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EARTHQUAKE RESISTANT DESIGN: A VIEW PROM THE PRACTICE

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ABSTRACT

The main stages of the design process of structures that will be built in seismic areas are described, and differences with conventional design are shown. The importance of the collaboration between all the members of the interdisciplinary team that is necessary in the design and construction process is emphasized. The methods for seismic analysis and design are commented, and their limitations indicated. The 1994 Northridge earthquake is employed to show the uncertainties of the analysis and design methods. Finally, several points are suggested in order to improve the design and construction of buildings in seismic areas.

KEYWORDS

Connections; Northridge earthquake; professional practice; rigid frames; seismic design.

INTRODUCTION

The aim of this paper is to present the main aspects of structural design in seismic areas, from the point of view of a professional engineer; for him, seismic engineering is not an end in itself, but an applied science, which will provide the relevant information to make safe and economical designs.

Technical literature is so huge that it is impossible to keep pace with the developments in the area, specially considering that seismic engineering is not the only discipline that the professional engineer needs to develop his activities. This is one of the reasons why building codes are of such an importance; through them, the engineer receives the results of research and the experience of distinguished professionals. New knowledge, obtained through theoretical and experimental research, and through the study of building behavior during strong earthquakes, must be rapidly included in the codes.

Seismic engineering is more and more complicated; nevertheless, an effort must be made to simplify the methods of analysis and design to permit that well prepared engineers be able to solve, by themselves, most problems, and should require the collaboration of specialists only for designing very important and complex structures.

THE DESIGN PROCESS

It starts with the identification of a need which must be met and which requires some type of construction. It also includes a socio-economic study which will show its feasibility.

The site where the new building will be erected is usually not open to discussion, even though it is a highly seismic area such as Mexico City, San Francisco or Tokyo. Once this reality has been accepted, the building to be constructed has to be safe, and its operation should not be seriously affected by earthquakes.

This paper will focus only on urban buildings of several stories for office or residential use, which constitute a large percentage of constructions. These are also the structures on which most of the efforts made by seismic engineering have been focused.

Once the decision to build has been made, the next step consists of making an architectural pre-design for the building, which will consider every economical and functional aspect. A multidisciplinary team will have to be involved at this point, to collaborate with the architect. Since the beginning, the architect should be specially mindful of the restrictions imposed by the facilities and by the equipment required for the operation of the building and, above all, by the need to have a structure, which is indispensable to maintain the construction's shape, to create the spaces which make it up, and to support, in a safe and economical way, the different loads and actions which will act upon it during all of its useful life.

The architect plays a very important role as head and coordinator of the interdisciplinary team required to develop the project of any relevant urban construction, and he should accept his responsibility as team director. It is not unusual that buildings with configurations chosen by an architect undergo damages because their seismic behavior is inadequate. Once a disadvantageous configuration has been chosen, it might be impossible to obtain a healthy building, even though the structural design is correct. Not only the structural engineer, but the architect as well, should be familiar with the effects that the characteristics of the materials, the geometrical configuration and the distribution of elements that support the vertical an horizontal loads, have on the seismic behavior of the building. And both should be very much aware of the responsibility that they share.

When choosing a structural system to resist earthquakes, the horizontal and vertical configurations of the building, and the distribution of masses and stiffness along its height, have to be carefully considered. Some of the most important rules are:

1) Simple horizontal configurations, such as circles or rectangles that are not too elongated, are desirable to avoid force concentrations in localized areas, and achieve adequate work of the structure as a whole.

2) The rigidity centers of the different floors must be close to the mass centers, to minimize the deformations and stresses produced by torsion.

3) The floor masses and the story stiffness must be distributed uniformly from the base to the top of the building. Sudden changes between successive floors should be avoided, because they tend to produce very large deflections in some stories and to concentrate of them the ductility demands, which might become excessive.

4) The design must be made in such a way that the columns will be stronger than the adjacent beams, because the inelastic deformation capacity is larger in the beams, which have minimal axial forces and, in addition, a column failure is more critical than a beam failure.

5) Continuous and simple paths should be provided from the places in which the seismic actions are applied to the foundation. It is convenient to have highly redundant structures, in order to have several possible load paths; in this way, structural safety will not depend on just one or a few structural elements.

Contemporary structures frequently present problems that were unusual some years ago, when the architecture was mainly based on prism-like shapes, whereas contemporary buildings are usually extremely irregular, both in plan as in elevation.

Differences in focus between structural engineers and architects are clear: one distinguished American researcher has stated that "from the point of view of resistance to gravitational loads, architectural and structural decisions may be made separately, but as to seismic resistance, separating the engineer from the architect is an open invitation for disaster". However, the current architectural trend for high-rise and medium-height buildings is to obtain free shapes that will create an eye-catching exterior, and to provide irregular interior spaces which are attractive for the occupants of the building. In addition, architects are looking for original buildings, with their own personality. Frequently, the structural engineer does not play a dominant role in the determination of the building shape. The architect feels free from structural restrictions, and the engineer has to look for places where to place the elements that will resist the gravitational loads and seismic actions, without interfering either with the shape or with the operation of the building, paying attention to the interconnection between the structural members, to obtain the necessary continuity along the height of the structure. In this way hybrid solutions are obtained, which are not always convenient from the point of view of seismic behavior; these solutions can be analyzed only thanks to the huge power of electronic computers.

In this stage the most convenient material is selected, the structural system is chosen, and the best possible positions are chosen for beams, columns, shear walls and vertical bracing, taking into account the restrictions imposed by the architectural project. The goal is to obtain a structure capable of resisting intense dynamic actions, which change direction a good number of times during each earthquake, without deteriorating, as deterioration will decrease its resistance to later earthquakes. Elements with adequate stiffness and strength are to be distributed in such a way as to eliminate excessive torsion, and an adequate balance between strength and stiffness has to be achieved. The mechanism of energy absorption shall also be chosen, as the capacity of the structure to provide an adequate inelastic response during intense earthquakes will depend on this mechanism.

Preliminary designs should not be limited to the superstructure; they will have to include the foundation as well. The architect, apparently, has now very little influence, because this part of the construction is no concern of his, since it is below ground, will not be seen, nor will interfere with the building operation. However, the type and characteristics of the foundation will be determined, in good measure, by architectural decisions, which will dictate how and where the superstructure loads will be transmitted to the foundation. For example, a poorly conceived architectural project for a tall, slender building might, under severe seismic actions, cause significant net tensions at some columns; this will cause problems, that might be difficult to solve, in the design of the foundation.

The main collaborator of the structural engineer is now the specialist in soil mechanics. Their joint work should be easier than work with the architect, as they are both engineers and are pursuing the same goals: the safety and adequate behavior of the building. However, this joint activity has not been as fruitful in the past as could have been expected. Here there is another possible source of problems which are very significant in seismic areas, in which the soil-foundation-superstructure interaction has great relevance.

From among several feasible preliminary designs, the best one is chosen. This one, when developed to the necessary level of detail, becomes the final project.

The actions that the structure will have to resist are next determined. Live loads depend only on the purpose that the building will serve and not on the construction characteristics nor on its geographical location. Nevertheless, increments in these loads, because of overloads, whether intentional or not, or because of a change in use, are more important in buildings built in seismic areas, because not only the vertical loads are increased, but also the earthquake actions. Dead loads depend on several properties of the building, among them the weight of the structure itself; their determination is not different for structures to be built in seismic areas. On the contrary, and differently from wind effects, that in urban constructions depend almost exclusively on their shape and external dimensions, seismic response will be a function of the structure's properties and of the relationship between them, the ground properties in the site, and the characteristics of the earthquakes. Here lies one of the most important peculiarities of engineering in seismic areas.

Once the loads have been determined, structural analysis is used to assess their effects on the structure.

Because of their great complexity, most of the buildings should be drastically simplified to be analyzed; not the real building, but a model of the same, is analyzed.

The selection of a satisfactory model, that will include the most significant characteristics of the building, is fundamental. The most refined, most mathematically elegant and accurate methods, are useless if the model does not adequately represent the real building, or if it is not built so that it will behave as it was supposed. Agreement between the model and the building is much more important, and much more difficult to achieve, in seismic analysis, as compared to the analysis for vertical load, because of phenomena such as the interaction of structural and non-structural elements, the relationship among strength, stiffness and ductility, the space configuration of the building, and the joint behavior of foundation and superstructure.

The result of the analysis are the flexural moments, shear and normal forces, and torsion moments, which will have to be resisted by each one of the members that make up the structure and by the complete structure. Design consists in determining their dimensions in such a way as to obtain adequate resistance and satisfactory service conditions. If even one of these goals is not attained, it might be necessary to make a new analysis, with the modified dimensions of the structural elements.

Adequate seismic behavior demands a much more careful design than the one required for gravitational loads; it will allow to prioritize the possible forms of failure, and avoid phenomena that will cause drastic declines in resistance or energy absorption capacity.

Sometimes, because of the design results, it is necessary to modify the architectural project which, in turn, might make new analysis and new designs necessary.

The process ends with the formulation of the documents, drawings and specifications, that are necessary to convey the results to the builder.

The construction process also demands the greatest care, specially in the details from which will depend the survival of the structures during intense earthquakes.

LOADS

The goal of an urban building is to create spaces in which people can live and work in adequate conditions of safety and comfort. For this purpose, the structure must have enough strength to resist the combined effects of vertical loads and winds or earthquakes, and adequate stiffness to keep the deformations below certain limits, in order to avoid panic among the occupants, at least in not too intense earthquakes, to reduce damages to non-structural elements and installations, and to control second order effects, which arise from the interaction of vertical loads and horizontal drift; these effects may have significant influence on the resistance of flexible structures, and can even originate instability failures.

Dead and live loads are generally determined very accurately. On the contrary, there are very large uncertainties relative to the actions produced by wind and earthquakes, as these are natural phenomena, out of the control of man. However, intense winds are frequent, much is known about them, and their design values can be determined in a reliable way. The opposite is true of earthquakes, whose intensities and characteristics are, at least for the time being, and maybe forever, unpredictable.

Many of the uncertainties of design in seismic areas are caused by lack of knowledge of the maximum actions that the structure might be exposed to. These uncertainties make seismic design different from all other structural design problems.

The basis for seismic design of buildings is not the most intense earthquake that they will have to resist, which is unknown, but the earthquakes that have occurred in the past at the place where they will be built. Since the quantitative information is very limited, just a few decades, which are but an instant in the life of our planet, very little is known about the design earthquake.

The probable intensity of the design earthquake depends also on the return period considered, relative to the useful life of the structure; here another source of uncertainty arises, because buildings are not demolished when their useful design life ends. They are preserved until they stop being economically convenient.

There is growing evidence that, for very long return periods, which are adequate for revising failure limit states, the maximum possible soil acceleration does not depend on the seismicity of the area or on the distance to a significant geological fault. For example, it is believed that for return periods measured in thousands of years, there can be maximum levels of earthquakes, similar to those along the Western coast, in vast areas of the Eastern and the Mid-Western United States. The XVIII century Lisbon earthquake, and the earthquake of New Madrid in the XIX century, are evidence of this hypothesis. What would happen with modern seismic engineering if a devastating earthquake would shake New York City, for example?.

Structural engineering in seismic areas faces a problem for which there is apparently no solution: to choose structural systems, and to size their elements, so that they will resist unknown loads by means of response mechanisms that are not totally understood.

The amplitude and frequency content of the waves produced by an earthquake at a given site depend on the magnitude of the earthquake, the distance from the site to the area where the earthquake was generated, the mechanical properties of the medium they travel through to arrive at the site, and the characteristics of the soil at the site.

There may be amplifications of waves of certain frequencies when they travel through soil strata with certain properties, and the amplitude of the movements can increase or decrease, and their frequency vary. The characteristics of the soil at the site are determinant: everybody knows the huge amplifications typical of the area of the ancient lakes on which a large part of Mexico City is built, on top of thick layers of clay with a large content of water.

Since the waves which originate ground movements in the place where a structure will be erected are coming from the rupture area of a fault, they arrive at the building with a certain direction, but because of the characteristics of these waves, their interaction and the local, geological and topographic effects, the movement of the soil becomes random, predominantly horizontal, with some directional emphasis, and with a vertical component which may be important. The effects of an earthquake on a certain building are affected even by the characteristics of the neighboring structures, their geometry, mass and type of foundation.

The study of the mechanisms which originate the earthquakes and how their effects are transmitted to the soil surrounding the rupture area is not the responsibility of structural engineers; it belongs to geophysicists, geologists and seismologists. The structural engineer is interested in the most unfavorable seismic actions that the building which is going to be constructed will have to resist. He is not trained to assess them: he lacks basic training and timely access to all the relevant information and, above all, he does not have enough time.

There are agencies in educational, professional and official institutions that undertake the study of seismology, seismicity and seismic risk which are necessary to determine the design actions. These actions are made available to the engineers who will project the structures by means of a building code, which is legal and mandatory. All the information obtained through time-consuming and sophisticated studies performed along many years of measurements of real earthquakes, of interpretation of the information obtained, and development and application of complex and elegant mathematical models, is reduced to two main data on which the structural design is based: the seismic coefficient and the design spectrum.

The seismic coefficient, c, is an index of the design action; it is the basis for the design spectra, and is used directly to evaluate the horizontal actions that will have to be resisted by the structure, when the evaluation is performed with static methods.

The following table shows the evolution of the seismic coefficients specified, since 1942 until today, in the successive Mexico City building codes, corresponding to the most common types of structures.

SEISMIC COEFFICIENTS IN MEXICO CITY BUILDING CODES.

YEAR	GROUP	TYPE OF STRUCTURE	SEISMIC COEFFICIENT			SEISMIC COEFFICIENT/Q				
			ZONE I	ZONE II	ZONE III	Q	ZONE I	ZONE II	ZONE III	GROUP A
1942	III TO VI		0.025							I.- x 4.0 II.- x 2.0
1957*	B	TYPE I	0.05	0.06	0.07					x 2.0
1966	B	TYPE I	0.04		0.06					x 1.3
1976	B	TYPE I	0.16	0.20	0.24	4.00	0.04	0.05	0.06	x 1.3
1985*	B	TYPE I	0.16	0.27	0.40	4.00	0.04	0.068	0.10	x 1.5
1987	B	TYPE I	0.16	0.32	0.40	4.00	0.04	0.08	0.10	x 1.5
1993	B	TYPE I	0.16	0.32	0.40	4.00	0.04	0.08	0.10	x 1.5

* *Emergency codes*

The 1942 code was the first code with recommendations for seismic design. Buildings to be used for housing and offices were designed with a coefficient of 0.025. The weight of the floors was multiplied by this coefficient to obtain the equivalent horizontal forces, as if the structure was a rigid body. These forces were applied to the frames according to their tributary areas, without taking into consideration their relative stiffness. The walls were not included in the analysis, and no torsion was considered. The seismic coefficient was independent from the geometrical and structural characteristics of the building. Lateral displacement of the floors was not revised. Buildings no higher than 16 meters (five stories, approximately), did not require seismic design.

Schools, hospitals and other important buildings were designed with a seismic coefficient of 0.10.

Even it they were so rough, the recommendations were not completely illogical in a time in which the buildings had few stories, their facades were made of stone and were very stiff and resistant and, in addition, they possessed many interior walls, also with high stiffness and resistance. These elements have doubtless been responsible for the good behavior of the majority of these structures during intense earthquakes.

After World War II, because of operation requirements, the development of new materials and new construction systems, as well as the architectural fashion, office buildings were constructed with facades made of thin sheets of metal and glass, without interior resistant walls, with the exception of the walls for elevators and facilities which, in many cases, because of their position, caused significant torsion, which was not included in the analysis. The lateral resistance and stiffness of these buildings are provided, almost one hundred per cent, by the structure. All the previous factors were combined with larger spans and more resistant materials, resulting in smaller beams and columns, and less stiffness. Concrete structures with flat or waffle slabs, and mixed structures, with steel columns and reinforced concrete flat slabs, were used. Facing the need to provide parking areas, apartment buildings with free ground floors and upper floors with many dividing walls were common. All of this caused very significant decreases in resistance and lateral stiffness, and concentrations of deformation demands on some stories, at a time when nothing was known about the influence of these factors on seismic response.

The 1942 code was enforced until the July 28, 1957 earthquake. There are, then, a good number of buildings, built in Mexico City, between the end of the 40's and 1957, with a high seismic risk. The situation is similar in all seismic areas in the world, as shown in Kobe, Japan, in January of last year.

It is obvious how slowly new knowledge is incorporated into the codes. Many of the limitations of the 1942 regulations were known before July 1957, as seismic engineering had evolved considerably during those fifteen years; however, it was necessary a very destructive earthquake to have the regulations modified. And this situations has been repeated over and over again, in Mexico City and in the whole world.

The effects of the July 28, 1957 earthquake showed that the response of the structures under a certain earthquake, depends on their own characteristics and on the type of soil on which they were erected. To take these factors into account, Mexico City was divided in three areas in the emergency code issued in 1957, immediately after the earthquake. The seismic design coefficient was varied according to the area and the characteristics of the building. The highest coefficient corresponds to Zone III, with powerful layers of highly compressible clay of lake origin.

The 1966 building code seismic coefficients are somewhat smaller than those in the 1957 regulations; the factor by which they have to be multiplied to design Group A buildings also decreases, from 2.0 to 1.3. The smaller values were due to the fact that the intensity of the earthquake was underestimated, and most of the damages were blamed on construction defects.

In 1957 there were no instruments in Mexico City to measure the intensity of the earthquake; the coefficient of 0.06 in Zone III was obtained, mainly, from approximate measurements of the story drift at the base of the Torre Latinoamericana, combined with the theoretical value of the story stiffness. It was determined, therefore, not the soil acceleration, but the response of a structure with a fundamental vibration period very far away from that of the soil on which it is supported.

The 1976 code was not due to any seismic event, but to the desire to update the 1966 regulations. The concept of ductility was introduced for the first time, by means of the Q factor, which depends on the characteristics of the structural system, by which the coefficient c is divided to estimate the design forces; in this way, the different capacities of energy dissipation due to inelastic behavior of the different materials and structural systems are taken into consideration. The values of c and Q are chosen in a way which gives, for the most common types of construction, the same 1966 design forces. The requirements that have to be met to use the different values of the ductility factor are not clearly stated; for example, the convenience that the plastic hinges of the energy absorption mechanism will form at the beams is not indicated, equal Q factors are set for steel rigid frames with ordinary beams and with open-web beams, in which no plastic hinges can be formed. The design is made for the simultaneous action of 100% of the seismic forces in one direction and 30% of the forces in an orthogonal direction.

After 1957 there were several earthquakes, of different intensities; in general, the response of the buildings was acceptable from the standpoint of their resistance, but they frequently underwent excessive damages in non structural elements; there were a few buildings that collapsed during this period, but the causes were attributed to construction defects, combined with inadequate structural systems. In accordance with the behavior of the buildings between 1957 and 1985, and with seismicity studies undertaken during this period, it seemed that the design seismic coefficients were adequate and even conservative, even though it would maybe be convenient to reduce the admissible story drifts.

The terrible effects of the earthquakes of September 19 and 20, 1985, caused the immediate issuance of a new emergency code. In this code, the design seismic coefficients are significantly increased in Zone III, as all the collapsed buildings and most of those that underwent significant damages were in that zone; the seismic coefficients in Zone II were increased in a lesser proportion, and there was no modification for those in Zone I, in which there were no damages. The Q values were also modified, reflecting the behavior observed in the different types of buildings, but Q = 4.0 was preserved for the most common structural systems, though more stringent requirements had to be met. In the 1976 Code, Q factors of up to 6.0 were admitted, even though it seems that very few, or maybe no buildings, were designed with this value since, if used, the design was always governed by lateral displacements; in 1985 the maximum ductility factor was limited to 4.0. Even though the admissible story drifts were almost not modified, the regulation became much more restrictive, in Zones II an III, since the limits must be complied for much larger seismic forces.

In 1987 the seismic coefficients of Zones I and III have been preserved, and the one in Zone II has been increased. The end result, compared to 1976, is that coefficient c in Zone I has not been modified, and those of Zones II and III have been increased in practically the same percentages. The Q factors, now called "seismic behavior factors" to indicate that they depend on other aspects, besides the ductility of the structures, do not have significant changes.

The 1993 and 1987 codes are identical in all aspects relative to structural design.

The importance factor has changed several times through the years. The seismic design coefficients for Group B are multiplied by the importance factor to obtain those corresponding to Group A constructions.

Independently of the little or great depth and reliability of the studies with which the values of c and Q are determined, once published in the building code they become mandatory for the engineers who design the structures that are built in Mexico City. These engineers have to accept responsibilities based on them, in spite of having had a very limited participation in their selection.

The magnitudes of the seismic coefficients arise, mainly, from the observation of the behavior of constructions during the earthquakes that have shaken the city since the 40's: this is why they have been preserved without change in Zone I, and they have grown significantly in the other two zones.

The studies of seismic risk, and the research on the significant earthquakes that have happened in the past, are scarcely reflected in them. The fact that the buildings erected on the firm soil that surrounds the ancient lakes have not undergone great damages, only indicates that the last destructive earthquakes have originated very far away from Mexico City, along the Pacific coast, but it is known that in the past there were earthquakes with different characteristics which caused damages on structures maintained intact during the last decades. It is also interesting to mention that the seismic coefficients of the 1985 Emergency Code, published one month after the September earthquakes of that year, have been preserved almost without change in 1993, after eight years of very intensive studies on seismicity and seismic risk.

If the evolution of the design elastic spectra, which are the basis of modal dynamic analysis, is studied, aspects similar to those mentioned in relation to the seismic coefficients are found, and when comparing them to the spectrum of the accelerogram registered on September 19, 1995 at the Ministry of Communications and Transportation (SCT), new and serious uncertainties appear.

The SCT spectrum shows accelerations of the order of g in elastic structures of one degree of freedom with a fundamental natural period of 2 seconds and five percent damping, which grow up to 1.67 g, approximately, if damping is reduced to two percent of critical. It is clear that the structures built in that area which have a first mode of vibration with a natural period close to 2 seconds, run serious risk of entering into a condition close to resonance, with huge increments in the dynamic effects.

The risk becomes worse in structures with a somewhat lesser fundamental period, because if they undergo deterioration during the first part of a long earthquake, they lose stiffness and get closer and closer to the critical condition. This phenomenon gets worse with time, after a succession of earthquakes, even if they are not too strong. To all the uncertainties of seismic design one more is added: very little is known about the dynamic response of a building when it has just been built, and still less about how it will be modified when, in an undetermined future, it will have to resist a big earthquake.

In structural design the maximum actions that the structure will have to resist are, almost always, of interest, regardless of the complete history of the load process; the response spectra arises from an application of this concept to seismic design. However, the load history is now of interest, because the structure may deteriorate during long earthquakes or during several successive seismic events, as it have to sustain a series of oscillations into the inelastic range of response; the response spectra do not consider the duration of the earthquakes and since, in addition, they are determined by combining and smoothing the elastic spectra corresponding to several real earthquakes, and correcting them by inelasticity in a quite arbitrary fashion, the design that is based on them may have serious deficiencies.

The Mexico City building code seismic coefficients and response spectra have been chosen in such a way that the structures de not become too expensive, and the role of preventing real actions to be bigger than design actions has been left to ductility and damping, thus precluding the collapse. Nevertheless, the ductility demand may be excessive in structures with dynamic characteristics which make them close to resonance, and that have a damping of less than the five percent considered in the regulations. The ductility demands imply very important deformations and considerable and progressive structural damages, that may lead to collapse.

The degree of structural damping is not known and, in addition, it changes in the different stages of the response to earthquakes. Still less is known about the damping corresponding to the complex set made up by structure, walls and partitions, slabs and stairs, foundation, but it is certain that many modern constructions, that lack stiff and resistant walls almost completely, have damping smaller than five percent, and their seismic response may be increased substantially. This problem becomes worse in welded steel structures.

Some structures designed according to the regulations in force have safety levels lesser than the desirable ones, and may even collapse in case of earthquakes similar to those of the last decades. A clear case are the buildings with a fundamental period of vibration between 1.5 and 2 seconds, built in the city zones where the soil vibrates with a similar period. It is no coincidence that the larger number of collapsed structures, or structures with serious damages, had these characteristics, and it is no coincidence, either, the excellent behavior of buildings such as the Torre Latinoamericana or the Pemex Tower, which, because of their great height and slenderness, have fundamental periods around four seconds. Rigid structures built on resistant soils may be another critical case, in the event of any major earthquake with epicenter close to the city.

All these phenomena are not covered adequately in the codes; structural engineers will be responsible for avoiding them, by taking measures which might be as drastic as not designing structures with a number of stories that make them dangerous for the zone in which they will be built. The cooperation of owners and architects is indispensable in this sense.

ANALYSIS AND DESIGN

The goal of the current methods of analysis an design is to compare the foreseeable behavior of new structures, that have not yet been built, with the behavior of similar structures that have had a satisfactory behavior.

Stresses and deformations computed with methods based on a linear elastic response of materials and structures bear very little relationship with the real behavior of constructions in service conditions, and none in the proximity of collapse; the computed stresses are only an index of comparison between new and already existing structures.

Stresses are not the only index: in plastic design, resistance of elements or systems are compared, and in other cases, comparisons are made between resistance of members or critical sections, maximum deformations or dynamic characteristics.

Modern building codes recommend design methods based on load and resistance factors, with resistance factors generally lesser than the unit, and load factors almost always larger than one, to take into consideration the possibility that real actions exceed the actions of design, and that resistance are less than the nominal values.

Even though the load and resistance factors are determined with statistic and probabilistic methods, they are calibrated to obtain structures that do not differ significantly from those built up to now. It is accepted, then, that the theories of probabilities and structural reliability can not, on their own, solve the problem of structural safety; the design of new structures is still based on the comparison with those already existing.

ANALYSIS

The most common method of seismic analysis is the static one. Lateral forces are distributed along the height of the building following a triangular law of variation, with the apex at the base, trying to reproduce, approximately, the response of the building in its first mode of vibration; some codes take into consideration the higher modes, in a roughly approximated way, applying a fraction of the total horizontal force at the upper level. In general, the base shear is made to depend on the fundamental vibration period, determined with an empirical formula based on the general dimensions of the structure or assessed, more accurately, with Rayleigh's formula.

The buildings and earthquakes characteristics are taken into account with methods of dynamic analysis, applied to a model formed by masses concentrated on the floors, connected by springs, whose lateral and torsion stiffness depend on the characteristics of the structure and of the elements, which, structural or not, contribute to resist seismic forces. It was usual to assume that the structure is supported on undeformable soil, that the floor systems are rigid, and that the columns may undergo lateral deformation, but not in the vertical direction; the current tendency is to consider the interaction with the soil, above all when it is highly deformable, and the computer programs used in the analysis include the columns changes in length.

The history of the response of the structure under specified soil accelerations, which vary with time, may be obtained with a modal analysis; once the complete history is known, maximum actions are evaluated.

The maximum values of the responses can also be obtained with a spectral modal analysis; the maximum responses of each mode are determined, using the code design spectra. These responses are combined to obtain the maxima of the complete system, employing, usually, the method of the square root of the sum of the squares.

Once the horizontal forces have been determined by static or dynamic methods, they are distributed among the vertical resistant elements of the building in proportion to its lateral stiffness, and the analysis is made as if they were static forces.

The dynamic approach is considered more appropriate than the static one, specially for irregular buildings, but it still has many shortcomings which are caused, mostly, by the assumption that the response is elastic up to failure, in spite of the fact that the design is made so that inelastic behavior start under earthquakes of medium intensity.

Very important structures are sometimes analyzed by taking into consideration their inelastic behavior under earthquakes of adequate characteristics. The inelastic response, along time, is determined with a step by step process of integration. This is the most accurate seismic analysis method, at least from a conceptual standpoint; however, since neither the intensity, nor the duration, nor any other characteristic of future earthquakes are know, and the method is complex and expensive, it is mainly used as a research tool; few real buildings are designed using this method.

The trend during the last few years has been to emphasize the many hypothesis, not very accurate, of static seismic analysis, and consider that the dynamic analysis is the perfect solution. This may be true from a theoretical standpoint, but the uncertainties of the dynamic analysis are so many that their results may be very far away from representing, in a reasonable way, the real behavior of the structure.

Uncertainties begin where they always begin: it is impossible to predict the characteristics of future earthquakes at a certain location.

The damping of the structures, on which their dynamic response depends in a good part, is not known either, and still less is known how it changes along time, since this depends on the characteristics of the structure and the non-structural elements, as well as on complex interactions between them.

Another serious uncertainty arises from the reduction that is made in the linear elastic response to consider the ductility of the structure. The most common procedure consists in dividing it by a factor, which depends mainly on the structure's ductility and over-resistance, which varies from one to four or more. The influence that this factor has on the end results of the dynamic analysis is obvious; however, its values can not be computed. They are a product of the judgment of groups of experts, which are based mainly on the behavior of the different structural systems during earthquakes, and which are frequently unpleasantly surprised, when in later earthquakes this behavior is very different from the expected one.

The determination of the mathematical model necessary for analysis is also subjected to important uncertainties. Lateral loads are frequently resisted by shear walls and braced frames, combined with rigid frames. It is difficult, and sometimes impossible, to satisfactorily determine the stiffness of these complex systems. The non-structural interior walls and facades may considerably increase the building stiffness, unless they are built to move freely with respect to the structure. Floor systems and stair ramps also modify lateral stiffness.

And all these stiffness, as well as those of the frames, whether of steel or reinforced concrete, change during an earthquake, because of cracking and progressive deterioration, and during the time, when a succession of earthquakes occurs. In addition, the deterioration it not uniform, and this alters the distribution of seismic forces among the resisting elements, and modifies the torsion, which can be substantially increased.

The soil-foundation-structure interaction introduces new unknown or ignored phenomena. To evaluate the design seismic forces, it is usually assumed that the ground movement, at the basis of the foundation, is the free field movement, which is reasonably correct only if the soil is stiff. When the soil is soft, the building fundamental period tends to increase, because of the rotational component of the foundation's movement. In addition, a good part of the energy transmitted to the structure during the earthquake is lost by radiation of the seismic waves through the foundation, and because of the damping of the soil material, due to the hysteretical inelastic action which takes place in it. As a consequence, seismic forces are reduced, while lateral drifts and PΔ effect grow.

It is clear that the seismic response depends largely on the type of foundation; this effect is usually not considered in the analysis, and the existing knowledge on it is very scarce.

And what can be said about the influence of the neighboring structures and their foundations? It is not known how to determine it, and even if it were possible it would not be of much use, as the future changes in the structures close to a given site are not known.

All of this makes it obvious that the dynamic methods, theoretically very accurate, are not that accurate when applied to real constructions. And there are also doubts of what is the use of theoretical studies, of great apparent accuracy, in which is studied, for example, the elastoplastic behavior of isolated two-dimension rigid frames, from which all the factors that make the behavior of real structures so extraordinarily complex are removed.

When the analysis is finished, the results are a set of horizontal actions which reproduce, with more or less accuracy, the inertia forces that the ground movement causes on the structure. The design actions are obtained considering these forces as static forces, and using any of the usual methods of analysis. The most common are matrix methods, performed with computers, which are based on an unlimited linear elastic behavior of the structure; however, codes allow redistribution, more or less arbitrary, of moments and shears, to take into consideration, even though in a roughly approximate way, the effect of the inelastic deformations that precede the failure.

DESIGN

Structures may be designed, at least in theory, to have enough resistance and stiffness to respond in a predominantly elastic way, under the combination of gravitational and seismic actions, but, on so doing, the solutions are much more expensive than the traditional ones, which have had, in general, a satisfactory behavior under real earthquakes. This solution is recommended for special structures, such as nuclear power plants, in which damages have to be maintained at very low levels, even under very strong earthquakes.

In most structures the previous solution is not justified. This is the origin of the current philosophy for the design of buildings which will be built in seismic areas, which determines the criteria to set the levels of load indicated in modern codes: buildings should resist minor earthquakes without damages, moderate earthquakes without structural damages, but with some damages in non-structural elements, and very strong earthquakes without collapse, but with non-structural and structural damages which may be severe. It is accepted the possibility that the structure undergoes significant damages, but lives should not be lost. The goal of the codes is to obtain structures that will respond elastically under earthquakes that might be expected to occur several times during the lifetime of the building, and will survive the maximum intensity earthquake which might occur during its lifetime without collapse. To avoid collapse during the most strong earthquake, the members, and the structure as a whole, must have enough ductility to absorb and dissipate energy by means of postelastic deformations, which demand important excursions into the inelastic range, with little or no loss of resistance. The necessary ductility may be associated, in extreme cases, with very large permanent deformations; even though collapse does not occur, the structure's damages may be so important that might not be economical to repair them, and the structure might be considered a total loss.

After the last strong earthquakes (Mexico, 1985; Northridge, 1994; Kobe, 1995), it has been questioned, if not the philosophy itself, at least the level of damages which should be allowed during intense ground movements, because the cost of repairs and reinforcement is so high that surely can be justified an increase in the initial investment, to decrease the risk that important damages might occur to the structure, architectural items or installations.

The main characteristics that a building that is going to be built in a seismic area should have, are enough resistance and stiffness and adequate ductility.

Ductility is not indispensable, in theory, since structures may be built, in theory also, that will respond elastically during earthquakes of any intensity, but since the characteristics of the most unfavorable earthquake that the structure will be exposed to are not known, nor it is known the way in which the structure will respond to that earthquake, ductility cannot be suppressed, at least in critical areas of the structure, without running the risk that the real behavior will be very much below the desired behavior.

In their book "Reinforced Concrete Structures", R. Park and T. Paulay state the following:

"Since it is impossible to accurately predict the characteristics of the ground motions that may occur at a given site, it is impossible to evaluate the complete behavior of a multistory frame when subjected to very large seismic disturbances. However, it is possible to impart to the structure features that will ensure the most desirable behavior. In terms of damage, ductility, energy dissipation, or failure, this means a desirable sequence in the breakdown of the complex chain of resistance in a frame. It implies a desirable hierarchy in the failure mode of the structure. To establish any sequence in the failure mechanism of a complex chain, it is necessary to know the strength of each link.

In spite of the probabilistic nature of the design load or displacement pattern to be applied to the structure, in the light of present knowledge, a deterministic allocation of strength and ductility properties holds the best promise for a successful response and the prevention of collapse during a catastrophic earthquake. This philosophy may be incorporated in a rational capacity design process. In the capacity design of earthquake-resistant structures, energy dissipating elements of mechanisms are chosen and suitably detailed, and other structural elements are provided with sufficient reserve strength capacity, to ensure that the chosen energy-dissipating mechanisms are maintained at near their full strength throughout the deformations that may occur".

This book was published in 1975, but the aspects pointed out are still valid twenty years later; they are the basis of the capacity design of ductile structures, which has been incorporated, to a larger or lesser degree, in all the contemporary codes for seismic design.

In capacity design, certain elements of the system which resist horizontal forces are chosen, and they are designed and detailed so that they will dissipate energy under the severe deformations imposed upon the system. Inelastic response is concentrated in the critical regions of the chosen members, the plastic hinges, and the rest of the structure is protected against failure by providing a strength larger than the one which corresponds to the development of the maximum possible resistance of the potential plastic hinges.

Capacity design is not an analysis technique, but a powerful design tool; the simplicity and attractiveness of the method come from the fact that the designer commands the structure "what to do", instead of asking it, by means of an analysis, "what it can do". The goal pursued is to have a desirable and predictable behavior during extreme earthquakes whose characteristics are, in spite of all the studies performed, unknown.

If the failure limit state corresponds to the formation of a mechanism with plastic hinges, and these hinges have enough rotation capacity to admit, without loss of resistance, the great inelastic deformations corresponding to the mechanism, an upper limit is imposed on the intensity of the lateral actions that a future earthquake, of unknown characteristics, may cause.

An important part of the knowledge on which the project and structural design of constructions that will be built on seismic areas is based, comes from the observation of the behavior of real buildings during real earthquakes. From the study of collapsed or heavily damaged structures, and of those structures that did not undergo damages, or damages were very small, it is apparent that structures, in general, have a much larger seismic resistance than the one attempted to in the codes. But it is also clear that that vital extra resistance may be easily lost, if are neglected aspects that, not very important in structures which will have to resist only predominantly vertical loads, become life or death issues when a big earthquake hits them.

Among the main factors which may cause drastic decreases of strength of structural systems, during long duration earthquakes, are the following:

Defective architectural design, which causes very large torsion, sudden changes in stiffness from one story to another, excessive ductility demands in localized areas, incompatible forms of vibration of the parts of an irregular construction, transmission of vertical, gravitational and seismic loads to localized and poorly chosen areas of the foundation and the soil.

Low redundancy.

Premature non-ductile failures, because of cracking, shear or buckling.

Excessive demands of ductility on critical sections or elements.

Poorly designed or poorly built connections and other details, which stimulate brittle failures.

PΔ effect, increased by gradual loss of stiffness in the construction and its interaction with the soil.

Pounding of adjacent buildings.

Defective construction, because of use of inadequate materials and/or insufficient supervision.

Resonance by coincidence among the earthquake characteristics and the forms of vibration of the soil and the building.

Modifications of the structure and/or non-structural elements; increase in the number of floors, elimination or change of position of walls.

Live loads larger than design live loads, above all at the upper levels, because of change of use of the building or creation of dead files.

Deterioration of the structural system along time, due to weathering, earthquakes, differential settlement of the foundation.

Changes in the resisting elements stiffness, because of progressive deterioration of walls and/or structural members.

Reinforcement of the foundation, or change of its characteristics (for example, placement of piles in a building which did not have them), required by inadequate behavior, which may cause an increase in the effects transmitted by the soil to the superstructure during an earthquake, and a decrease of the amount of energy dissipated by radiation or by inelastic behavior of the soil.

RELATIONSHIP BETWEEN RESEARCH AND PROFESSIONAL PRACTICE. THE NORTHRIDGE EARTHQUAKE.

As in almost every human activity, in structural engineering practice comes before theory and research, art comes before science.

Researchers are almost always concerned with solving problems caused by structural engineers when they venture into unknown territory, rationalizing and improving the methods for design, or determining the causes of collapses or other problems. Examples of this are the studies on dynamic stability of suspension bridges, which followed the failure of the Tacoma Bridge in 1940, and research relative to the brittle failure of steel, caused by the great number of ships that failed in this way during the Second World War, as well as the XIX century studies on fatigue, performed after the rupture of locomotive shafts subjected to a very high number of load cycles.

That art precedes science is obvious in seismic engineering. This can be clearly seen in the variation, along time, of the design seismic coefficients, as a result of the effect of earthquakes on already existing buildings. Another example is the still unfinished history of the design of welded joints between beams and columns of steel rigid frames, from the practice followed when the first high-rise buildings of welded steel were built, to the behavior of connections during the January 17, 1994 Northridge earthquake, and the consequences derived from it.

The June 1949 AISC specifications already permit the use of welding for rigid frames connections, but do not give any information about their design. In the 1961 printing of the 5th edition of the AISC Steel Construction Manual, details are given on riveted, rigid connections, to resist the wind, and indications are provided for the design of the connecting elements, angles, T's and rivets, but no mention is made of the joint design itself.

The first welded high-rises belong to that time; the common area for beams and columns was not revised, under the assumption, probably, that if the column could resist all the actions that were acting upon it, there was no reason why local problems should arise. If the beam was directly joined to the column, the design consisted on specifying complete penetration welds on the flanges and in sizing the filet welds in the web, to transmit the shear force. It was common practice to use horizontal plates to connect the beam flanges, and to place one or two vertical angles on the web. In general, all the frames of the buildings were rigid, in both directions, and in most connections the columns received one or two beams in the web, in addition to those of the flanges.

During the years 1952-53, a 22 story building was built in Mexico City with a totally welded steel structure (even the beams and columns, of sections I and H, were made with three welded plates, since in Mexico there was, and still there is, no production of rolled shapes of the necessary sizes, and importation was banned); steel was A7, with welding characteristics much lower than those of A36, which replaced it a few years later. Seismic forces were obtained by multiplying the weight of each level times a seismic coefficient, constant with the height, of 0.025. No torsion nor relative stiffness were considered. No deformations were computed. No measures were taken so that the structure could preserve its resistance in the inelastic range; no attempt was made to make the columns more resistant than the beams. The structure was formed by orthogonal rigid frames, which included all the columns; the methods of analysis were approximate. In the more than forty years that have elapsed, the building behavior has been excellent, in spite of the fact that during these years two strong earthquakes occurred, in 1957 and 1985. This is, doubtless, a demonstration of the advantages of steel rigid frames for structures built in seismic areas, when they have high redundancy, the earthquake effects are distributed uniformly, and they are built with relatively thin wall profiles which are easily welded, without creating discontinuities nor excessive residual stresses. The correct behavior of the rigid beam-column joint has also been shown, as in many other times, including the case in which the beam connects to the column web.

A well known textbook, published in 1957, has pictures of rigid welded connections between beams and columns, in buildings built in California; the beams arrive to the flanges and to the web of the column, and they are joined to them with plates and angles; the horizontal plates of the beams that connect to the column web are acting as stiffeners for the ones connected to the flanges. It is stated that, to achieve a high degree of stiffness, stiffeners may be required among the flanges of the column "in case there are no beams rigidly connected to the web", so that their flexion does not cause a non-uniform distribution of stresses at the butt-weld of the upper plates, but that there is no analysis that indicates when they are necessary, so the decision of placing them or not, lies on the judgment of the designer. There is also mention of laboratory studies which indicate that it is a good practice to design the upper plates butt welds with tension allowable stresses 20% less than those recommended by AISC, and that the upper flange may be butt-welded to the column, without a horizontal plate, but that, when doing so, ductility is lost.

The main problems that affect the connections design and behavior were already identified, but nobody knew how to solve them, and they were left "to the judgment of the designer".

In December 1958, AISC published its first recommendations for plastic design of continuous beams and one or two-story rigid frames; all connections whose stiffness is essential to insure the continuity assumed in design, will be capable of resisting the moments, axial and shear forces produced by the design factored loads. If necessary, stiffeners will be used to preserve the continuity of the flanges of the members interrupted in their union with other members of rigid frames; the stiffeners will be placed in pairs, on both sides of the web of the member which goes through the joint.

It is recognized that stiffeners may become necessary, but not when, nor are there any recommendations to design them.

Towards the end of the 50's an experimental and analytical study was made of a representative group of rigid welded beam-column connections under static load. This study provided the design rules recommended since 1961 in AISC specifications, which have been preserved until now, clarified and expanded. The specimens were formed by W columns and rolled beams; in most of them the beams were connected to the column flanges; but some had four beams, joined to the web and to the flanges, it was concluded that the beneficial effect of the stiffening provided by the beams connected to the web more than offset the unfavorable effects of the triaxial state of stresses created by them, so that it is conservative to design joints with web-beams as if there were none. The results of this research, and the recommendations for the design of beam-column joints derived from them, were published in 1959.

In November 1961, AISC included these recommendations, incomplete, in its specifications for plastic design, and provided rules to revise the panel zone by shear, when only one beam is supported on the column, or when the end moments of the two beams are unbalanced. In the main body of the specification, for design by allowable stresses, the sizing of the rivets, bolts or welds is mentioned, but nothing is said about the joint itself.

In the sixth edition of the AISC Manual for Steel Construction, published in 1963, details are shown for welded rigid joints, for beams joining the column flanges or the web.

The problem of the design of the beam-column joint under static or wind loads can be considered solved since the 50's. During those same years a study of cyclically loaded joints was started, trying to reproduce their behavior under seismic actions.

The results of experimental research, baked by theoretical studies, made along more than thirty years, seemed to indicate that the rules developed for joints under static load were also applicable, with few changes, to structures built in seismic areas, if they were complemented with a revision of the panel zone.

The three versions of the AISC specification in force right now, for design by allowable stresses, plastic design and load and resistance factor design, have similar recommendations for the design of beam-column joints of welded rigid frames, based on the studies reported in 1959 and in later research which, in general, ratify their results. It is checked if stiffeners (also called continuity plates) are needed in front of the beam tension flange, to prevent the welding from cracking if the column flange is too flexible, facing the compressed flange, so that the column web will not buckle, or facing any of the flanges, so that it will not fail by local plastification. Shear resistance and buckling of the panel zone are also checked, and rules are given to reinforce it, if necessary.

In the Recommendations for seismic design of structural steel buildings, published by AISC, in its second edition, in June 1992, rules similar to those of the general specification are given, and specific details for joints with beams connected to the column flanges or to the web are provided. The inelastic action may be concentrated in the beam or column end, or in the panel zone, and the designer will decide which is the most adequate particular zone in each case; for this purpose, he sizes the rest of the structure to remain in the elastic range. It is advantageous to have the plastic hinges forming at the beams, so it must be checked that the columns which converge at each joint have greater resistance than the beams; however, it is stated that it is convenient to have a partial plastification of the panel zone, in order to decrease the ductility demand at the end of the beams. It is assumed that in most rolled beams, strain-hardening allows the whole plastic moment to be transmitted by the flanges, which join the column with complete penetration welds.

In the last editions, the AISC Steel Construction Manual preserves the details proposed in previous editions for rigid frames connections.

The requirements which should be met in the design of joints should make sure that plastic deformations which may arise in them during the response to severe earthquakes will not happen in the connection elements, but in some of the two adjacent zones, the beam or the joint. The design is not made for the forces obtained in the analysis, but for the nominal strength of the members which are really used in the structure, thus precluding joints from failing before the necessary inelastic deformations. This is true, even when the members are overdesigned for resistance, for example, when their dimensions are increased to reduce the structure's story drift.

Until the Northridge earthquake it was believed that welded steel rigid frames were the most convenient structural system to resist intense seismic actions, because by taking the adequate measures to avoid instability or brittle failures, structures were obtained that responded in a ductile way up to collapse, which happened when a mechanism was formed, with plastic hinges mainly at the beams, preceded by absorption and dissipation, by inelastic behavior, of great amounts of energy. In addition, in past earthquakes, significant damages were rarely reported, and never collapses, in rigid frames built in accordance with contemporary practice.

However, the Northridge earthquake produced brittle failures in several thousands of connections, in more than one hundred buildings of heights between one and twenty six stories; there were no collapses, and no lives were lost, but the magnitude of the damages was such that it will be necessary to invest several thousand million dollars to repair the damaged structures and to reinforce potentially dangerous buildings.

The Northridge earthquake has shaken the confidence in the welded rigid frame as the preferential structural system in seismic areas. Once again it has been shown that in seismic engineering blind trust is not justified, above all, it is deposited in a connection in which force is transmitted through only one complete penetration weld, whose survival depends on a large number of parameters, many of them related with the quality of labor. The problem gets worse when these connections, responsible for the integrity of the building, are reduced to a minimum, so the failure of very few endangers the whole structure.

Seismic design of buildings with welded rigid frames is based on the assumption that they may undergo significant inelastic deformations, which are mainly concentrated on plastic hinges placed at the ends of the beams, without loss of strength, which makes them capable of dissipating, in a benign way, the energy that they receive from earthquakes; the damages should be limited to moderate plastic flow and localized buckling, without brittle fractures. Based on this assumed behavior, building codes require that structures of this kind have a design strength, when loaded laterally, several times smaller than the necessary to behave 100% elasticaly.

Additional requirements, to limit the story drift, make resistance substantially larger than the minimum required. In most of the structures built in areas of high seismicity, plastification should no start until the earthquakes reach intensities of the order of $^1/_3$ to $^1/_2$ of the design intensity. This kind of design has been based on historical precedents, on the observation of the behavior of buildings during earthquakes, on research, which has included laboratory tests of models made up by beams and columns, and in non-linear analytical studies.

The observation of the damages caused by the Northridge earthquake shows that, against assumptions, brittle fractures started in many cases at the connections, when the ductility demand was very low and, in some buildings, while the response was still elastic. The great number of brittle fractures exceeded expectations by a large factor.

Due to lack of data and experience relative to the effects of largest and longest earthquakes, there is considerable uncertainty on the behavior of the different types of buildings in seismic events of great magnitude. It is believed that seismic risk in those events depends, in a great measure, on the particular ground motion at each specific site, and on the characteristics of each individual building. The generalizations relative to the probable behavior of certain types of constructions might, then, lack meaning.

The five problems most frequently mentioned, among the possible causes of the poor behavior of connections, have been:

1.- Incorrect workmanship, specially in field complete penetration groove welds.

2.- Pre-existing cracks in the weld metal or in the adjacent base metal.

3.- Residual stresses at the joints, generated during the construction of the structure, including shop and field welding.

4.- Failure of the column flange, caused by tensions in the through-thickness direction.

5.- Basic configuration problems at the joints.

The great number of failed connections in structures made by different contractors, eliminates the first factor as the main cause of the failures, unless the errors are in the standards which govern the making of the welds.

In early rigid frame construction, nearly every column participated in the resistance to horizontal forces, so the structural systems had a high degree of redundancy. In an attempt to obtain economic solutions, in recent practice in California, the lateral resistance is provided with the columns and beams of a few bays, at the periphery of the building; the remaining columns are designed to resist only vertical load, and the beams are supported on them with flexible connections. The few beams and columns on which the lateral stability of the building depends, and the welds among them, are very large, which makes the problems mentioned above more critical, and contribute to the poor performance of the connections. Further, if the resistance to lateral loads depends on a few elements, the failure of only few joints may cause a significant decrease of this resistance.

Failure have been due to a combination of all the factors mentioned plus two aspects to which so far, at least apparently, very little attention has been paid: the almost instantaneous nature of the forces caused by the earthquakes and the complex tridimensional stresses at the connections, which arise from the process of manufacture and assembly of the structure, aggravated by the earthquake effects.

Ductile failures are associated with shear stresses. Fast strain rates, and low temperatures, cause increments in shear strength, and the relationships between stresses and resistance change; both phenomena cause a decrease in ductility and any of them, or the combination of both, may be the reason why steel, very ductile in normal conditions, becomes completely brittle. Low temperatures had probably no influence in the performance of most of the damaged structures (even though ambient temperature at the time of the earthquake was 4ºC, and several of the fractures happened in structures under construction, not yet protected), but the velocity of load application must have played an important role.

In a zone of a structure subjected to triaxial stresses, with three equal principal stresses, a ductile failure can never happens, as shear stresses are null in all directions. Principal stresses are never equal; however, the irregular and restricted shrinkage of the weld material deposited between beam and column flanges, and of the adjacent base material during cooling from the liquid state to ambient temperature, produces very high residual tension stresses in three orthogonal directions; their combination with earthquake stresses produces a drastic decrease in the relationship shear stress/axial stress, which promotes brittle failures. The importance of the problem increases because of stress concentrations due to discontinuities in the material (due, for example, to weld defects, or to the rolling process), and by changes in the direction in which the internal forces are transmitted.

The combination of all the effects above mentioned may make it impossible to build connections with the flanges of the beams butt-welded to the column flanges that do not fail by brittle fracture under the almost instantaneous actions of an earthquake; of course, assumptions such as the one which allowed, in many cases, to design the web connection without considering any bending moment, under the assumption that strain-hardening allowed the flanges to transmit, by themselves, the complete plastic moment of the beam, are invalidated, as the beam will fracture under smaller deformations than those corresponding to strain hardening.

Rigid frame non-linear deformations are obtained as inelastic flexural or shear strains develop within discrete areas of the structure. When inelastic deformations are large, plastic hinges are formed, which admit significant concentrated rotations, at constant or almost constant load, thanks to yielding of tension fibers and local buckling of the compressed ones. If a sufficient number of plastic hinges are developed, the frame becomes a mechanism, that undergoes lateral deformation. To this behavior corresponds a significant energy dissipation, particulary if the number of members involved in the mechanism is large, as well as significant local damages in the plastic hinges. It is not convenient to have plastic hinges forming in the columns, as mechanism might develop with the participation of few elements, the so called "story mechanisms", associated with little energy dissipation. In addition, these mechanisms cause local damages in elements that are critical to resist gravity loads.

The methods for the design of connections recommended before the Northridge earthquake were based in the formation of plastic hinges at the beams, adjacent to the face of the columns, or in the panel zone. The plastic hinge at the end of the beam generates great deformation demands through the thickness of the column flange, in the welding metal and in the heat affected areas, which can cause a brittle failure. To obtain a more reliable behavior, it is recommended to dimension the connection in such a way that the plastic hinge is developed far away from the face of the column, at a distance no less than one half the depth of the beam; for this purpose, the beam-column joints should be reinforced with coverplates, gusset plates, lateral plates, etc., and all the elements of the frame should be designed to resist the actions corresponding to the formation of the hinges, including strain-hardening effects.

The principles of capacity design are identified: the regions in which plastic hinges will be formed are chosen, and the rest of the structure is dimensioned to achieve that end.

There is evidence that plastic flow of the panel zone may increase the capacity of the connection to admit plastic rotations. However, there is also the concern, and some evidence, that if the shear deformation is excessive, cracks will form in the column flange in front of the beam flange. If the local curvature is significant, it may contribute to the failures of the joint. This suggests that it might be convenient to design the panel zone in a conservative way.

Doubler plates and, specially, the welds associated with them, produce unfavorable effects; it is recommended to select, if practical, column sizes that will not require the addition of those plates.

Additional welds in bolted vertical shear plates are not recommended, because when the induced joint rotations are large, they apparently contribute to the potential failure of the plate. It has even been suggested to use elongated horizontal holes, in order to limit the moment generated in the shear plate, to protect its capacity to resist vertical forces in the event of a flexural failure.

Against the requirements of the codes in force, it is convenient always to reinforce the connection with horizontal stiffeners (continuity plates), of a thickness at least equal to the flange of the beam (without including the beam coverplate) or one half the effective total thickness (beam flange plus coverplate).

The conclusion of the previous discussion is that the methods for welded connection design, based on laboratory tests made along more than three decades, and backed by analytical studies, were wrong, were applied to joints very different from those for which they were obtained, or the Northridge earthquake had unexpected characteristics. None of these conclusions give peace of mind to the structural engineer who sees himself, once again, sailing through unknown waters, in spite of the fact that they seemed to have being explored enough.

Reanalyzing the buildings damaged by the earthquake, it was found that it was impossible to predict the places in which the damages were found, and that their distribution was practically random, with very little relationship with the areas in which the analyses predicted the biggest demands of strength or ductility.

The huge uncertainties of seismic design, and the little value of the very sophisticated dynamic analyses, are again evident.

In the near future there will probably be an increase in the use of semi-rigid welded connections and high strength bolts in beam-column joints, and more complex and promising solutions will be used, such as seismic isolation of the buildings, to limit the energy that they receive during a seismic event, or the use of dampers, to dissipate that energy without inelastic deformations of the structure.

CONCLUSIONS

Design should be made with a reasonable seismic coefficient, not too low and not too high, and with design spectra that take into consideration the probable characteristics of future earthquakes and the soil-foundation-structure interaction.

Once the seismic coefficient and the design spectrum have been chosen, the goal will be to obtain constructions in which the phenomena that are unfavorable to their behavior during earthquakes should be suppressed, and which may resist, without collapse or excessive deterioration, a considerable number of load cycles of high magnitude. This will be done by means of an adequate architectural and structural design, and careful construction, that will require very careful supervision as well, in which the structural engineer should be involved.

The main attention should be focused on the height and configuration of the building, to avoid resonance phenomena and geometrical or structural anomalies, in plan or elevation, that will produce excessive torsion or weak stories, and on the design and construction of the structure, always looking for the most desirable behavior under strong earthquakes, which involves avoiding premature brittle or instability failures. All of this demands a lot of care, mostly in the areas where there will be great demand for ductility. And it also demands a much closer collaboration between the architect and the structural engineer, and between the structural engineer and the specialist in soil mechanics and foundations.

The method of analysis becomes secondary if capacity design is used; there is no doubt that in special cases dynamic analyses are required, above all in constructions with very complicated geometry, but in most buildings it is enough to have forces obtained with a static analysis, since the characteristics of the structure will prevent those forces from being exceeded.

It is my opinion that during the last decades an excessively scientific character has been given to structural engineering in seismic areas. Structural engineers and builders have been depending too much on theoretical research, upheld, many times, on not too firm bases, up to believing that the results of a dynamic analysis performed with a computer, based on the design spectra produced by the research, will allow to design structures which are earthquake-proof.

It is forgotten that it is not the structure which is analyzed, but a model not too similar to it, that the spectra arise from earthquakes that have happened in the past, not from the ones that will be in the future, and that their maximum ordinates have been cut in a quite arbitrary manner. It is forgotten that structural engineering in general, and seismic engineering in particular, are an art, with solid scientific bases, but not an exact science. It is forgotten that, given the basic parameters, there are several solutions for a given problem, and that the behavior of a building will depend, in a good part, on aspects which are difficult to assess quantitatively, and which are not included in traditional specifications and codes.

Building codes should provide standards so that structures will be designed with reasonable seismic coefficients, and they have to specify analysis and design methods congruent with the importance of the structures. But these are not the only aspects that they have to cover, nor even the most important. They will have to pay special attention to the characteristics which will give buildings the additional resistance that allows them to resist ground motions larger than those prescribed in the codes without collapsing and, in many cases, without significant damages. Even though some of these aspects are covered in the codes, they have received, and they are still receiving, scant attention.

In general, emphasis should be shift from analysis to design. Each significant earthquake forces modifications to the seismic coefficients and the design spectra, and it is impossible to foresee, with enough accuracy, the seismic actions that a certain building will have to resist in the future, since that depends, in a great measure, on its own characteristics and the characteristics of the site, which, in addition to being only imperfectly known, change with time, and vary substantially from one earthquake to another. The best way to resist these unknown earthquakes is by giving to the structure the necessary ductility to put an upper limit to its effects, and the necessary strength and stiffness to resist the earthquakes with localized damages, easy to repair. Of course, events such as the Northridge earthquake show that there is not yet enough knowledge on the seismic response of structural elements and their connections, to be able to provide, in all cases, the necessary ductility and strength.

Building codes should be addressed to every professional involved in the process of design and construction, and not only to structural engineers, who go unnoticed and ignored when the structures behave correctly, but are usually the only ones to blame for everything when problems arise, in spite of the fact that they develop their activities in a frame defined by authorities, owners, urbanists, university researchers, architects and specialists in other areas.

Responsibilities should be shared among code writers, city authorities, owners, architects, structural engineers, specialists in soil mechanics and foundations, and builders, as all of them contribute to the final characteristics of the building and to the state in which it will be when, maybe many years after it was built, will have to resist a strong earthquake.

Who is accountable when a structure collapses, and it is shown that it was designed in accordance with the regulations in force, and that it was built following accepted practices? Who sets the risks accepted by society? The groups of experts who write the codes, are they impersonal, and do not have any responsibility? And the architects, who set the spatial characteristics of structures, and the owners, who will determine their usage and maintenance?.

Applied research must be headed to the solution of real problems, on which the safety of the structures will depend, and every effort must be made to transmit its results, as soon as possible, to those that will use them. This does not happens today, as most of the papers in the specialized journals seem to be addressed to other researchers, and the papers that describe applications of a practical nature have almost totally disappeared. There are even many books that are difficult to read, because the author, who is almost always a researcher or a college professor, frequently does not try to clarify basic problems; he just refers the reader to publications to which the reader does not have easy access to, nor does he have time to read and digest. All of this is a consequence of a vicious circle that should be broken: structural engineers write little; the editorial committees of prestigious journals are composed, almost entirely, by professors and researchers, who prefer theoretical studies, and almost always reject what professional engineers have written; as a consequence, they write even less. And, unfortunately, there are very few university professors or researchers who are concerned with the transmission of knowledge to those who will apply it.

The structural engineer makes his designs in compliance with the formal requirements that regulations impose upon him and, since he is not an expert in seismic engineering, nor is this the only aspect of design that worries him, he does not take the physical phenomena that underlie them in great consideration. For this reason, it is vital that codes and regulations are continuously updated in the aspects of interest to the structural engineer; it can never be forgotten that his goals and interests, and those of his clients, are not necessarily the same as those of the researcher; it should be always remembered that, even though research is basic for the development of structural engineering in seismic areas, it is a dangerous practice to put the writing and updating of building codes in the hands, almost exclusively, of professors and researchers, with very little input from the engineers who will be using them.