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INTRODUCTION

The availability of sophisticated analysis techniques is overwhelming. Numerous universities and major computer networks support and make available powerful programs tailored to the "exact" analysis of reinforced concrete buildings.(1) Such programs have been developed to provide analysis routines for the solution of both vertical and lateral load conditions on a variety of possible building models. These programs are providing useful analysis tools, when properly used, that have assisted in reducing the number of inaccuracies caused by hand analysis techniques and their simplifying assumptions. Of course, such programs, by virtue of their pounds of documentation and the unbelievable amount of output they produce, carry with them an air of sophistication and accuracy second to none.

Unfortunately, all of the available, automated routines are forced to make a large number of questionable assumptions about building geometry, material properties, service loadings, earthquake force levels, and construction techniques. For this reason, their proper use depends solely on the intuition of the design engineer, who must use them within an acceptable application, understand their limitations and inherent inaccuracies, and then accept full responsibility for their results.(2)

To present a paper on the practical application of computer analysis to all types of reinforced concrete structures under all loading conditions is certainly far beyond the scope of this work. This discussion shall therefore be limited to the analysis of a special class of structures; namely middle-rise (six to sixteen story) concrete shear wall buildings with interacting concrete or steel moment frames. For their height, this type of building represents a proven structural scheme, that when conceived and oriented into a complete and balanced system, has historically performed outstandingly in both minor and major earthquakes.

Within the typical design office, lateral force analysis for earthquake motions has grown from an original, arbitrary equivalent wind technique, through the equivalent static force methods developed by the Structural Engineers Association of California (SEAOC) (3), to the direct application of dynamic analysis for individual structures. In all of these methods, assumptions and

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analysis techniques based on elastic material behavior are utilized.

It is recognized that non-linear analysis techniques are rapidly being developed and have earned a place as an indispensible tool in the research community.(4) Generally speaking, currently available, non-linear analysis techniques do very well with simple frame-type lateral force systems, but have substantial difficulty in properly modeling and solving the complex behavior of concrete shear walls and floor diaphragms.(5) Their use requires a specially skilled worker and, in general, their run costs are very expensive. Thus, it can be said that they are a long way from finding common application within the design office.

For these reasons, this discussion will be generally limited to the elastic analysis techniques currently available and in use today, and particularly limited to an application aimed at giving the best elastic solution for the buildings under consideration. For purposes of organization, it will define the total structural analysis task for earthquake loading in terms of three phases; the overall building response, the internal force distribution to the lateral force resisting elements, and the local force and stiffness distribution within each of the lateral force resisting elements.

In working with computer aided structural analysis, we have found it practically impossible to create and use one computer model that will adequately represent all structural characteristics and present usable results in all three phases at once. On the contrary, because of modeling problems and impossibilities that require the bounding techniques of parametric studies, we have found that only through specialized models, tailored to match only the immediate phase, can a rational analysis be performed that is usable and affordable.

OVERALL BUILDING RESPONSE

The overall building response phase of the analysis is aimed at generating the general design forces, floor shears and overturning moments for the structural system. The methods used generally vary with the complexity of the building and the symmetry of the lateral force resisting system. It is important to realize that all currently codified procedures for evaluating overall building response involve, at various levels of sophistication, the dynamic characteristics of the structure.

Original lateral force analysis procedures were based on a nominal design for an arbitrary wind load applied to the structure. This technique followed from the post-earthquake observations of buildings in the 1906 San Francisco Earthquake. Observations after the 1933 Long Beach Earthquake led to the use of a procedure based on lateral forces derived from the application of a design acceleration to the mass of the structure. Further observations on the performance of high-rise buildings led to the adoption, in 1949, of the first method for varying the vertical distribution

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of the lateral design forces over the height of the building. The concept of establishing lateral design forces as a percent of gravity, and adjusting them over the height of the building has since evolved into the current techniques adopted by the Uniform Building Code at the recommendation of SEAOC. This evolution has generally followed the development of a clearer understanding of the dynamic characteristics of structures attained from numerous field observations and recorded data from actual earthquakes, and improved analysis techniques.

Throughout this evolutionary process, empirical performance observations have been transformed into lateral force design routines that recognize that valid principles of structural dynamics, and use the commonly available methods of elastic structural analysis. We, therefore, find ourselves working with a design procedure that sizes and strengthens a structure's elements to resist "elastic" lateral forces that are many times smaller than those expected. The difference, accounted for in terms of overall "ductility", is made up by a properly conceived and balanced lateral force resisting system with elements detailed to develop that overall ductility. Therefore, this resulting lateral force analysis and design process, often referred to as a dynamic analysis, provides only a method of deriving a sufficient amount of information to ensure a safe design based on the empirical observations of the past and our easily used elastic analysis procedures.

The details of a dynamic analysis are rather general and few can agree on what they should be. The analysis process will always involve a stiffness, mass and damping model of the structure that is used to determine the elastic structural characteristics of the system. This model is then excited by one of a variety of real or pseudo forcing functions in order to derive the design forces. At the practice level, the practical time and economic constraints of building design and the high level of uncertainty in the method have led the term into usually meaning a pseudo dynamic analysis using the response spectrum technique of analysis. This method of analysis is either than applied step-by-step with the help of a computer or as simplified and outlined in the Uniform Building Code. This distilled and simplified code method of dynamic analysis is commonly referred to as the equivalent static method.

The details of the response spectrum method of analysis are well documented, taught and generally known. Simply put, a mathematical model of the structure of some level of complexity is conceived that includes a representation of the stiffnesses and masses of the system and their respective distributions. From this model, and a response spectrum that has been adjusted for damping and ductility, the period, mode shapes, modal forces and modal displacements for the building model are derived. These values are combined in some fashion, usually by the square root of the sum of the squares (SRSS) of the individual modal contributions, into the lateral force design values. One of the major

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gray areas in this analysis technique, and the area subject to the biggest debate has to do with the selection of the mathematical model to be used. The possibilities vary drastically from a very simple, single column model with a lump of mass at each floor to the highly detailed, complete 3D models that offer a design SRSS number for every axial load, shear and moment.

The equivalent static method represents a simplified design procedure for applying the sound principles of dynamic analysis while taking advantage of some common characteristics of structures. From the generally uniform distribution of mass throughout the height of a structure, the uniformly decreasing stiffness of a structure with height, and the general regularity and completeness of most structural systems, it has been shown that the lateral design forces for low to mid-rise buildings will be primarily the result of only the first translational mode of vibration.(6) From this observation then, the dynamic analysis of structures conforming to these assumptions can be limited to solely the fundamental mode which then requires the calculation only of the fundamental period of the structure and the fundamental mode shape.

Studies have shown throughout the development of the equivalent static methods that the fundamental period of this class of buildings can be adequately enveloped and described as a function of the height and width of the structure and that the fundamental mode shape can be described as varying from triangular to parabolic depending on the period.(7) These studies are generally based on actual measured periods of structures and their overall response to strong motion. As a result, in addition to accurately stating the period and mode shape of the structure, these empirical values represent a correcting link between building performance and the theoretical building dynamic characteristics; a link that does not exist in the rigorous, theoretical dynamic solution.

Thus for the class of mid-rise concrete shear wall buildings under consideration, it becomes obvious that a response spectrum-type dynamic analysis is necessary only in those cases which violate the assumptions from which the equivalent static method is derived. That is, for example, for buildings with highly concentrated masses in isolated locations, severe stiffness discontinuities due to a change in building configuration or lateral force system, or for incomplete or poorly configured structural systems. It must be re-emphasized, though, that in applying a direct dynamic analysis to such systems, the period and mode shape correcting link is not available and it therefore becomes imperative that the analysis include all factors critical to determining the dynamic characteristics of that system.

For example, it has been our experience, that for mid-rise concrete shear wall buildings, foundation conditions play an important role in the period determination of the structure. For an example of this effect, consider a fifteen story, steel frame/ concrete shear wall hospital we recently analysed and designed.

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The upper eight stories of the structure consist of 23.2m x 65.8m (76 feet x 216 feet) rectangular floors with the lower seven floors providing the gradual stepwise transition from the rectangular eighth floor to a 54.9m x 65.8m (180 feet x 216 feet) square basement configuration. The lateral force system was composed of four major longitudinal and four major transverse shear walls that created a balanced configuration for both translational and torsional motions.

Early dynamic analysis studies of this structure were carried out using simplified stick column models of the shear walls linked with pin-ended rigid links into two-dimensional models. Refer to Figure 1 for the significant results for the transverse direction. The initial dynamic analysis using a fixed base assumption resulted in a fundamental period of 0.5 seconds and a cantilevering type of fundamental mode.

The foundation conditions at the site were highly variable with the bedrock surface sloping from the ground surface at the east end to about 22.9m (75 feet) below the surface at the west end. The structure is basically caisson supported to bedrock. For purposes of analysis, the foundation springs were calculated based on the soil deformation characteristics under the caisson loadings and the P/A deflections of the caissons. A second, flexible foundation dynamic analysis then, using the rotation springs at the base of each shear wall column, produced a 100% increase in fundamental period to 1.0 seconds and a fundamental mode resembling a rigid block-type motion.

Given a basic understanding of the role of dynamic analysis, a few comments are in order concerning the recent emphasis toward using dynamic analysis to provide a "more accurate" overall building response for buildings that generally meet the assumptions of the equivalent static method. This trend is no doubt encouraged by the availability of inexpensive, easy to use, dynamic analysis programs. Unfortunately, we need only recall that while the analytical technique can produce many significant figures, the input data and assumptions are, at best, good for only one significant figure. Consider, for example, the high variability of input motions, structural and non-structural stiffness, foundation conditions, damping, etc.(8)

In short, it is imperative that for the design process, we must keep in mind that the overall building response determination is an empirical exercise based on past experience and resist the tendency to hide behind any sophisticated analysis that is mostly meaningless.

INTERNAL FORCE DISTRIBUTION TO THE RESISTING ELEMENTS

Given an overall equivalent static building response in terms of generated floor forces, story shears and overturning moments, or an overall dynamic building response in terms of modal floor forces, shears, and overturning moments for all

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significant modes, it becomes necessary to distribute these forces to the lateral load resisting system. Any such distribution must pay especially careful attention to three critical parameters: the stiffnesses of the shear walls, the flexibility of the diaphragms and the minimum and inherent torsion within the system.

Within this phase, the computer stands as an invaluable tool in providing a quick, and easily attainable three-dimensional solution that can, if modeled properly, accurately provide a designable force distribution. However, if internal force distribution is carried out without proper attention given to these areas and/or without proper statics checks for identifying and intuitively verifying the resulting force distributions, the distributions will be erratic. Instead of a usable design value, the designer will be left with a distribution plagued with fictitious hard spots, unnecessary transfers of shears through diaphragms, wastefully conservative designs in some areas, and a lack of strength in others.

Shear Wall Stiffness

The modeling of concrete shear walls for stiffness and local force distribution has always been a problem within any computer analysis. Analysis routines have always been forced to idealize shear walls with a finite number of elements that are intended to represent their actual configuration. Time and economic constraints have led to idealizations using either a beam and column type model, a gross finite-element model using story deep elements, or some combination of the two. It will be shown later in the local force distribution section, that such two-dimensional shear wall models can be erratic and very sensitive to slight modeling changes. However, for purposes of discussion within this phase, assume that a suitable shear wall model is available for use in the three-dimensional distribution process.

Flexibility of Diaphragms

Traditionally, concrete diaphragms in concrete shear wall buildings have been assumed to be rigid allowing for the internal force distribution to be based on only the relative rigidities of the lateral load resisting system and the building geometry. This assumption greatly reduces both the input necessary to set up the model, and the computational effort required to solve the 3D lateral force distribution. This assumption works well for concrete moment frame structures where the lateral force system is not as rigid as a typical concrete slab diaphragm, and for concrete shear wall structures with regularly placed, full height shear walls of fairly uniform stiffness. However, it does not even come close to properly representing diaphragms in irregularly shaped buildings with major openings or diaphragms carrying major shear transfers between shear walls.

Consider again the fifteen story, concrete shear wall hospital

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mentioned earlier. An excellent example of the invalidity of a rigid diaphragm condition occurred at the seventh floor. For this particular structure, we were dealing with an 8.3cm (3-1/4 inch) lightweight concrete filled metal deck diaphragm and a number of 55cm (20 inch) thick concrete shear walls. As previously noted, and shown in Figure 2, the upper floors, eight to fifteen, of the building consist of rectangular floors with four primary transverse and three longitudinal shear walls. At the seventh floor, the building grows into an "L" shape that includes the extension of one transverse shear wall and the addition of a fourth longitudinal wall.

The original, three-dimensional model, with a mathematically rigid diaphragm and equivalent columns for shear walls produces a 8900 ^{kN} (2000^{k}) shear transfer to the added fourth longitudinal wall (line 7.8) under the longitudinal lateral design forces, while creating a shear reversal in the two short longitudinal shear walls. The total shear at this story is about 69,400^{kN} (15,600^{k}). In a subsequent, three-dimensional model composed of the same equivalent columns linked by a flexible diaphragm, only 1070^{kN} (240^{k}) shear was transferred to the new wall, an 8 times reduction, with no shear reversal in the others. In the flexible diaphragm model, all of the floor slabs were modeled with finite elements. The stiffening effects of the structural steel chords and the perpendicular shear walls were added to the model with truss elements.

While the condition at the seventh floor is the most dramatic example of the softening effects of flexible diaphragms, it is certainly not the only one seen in this analysis. Consider the transverse shear distribution to the shear walls in the transverse direction, shown in Figure 3, and the longitudinal shear distribution shown in Figure 4. The solid lines represent the original distributions based on the rigid diaphragm analysis. The dotted lines are the shear diagrams from the flexible analysis. It is interesting and instructive to note that while the basic shear distributions are about the same, the sharp shear transfers have been tempered and the entire diagram takes on a more believable smoothness. While this resulting smoothed distribution had no effect on the final shear wall design, the elimination of the sharp transfers had a significant effect on the diaphragm design. In one case, it made the difference between a workable boundary-type design and a nearly impossible design.

As an interesting sidelight to this analysis, we were quite surprised to find that when comparing the deflections of the two analyses, only very small differences were found. Consider again the seventh floor diaphragm. The flexible diaphragm distortion from the rigid diaphragm position is shown in Figure 5. The maximum additional deflection attributable to the flexible diaphragm occurred at the inside corner of the "L" and was a little more than 8mm (.31 inches). Obviously, this is far less than the deflection it would take to close the allowable shrinkage cracks in the floor slabs. Please realize that this fact alone

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raises serious doubts about the rigidity of any concrete diaphragms working in a shear wall system, and certainly warrants careful consideration in applying one of the most common analysis assumptions made today.

Minimum and Inherent Torsion

Engineers have long realized and understood the need for considering the effects of torsion, both in the resisting and driving sense. Any internal distribution of shears to the lateral system must be done with regard for the basic eccentricity between the center of mass, and the center of rigidity, i.e., with regard for the inherent torsion in the resisting system. In addition, it has been shown that a driving-type of torsion, that created within a torsional-type mode in the system need to also be considered.

A three-dimensional internal force distribution, done as a stiffness analysis problem, has the inherent ability to adjust the shear distribution for the effects of the actual eccentricity at each floor within a structure. Unfortunately, the actual eccentricity and resultant redistribution is only available through the deflected shape of the system under load. Thus, the only way to identify the actual eccentricities is to apply some basic principles of geometry to each rotated floor diaphragm to determine its center of rotation which can then be equated to the center of rigidity. Comparing this center with the known center of mass and floor shear, the actual effective torsional moment can be calculated.

Unfortunately, it is not enough to merely account for the inherent torsion in a resisting system. For lateral force resisting systems with major mass or stiffness eccentricities, conventional dynamic analysis can be used to identify and add the effects of torsional modes of vibration. However, for relatively symmetric, or total symmetric building systems with well-balanced and complete lateral systems, no currently available dynamic analysis can so thoroughly model material properties, construction tolerances,mass distributions, element stiffnesses and out-of-phase input motions to accurately include the effects of accidental or minimum torsion. Therefore, the effects of accidental torsion must be inputed to the system as an additional static torsional moment.

Newmark(9) has determined theoretically that the accidental torsion in a symmetric building, arising from the earthquake wave motions can cause a rotational-type component of input motions. The resulting torsional moments, when expressed as equivalent eccentricities can exceed 5% of the long building dimension for buildings with periods between 0.5 seconds and 1.0 seconds and 10% for periods shorter than 0.5 seconds.

The preliminary, unpublished results of a recent full size shaking test of a three-story concrete precast structure in the Soviet Union, actually demonstrated presence of accidental torsion

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in a symmetric building. A test structure, 9m x 12m with three-3m stories was constructed in the Soviet Union in 1975. It was shaken by a series of nearby explosions. The explosions produced an eight second strong motion record with a maximum acceleration of 19% g. The structure had solid full height concrete shear walls on the sides parallel to the explosions and was open on the other two sides. The structural response was instrumented by the Seismic Engineering Branch of the United States Geological Survey. Acceleration data was recorded at twelve stations within the structure which were used to accurately define the translational, torsional, and vertical modes of vibration. Visual comparison of the modal contributions to the total deflected shape indicates that over half of the motion at the roof, at some instances in time, is due to the fundamental torsional mode for this theoretically symmetric structure.

The existence of accidental torsion has led to the inclusion in the Uniform Building Code of minimum torsional moments and the required omission of any resultant, reducing effects. A number of Central and South American codes as well as in the proposed design criteria recently prepared by the Applied Technology Council have the additional requirement that the minimum torsional moments be added on top of the inherent torsion of the resisting system. In either case, since these adjustments are due to conditions that cannot be taken care of with any dynamic analysis, they must be accounted for with static moments added to the internal distribution phase of the analysis. It fortunately represents a task that can be defined and carried out but unfortunately requires a substantial iterative process.

LOCAL FORCE AND STIFFNESS DISTRIBUTION WITHIN EACH LATERAL RESISTING ELEMENT

Given an internal distribution of the lateral forces to the resisting elements, the final analysis step requires the derivation of the actual design axial loads, shears and moments needed for detailing the elements. Of course, inherent in this local distribution is the estimation of the local and overall stiffness characteristics for the resisting element. Both the local force distribution and the overall stiffness characteristics for an element can be derived by a variety of hand techniques using appropriate assumptions for very simple walls or by a number of available computer analysis techniques for any type of wall. In either case, it is important that the user understand the limitations of the technique being used and the sensitivity of the system to slight changes in its assumptions.

Contrary to popular assumption, shear walls do not usually come in a constant size and without openings or other functional disruptions. More than likely, the designer takes his shear walls where he can get them, works to provide a symmetric, balanced lateral force resisting system, and ends up with walls filled with holes and radical discontinuities. Generally then, we know that shear walls vary from solid walls, to walls with regular patterns

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of openings, to walls with random patterns of openings, to walls that have a multiplicity of opening patterns. In each case, computer models of varying levels of complexity are required to adequately model an element's local force distribution and stiffness characteristics.

A solid shear wall can very accurately be modeled as a cantilever column by properly providing the shear and bending properties of the equivalent section. A shear wall with a regular pattern of large openings throughout its height can be modeled with a beam and column equivalent frame model. A shear wall with a random set of openings behaves most like a "pierced plate" and therefore cannot be accurately modeled as a frame. For overall stiffness, it can be considered as an equivalent solid column with adjusted properties. For a general, detailed analysis it can only be accurately modeled with a detailed finite element model. For models that exhibit two or more types of opening patterns, a general detailed analysis can take advantage of both beam and column models and finite element models as they apply to the opening patterns, provided care is taken in the transition zones between the models.

It has been our experience that while each of these modeling techniques can be very successful at creating a useful force and stiffness distribution within a lateral resisting element, they can also predict totally fictitious and useless results when used improperly or without a full understanding of their inherent assumptions. Specifically, we have found that beam and column models tend to be very sensitive to the treatment of their panel zones. Finite element models generally provide better answers but are always hard to prepare, expensive to run, and very difficult to interpret. Combinations of beam/column models and finite element models tend to be efficient but can error drastically at their point of transition. And finally, we have found that all local distributions within shear walls involving openings at or near their bases are very sensitive to the base fixity conditions.

Panel Zones in Beam and Column Models

It is fairly common practice, when modeling a concrete shear wall with a fairly regular set of openings, to model the piers as columns and the spandrels as beams. The line members normally follow the pier and spandrel centerlines and their widths are accounted for by rigid arms under the elastic assumption that plane sections remain plane. This particular model, however, neglects the shearing distortion of the panel zone. Depending on the geometry of the wall, and the relationship between pier and spandrel size, this model can error substantially in the local stiffness of the wall and the local design forces.

Consider, as an example of an improperly modeled panel zone, the 12.2m (40 foot), one-story solid wall shown in Figure 6. Applying a 3560kN (800k) load at the hypothetical roof level to an equivalent column model results in an overall deflection of

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