UPDATING REINFORCED CONCRETE DESIGN RECOMMENDATIONS FOR STRONG SEISMIC ZONES

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SUMMARY

This paper presents a discussion of some general concepts which should be considered in the formulation of reinforced concrete design provisions for strong seismic zones. Topics include: Controls on irregular building configurations; compatibility of design base shears and related details with design practice and construction feasibility; and controlled diagonal cracking as a viable method of shear wall energy absorption.

INTRODUCTION

In the review of the damage reports of recent strong earthquakes (Anchorage, 1964; Caracas, 1967; San Fernando, 1971; Managua, 1972; Imperial Valley, 1979; El Asnam, 1980), modern reinforced concrete structures provide some of the most spectacular examples of serious damage and collapse. This performance record should not be used to imply that reinforced concrete (R/C) buildings cannot resist strong ground motion; but it should be used as a justification to correct the poor configurations, brittle details, and construction omissions that were the actual causes of failure. Non-symmetrical bracing systems with severe torsional response; stiff box walls or infilled walls on brittle first story columns; lack of adequate shear reinforcing and confinement hoops, poor concrete, design omissions, and unauthorized construction changes are all examples of the initiating sources of failure; and it is these deficiencies that should be controlled rather than the total condemnation of reinforced concrete. Indeed, in nearly all of the cited earthquakes, the predominant type of building construction was R/C; and many of these buildings had excellent performance. In all fairness, if structural steel buildings were to have been in the majority, then perhaps similar damage statistics could have resulted due to excess drift and flexibility, bracing connection failures, and column buckling. The objective of this paper is to present some general concepts that need to be considered in order to best assure satisfactory R/C building performance under moderate (damage control) and major (collapse prevention) earthquake ground motion

Areas to be discussed are: (1) controls on irregular configurations in lateral force resisting systems; (2) compatibility of design base shear values and related details with design practice and construction feasibility; and (3) controlled diagonal cracking as a viable method of shear wall energy absorption.

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CONTROLS ON IRREGULAR CONFIGURATIONS OF LATERAL FORCE RESISTING SYSTEMS

The following non-unique but dramatic failures have done much to encourage code provisions related to the control of structural irregularities: The Olive View Hospital, San Fernando, where a stiff box system supported on a flexible first-story frame suffered severe first-story deformations; the Banco Central, Managua, where a flexible frame was stiffened in its plan view by an elevator core on one side of the building and large damaging torsional deformations occured due to this eccentricity of a major stiffening element; and the County Services Building, El Centro, with a discontinuous shear wall supported by partially confined columns and where the large overturning moment forces from the shear wall crushed the supporting columns. These and other similar lessons have led to present requirements for: strong column - weak beam relations in ductile frames; recognition and calculation of torsion effects in eccentric bracing systems; and full ductile confinement reinforcing in columns supporting discontinuous walls. Further controls are needed to: limit differences in relative strengths and stiffnesses in successive stories; limit the amount of calculated torsion shear to 1/3 to 1/2 of the direct shear in any shear wall; and the design of columns under discontinuous walls for a factor of 3 to 4 times design axial load so as to represent the actual maximum earthquake actions.

Explicit Recognition of Maximum Earthquake Input and Response

New code proposals such as the SEAONC recommendations for the City of San Francisco and the SEAOC revision of their lateral force design provisions employ the explicit ATC3-06 Base Shear Format of $V=C_oSW/R_W$ where C_oS is a representation of the site maximum earthquake response spectrum and R_W is the structural system factor for a working stress design basis. In these new proposals there is also a statement of the maximum force level that may occur in the inelastic yielding structure when it experiences the maximum earthquake deformations. This force level is $3R_W/8$ times the design base shear force level for values of R_W ranging from 6 to 12. With this explicit definition of maximum input and force level, designers are obliged to verify structural integrity and stability at the maximum deformations due to the C_oS input, and verify vertical load capacities of critical elements at the $3R_W/8$ multiple of the design force level. This latter requirement is particularly applicable to columns supporting discontinuous shear walls and the footings for these columns. In these critical members, the nearly full elastic response overturning moment on the rigid wall can produce high axial load demands in the supporting column; therefore, both an extra capacity and a full ductile confinement are required.

COMPATIBILITY OF DESIGN BASE SHEAR VALUES AND RELATED DETAILS WITH DESIGN PRACTICE AND CONSTRUCTION FEASIBILITY

The low seismic design values resulting from the use of K=0.67 or $R_W=12$ in $V=KCSW$ or C_oSW/R_W for ductile frame systems are based on a long history of practice, some (not much) actual earthquake performance, and scattered amounts of analytical studies and element test programs.

Originally these low load levels stemmed from an equality to wind loads: the old-fashioned riveted steel frame buildings, with reinforced concrete floors and concrete fire-proofing,and with strong masonry partitions,performed well in San Francisco, 1906, and in Tokyo, 1923, with lateral seismic design near to specified wind loads. And, of course, when ductile concrete frames were developed, the main objective was to create a system equivalent to this past satisfactory performance and with equal (K=0.67) design loads. However, neither the building systems nor their design procedures have remained the same. Many steel frames and concrete frames have changed from multiple bay configurations to perimeter frames or tube frames. Flooring in steel is metal decking. Flooring in concrete may be pre-cast planking with topping. High strength steels and concrete have been developed; and strength design has definitely changed the design sizes of members. In order to preserve "equivalent performance" in light of these changes, more and more design checks and details are required for both materials. Panel zone doubler plates and stiffeners for steel; and joint shear analysis, member shears at the frame limit state, and extra confinement hoops and stirrups for concrete are some examples. This has led to one of the most irrational positions of modern structural engineering.

In our most important high rise structures - we have elected to retain the low (K=0.67, R_W=12) design loads, and then we have progressively piled on numerous design checks (for relative beam-column strengths, drifts, P-Delta effects, etc.) along with the aforementioned extra details in order to justify ductile performance as required to absorb energy when these low design levels are exceeded. All of which has now become so complex - that there are high probabilities that either or both design omissions will occur or construction flaws will be produced. Thus, in order to retain an old-fashioned design concept and maintain material cost competiveness - our own high rise design and construction process has become most unreliable. If we would draw back and take look at the entire process of "ductile frame design", a much wiser, more economical, and definitely more reliable procedure would be to use a higher design level (K=0.8 or 1.00, or R_W=10), and reduce some of the complex design checks and relax some of the very tight detailing requirements.

Design Objectives

In order to provide a rational basis for improvements in the design process, it is useful to review the twofold objectives of seismic resistant design: Protection against both structural and non-structural damage due to moderate (50 percent exceedence in 50 years) ground motion; and prevention of life endangering failure or collapse due to major earthquakes (10 percent exceedence in 100 years). These objectives should be achieved by methods having reasonable costs along with acceptable reliability both in conformance with design provisions and in the construction of the building.

Development and Consequences of Ductile Moment Resisting Concrete Frame Provisions

Early in our modern earthquake experience (Caracas 1967 and San Fernando 1971) it was evident to all that ordinary R/C frames were inadequate to resist strong ground motion. Ductile Moment Resisting Concrete Frames (DMRCF) were developed and incorporated into the Uniform Building Code (UBC), and the 1973 UBC outlawed the use of ordinary R/C frames as a seismic resisting system in seismic zones 3 and 4. However, DMRCF's are rather complex to design, and even more difficult to construct: the various load combinations, member dimension and strength checks, details required for ductile yielding, and the requirements for closely spaced hoops and stirrups have made for increased costs and possible errors and omissions both in the office and in the field. As a result;

- drafting rooms are filled with requests for large-scale rebar placement details,

- ingenious, but untested, systems of prefabricated rebar hoop cages are invented to reduce rebar placement difficulties and tying costs.

- change orders for small 3/8 inch aggregate and workability admixtures swamp the engineer.

- and the air is bright blue with the caustic remarks of rebar and concrete workers as they struggle to fit the steel and squeeze in the concrete.

A Needed Balance Between Ductility and Feasible Confinement Details

Under the response effects of the major (EQ-II) level of ground motion, the hinging region of beams in DMRCF's will undergo several reversed cycles of large inelastic flexural rotations. Top and bottom longitudinal steel will be extended, leaving a section almost devoid of concrete at the beam-column junction. In order for the beam to maintain load carrying capacity, it is necessary to have closely-spaced hoop stirrups in these hinging regions. These hoops serve to confine the concrete under the severe flexural strains; help longitudinal bars to maintain dowel shear resistance at the separated beam-column face; and prevent diagonal shear crack deterioration of the beam shear span. In order to provide this needed hoop reinforcement, code provisions call for stirrup design based upon a zero value of concrete shear resistance, and yield moments at the beam ends, and a 1.4 factor on dead and live load. The end result is that designers are often forced to specify 2 to 3 inch hoop stirrup spacing to meet shear requirements. The clear space is further reduced when over-lapping hoops are used.

Good, feasible construction, on the other hand, requires a rather wide hoop spacing of about 5 to 6 inches in order to place longitudinal steel, pour concrete, and consolidate the concrete with the vibrator. The paradox is that good homogeneous concrete cannot be reliably placed in the closely spaced hoop arrangements necessary to insure ductile behavior. Some compromise is therefore necessary between the choice of the inelastic design level (the R_w factor) and the need for closely spaced reinforcing. A minimum allowable Stirrup Hoop spacing of at least 4 inches or, perhaps, 6 longitudinal bar diameters would greatly improve chances for homogeneous uniformly consolidated concrete. The small loss in confinement and "basketing" from that given by 2 to 3 inch spacing would be well made up by superior concrete and well-bonded longitudinal steel. The statistical chances of good concrete on the majority of projects would be very much improved.

The same close spacing problem also occurs in columns, in ductile shear-wall boundary elements, and in beam-column joints. Excessive concern for shear resistance where axial stress is less than $0.10f'_c$ in columns, and too many hoops and cross-ties in columns can easily result in rock pockets and, therefore, "no concrete" where it is really needed for shear resistance and for bar development. The boundary elements of shear walls are certainly crowded by both the presence of hoops and the required hook anchorage of horizontal wall shear steel within the boundary element. A permitted hoop spacing of 6 longitudinal bar diameters would help to reduce this crowding, with no real loss in confinement.

THE FALLACY OF EQUAL DESIGN FACTORS FOR STEEL AND CONCRETE

There exists, within the materials industry, a rather irrational rule that seismic design loads must be nearly equal for steel and concrete structural systems. The reasoning being that economic competiveness between these materials is dependent on the design values. Another prevailing custom is that no matter how much structural systems change, their seismic loads must not increase significantly from the values used for past out-moded structures. These "industry" rules should be reviewed in the light of the real differences in the performance of steel and R/C structures; each have their own unique weaknesses and strengths with respect to meeting criteria for damage control and stability.

Moderate Quake (EQ-I) Performance

R/C frames can exhibit some noticeable cracking and spalling with a possibility of costly repairs. However, the inherent stiffness of the structure provides excellent control on non-structural damage. Steel frames may have localized yielding, but this is seldom noticeable and repairs are seldom necessary. However, many "cost competitive" steel frames may have excessive flexibility, and costly non-structural damage can occur both on interior elements and exterior cladding.

Major Quake (EQ-II) Performance

R/C structures can have extensive cracking, yielding, and deterioration on walls, joints, and columns; but the type of stable, confined inelastic behavior, with its changing period, and increasing damping can control the otherwise high elastic forces and deformations. Damage can usually be repaired. Steel moment frames may not crack or fracture, but excessive plastic deformations may render the building non-repairable. Steel braced frames may offer better controls on deformations, and bracing can possibly be replaced if seriously deformed or ruptured.

Base Shear System Factors

The value of the K factor in $V=KCSW$ or the R_W factor in $V=C_0SW/R_W$ should be considered as the best guess for the preliminary design of structural members. Such that, for a given material and lateral force resisting system, this preliminary or first trail design needs little or no modifications to meet drift control criteria at the moderate EQ-I level, or overstress limits and P-Delta criteria at the major EQ-II level. R_W should not be visualized as a "ductility factor", since the particular design method (WSD or USD) and other design requirements can significantly increase the "yield" level of a given structure beyond the level indicated by the design base shear.

Structural Period

The values of structural period T to be used in the base shear evaluation should also meet the objective of little or no modifications to the first trail design based on the specified base shear. The period T should be based on the uncracked R/C structure with recognition of all non-structural stiffening elements such as in-fill panels and exterior panel surfaces. The use of a cracked-section or slightly inelastic structure stiffness for period evaluation can conceivably lead to periods nearly equal to steel frame periods. However, in the 1971 San Fernando earthquake, concrete frame structures responding in this cracked state condition suffered repair costs for both structural and non-structural damage equal to a significant fraction of building value. Therefore, if damage mitigation is an objective, then design periods must reflect the stiffness of the non-damaged structure.

In summary, if economic competition must exist between different materials and systems of construction, then factors based upon ease and reliability of design and construction and demonstrated performance of damage control - and collapse prevention should be considered in the prevailing argument rather than equality of K and R_W values, periods, and design base shears.

CONTROLLED DIAGONAL CRACKING AS A VIABLE MODE OF SHEAR WALL ENERGY ABSORPTION

Probably because of the association with the word "shear" and its connotations of brittle diagonal crack failure, shear walls have been penalized by either an extra load factor (U=2.0) or a low understrength factor (ϕ=0.60). This paranoia which visualized the virtual shattering of a wall if shear cracking were to occur, is currently supplimented by photos of isolated wall element tests where diagonal web crushing occurs at repeated cycles of large deformations. Yet, in all recent earthquake experience, there have been no instances of serious damage in walls having design levels and reinforcing details reasonably close to current code requirements. Perhaps some of this excellent performance is due to the fact that wall elements in buildings have three-dimensional constraints and load-sharing from framing members and floor slabs; such that shear span ratios are low and there is load redistribution between wall elements floor slabs, and framing when overstress cracking occurs. It is worth mentioning, that the large-scale building test in Japan required a high special single story load application in order to induce a sliding shear failure in the shear wall building. However, in spite of real performance, current literature and research reports and associated code proposals recommend that the shear resistance of a wall should exceed the flexural capacity such that seismic input energy is absorbed by "ductile flexural yielding" rather than by the possibly "brittle or rapidly degrading" mode of diagonal shear cracking.

This "flexure" concept might be valid if: (1) shear walls were isolated vertical cantilever beams, with moment-to-shear ratios greater than about two or three, rather than their usual configuration which is constrained flexurally at each floor level by monolithic or continuous floor system connections; (2) shear panels had little or no shear reinforcing such that a single brittle diagonal crack failure might occur; and or (3) cyclic load repetitions far beyond the yield range, and larger than maximum ground motion response, were to be applied such that diagonal strut crushing would occur at the compression toe of the panel.

For the seismic design load levels given in new code proposals, the excessively large excursions beyond yield are not likely to occur during the major earthquake ground motion. The design procedures either forbid the irregular structural system configurations which might experience these large deformation demands, or they require extra levels of analysis which can forecast locations of large inelastic demand and incorporate corresponding design corrections.

With the limits on maximum shear stress, with the required two curtains of anchored shear reinforcing grids, and with ductile edge members (where the extra confinement ties contribute to shear resistance), the wall panels will form a progressive pattern of closely spaced diagonal cracks for each load direction. This diagonal pattern of cracks will be well contained by the reinforcement grid. Under the cyclic input of major seismic ground motion, the shear walls can form stable non-linear softening hysteresis loops with progressively increasing damping values. This load deformation behavior provides a very effective means of preventing a build-up of response that would otherwise occur due to harmonic matching of input motion and structural period. With the continuous changing of stiffness and increased damping in both load directions, large amounts of response deformations are not likely to occur. In summary, distributed shear panel cracking is a much more effective (and repairable) mode of controlling seismic response and of absorbing energy than the flexural yielding mode for the majority of wall configurations and loading conditions prevailing in current building practice.