THE DESIGN OF PUBLIC BUILDINGS+

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SYNOPSIS

This paper may be regarded as complementing that by Mr. I. C. Armstrong elsewhere in this volume. The author discusses in general the present attitude of his office towards aseismic design, the changes and developments since a report on the same subject at a symposium in 1968(1).

INTRODUCTION

The Code of Practice for the structural design of public buildings was last reviewed in 1970. It is intended to bring out a new edition later this year. Researchers in many countries are investigating at present the effects of the 1971 San Fernando Earthquake and it seemed prudent to await the result of these. Hopefully, too, the appearance of several new SANZ (Standard Association of New Zealand) Codes is expected. While some of the proposed changes discussed below have not yet appeared in print as amendments, many are already applied on a trial basis to buildings designed in the various offices under the writer's technical control.

PURPOSE AND NEED FOR A PUBLIC BUILDING C.O.P.

This subject was dealt with in detail in the above mentioned report(1). A review of the statements then made indicates that they still hold. One reason given was speedier application of the findings of the researchers and lessons from earthquake damage than through total dependence on SANZ Codes. Re-organisation of the National Codes into By-Law and Code of Practice (C.O.P.) parts is a great improvement with regard to facilitating adoption by local bodies but the production phase seems as time consuming an effort as ever. The writer now realises that this is not only due to N.Z.'s traditional method of producing Codes by voluntary effort from people with full time jobs but simply because highly competent, professional assistance free of industry ties is just not speedily available in this country. Could it be that contrary to some opinions the glamour of "Management" has cost us too many of our technically skilled people? And what about rough drafts completed by Project Committees that cannot be circulated for months because SANZ, - inadequately funded and thus short of technical advisers - is unable to deal with them!

Experienced technical staff in the writer's office too have only limited time available for research. In order to produce an up-to-date code they must concentrate on the areas where they believe national codes are either dated or inadequately covering the interests of the government as owner and insurer of its buildings. The resulting composite document has user disadvantages as it requires at times considerable cross referencing. However, any attempt to produce a code complete in itself would defeat the object of speedy up-dating.

DUAL STANDARDS

The writer considers it of utmost importance that the current wide gap between what most informed engineers in this country consider up-to-date seismic design practice and the minimum permitted by national codes should be eliminated. To cite a most serious example:

The current N.Z.S. Chapter 8 states in Clause 8.41 without qualification that "the building as a whole shall be designed with consideration for adequate ductility." N.Z.S. 3101P 1970 gives methods of detailing R.C. for adequate ductility (or perhaps nearly adequate) but neither of these provisions are currently enforced for the bulk of non public buildings because N.Z.S. Chapter 9.3A Clause 1.3. requires ductility only where in the opinion of the "Engineer" special circumstances exist. The term "Engineer" in this context has such a wide meaning that it includes many who are not equipped to judge the implications of lack of ductility.

Professional engineers, the writer included, intuitively dislike mandatory requirements, for if dated they may hamper those who are well informed and - this is important in this design and build age - free to apply engineering considerations without undue economic pressures. Since this is frequently not the case, up-to-date code provisions in some detail are of great assistance to those wishing to produce a sound engineering structure under competitive conditions. A recent example known to the writer concerns a number of reinforced concrete buildings produced by two consultants and Ministry of Works for the same client on the same site. One of the consultants and Ministry of Works applied the ductility provisions of N.Z.S. 3101P but the designer of the third structure gave no consideration to the provision of ductility. No doubt his structure was cheaper.

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DUCTILITY REQUIRED FOR LOW BUILDINGS

The M.O.W. Code of Practice has since 1968 required all framed buildings (in all seismic zones) higher than one storey with light roofs and all structures with heavy roofs to be detailed for ductility. Such stringent requirements have at times resulted in criticism of the Department by uninformed persons. The recent failures and serious damage to single storied buildings at San Fernando, two and three storied buildings at Tokachi-oki in "normal" reinforced concrete designed to N.Z.S. Code levels or higher (0.2g at Tokachi-oki) are full justification for our ductility requirements. These are currently being further revised to take latest test information into account.

ASSISTANCE TO SANZ

M.O.W. recognising that the best way of eliminating dual standards is by way of assisting SANZ is actively engaged in helping to up-date several national codes. Of particular interest to the writer's office are naturally the chapter on "General design and loadings" and the Materials Codes. It is unlikely that there will be any significant differences between our C.O.P. and the new chapter 8 or the concrete design code although revision of the latter, considering ACI 1970 and San Fernando has not yet been started. Interim provisions on the lines indicated in the second part of this paper will thus be required.

DAMAGE CONTROL

Older N.Z. Standards were concerned to a very limited degree with the prevention of damage but this is no longer the case (refer to paper on proposed N.Z.S. General Structural Design and Loadings Code). The degree to which damage must be prevented in a "Public Building" required to function following disaster is greater than that which a national code can prescribe for non public buildings and this difference will again be reflected in our provisions.

The absence of earthquake insurance concessions in this country for buildings of better than minimum code level earthquake resistance (whether adequate or not) are a hindrance to good practice. Clients are difficult to convince that it is worthwhile to spend on the structure more than the absolute code minimum required when the return to be derived from such extra capital expenditure is dependent on rather limited statistical evidence. A reason given for uniform earthquake insurance rates (often lowest for the highest seismic risks because of fire rating considerations) has been the high cost of the necessary professional investigation to determine the risk. The writer believes that the real benefit to the insurer would not be derived from the difference in insurance rates but the saving following the occurrence of a severe earthquake. This applies particularly with regard to non structural earthquake damage resulting typically in the greatest monetary loss in modern buildings. In this respect the present chapter 8 provision allowing 0.25% drift without separation clearance of parts is particularly objectionable in the light of current knowledge or ductile building behaviour. Separation is widely avoided by just meeting the above stiffness level.

LOW EARTHQUAKE RESISTANCE NOT IN NATIONAL INTEREST

It has been suggested that earthquake resistant design, particularly damage control is largely an economic exercise and that a fairly low level of resistance is adequate provided sufficient funds are set aside and invested in a manner that would ensure their availability following disaster. In a small country of limited local resources already unable to cope with normal building activity provision of low earthquake resistance would require massive import not only of materials but also of labour. This would lead for years following seismic disaster to unacceptable disruption of the social structure. Additionally, material damage and life risk are often only a hair's breadth apart. A meagrely separated masonry partition may become the cause of totally altered structural behaviour (as in the Caracas earthquake) or it may be crushed and block vital exist ways.

New Zealand has been fortunate so far in sustaining relatively few casualties from earthquakes compared to say road deaths and other voluntary risks. Such a comparison neglects the difference between the problem of dealing with casualties occurring at a fairly predictable rate day after day, year after year and a catastrophe which requires a total immediate rescue effort comparable to one normally spread over decades. Do we equate the cumulative pains of shaving to that suffered in a major fracture of a limb? In a country where the standard of living allows a substantial and increasing expenditure on the decoration of buildings (expensive finishes, pools and fountains) - the expenditure on earthquake resistance should keep pace.

What are then the principal changes proposed?

INCREASED BUILDING RESPONSE DUE TO OVER-STRENGTH

Ductility demand is reduced by over-strength but elastic response is increased, because the yield level is raised. It is important that building services engineers should be made aware that their equipment will be subjected to much greater accelerations than is evident at first sight. The same considerations apply to parts and fixings that are to remain elastic. The principal reason for the difference between building response deduced from the loading code and the possible maximum is due to a number of built-in over-strength factors which prevent a building from yielding at code levels. The present code requires a load factor of 1.25, the actual yield strength of the steel, rounding up of bar numbers or section sizes may account for a further 25% increase. 0.8 may be the material under capacity factor used.

Some well designed buildings will thus be overstrength by a factor of $2 = \frac{1.25 \times 1.25}{0.8}$. Many are likely to have at least 50% over capacity. While such a situation is favourable with regard to our (probably over optimistic) ductility reduction factor of 4 it results in high building accelerations. The elastic response of a building is related to capacity at yield and hence is a function of the horizontal design co-efficient C_d. Where triangular distribution applies the local seismic co-efficient K_x (NZS Chapter 8) at top is $2C_d$. Accelerations at the top of buildings would thus generally be $3C_d$ to $4C_d$. Damage potential is a function of both pseudo velocity and pseudo acceleration as well as more complex dynamic response considerations but for practical code purposes the following is useful:

The force on a part or proportion of a building should be taken to be

$F_s = C_p \, C_{dp} \, W_p$ and

$C_p = C_d \, (K_p \, R_p)$ and

W_p = seismic load for part

C_d = horizontal seismic force design co-efficient for building

$C_{dp} = S \times \gamma$ as related to the part.

K_p is a multiplier for C_d as discussed previously where

$K_p \ngtr 1.5 \, K_x > 1.5$

R_p is a risk factor related to the part

In cases where essentially triangular distribution does not apply or where a part is located at a lower level of a building appropriate modifications should be made. Although the proposal appears complex this is not so in practice. The code will give suitable combinations of $(K_p \, R_p)$ and in many cases $C_{dp} = 1$. e.g.,

A masonry partition with a fire rating located at the top storey of a reinforced masonry non-public building, seismic zone A, ($T < 0.45$ seconds) would be designed for: (refer to paper on proposed loading code elsewhere in this volume)

$C_d = CIS\gamma R = 0.2 \times \frac{3}{4} \times 1.6 \times 1.2 \times 1 = 0.29$

$K_p = 3$, $R_p = 1\frac{1}{3}$ therefore

$R_p \, K_p = 4$

$C_p = 4C_d$

$C_{dp} = S \times \gamma = 1.0 \times 1.2. = 1.2$ therefore

$F_s = 4 \times 0.29 \times 1.2 = 1.4 W_p$

It is found that the reinforcing requirements are very moderate using ultimate strength design procedures.

There are cases where it becomes necessary to establish upper and lower limits for C_p. It is important to note here that peak building accelerations will occur in relatively small earthquakes and therefore, quite frequently. Reference to typical NZ response spectra indicates that for 2% damping - a reasonable value for stress levels below yield - the amplification of ground acceleration may be as high as 3.2 ($T = 0.3$ seconds). A $3C_d$ acceleration at the top of a structure with $C_d = 0.15$ represents therefore only the magnification of a ground spectrum acceleration of $3 \times 0.15 = 3.2 S_a \times 2$ therefore $S_a = 0.07$, which is only 21% El Centro.

If for purposes of this exercise the response to $\frac{1}{2}$ El Centro type motion is used as the maximum in Zone A for parts not likely to fail in a brittle manner we may establish this level at $0.165 \times 2 \times 3.2 = 1.1$g. This figure should be multiplied by a R_p factor. For seismic Zone C presumably $\frac{1}{4}$ El Centro would be appropriate. On the other hand parts likely to fail in a brittle manner should withstand the accelerations of responses to 40 or 50%g ground acceleration.

As previously stated our reference point is earthquake performance and here we have all the evidence to support the above reasoning - from turned over lift machinery, buckled water tanks, failed parapets and last though not least the alarming experience of occupants in high buildings shaken by motions not even noticed on the ground.

HEIGHT LIMITS

In view of the sometimes contentious nature of this subject it is interesting to follow the history of height limits in recent years and apply hind sight to the reasons for and against these. Height limits for structures, particularly reinforced concrete were common prior to 1960 in California, Japan and New Zealand. When dynamic earthquake analysis methods were developed the continued existence of height limits was considered by many as unjustified. An undeniable benefit of the 160 ft. height limit for reinforced concrete in California was that it resulted in greatly increased funding for the concrete research which led to Blume, Newmark and Corning's book on "Design of Multi-Storey Buildings for Earthquake Motions", and the consequent removal of this height restriction. The result of this research work proved of course the prudence of the height limit since the investigation indicated that reinforced concrete frames as built up to that time were totally inadequate to resist severe earthquakes.

Unfortunately many reinforced concrete frames built in accordance with the above named work are likely to prove almost as inadequate as their predecessors because notwithstanding the giant stride made one vital consideration had been overlooked - the existence of extremely high joint shears due to the concentrated reaction of the yielding beam reinforcing. Compare the recommendations for joint stirrups on page 158, fig. 6.6. with the cages of 5/8" and heavier rods at close centres later tests indicated as being necessary. And just as designers thought that all was well researchers at Canterbury University showed that if one took the axial load off columns one could rotate the nicely tied exterior beam column joint together with the beam in one piece out of the columns. So designers went back to their drawing boards and talked architects into putting sun shields, supported by massive beam projections beyond the exterior columns, on to their buildings. But what of the high buildings already up? How fortunate that the N.Z. Code loading levelled out at $T = 1.2$ seconds rather than followed El Centro response. So there was extra margin for high buildings! - or was there? El Centro's 0.33g is now regarded as inadequate. We must now think in terms of 0.5g or even 1g. And what are these ugly humps at 1 second and longer periods reported by Seed[2]? Concurrent earthquake effects, column beam joint shears due to yielding of all beams framing into a joint, vertical earthquake effects! What next?

Do the same considerations apply to low structures? Many do but there are differences. For low structures:

1. Analysis is more predictable. Simplifying assumptions are of less consequence. How many stories will form simultaneous plastic hinges? Horizontal accelerations at higher levels due to rocking? Dynamic amplification of torsions?
2. Details are <u>much closer</u> in scale to the sizes tested in laboratories.
3. Margin over code minimum is usually greater than that of a high building.

And, all failing, a small building comes down with a smaller thump than a large one!

The writer's feelings thus are that we have moved too fast upward in reinforced concrete and if, notwithstanding this, there will be no height limit for reinforced concrete in our next code of practice that is due to the fact that none will be needed - practical detailing considerations impose their own height limits. The time-worn joke about checking the reinforcing cage by putting a canary in has finally become true. The only problem is how to put the canary in!

Height limits will be continued for reinforced hollow masonry. It is becoming increasingly obvious that ductility under dynamic loadings is difficult to achieve even under laboratory conditions. The material is thus put to best use in low structures designed for high loadings that will ensure near elastic behaviour in most earthquake situations.

LOAD FACTOR EQUATIONS, EARTHQUAKE FORCE MULTIPLIERS AND REDUCTION FACTORS

These will be identical to those proposed for the N.Z.S. code and discussed elsewhere at this symposium by the writer.

DUCTILITY OF REINFORCED CONCRETE FRAMES

Current provisions are partly based on SEAOC. New M.O.W. Ductile frame detailing will take into consideration SEAOC, ACI 1971 and work done at Canterbury University. More detailed information is given in Mr. Armstrong's paper.

SHEAR WALL STRUCTURES

The satisfactory earthquake performance of low and medium height buildings in this structural form cannot be ignored. It may, however, have been due principally to two factors - accidental overstrength and inherent stability of damaged walls to in-plane loadings. Recently there has been a trend by some designers to provide buildings with minimum code strength shear walls only. This is dangerous and higher loadings must be specified for a structural form that dissipates seismic energy in a shear mode.

DUCTILE SHEAR TOWERS

Structures of this type may be regarded either as vertical cantilever beams of such dimensions that hinge formation is not a stability problem or as deep membered frames detailed to prevent non-ductile failure of the coupling beams.[3] The continuing confidence of the writer in this form of construction is reflected in the relatively low structure type coefficient required. There will be no change from our past design approach (1) (Part B). The SEAOC provisions relating shear stress in deep beams to $^{H}/_{D}$ ratios were discontinued in 1969 as a result of a personal communication from Dr. Paulay.

REINFORCED MASONRY

The writer is hopeful that M.O.W. may be able to essentially accept the new SANZ code now in preparation provided there is no change of heart from the proposed high force multipliers contemplated at present.

It is, however, proposed to adhere to the current minimum requirements for closely spaced rebars in both directions in order to encourage the development of a controlled, distributed crack system whatever its cause - seismic, thermal, shrinkage or settlement.

The writer fails to fully understand the opposition in some parts of industry to the use of horizontal reinforcing. Horizontal bars are easier placed than vertical ones and in addition do not increase flexural strength which in turn increases demand on shear strength. It appears that this opposition stems from the use of two skin structural wall construction in $3\frac{5}{8}$" wide masonry units in some of the less earthquake conscious parts of New Zealand. The practice has developed as a result of leakage problems with single skin construction. $3\frac{5}{8}$" walls of concrete masonry units cannot satisfactorily be reinforced and grouted under field conditions even when uni-directionally reinforced only and much less so when the tiny cavities are blocked by two-way reinforcing.

The use of thin reinforced masonry walls should be discouraged for other reasons too. Supervision of masonry construction is difficult. Sliding of the bottom construction joint is a frequent failure initiating mechanism in tests and this very joint is in practice often the worst due to the presence of mortar droppings. Wider cavities ease cleaning and reduce the likelihood of the entire construction joint being covered by droppings. Reinforced hollow masonry at its worst is concrete with dubious construction joints at 16" centres vertically and 8" centres horizontally. The wider the wall the greater the proportion of homogeneous material provided of course all cells are filled and units with open end and recessed webs are used. This construction minimises the discontinuity of internal vertical joints and reduces it essentially to that at the face shelves.

Vertical mortar joints are particularly easily weakened in practice as block layers adjust blocks after laying thus destroying what little bond there may be available in these regions. Again the wider wall is at an advantage here.

REINFORCED MASONRY RESEARCH

The writer's office through the Ministry of Works Central Laboratory has initiated research into a number of aspects in reinforced hollow masonry construction that appear to be particularly undesirable. One is the frequent lack of adhesion of grout to face shells. Present indications are that lack of adhesion is due to the faces of the blocks being drawn inwards towards the grouted core when initial shrinkage takes place. Separation occurs when the shells elastic resistance overcomes the adhesion. It appears that the only satisfactory way of overcoming the problem is by the addition of a suitable expansive agent. Mere change of mix proportions is not satisfactory because of the difficulty involved in grouting relatively small spaces with low slump mixes. The high cost of additives makes it necessary to determine optimum proportions and this work is not yet complete. It should be noted in this context that extensive spalling of block faces occurred at San Fernando.

Another research project involves the investigation of lap slices in grouted cavities. Lapping of bars results in bursting forces that are capable of splitting blocks before the strength of the bars can be developed. The work it is hoped will establish lap lengths for a variety of bar thicknesses, bar diameters and types of masonry units. It is suspected that open end and recessed web units may be particularly vulnerable to splitting unless grout adhesion can be improved. If proved this would be unfortunate in view of the many other desirable properties of these units.

STRUCTURAL STEEL

Structural steel in larger sections is an imported material and Government policy has been to minimise its use. Ministry of Works have therefore limited its application to structures where reinforced concrete was considered to be unsuitable. This was the case of the Wellington Postal Centre where structural steel was used to provide more predictable earthquake resisting performance. The International Airport Terminal buildings in Auckland were designed in steel to give greater flexibility for future development.

The increasing cost of reinforced concrete detailed for ductility and the improved overseas funds position of New Zealand will, however, allow somewhat more liberal use to be made of structural steel in future Government buildings, probably again in the very high multi-storey buildings and the very low light structures.

One of the remaining uncertainties in the performance of structural steel are the joints particularly where large boxed columns are used. Massive welding from one face is required and extensive ultrasonic testing must be employed. Nevertheless there are no full scale test results reported in the English literature that would indicate that locked in stresses in this type of joint are self relieving on yielding without prior significant crack initiation. To overcome this problem the department is currently designing a major structure in spun, flanged steel columns thereby avoiding some of the above-mentioned joint problems. The effect of concurrent beam yielding has, however, not yet been investigated in Japan where the system originates and theoretical investigations and tests will be carried out on this problem in co-operation with Ministry of Works Central Laboratories.

CONCLUSION

Ministry of Works practice in earthquake resistant design since the previous report required no radical changes. The bulk of public buildings 3 to 12 storeys in height are being and will be designed in reinforced concrete. Squat reinforced concrete shear walls are being designed for 100% increased loadings over that of ductile reinforced concrete frames. Detailing of reinforced concrete beam column joints to allow for the latest test results, the concept of concurrent beam yielding and vertical earthquake effects has been further developed and is used in buildings of all heights. All spandrels in ductile coupled shear walls are being diagonally reinforced. Reinforced hollow masonry is being designed for 140% higher earthquake loadings than ductile reinforced concrete frames. Detailing remains unchanged and height restrictions continue. Increasing use of structural steel in forseen for buildings over 12 to 15 storeys in height. It is recognised that parts and portions of buildings have to sustain even higher and more frequent accelerations than was previously thought likely.

REFERENCES

1. Seismic Design Public Buildings, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 2, No. 4, 1969.
2. Seed, Influence of Soil Conditions ASCE, SM1, January 1969, etc.
3. Paulay, P.29, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 5, No. 1, 1972.
4. Skinner, R. I., N.Z. D.S.I.R. Bulletin 166.