GUIDE TO THE DESIGN OF COMMON IRREGULARITIES IN BUILDINGS

2012/2015 IBC® and ASCE/SEI 7-10

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Introduction

It is not uncommon to encounter buildings with structural irregularities. The plan layout, aesthetics, space planning and other issues often result in buildings having irregularities.

Past earthquakes have demonstrated that buildings having irregularities have performed poorly and suffered greater damage when compared with buildings having regular configurations. This is due to the fact that the seismic demands imposed on the structure by the ground motion generally tend to be well distributed throughout the structure. This in turn will lead to the dispersion of energy dissipation and damage. However, this is not the case for irregular structures wherein the seismic demands tend to concentrate in the zone of irregularity, resulting in major damage and failure of structural elements in these areas.

The analysis methods that are typically used in the design of structures cannot capture adequately the demands imposed on the irregular structure and thereby lead to deficient design in the zone of irregularity.

Structural irregularities can be classified into two types:

a. Horizontal irregularities

b. Vertical irregularities

Table 12.3-1 describes the circumstances under which buildings must be designated and having a plan irregularity. Horizontal irregularities that occur due to torsion, reentrant corner and diaphragm discontinuity require larger force transfer through the diaphragm. The most critical of the discontinuities is the out-of-plane offset (Type 4 horizontal irregularity) in the lateral force-resisting system. Such offsets impose large demands on the diaphragm as well as the lateral system. The example problem in this design guide gives a step-by-step method for designing diaphragms, collectors and chords for a building having Type 4 horizontal irregularity.

Vertical irregularities primarily affect the load distribution over the building height. Table 12.3-2 describes the circumstances under which buildings must be designated and having a vertical irregularity. The example problem in this design guide gives a step-by-step method for designing diaphragms, collectors and chords for a building having Type 4 vertical irregularity. Due to this irregularity the loads imposed at various levels can be significantly different from the one assumed in the Equivalent Lateral Force (ELF) analysis procedure given in the code.

ELF analysis procedure may not be able to capture adequately the distribution of lateral forces over the height of the building for the following conditions:

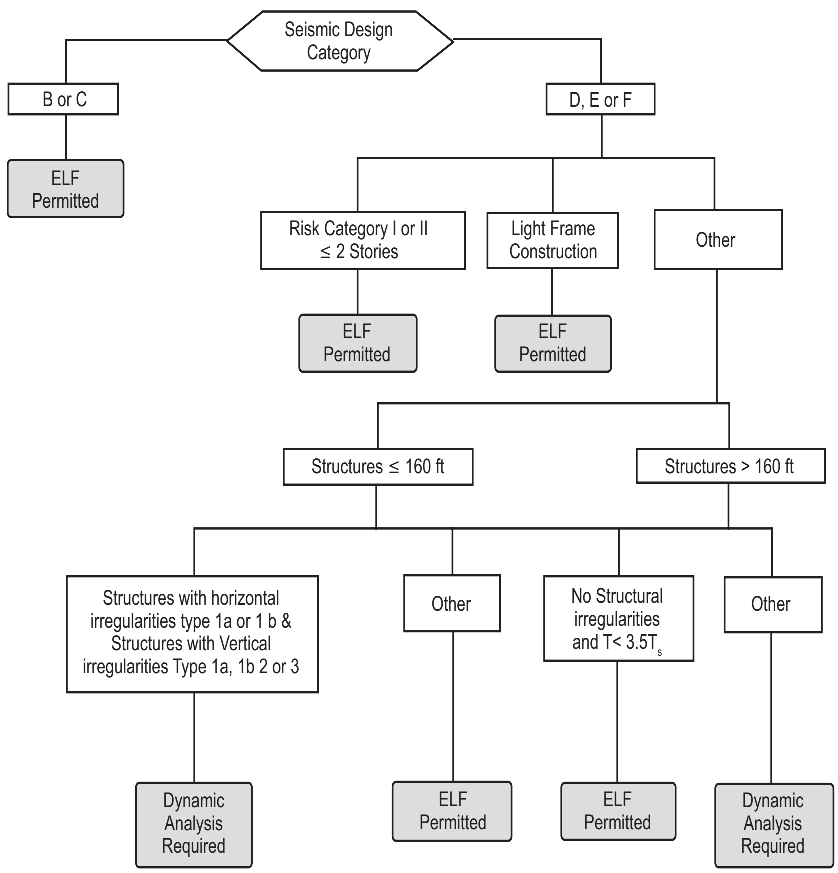
- Irregular distribution of mass
- Irregular distribution of stiffness
- Irregular distribution of story strength
- Buildings where lateral motions in two orthogonal directions and torsion motions are coupled
- Buildings that have significant change in shear from one level to the next
- Diaphragms that have a significant offset in the center of mass from one level to the next
- Out-of-plane and in-plane discontinuity in the lateral load-resisting system

The main advantage of the Dynamic Analysis method compared with ELF is that it is better in estimating the maximum displacement response. Further, the inelastic static and dynamic methods are superior when compared with elastic methods in interpreting structural discontinuities according to the experimental studies carried out by Professor Moehle and Alarcon in 1986 in a paper titled "Seismic Analysis Methods for Irregular Buildings."

According to the study carried out by Professor Krawinkler and Al-Ali in 1998 and their paper titled "Effects of Vertical Irregularities on Seismic Behavior of Building Structures— Report No. 130", the effect of mass irregularity is the smallest, the effect of strength irregularity is larger than the effect of stiffness irregularity, and the effect of combined stiffness and strength irregularity is the largest. This was based on the study of a 10-story building model with SMRF.

Permitted Analytical Procedures

Table 12.6-1 describes the various scenarios under which a particular method of analysis is permitted. For buildings in SDC B or C, ELF analysis is permitted. See flow chart below (reproduced with permission with S.K.Ghosh and Associates Inc.).



For buildings in SDC D, E or F, the trigger for Dynamic Analysis (Modal Response Spectrum Analysis or Seismic Response History Procedures) is based on the height of the building—buildings that are less than 160 feet in height and have horizontal torsional irregularities of Type 1a or 1b and vertical irregularity Type 1a or 1b, 2 or 3, Dynamic Analysis is required.

For buildings greater than 160 feet in height and having no irregularities and period T less than 3.5Ts, ELF procedure is allowed. For all other buildings, Dynamic Analysis is required.

It should be noted that for buildings in Risk Category 1 or 11 and less than two stories and for light-frame construction, ELF is permitted.

As noted in the ASCE7-10 commentary, ELF is not allowed for buildings with certain irregularities due to the fact that the assumption of gradually varying distribution of mass and stiffness over the height of the building with negligible torsion is no longer valid. The basis of 3.5Ts limitation is based on higher modes becoming more dominant in taller buildings and as a result the ELF method may underestimate the design base shear and also may not capture correctly the vertical distribution of seismic forces in taller buildings.

For the example problems in the design guide, since the building is less than 160 feet in structural height, ELF procedure is permitted for buildings having Type 4 plan irregularity and Type 4 vertical irregularity.

Code Requirements for Irregularities

The code requires that all of the plan irregularities except for type 2 (reentrant corner irregularity) and Type 3 (diaphragm discontinuity irregularity) requires that the building shall be analyzed using a three-dimensional model and with a minimum of three degrees of freedom—two translational and one rotational degree of freedom. In addition, the model shall include diaphragm's stiffness where the diaphragm has not been classified as rigid or flexible per Section 12.3.1. For buildings having flexible diaphragm, the code makes an exception for Type 4 horizontal irregularity—the above noted requirement is not required. Each example in the design guide addresses the various other requirements for horizontal and vertical irregularities as required by Tables 12.3-1 and 12.3-2.

Issues Related to Steel Diaphragms

The design of diaphragms for steel-framed buildings encompasses three distinct types: the steel deck, the composite deck (or slab), and the horizontal truss.

Steel Deck

The first type is the steel deck diaphragm, in which the diaphragm shear is carried by the deck, while framing members resist the corresponding axial forces at chords and collectors. The shear transfer between the deck and the chords and collectors is achieved by welds, shot pins, or screws. As the framing members resist axial forces, their connections must transfer these forces, along with the gravity shear. Such connections are part of the seismic load-resisting system and subject to the requirements and limitations in ASCE 7 Chapter 12 and AISC 341, including design for the overstrength-level load in SDC C through F, use of notch-tough welds, and bolt pretension and faying-surface preparation.

Typical design of these decks is based on idealization of the diaphragm as a flexible element spanning laterally to (or across) effectively rigid lateral supports at frames and walls. In the case of steel decks in moment-frame structures, ASCE 7 requires diaphragm stiffness to be included in the analysis. This adds some complexity to the modeling (and introduces several pitfalls), and generally does not lead to a better distribution of resistance, except when such analysis exposes a torsional irregularity. These pitfalls include improper in-plane resistance to principal stress in modeled deck elements and the corresponding lower axial forces in chords and collectors. In the author's opinion the modeling of steel decks should neglect the membrane stiffness other than shear stiffness for the most appropriate design forces to result.

The analysis of the diaphragm itself typically assumes that the collectors are uniformly loaded in shear by the diaphragm along their length. That is, the analysis is based on the deck shear per unit length being the total shear divided by the total length. This is clearly an idealization, and some ductility is required to affect this distribution. This is an area of ongoing research.

Composite Deck or Slab

The second type of diaphragm consists of a concrete element, either over a steel deck or as a formed slab. The shear in these diaphragms is resisted by the concrete with steel reinforcement. (A composite steel deck may serve as reinforcement.) As in the case of the steel deck diaphragm, chord and collector forces may be resisted by the steel framing members. Alternatively, reinforcement in the deck may be provided to resist chord and collector forces. Such a design approach is similar to the design of concrete diaphragms, and the reader should refer to that section for discussion of compressive stress, confinement, effective width, and eccentricity.

The shear transfers from deck to chord and collector (and the moment-frame or braced-frame beam) by means of composite shear studs. (As discussed in the example, a resistance factor of 0.65 is recommended for this application.)

Typically this deck type is idealized as rigid, and a three-dimensional building analysis with accidental eccentricity gives the distribution of forces among the frames, along with transfer forces. The combination of diaphragm forces and transfer forces can be achieved through examination of separate building analyses for each diaphragm force and for the Equivalent Lateral Force (or a modal response spectrum analysis). Alternatively, the combined effects of transfer forces and diaphragm forces may be post-processed from a single analysis (Sabelli et al., 2009).

Distribution of shear along the depth of the diaphragm may be assumed to be uniform, or it may be assumed that only the depth of diaphragm necessary to provide sufficient strength is active. Either assumption is an idealization and requires some level of ductility in the diaphragm or collector line.

Horizontal Truss

The third type of diaphragm is the horizontal truss. This type is sometimes employed in nonbuilding structures and industrial buildings. It is rarely used in other building types but may be used in areas of unusually high diaphragm forces or across open areas at large atria.

In this type, the deck is not considered to provide shear resistance. Instead, beams span laterally between the panel points of a horizontal truss (or system of horizontal trusses) that span between frames or walls.

The truss members and their connections are typically designed to limit ductility demands by use of the overstrength-level forces. Alternatively, they may be detailed similarly to Special Concentrically Braced Frames, with the diagonal members and their connections subject to the requirements for braces.

Issues Related to Concrete Diaphragms

Most concrete diaphragms can be classified as either rigid or semi-rigid. For rigid diaphragm assumption, no horizontal irregularity is permitted and the span-to-depth ratio should be less than 3. For semi-rigid diaphragm, explicit modeling of the diaphragm stiffness is required in the structural analysis with appropriate stiffness modifier—ACI 10.10.4.1 gives recommendations for modeling cracked section properties.

As discussed in the ACI commentary, for most concrete buildings where one can expect inelastic behavior due to seismic demands, it is desirable to limit the inelastic behavior of floor and roof diaphragms and further to limit any inelastic action to occur only in intended locations of the vertical lateral load-resisting system (LLRS) that are specifically detailed for a ductile behavior. However, where diaphragms may reach their flexural or shear strength before the yielding in the LLRS, the diaphragm has to be designed to provide the necessary strength.

The minimum thickness of diaphragm per code is 2 inches thick (concrete slabs and composite slabs serving as structural diaphragm). The minimum reinforcement is the reinforcement to be provided for temperature and shrinkage.

The code now allows for diaphragms to be designed using the code provisions for flexural design (Section 10.2 and 10.3 of ACI) ignoring the nonlinear distribution of strain as applicable to deep beams. This implies that all of the chord reinforcement need not be located at the opposite edges of the diaphragm and the longitudinal reinforcement can be considered to contribute to the flexural strength of the diaphragm. This reduces the area of longitudinal reinforcement required at the edge of the diaphragm, but it should not lead to eliminate all boundary reinforcement as noted in the ACI commentary.

For buildings with horizontal irregularities Type 1a, 1b, 2 and 3 or 4 and vertical irregularity type 4, the diaphragm design forces have to be increased by 25% for the connection of the diaphragm to the vertical lateral load-resisting system.

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