GUIDELINES AND PROCEDURES FOR STRENGTHENING OF BUILDINGS

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SUMMARY

General design guidelines developed for the New Zealand Ministry of Works and Development for the seismic strengthening of buildings are presented, together with a discussion of the levels of risk associated with alternative levels of strengthening and the economic comparison of those alternatives.

A variety of approaches to strengthening are described. These cover unreinforced masonry buildings and also buildings of reinforced concrete or with structural steel frames incorporating rivetted beam to column joints. Considerations which govern the design of various details are discussed.

INTRODUCTION

The New Zealand Ministry of Works and Development is responsible for assessing the seismic resistance of more than 5,500 state owned buildings, setting guidelines for strengthening schemes and designing and implementing strengthening measures. Most of these buildings were built to earlier codes, and hence do not comply with earthquake resistance requirements of current codes. The worst risks are presented by the unreinforced masonry structures constructed before 1935. About 1,000 of the buildings are in this category, and priority in replacement or strengthening is being given to these.

DESIGN LEVELS FOR STRENGTHENING

Where a building is to be strengthened and refurbished, as an alternative to demolition and replacement to provide an "unlimited" future life, i.e. a future life of more than 50 years, one principle adopted is that the risk to life in the strengthened building must not be significantly higher than the risk in a new building built to present seismic codes. On the other hand, the acceptable property risk is determined on economic grounds.

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New Zealand is divided into 3 seismic zones, denoted A, B and C respectively. Zone A has the highest level of seismicity, and for that zone the national design loading code, NZS4203 (Ref. 1), sets levels of design lateral forces which can cope with the building response to an MM9 earthquake, with maximum ground accelerations up to 0.4g. For coping with the very infrequent earthquakes in Zone A that have intensity greater than MM9 reliance is placed on the existence of a greater ductility capacity than the minimums called for by the provisions of NZS4203.

It is necessary to decide on the minimum permissible seismic strength in Zone A for a building upgraded for unlimited future life. In a number of regions in Zone A the assessed return period for MM9 or greater earthquakes is 220 years and the probability of such an event occurring in a 60 year building life would be approximately 24 percent. For such a high probability of occurrence, a low life risk in an MM9 earthquake is essential. This indicates a minimum strength of $\frac{2}{3}$ of NZS4203 seismic design loads. A building strengthened to this level is theoretically subjected to a 50% overload in an MM9 earthquake, and withstands it by calling on reserve seismic capacity which includes the ability of shear-wall systems under in-plane loading to deflect a long way beyond the limit of ductile response deformation before collapse. Because of the larger response deformations involved, $\frac{2}{3}$ code level strengthening will result in greater building damage in strong earthquakes than full code strengthening.

Similar conclusions as to the acceptability of $\frac{2}{3}$ code level strengthening apply in seismic Zones B and C, where NZS4203 sets seismic design loads at 83% and 67% respectively of the Zone A values.

ECONOMIC COMPARISON OF STRENGTHENING OPTIONS

Normal practice of the Ministry is to compare the economics of full code strengthening with $\frac{2}{3}$ code strengthening for each building. Preliminary designs for the two alternatives are developed far enough to enable rough order of cost estimates to be prepared. The probable average annual seismic damage loss is then calculated by taking for each intensity level on the MM scale the product of the probability of an earthquake of that strength occurring during any 12 month period with the probable damage cost corresponding to such an earthquake, and then summing all such products over the appropriate range of intensities. New Zealand Treasury guidelines require that if $1,000 of damage are predicted to occur n years after the present date, the present value of that sum must be calculated as $1,000 divided by 1.1^n. (This rule is based on the premise that any Government capital expenditure project such as a strengthening scheme, should increase in capital value in step with inflation and should also earn an income of at least 10% per year. Hence the present value of an annual seismic damage loss of M summed over a 60 year life of a building would be:-

$$M\left(\frac{1}{1.1} + \frac{1}{1.1^2} + \text{----} + \frac{1}{1.1^{60}}\right)$$

which is 9.96M.

The sum of the cost of strengthening and refurbishing plus 10M for the full code option is compared with corresponding sum for the $\frac{2}{3}$ code alternative, to obtain the economic comparison. The fact that the level of life risk for the $\frac{2}{3}$ code alternative, although very low, is still greater than that for the full code option is also taken into account.

Strengthening for limited life is also considered as an alternative. A strengthening scheme that will achieve ½ code seismic strength is considered as an acceptable option if the future life of the building is limited to 25 years and if its cost is less than half replacement cost. The smallest return period in any locality in New Zealand for an earthquake strong enough to cause significant risk to life in a building strengthened to that standard is 220 years, and the risk of such an earthquake occurring during a 25 year period is of the order of 10%. Even though this represents an appreciably greater life risk than a replacement building to full code standard, it is considered acceptable because of the finite period of the life risk. Also, on a national scale, being able to strengthen 2 buildings for the cost of replacing one can even result in a risk reduction, because of the amount of money available each year to replace or strengthen hazardous buildings is limited. However usually the economics of ½ code strengthening does not compare favourably with $\frac{2}{3}$ code strengthening, taking into account the significantly higher life risk and the limited future life. It is only when the $\frac{2}{3}$ code standard requires a different form of strengthening scheme that it may become appreciably more expensive than the ½ code alternative.

Strengthening schemes achieving less than ½ code level are usually not considered because the resulting life risk is too high.

UNREINFORCED MASONRY BUILDINGS

Guidelines for Design of Strengthening

The guidelines set for design of strengthening schemes are as follows:-

1. To Strengthen to Current Code Requirements

(a) Strength and Ductility

Strengthen the structure of the building, using materials with predictable properties, to resist the earthquake forces specified in NZS 4203, with sufficient ductility to provide the necessary energy absorption. The S-factor chosen for calculation of the Cd coefficient should be related to the ductility of the seismic resisting system.

(b) Deformation Characteristics

Ensure that the deformation characteristics of the new structure are such that collapse of the existing gravity load bearing elements is unlikely under ultimate deflections into the inelastic range.

(c) Integrity

Tie all structural elements together in such a manner that no collapse of any portion of the structure will occur under design seismic loading or ultimate post-elastic design seismic deflections.

(d) Non - Structural Elements

Support all potentially hazardous non-structural elements (such as parapets, gables, masonry partition walls etc) in such a manner as to prevent collapse under design seismic forces.

(e) Aesthetics

Design strengthening work in such a manner as to minimise the effect on the external appearance of the building.

2. To Strengthen to $\frac{2}{3}$ or ½ Current Code Requirements

As above except that design earthquake forces are to be taken at $\frac{2}{3}$ or ½ those set down in NZS4203 for the level of ductility that is achieved in the strengthened building.

Form of Strengthening

This takes into account architectural replanning and upgrading. One extreme consists of replacing most of the building with a complete new building, retaining only one or more facades, which form some of the exterior walls of the new building. At the other extreme are projects where all the elements of the original building are retained, including all internal masonry walls.

The unreinforced masonry walls are retained and relied upon only for carrying gravity loads and are not designed to resist any in-plane seismic loads. The lateral seismic loads on the structure is resisted by a system of new concrete shear walls, some of which are usually provided in the form of reinforced sprayed concrete backing to the unreinforced masonry walls. This backing also supports the masonry elements against seismic face loads. For very thick walls satisfying minimum strength requirements, with a substantial length between wall openings and a storey height-to-thickness ratio no greater than 10, seismic face loads may be assumed to be resisted by the existing masonry walls without backing if the building is in an area of low seismicity with a long return period for MM8. (For example in Auckland Zone

C the return period for MM8 is assessed at 1,400 years). Special dispensation for omitting backing in thick walls is also made in historic buildings where the interior character as well as the exterior character requires to be preserved and the occupancy is low.

Design Stiffness of Strengthening Shear Walls

The design guidelines already given require the design deformations of the strengthening structure to be limited to values small enough to avoid collapse of the existing gravity load bearing elements. The criteria adopted for complying with this requirement is to keep the strains in unreinforced masonry walls, resulting from design seismic loads on the structure, within the elastic strain limits of reinforced concrete. Where the masonry walls are backed with reinforced concrete, this limitation can be achieved by apportioning in-plane lateral loads to the concrete walls on the basis of elastic analysis and then designing the concrete walls by ordinary ultimate strength procedures, with concrete compressive strains not exceeding 0.003. Also, conditions where a small increase in loading above design load would result in large increases in deformation should be avoided. This means that uplift and foundation failure in shear walls should not occur below 1.2 times the loading at which the wall yields.

Ductility Capacity and Design Loads

Testing which the Ministry's Central Laboratories have carried out (Ref. 2) to date indicates that a composite wall consisting of original brickwork of reasonable quality backed with a reinforced concrete strengthening layer can withstand in-plane cyclic deformations of 2½ times the yield deformation of the concrete without any noticeable cracking of the brickwork or failure of adhesion between the brickwork and concrete. Hence, in unreinforced masonry buildings, the procedure adopted has been to design the concrete backing walls and other forms of concrete shear walls for seismic loads which are based on an assumed ductility factor of 2.5. In NZS4203 the design lateral force coefficient is given by the product CISMR, where the product of S and M is governed by the ductility capacity of the structure. If a structure has to withstand the full design earthquake in the elastic range, the SM product is set at 4 to 5, while in a structure having the highest ductility level is set at 0.64. Hence for shear walls backing unreinforced masonry in strengthening schemes, the SM product is set at 2.0, which corresponds to a "limited ductility" response involving a ductility factor of 2.5. In a structurally stiff public building in Seismic Zone A, this would result in a design lateral force coefficient for ultimate strength of 0.39g in flexure and 0.78g in shear.

In the maximum credible earthquake the ductility demand may amount to more than 2.5. This could result in damage such as cracking of brickwork and loss of adhesion to the backing concrete in some areas of strain concentration. Such damage is considered acceptable even in a historic building for an earthquake with a return period of 400 years or more, provided that the life risk is low and repairs can restore the original appearance.

Tying of Masonry to Concrete Backing

This is achieved by a combination of epoxy grouted metal ties and adhesion of brickwork to concrete backing. Ties at 1.2 metres centres have sufficient anchorage to resist all face load forces, and the adhesion is relied upon to minimise the risk of small portions of brickwork between ties breaking loose. The surface of the brickwork is cleaned to provide optimum adhesion conditions.

Cavity Walls

These present the problem that, if the backing layer is placed against the inner skin and tied to it, the problem of providing support against face loads to the outer skin arises. Continuation of ties across cavity into outer skin involves costs and problems and there is the risk of corrosion of ties in the cavity in the New Zealand climate. If the building is in grounds which can be planted around the outside walls to keep people away from the walls, no additional tying is provided for the outside skin. The most usual measure adopted is to grout the cavity and rely on the adhesion of grout and the existing wire ties across the cavity to hold the outer skin. Although it is not possible to clean the brickwork surfaces in the cavity, cores taken through walls with grouted cavities have indicated good adhesion where an expansive additive has been used.

Timber Backing Walls

These have shown favourable costs in some schemes, and simply consist of providing a timber structural wall either as a replacement for the inner skin of a cavity wall or else placed against the inner face of a solid masonry wall. Ties grouted into the brickwork are connected to the studs of the timber wall, and plywood sheathing is fixed to the studs to resist in-plane loading. This virtually converts the brick walls to timber framed veneers and it is appropriate for single storied construction.

Thin Coatings

Thin coatings of GRC and steel fibre reinforced concrete applied to both wall faces have been tested by our central laboratories and reported elsewhere (Ref. 3).

On one part of the strengthening work for Wairarapa College, coatings of GRC on the inner face of masonry walls have been used for resisting face loads. On another scheme where the outside of the building was plastered and hence exterior GRC coats were acceptable, one option investigated for reduced costs was GRC coatings on both faces of exterior walls, to provide flexural tension membranes.

Floor Diaphragms

Whenever possible existing floors are utilised by tying them into the new backing for the walls. Existing concrete floors usually have sufficient strength and only require a positive connection to the new backing and any new walls. Existing timber floors on the other hand have been found to require strengthening as well as tying in. Structural plywood or particle board applied as an overlay or to the ceiling has been found to be cheaper than steel bracing for this purpose.

STRUCTURAL STEEL AND REINFORCED CONCRETE BUILDINGS

Until the publication of NZS4203:076 (Ref. 1) and NZ MWD Code of Practice for Seismic Design of Public Buildings:1968 (Ref. 4), the designers of the moment resisting frames to resist earthquake loads paid only lip-service to ductility requirements. In particular most frames designed prior to the 1970s are deficient in both column shear capacity and beam-column joint capacity.

However, as an exception, the results of full scale testing of a concrete encased rivetted steel beam-column joint from a 7 storey building (Ref. 5) should be noted. The unit proved to be more ductile and twice as strong as analysis suggested with failure occurring in the beam at 30% in excess of the beam nominal design strength. The results indicated that testing is desirable to make a realistic assessment of concrete encased rivetted beam-column joints.

The strengthening approach has been to limit interstorey drift to protect columns by adding shear walls or diagonally braced steel frames.

Shear Walls

These are designed to current code requirements and are slightly offset from existing frame grid lines, so as to pass along the faces of existing beams or columns. This avoids complicated junctions with beams and columns and allows vertical wall reinforcement to penetrate existing floors. They can also be added on the outer faces of the building where this is architecturally acceptable. The tying into the existing structure in the latter case is more difficult, but such a scheme for a 7 storey building showed an estimated 26% saving in construction cost and did not disrupt the functioning of the building to the same extent as other methods.

Diagonally Braced Steel Frames

Both concentric and eccentric diagonally braced steel frames have been considered. Although, they are usually more expensive, up to 2½ times as expensive as shear walls, they have the advantage of involving much less site work inside the building and easier connections through the floors between stories. This results in much less disruption to the building occupants. Steel frames have the further advantage over shear walls that they do not have to be built from the bottom up and hence permit a more flexible construction sequence to coincide with the owner's refurbishing programme.

As with shear walls, the new frames should be offset to the sides of existing frames, to reduce complications in joints to beams and columns. The resulting eccentric effects in connections to existing members must be considered together with deflections at full design ductility that result from the new system.

Eccentrically braced frames may be designed for a low SM of 0.64 (in terms of NZS4203) because of their good ductility but difficulty has been experienced in providing stiffness compatibility for a stiff building even for response in the elastic range.

Concentrically braced frames provide better stiffness over the elastic response range, but show a sharp stiffness degradation when cycled in-elastically. Use of such elements requires a special study of deflections at various response levels.

CONCLUSION

Space limitations have restricted any considerations of detail, particularly for structural steel and reinforced concrete buildings. The whole field of strengthening of buildings is in a development situation and it will continue to call for much innovation and testing on the part of designers.

References

1. Including amendments Nos. 1 and 2 and draft amendment A3, NZS4203:1976 Code of Practice for General Design Loadings for Buildings. Standards Association of New Zealand.

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5. P J Wood. Cyclic Testing of a Concrete Encased Rivetted Beam Column Joint. New Zealand Ministry of Works and Development Central Laboratories Report 5-82/2.