Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VIII)

SG-11

SEISMIC DESIGN IN REINFORCED CONCRETE THE STATE OF THE ART IN NEW ZEALAND

Thomas Paulay

Department of Civil Engineering, University of Canterbury,
Christchurch, New Zealand.

SUMMARY

Some highlights of the evolution over the past two decades of a seismic design strategy, largely developed and used in New Zealand for reinforced concrete buildings, are reviewed. The outlines of the philosophical concepts of a capacity design methodology is complemented with illustrations how the expected ductility demands in various structural systems may be met with high quality in detailing.

INTRODUCTION

This review attempts to highlight features in recent developments of concrete design in New Zealand. It concentrates on seismic aspects. Many of these have been studied in New Zealand where they have been incorporated into design codes. Although most design recommendations, particularly those relevant to the behaviour of reinforced concrete elements under reversed cyclic inelastic actions, originated from theoretical and experimental research, some are based on less quantifiable common sense engineering judgments. The basic design philosophy adopted is deterministic in the context that the designer should be able to determine the behaviour of the structure when it is exposed to the most severe seismic action. The philosophy is currently being applied. Details of its application have been quantified. A concurrent, and equally important effort, concentrates on quantifying the goodness in detailing of the reinforcement in potential plastic regions to ensure that ductility potential equal to or in excess of that expected, will be available.

CONCEPTS AND APPLICATION OF A CAPACITY DESIGN PHILOSOPHY

To be able to predict inelastic respone with reasonable certainty, a hierarchy in the chain of resistance had to be established. Thus strengths and capacities need to be compared. It is for this reason that the term "capacity design" was coined. In the capacity design of earthquake resisting structures, elements of primary load resisting systems are chosen and suitably designed and detailed for energy dissipation under severe inelastic deformations. All other structural elements are provided with sufficient strength so that the chosen means of energy dissipation can be maintained.

To quantify strengths, a few new definitions had to be introduced. Ideal or nominal strength, S_i, commonly used in (ultimate) strength design approaches, is obtained from first principles of structural mechanics with specified material strengths. The dependable strength, available for earthquake resistance, $S_{ed} = \phi S_i$, where ϕ is a specified strength reduction factor, is chosen so that it is equal to or larger than the strength demand, S_e, which results from the code specified earthquake load. This means that $S_{ed} \geq S_e$. During a large inelastic seismic pulse, the overstrength of a component $S_o = \lambda_o S_i$ may be developed, where λ_o quantifies the contribution of the maximum likely strength of the materials, particularly that of the steel, which may be mobilized. Typically $1.25 < \lambda_o < 1.60$. When strengths of adjacent elements are compared, for example those of beams and columns in ductile frames, it is convenient to relate overstrength to the required strength thus : $\phi_o = S_o/S_e$.

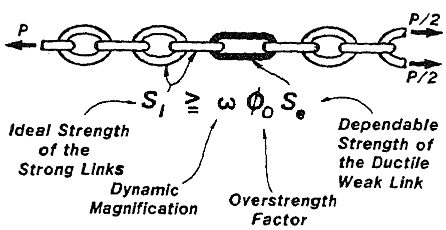

Fig.1 - Strength Hierarchy of Links in a Chain.

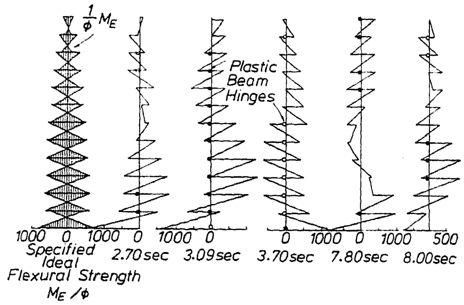

Fig.2 - A Comparison of Column Moments Resulting from Lateral Static and Dynamic Loading.

The essence of the capacity design approach is illustrated with the simplistic example of a ductile chain, shown in Fig. 1. It is assumed that all but the central link are brittle. Therefore they should not be permitted to fail due to overload. A weak but very ductile link must be chosen so that its dependable strength, S_{ed}, is equal or larger than the specified strength demand, S_e. To prevent a brittle failure, the ideal strength of the strong links, S_i, must be at least equal to the overstrength of the weak link, $\phi_o S_e$. In certain components of building structures the load transmission may also be affected by dynamic effects. Hence, for the sake of generality, as Fig. 1 shows, a further factor, $\omega > 1.0$, is introduced. The behaviour of such a chain (structure) is thus governed by the ductile response of its weak link.

Moment Resisting Ductile Frames The primary aim in multistorey frames is to avoid simultaneous plastic hinges at both ends of all columns in a storey. This design leads to the generally accepted "strong column-weak beam" system. Without imposing economic penalties, columns in upper storeys can be provided with sufficient flexural strength to eliminate the possibility of a plastic hinge developing. This is achieved with the relationship shown in Fig. 1. Typical values for the two important factors are $1.2 < \phi_o < 1.6$ and $1.2 < \omega < 1.9$. The dynamic magnification factor, ω, quantifies the deviation of column moments during an earthquake, as shown in Fig. 2, from the pattern predicted by an elastic analysis using lateral static loading. Two of the major practical advantages of this strategy are that : (1) end regions of columns need be detailed for very limited ductility only and (2) lapped splices may be used at the bottom end instead by midheight of upper storey columns.

To reduce excessive strength development in "weak" beams, which would increase the required strength of "strong" columns, the redistribution of design moments between critical sections of beams at any level, allowing maximum values to be reduced by up to 30%, may be adopted. This is in recognition of inelastic response during large earthquake actions. Moment redistribution hardly affects curvature ductility demands (Ref.1). It also enables congestion of beam reinforcement to be reduced.

Structural Walls The thrust of development in New Zealand was directed towards establishing criteria which would enable cantilever walls to exhibit ductilities and energy dissipating characteristics, comparable to those achieved in ductile frames. This may be readily achieved if flexure rather than shear dominates wall response. Therefore, using the principles of Fig.1, the design shear force for a wall is estimated by $V_{wall} = \omega_v \phi_o V_E$, where V_E is the shear assigned to the wall by the code specified lateral load, and the dynamic shear magnification, ω_v, attempts to allow for the increase of wall shear due to higher mode effects on the inelastic wall. Thereby diagonal tension failures may be precluded.

To enable ample curvature ductility to be developed in the potential plastic hinge region of a cantilever wall, the neutral axis depth in the critical section, relative to the length of the wall, is to be kept relatively small. If this is not possible, a part of the flexural compression zone is to be confined, as shown in Fig. 14, to enable large concrete strains, typically $0.004 < \epsilon_c < 0.010$, to be developed without loss of flexural resistance. Wall thickness in the flexural compression zone of the plastic hinge at the base of the wall must be limited to ensure that premature failure during reversed cyclic displacements due to inelastic out-of-plane buckling does not occur. To prevent premature failure of the plastic hinge due to diagonal compression, the maximum computed shear stress should be limited with the increase of the expected ductility demand. Flexural ductility can be readily developed also in squat walls, typically with aspect ratios less than two, provided that significant sliding movements are prevented, as shown in Fig. 16.

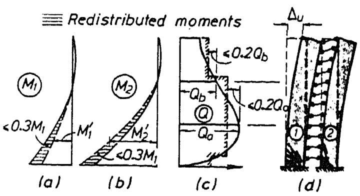

Fig.3 - The Inelastic Response of Coupled Walls.

Because of their large stiffness and dispersed energy dissipation potential, coupled structural walls are considered to be particularly suited for earthquake resistance. As Fig. 3 shows energy dissipation is primarily assigned to ductile coupling beams. For optimal beam strength vertical redistribution of design beam shear forces, as shown in Fig.3, can be utilized. To relive the flexural demand on the tension wall, moment and shear redistribution to the compression wall usually results in significant saving in vertical wall reinforcement without reducing to total resistance of the structure.

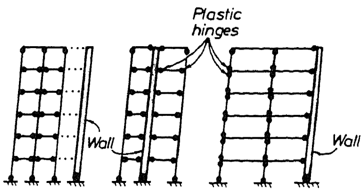

Fig.4 - Energy Dissipating Mechanisms in Hybrid Structural Systems.

Hybrid Structural Systems Interactive frames and walls offer many advantages. As Fig.4 shows, plastic hinges in frames may be more freely chosen because walls can ensure that a "soft storey" will not develop. A weak column-strong beam system may also be used because column hinges will develop simultaneously over several storeys, as indicated in Fig.4(c). In this case, however, end regions of columns must be provided with the full required amount of confining reinforcement. The capacity design of components of hybrid structures is very similar to that described previously.

THE QUALITY OF DETAILING

Concrete is brittle. Deformed steel bars usually possess ample ductility. The art in the design for structural survival consists foremost of the skillful combination of these two materials to produce sufficiently ductile composite response in all critical regions. Detailing of reinforcement includes this singularly important aspect of seismic design.

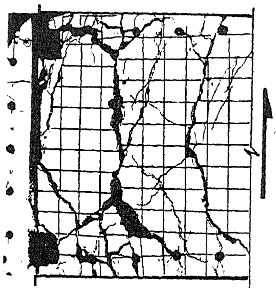

Fig.5 - Large Shear Displacements Along Interconnecting Flexural Cracks Across a Plastic Hinge of a Beam

Plastic Hinges in Ductile Frames   As long as the amount of flexural reinforcement in the top and the bottom of a beam section is not very different, concrete plays a relatively minor role in flexural strength and ductility. Hence it does not require special attention. However, great care must be taken with the protection of each beam bar against buckling when subjected to large cyclic strains. To ensure that large plastic hinge rotations can be sustained, both the spacing of ties and their strength is considered to be important. Rotational ductilities of plastic hinges of up to 8 were repeatedly attained when the spacing of a tie, stabilizing a bar, did not exceed 6 times the diameter of that bar. This close tie spacing is only required in the clearly defined plastic hinge length of a member.

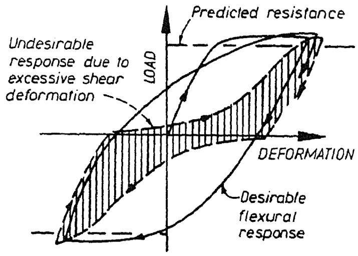

Fig.6 - A Comparison of Hysteretic Responses.

Although stirrup reinforcement, provided in the traditional way in plastic hinge regions, should prevent diagonal tension failure, large sliding displacements along interconnected wide flexural cracks, forming a failure plane, as shown in Fig.5, may occur. The ensuing loss of energy dissipation, as shown by the response corresponding with the dashed curve in Fig.6, may be significant when the shear stress is large, as in short span beams, and the magnitudes of beam shear force associated with each direction of earthquake attack are similar. Hysteretic response is greatly improved when diagonal reinforcement, to resist at least a part of the seismic shear force, is provided as shown in Fig. 7.

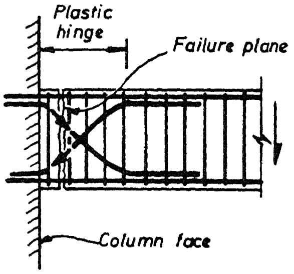

Fig.7 - The Control of Sliding Shear in a Plastic Hinge by Diagonal Reinforcement.

Over 15 years of research in New Zealand (Ref.2) assisted in the identification of the mechanisms providing confinement of concrete in plastic hinges of columns, which for example are expected at foundation level. Confining reinforcement with appropriate amounts and configuration will transform brittle response of concrete into a ductile one. Moreover, the compression strength of the confined concrete may be significantly increased. These properties are illustrated in Fig. 8. In rectangular columns it is vital that confinement be provided by holding closely spaced large diameter vertical bars in position as seen in Fig.9(a). Even large amounts of hoop reinforcement, often used in the form shown in Fig.9(b), will be ineffective in both roles, to confine a concrete core and to protect column bars against buckling.

Beam-Column Joints   The strategy used in New Zealand for the design of joints in ductile frames rests on the precepts (Ref.3) that, in terms of both stiffness strength, joint response should not control the response of frames. This is because the basic mechanisms, one transmitting shear forces and the other providing anchorage by bond to beam and column bars, are unsuitable for energy dissipation. Moreover, possibilities for the repair of joints are very limited. The transmission of shear forces is based on the models shown in

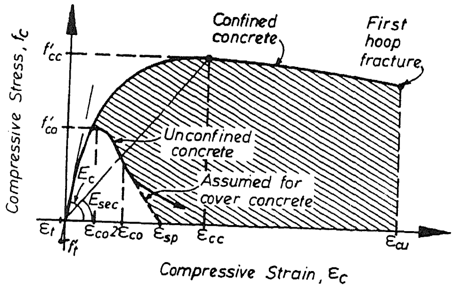

Fig.8 - Stress-Strain Curves for Confined and Unconfined Concrete (Ref.2).

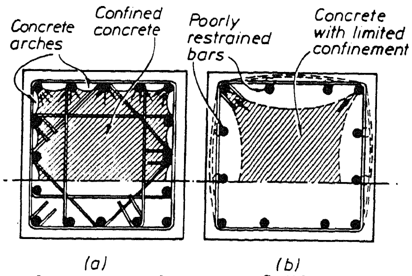

Fig.9 - Contribution of Ties to the Confinement of Compressed Concrete.

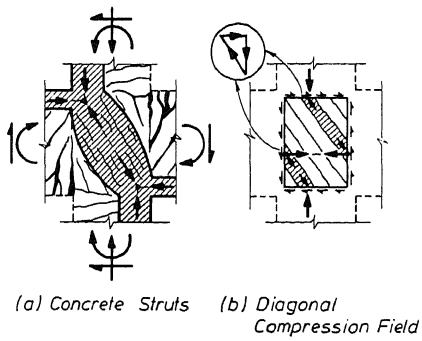

Fig.10 - Mechanisms of Shear Resistance at an Interior Beam-Column Joint.

Fig.10. The concrete strut mechanism (Fig.10(a)) is very efficient as long as members around the joint are elastic. A truss mechanism, requiring both horizontal and vertical joint shear reinforcement, may be fully mobilized when plastic hinges develop adjacent the joints. If premature bond failure, leading to large increases of storey drift, is to be avoided, the diameter of bars passing through a joint must be limited.

Some unconventional solutions attempt to eliminate critical aspects of joint performance. By relocating beam plastic hinges, as shown in Fig.11, yield penetration along beam bars into the joint core, the major cause of premature bond slip, can be prevented. To reduce congestion of horizontal joint shear reinforcement, usually arranged in a form shown in Fig.9, large diameter hoops can be placed outside around a column, if horizontal haunches, as shown in Fig.12, are used. A possible arrangement of joint reinforcement for such a specific case is shown in Fig. 13.

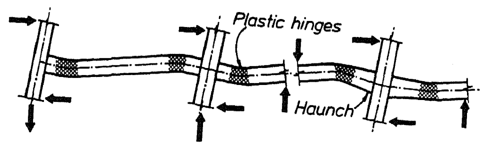

Fig.11 - Beams with Relocated Plastic Hinges.

Ductile Structural Walls Although vertical boundary elements for cantilever walls are desirable, they are not considered to be essential when the thickness of the wall is sufficient to ensure that out-of-plane buckling does not occur. Typical details of the confinement of the concrete within the computed compression zone over approximately 900 mm, are shown in Fig. 14. The principles used are the same as those applied in Fig.9. When coupled walls, such as seen in Fig.3, are used, it is essential to ensure that unusually large deformations in the coupling system can be sustained without strength degradation. Conventionally reinforced coupling beams, like short columns, usually fail in diagonal tension (Fig.15(a)). If sufficient stirrup reinforcement is provided, failure with limited ductility is typically due to sliding shear (Fig.15(b)). For this reason diagonal reinforcement, ensuring excellent hysteric response, is routinely used in New Zealand (Fig.15(c)). The same concept can also be used in squat walls, such as shown in Fig.16, where the prevention of sliding shear failure with the use of additional diagonal bars, as in Fig.16(c), will then ensure very ductile flexural response. Diagonal reinforcement is also used extensively in New Zealand in short spandrel beams of ductile tube frames.

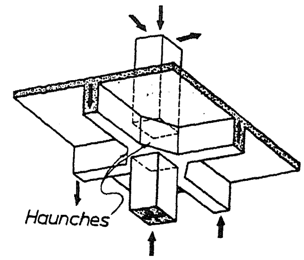

Fig.12 - An Interior Joint Formed by Horizontal Beam Haunches.

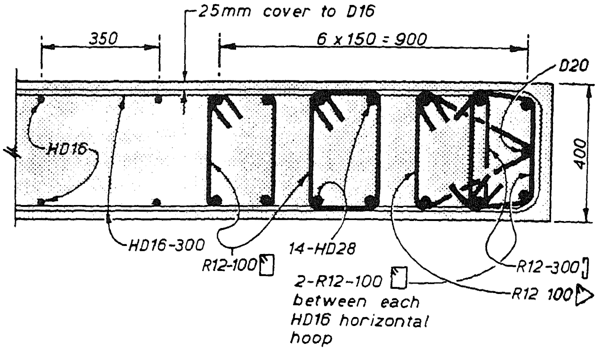

Fig.14 - The Confinement of Compressed Regions of a Wall Section.

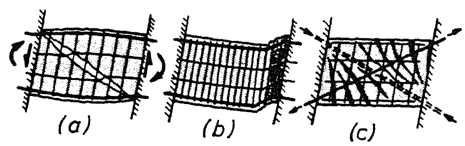

Fig.15 - The Mechanisms of Shear Resistance in Coupling Beams.

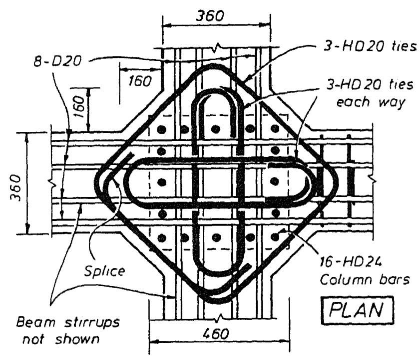

Fig.13 - Joint Shear Reinforcement Formed Within Horizontal Beam Haunches.

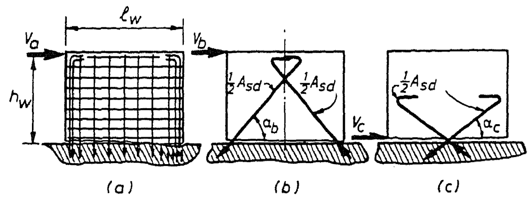

Fig.16 - Conventionally Reinforced Squat Walls with Additional Diagonal Bars

CONCLUSIONS

By necessity only a few highlights in the evolution of the current seismic design strategy in New Zealand could be mentioned briefly. Descriptions of the design procedure intended to emphasise the designer's determinations to "tell the structure what to do". Examples were presented to manifest attempts to quantify goodness in detailing, which in terms of seismic response will make structures extremely tolerant, so that they can perform "as they were told to".

REFERENCES

1. Otani, S., Li, S. and Aoyama, H., "Moment-Redistribution in Earthquake Resistant Design of Ductile Reinforced Concrete Frame", Transactions of the Japan Concrete Institute, 9, 581-588, (1987).

2. Priestley, M.J.N. and Park, R., "Strength and Ductility of Concrete Bridge Columns under Seismic Loading", ACI Structural Journal, 84 (1), 61-76, (1987).

3. Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley & Sons, New York (1975).