Seismology and Structural Standards Committee

Seismology Publication

April 2005

Using a Concrete Slab as a Seismic Collector

Scope:

This document presents a methodology endorsed by the SEAONC Seismology and Structural Standards Committee on the development and use of reinforced concrete slabs as seismic collectors. "Using a Concrete Slab as a Seismic Collector" proposes an alternative design approach to the design of seismic collectors constructed of reinforced concrete. This paper is the culmination of the SEAONC Concrete Sub-Committee's work over a period of two years.

The 1997 Uniform Building Code is the base reference for this document.

Symbols and Abbreviations:

See Publication for Symbols and Abbreviations (in context)

Commentary:

The building industry, specifically the structural engineering profession, has long struggled with a reasonable method to construct reinforced concrete collector elements in parking garages, flat slab and other reinforced concrete structures that can accommodate high force demands associated with Ω_0 and other code provisions. This publication presents the reader with some alternative methods that make the design and construction of reinforced concrete collector elements more tenable for certain types of structures.

References:

Recommend Lateral Force Requirements and Commentary (SEAOC Blue Book), SEAOC, 1999.

Design Example of Seismic Resisting Diaphragms (draft copy), Dr. Joe Maffei, 2002.

Authorship:

This publication represents views endorsed by the Seismology Committee. It may differ from the views, methods, policies and interpretations of some building authorities. Engineers are cautioned to ascertain such views, methodologies, policies and interpretations in advance of design.

The document was approved by the SEAONC Seismology Committee March 2005.

2004-2005 SEAONC Seismology Committee Leadership:

Kevin Moore, S.E., Chair   Mark Moore, S.E., Vice-Chair   Gary Mochizuki, S.E., Past-Chair

Acknowledgement

The following persons actively participated in the preparation of this document and the chair of the sub-committee acknowledges their efforts and expresses his sincere appreciation and gratitude for their hard work. The Concrete Sub-Committee appreciates and thanks the SEAONC Seismology and Structural Standards Committee and the SEAOC Seismology Committee for their review and comments on this paper. The Committee also extends it gratitude to Dr. S.K. Ghosh for reviewing this paper.

Portions of this document were adopted, by permission from the author, from the draft document for *Design Example of Seismic Resisting Diaphragms*, by Dr. Joe Maffei. Concrete Sub-Committee members wish to acknowledge his contributions to this document. For a more in-depth discussion of diaphragm and collector design issues the reader is referred to Dr. Maffei's document.

Lawrence Liao, S.E.
Ray Bligh, S.E.
Jessica McInerney, P.E.
Bruce Starkman, S.E.
Ted Zsutty, S.E.
Vahid Sattary, S.E.
Joe Maffei, S.E.

Badri K. Prasad, S.E.
Chair – SEAONC Concrete Sub-Committee

1. Introduction

Collectors are elements of the floor or roof structures that serve to transmit lateral forces from their location of origin to the vertical seismic-force-resisting elements (e.g., walls or moment frames) of the building. Typically, collectors transfer earthquake forces in axial tension or compression. When a collector is a part of the gravity-force-resisting system, it is designed for seismic axial forces along with the bending moment and shear force from the applicable gravity loads acting simultaneously with seismic forces.

When subjected to lateral forces corresponding to a design earthquake, most buildings are intended to undergo inelastic, nonlinear behavior. Typically, the structural elements of a building that are intended to perform in the nonlinear range are the vertical elements of the seismic-force-resisting system, such as structural walls or moment frames. For the intended seismic response to occur, other parts of the seismic-force path — particularly floor and roof diaphragms, collectors and their connections to the vertical seismic-force-resisting elements — should have the strength to remain essentially elastic during an earthquake. This is the intent of most building codes and for this reason diaphragms and collectors should be designed for larger seismic forces than those for which the walls or moment frames are designed.

2. Building Code Requirement

The seismic design requirements for collector elements are addressed in Chapter 16 of the 1997 UBC as follows:

> ***"1632.2.6 Collector elements.*** *Collector elements shall be provided that are capable of transferring the seismic forces originating in other portions of the structure to the element providing the resistance to those forces."*
>
> > *"Collector elements, splices and their connections to resisting elements shall resist the forces determined in accordance with Formula (33-1). In addition, collector elements, splices, and their connections to resisting elements shall have the design strength to resist the combined loads resulting from the special seismic load of Section 1612.4."*

The "Special Seismic Load Combinations" of Section 1612.4 includes the estimated "Maximum Earthquake Force", E_m, which is defined by Equation (30-2) as the basic seismic design force multiplied by an overstrength factor, Ω_o.

The intent of the Ω_o–amplification factor is to allow for the likely overstrength of the vertical seismic-force-resisting elements so that major yielding does not occur in collectors and their connections prior to yielding and inelastic response at the vertical elements of the building's seismic-force-resisting system.

3. Collector Design Issues

The traditional seismic design practice was generally based on providing discrete collector elements having special reinforcement to transfer the entire required seismic load to the end of the seismic-resisting vertical element. This practice was partly due to, and based on, the low collector force requirements in the older seismic design codes. (The previous editions of Uniform Building Code prior to the 1997 edition required that collectors be designed only for the diaphragm design force without multiplying by the system overstrength factor to account for the maximum earthquake effect).

Applying the system overstrength factor per 1997 UBC effectively increases the collector load demand by approximately 200% to 300%. It often becomes impractical to provide a collector element that is concentric with the shear wall/moment frame that has adequate strength to resist the full seismic force and transfer it to the ends of the vertical seismic-force resisting element. Additionally, by concentrating all the collector reinforcement in a small region in line with the wall, the elastic stiffness of the adjacent floor slab may be underestimated. This "traditional" methodology can result in a condition where the floor slab having larger area and being stiffer than the collector element, would initially resist the collector seismic tension. If the floor slabs are not adequately reinforced for the seismic tension force, significant cracking may occur, until the reinforced collector element starts yielding and reaches its full strength.

4. Illustration of Methodology

The purpose of the following discussion is to illustrate an alternative collector design approach where part of the seismic load is resisted by the reinforcement directly in line with the shear wall, which transfers the force directly to the end of the shear wall. The balance of seismic force is resisted by reinforcing bars placed along the sides of the wall and uses the slab shear-friction capacity at the wall-to-slab interface to transfer seismic forces to the wall (for an example, see Figure 1). (In this document shear wall is used to represent vertical seismic force-resisting element, the condition for moment frames or other systems are similar).

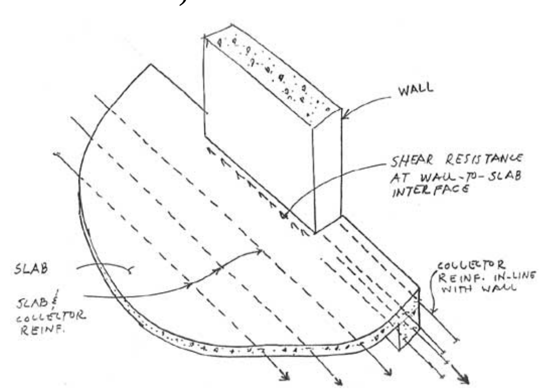

Figure 1 – Perspective View of Wall and Collector

It is noted that where the slab is adjacent to a shear wall, and is used to resist seismic "collector forces", there is an eccentricity between the resultant of collector force in the slab and shear wall

reaction. This eccentricity can create secondary stresses in the slab transfer region (or "diaphragm segment") adjacent to the wall (see Figure 4). For a complete and consistent load path design, the effect of seismic force eccentricity in this "diaphragm segment" must be checked to determine that adequate reinforcement is provided to resist the induced stresses. Section 6.6 presents a method for determining the magnitude of the eccentric force and the required added reinforcement in the slab transfer region; an application of this method is illustrated in the design examples (attached).
The design examples use a rational load path for collector forces and outline a design process that satisfies the 1997 UBC requirements. The buildings used as a basis for the design examples were deliberately selected to be simple and structurally regular. These examples do not present a comprehensive list of all collector design issues, nor do they provide a commentary on all of the relevant building code requirements.

Two design examples are provided; the first example illustrates the collector design for a post-tensioned roof slab in a building having concrete bearing shear wall seismic force-resisting system. The second example considers the same structure without post-tensioning.

5. Slab Effective Width

A key design issue in this approach is to determine the effective width of slab adjacent to the shear wall that is used to resist collector forces. Where a narrow effective width is assumed, eccentric force effects become small, but more reinforcement may be required to drag the collector forces in-line with the wall. On the other hand, if a wide slab width is used as collector, more force can be transferred through the slab, reducing reinforcing bar congestion at the end of the wall; however, secondary stresses caused by force eccentricity would be larger.

The procedure outlined in these examples proposes to treat the choice of the effective slab width as a design parameter to be selected by the designer. The first example uses an assumed 45- degree influence line to determine the effective slab width, in the second example the effective slab width is arbitrarily selected to be equal to the wall length (see Section 6.3). For both cases, the resulting force eccentricity should be checked and, if required, additional reinforcement should be provided in the slab transfer region.

6. Collector Design Procedure

The following is a suggested outline for collector design procedure

6.1 Determine Collector Design Forces

Determine the seismic forces distribution to the vertical seismic force-resisting members by conventional analysis and draw collector force diagram along the line of seismic force-resisting members (Figure 2).

Note that a linear variation of collector force along the line of vertical seismic load resisting member assumes that the tributary width of the slab is constant and the collector (which, in this case refers to both the element in line with the wall and its adjacent slab section) is stiffer than the other connecting members.

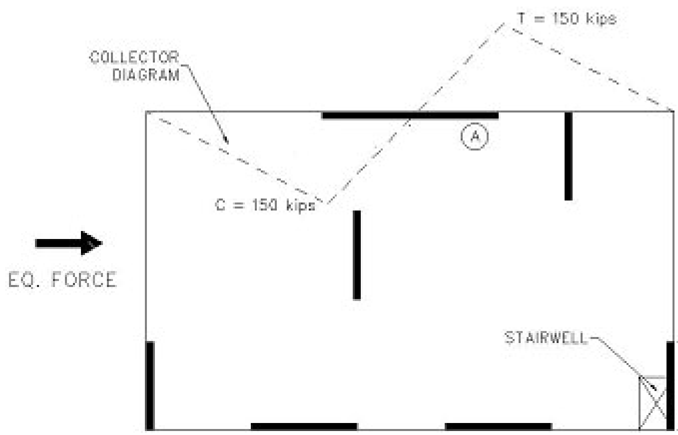

Figure 2 – Collector diagram for Wall A

6.2 Determine the Steel Area Directly in Line with Shear Wall

It is proposed that the section of the collector that is directly in line with the wall be designed for all the applicable gravity load demand plus a reasonable portion of the total collector force that the designer can select considering the required number of reinforcing bars and practical limitations of reinforcing bar congestion at the end of the wall. Then, the balance of the collector force will need to be resisted by the adjacent slab section in accordance with the following design procedure.

6.3 Select Effective Slab Width to Resist Collector Forces

Example No. 1 presents a method to assign the effective slab width to resist collector forces based on an assumed 45-degree influence line, which originates from the "point of zero force" along the collector force diagram (see Figure 3).

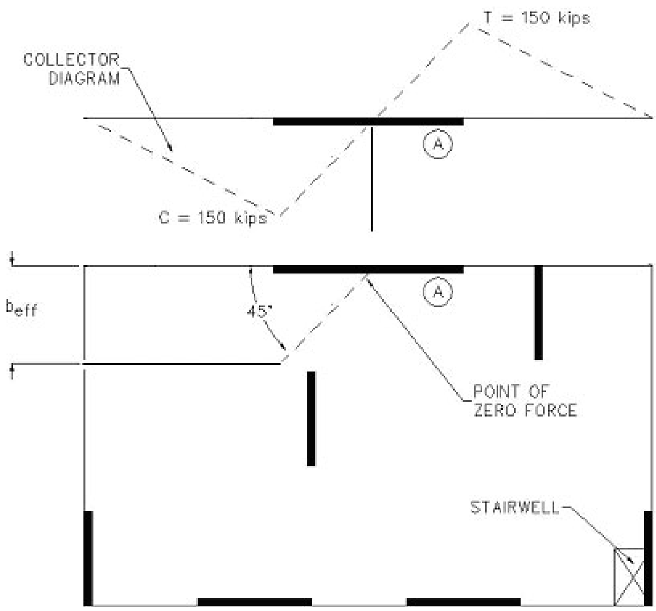

Figure 3 –Effective Slab Width as Collector

Example No. 2 arbitrarily uses the following equation to assign an assumed effective slab width.

$$B_{EFFECTIVE} = t_{WALL} + n \cdot \left(\frac{L_{WALL}}{2} \right)$$

where, n is the number of sides that slab exists adjacent to the collector.

Also, it is proposed that the designer may choose any other slab width that satisfies the check for secondary eccentric stresses as outlined in Item 6.6, below.

6.4 Determine Required Steel Area to Resist Collector Tension

Determine the net tension force, T_{NET}, and the required steel area, A_S, at each section along the collector. (In these examples the required steel area is calculated only at the maximum force location). For most reinforced concrete slabs the net tension force is equal to the calculated collector tension, F_T. For pre-stressed concrete sections, the UBC Section 1921.6.12 allows the use of the slab pre-compression force from unbonded tendons, F_{PT}, when calculating T_{NET} as it is illustrated in Example No. 1. Hence:

$$T_{NET} = F_T \Omega_O - F_{PT} \qquad \text{UBC Section 1921.6.12}$$

$$A_S = \frac{T_{NET}}{\phi \cdot f_y}$$

where, F_T is the calculated collector tension force, Ω_O is system overstrength factor from UBC Table 16-N, ϕ is capacity reduction factor per UBC 1909.3.2.2, and f_y is yield strength of reinforcing steel.

The reinforcing area, A_S, represents the total area of the required reinforcing steel. Part of this steel may be placed in the slab element directly in-line with the wall (see Item 6.2), and the balance may be distributed throughout the effective slab width adjacent to the wall. The collector reinforcement shall be placed, as much as practicable, symmetrically about the centroid of the concrete section in order to prevent additional out-of-plane slab bending stresses. Additional calculation shall be performed to determine the effect of collector force eccentricity relative to the shear wall reaction and required reinforcement, as discussed in Item 6.6.

For pre-stressed concrete sections, the magnitude of pre-compression force, F_{PT}, depends on the assumed effective slab width, which should be selected based on engineering judgment and verified by calculation to determine the required added reinforcement to resist effect of eccentric forces and secondary stresses.

6.5 Check Collector Compression Stress

Determine the total compression force, C_{NET}, and check concrete compressive stress at each section along the collector. (In these examples concrete compressive stress is checked only at the maximum force location). For most reinforced concrete slabs the total compression force is equal to the calculated collector compression, F_C. For pre-stressed concrete sections, since pre-compression force, F_{PT}, was used to reduce the net tension, T_{NET} (see Item 6.4), it must be accounted for in calculating to the total compression force, C_{NET}. Hence:

$$C_{NET} = F_C \Omega_O + F_{PT}$$

The 1997 UBC design concept for collectors in compression is that transverse reinforcement must be provided where large collector compressive forces exist, as stated in the following code section:

> ***"1921.6.2.**3 Structural-truss elements, struts, ties and collector elements with compressive stress exceeding $0.2f'_c$ shall have special transverse reinforcement, as specified in Section 1921.4.4, over the total length of the*
>
> *element. The special transverse reinforcement may be discontinued at a section where the calculated compressive stress is less than $0.15f'_c$. Stresses shall be calculated for the factored forces using a linearly elastic model and gross-section properties of the elements considered."*

The Concrete Subcommittee interprets that the $0.2f'_c$ stress criterion should apply to collector forces before they are magnified by the Ω_o factor. If Ω_o-magnified forces are used, then the $0.2f'_c$ criterion becomes $0.45f'_c$ for consistency with the intent of the UBC and established collector design practices. (It should also be noted that ACI has adopted a code change for the upcoming 2005 ACI 318 code that clarifies this requirement). The limit of 0.45 was selected based on the following arithmetic:

$$0.2\ (\Omega_o = 2.2 \sim 2.8\ ) = 0.44 \sim 0.56$$

Hence:

$$\frac{C_{NET}}{A_C} \le 0.45 \cdot f'_c \qquad \text{"Modified" 1997 UBC 1921.6.2.3}$$

where C_{NET} is the total compression force, and A_C is the gross cross-sectional area of the effective concrete section in compression. It should be noted that the magnitude of A_C depends of the assumed effective slab width. Since the resultant of concrete compression forces would be eccentric relative to the shear wall, additional calculation shall be performed to determine the effect of collector force eccentricity relative to the shear wall reaction and required reinforcement, as discussed in Item 6.6.

6.6 Check Diaphragm Segments for Eccentricity

For conditions where all or part of collector reinforcement is placed at the sides of the shear wall, the transfer region (or the diaphragm segment adjacent to the wall) should be designed to resist the seismic shear and in-plane bending moment resulting from the eccentricity of the portion of collector force that is not transferred directly into the end of the shear wall. In keeping with the code intent to design collectors and their connections for the "maximum expected seismic force", E_m, the stresses due to collector eccentricity in that diaphragm segment adjacent to the wall shall be determined using an overstrength amplification factor, Ω_o.

It should be noted that the specific diaphragm configurations, such as, slab thickness variations, location of framing members, opening patterns, and other local conditions, could produce a complex stress state in the transfer region of eccentric collectors and affect the required slab reinforcement. Due to the vast variation of diaphragm configurations in actual design situations, a single all-encompassing design procedure could not be presented to be applicable to all possible cases. Hence, for the discussions in this section only an example of a simple diaphragm segment is provided to illustrate the general design requirements and a simplified rational procedure to satisfy these requirements.

Figure 4 shows an idealized partial plan at the edge of a diaphragm with the seismic resisting wall "*a-d*" and the seismic collector located eccentrically at a distance "e" relative to the wall. The figure also shows the diaphragm segment adjacent to the wall with internal forces acting on the free-body "*abcd*", (except the components of tension/compression forces perpendicular to the free-body diagram are ignored in this example for sake of simplicity.) The collector design force is designated as F_c; and in a general sense it consist of a compression and tension collector portions and the portion of diaphragm shear force along Line *bc*, respectively, designated as $(F_c)_{comp}$, $(F_c)_{tens}$, and V_d. Considering the seismic amplification factor and collector eccentricity, the maximum eccentric moment acting on the free-body *abcd* is calculated as:

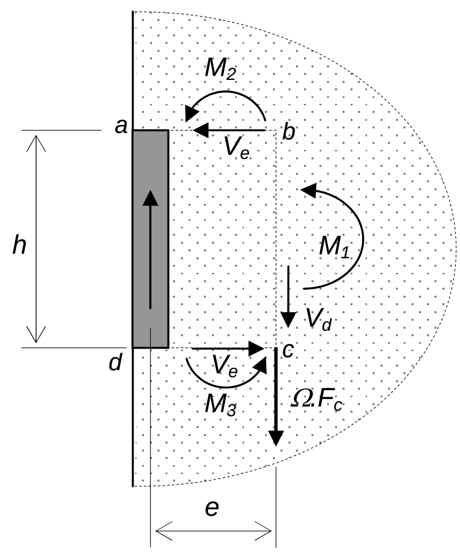

Figure 4 – Diaphragm Segment Plan

$$F_c = (F_c)_{comp} + (F_c)_{tens} + V_d$$
$$M_e = (\Omega_o F_c)e$$

The applied eccentric moment should be resisted by the combined action of all the diaphragm internal forces, thus:

$$M_e = V_e h + M_1 + M_2 + M_3$$

where, the magnitude of internal forces, V_e, M_1, M_2, M_3, could be calculated in a rigorous analysis in accordance with their relative stiffness. However, for practical design purposes, the calculation can be greatly simplified by using capacity design concept. The following discussion presents a possible procedure for determining the diaphragm segment design capacities. For example, the moment capacity of the slab region under direct tension from collector force, i.e. moment M_3 in the Figure 4, may be conservatively neglected. Furthermore, the shear capacity V_e shall be calculated using only the capacity of shear reinforcing bars and neglecting the contribution of concrete section under tension. Hence, the strength limits for the shear force V_e, and bending moment M_2, are determined as:

$$M_2 = \varphi f_y A_{S2}(je)$$
$$V_e = \varphi A_{sv} f_y$$

where, A_{S2} is the reinforcement areas perpendicular to section *ab*, *(je)* is the effective moment arm, and A_{sv} is the smaller of the reinforcing bar area parallel to sections *ab* and *dc*. Then, the required flexural strength, M_1, can be calculated as:

$$M_1 = M_e - (V_e h + M_2)$$

(Alternatively, it may be assumed that the bending moments M_1, M_2, and M_3 would reach their allowable strength limit and calculate the required shear, V_e, to satisfy the basic equilibrium of forces. A similar procedure may be used for other combinations of bending moments and shear force.)

For conditions where the eccentricity, e, is small relative to the dimension h, it is reasonable to assume that the relative stiffness associated with the actions M_1 and V_e is much larger than the other actions; hence, without being too conservative, the contribution of the moments M_2 and M_3 may be neglected, which simplifies the equation for M_1, to the following:

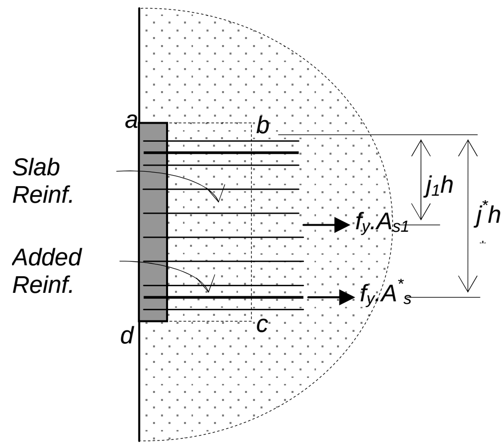

Figure 5 – Diaphragm Segment Reinforcement

$$M_1 = M_e - V_e h$$

The required moment capacity, M_1, can be computed by taking into account the effect of the slab's distributed reinforcement that is provided for gravity loads, but is in excess of what is needed to resist seismic load combinations. For cases where the available distributed reinforcement is not adequate to satisfy the required strength, supplemental reinforcing steel should be provided at the eccentric force transfer zone. Figure 5 shows an arrangement of various reinforcing bars perpendicular to section bc; and illustrates the terms used in the following computation for moment M_1.

$$M_1 = \varphi f_y \{A_{S1} (j_1 h) + A^*_S (j^* h) \}$$

where, A_{S1} is the available area of the distributed slab reinforcement perpendicular to the section bc that can be used for seismic load combination, $(j_1 h)$ is its effective moment arm; A^*_S is the area of supplemental reinforcement, and $(j^* h)$ is the effective moment arm of the supplemental steel.

6.7 Check Diaphragm Segment Shear Strength

The slab shear stress demand should be checked for the shear force transfer region adjacent to the wall and, where required, additional reinforcement shall be provided. Two shear force transfer mechanisms should be considered. First, slab shear strength should be evaluated in accordance with UBC Section 1921.6.5, and considering the contribution of all available slab reinforcement. It should be noted that the common diaphragm proportions and support layout often create a condition that the diaphragm shear strength is less than the shear corresponding to the nominal flexural strength. Thus the strength reduction factor, φ, for shear should be taken as 0.60 according to 1997 UBC Section 1909.3.4.1.

$$V_u \leq \varphi\, A_{cv} \left(\alpha_c \sqrt{f'_c} + \rho_n\, F_y \right) \qquad \text{UBC Equation (21-7)}$$

where, A_{cv} is the net area of the concrete section bounded by the slab thickness and length of the wall, α_c is the ratio of the width to length of the diaphragm segments, which in this case is equal to effective slab width to the length of the wall, and ρ_n is the ratio of distributed shear reinforcement perpendicular to the wall.

Special attention must be given to A_{cv} when the vertical seismic force-resisting member is not continuously connected to the diaphragm. For example, for an exterior wall that is 25 ft long, but is located adjacent to a 10 ft wide stair opening, then the length used in calculating the shear area is 15 ft.

6.8 Check Shear-Friction at Wall-to-Slab Interface

The strength of transfer mechanism by shear friction at the face of the supporting wall and/or frame should be checked. For this mechanism the potential sliding plane should be identified; for most practical design cases the potential sliding plane is taken as the vertical plane at the interface between the wall and the slab. In accordance with the requirements of 1997 UBC Section 1911.7.7 the shear-friction reinforcement can include all reinforcement that crosses this plane, as long as it is not used to resist direct tension. Hence, the area of the required shear transfer reinforcement, A_{VF} per foot of wall length is calculated as:

$$A_{VF} = \frac{V_n}{\mu \cdot fy \cdot L_w} \qquad \text{1997 UBC 1911.7.4.1}$$

where, L_w is length of interface between the wall and the slab.

7. Other considerations for collector design

7.1 Using gravity slab reinforcement

The collector design procedure in these examples assumes that a portion of seismic collector load is resisted by the shear strength along the wall interface with the floor slab. For this load path the slab longitudinal reinforcement parallel to the shear wall can be used to transfer the balance of the collector force to the side of the wall.

For design efficiency, a portion of the slab reinforcement that is provided for gravity loads, but is in excess of what is needed to resist seismic load combinations, can be used to resist collector or diaphragm forces, provided they meet the special detailing requirements for seismic-force-resisting systems. The seismic detailing requirements are more stringent for this condition than for typical non-seismic slab design. These detailing requirements account for conditions resulting form seismic load reversals. The main detailing issues in regard to using slab gravity reinforcement for seismic diaphragm include the following:

- Minimum diaphragm reinforcement ratio and required bar spacing;
- Diaphragm reinforcement splices and development length;
- Symmetric distribution of diaphragm reinforcement at top and bottom of section to resist seismic net axial force without inducing additional slab bending moment.

7.2 Check Local Stress Concentration at face of Wall

The stress concentration at a face of the LFRE should be studied. It is the sub-committee's opinion that the local failure will trigger the re-distribution of the load and eventually the interface between the LFRE and slab will carry most of the load. The local failure will not cause collapse or have significant impact on the load transfer from the strain compatibility of the slab.

7.3 Detailing of reinforcement

Planar elements such as shear walls and diaphragm slabs have a better post-cracking behavior if the reinforcing is reasonably distributed over regions of high shear and axial stress rather than being concentrated in narrow groups near the edges of these elements. Distributed reinforcing allows the formation of multiple narrow cracks over the stressed region, while the stiffening effect of concentrated group reinforcing results in a few wide cracks with possible localized spalling. Note that the current shear wall provisions allow and encourage the use of vertical reinforcing distributed over the wall section rather than in concentrated boundary elements. Similarly, provision of distributed steel in an assigned effective width of a collector element can result in better post cracking performance than if the collector is made up of large diameter bars in and closely adjacent to the vertical lateral force resisting element.

Using smaller bars in larger amounts to spread over a wide band of slab will result in a better stress distribution than using smaller quantities of big bars. Since a wide slab band is used in collector design, this check seldom becomes critical. This idea is supported from the study of finite-element analysis of examples indicating the stress distribution is spread out in a wide band across the slab section. Even with the existence of shrinkage cracks, the reinforcement in the slab keeps the diaphragm inertia force distributed in a relatively wide band until the failure line forms. It is believed that the concentration of bars in collectors changes the force distribution; it attracts the force to the narrow band formed by these bars and causes early overstress in the narrow region.

APPENDIX "A"

COLLECTOR ELEMENT TIE REQUIREMENTS:

The following discussion is to provide a justification for not providing closed ties (such as those required for gravity load bearing ordinary tied columns) in collector elements when gross section compression stress due to $\Omega_o E_h$ does not exceed $0.45f'_c$.

In order to compare collector element stress levels and related capacities, the following specific, but reasonably typical, example will be used.

Given a collector element with: gross section A_g, $f'_c = 5$ ksi, $f_y = 60$ ksi,
modular ratio $n = 7$, steel ratio $\rho = 0.01$, steel area $A_s = \rho A_g$

Transformed Area $A_t = A_g + (n-1)\,\rho\,A_g = 1.06\,A_g$

Nominal Compressive Strength $P_n = [0.85f'_c + \rho\,f_y]A_g$, (concrete strain $\varepsilon_c = 0.003$),
with Section Stress $f_n = P_n/A_g = [0.85f'_c + \rho\,f_y] = 4.85$ ksi

For the condition where the maximum expected seismic load $\Omega_o E_h$ causes a gross section compression stress $\Omega_o E_h/A_g = 0.45f'_c$, the limit that requires confinement ties;

Concrete stress $f_c = \Omega_o E_h/A_t = \Omega_o E_h/(1.06)A_g = 0.45f'_c/(1.06) = 0.42f'_c$
Steel stress $f_s = nf_c = 2.94f'_c = 14.7\text{ ksi} = 0.25f_y$

Demand / Nominal Capacity ratio = 0.45(5ksi)/4.85ksi = 0.46

Conclusion: Actual Maximum Seismic Load could be more than doubled before reaching nominal strength where concrete strain = 0.003 requires special confinement ties. As long as the gross section stress is below $0.45f'_c$, the steel compression stress is less than one fourth of yield stress and the concrete stress is well below the crushing stress of $0.85f'_c$. At these stress levels, the buckling of the bars in compression can be restrained by the non-crushed integral concrete cover. This assumes that the actual maximum seismic load will not greatly exceed $\Omega_o E_h$ and that the collector steel has not been elongated by strains beyond the ension yield stress f_y. Note that the collector steel has been designed to resist tension
force $\Omega_o E_h$ with stress equal or less than ϕf_y .

Comparison of Failure Consequences: Ordinary tied columns that support gravity loads (not seismic loads which are expected to exceed the design value E_h) require square ties that are spaced at $16d_b$ in order to prevent bar buckling.

Safety against overload is provided by the low ϕ = 0.70 and 1.2D + 1.6L, such that concrete crushing beyond 0,85f'_c and/or steel yield are not likely to occur.

However if overload were to occur, the consequence would be loss of vertical load capacity and collapse. Also note that permanent dead load creep effects in the concrete can increase bar stress and increase the potential for bar buckling in columns, whereas these effects are not present in the collector element with short time seismic loading.
For the case of a collector element without closed ties, safety against buckling is provided by the compression stress limit of 0.45f'_c with the maximum expected load of $\Omega_o E_h$. In the unlikely event of overload, the bars could buckle and local damage could occur. However the collector element would retain concrete compression capacity and steel tension capacity. The loss of vertical load support and local collapse in the damaged region of the slab would be prevented by the catenary suspension action of the tension steel.

Therefore it is concluded that closed ties are not required for collector elements with stress not exceeding 0.45f'c since adequate safety is provided against overload and in the case of overload, bar buckling failure does not result loss of function of the collector or a life endangering collapse.