dimensional equations are essentially the design equations presented by the PRESSS report.[1] The whole design procedure, along with the complete dimensional design equations and their derivations, are also presented by Stanton and Nakaki.[1]

Referring to Fig. 3, at the design drift level, the force resisted by the concrete in any wall panel C_{des} can be expressed as follows.

$$C_{des} = P_{des} + W + F_{sc_L} - F_{sc_R}$$

where

C_{des} = compression reaction on one wall panel at design drift

P_{des} = force in the prestressing tendon at design drift

W = total gravity load from all floors on one wall panel, including panel self-weight

F_{sc_L} = total yield force of all shear connectors in joint to left of panel

F_{sc_R} = total yield force of all shear connectors in joint to right of panel

The forces in the shear connectors F_{sc} at each end of a wall panel are equal. Although a different capacity for the shear connectors can be used at each end, they are assumed to be equal to simplify the calculations.

Therefore, for an interior panel

$$F_{sc_L} - F_{sc_R} = 0$$

The relative position of the compressive force as a fraction of wall-panel length α_{des} can be found from the equilibrium of forces as follows.

$$C_{des} = k_1 f_g' \left(2\alpha_{des} \ell_w\right) t_w \tag{1}$$

where

k_1 = uniform stress in Whitney rectangular equivalent stress block divided by f_g'

f_g' = the smaller of the compressive strengths of grout bed and wall concrete

ℓ_w = horizontal length of one wall panel

t_w = thickness of wall panel

$$\alpha_{des} = \frac{0.5C_{des}}{k_1 f_g' \ell_w t_w} = \frac{0.5\left(P_{des} + W + F_{sc_L} - F_{sc_R}\right)}{k_1 f_g' \ell_w t_w} \tag{2}$$

The moment capacity of each wall panel $M_{cap,panel}$ is

$$M_{cap,panel} = \ell_w \left(C_{des}\left(0.5 - \alpha_{des}\right) + 0.5\left(F_{sc_L} + F_{sc_R}\right)\right) \tag{3}$$

The total moment capacity resisted by all panels is

$$M_{cap,wall} = \Sigma M_{cap,panal} \tag{4}$$

In general, the wall can be divided into n panels. Each wall panel has its own α_{des}, C_{des}, and $M_{cap,panel}$. All interior panels have the same α_{des} and C_{des} and, thus, the same moment capacity. Therefore, there are only three cases to be considered when obtaining α_{des}, C_{des}, and $M_{cap,panel}$. These cases are: the left panel, the middle or interior panels, and the right panel. The calculations of α_{des}, C_{des}, and $M_{cap,panel}$ for the three cases are presented in the following sections.

Left exterior panel

Referring to Fig. 2 and 3, the following nondimensional parameters are defined

$$\phi_1 = \frac{h_w}{\ell_w} \tag{5}$$

where

h_w = total height of wall panel

$$\phi_2 = \frac{h_{eff}}{\ell_w} \tag{6}$$

where

$h_{eff} = \frac{M_{des}}{V_{des}}$ = height above foundation of lateral load resultant on wall

M_{des} = moment demand at design drift

V_{des} = design base shear

$$\rho_1 = \frac{A_p}{\ell_w t_w} \tag{7}$$

$$\rho_2 = \frac{F_{sc}}{\ell_w t_w f_g'} \tag{8}$$

where

A_p = area of post-tensioning tendon

F_{sc} = total yield force of all shear connectors in one vertical joint

Note that f_g' is the smaller of the compressive strengths of the grout bed or the wall concrete. In general, the grout compressive strength is less, so the damage during an earthquake will primarily occur in the grout instead of the wall. This design objective is recommended because repairing the grout is typically easier than repairing the wall.

$$\Omega = \frac{h_w\left(\gamma_c + \gamma_L\right)}{f_g'} \tag{9}$$

where

γ_L = ratio of w_{floor} to $t_w h_w$

w_{floor} = distributed vertical load on the wall, at base, from all floors

γ_c = density of concrete

$$\psi_o = \frac{f_{po}}{f_g'} \tag{10}$$

$$\psi_L = \frac{f_{p,des_L}}{f_g'}$$

where

f_{po} = stress in post-tensioning tendon after losses at zero drift

$f_{p,des}$ = stress in post-tensioning tendon at the design drift

From Eq. (2)

$$\alpha_{des_L} = 0.5\frac{\left(A_p f_{p,des_L} + \ell_w t_w h_w\left(\gamma_c + \gamma_L\right) - F_{sc}\right)}{\ell_w t_w k_1 f_g'}$$

Substituting the corresponding nondimensional parameters previously defined, Eq. (2) becomes

$$\alpha_{des_L} = \frac{0.5}{k_1}\left(\rho_1\psi_L + \Omega - \rho_2\right) \tag{11}$$

The distance from member compression face to neutral axis divided by member depth at design drift is

$$\eta_{des_L} = 2\frac{\alpha_{des_L}}{\beta_1} = \frac{1}{k_1\beta_1}\left(\rho_1\psi_L + \Omega - \rho_2\right) \tag{12}$$

where

β_1 = depth of equivalent compressive stress block divided by the neutral axis depth

The elongation of the post-tensioning tendon Δ_{P_L} at the design drift level is determined using Eq. (13).

$$\Delta_{p_L} = \theta_{des}\ell_w\left(0.5 - \eta_{des_L}\right) \tag{13}$$

where

θ_{des} = interface rotation at design limit state shown in Fig. 3

Thus, the increase in stress in the post-tensioning reinforcement at the design drift is

$$\Delta f_{p_L} = E_p\frac{\Delta_{p_L}}{h_w} = E_p\left(\frac{\theta_{des}\ell_w}{h_w}\right)\left(0.5 - \eta_{des_L}\right) \tag{14}$$

where

E_p = post-tensioning strand modulus of elasticity

Substituting Eq. (12) and the corresponding nondimensional parameters previously defined, Eq. (14) becomes

$$\Delta f_{p_L} = E_p \frac{\theta_{des}}{\phi_1}\left[0.5 - \left(\frac{\rho_1 \psi_L + \Omega - \rho_2}{k_1 \beta_1}\right)\right]$$

The total stress in the post-tensioning reinforcement at the design drift level is determined using Eq. (15)

$$f_{p,des_L} = (f_{p_o} + \Delta f_{p_L}) \quad (15)$$

and must be less than the yield stress f_{py} if the system is to remain elastic.

Substituting Eq. (14) and the corresponding nondimensional parameters previously defined, Eq. (14) becomes

$$\begin{aligned}\Delta f_{p_L} &= f_{pdes_L} - f_{po} = \psi_L f'_g - f_{po} \\ &= E_p \frac{\theta_{des}}{\phi_1}\left[0.5 - \left(\frac{\rho_1 \psi_L + \Omega - \rho_2}{k_1 \beta_1}\right)\right]\end{aligned}$$

Furthermore, with algebraic manipulation of the previous equation the nondimensional parameter Ψ_L becomes Eq. (16).

$$\psi_L = \frac{f_{po} + \dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\left(0.5 k_1 \beta_1 - \Omega + \rho_2\right)}{f'_g + \left(\dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\right)\rho_1} \quad (16)$$

Substituting Eq. (11) into Eq. (1), the equation for the compressive reaction in the left wall panel at design drift becomes Eq. (17).

$$C_{des_L} = \ell_w t_w f'_g \left(\rho_1 \psi_L + \Omega - \rho_2\right) \quad (17)$$

Substituting Eq. (11) and (17) into Eq. (3), the moment capacity of a left exterior panel can be written as Eq. (18).

$$M_{cap_L} = \frac{\ell_w^2 t_w f'_g}{2}\left[\rho_1 \psi_L + \Omega - \frac{1}{k_1}\left(\rho_1 \psi_L + \Omega - \rho_2\right)^2\right] \quad (18)$$

Right exterior panel

Using the same procedure, the following equations are derived for a right exterior panel.

$$\alpha_{des_R} = \frac{0.5}{k_1}\left(\rho_1 \psi_R + \Omega + \rho_2\right)$$

$$\psi_R = \frac{f_{po} + \dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\left(0.5 k_1 \beta_1 - \Omega - \rho_2\right)}{f'_g + \dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\left(\rho_1\right)} \quad (19)$$

$$C_{des_R} = \ell_w t_w f'_g \left(\rho_1 \psi_R + \Omega + \rho_2\right)$$

$$M_{cap_R} = \frac{\ell_w^2 t_w f'_g}{2}\left[\rho_1 \psi_R + \Omega + 2\rho_2 - \frac{1}{k_1}\left(\rho_1 \psi_R + \Omega + \rho_2\right)^2\right] \quad (20)$$

Interior panel

All interior panels are identical. Using the same procedure for the exterior panels, the following equations can be derived for a typical interior panel.

$$\alpha_{des_M} = \frac{0.5}{k_1}\left(\rho_1 \psi_M + \Omega\right)$$

$$\psi_M = \frac{f_{po} + \dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\left(0.5 k_1 \beta_1 - \Omega\right)}{f'_g + \dfrac{E_p \theta_{des}}{k_1 \beta_1 \phi_1}\left(\rho_1\right)}$$

$$C_{des_M} = \ell_w t_w f'_g \left(\rho_1 \psi_M + \Omega\right)$$

$$M_{cap_M} = \frac{\ell_w^2 t_w f'_g}{2}\left[\rho_1 \psi_M + \Omega + 2\rho_2 - \frac{1}{k_1}\left(\rho_1 \psi_M + \Omega\right)^2\right] \quad (21)$$

The total wall

A wall can be divided into n equal panels. The minimum number of panels is two. From Eq. (4), the total moment capacity of a split wall $M_{cap,wall}$ is:

$$M_{cap,wall} = M_{cap_L} + \left(n-2\right) M_{cap_M} + M_{cap_R} \quad (22)$$

Substituting Eq. (18), (20), and (21) into Eq. (22)

$$M_{cap,wall} = \frac{\ell_w^2 t_w f_g'}{2}\left(\begin{array}{l}\rho_1\left[\psi_L + (n-2)\psi_M + \psi_R\right] + n\Omega + 2\rho_2(n-1) \\ -\frac{1}{k_1}\left\{\rho_1^2\left[\psi_L^2 + (n-2)\psi_M^2 + \psi_R^2\right] + 2\rho_1\Omega\right. \\ \left.\left[\psi_L + (n-2)\psi_M + \psi_R\right] - 2\rho_1\rho_2(\psi_L - \psi_R) + n\Omega^2 + 2\rho_2^2\right\}\end{array}\right) \tag{23}$$

The moment capacity of the wall $M_{cap,wall}$ must be equal to the design moment M_{des}. It is clear from Eq. (23) that a portion of $M_{cap,wall}$ is resisted by the shear connectors M_{sc}.

$M_{sc} = \omega M_{des}$

where

ω = ratio of the moment resisted by the shear connectors to the total design moment

Therefore,

$$M_{cap,wall} = M_{des} = \frac{M_{sc}}{\omega} \tag{24}$$

From Fig. 3, the total moment resisted by the shear connectors is

$$\begin{aligned} M_{sc} &= (n-1)F_{sc}\ell_w \\ &= (n-1)\rho_2 \ell_w t_w f_g' \ell_w \\ &= (n-1)\rho_2 \ell_w^2 t_w f_g' \end{aligned} \tag{25}$$

Substituting Eq. (23) and (25) into Eq. (24), the following nondimensional equation can be obtained.

$$\begin{aligned} &\rho_1\left[\psi_L + (n-2)\psi_M + \psi_R\right] + n\Omega + 2(n-1)\rho_2 \\ &- \frac{1}{k_1}\left\{\rho_1^2\left[\psi_L^2 + (n-2)\psi_M^2 + \psi_R^2\right]\right. \\ &+ 2\rho_1\Omega\left[\psi_L + (n-2)\psi_M + \psi_R\right] \\ &\left.- 2\rho_1\rho_2(\psi_L - \psi_R) + n\Omega^2 + 2\rho_2^2\right\} \\ &= \frac{2(n-1)\rho_2}{\omega} \end{aligned} \tag{26}$$

Equation (26) is a nondimensional design equation that depends on the geometry of the wall; material properties; level of prestress; area of the tendon; capacity of the shear connectors; self-weight of the wall; and any superimposed gravity load, design loads, and drifts. Equation (26) must be satisfied if the wall moment capacity is equal to the design moment. An optimum design for a particular wall and loads is achieved when ω is chosen such that Eq. (26) is satisfied and the residual drift is zero, along with other constraints that are discussed in the following sections.

Acceptance criteria

An optimum design for the wall must meet the following criteria. The equations are taken from the PRESSS report[1] equations, but written in nondimensional form.

Yielding of post-tensioning reinforcement The post-tensioning reinforcement should not yield if the wall is to re-center after an earthquake. This is achieved by satisfying the following criteria.

$$\frac{\psi_L f_g'}{\alpha f_{py}} \le 1 \tag{27}$$

$$\frac{\psi_M f_g'}{\alpha f_{py}} \le 1 \tag{28}$$

$$\frac{\psi_R f_g'}{\alpha f_{py}} \le 1 \tag{29}$$

Following the PRESSS procedures, the α parameter was assumed to be equal to 1.0. However, a smaller value for α may be warranted pending further studies.

Uplift of end panel To ensure that the end wall panel does endure an uplift condition, the ratio of uplift force to hold-down force κ_o must be checked.

$$\begin{aligned} \kappa_o &= \frac{F_{sc}}{N_o} = \frac{F_{sc}}{A_p f_{po} + W} \\ &= \frac{F_{sc}}{\ell_w t_w \rho_1 \psi_o f_g' + \ell_w t_w h_w(\gamma_c + \gamma_L)} \\ &= \frac{F_{sc}}{\ell_w t_w f_g'\left[\rho_1\psi_o + \frac{h_w(\gamma_c + \gamma_L)}{f_g'}\right]} \end{aligned}$$

where

N_o = total axial force on one wall panel from gravity plus post-tensioning at zero drift

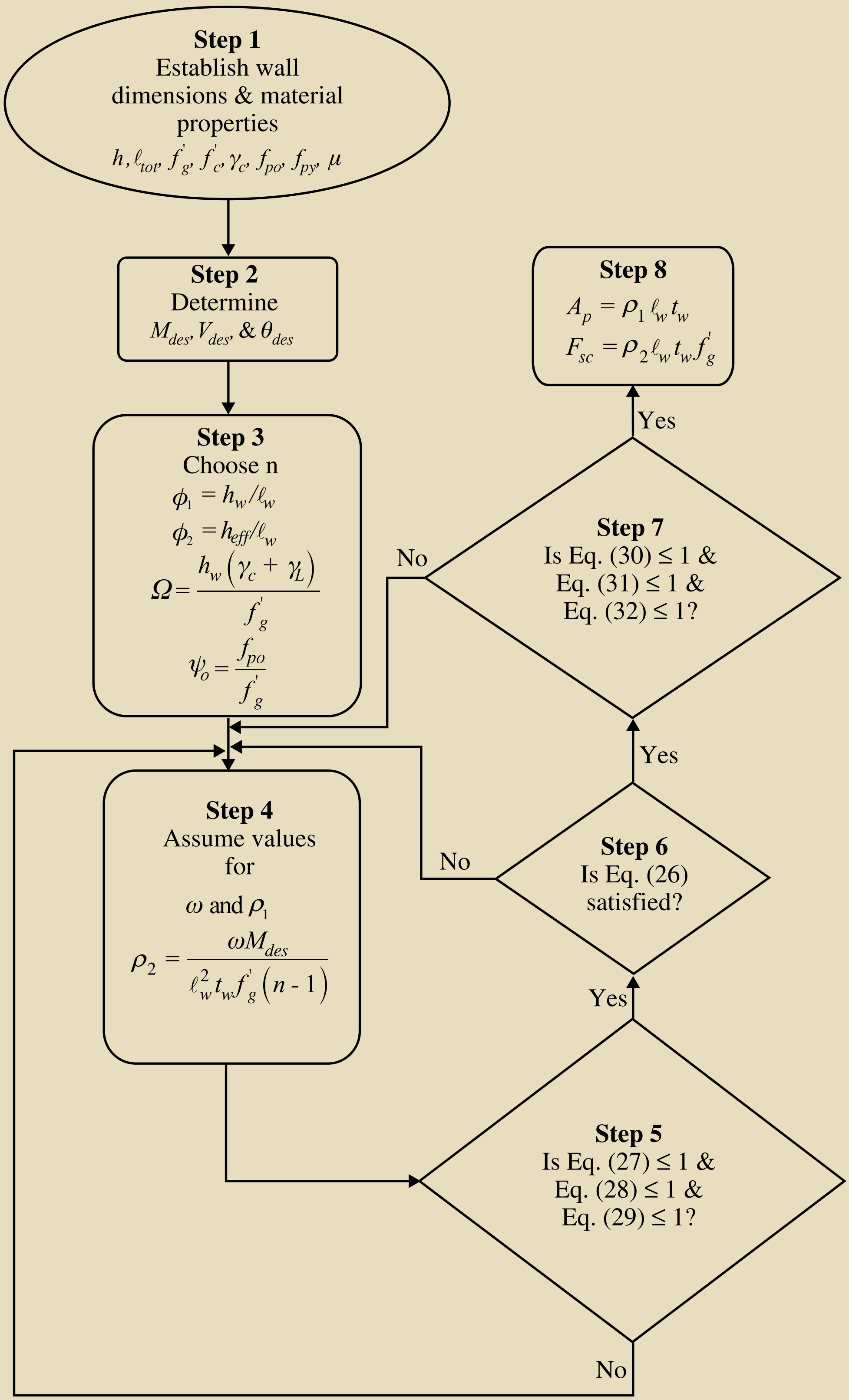

Figure 4. Unbonded post-tensioned split precast concrete walls can be designed nondimensionally with the use of this flowchart. Note: A_p = area of post-tensioning tendons; f'_c = concrete design compressive strength; f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{p0} = stress in post-tensioning reinforcement after losses at zero drift; f_{py} = specified yield stress of post-tensioning reinforcement; F_{sc} = total yield force of all shear connectors in one vertical joint; h_{eff} = height above foundation of lateral load resultant on wall; h_w = total height of wall panel; ℓ_w = horizontal length of one wall panel; $\ell_{w,tot}$ = total length of the wall; M_{des} = moment demand at design drift; n = number of wall panels per wall; t_w = thickness of wall panel; V_{des} = design base shear; γ_c = density of concrete; γ_L = ratio of w_{floor} to $t_w h_w$; μ = coefficient of friction against concrete; ϕ_1 = ratio of h_w to ℓ_w; ϕ_2 = ratio of h_{eff} to ℓ_w; ψ_o = ratio of f_{po} to f'_g; ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state; ω = ratio of the moment resisted by the shear connectors to the total design moment.

$$\kappa_o = \frac{\rho_2}{\rho_1 \psi_o + \Omega} \le 1 \tag{30}$$

Residual drift One of the major objectives of this design concept is to provide shear connectors between wall panels capable of reducing the peak drift during an earthquake and also allowing the wall to re-center after the earthquake. For that to happen, the restoring moment due to the post-tensioning force plus the wall self-weight and any imposed gravity loads must be greater than the moment corresponding to the shear connectors.

$$\rho_{RZD} = \frac{\kappa_o \left(n - 1 + 2\alpha_{o,avg}\kappa_o\right)}{n\left(0.5 - \alpha_{o,avg}\right)} \le 1 \tag{31}$$

where

$\alpha_{o,avg}$ = the average of α_o across all panels

α_o = relative position of the compressive force as a fraction of wall-panel length at zero drift

Each panel has its own α_o and can be found in a manner similar to that previously derived at the design drift level.

$$\alpha_{o_L} = \frac{0.5}{k_1}\left(\rho_1 \psi_o + \Omega - \rho_2\right)$$

$$\alpha_{o_R} = \frac{0.5}{k_1}\left(\rho_1 \psi_o + \Omega + \rho_2\right)$$

$$\alpha_{o_M} = \frac{0.5}{k_1}\left(\rho_1 \psi_o + \Omega\right)$$

$$\alpha_{o,avg} = \frac{0.5\left(\rho_1 \psi_o + \Omega\right)}{k_1} \le 1$$

Sliding versus rocking Satisfying the following force ratio ensures that the panel slides rather than rocks.

$$\rho_{Roc} = \frac{\kappa_o}{\mu \phi_2}\left[\left(0.5 - \alpha_{o,avg}\right) + \frac{\left(n - 1 - 2\alpha_{o,avg}\kappa_o\right)}{n}\right] \le 1 \tag{32}$$

Step-by-step nondimensional design procedure

A simplified design procedure was devised to eliminate lengthy and iterative calculations in the design of the hybrid wall system. The new step-by-step procedure is based on the nondimensional design equations derived previously. **Figure 4** summarizes the nondimensional design procedure in a flowchart.

It should be noted that while the following procedure with nondimensional equations is iterative, the noniterative nondimensional design charts are developed based on this procedure. The designer may choose to create a spreadsheet based on the procedure shown in the Fig. 4 flowchart, but the use of the design charts will allow the designer to avoid any iterative solution.

Step 1: Determine material properties and wall dimensions

Wall dimensions are assumed from a preliminary design or architectural considerations. These include the length of the wall $\ell_{w,tot}$, the height of the wall h_w, and the thickness of the wall t_w. Material properties are also known and include the following:

- Grout: strength f'_g, stress block coefficients β_1, coefficient of friction against concrete μ, and the grout stress block parameter k_1, which is usually taken as 0.85
- Concrete: design compressive strength f'_c and density γ_c
- Post-tensioning reinforcement: Young's modulus E_p and yield stress f_{py}
- Connectors: cyclic load-displacement relationship
- Prestress: design prestress level and effective prestress after all losses f_{po}

Step 2: Obtain M_{des}, V_{des}, and θ_{des}

Design loads can be obtained using either FBD or DBD methods.[5] Interface rotations θ_{des} can be calculated from the drift ratio using the geometry of the system.

Step 3: Determine nondimensional parameters related to wall geometry

Choose the number of panels n. Determine ϕ_1, ϕ_2, Ω, and Ψ_o from Eq. (5), (6), (9), and (10).

Table 1. Range of parameters used in the parametric studies.

Parameter	Minimum	Maximum
$\frac{h_w}{\ell_w}$	2.25	10
$\frac{A_p}{\ell_w t_w}$	0.001	0.0057
Ω	0.04	0.05
Number of panels n	2	3
f'_g, ksi	6	
f_{po}, ksi	175	
θ_{des}, %	2	

Note: A_p = area of post-tensioning tendons; f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; h_w = total height of wall panel; ℓ_w = horizontal length of one wall panel; n = number of wall panels per wall; t_w = thickness of wall panel; γ_c = density of concrete; γ_L = ratio of w_{floor} to $t_w h_w$; ϕ_1 = ratio of h_w to ℓ_w; ρ_1 = ratio of A_p to $\ell_w t_w$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state. 1 ksi = 6.89 MPa.

Step 4: Determine proportion of moment capacity contributed by shear connectors and post-tensioning reinforcement

Assume a value for ω, the ratio of the moment in the shear connectors to the design moment. A good initial assumption for ω is 0.45 to 0.5. Assume a value for ρ_1 and calculate the value for ρ_2 using Eq. (25).

Step 5: Check whether tendons yielded

Tendons must remain elastic if the wall is to re-center after an earthquake event. Therefore, Eq. (27) through Eq. (29) must be satisfied. If any of these equations are not satisfied, revise the values assumed in step 4.

Step 6: Assess moment strength of the wall at design drift level

If all tendons remain elastic at the design drift level, assess the moment strength of the wall using Eq. (26). If the moment capacity of the wall is unsatisfactory, either less than M_{des} or greatly above M_{des}, revise the values assumed in step 4.

Step 7: Check acceptance criteria

If the moment strength is satisfactory, check the acceptance criteria in Eq. (30) to (32). If any of these equations are not satisfied, revise the values assumed in step 4.

Step 8: Calculate tendon area and shear-connector capacity

Use Eq. (7) and (8) to determine the area of tendons and the capacity of the shear connectors.

Parametric study

The nondimensional parameters were varied in a parametric study involving an optimization process to illustrate the proposed simplified nondimensional design procedure. The objective of the optimization program was to ensure that the moment capacity of the section is equal to M_{des} (Eq. [26]) and to satisfy additional constraints (Eq. [27]–[32]) discussed in the previous section and summarized in Fig. 4. **Table 1** lists the range of parameters used in the parametric study. The range of parameters used in this study was limited to showing the application of the proposed design charts.

Microsoft Excel's Solver feature calculated the optimum solution for Eq. (26) by varying ω, ψ_L, and ψ_R (while ensuring that ρ_2 calculated using Eq. [16] is equal to ρ_2 calculated using Eq. [19]) and subjected to the following constraints:

- to ensure that the post-tensioning reinforcement does not yield (Eq. [27]–[29])
- to ensure that the end wall panel does not endure uplift (Eq. [30]
- to achieve zero residual drift by maintaining $\rho_{RZD} = 1$ (Eq. [31])
- to ensure that the panels slide rather than rock (Eq. [32])

Results from the parametric studies

The results of the optimization process revealed that for a certain specified ρ_1, the calculated optimum value for ρ_2 was independent of ϕ_1. Thus, a figure for ρ_1 versus ρ_2 can be plotted. It should also be noted that the obtained optimum value ω ranges from 0.46 to 0.49.

Based on the results of the parametric studies, **Fig. 5–10** show a set of charts generated with the nondimensional parameters. The designer can calculate the required areas of post-tensioning tendon A_p and the force in the shear connectors F_{sc} by utilizing Fig. 5–7 for $\Omega = 0.04$ and Fig. 8–10 for $\Omega = 0.05$.

For example, if $\Omega = 0.04$, in order to obtain ρ_1 from Fig. 5 ($n = 2$) or Fig. 6 ($n = 3$), the designer needs to calculate the ratios ρ_2/ω (y-axis) and ϕ_1 (x-axis) from the required design moment strength M_{des}, f'_g, and the dimensions of the wall panels. After calculating these ratios, two lines are drawn: one parallel to the y-axis and the other parallel to the x-axis. The point of intersection of these two lines is the required ratio ρ_1. Afterward, the designer can enter Fig. 7 to obtain ρ_2 by knowing ρ_1. Once ρ_1 and ρ_2 are known, A_p and F_{sc} can be calculated using Eq. (7) and (8), respectively.

Design procedures

The design charts presented in this article are only a small sample to illustrate the effectiveness of the method. In actual design practice, a full set of design charts (with varying f'_g, f_{po}, θ_{des}, Ω, and n) can be developed and the designer may interpolate data between the charts. The following noniterative steps should be followed when using the developed design charts:

1. Obtain the applied moment and design drift (M_{des} and θ_{des}).

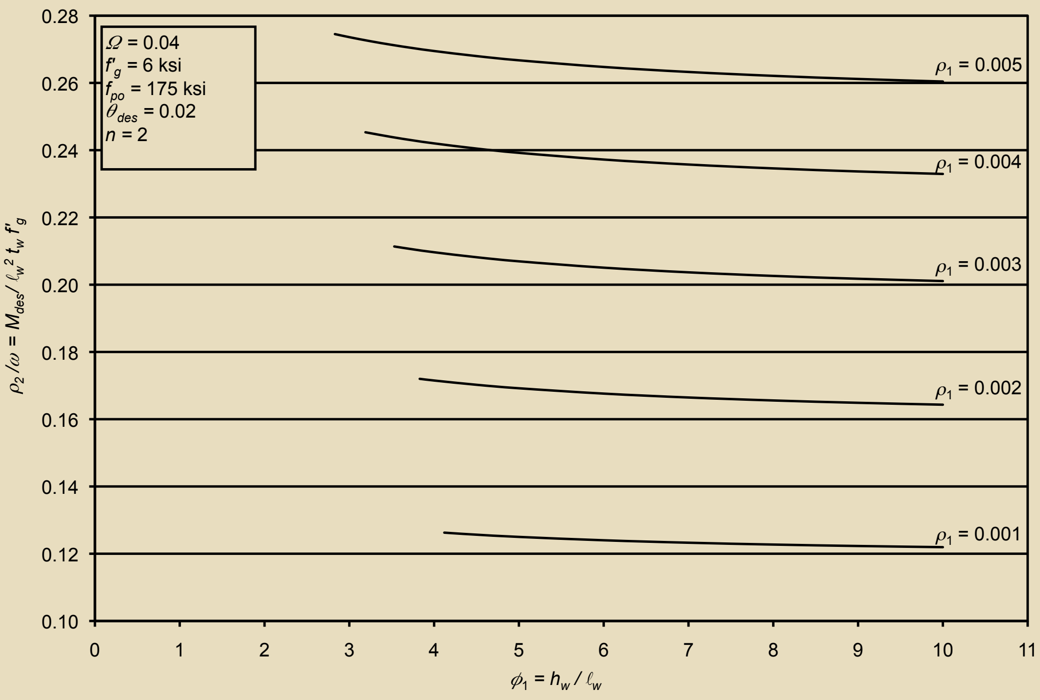

Figure 5. This graph plots ρ_2/ω versus ϕ_1 for two wall panels (n = 2) and Ω = 0.04. Note: f'_g;= smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; h_w = total height of wall panel; ℓ_w = horizontal length of one wall panel; M_{des} = moment demand at design drift; n = number of wall panels per wall; t_w = thickness of wall panel; ϕ_1 = ratio of h_w to ℓ_w; ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state; ω = ratio of the moment resisted by the shear connectors to the total design moment.

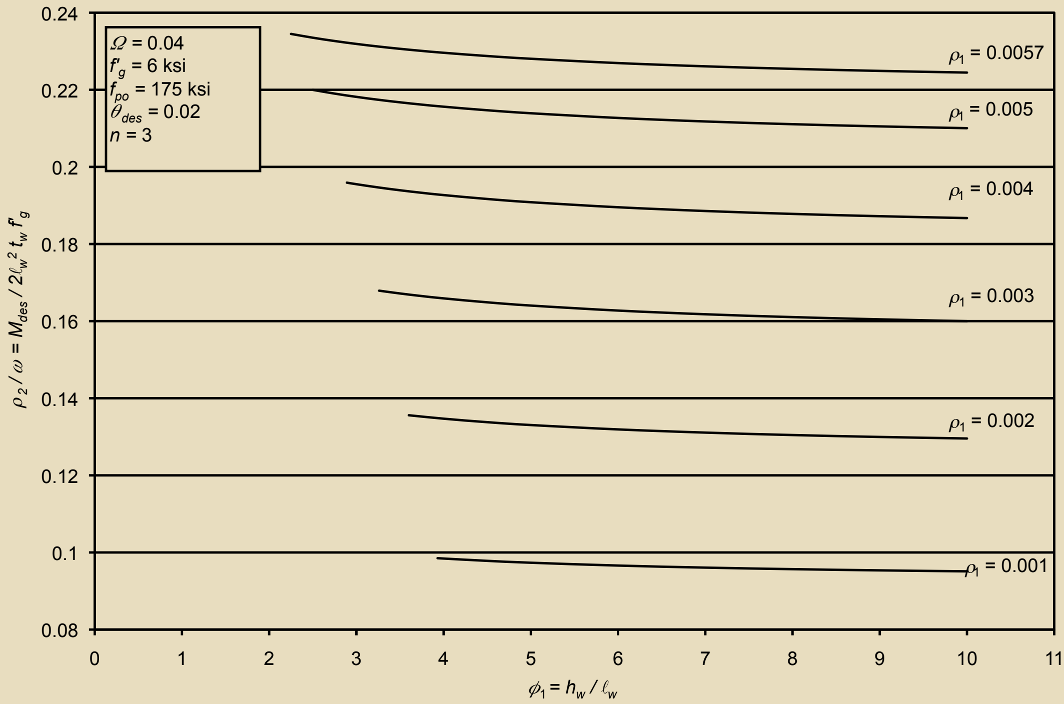

Figure 6. This graph plots ρ_2/ω versus ϕ_1 for three wall panels ($n = 3$) and $\Omega = 0.04$. Note: f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; h_w = total height of wall panel; ℓ_w = horizontal length of one wall panel; M_{des} = moment demand at design drift; n = number of wall panels per wall; t_w = thickness of wall panel; ϕ_1 = ratio of h_w to ℓ_w; ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state; ω = ratio of the moment resisted by the shear connectors to the total design moment.

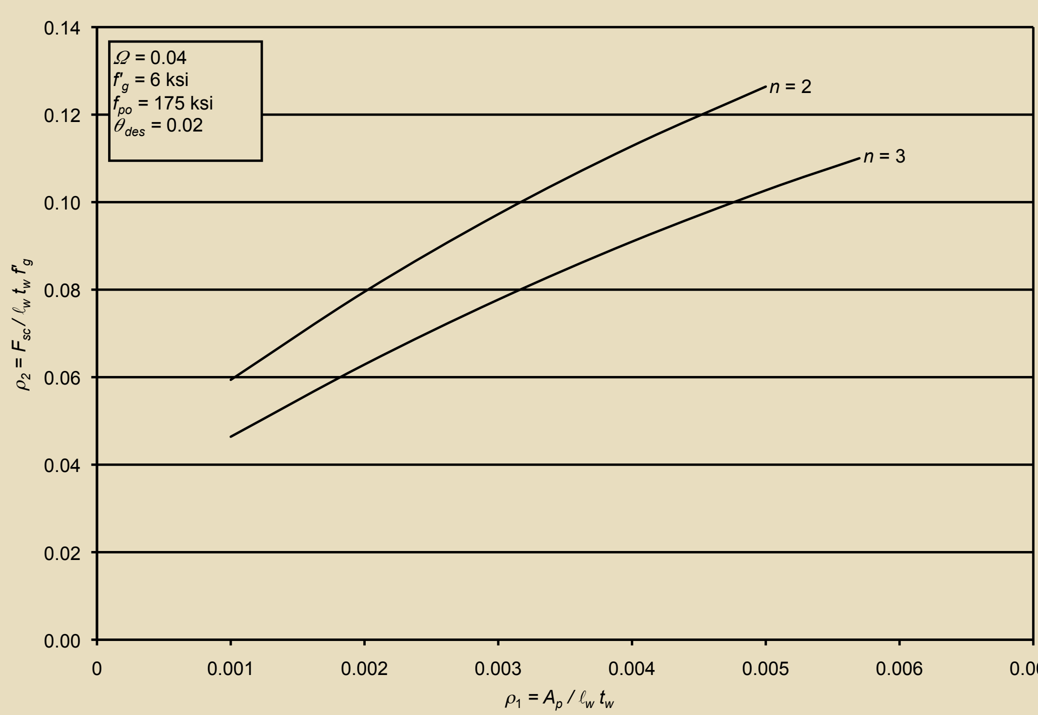

Figure 7. This graph plots ρ_2 versus ρ_1 for $\Omega = 0.04$. Note: A_p = area of post-tensioning tendons; f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; F_{sc} = total yield force of all shear connectors in one vertical joint; ℓ_w = horizontal length of one wall panel; n = number of wall panels per wall; t_w = thickness of wall panel; ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state.

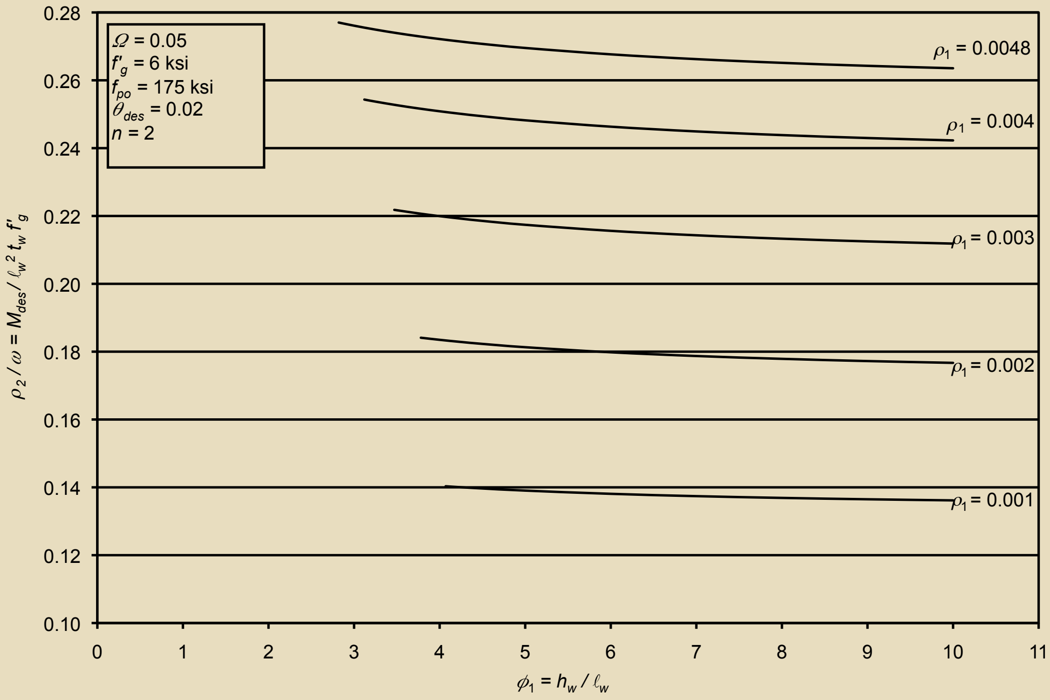

Figure 8. This graph plots ρ_2/ω versus ϕ_1 for two wall panels (n = 2) and Ω = 0.05. Note: f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; h_w = total height of wall panel; ℓ_w = horizontal length of one wall panel; M_{des} = moment demand at design drift; n = number of wall panels per wall; t_w = thickness of wall panel; ϕ_1 = ratio of h_w to ℓ_w; ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; θ_{des} = interface rotation at design limit state; ω = ratio of the moment resisted by the shear connectors to the total design moment.

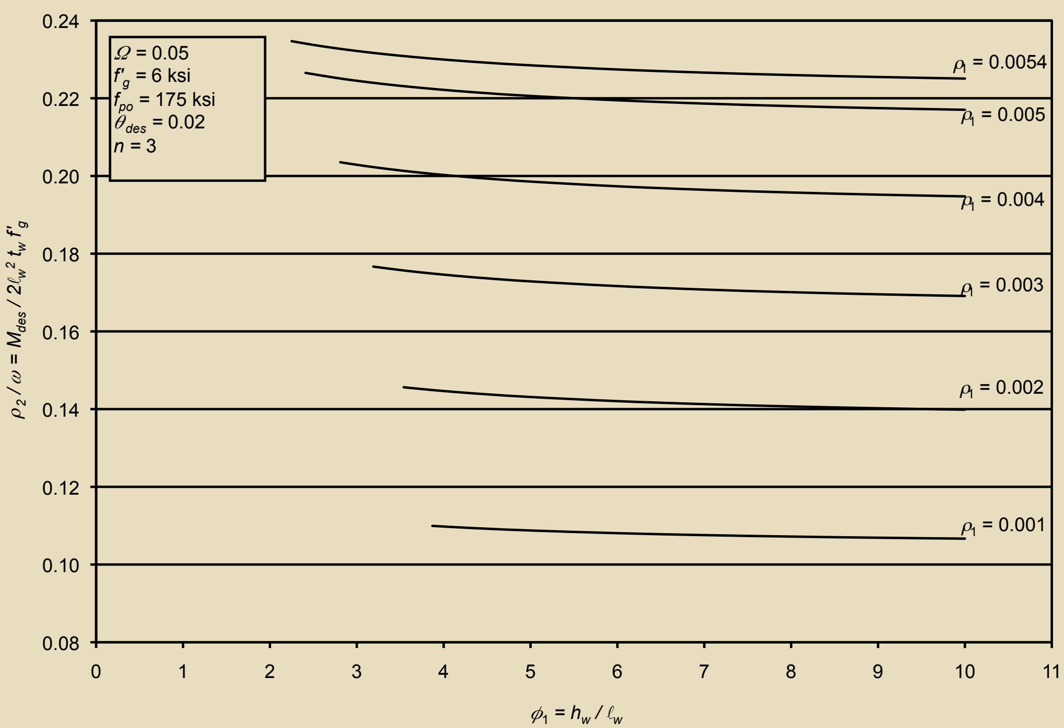

Figure 9. This graph plots ρ_2/ω versus ϕ_1 for three wall panels (n = 3) and Ω = 0.05. Note: n = number of wall panels per wall; ϕ_1 = ratio of h_w to ℓ_w; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g; ω = ratio of the moment resisted by the shear connectors to the total design moment.

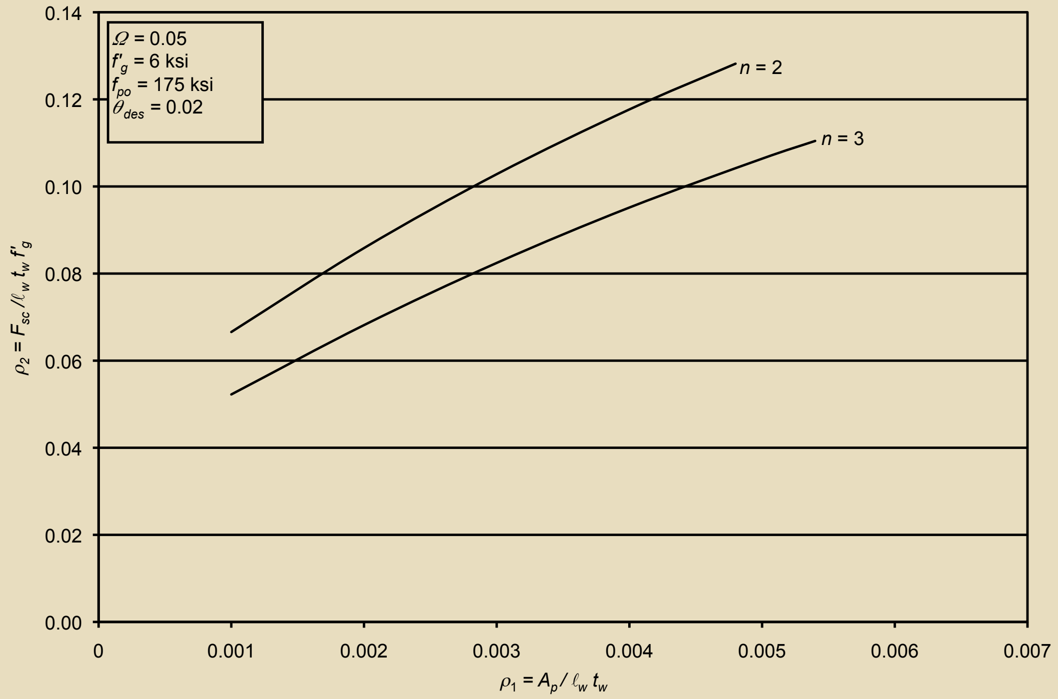

Figure 10. This graph plots ρ_2 versus ρ_1 for Ω = 0.05. Note: ρ_1 = ratio of A_p to $\ell_w t_w$; ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_g$; Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_g.

Table 2. Material properties and design information

Parameter	Value	Comments
h_w, ft	75	Wall height
ℓ, ft	30	Total length of the wall
t_w, ft	1	Wall thickness
n	2	Number of wall segments
f'_g ksi	6	Grout compressive strength
γ_c, lb/ft^3	145	Unit weight of concrete
μ	0.5	Friction coefficient
E_p, ksi	28,500	Modulus of elasticity of post-tensioning reinforcement
f_{py}, ksi	240	Yield stress of post-tensioning reinforcement
f_{po}, ksi	175	Effective prestress after all losses
M_{des}, kip-ft	43,296	Design moment
V_{des}, kip	850	Design shear
w_{floor}, kip/ft	27.67	Floor load
θ_{des}, %	2	Interface rotation

Note: E_p = post-tensioning strand modulus of elasticity; f'_g = smaller of the compressive strengths of grout bed and wall concrete; f_{po} = stress in post-tensioning reinforcement after losses at zero drift; f_{py} = specified yield stress of post-tensioning reinforcement; h_w = total height of wall panel; ℓ = total length of the wall; M_{des} = moment demand at design drift; n = number of wall panels per wall; t_w = thickness of wall panel; V_{des} = design base shear; w_{floor} = distributed vertical load on the wall, at base, from all floors; γ_c = density of concrete; μ = coefficient of friction against concrete; θ_{des} = interface rotation at design limit state. 1 in. = 25.4 mm; 1 ft = 304.8 mm = 0.3048 m; 1000 lb = 1 kip = 4.448 kN; 1 ksi = 6.89 MPa.

Table 3. Design calculations per PRESSS procedures, panel parameters

	Left panel	Right panel
ℓ_w, ft	15	15
h_{eff}, ft	50.93	
W_{panel}, kip	163.1	163.1
$W = W_{panel} + \ell_w w_{floor}$, kip	578.2	578.2
C_c, kip	11,016.0	11,016.0

Note: C_c = compressive capacity of one wall panel; h_{eff} = height above foundation of lateral load resultant on wall (M_{des}/V_{des}); ℓ_w = horizontal length of one wall panel; w_{floor} = distributed vertical load on the wall, at base, from all floors; W = total gravity load from all floors on one wall panel including panel self-weight. 1 ft = 304.8 mm; 1 kip = 4.448 kN.

2. Choose the number of wall panels n.

3. Calculate $\phi_1 = h_w/\ell_w$.

4. Calculate $\rho_2/\omega = M_{des}/(n\text{-}1)\ell_w^2 t_w f_g'$.

5. Find ρ_1 by using figures similar to Fig. 5, 6, 8, or 9.

6. Find ρ_2 by using figures similar to Fig. 7 or 10.

7. Calculate A_p using Eq. (7).

8. Calculate F_{sc} using Eq. (8).

Numerical design example

A design example was developed to illustrate the effectiveness of the developed charts. This example compares the original PRESSS procedure with the developed nondimensional-charts procedure. The design example is a six-story shear wall, and **Table 2** shows the given design parameters. Lateral forces are assumed to be directed from left to right.

Solution using PRESSS procedures

The PRESSS design procedure is iterative. Values for A_p and F_{sc} are first assumed and then all acceptance criteria are checked. Many A_p and F_{sc} combinations can be found to satisfy a given wall design; however, an optimum design is one that gives a moment capacity equal to the required design moment capacity while keeping ρ_{RZD} equal to 1.0. A value of ρ_{RZD} much less than 1.0 also yields zero residual drift, but the force due to the shear connectors will be small compared with the clamping force. This means a high response of the system is expected during an earthquake (Fig. 1). The high response is not a desired effect; therefore, solutions found for the design charts are all based on ρ_{RZD} equal to 1.

For the sake of brevity, the PRESSS design equations are not shown here and the reader is referred to the PRESSS report.[1] The design results using the PRESSS procedures are shown in **Tables 3–6**.

Solution using the developed design charts

Based on the developed design charts the design is noniterative and consists of the following steps.

1. Calculate $\phi_1 = \dfrac{h_w}{\ell_w} = \dfrac{75}{15} = 5$

2. Calculate $\gamma_L = \dfrac{w_{floor}}{t_w h_w} = \dfrac{\frac{27.67}{12}}{12\left[75(12)\right]} = 0.0002135$

3. Calculate

$$\Omega = \frac{h_w\left(\gamma_c + \gamma_L\right)}{f_g'} = \frac{75(12)\left(0.0000868 + 0.0002135\right)}{6} = 0.045$$

4. Calculate $\dfrac{\rho_2}{\omega} = \dfrac{M_{des}}{\ell_w^2 t_w f_g'} = \dfrac{43{,}296(12)}{180^2(12)(6)} = 0.2227$

5. From Fig. 5 ($\Omega = 0.04$) for $\phi_1 = 5$

   and $\dfrac{\rho_2}{\omega} = 0.2227 \Rightarrow \rho_1 = 0.003435$

   From Fig. 8 ($\Omega = 0.05$) for $\phi_1 = 5$

   and $\dfrac{\rho_2}{\omega} = 0.2227 \Rightarrow \rho_1 = 0.003182$

   By linear interpolation for $\Omega = 0.045 \Rightarrow \rho_1 =$ 0.003309

6. From Fig. 7 ($\Omega = 0.04$) and $\rho_1 = 0.003435 \Rightarrow \rho_2$ = 0.1032

Table 4. Design calculations per PRESSS procedures: Cycle 1

	Left panel	Right panel	Comments
$F_{sc,net}$, kip	-1000.0	1000.0	
Establish conditions immediately after liftoff			
P_o, kip	1139.3	1139.3	
N_o, kip	1717.5	1717.5	
C_o, kip	717.5	2717.5	
α_o	0.0326	0.1233	Average = 0.07795
η_o	0.0868	0.3289	
κ_o	0.5823	0.5823	
Establish conditions at design load and drift			
η_{des}	0.1209	0.3430	
P_{des}, kip	1420.6	1255.8	
N_{des}, kip	1998.8	1834.0	
C_{des}, kip	998.8	2834.0	
α_{des}	0.0453	0.1286	
Δ_p, in.	1.4	0.6	
Δf_p, ksi	43.2	17.9	
$f_{p,des}$, ksi	218.2	192.2	
$M_{cap,panel}$, kip-ft	14,312	23,287	
$M_{cap,wall}$, kip-ft	37,599		
ρ_{MOM}	1.15		> 1.0 N.G.
ρ_{fpo}	0.89	0.79	≤ 1.0 O.K.
ρ_{UPL}	0.58	0.58	≤ 1.0 O.K.
ρ_{RZD}	0.75	0.75	≤ 1.0 O.K.
ρ_{ROC}	0.30	0.30	≤ 1.0 O.K.

Note: Assume thirty 0.6-in.-diameter tendons, with A_p = 6.51 in.^{2} and F_{sc} = 1000 kip. The moment capacity of the wall is lower than required. Therefore, higher values for both A_p and F_{sc} are assumed and another cycle is performed. A_p = area of post-tensioning tendons; C_{des} = compressive reaction on one wall panel at design drift; C_o = compressive reaction on one wall panel at zero drift; $f_{p,des}$ = stress in post-tensioning reinforcement at the design drift; F_{sc} = total yield force of all shear connectors in one vertical joint; $F_{sc,net}$ = net vertical force on one panel from all shear connectors; $M_{cap,panel}$ = moment capacity of a panel; $M_{cap,wall}$ = total moment capacity of wall; N_o = total axial force on one wall panel from gravity plus post-tensioning at zero drift; N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift; N.G. = no good; P_o = force in post-tensioning reinforcement at zero drift; P_{des} = force in the post-tensioning reinforcement at design drift; W = total gravity load from all floors on one wall panel including panel self-weight; α_o = relative position of the compressive force as a fraction of wall panel length at zero drift; α_{des} = distance from center of compressive reaction to edge of member divided by member depth, at design drift; Δf_p = increase in stress in prestressing tendon between zero drift and design drift; Δ_p = axial deformation of post-tensioning steel tendon between zero drift and design drift; η_o = distance from member compression face to neutral axis divided by member depth at zero drift; η_{des} = distance from member compression face to neutral axis divided by member depth at design drift; κ_o = the ratio of uplift force to hold-down force; ρ_{fpo} = stress ratio to ensure that post-tensioning reinforcement does not yield at maximum drift; ρ_{MOM} = demand/capacity ratio for overturning moment on wall; ρ_{ROC} = force ratio to ensure that the panel slides rather than rocks; ρ_{RZD} = parameter ratio controlling the residual drift; ρ_{UPL} = ratio of uplift force to hold-down force on one panel. 1 in. = 25.4 mm; 1 ft = 304.8 mm = 0.3048 m; 1000 lb = 1 kip = 4.448 kN; 1 ksi = 6.89 MPa.

Table 5. Design calculations per PRESSS procedures: Cycle 2

	Left panel	Right panel	Comments
$F_{sc,net}$, kip	-1400.0	1400.0	
Establish conditions immediately after liftoff			
P_o, kip	1291.2	1291.2	
N_o, kip	1869.4	1869.4	
C_o, kip	469.4	3269.4	
α_o	0.0213	0.1448	Average = 0.08305
η_o	0.0568	0.3957	
κ_o	0.7489	0.7489	
Establish conditions at design load and drift			
η_{des}	0.0978	0.4053	
P_{des}, kip	1629.5	1370.8	
N_{des}, kip	2207.7	1949.0	
C_{des}, kip	807.7	3349.0	
α_{des}	0.0367	0.1520	
Δ_p, in.	1.4	0.3	
Δf_p, ksi	45.9	10.8	
$f_{p,des}$, ksi	220.9	185.8	
$M_{cap,panel}$, kip-ft	16,114	27,981	
$M_{cap,wall}$, kip-ft	44,095		
ρ_{MOM}	0.98		$\leq$ 1.0 OK
ρ_{fpo}	0.90	0.76	$\leq$ 1.0 OK
ρ_{UPL}	0.75	0.75	$\leq$ 1.0 OK
ρ_{RZD}	1.02	1.02	> 1.0 N.G.
ρ_{ROC}	0.38	0.38	$\leq$ 1.0 OK

Note: Assume thirty-four 0.6-in.-diameter tendons, where A_p = 7.38 in.^{2} and F_{sc} = 1400 kip. The wall capacity is slightly greater than required. However, ρ_{RZD} is slightly larger than 1.0, which means that some residual drift will be expected. Slightly decreasing the number of tendons reduces the wall capacity and at the same time reduces ρ_{RZD}. A new assumption is made for A_p and F_{sc}, and another cycle is performed. A_p = area of post-tensioning tendons; C_{des} = compressive reaction on one wall panel at design drift; C_o = compressive reaction on one wall panel at zero drift; $f_{p,des}$ = stress in post-tensioning reinforcement at the design drift; F_{sc} = total yield force of all shear connectors in one vertical joint; $F_{sc,net}$ = net vertical force on one panel from all shear connectors; $M_{cap,panel}$ = moment capacity of a panel; $M_{cap,wall}$ = total moment capacity of wall; N_o = total axial force on one wall panel from gravity plus post-tensioning at zero drift; N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift; N.G. = no good; P_o = force in post-tensioning reinforcement at zero drift; P_{des} = force in the post-tensioning reinforcement at design drift; W = total gravity load from all floors on one wall panel including panel self-weight; α_o = relative position of the compressive force as a fraction of wall-panel length at zero drift; α_{des} = distance from center of compressive reaction to edge of member divided by member depth, at design drift; Δf_p = increase in stress in prestressing tendon between zero drift and design drift; Δ_p = axial deformation of post-tensioning steel tendon between zero drift and design drift; η_o = distance from member compression face to neutral axis divided by member depth at zero drift; η_{des} = distance from member compression face to neutral axis divided by member depth at design drift; κ_o = the ratio of uplift force to hold-down force; ρ_{fpo} = stress ratio to ensure that post-tensioning reinforcement does not yield at maximum drift; ρ_{MOM} = demand/capacity ratio for overturning moment on wall; ρ_{ROC} = force ratio to ensure that the panel slides rather than rocks; ρ_{RZD} = parameter ratio controlling the residual drift; ρ_{UPL} = ratio of uplift force to hold-down force on one panel.
1 in. = 25.4 mm; 1 ft = 304.8 mm = 0.3048 m; 1000 lb = 1 kip = 4.448 kN; 1 ksi = 6.89 MPa.

Table 6. Design calculations per PRESSS procedures: Cycle 3

	Left panel	Right panel	Comments
$F_{sc,net}$, kip	-1360.0	1360.0	
Establish conditions immediately after liftoff			
P_o, kip	1253.0	1253.0	
N_o, kip	1831.2	1831.2	
C_o, kip	471.2	3191.2	
α_o	0.0214	0.1448	Average = 0.0831
η_o	0.0750	0.3863	
κ_o	0.7427	0.7427	
Establish conditions at design load and drift			
η_{des}	0.0969	0.3965	
P_{des}, kip	1582.1	1337.5	
N_{des}, kip	2160.3	1915.7	
C_{des}, kip	800.3	3275.7	
α_{des}	0.0363	0.1487	
Δ_p, in.	1.5	0.4	
Δf_p, ksi	46.0	11.8	
$f_{p,des}$, ksi	221.0	186.8	
$M_{cap,panel}$, kip-ft	15,766	27,462	
$M_{cap,wall}$, kip-ft	43,228		
ρ_{MOM}	1.00		≤ 1.0 OK
ρ_{fpo}	0.90	0.77	≤ 1.0 OK
ρ_{UPL}	0.74	0.74	≤ 1.0 OK
ρ_{RZD}	1.00	1.00	≤ 1.0 OK
ρ_{ROC}	0.37	0.37	≤ 1.0 OK

Note: Assume thirty-three 0.6-in.-diameter tendons, where A_p = 7.16 in.^{2} and F_{sc} = 1360 kip. All conditions are satisfied. Therefore, use thirty-three 0.6-in.-diameter tendons, where A_p = 7.16 in.^{2} and F_{sc} = 1360 kip. A_p = area of post-tensioning tendons; C_{des} = compressive reaction on one wall panel at design drift; C_o = compressive reaction on one wall panel at zero drift; $f_{p,des}$ = stress in post-tensioning reinforcement at the design drift; F_{sc} = total yield force of all shear connectors in one vertical joint; $F_{sc,net}$ = net vertical force on one panel from all shear connectors; $M_{cap,panel}$ = moment capacity of a panel; $M_{cap,wall}$ = total moment capacity of wall; N_o = total axial force on one wall panel from gravity plus post-tensioning at zero drift; N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift; P_o = force in post-tensioning reinforcement at zero drift; P_{des} = force in the post-tensioning reinforcement at design drift; W = total gravity load from all floors on one wall panel including panel self-weight; α_o = relative position of the compressive force as a fraction of wall-panel length at zero drift; α_{des} = distance from center of compressive reaction to edge of member divided by member depth, at design drift; Δf_p = increase in stress in prestressing tendon between zero drift and design drift; Δ_p = axial deformation of post-tensioning steel tendon between zero drift and design drift; η_o = distance from member compression face to neutral axis divided by member depth at zero drift; η_{des} = distance from member compression face to neutral axis divided by member depth at design drift; κ_o = the ratio of uplift force to hold-down force; ρ_{fpo} = stress ratio to ensure that post-tensioning reinforcement does not yield at maximum drift; ρ_{MOM} = demand/capacity ratio for overturning moment on wall; ρ_{ROC} = force ratio to ensure that the panel slides rather than rocks; ρ_{RZD} = parameter ratio controlling the residual drift; ρ_{UPL} = ratio of uplift force to hold-down force on one panel. 1 in. = 25.4 mm; 1 ft = 304.8 mm = 0.3048 m; 1000 lb = 1 kip = 4.448 kN; 1 ksi = 6.89 MPa.

From Fig. 10 ($\Omega = 0.05$) and $\rho_1 = 0.003182 \Rightarrow \rho_2 = 0.1054$

By linear interpolation for $\Omega = 0.045 \Rightarrow \rho_2 = 0.1043$

7. Calculate $A_p = \rho_1 \ell_w t_w = 0.003309(180)(12) = 7.15$ in.^{2} (4613 mm^2)

   $F_{sc} = \rho_2 \ell_w t_w f_g' = 0.1043(180)(12)(6) = 1352$ kip (6014 kN)

   Use thirty-three 0.6-in.-diameter (15 mm) strands, where $A_p = 7.16$ in.^{2} (4619 mm^2) and $F_{sc} = 1352$ kip (6014 kN).

Design of the shear connectors is beyond the scope of this paper. This paper is concerned only with finding the optimum combination of A_p and F_{sc}. The actual choice of number and diameter of strands and type of shear connectors is also left to the designer. The PRESSS report[1] has guidelines for the design of U-shaped flexural connections as shear connectors.

Conclusion

The following conclusions are made:

- The PRESSS procedures to design hybrid walls can be modified into a nondimensional procedure that can be solved with optimization routines for a wide range of parameters.

- The solution of the nondimensional formulation can be summarized into nondimensional charts. A sample of the nondimensional charts is presented in this article.

- A design example is included to illustrate the effectiveness of the nondimensional design charts in designing the hybrid walls. The design procedure following the PRESSS report[1] results in significantly more calculations as more wall segments are used because calculations must be made for each wall segment separately. The design steps needed when using the developed design charts are unrelated to the number of wall segments because only the charts corresponding to the number of wall segments n are used.

- A full set of design charts that include all ranges of parameters (f_g', f_{po}, θ_{des}, Ω, and n) can be developed. Such charts easily provide the designer with insight into the effects of various parameters on the design.

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Notation

A_p = area of post-tensioning tendons

C_c = compressive capacity of one wall panel

C_{des} = compressive reaction on one wall panel at design drift

C_{des_L} = compressive reaction on left wall panel at design drift

C_{des_M} = compressive reaction on interior wall panel at design drift

C_{des_R} = compressive reaction on right wall panel at design drift

C_o = compressive reaction on one wall panel at zero drift

E_p = post-tensioning strand modulus of elasticity

f_c' = concrete design compressive strength

f_g' = smaller of the compressive strengths of grout bed and wall concrete

$f_{p,des}$ = stress in post-tensioning reinforcement at the design drift

f_{p,des_L} = stress in post-tensioning reinforcement of left wall panel at the design drift

f_{p,des_M} = stress in post-tensioning reinforcement of an interior wall panel at the design drift

f_{p,des_R} = stress in post-tensioning reinforcement of right wall panel at the design drift

f_{po} = stress in post-tensioning reinforcement after losses at zero drift

f_{py} = specified yield stress of post-tensioning reinforcement

F_{sc} = total yield force of all shear connectors in one vertical joint

F_{sc_L} = total yield force of all shear connectors in joint to left of panel

$F_{sc,net}$ = net vertical force on one panel from all shear connectors

F_{sc_R} = total yield force of all shear connectors in joint to right of panel

h_{eff} = height above foundation of lateral load resultant on wall (M_{des}/V_{des})

h_w = total height of wall panel

k_1 = uniform stress in Whitney rectangular equivalent stress block divided by f_g'

ℓ_w = horizontal length of one wall panel

$\ell_{w,tot}$ = total length of the wall

$M_{cap,panel}$ = moment capacity of a panel

M_{cap_L} = moment capacity of left wall panel

M_{cap_M} = moment capacity of interior wall panel

M_{cap_R} = moment capacity of right wall panel

$M_{cap,wall}$ = total moment capacity of wall

M_{des} = moment demand at design drift

M_{sc} = total moment capacity of all U-shaped flexural plate connectors

n = number of wall panels per wall

N_{des} = total axial force on one wall panel from gravity plus post-tensioning at design drift

N_o = total axial force on one wall panel from gravity plus post-tensioning at zero drift

P_{des} = force in the post-tensioning reinforcement at design drift

P_{des_L} = force in the post-tensioning reinforcement of left wall panel at the design drift

P_{des_M} = force in the post-tensioning reinforcement of an interior wall panel at the design drift

P_{des_R} = force in the post-tensioning reinforcement of right wall panel at the design drift

P_o = force in post-tensioning reinforcement at zero drift

t_w = thickness of wall panel

V_{des} = design base shear

w_{floor} = distributed vertical load on the wall, at base, from all floors

W = total gravity load from all floors on one wall panel, including panel self-weight

α = a factor less than or equal to 1.0 to ensure that the post-tensioning steel does not yield

α_o = relative position of the compressive force as a fraction of wall-panel length at zero drift

α_{o_L} = relative position of the compressive force as a fraction of the left wall-panel length at zero drift

α_{o_M} = relative position of the compressive force as a fraction of the interior wall-panel length at zero drift

α_{o_R} = relative position of the compressive force as a fraction of the right wall-panel length at zero drift

$\alpha_{o,avg}$ = the average of α_o across all panels

α_{des} = distance from center of compressive reaction to edge of member divided by member depth, at design drift

α_{des_L} = distance from center of compressive reaction to edge of left wall panel divided by wall depth, at design drift

α_{des_M} = distance from center of compressive reaction to edge of interior wall panel divided by wall depth, at design drift

α_{des_R} = distance from center of compressive reaction to edge of right wall panel divided by wall depth, at design drift

β_1 = depth of equivalent compressive stress block divided by the neutral-axis depth

γ_c = density of concrete

γ_L = ratio of w_{floor} to $t_w h_w$

Δf_p = increase in stress in prestressing tendon between zero drift and design drift

$\Delta f_{p\infty}$ = increase in stress in post-tensioning tendon between zero drift and design drift when concrete and grout strengths are infinite

Δ_p = axial deformation of post-tensioning steel tendon between zero drift and design drift

Δ_{P_L} = the elongation of the tendon at the design-drift level

η_o = distance from member compression face to neutral axis divided by member depth at zero drift

η_{des} = distance from member compression face to neutral axis divided by member depth at design drift

θ_{des} = interface rotation at design limit state

κ_o = ratio of uplift force to hold-down force

μ = coefficient of friction against concrete

ϕ_1 = ratio of h_w to ℓ_w

ϕ_2 = ratio of h_{eff} to ℓ_w

ψ_L = ratio of f_{p,des_L} to f'_c

ψ_M = ratio of f_{p,des_M} to f'_c

ψ_o = ratio of f_{po} to f'_c

ψ_R = ratio of f_{p,des_R} to f'_c

ρ_1 = ratio of A_p to $\ell_w t_w$

ρ_2 = ratio of F_{sc} to $\ell_w t_w f'_c$

ρ_{fpo} = stress ratio to ensure that post-tensioning reinforcement does not yield at maximum drift

ρ_{MOM} = demand/capacity ratio for overturning moment on wall

ρ_{ROC} = force ratio to ensure that the panel slides rather than rocks

ρ_{RZD} = parameter ratio controlling the residual drift

ρ_{UPL} = ratio of uplift force to hold-down force on one panel

ω = ratio of the moment resisted by the shear connectors to the total design moment

Ω = ratio of $h_w(\gamma_c + \gamma_L)$ to f'_c

About the authors

Elias I. Saqan, PhD, MPCI, is an associate professor and chair of the Department of Civil Engineering at the American University in Dubai, UAE.

Rami A. Hawileh, PhD, is an assistant professor for the Department of Civil Engineering at the American University of Sharjah, UAE.

Synopsis

The Precast Seismic Structural Systems (PRESSS) research proposed five different seismic structural systems made from precast concrete elements. These systems formed various parts of the structural framing in the PRESSS phase III building that was tested at the University of California at San Diego.

Based on the PRESSS evaluation, one of the structural systems that has the potential to eliminate residual drift after an earthquake was the unbonded, post-tensioned, split precast concrete wall system. The PRESSS report outlines a dimensional design procedure for the unbonded post-tensioned split precast concrete walls. This procedure is iterative, requiring lengthy calculations to achieve an optimum design. This study developed a set of new nondimensional parameters and procedures for the design of such walls. The goal of this research was to develop a set of nondimensional design charts that require no iterations. Such charts are based on an optimum design of zero residual drift while the moment capacity of the wall is equal to the applied design moment. The results of these studies were used to generate nondimensional design charts. A numerical design example using the conventional iterative PRESSS design procedure and the new non-iterative design charts procedure is presented, and the methods are compared.

Keywords

Design chart, nondimensional parameter, seismic, self-centering, shear force, split wall, unbonded post-tensioning.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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