DAMAGE AVOIDANCE DESIGN OF WAREHOUSE BUILDINGS USING THE PRECAST HOLLOW CORE WALLS SYSTEM

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ABSTRACT:

The need for new seismic design philosophy that has performance traits of conventional ductile concrete monolithic wall systems but without permanent damage potential is presented. Innovative construction methods are proposed that avoid earthquake induced damage by using an entirely precast wall system. The walls consist of a mix of seismic wall panels and non-seismic infill cladding panels. The seismic wall panels are used to seat the rafters. Lateral loads are transmitted via diaphragm action through a roof truss. Damage Avoidance Design (DAD) principles are incorporated by using rocking walls with armoured steel seating. A seven-step design procedure is proposed which includes: assessment of seismic hazard (demand); setting design target displacements (capacity); estimation of effective damping; calculation of the base shear capacity in the form of a lateral pushover curve; design unbonded tendons and/or fuse-bars; re-evaluation of effective damping; and an assessment of seismic resistance adequacy via a demand vs capacity evaluation of the structure.

KEYWORDS: seismic wall, non-seismic wall, armoured steel seating

1. INTRODUCTION

The general problem is related to the seismic design approach presently used which generally leads to construction of monolithic connections where plastic hinge zones (PHZ) are produced at wall-foundation interfaces. This can lead to permanent irreparable earthquake induced damage to structures. Seismic damage may also be exacerbated through out-of-plane seismic loading leading to instability of the structure due to P-Δ effects of gravity load from the roof. To overcome performance deficiencies that arise from ductile (conventional) seismic design it is evident that it is desirable to change to alternative design concepts that are capable of maintaining the post-earthquake serviceability of structures. Seismic isolation is one approach where post-earthquake damage is minimised and a higher level of performance can be specified. However, seismic isolation is inappropriate for certain structural configurations including large warehouse type structures. Another promising design approach is to use rocking structures mechanism. When coupled with a "Damage Avoidance Design" philosophy, similar to that proposed by Mander and Cheng (1997) for bridges, damage-free performance can be attained by utilizing steel-armouring details at critical rocking connections. Longitudinal wall (or column) reinforcement is terminated at the foundation beam interface and the walls (or columns) are free to rock.

The purpose of this research is to expand existing design procedures for rocking structures, and to make them applicable for the seismic design of multi-panel precast hollow core walls that are free to rock on their foundations. Following the development of design procedures, a design example and detail of typical warehouse building will be demonstrated.

2. BASIC CONCEPT OF ROCKING WALL IN WAREHOUSE BUILDINGS

Figure 1 shows a proposed prototype warehouse building constructed using precast hollow core walls. The multi-panel walls consist of seismic and non-seismic wall panels. Figure 1(a) shows the horizontal bracing elements that form a roof diaphragm to transmit inertia loads (or wind induced forces) to the seismic wall units. Figure 1(b) shows the plan view of a warehouse with their seismic and non-seismic walls, wind trusses and rafter with portal frame. Figure 1(c) shows the cross-section X-X of the portal frame together with seismic wall panel. The detail (Connection A) shows the connection between top of the walls and edges of portal frames and Connection B depicts the joint detail between foundation beam and bottom of the walls.

3. SEISMIC RESISTANCE OF ROCKING PHCW

Based on the foregoing design strategy it is now necessary to derive some basic equations that relate wall seismic resistance along with spread footing interaction to the base shear capacity of a rocking precast hollow core wall system. Figure 2 shows all the forces acting on multi-panel walls and their interaction with their supporting spread footings. In order to prevent uplift of the spread footing, it is necessary to limit the lateral load base shear capacity of the walls. This can be achieved by optimizing the size of fuse-bars and prestressing tendons. This is considered necessary, otherwise tension piles may need to be provided beneath the footing, or else the soil be permitted to fail. The maximum soil pressures can be minimised by locating each seismic wall panel at the center of each spread footing unit. The total length of spread footing depends on the number of wall panels placed on top of foundation block.

The lateral resistance of the seismic wall system is provided by the combination of roof loading and self-weight of each wall panel. The unbonded post-tensioned tendons and/or fuse-bars also add to the lateral resistance. Figure 2(a) shows the seismic resistance of multi-panel walls due to all the forces acting from roof to the spread footing. The lateral load capacity of a multi-panel wall system is given by the following equation:

$$F = \frac{B}{2H}(W_r + nW_w + T) \tag{3.1}$$

where $B =$ panel width, $H =$ panel height, $W_r =$ weight of roof reaction from rafter, $W_w =$ weight of one wall panel, $n =$ the number of wall panels, $T =$ total tension forces provided by prestress and mechanical energy dissipators, if any. Simplification of base shear capacity of the wall using equation (3.1) is as follows:

$$C_c = \frac{F}{W_r + nW_w} = \frac{F}{W_s} = \frac{B}{2H}\left(1 + \frac{T}{W_s}\right) \tag{3.2}$$

where $W_s = W_r + nW_w =$ structural seismic weight. By taking moments at the discontinuity point of the strip footing (refer to Figure 2(b)), the maximum eccentricity on foundation reaction is derived as:

$$FH = (W_s + W_f)e \tag{3.3}$$

in which $W_f =$ weight of foundation strip footing and $e =$ eccentricity underneath foundation beam. The ratio of eccentricity (e) over total length of strip footing (L_s) can be defined as:

$$\frac{e}{L_s} = \frac{e}{nB} = \frac{FH}{nB(W_s + W_f)} \tag{3.4}$$

Substituting equation (3.1) into equation (3.4):

$$\frac{e}{L_s} = \frac{e}{nB} = \frac{B}{2H} W_s \left[1 + \frac{T}{W_s}\right] \frac{H}{nB(W_s + W_f)} \tag{3.5}$$

$$\frac{e}{B}=\frac{1}{2}\left[\frac{1+T/W_s}{1+W_f/W_s}\right] \tag{3.6}$$

Ideally for no tension uplift $e < L_s/6$, where

$$\frac{e}{B}<\frac{L_s}{6B}=\frac{nB}{6B}=\frac{n}{6} \tag{3.7}$$

By relating base shear capacity of the system with eccentricity underneath foundation block where equation (3.6) is equal to equation (3.7), the eccentricity ratio becomes

$$\frac{e}{B}=\frac{1+T/W_s}{1+W_f/W_s}<\frac{n}{3} \tag{3.8}$$

The tension force limit for prestress and/or mechanical energy dissipator's, if any, is given by:

$$T<\frac{n}{3}\left(W_s+W_f\right)-W_s \tag{3.9}$$

Also, before any prestress of unbonded post-tensioned tendons can be applied, the right hand side of equation (3.9) must be positive where

$$n>\frac{3}{1+W_f/W_s} \tag{3.10}$$

For a light foundation where $W_f \to 0$, in order to prevent the uplifting of light foundation beam the number of wall panels should be greater than 3 $(n>3)$. Also, for example, if the foundation beam is designed to carry six panels plus the roof loading, the ratio of tension force over seismic weight of the system is derived as:

$$\frac{T}{W_s}<\frac{6}{3}\left(1+\frac{W_f}{W_s}\right)-1=1+\frac{2W_f}{W_s} \tag{3.11}$$

By substituting equation (3.11) into equation (3.1), the design base shear capacity becomes:

$$C_c=\frac{B}{H}\left(1+\frac{W_f}{W_s}\right) \tag{3.12}$$

Figure 2(c) shows the interaction between multi-panel walls and spread footings during ground shaking. The steel channel on top of the walls is used to transmit lateral forces from one seismic wall to the next seismic wall through its web. If the lateral displacement is large, a plastic hinge mechanism will occur at the V-cut location on its flange. During earthquake excitation, the seismic and roof loads are transferred to the spread footing and the shear key/pintels underneath seismic wall will prevent the walls from sliding. The non-seismic walls transmit gravity load through the rubber pad placed between the wall and foundation beam.

4. DESIGN PROCEDURE FOR PRECAST HOLLOW CORE WALLS

The design procedure for rocking PHCW by incorporating unbonded tendons and fuse-bars together with the theoretical background is proposed. This procedure involves seven steps as described below and summarised in Figure 3.

STEP 1: Determine Seismic Demand of DBE and MCE based on the Hazard Exposure

Two desired level of ground motions are identified namely, basic design earthquake (DBE- 10% probability in 50 years) and maximum considered earthquake (MCE- 2% probability in 50 years). For example, the values for DBE and MCE at Wellington (New Zealand) are 0.4g and 0.8g, respectively. Damping reduction factors as mentioned above with different level of effective viscous damping are considered in spectrum seismic demand.

STEP 2: Determine the maximum response displacement, $\Delta_{\max}^{DBE}$ and $\Delta_{\max}^{MCE}$

For the DAD philosophy, the performance objective for DBE is that the structure remains elastic during ground shaking with target design drift, $(\theta < 2\%)$. Under MCE, the structure is allowed to yield especially supplemental energy dissipators but no structural damage should exist in wall $(\theta \leq 4.0\%)$. The target design drift is calculated based on performance criteria and target design displacement at effective height of structures as defined below:

$$\Delta_{\max}^{DBE} = \theta_{\max}^{DBE} H_{eff} \qquad \text{and} \qquad \Delta_{\max}^{MCE} = \theta_{\max}^{MCE} H_{eff} \tag{4.1}$$

STEP 3: Estimate total effective damping of the system (ξ_{eff})

Total effective damping of the structure can be estimated from which the damping factors B_a, B_v and B_d.

STEP 4: Calculate the required base shear capacity of the structures, C_c^{DBE} and C_c^{MCE}

The required base shear capacity of the structure can be calculated from modified equation (3.1) and (3.2) as follows:

$$C_c = \frac{0.25g}{\pi^2 \Delta}\left(\frac{F_v S_1}{\phi B_v}\right)^2 \leq \frac{F_v S_1}{\phi T_v B_a} \tag{4.2}$$

where $\Delta \leq \dfrac{0.25g}{\pi^2}\left(\dfrac{F_v S_1}{\phi B_d}\right) T_d$

STEP 5: Design energy dissipator and unbonded tendons

Once the required base shear capacity of the structure is known, calculate the required cross-sectional area of unbonded fuse-bars and unbonded tendons.

STEP 6: Evaluation of hysteresis damping and total effective damping of the system

Calculate total effective damping by summing the intrinsic, radiation and hysteretic damping of the system. These values should be checked with the estimates in Step 3, and adjustments to the solutions in Steps 4 and 5 made accordingly.

STEP 7: Assessment of Seismic Capacity

The seismic capacity of precast hollow core wall system can be assessed by checking that the base shear capacity of the system is bigger than the base shear demand. If the design does not conform to above equation, then Steps 4 to 6 are repeated until they converge and the design is considered acceptable.

5. CONCLUSIONS AND RECOMMENDATIONS

The concept development of rocking wall panel structures along with the proposed design procedures for the construction of warehouse buildings is presented. Based on this study, the following conclusions are drawn:

1. The end-user community now is becoming more demanding requiring minimal and preferably no seismic damage. By using Damage Avoidance Design (DAD) philosophy along with the proposed design procedure, the repairable damage to industrial buildings will be minimized (re-prestress fuse-bars) and irreparable damage to the structures can be avoided.
2. In order to avoid any damage to the wall and strip footing, these structural elements should be discontinuous and require steel-steel or concrete-rubber protection against rocking motion impact. During earthquake excitation, the rocking toe experiences a high point stresses between foundation-wall interfaces. By providing steel-steel rocking interface, the rocking wall behave bilinear elastic fashion and keeps the self-centring characteristics, therefore, no residual displacement or permanent damage is expected to occur.
3. The design process proceeds without the need to determine the fundamental period. This is useful, as for rocking structures the period constantly changes. Once designed and detailed, the adequacy of the design can be checked against rapid IDA curves.

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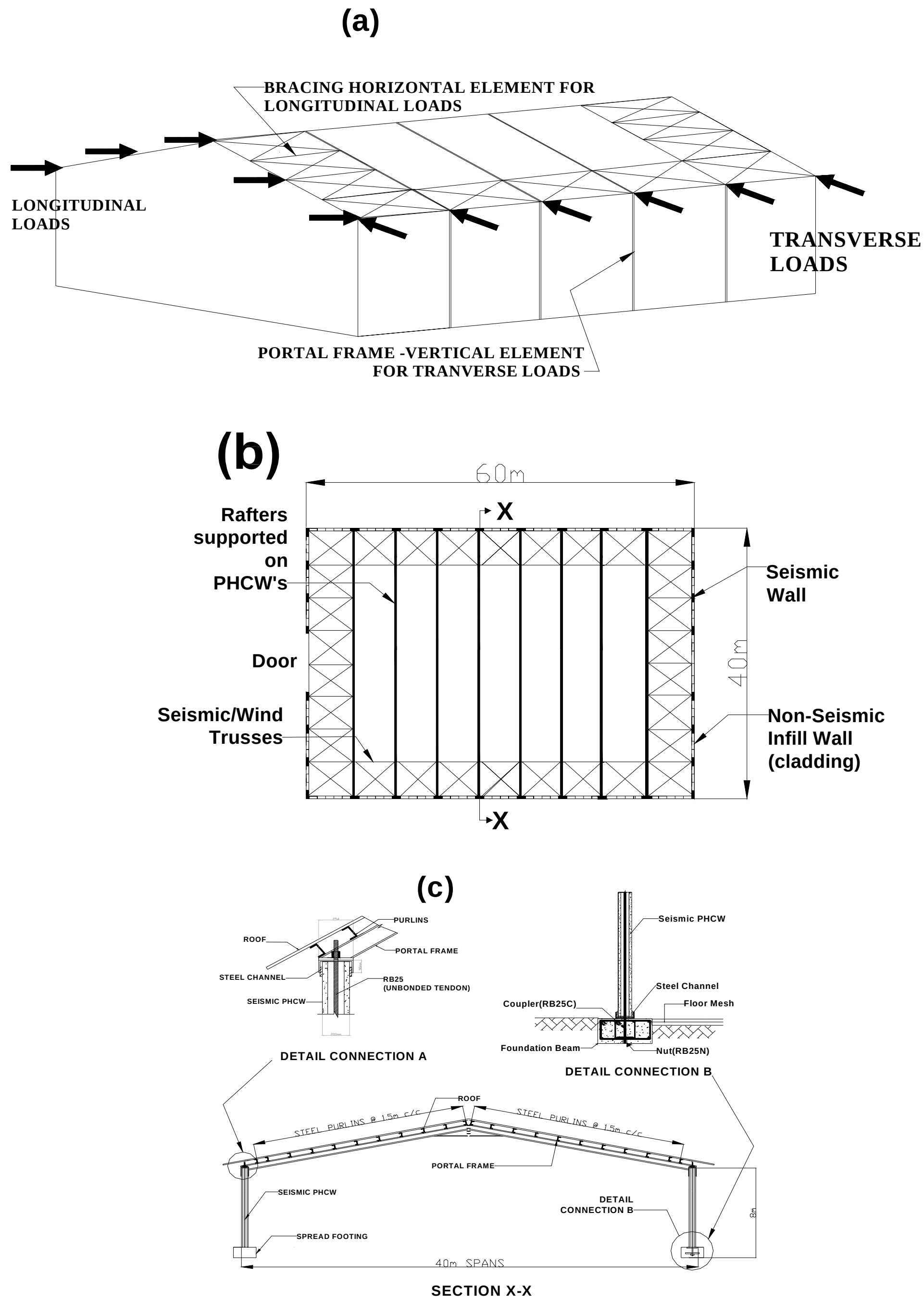

Figure 1: Concept overview of warehouse building; (a) distribution of transverse and longitudinal loading arising from either wind or earthquake effects; (b) plan view of warehouse showing lines of portal frames seated on PHCW; and (c) steel portal frame setting on PHCW together with detailing connection at top and bottom of the wall.

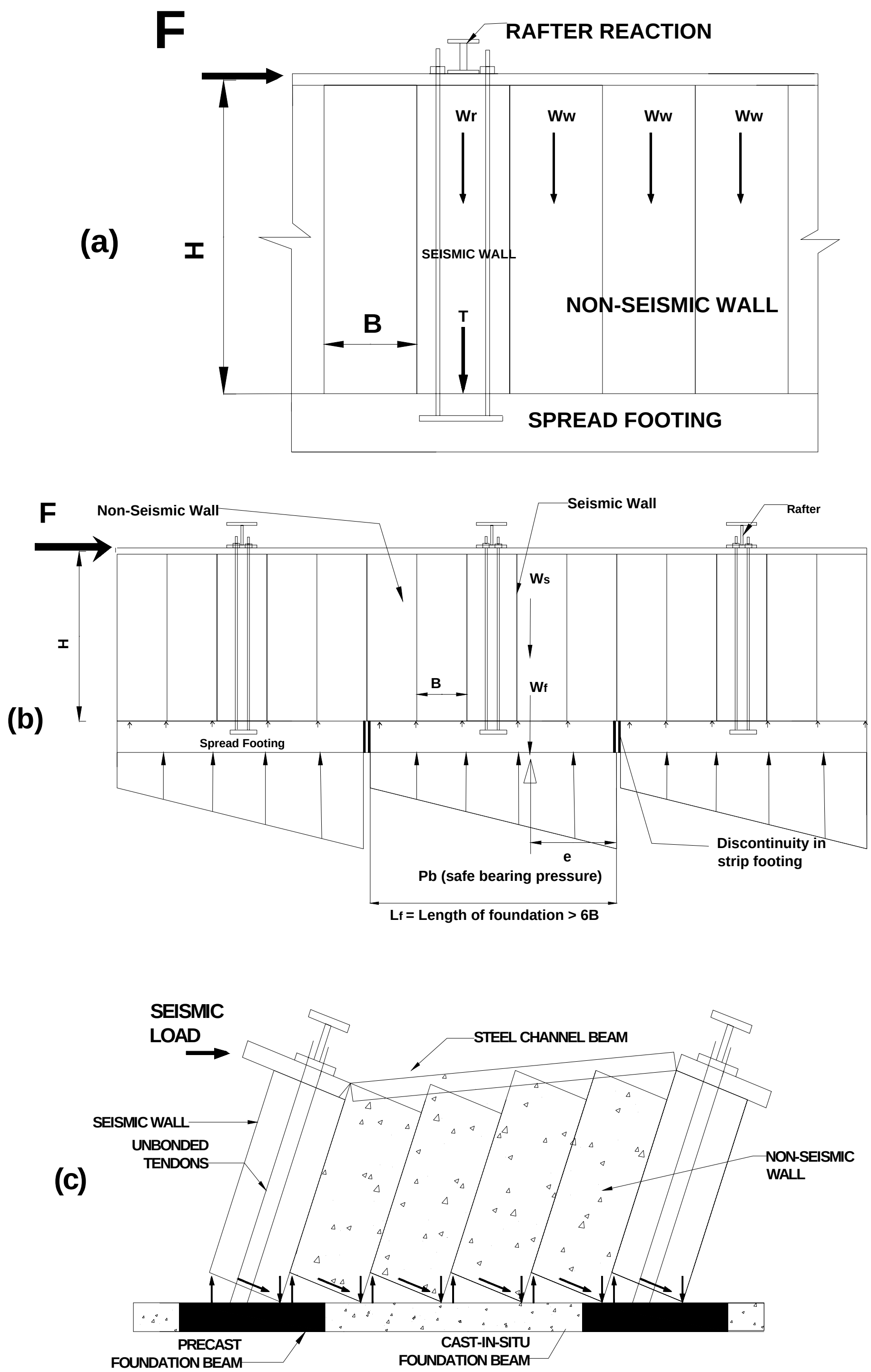

Figure 2: The forces and interaction of multi-panel acting on spread footing; (a) seismic resistance in multi-panel wall system; (b) distribution of soil bearing pressure underneath spread footing; and (c) interaction between multi-panel walls and spread footing during ground shaking.

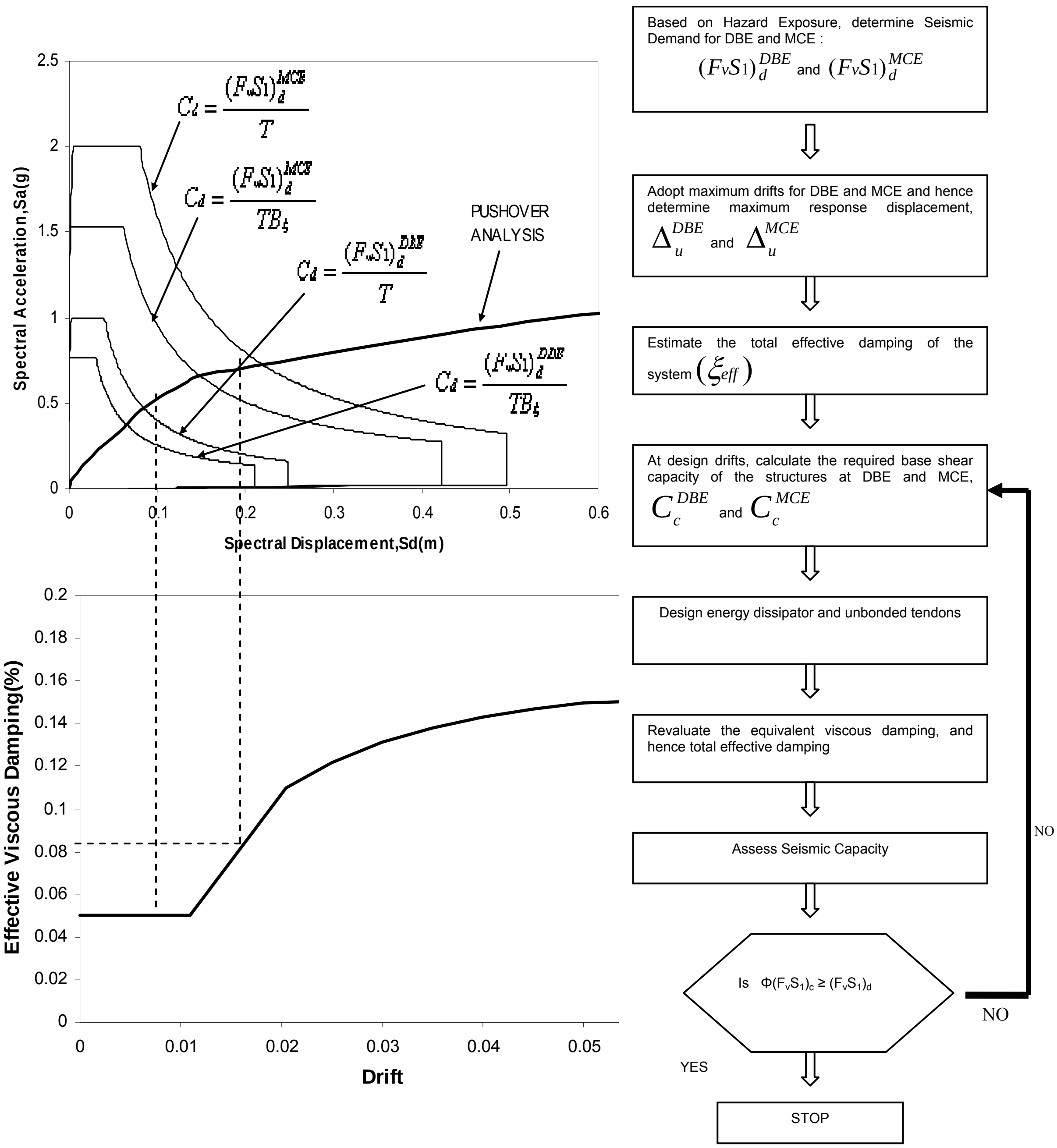

Figure 3: The flow chart of proposed design procedure for rocking precast hollow corewalls by adopting Damage Avoidance Design (DAD) philosophy; (a) arrangement of multi-wall panel system together with foundation beam; (b) steel portal frame sitting on seismic wall; (c) rocking base plate and pintles; and (d) reidbars and pintles are welded to steel channel acting as rocking toes.