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Procedia Engineering 199 (2017) 3546–3551

Procedia Engineering

www.elsevier.com/locate/procedia

X International Conference on Structural Dynamics, EURODYN 2017

Performance-based seismic design framework for RC floor diaphragms in dual systems

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Abstract

Floor diaphragms play several roles in the seismic response of dual systems: support vertically spanning components, transfer lateral forces to walls and frames, provide restraint to columns and walls, tie the structure together, and enable redundant load paths for lateral forces. In many buildings after events as recent as the 2010-2011 Christchurch earthquakes, floor diaphragms were unable to perform one or more of these functions, leading to extensive damage and collapse. Research consistently highlights three underlying causes: a failure to ensure the integrity of the load path, underestimation of in-plane forces, and poorly understood interactions with walls, supporting beams and RC moment frames. Most recent and ongoing research has focused on modifying the prescriptive code provisions, and indeed many codes –but not all− have been consequently updated. Such prescriptive rules, however, are not helpful for assessing existing buildings, comparing alternate means and methods, or in displacement-based design (DBD) for which a performance-based design (PBD) framework is necessary. This paper proposes a new performance-based framework for the seismic design of reinforced-concrete (RC) floor diaphragms with or without precast elements. The floor diaphragm performance limit states (LS) are re-defined in terms of the observed failure modes (FM). Results from prior research on these failure modes are used to select damage measures (DM, *e.g.* 'crack width') for pairs of FM and LS. Expressions for DMs in terms of engineering demand parameters (EDP, *e.g.*, 'strain') are derived from experimental results or from first principles. EDPs are the basic output from numerical analysis in PBD; this paper comments on the suitability of different analytical approaches.

Keywords: performance-based seismic design; floor diaphragms; dual systems; displacement-based design

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10.1016/j.proeng.2017.09.512

1. Introduction

In seismic design, force-controlled failure modes are precluded using a capacity design approach. The most common code approach for floor diaphragms is to consider them “brittle” elements (force-controlled) and its shear strength is checked against the analysis forces at over-strength. Yielding of the steel in the chords, being a ductile failure mode, is usually permitted, so it is unusual to multiply the analysis forces by any over-strength factor. Recent research, however, shows that current code approaches fail to address observed failure modes and preclude non-ductile modes from occurring. Furthermore, damage associated with ductile failure modes is not normally explicitly assessed.

Performance-based earthquake engineering (PBEE), and its design component (PBD), aims at a reliable prediction of a building’s performance in an earthquake, defined as an attained limit state for a given level of ground shaking intensity. Typically, three levels of performance are defined, although in recent approaches a continuum, probabilistic-based treatment of risk is followed. The attainment of any given limit state is assessed through quantifiable damage measures (DM − crack width, buckling of rebar, etc.) which can be used to estimate whether repair is possible and what are the cost and downtime impacts of the repair process. In this paper, the damage to floor slabs observed during experiments and real earthquakes and reported in the literature is used to describe each limit state qualitatively. This paper reports the new framework, developed as part of one of the authors’ PhD dissertation [1].

2. Performance objectives

There are two approaches to defining performance objectives (POs) [2]: a) paired performance and hazard levels (discrete POs), and b) the convolution integral approach (PEER approach) that treats both hazard and vulnerability as continuous functions. The PEER approach is generally better suited for risk assessment and for comparing design options in terms of a design variable (DV), requires knowledge of the fragility function of each component, and involves non-linear response history analyses (NLRHA). This research adopts the simpler discrete POs approach as outlined in Figure 1, which is readily adaptable into current code prescriptions. Three objectives are used in this paper: immediate occupancy (IO), life safety (LS), and collapse prevention (CP). The above limit states are common among various publications but the terminology varies. For example, in [3] they are called Level 1, 2 and 3, respectively, while Vision 2000 uses “operational”, “life safe” and “near collapse”. In different publications “life safe” denotes different limit states, while IO can be ‘operational’ or ‘fully operational’ depending on non-structural performance.

3. General PBEE Framework

The damage measures (DM) need to be related to variables extracted from numerical analysis, called engineering demand parameters (EDPs). Earlier versions of PBEE used only global EDPs from elastic models, such as drift, to estimate performance. Although such parameters are still useful *e.g.* to compute non-structural damage, non-linear models allow a better understand damage and statistical treatment. The response variables obtained from non-linear models can be global (chord rotation, inelastic drift) or local (material strain, curvature). Which EDP to use depends on the damage measure and the element analysed. Different EDPs warrant different analytical techniques. Global EDPs can be obtained from elastic analysis. Displacement-based, global EDPs can be calculated from elastic analyses with appropriate assumptions regarding inelastic response; current computational capabilities allow these EDPs to be best computed using lumped-plasticity non-linear analyses. Displacement-based local EDPs, on the other hand, require recovery of response values at a fibre and section levels, so usually warrant fibre-based lumped plasticity or distributed plasticity approaches. The approach followed in this research is summarised in Figure 1.

4. DMs and EDPs to describe diaphragm performance

A one-on-one relationship can be established between each identified failure mode and EDPs. Figure 1 shows this relationship for IO, LS and CP limit states. The EDPs are colour-coded by analytical method: green for elastic models, yellow for global non-linear models, red for local non-linear models. A more detailed description can be found in [1].

4.1. Immediate Occupancy (Serviceability) Limit state

The following limit states should be checked against a frequent earthquake: (a) flexural cracking due to tension in the chords; (b) diagonal cracking due to in-plane shear stress; and (c) localized cracking at discontinuities. Cracks form early on during cyclic response and tend to be wide and concentrated, as opposed to smeared crack patterns observed in the laboratory [4], [5]. Shrinkage, temperature change and beam elongation all contribute to crack initiation and propagation. Some cracking is usually unavoidable, and enough reinforcement should be provided to keep the crack width tight. In-plane diagonal cracking leads to damage to finishes, and to force and stiffness degradation. Crack widths should be constrained, otherwise leading to premature yielding of the shear reinforcement. Openings and re-entrant corners lead to localised cracking, and it is unclear how localised damage affects the response, but typical detailing solutions for non-seismic applications will mitigate the problem. The DM for these limit states is crack width, and the corresponding EDPs are rebar strain for flexural cracks and shear stress to assess shear cracking.

4.2. Life Safety (Ultimate) Limit State

At the design-level earthquake, verify: (a) strain in the chord steel; (b) yielding in internal ties and crushing of struts; (c) upwards buckling of the topping due to diagonal compression; (d) positive moment failure near supports; (e) torsional failure of precast units. If the diaphragm is seen as a beam (supporting horizontal loads), clearly the same checks for the extreme compression and tension fibres apply to both a beam and a diaphragm. However, diaphragms are deep beams, which behave like D regions, and beam theory is not generally applicable. Therefore, these checks can only be made at the mid-span. The chords can be designed either to remain elastic or to yield. Nakaki [6] notes that the current code design procedures allow the chords to yield and argues that this is undesirable because: i) yielding prevents cracks from closing, encouraging buckling; ii) reduces shear strength; iii) risks allowing deformation to concentrate in a single crack; iv) it is incompatible with the codes' assumptions. These concerns seem to be supported by the observations by Scarry [7]: "…what was observed in Christchurch was a small number of wide cracks, with very limited bond breakdown. As a consequence, the strain in the reinforcing became excessive, leading to bar fracture." The EDP is rebar strain; modelling should account for initial strain and beam elongation, so fibre-section non-linear elements should be used. Compression and tension fields develop within the diaphragm and their response can be defined from strain in the steel and concrete [8], or correlated with forces from an elastic model. It has been observed in two-stage floor construction, where a thin topping is cast over prefabricated planks, that the topping tends to buckle under in-plane compression forces [9]. Thus, the EDP for diagonal crushing/buckling is the force in the diagonal strut. The resistance to buckling depends on the topping thickness and on the presence of vertical ties. Positive moment cracks may develop either at the end of the hollow core unit, or in line with the support [10]. The moment

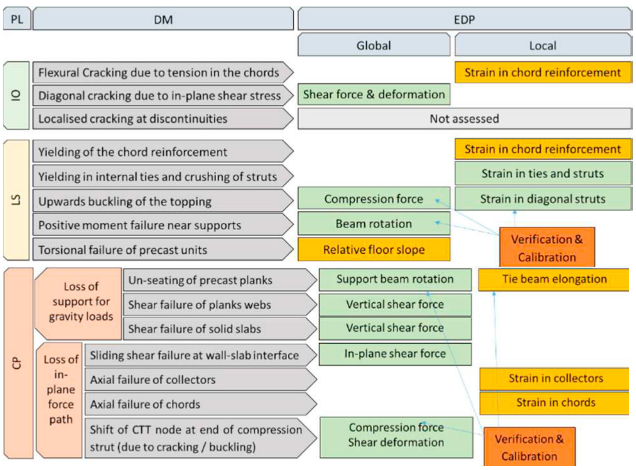

Figure 1. Schedule of limit states (LS, IO, CP), DMs and EDPs for diaphragms

equilibrates the force-couple generated by tension in the slab and the corresponding reaction along the beam centroid; the corresponding EDP is the support beam rotation. Deformation compatibility may induce torsion in the precast units under certain conditions, as described in [11]. Relative deformations between supports (the EDPs) depend on local, rather than global, inelastic demand, and should be captured from a non-linear model.

4.3. Collapse Prevention Limit State

This limit is reached when the system can no longer safely carry gravity loads, due to (a) un-seating of precast planks [8], [12]; (b) shear failure of precast planks webs [13]; or (c) shear failure of solid slabs. It is be reached if the lateral load path is severely disrupted as a result of: (d) sliding shear failure at wall-slab interface; (e) axial failure of collectors that carry the compression strut into the walls; (f) axial failure of chords; and (g) shift of CTT node at the end of the compression strut (arch) due to opening of cracks and/or buckling of the topping [11]. Beam elongation and shrinkage induce tension within the floor; the planks tend to move away from their supports, compounded by the rotation of the support beams and the frame. If the planks cannot follow this rotation and the seat is not wide enough, they may un-seat. Even if no geometric unseating occurs, the stresses resulting from prying action may lead to shear-flexural failure of the planks. The combined effect of support beam rotation and beam elongation, and the applied gravity loads, provide the necessary EDPs for these failure modes. Significant damage and collapse observed during the Northridge and Christchurch earthquakes was ultimately linked to the loss of integrity of the load path or the inability of the floor diaphragm to provide lateral stability to vertical elements. The response of the vertical systems under the redistributed lateral forces can no longer be reliably predicted, a condition regarded as near-collapse. The connection between the diaphragm and vertical elements can fail in shear (at the interface) or tension (in the collector), while diagonal struts can fail due to either buckling of the topping (compressive failure) or tensile failure of the reinforcement that equilibrates the diagonal strut. In this case, the CTT node relocates, leading to spread of damage until a new equilibrium condition is reached. Limiting compressive stresses in the topping (from an elastic model), and material strains from, for example, a non-linear reticulate model, are suitable EDPs for these failure modes.

5. Displacement-based implementation

The framework described above can be used to incorporate diaphragm design and response into DDBD [3], as described below and summarized in Figure 3. For a detailed discussion see [1]. First (step 1), relevant POs are defined. The diaphragm response is defined by the relative distribution of strength between end and interior walls, β_V, the

Step 1. For each limit state (DL, LS, CP) identify hazard level (*e.g. PGA=1.3g* for CP) and failure modes **(**Figure 1**)**

Step 2. Determine general characteristics of the response:
- Trial values: β_V=2.0, L_s=20m, B=11m, t_s=70mm, A_{se}=6400mm^2 Compare A_{sw}=175, 350 & 700mm^2/m
- Consider the effects of initial strain: ε_0=1.10mm/m
- Find if the chord's tensile strength can be developed: $T_e=(f_y-\varepsilon_0 E_s)A_{se}$=2.32MN > $f_v L t_s$ =2.14MN

  ∴ *Tensile strength cannot be developed* (A_{se} is very large)

Step 3. Define diaphragm in-plane deformation limits at each limit state, $\Delta_{d,LS}$
- Determine flexural deformations
- Determine shear deformations
- Combine shear and flexural deformation, shown in Figure 3b

Step 4. Determine the limiting drift ratio governed by: [11]
- Hollow-core plank un-seating: θ_{CP}=1.55%
- Positive moment failure of the hollow-core planks: θ_{CP}=1.34%
- Failure due to incompatible deformations: θ_{LS}=1.25%
- Torsional failure of hollow-core planks: *assumed not critical*

Step 5. Is the diaphragm performance within the target limit state?

∴ A_{sw}=700mm^2/m is required to meet target performance

Step 6. Select the governing wall and find equivalent SDOF system

Both walls have the same strength since β_V=2; for θ_{CP}=1.34% the required wall shear strength is 6MN

Step 7. Calculate horizontal forces and diaphragm internal forces; design force-controlled elements

Diaphragm design forces at over-strength: V=3.75MN, M=18.75MNm, which requires A_{sw}=350mm^2/m

Figure 2. Step-by-step PBD procedure

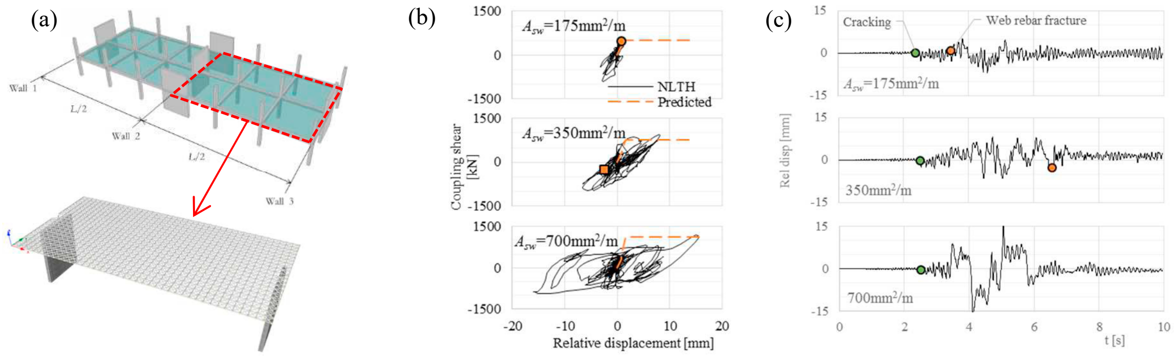

Figure 3. (a) Model used for NLTH verification; (b) calculated and simulated force-displacement curves showing CP performance point; (c) relative displacement time histories showing first occurrence of web rebar fracture at CP limit state.

aspect ratio (L_s/B), the topping thickness t_s, and the plank support details. These all relate to deformation-controlled response; therefore, they are defined at the onset (step 2), much like the wall length and plan layout are defined at the start of the normal DDBD. The diaphragm response is influenced by whether the chord strength can be developed, which in turn depends on the shear strength of the chord-web interface. If the chord is too strong for the web, performance will be impaired by wide longitudinal and diagonal cracks forming along the interface and a reduced flexural strength. The available strain in the chord rebar is also limited by previous inelastic excursions and by cracking and yielding due to beam elongation of the transverse frames. The effect of previous cycles can be accounted for by subtracting an initial strain ε_0 from the characteristic strain $\varepsilon_{LS,C}$ at each limit state**.** The initial strain, ε_0, can be due to: (a) drying shrinkage, (b) temperature, (c) beam elongation ($\varepsilon_0 = \Delta L/2L_{SP}$ with ΔL obtained as in [1], [11]), and (d) previous inelastic excursions. The available flexural strength is reduced by replacing f_y with $f_y-\varepsilon_0 E_s$. Force-deformation curves can be defined and combined into a single curve as in Figure 2b (step 3). Out-of-plane deformation limits are governed by un-seating, positive moment failure, incompatible deformation and torsional failure of the planks, so inter-storey drift limits are imposed to preclude these failure modes (step 4). The next step (5) is to determine whether the diaphragm's POs are met. When a building with multiple walls is designed using DDBD [3], the deformation profile at each limit state is determined for each wall separately. One of the walls (called *rigid*) has the smallest roof displacement, while other walls (called *flexible*) and frames are bound to the displacement profile of the *rigid* wall by virtue of the rigid diaphragm. However, if the diaphragm is not rigid, and even more when it is allowed to deform inelastically, the deformations of the *flexible* walls and frames can be larger than that of the *rigid* wall. The difference is the diaphragm displacement, computed in step 3. If the deformation of the *rigid* wall plus the deformation of the diaphragm exceeds the deformation capacity of the *flexible* wall, then the *flexible* wall governs the design. The diaphragm's performance point at any given limit state is found (step 5) as follows: i) find the diaphragm inertia forces as discussed using an appropriate method such as pESA [14] or FMR+RM [1]; ii) find the internal coupling forces for the selected value of β_V; iii) find the corresponding in-plane deformation from the force-deformation curve found in step 3; and iv) if the diaphragm performance at the deformation found above exceeds the target limit state, re-design the diaphragm (*e.g.*, increase the reinforcement or t_s), and repeat step 3. Once the performance is satisfactory, select whether the *rigid* or the *flexible* wall govern the design, and use the corresponding displacement profile to find the equivalent single-degree-of-freedom system (step 6). The horizontal inertia forces acting on the diaphragm can be computed based on the system's properties. The FMR+RM method described in [1] is recommended, with the internal forces computed using β_V. Force-controlled failure modes are checked against these forces and the corresponding elements designed. These would typically include friction shear connection to the walls, in-plane shear, and buckling of the in-situ topping. Collectors can now be designed to develop the walls' over-strength.

6. Example verification through non-linear dynamic analysis

A reticulate model [1][9] of half a diaphragm (Figure 2a) was excited with the Morongo Valley record of the 1986 Palm Spring earthquake (PEER NGA 572) scaled to PGA=1.3g. The results for three different densities of web reinforcement (A_{sw}=175, 350 and 700mm2/m) shown in Figure 2b and c, suggest that fracture of the web reinforcement occurs in all but the most densely reinforced diaphragm. Also, note how the heavily reinforced slab provides much greater ductility. The reticulate model uses non-linear fibre-section truss elements with reinforcement in the main orthogonal directions, and without reinforcement along the diagonals. The reinforcement fibres represent the corresponding proportion of web (or chord) reinforcement, and the concrete section is selected to give equivalent properties to a quadrilateral, plane-stress, element in the elastic range. The walls are modelled with plastic hinge fibre section force-based elements. The software used was SeismoStruct.

7. Conclusions

The seismic response of diaphragms has been analysed from the point of view of performance-based design (PBD) based on observed damage during actual records and past experimental results. Three limit states (immediate occupancy, life safety, and collapse prevention) have been described using suitable damage measures (DM). For each DM, engineering demand parameters are identified in the literature and, when necessary, derived as part of a broader research program. Appropriate analytical techniques to each DM are discussed. From the above, a new framework for PBD of horizontal floor diaphragms was developed.

A new method for predicting the force-deformation response of a floor diaphragm and for the incorporation of diaphragm design within a DDBD framework is presented. The method is able to predict, through very simple calculations and with good accuracy, the force-displacement response of a floor diaphragm in its own plane. The response of a floor diaphragm in its plane is influenced by the ability of the slab to transfer enough shear into the chords to develop the chord reinforcement's strength. Another important influencing factor is the web reinforcement ratio, as well as the ratio of web-to-chord reinforcement. A practical method to account for these effects is proposed.

Diaphragms with usual levels of web and chord reinforcement tend to develop a single crack at the critical region. The available ductility is limited because the plastic deformation is concentrated along a narrow band either side of the crack. Highly reinforced diaphragms, in contrast, can have much longer plastic hinge lengths, and will develop multiple smeared cracks as the plasticity is spread over a larger 'fan' area, much like a wall. Significant ductility is available in this case. However, this occurs only at unusually high levels of web reinforcement, as shown in an example. The proposed method is able to account for these differences in behaviour.

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