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On the seismic behavior and design of long span precast concrete diaphragms

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ABSTRACT: Floor systems are intended to provide diaphragm action between elements of the lateral-load resisting system during seismic response. Recently, significant design issues have been raised related to precast concrete diaphragms in long floor span structures. These issues include the possibility of larger than expected forces, joints in critical locations that may not possess sufficient strength or ductility, and the potential for large drift demand on the gravity system due to excessive diaphragm flexibility.

An analytical research program has examined the behavior of floor diaphragms in long floor span precast concrete structures, focusing on parking structures. A two stage approach was taken in the research: Nonlinear static (pushover) analyses of topped and pretopped floor diaphragms were performed to determine the effect of structural dimension and construction details on the service level stiffness and ultimate capacity of the diaphragms; and, diaphragm demands were established for structures with diaphragms of varying strength and flexibility through nonlinear transient dynamic analysis.

Design recommendations are proposed based on a comparison of the diaphragm capacities to the diaphragm demands. These recommendations include provisions for the appropriate design strength, allowable flexibility, and ductility requirements for precast concrete floor diaphragms in long span construction, including parking structures.

1 INTRODUCTION

Precast construction is commonly and effectively used for building systems with long floor spans. These structures create a demanding condition for the diaphragm by virtue of their configuration. Recent performance of diaphragms in one such long floor span structural system, the parking structure, has raised questions about the adequacy of current design procedures. A research project was undertaken to examine the seismic performance of precast concrete structures with long span diaphragms for the purpose of producing new seismic design recommendations. The research project involved: (1) determining the capacity of precast concrete diaphragms of different dimension and detail; (2) establishing the seismic demands for structures of varying diaphragm strength and flexibility; and (3) producing design recommendations for the appropriate design strength, allowable flexibility, and ductility of precast concrete floor diaphragms in long span construction.

2 BACKGROUND

The unexpected performance of precast parking structures in recent earthquakes (Iverson and Hawkins 1994) has led to a reexamination of the seismic design procedures in-place for diaphragms in general, and precast diaphragms in particular. Currently, diaphragms are designed in U.S. codes (UBC 1997) for a diaphragm design force, F_{px}, as given by code equivalent lateral force (ELF) distributions (See Fig. 1a); diaphragm reinforcement (chord, collector, web) is sized using a horizontal plate girder analogy (See Fig. 1c) in which the web reinforcement is to carry the entire shear and the chord steel is to carry the entire in-plane flexure (PCI 1999). These procedures rely on elastic diaphragm response but may not accomplish this goal (Nakaki 2000). Researchers and practitioners have therefore advocated prescriptive elastic diaphragm design (Ghosh 1999). However, evidence exists to show that establishing a reliable and economical elastic design may be difficult (Fleischman et al 1998), and even so may not produce acceptable drift performance for flexible diaphragm structures (Fleischman et al 1996). A design approach that targets performance requirements may be a viable alternative.

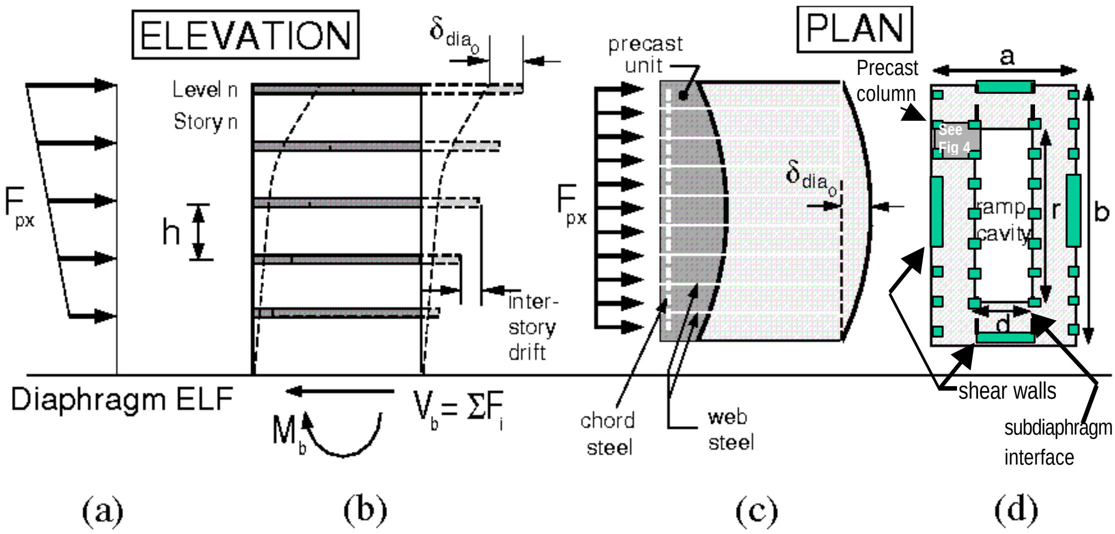

Figure 1. (a) Diaphragm ELF; (b) interstory drift; (c) typical precast diaphragm; (d) parking structure plan.

2.1 Issues related to elastic diaphragm design of long floor span structures

Diaphragm flexibility modifies the dynamic properties of both wall and frame structures. As expected, the dynamic load distribution for flexible diaphragm structures differs from rigid diaphragm structures (Fleischman and Farrow 2001). However, the greatest dissimilarity with the current ELF design pattern occurs in inelastic wall structures following formation of a base plastic hinge. Instantaneous effective centroids consistently occur well below the centroid of the ELF pattern, leading to larger than anticipated diaphragm forces during seismic response (Rodriguez et al 2002), including less frequent but extreme force events (Fleischman et al 2002). The potential for these extreme forces is greater in lower levels of the structure, in opposition to the ELF pattern. Extreme diaphragm force values do not develop in frame structures, however, the force distributions can possess higher values at lower levels.

The significant differences found between actual and code-based diaphragm forces led to a statistical examination of seismic demands for code-designed structures with flexible diaphragms (Fleischman et al 2002). Demands were established for seismic hazard at two levels corresponding to 10% and 2% probability of exceedence in 50 years, termed design basis (DBE) and maximum considered earthquake (MCE), respectively. The studies indicated that current diaphragm designs do not assure elastic diaphragm behavior for the high seismic zone DBE. The drift, force and deformation demands for these inelastic diaphragm structures were found to be higher in lower regions of the structure, particularly for wall lateral systems (See Fig. 2). Based on comparisons with expected behavior, it was concluded that current designs of flexible diaphragm structures could produce inadequate seismic performance due to: (a) exceedence of acceptable drift limits; or (b) large unintended ductility demand on the diaphragm. Hence the recommendation of prescriptive elastic design seems warranted.

Further studies were performed to determine the design strengths required to produce elastic diaphragm behavior. These studies indicated that substantial increases in current design strength were required to assure elastic behavior in a MCE (thick solid line in Fig. 2b). Given that the diaphragm is a portion of the structure not typically considered for special costs, these increases may be economically unattractive. Furthermore, elastic diaphragm behavior was found to be no guarantee of adequate drift performance (See Fig. 2a). For these reasons, an approach was pursued in which elastic behavior is enforced only to the DBE, an outcome realized through modest design force increases (thin solid line in Fig. 2b). Clearly, however, this design approach requires the accommodation of inelastic behavior during a MCE event, i.e. it must include provisions for ductility. Given the typical configurations of long floor span structures, including parking structures, the design must also address gravity system drift. The need to address multiple objectives regarding elastic behavior, drift performance, and ductility demand lends itself to concepts associated with a performance-based design approach.

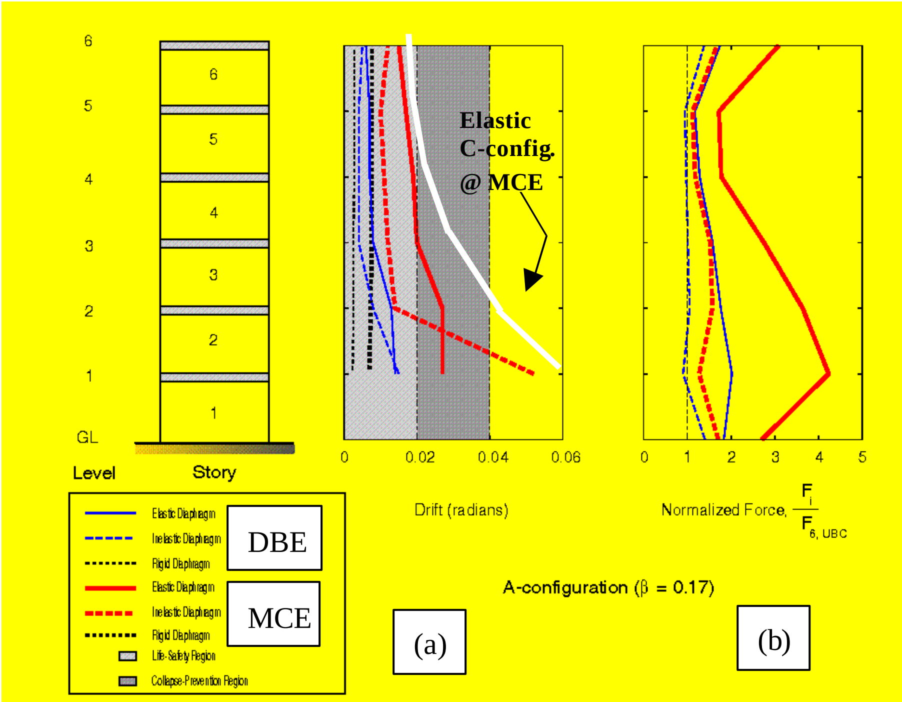

Figure 2. Current code and elastic diaphragm demand profiles: (a) interstory drift; (b) force.

2.2 Design approach using performance requirements

Performance requirements were created using drift and damage acceptance criteria based on a recent rehabilitation document for existing structures, the FEMA 273 guidelines (NEHRP 1997). These guidelines provide the most explicit accounting yet of expected performance for diaphragms. Two main FEMA 273 structural performance levels exist: life-safety and collapse-prevention. As such, the following performance was selected as the basic objective of the design approach: (1) Elastic diaphragm behavior is the DBE target while exhaustion of the diaphragm's probable deformation capacity is permitted in a MCE; and (2) Gravity-system elements remote to the lateral-system are to remain within life-safety drift limits for the DBE and collapse-prevention drift limits for a MCE.

The appropriate design strength meeting the performance requirements, $R_{n,req}$, is shown schematically in Fig. 3; a corresponding design overstrength $\Omega = F_{req}/F_{px}$ is obtained. A flexibility index, β, represents diaphragm deformation expressed as a percentage of allowable drift: $\beta = \delta_{diao}/h\ \phi_{all}$ where δ_{diao} is the maximum in-plane diaphragm deflection at nominal design strength $R_{n,px}$; h is floor-to-floor height; and ϕ_{all} is the allowable drift, assigned a value of 0.02 radians, the transient drift angle anticipated for concrete frames at the life safety structural performance level. The frame value is applicable because β has pertinence to the drift of the gravity system. $R_{n,px}$ and δ_{diao} are indicated in Figure 3. To determine Ω as a function of β, the research examined diaphragm capacity (stiffness, strength and ductility) as well as diaphragm demand (force, deformation and drift), as described next.

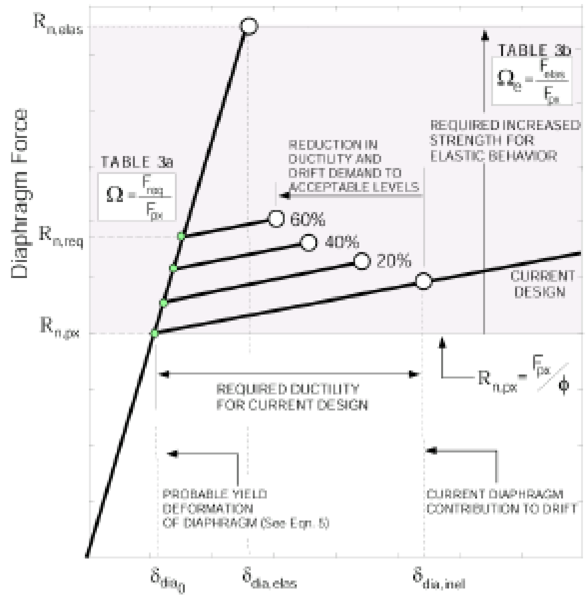

Figure 3. Schematic indicating relationship between diaphragm overstrength and drift/ductility reduction.

3 DESCRIPTION OF ANALYTICAL RESEARCH PROGRAM

Long span diaphragms of varying dimension, strength and detail are evaluated in the analytical research. A two-stage approach is adopted: diaphragm capacity is determined through nonlinear static analyses of individual diaphragms; diaphragm demand is established through nonlinear transient dynamic analysis of the entire structure. A three-bay side-by-side parking structure configuration under symmetric in-plane response is chosen for the study.

3.1 Prototype structure

A prototype parking structure (Fleischman et al 1998) developed for previous investigations is used as the baseline structure. The prototype structure represents the extreme (flexible) configuration for the study. This configuration is comparable to structures that performed poorly during the Northridge earthquake (Iverson and Hawkins 1994). Subsequent configurations are created by reducing the transverse dimension of the floor, and hence the clear span between lateral elements. Thus, length and aspect ratio change with configuration (See Table 1) where dimensions are indicated in Figure 1d.

Table 1. Floor Diaphragm Configurations

Configuration	Length, b (m)	Depth, a (m)	Subdiaphragm Aspect Ratio (r/d)	b, Diaphragm* Flexibility Index	Diaphragm Design Force, Fpx (kN)
A	58.5	57.6	1.52	0.17	3960
B	78.0	57.6	2.54	0.30	5293
C	97.5	57.6	3.55	0.51	6628

* Topped Diaphragm.

The prototype structure's gravity load system is precast floor construction, including a floor system comprised of precast double tee units spanning interior inverted tee girders and perimeter spandrels, and supported by precast columns. Topped and pretopped floor systems are evaluated (See Figure 4). For the topped systems, the floor diaphragm is completed by a cast-in-place topping slab. Steel reinforcing bars in the topping slab serve as collector and chord steel. Mechanical connectors in the flange of the precast units and welded wire fabric (wire mesh) in the topping slab serve as web reinforcement (See Fig 4a). For the pretopped systems, chord forces are carried via chord steel included within pour strips. Mechanical connectors in the pretopped units provide the web reinforcement (See Fig. 4b).

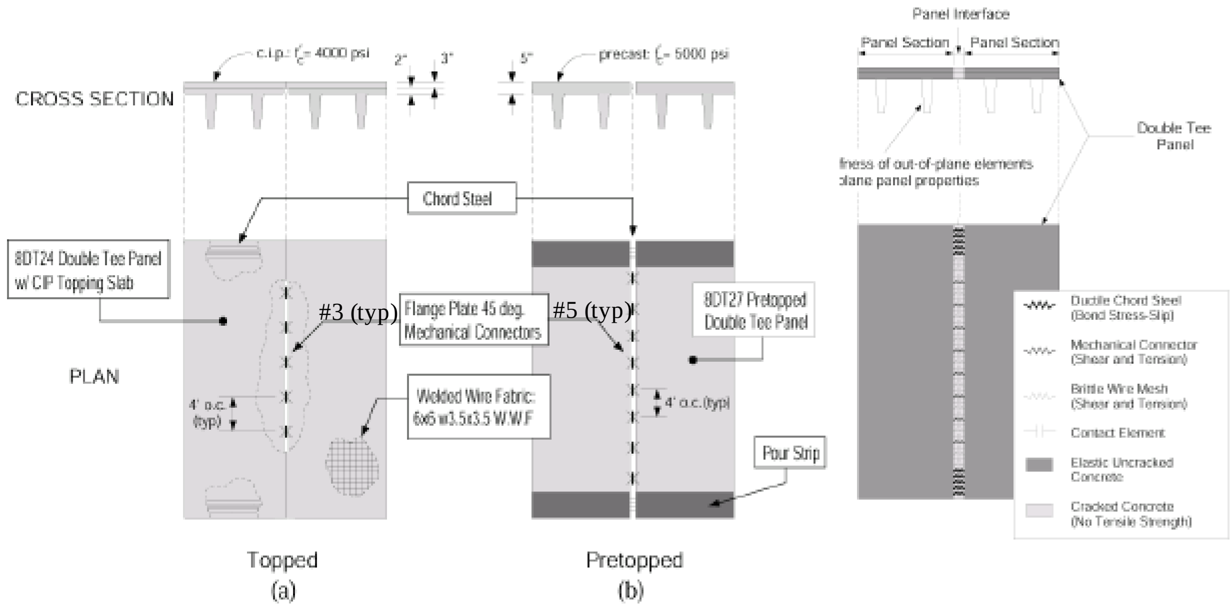

Figure 4. Web detail: (a) topped; (b) pretopped; (c) FE model. Note: 1' = 0.3048m, 1" = 2.54cm, 1psi=6.98KPa

3.2 **Study on diaphragm capacity**

Nonlinear static analyses were performed on detailed finite element models of individual diaphragms to determine stiffness, strength, and ductility capacity. The diaphragm models, developed using the finite element program ANSYS, capture the nonlinear behavior of reinforcing elements acting across joints between precast units through nonlinear springs possessing both tension (axial) and shear resistance (Farrow 2002) determined by: bond stress-slip relationships (chord steel); empirical data (shear-friction and mechanical connectors). Lateral system anchorage is assumed sufficient.

Loading of the diaphragm is generated by body forces representing uniform acceleration. The forces act in the plane of the diaphragm and are directed transversely, consistent with the direction of F_{px} in Figure 1c. The models are subjected to incrementally increasing levels of lateral loading (acceleration) until overall failure is incurred. This type of analysis, commonly used for lateral systems, is termed a nonlinear pushover analysis. Here, the pushover analyses provide the load-deformation response curve of individual floor diaphragms. Thus, estimates of the diaphragm initial stiffness and ultimate strength are obtained. Additionally, the detailed finite element models provide: (a) internal force paths; (b) the sequence and occurrence of limit state (failure) events; and, (c) relationships between global diaphragm deformation and local ductility demand. In anticipation of remediative measures, the pushover analyses are continued after nonductile limit states by replacing the failed elements

3.3 **Study on diaphragm demand**

Nonlinear transient dynamic analyses were performed on multi-degree-of-freedom structural models to determine force, deformation and drift demands (Fleischman et al 2002). Generalized coordinates are used to describe each diaphragm as a single-degree-of-freedom (DOF) oscillator. A polynomial curve-fit of deformation patterns from the finite element analyses determines the shape functions. Thus, each level of the structure is described by two DOFs representing strong-axis translation and in-plane rotation of the lateral-system; and one DOF representing horizontal translation about the diaphragm weak axis. The nonlinear time-history analyses are performed using DRAIN-2DX. The diaphragm is modeled using a bilinear spring with stiffness-degrading hysteretic properties. Elastic stiffness, yield load, and post-yield stiffness from the pushover analyses are transformed using the generalized coordinate treatment. Rayleigh damping of 2% is specified for the first and last significant structural modes.

4 ANALYTICAL RESULTS

4.1 Diaphragm pushover analyses

Figure 5 shows typical topped and pretopped diaphragm pushover curves. The following basic strength limit states were identified (failure refers to loss of load-carrying capacity as determined by deformation values at fracture or pullout in experiments): (1) failure of subdiaphragm interface joints; (2) web reinforcement shear failure; (3) web reinforcement tensile failure; and (4) chord steel flexure failure. Of these, only the chord steel flexural limit state is a desired outcome for the diaphragm in an overload situation as it is ductile, i.e., large inelastic deformations will be accommodated prior to a failure. In contrast, the shear failure limit state likely ends the diaphragm's ability to provide force transfer locally, and could lead to a progressive collapse of the entire structure.

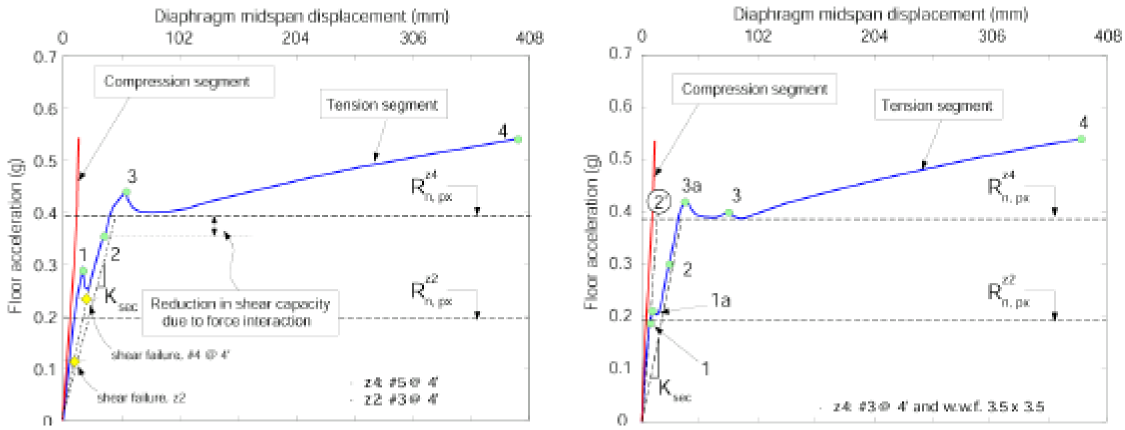

Figure 5. Pushover analyses, C-configuration: (a) pretopped; (b) topped.

The pushover analyses indicate that tension/shear force combinations can produce nonductile failure (limit state 2) at load levels below design strength for topped or pretopped diaphragms alike. The force combinations are in large part a consequence of complex load paths due to openings present in diaphragms, in this case ramp cavities. It can therefore be argued that irregular floor plans be addressed on a case-by-case basis, possibly through a rational approach such as the strut-and-tie method. However, it is as important to realize that even in cases where the shear limit state does not occur prematurely, for instance a simple diaphragm absent of force combinations, it still will coincide with the diaphragm nominal design strength ($R_{n,px}$), i.e. prior to developing the flexural capacity, because current design employs the same strength reduction (ϕ) factors for the web and the chords. This result implies that a nonductile diaphragm failure is the likely outcome in an overload condition.

Limit state 3 occurs because tensile deformation demand is imposed on the web reinforcing. These deformation demands, due to the strain-curvature compatibility associated with in-plane flexure of the diaphragm, concentrate at joints between precast units in high in-plane bending regions and become significant after the chord steel yields and significant debonding initiates. Tensile deformation capacity is not currently considered in design. Thus, standard web reinforcement, intended simply for shear transfer, can instead fracture from exhaustion of its tensile deformation capacity in regions where the chord steel yields. Welded wire fabric possesses poorer ductility characteristics than mechanical connectors, and thus fractures at a smaller tensile deformation demand (δ_{3a} vs. δ_3).

Regarding service level stiffness, deformation compliance in the panel joints decreases both flexural and shear rigidity. While shear deformation is non-negligible in topped systems, pretopped systems are significantly more flexible at service levels. Effective elastic moduli are proposed (See Table 1).

Table 2. Diaphragm elastic rigidity properties

	Topped Diaphragm					Pretopped Diaphragm			
Connector	none	#3	#4	#5	SRF[a]	#3	#4	#5	SRF[a]
E_{el} (ksi)	1400	1725	1925	2150	C_1 = AR/5	950	1275	1600	C_1 = 0.8
G_{el} (ksi)	1150	1200	1250	1300	C_2 = 0.57	300	450	580	C_2 = 0.67

[a]SRF: Stiffness reduction factor for initial elastic slope to design level secant stiffness. **Note**: 1ksi = 6.895Mpa.

4.2 Seismic demands on structures with long span diaphragms

Analyses for various diaphragm flexibilities were performed at incrementally increased strength levels over nominal until the performance requirements are met. An appropriate diaphragm design force level is therefore the larger of the values producing elastic DBE behavior and acceptable MCE drift performance. For wall structures, diaphragms with a flexibility index $\beta < 0.25$ were shown to be controlled by the former, while diaphragms with a flexibility index of $\beta > 0.25$ are controlled by the latter. The resulting overstrength factors appear in Table 3a. For reference, elastic design overstrength factors, Ωe, are shown in Table 3b. For frame structures Ω =1 with a constant diaphragm force pattern.

Table 3. Diaphragm overstrength values: (a) basic design approach; (b) elastic design.

	(a) W					(b) W_e				
b / **stories**	0.2	0.25	0.3	0.35	0.4	0.2	0.25	0.3	0.35	0.4
1	1.0	1.1	1.2	1.4	1.55	1.9	1.85	1.8	1.7	1.6
2	1.2	1.3	1.45	1.7	1.85	2.3	2.25	2.15	2.05	1.95
3	1.4	1.5	1.7	1.95	2.2	2.7	2.6	2.5	2.4	2.3
4	1.6	1.7	1.95	2.2	2.5	3.1	3.0	2.9	2.75	2.6
5	1.8	1.95	2.2	2.5	2.7	3.45	3.35	3.25	3.15	2.9
6	2.0	2.15	2.45	2.8	3.0	3.8	3.65	3.5	3.35	3.2

The Ω factors capture the important interrelation of diaphragm flexibility and diaphragm strength in describing the seismic performance of long floor span structures. These factors are intended to produce the desired DBE elastic behavior and MCE drift performance. The use of these factors will be meaningless, however, if nonductile limit states circumvent the desired diaphragm behavior. Thus, a detailing philosophy compatible with achieving ductile limit states is needed, as described next.

5 DUCTILE DETAILING IN LONG SPAN PRECAST DIAPHRAGMS

Special detailing requirements have not been a traditional part of diaphragm design. However, the approach proposed here requires detailing for the MCE event. As it may be prudent even in an elastic diaphragm design to anticipate ductility demand, a viable diaphragm design should promote the formation of a desirable mechanism in the event of an overload.

5.1 **Elimination of shear limit states**

An essential detailing requirement for precast concrete diaphragms is the elimination of nonductile failure modes involving the web reinforcement. This objective can be readily accomplished through a capacity design for the web reinforcing relative to the chord reinforcing, i.e. the use of $\phi = 0.6$ for the web reinforcement (and the collector/anchorage steel), while restoring the ϕ factor for chord steel to 0.9. This approach provides the web reinforcement sufficient overstrength to fully develop the flexural strength of the diaphragms. The ϕ factors are to be used in combination with the Ω factors. The capacity design approach may not be sufficient for cases in which maximum shear occurs in the diaphragm high bending region since web reinforcement could fail due to compatibility-induced tension deformation regardless of shear strength. Thus, the shear friction contribution of welded wire fabric should not be included in high in-plane bending regions of the diaphragm unless an elastic design is selected. The shear-friction contribution can be included in low in-plane bending regions.

5.2 **Detailing for tension limit states**

If no assurance exists that diaphragm loads will remain below design strength, the web reinforcement must possess sufficient tensile deformation capacity to meet ductility demand compatible with the achievement of a flexural limit state in the chord reinforcing. It can be argued that loss of web reinforcement in the midspan regions of diaphragms has minimal consequence since this region attracts low shear force under transverse lateral loads. However, it should be recognized that irregular floor plans, end restraint, or diagonal loading may produce significant shear forces in high bending regions of the diaphragm. Localized web reinforcement failure must be accepted in high bending regions for the MCE as neither welded wire fabric nor standard mechanical connectors possess sufficient ductility to meet the MCE tensile deformation demands at the overstrengths proposed. However, alternatives can include elastic design or the use of tension-compliant web reinforcement.

6 **SEISMIC DESIGN RECOMMENDATIONS FOR LONG SPAN PRECAST DIAPHRAGMS**

Based on the results presented in this paper, the following design recommendations are made for long floor span precast diaphragm structures in high seismic regions: (1) the use of a diaphragm overstrength factor Ω as given in Table 3 for wall lateral systems and $\Omega = 1$ for frame lateral systems; (2) a capacity design for detailing the web reinforcement by using $\phi = 0.6$ while using $\phi = 0.9$ for the chord reinforcement; and (3) acceptance of web reinforcement tension failure in regions of high bending unless elastic design or tension compliant web reinforcement is used. It is noted that the above recommendations are to be used in conjunction with findings from previous work (Fleischman et al 2002) including: (1) the calculation of a diaphragm flexibility index β based on effective elastic moduli; (2) the inclusion of the elastic contribution of the diaphragm in drift calculations; (3) a restriction on building certain configurations ($\beta > 0.4$), comparable to the longest span parking structures of recent construction, as no design strength increase exists to realize adequate drift performance; (4) a constant strength pattern based on the top-level lateral force.

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