COST EFFECTIVENESS OF CODE BASE SHEAR REQUIREMENTS FOR REINFORCED CONCRETE FRAME STRUCTURES

D. G. Elms* and D. Silvester**

ABSTRACT

The appropriateness of the overall base shear levels prescribed by the New Zealand Loadings Code NZS4203:1976 is investigated for reinforced concrete frame buildings. Six-storey structures were designed to different base shear levels and total costs were computed: total cost takes account of capital cost, averaged direct economic loss due to earthquakes, and indirect earthquake losses. Damage levels were obtained from computer time-history analyses. It is shown that the code base shear levels are of the right order of magnitude for reinforced concrete frame buildings, but that the total cost of such buildings is insensitive to design base shear level. The increase in capital cost of a concrete frame building due to earthquake design requirements is of the order of 4%.

INTRODUCTION

Following the introduction of the present New Zealand Loadings Code, NZS 4203:1976, various criticisms were made to the effect that the code was too severe and led to unnecessary cost penalties. The criticisms were widely voiced: at one stage a spokesman for the Prime Minister's Department said that complaints had been made that the standards were excessive and were adding considerably to the cost of houses and other buildings(1).

In fact, the drafters of NZS 4203:1976 had aimed to leave the overall level of the code unchanged from that of the earlier Chapter 8 of NZS 1900, while expressing the ductility requirements of the earlier code in more detailed form and introducing a series of factors designed to adjust the relative level of design of various buildings. Thus, the important matter of the appropriateness of the overall code level had not received serious consideration. However, it is by no means easy to arrive at a rational basis for the optimal level of any code. Many points are involved. Capital cost is one, safety considerations are another. Even if careful calculations of probable economic loss can be made, these do not necessarily give an adequate reflection of the wishes of the general public as to the acceptable amount to be spent on seismic protection, nor can they easily take account of the perceived losses to a community caused by injury and death.

Both for lack of data and for lack of a complete methodology, it is not yet possible to arrive at a truly optimal code level for a community. However, this paper looks at the question in a limited way with regard to NZS 4203:1976. By considering the design of a six-storey reinforced concrete frame structure for various base shear levels, and by deriving likely estimates of damage for different earthquake levels, the appropriateness of the present code level is investigated. Different results would, of course, be expected if the investigation were carried out on different building types and layouts, but the present study can certainly give an overall idea of the correctness of the code. Two other matters are also considered in the paper: the increase in capital cost of a building due to seismic provisions, and the sensitivity of the total cost (including the effects of possible damage) to the code base shear factor.

* Professor of Civil Engineering, University of Canterbury.

** Engineer, Perth, Western Australia.

METHODOLOGY

In order to obtain the optimum code level for a particular building, the building is first designed for various alternative code levels. The total expected cost of each design alternative is then worked out for a given location - say, Wellington. The total expected cost takes into account probable values of both direct and indirect seismic losses due to earthquakes, as well as initial capital costs. The optimum code level is, of course, that leading to the least total expected cost.

Figure 1 shows the general procedure for finding the total expected cost, which is essentially that followed by Whitman et al(2). The total expected cost consists of initial capital costs and probable future losses, discounted to an equivalent capital value.

Once a structural system has been chosen, it is designed for different levels of base shear. In this case, a six-storey reinforced concrete frame structure was chosen. It was designed for wind alone, and for base shear coefficients of 0.05 and 0.10 making three designs in all. It is a straightforward matter to use normal costing procedures to obtain the initial capital costs of these designs. In fact, the total capital cost does not have to be computed as it is only the additional costs incurred by the seismic provisions of the code that are required.

The first stage in obtaining the probable future losses is to subject the chosen designs to time-history computer analyses in order to determine estimates of the damage that will be caused by earthquakes of different intensity levels. This involves first scaling a suitable earthquake record to give a series of desired intensity levels. The maximum displacement responses of the buildings give indications of the degree of damage to be expected. From these results, a damage probability matrix is obtained for each design which relates the probabilities of different degrees of damage to a series of intensity levels. Assuming that for any chosen location a probability distribution can be obtained for the annual occurrence of earthquakes of different intensities, it is then a straightforward matter to calculate the probable cost of physical damage on an annual basis. The consequences of an earthquake are not restricted to physical damage, however. Other types of loss occur such as injury, loss of life and loss of business. We shall call these "incidental losses", though this should not be taken to imply that such losses are of secondary importance. It is extremely difficult to quantify such losses in monetary terms not so much because of a lack of data (though indeed data may be scarce) but rather because of the philosophical difficulties involved in defining loss at this level: for instance, although incidental economic losses incurred by one individual firm may on balance have little economic effect on society as a whole, they may have a very large effect on the firm itself. Whether or not a code should be concerned only with minimising total cost to society as a whole or whether it should also consider losses to individuals is a matter not easily resolved. However, despite these reservations, some attempt must be made to include incidental losses in the computations of probable future losses due to earthquakes.

INITIAL CAPITAL COSTS

The basic geometry of the six-storey reinforced concrete frame chosen for the study is given in Figure 2, which shows half of one frame. It is assumed that this represents one bay of a long building in which the lateral load transverse to the building is resisted by frames. The choice of such a structure was dictated largely because of the need to carry out a time history non-linear analysis, which can only, at present, be done for two-dimensional structures. However, certain other benefits result from such a structure, such as being able to neglect concurrency effects in column design and not having to include torsion in the initial analysis. Three different designs were carried out for different lateral loadings, using the same overall geometry:

Building I - wind load only
Building II - seismic load, $C_d = 0.05$
Building III - seismic load, $C_d = 0.10$

The buildings were designed to NZS 4203:1976 except that arbitrary values of base shear coefficient C_d were chosen for buildings II and III in order to achieve an adequate difference between the designs. It was found that, assuming a built-up site in Wellington for wind load calculations, that the base shear forces required for buildings I and II were virtually identical at 305 kN and 303 kN respectively: however, the assumed static distribution of lateral loads is different, being uniform for the wind load of building I and triangular for the seismic load of building II. The major difference between the two designs is that building II, like building III, was designed according to the capacity design procedure and was detailed for ductility, whereas for building I, excursion of members beyond the elastic range was not expected and there was no detailing for inelastic behaviour. Building I was designed using the normal capacity reduction factors of 0.9 for beams, 0.85 for shear and 0.7 for columns.

Assuming the code coefficients I, M and R are unity and that the structural type factor S is 0.8, buildings II and III have values of C of 0.063 and 0.125 respectively. The fundamental periods of buildings I, II and III were 1.99, 1.61 and 1.04 seconds. Thus building I is markedly more flexible than its seismically designed counterpart, building II.

The basic capital cost of one bay of the structure can be estimated as (in 1976 values) $\$300/m^2$. With an area of 870 m^2, this leads to a rough total cost of \$261,000 per bay. The cost of the slab floors will be about $\$35/m^2$: this cost is unlikely to be affected by seismic considerations. A pad footing foundation system is assumed, costing $\$5/m^2$. Beca and Tork(3) found that the cost of such a foundation would be virtually independent of the design base shear coefficient. The frame cost of building I would be about $\$15/m^2$ leading to a total structural cost (including foundations) of $\$55/m^2$, or roughly 20% of the whole. Taking the profit margin to be 10%, the non-structural costs will be about 70% of the whole. This agrees with Leslie's figure(4) of 68% for non-structural costs. In fact, we only need these overall figures roughly, in order to be able to express cost increases for seismic design as percentages of total capital cost.

The cost increases for seismic design fall into three categories: direct increases of structural costs, the cost to a client of an increase in construction time, and cost increases of nonstructural items due to code requirements for separation and detailing.

Structural cost increases occur only in the columns and beams of the frame. Using 1976 rates to cost the three frame designs, costs of \$12,856, \$15,993 and \$18,827 are obtained. The increases in cost of buildings II and III over building I thus represent about 6% and 11.4% respectively as percentages of the total structural cost \$52,200, of building I; that is, 1.2% and 2.3% of the total cost.

Seismic design requires confinement of concrete at and near beam column joints to ensure ductility. Increased longitudinal beam and column steel is generally needed, leading to congestion of steel and difficulty of placement. Gaps between bars must permit placement and vibration of concrete, and thorough detailing and inspection is required for good performance. The building designed for wind load could have the reinforcement prefabricated and lifted into place, whereas the others, because of reinforcement congestion, need to have the steel placed by hand. This delay of building completion leads to a longer construction period for seismically designed buildings. A reasonable estimate is that buildings II and III require an extra construction time of one week and one and a half weeks per floor respectively, compared with building I. The cost of this to an investor may be thought of conservatively as the interest he loses on his investment over the period. Assuming an interest rate of 10% per annum this effect therefore represents increases of 1.2% and 1.8% respectively in total cost. (The time of construction has, of course, also been taken into account in arriving at the structural costs already given.)

NZS 4203:1976 requires separation of non-structural elements such as stairways, rigid partitions, concrete claddings, glass windows and other brittle interior claddings in situations in which the interstorey deflection is more than 0.0006 of the storey height, with the deformation computations making due allowance for concrete cracking. The ideas behind the separation requirements have been dealt with in a recent article by Glogau(5). Both Glogau and Leslie(4) estimate the cost of providing separation and ductile fasteners for non-structural elements to be of the order of 2% of the total construction cost. As this cost is probably largely independent of the level of seismic design, we shall assume a 2% increase in cost for both buildings II and III.

Table 1 summarises the capital cost increases due to seismic design. It should be noted that the major cost increase is incurred in designing for earthquakes in the first place, and that thereafter the cost is relatively insensitive to increase in the design base shear coefficient. This insensitivity is due largely to the marked effect of non-structural costs. Assuming linearity of cost increase with increase in code level, the totals given in Table 1 lead to the expression

$$P = 2.7 + 27.2\ C \qquad (1)$$

for the percentage increase in total cost when designing for a seismic coefficient C. Thus, for zone C (for long-period buildings), C = 0.05 and the cost increase is 3.8%, while for zone A, the equivalent cost increase is 4.3%. These figures can of course only be taken as giving a rough indication of the order of cost increase to be expected for reinforced concrete frame buildings. The figures are repeated in the first row of Table 2. In general, the figures for long-period buildings would be more appropriate in most cases.

For certain buildings, such as, perhaps, those in the public sector, it might not be correct to include the cost to the owner of an increase in construction time. In such a case, the figures in row 2 of Table 2 are the appropriate ones, which show that the capital cost increase would normally be between three and four percent.

Finally, row 3 of Table 2 gives the percentage figures for structural cost increases alone.

PROBABLE FUTURE LOSSES

The next step is to determine the probable future losses due to earthquakes. As mentioned previously, an earthquake leads to two sorts of loss: economic loss due to direct physical damage, and 'incidental' losses due to injury, loss of life and economic disruption.

First, damage probability matrices had to be found for the three buildings. These were derived by carrying out time-history analyses for earthquakes of different intensities, and estimating damage levels from consideration of the responses obtained. As the three buildings had markedly different fundamental periods, it was necessary to avoid using an earthquake record whose spectrum would give undue bias to some frequencies at the expense of others. For this reason, the Jennings B1 artificial earthquake(6) was chosen, as this has a relatively uniform spectrum for periods between 0.5 and 2.0 seconds. In order to provide ground motions representing various earthquake intensities, the acceleration values of the basic B1 record were scaled as shown in Table 3.

The scaling factors were obtained from consideration of relationships between Modified Mercalli intensity and peak acceleration or velocity given by Gutenberg and Richter(7), Whitman et al(2), and Esteva and Rosenblueth(8).

The damage probability matrices for the three designs are shown in Tables 4, 5 and 6. These show the probabilities of different degrees of damage occurring for a range of earthquake intensities. Damage levels are given in terms of damage ratio - the ratio of damage cost to the total capital cost of the building. The probability figures were obtained from the time-history analysis results by considering column shears, beam curvature ductility demands and storey deformations. If column shear capacities were substantially exceeded then a strong probability of irrepairable damage or total collapse existed. Such a situation only occurred for building I: the analyses showed (Table 4) that there was a significant probability of total economic loss for an earthquake of MM VIII, that there was a 90% chance of complete collapse for MM IX, and that collapse due to column shear failure was certain for MM X. These results were due to the fact that the transverse steel for the wind-designed frame was inadequate to carry the shear loads imposed by moderate to severe earthquakes.

The degree of repairable structural damage involved was estimated from the maximum beam curvature ductility demands, and the damage to secondary elements, services and contents of a building were obtained from maximum values of interstorey deformation. Descriptive damage scales used by Whitman et al(2), and a relationship between damage ratio and peak interstorey displacement derived by Whitman(9) from a survey following the San Fernando earthquake were of great help in deriving the damage probability figures in Tables 4-6.

Clearly, the damage probability matrices for buildings II and III are very similar. However, as we shall see later, the differences are sufficient to provide significant economic differences between the two buildings.

The possibility of beam column joint damage was not taken into account. The joints of buildings II and III may be assumed to have been adequately detailed. However, substantial joint damage might well occur for building I at moderate earthquake levels. Such damage is difficult to repair and its economic consequences are serious, so that the probability figures of Table 4 could well have been more severe.

The final rows in Tables 4-6 give the expected damage ratio D for different earthquake intensities: these figures are obtained by multiplying the damage ratio scale in the left-hand columns by the probability figures and summing.

Estimates of the annual probabilities of significant earthquake motion for Wellington, Christchurch and Auckland are given in Table 7. These cities have been chosen as they are representative of code zones A, B and C. The table has been derived from Smith's work(10). The values for a MM value of X are based on a rough estimate and are not reliable, due to the lack of high intensity data in New Zealand. It has been assumed that in each city, 20% of the area consists of poor soil which will increase the intensity level by one. These figures can now be multiplied by the values of D in Tables 4-6 and summed to give estimates of the annual cost of physical damage as percentages of initial capital cost. They are then multiplied by a discounting factor corresponding to an annual interest rate of 10% to give present value estimates of physical damage costs : these are shown in Table 8.

A less tractable problem is the determination of incidental losses; that is, losses of all types other than the costs of direct physical damage. Two main types are considered here: economic losses, which include loss of rent to the building owner and impact on both building tenants and the local economy due to buildings unserviceability, and the effective losses caused by injury and death.

Treatment of these losses in an optimization process has been a controversial issue in the past. Different approaches have been suggested but no consensus has been reached. The question of the value of human life has been one of the main difficulties. Decision makers hesitate to openly accept the possibility of loss of life as the outcome of an acceptable decision. On the other hand it is argued that people accept the risk of death in their daily activities (such as driving) and that therefore a low probability of building failure should be admissible.

It is generally recognized that the public's set of values (public utility) is highly non-linear. A large catastrophic loss is considered far worse than the same loss spread over many events. A normal cost-benefit analysis can be objected to on the grounds that it does not take this non-linearity into account.

The method of analysis adopted in obtaining the results reported in this paper was to calculate a basic cost for each of the two main areas of incidental loss. These costs were then multiplied by severity factors which were intended to reflect public utility values and allow for their nonlinearity. Severity factor values were determined by the type of loss and its magnitude.

The calculations leading to estimates of incidental losses are rather lengthy and will not be reproduced here: they may be found elsewhere(11). The sources of information on which they were based are various. Estimates of the economic loss attendent on an earthquake were obtained from Munroe and Blair(12). Figures for the cost of injury given by Whitman(2) were used, while the basis of an estimation of the cost of loss of life was derived from Melinek(13) and Starr(14). Severity factors were estimated subjectively: this aspect of the work requires further investigation. The results of the incidental loss calculations are given in Table 9 as percentages of the initial capital cost of a building. It will be seen that the values obtained for building I are very high: this is a consequence of the high perceived value of total collapse, both in terms of economic loss to the community and in terms of injury and loss of life. The values obtained for building I in Wellington and Christchurch are in fact somewhat irrelevant as no building in these areas would be designed for wind alone. The figure for Auckland shows that in that city, the economic consequences of designing a building without detailing it for ductile behaviour are likely to be economically serious, though probably not catastrophic.

TOTAL EXPECTED COST

Tables 8 and 9 are combined with the capital cost increases of Table 1 to give total expected cost increases expressed as percentages of the initial cost of building I. These are shown numerically in Table 10 and graphically in Figure 3.

Figure 3 has no scale on the abscissa, for it is only intended as a graphic illustration of the numerical results. It can be seen that the total expected cost varies very little between buildings II and III for all three locations: the costs are remarkably insensitive to design base shear level, once the building has been adequately detailed for earthquake resistance. Building II, designed for a base shear level of 0.05g, shows a somewhat lower cost increase in all cases. This building has in effect been designed for code Zone B, while building III has a base shear 1.4 times greater than the Zone A requirement. The pattern is very little affected by a change in the interest rate used. It would appear from the results that the overall level of NZS 4203:1976 is of the right order. In the light of the uncertainty attending some of the assumptions on which the analysis is based, more than this cannot be said. Neither would it seem justified, in the light of the insensitivity of the cost figures to base shear level, to refine the analysis further, at least for reinforced concrete frame structures. It might well be, of course, that the total expected cost for some other structural type would be more sensitive to code level. However, this would seem to be unlikely.

What is clear from the results is that in no location is it cheaper to neglect aseismic design and detailing. Even in Auckland, where expected damage costs are far less than in other centres, the results unquestionably show the necessity for adequate detailing of a structure.

CONCLUSIONS

The following conclusions may be drawn. They apply to reinforced concrete frame buildings, and it should be particularly noted that they do not necessarily apply to other structural types.

1. The increase in the capital cost of an aseismically designed frame building compared with a similar building with no earthquake design requirements is of the order of 4%.

2. The increase in the capital cost of a code-designed reinforced concrete frame building in moving from Zone C to Zone A would seldom be greater than 2% and would generally be less than 1%.

3. The overall base shear levels required by NZS 4203:1976 appear to be of the right order of magnitude.

4. For a given location, the total expected cost of a building, which includes allowances for possible structural damage, economic loss, injury and loss of life, is markedly insensitive to the level of design base shear used.

5. It is imperative that all reinforced concrete frame buildings in New Zealand, even in Zone C, should be adequately detailed for ductility. Column detailing to avoid shear failure is especially important.

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TABLE 1

COST INCREASES AS A PERCENTAGE INCREASE ON THE TOTAL INITIAL CAPITAL COST OF BUILDING I

	Building II $C_d = 0.05$	Building III $C_d = 0.10$
Structural cost	1.2	2.3
Increase in construction time	1.2	1.8
Non-structural costs	2.0	2.0
Total cost increases	4.4%	6.1%

TABLE 2

COST INCREASES AS A PERCENTAGE INCREASE ON THE TOTAL INITIAL CAPITAL COST OF A BUILDING WITH NO EARTHQUAKE DESIGN REQUIREMENTS, FOR DIFFERENT C VALUES GIVEN BY NZS 4203:1976

	Long Period		Short Period	
	Zone C	Zone A	Zone C	Zone A
1. Total cost increases	3.8	4.3	4.9	5.9
2. Structural and non-structural costs	2.8	3.1	3.5	4.2
3. Structural cost alone	0.8	1.1	1.5	2.2

TABLE 3

SCALING FACTORS AND EQUIVALENT INTENSITIES FOR THE B1 RECORD

MM intensity	VI	VII	VIII	IX	X
Scaling factor	0.09	0.23	0.51	0.90	1.80

TABLE 4

DAMAGE PROBABILITY MATRIX - BUILDING I

Damage Ratio (%)	MM Intensity				
	VI	VII	VIII	IX	X
0.1	0.7				
0.5	0.3				
2		0.6			
5		0.4			
10			0.8		
30			0.1		
100*			0.1	0.1	
100^+				0.9	1.0
D	0.22	3.2	21.0	100	100

* - building standing but condemned

\+ - total collapse of building

TABLE 5

DAMAGE PROBABILITY MATRIX - BUILDING II

Damage Ratio (%)	MM Intensity				
	VI	VII	VIII	IX	X
0	1.0				
0.05		0.5			
0.5		0.5			
2.5			0.5		
6			0.5	1.0	
25					1.0
D	0	0.275	4.25	6.0	25.0

TABLE 6

DAMAGE PROBABILITY MATRIX - BUILDING III

Damage Ratio (%)	MM Intensity				
	VI	VII	VIII	IX	X
0	1.0				
0.05		0.5			
0.5		0.5			
2.5			1.0	0.5	
6				0.5	
25					1.0
D	0	0.275	2.5	4.25	25.0

TABLE 7

ANNUAL PROBABILITIES OF SIGNIFICANT EARTHQUAKE MOTION

	Modified Mercalli Intensity				
	VI	VII	VIII	IX	X
Wellington	0.233	0.073	0.026	0.0093	0.0013
Christchurch	0.065	0.026	0.012	0.0052	0.0008
Auckland	0.014	0.0047	0.0016	0.00022	0

TABLE 8

PHYSICAL DAMAGE COSTS

	Building					
	I		II		III	
	Annual Cost	Present value*	Annual Cost	Present value	Annual Cost	Present value
Wellington	1.89	19.8	0.22	2.3	0.16	1.68
Christchurch	0.95	9.97	0.11	1.15	0.079	0.83
Auckland	0.074	0.776	0.0094	0.099	0.0062	0.065

* Present value of the series of annual costs discounted at 10%

TABLE 9

INCIDENTAL LOSSES (ECONOMIC, INJURY AND LOSS OF LIFE)

	Building					
	I		II		III	
	Annual Cost	Present value*	Annual Cost	Present value	Annual Cost	Present value
Wellington	1145	12,000	0.0959	1.01	0.0654	0.69
Christchurch	517	5,420	0.0481	0.505	0.0332	0.348
Auckland	1.77	18.6	0.0026	0.027	0.00103	0.011

* Present value of series of annual costs discounted at 10%

TABLE 10

TOTAL EXPECTED COST INCREASES AS PERCENTAGES OF CAPITAL COST OF BUILDING I (FIGURES IN BRACKETS CORRESPOND TO 6% RATE OF INTEREST)

	Building		
	I	II	III
Wellington	12,020 (19,690)	7.71 (9.82)	8.47 (9.93)
Christchurch	5,430 (8,880)	6.05 (7.10)	7.28 (8.03)
Auckland	19.4 (31.6)	4.53 (4.60)	6.18 (6.22)

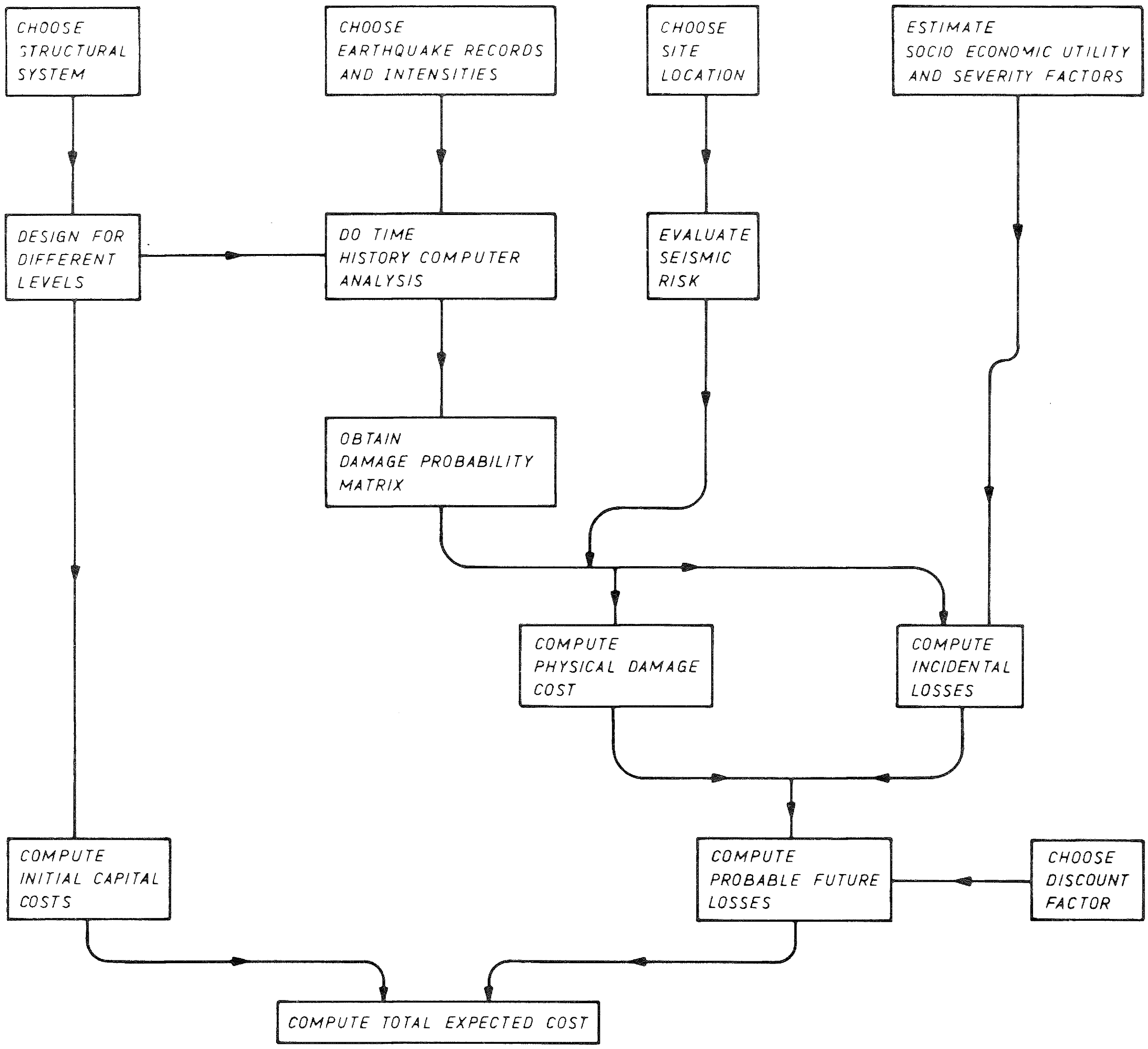

FIGURE 1: METHODOLOGY OF INVESTIGATION

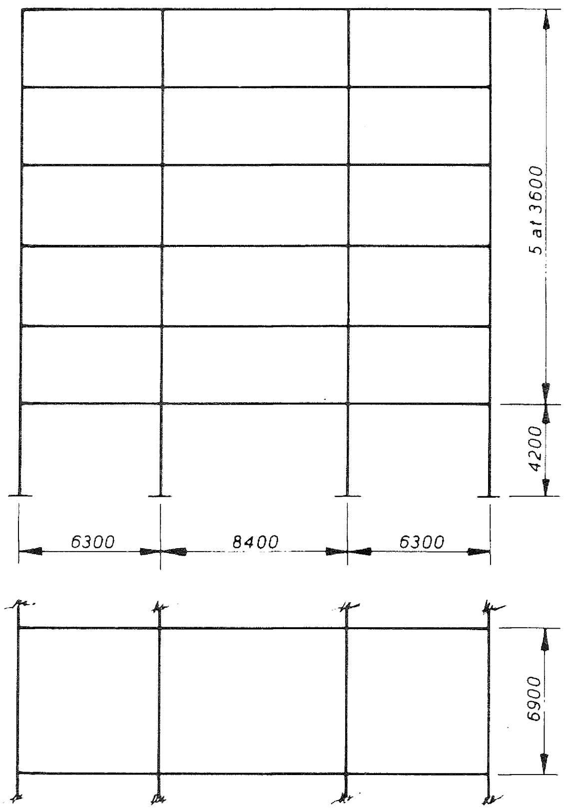

FIGURE 2: OVERALL FRAME DIMENSIONS

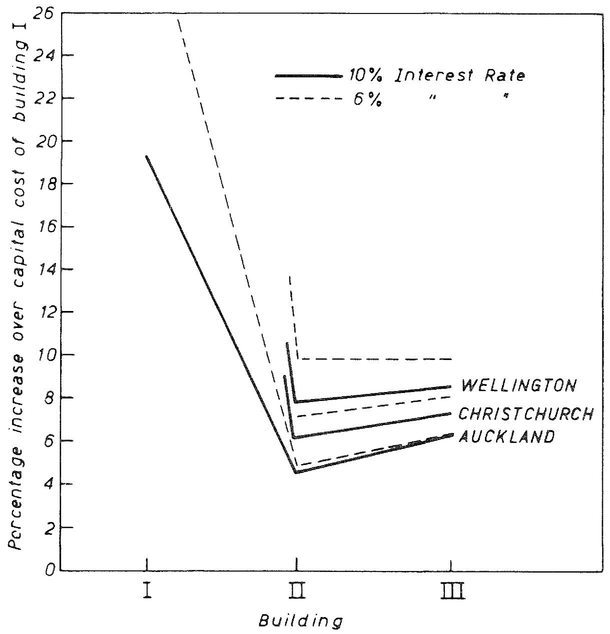

FIGURE 3: TOTAL EXPECTED SEISMIC COST INCREASES