A SIMPLIFIED EARTHQUAKE RESISTANT DESIGN METHOD FOR BASE ISOLATED MULTISTOREY STRUCTURES

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ABSTRACT

This paper describes the step-by-step design procedure of a simplified approach proposed for use in practical design of base isolated multi-storey structures. The proposed method can be used to accurately estimate the inertia forces, not only at the level of the isolation devices but throughout the height of the structure.

1. INTRODUCTION

As has been reported earlier in an accompanying paper [1] there is an increasing recognition of the Base Isolation (BI) technique and its benefits during the last two decades. Many practical BI systems have been implemented in various structures in countries which have high seismic risk. However, there is still no simple and reliable design procedure which is able to give the designer a clear insight into seismic behaviour of Base Isolated multistorey structures. Current practice still relies upon a series of deterministic dynamic inelastic time history analyses which seems to be the main reason why many structural engineers are still reluctant in apply this technique.

A study [2] was carried out to investigate in more detail the effects of various structural parameters and ground motion characteristics on the response of BI multistorey structures. It also reviewed the shortcomings of the current design methods. The results were then used to develop two simplified analysis methods for practical design.

The first method which is called the Code-Type approach can be used to accurately estimate the inertia forces, not only at the level of the BI devices but throughout the entire height of the multistorey structure. It is proposed for use as a preliminary design tool or even a final design tool for simple BI structures. The second procedure, which is based on the Component Mode Synthesis method [3,4], is suggested for final design purposes of more complex BI multistorey structures. This latter method enables the designer to evaluate the effects of the isolation devices on the contribution of each mode of vibration to the total response of the structure irrespective of the inelastic behaviour of the BI system.

In this paper, only the first method will be presented. Its step-by-step design procedure will be discussed and accompanied by an illustrative design example.

More detailed explanation regarding this latter method can be found elsewhere [2]. Prior to presenting the above simplified method it is felt necessary to discuss why simple design procedures are needed.

2. WHY SIMPLE METHODS ARE NEEDED

In this era of modern computers one might argue against the necessity of developing simple analysis methods for practical design purposes of BI multistorey structures. However, there are at least three main reasons why simple seismic analysis methods are, in fact, very desirable.

First, it is impractical to use mathematically precise analyses, such as deterministic inelastic time history analyses, at a preliminary design stage where the complete physical properties of the structure are still to be determined. It has been realized that the results of such analyses are highly dependent on the assumptions made in the formulation of the mathematical model. Unless a large number of analyses are carried out during the refined design process the results are generally no better than, and may in fact be inferior to, what could be achieved at

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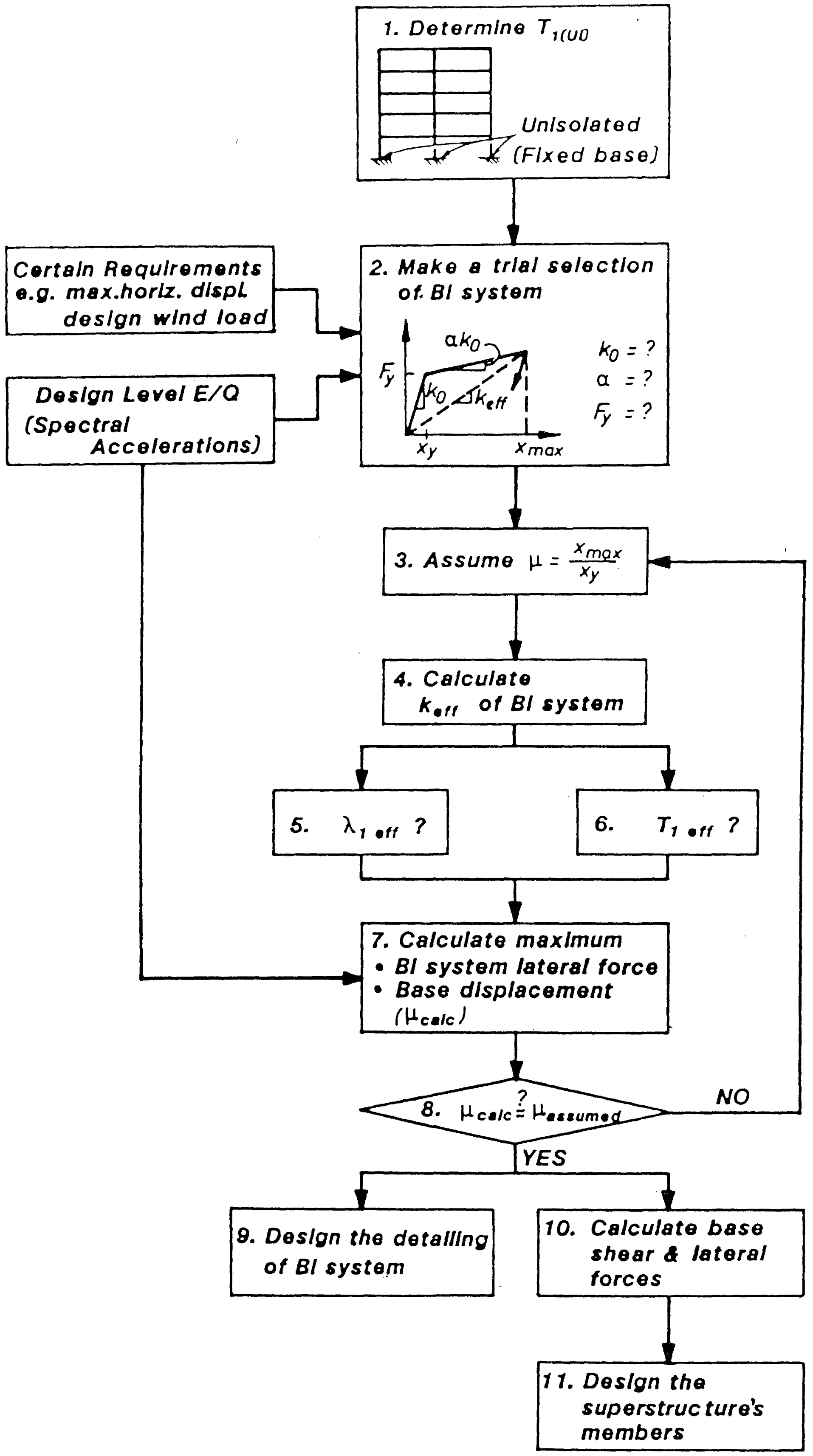


FIGURE 1: STEP-BY-STEP DESIGN PROCEDURE CODE-TYPE APPROACH FOR BI MULTISTOREY STRUCTURE

far lower cost by simpler, though less precise, approaches [5]. In design practice only a limited number of inelastic time history analyses are normally conducted because of cost and time limitations.

Second, simple design procedures enable the structural response to be readily estimated and visualised without the need of elaborate calculations. Simple approaches usually give the designer a clearer insight or a better feel towards the effects of varying the design parameters on the overall structural behaviour. This encourages the exercise of "engineering judgement" which is essential to a successful design [5].

Third, in order to promote the use of a new technique, such as Base Isolation, a reasonably simple yet reliable approach is required. Practitioners need to be ensured that is is possible to design a BI multistorey structure using a simple and familiar approach. It is also hoped that due to its simplicity and reliability the approach will eventually become incorporated in the general design code which in turn will enhance the confidence of structural engineers in adopting Base Isolation techniques.

3. THE PROPOSED CODE-TYPE DESIGN APPROACH

The proposed Code-Type approach is developed by adapting the well-known equivalent static lateral force analysis procedure to suit the seismic behaviour of BI multistorey structures. It is hoped that this similarity will help the designer to become familiar with this proposed approach. A flow chart shown in Figure 1 illustrates the step-by-step procedure of this simple design method.

STEP 1: Determine the fundamental period of the unisolated superstructure ($T_{1(UI)}$).

This first step can be carried out as usual by assuming that the superstructure is mounted on a rigid foundation. At a preliminary design stage approximate formulas as recommended by some codes [6,7,8,9] can be used to estimate the fundamental period of this fixed-base superstructure.

STEP 2: Make a trial selection of the BI system.

The required reduction of lateral inertia forces is normally the main consideration for selecting or predicting the idealized bilinear hysteresis loop parameters of a BI system, i.e. its initial stiffness, k_o, its post-yield stiffness, αk_o, and its yield strength, F_y. Other requirements such as the maximum allowable horizontal displacements at working loads (due to wind and small earthquakes) and ultimate load levels (stability of the BI system) should also be considered.

For this purpose a designer must know the design-level seismic load specified by the code for the particular site where the structure will be built, as well as the essential characteristics of a desirable BI system as discussed earlier [1].

STEP 3: Assume the maximum base displacement under the design-level earthquake and calculate the so-called maximum displacement ductility ratio, μ_{assumed}.

STEP 4: Obtain the effective stiffness of the BI system at the maximum base displacement by using Eq.1 or from the chart shown in Figure 2.

$$k_{\text{eff}} = k_o\left(\frac{1-\alpha}{\mu} + \alpha\right) \tag{1}$$

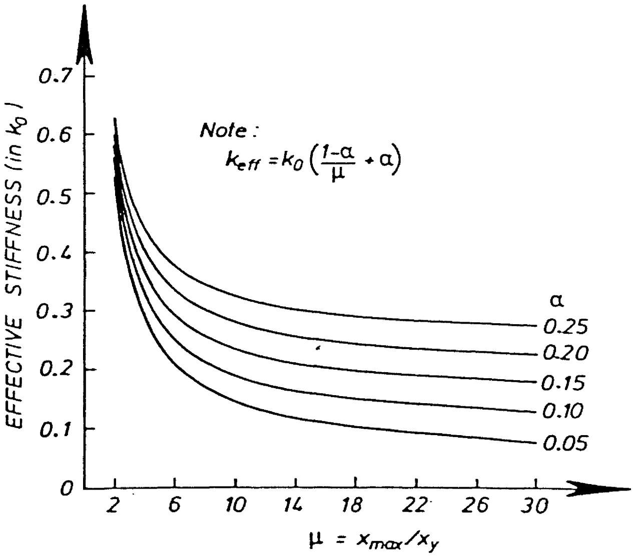


FIGURE 2: EFFECTIVE STIFFNESS OF BI SYSTEMS WITH BILINEAR HYSTERESIS LOOP MODEL

STEP 5: Determine the increase in damping due to the hysteretic behaviour of the BI system using Eq.2 or the chart shown in Figure 3. Then calculate the effective damping of the structure as the sum of the inherent damping of the structure and this additional hysteretic damping.

$$\lambda_{\text{add.}} = E_h = \frac{2}{\pi}R \tag{2a}$$

$$R = (1-\alpha)\left(\frac{\mu-1}{\mu^2}\right)\frac{k_o}{k_{\text{eff}}} \tag{2b}$$

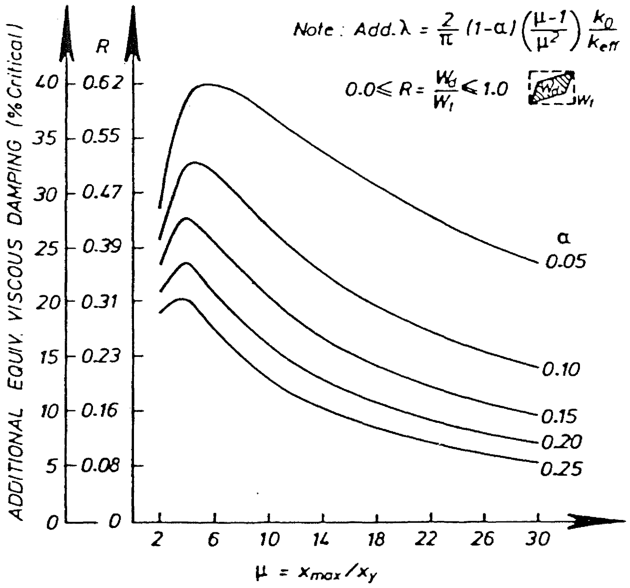


FIGURE 3: **ADDITIONAL DAMPING OF BI SYSTEMS WITH BILINEAR HYSTERESIS LOOP MODEL**

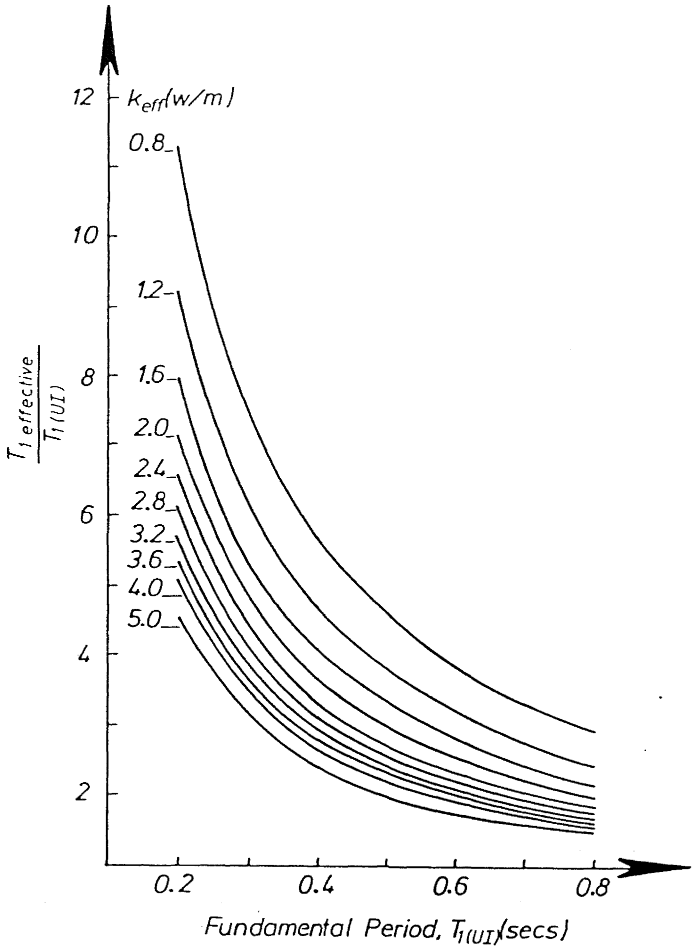


FIGURE 4: **EFFECTIVE FUNDAMENTAL PERIOD OF UNIFORM BI MULTISTOREY STRUCTURES**

In this study R is called the hysteresis loop ratio, i.e. the ratio of the hysteresis loop area to the area of the circumscribing rectangle. This value will be used further in Step 10.

STEP 6: Determine the effective fundamental period of the BI multistorey structure from the chart shown in Figure 4. Note that this chart is developed for BI multistorey structures with uniform floor mass and storey stiffness. Charts for other variations of floor mass and storey stiffness may be developed later. In the absence of such charts a proper modal analysis should be conducted to calculate the effective fundamental period of the BI multistorey structure.

STEP 7: Based on the effective fundamental period and effective damping of the structure determine the maximum BI system shear force from the appropriate elastic acceleration spectra specified by the loadings code (see examples in Appendix A). Then calculate the maximum base displacement and the maximum displacement ductility ratio, μcalc.

STEP 8: Compare the calculated maximum displacement ductility ratio, μ_{calc} with with maximum displacement ductility ratio assumed in step 2, $\mu_{assumed}$.

If the difference between these two values are relatively great, say above 5% or so, Steps 3 to 8 should be repeated. The calculated maximum displacement ductility ratio may be used as a new assumed value until the two values converge. It was found in the above mentioned study [2] that the convergence in this trial and error process is normally achieved very rapidly.

STEP 9: Detailed design of the BI system.

Some manuals and/or experimental test results of BI devices (10,11,12,13,14,15) can be used as a guidance in the detail design of the selected BI system. It is beyond the scope of this paper to discuss this step further.

STEP 10: Determine the base shear (first storey shear) and the lateral inertia force distribution over the entire height of the multistorey structure.

It is worth noting that the shear force of BI systems with thin hysteresis loops (low R) is usually 10% to 25% greater than the base shear of the superstructure. As the hysteresis loop becomes fatter, either due to the increase of the initial stiffness and the yield strength or to the decrease of the post-yield stiffness, the difference between the BI system shear force and the base shear decreases. The base shear may even exceed the BI system shear force [2]. For this reason, it is suggested that any estimation of the maximum base shear, V, which is based on the maximum BI system shear force, should be multiplied by a factor shown in Figure 5 in order to give a reasonably conservative estimate for the storey shears.

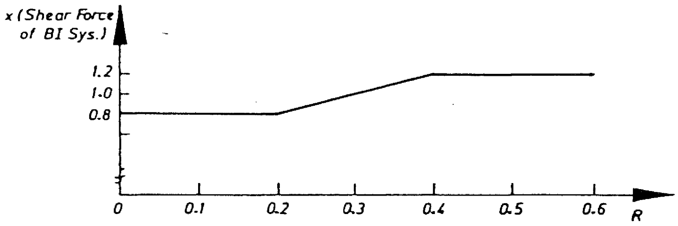


FIGURE 5: MODIFICATION FACTOR USED FOR PREDICTING BASE SHEAR FROM BI SYSTEM'S SHEAR FORCE

As mentioned earlier in Ref.1, that the equivalent static lateral force, F_i at floor i can be accurately predicted by the following formula :

$$F_i = V\frac{W_i h_i^p}{\Sigma W_i h_i^p} \qquad (3)$$

where V is the base shear, W_i and h_i are the weight and height of floor i respectively. The exponent p can be determined from the strong linear correlation with the BI system's hysteresis loop ratio [1]. For the NZ design-level earthquake in seismic zone A [7] the above correlations shown in Table 1 or Figure 6 were found [2].

STEP 11: Design the superstructure's members.

Once the lateral forces are determined the member forces can be computed and the members of the superstructure can be designed in more detail.

As in the use of the equivalent static lateral force procedure for non-isolated buildings, this Code-Type approach would, in general, be adequate for BI multistorey structures which have a uniform mass and stiffness configuration in all storeys or floors. The results of the investigation [2] show that this simple approach can reliably predict the response of short to medium-rise BI structures ($T_{1(UI)} \leq 0.8$ secs) with floor masses for lateral storey stiffness which do not differ by more than, say 25% in adjacent floors.

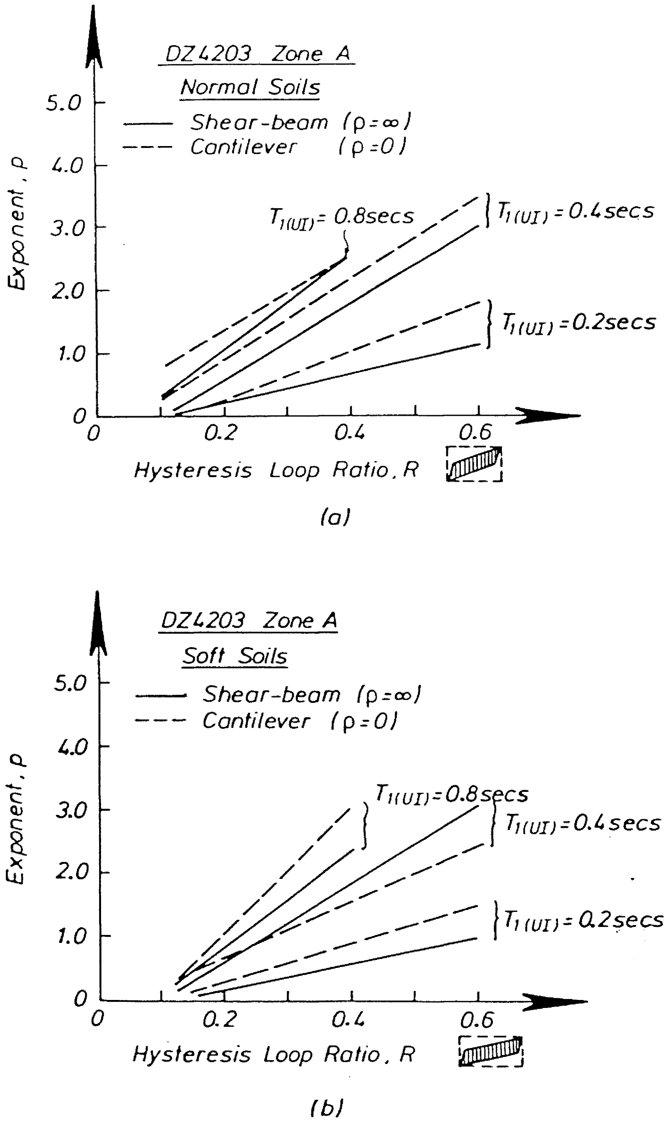


FIGURE 6: RELATIONSHIPS BETWEEN R AND P FOR DIFFERENT $T_{1(UI)}$ UNDER NZ DESIGN-LEVEL EARTHQUAKES FOR ZONE A

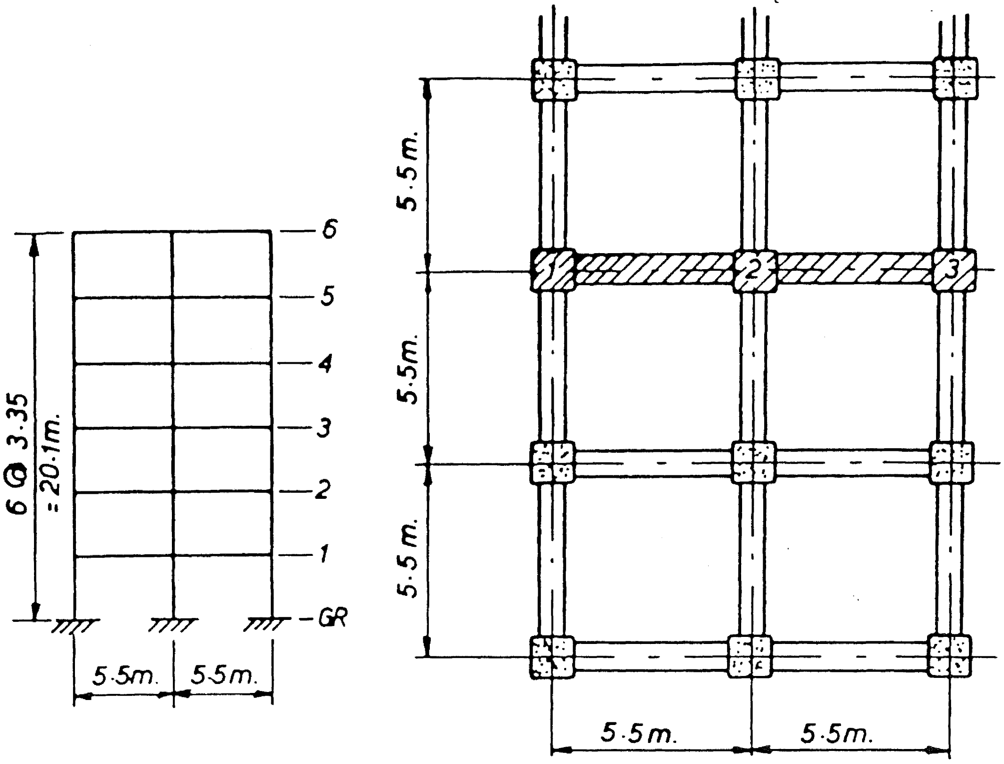
4. A DESIGN EXAMPLE USING THE CODE-TYPE APPROACH

To illustrate the step-by-step procedure of the proposed Code-Type approach a design example is presented in this section. The superstructure is a six-storey reinforced concrete moment-resistant frame shown in Figure 7. The dimensions of the frame structure are listed in Table 2 [16]. To suit it to the implementation of a BI system a stiff horizontal diaphragm is added across the columns at the base of the superstructure. It was assumed that the frames would be required to resist the component of earthquake motion in the plan of the frame only. The component in the perpendicular direction was assumed to be taken by some other resisting systems, for example shear walls. No torsional effects for the building as a whole were taken into account.

Table 1 : Correlations Between the Hysteresis Loop Ratio, R and the Exponent p for NZ Design-Level Earthquake in Zone A

Soil Condition	$T_{1(UI)}$ seconds	ρ	Linear Regression		
			A	B	r
Normal	0.2	0.0	-0.55	3.88	0.81
		∞	-0.31	2.57	0.66
	0.4	0.0	-0.40	6.45	0.83
		∞	-0.66	6.12	0.83
	0.8	0.0	0.16	5.90	0.79
		∞	-0.45	7.41	0.88
Soft	0.2	0.0	-0.32	3.05	0.77
		∞	-0.26	2.14	0.74
	0.4	0.0	-0.18	4.34	0.87
		∞	-0.64	6.20	0.89
	0.8	0.0	0.10	7.28	0.78
		∞	0.20	5.18	0.80

Note : Exponent $p = A + BR$
R = Hysteresis Loop Ratio
r = correlation coefficient
(~ 1.00 implies a perfectly linear correlation)
ρ = beam-to-column stiffness ratio [1,2]
ρ = 0.0 denotes a "cantilever-beam" structure
while
∞ denotes a "shear-beam" structure



ELEVATION

TYPICAL FLOOR PLAN
(not to scale)

FIGURE 7: BUILDING DIMENSIONS FOR THE SIX-STOREY SUPERSTRUCTURE (AFTER JURY [16])

Table 2 : Member Dimensions for the Six-Storey Superstructure (after Jury 16)

	FLOOR	
	1 - 3	4 - 6
Main beams (mm)	600 x 350	550 x 350
Columns 1 & 3 (mm)	500 x 450	450 x 450
Column 2 (mm)	550 x 550	500 x 500

The floor masses were derived using the New Zealand Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1976 [7]. Including the base mass, the total weight of the structure, W is 3322 kN. The building is assumed to be located on a normal soil in NZ seismic Zone A[7]. The inherent level of viscous damping possessed by the structure was assumed to be 5%.

STEP 1: The fundamental period of the non-isolated frame superstructure, $T_{1(UI)}$ is 0.8 secs [16].

STEP 2: Suppose it is desired to have a BI system with an initial stiffness, k_o = 10.0 W/m (33220 kN/m) and a yield strength, F_y = 0.05 W(166.1 kN). Lead-Rubber Bearings (LRB) are considered to be the BI systems in this first trial. The ratio between the post-yield stiffness and the initial stiffness of these LRB is approximately 0.15 [12].

STEP 3: Assume $\mu = 14.0$ (x_{max} = 80.0mm; $x_y = F_y/k_o$ = 5mm).

STEP 4: Using Eq.1 or the chart shown in Figure 2, the effective stiffness of the BI system at the maximum base displacement can be determined.

$$k_{eff} = k_o\left(\frac{1-\alpha}{\mu}+\alpha\right)$$
$$= 10.0\left(\frac{1-0.15}{14.0} + 0.15\right)$$
$$= 2.11 \text{ W/m (7009.4 kNm)}$$

STEP 5: Using Eq.2 or the chart shown in Figure 3 the hysteresis loop ratio, R and the additional damping due to the hysteretic behaviour of the BI system can be found as follows :

$$R = (1-\alpha)\left(\frac{\mu-1}{\mu^2}\right)\frac{k_o}{k_{eff}}$$
$$= (1-0.15)\left(\frac{14.0-1.0}{14.0^2}\right)\frac{10.0}{2.11}$$
$$= 0.27$$
$$\lambda_{add.} = E_h = \frac{2}{\pi}R$$

STEP 6: Thus, k_{eff} = 2.11 W/m and $T_{1(UI)}$ = 0.8 secs, the effective fundamental period of the BI structure can be estimated from the chart shown in Figure 4.

$$\frac{T_{1(eff)}}{T_{1(UI)}} = 1.9$$

T_{1eff} = 1.9 x 0.8 = 1.52 secs.

(from a more rigorous modal analysis: T_{1eff} = 1.47 secs.)

Note that the above superstructure does not have perfectly uniform properties in all storeys as assumed in the chart shown in Figure 4.

STEP 7: Based on the NZ design-level earthquake for normal soils in seismic zone A the elastic sprectral acceleration, S_a is 0.180 g for $T_{1(eff)}$ = 1.52 secs and λ_{eff} = 22% critical damping.(See Appendix A for the values of NZ spectral accelerations).

Thus, the maximum BI system shear force = 0.18 W or 598 kN. From the bilinear force displacement relationship it can be found that the maximum base displacement, x_{max} = 9.17 mm or μ_{calc} = 91.7/5 = 18.3.

STEP 8: μ_{cal} (= 18.3) > $\mu_{assumed}$ (=14.0);

i.e. a 30.7% difference.

Steps 3 to 8, therefore, should be repeated using μ_{calc} as the new assumed value until the convergence is achieved. After repeating the procedure for the third time with $\mu_{assumed}$ = 17.2 it is found that :

K_{eff} = 1.99W/m (6610.8 kN/m)

T_{1eff} = 1.60 secs (c.f. from modal analysis: 1.57 secs)

λ_{1eff} = 14.8% + 5% = 19.8% critical damping

R = 0.23

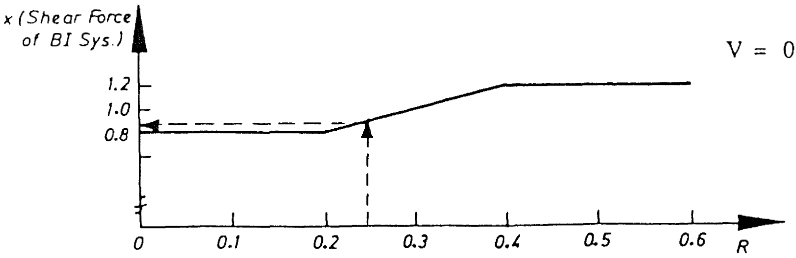
Maximum BI system shear force = 569.4 kN

Maximum base displacement = 85.9 mm

μ_{calc} = 85.9/5 = 17.18 ~ $\mu_{assumed}$ (= 17.2)

STEP 9: See Ref.12 for further guidance to select or design Lead Rubber Bearings in detail. Note, this step could be omitted if bearings with appropriate dimensions have already been selected in Step 2.

STEP 10: First the maximum base shear, V should be determined from the maximum BI system shear force using the modification factor from Figure 5 corresponding to R = 0.23.



V = 0.86 x 569.4 = 489.7 kN

The exponent p used in Eq.3 can be found from the linear correlations between R and p as listed in Table 1 or Figure 6a as follows :

For $T_{1(UI)}$ = 0.8 secs;
$\rho = \infty$, $p = -0.45 + 7.41\ R$
$\rho = 0.0$, $p = 0.16 + 5.90\ R$

As R = 0.23 the values of p are 1.25 and 1.52 for $\rho = \infty$ and 0.0, respectively.

The superstructure's beam-to-column stiffness ratio, ρ is about 0.70. Thus the value of p may be estimated in between 1.25 and 1.52, say 1.40.

If a smaller base shear is prefered a different type of BI system may be used instead of the chosen lead-rubber bearings. In the second trial a BI system with $\alpha = 0.05$ is considered. The initial stiffness and the yield strength remain the same, i.e. $k_o = 10.0$ W/m and $F_y = 0.05$ W. The same procedure is carried out. It coverges at $\mu = 16.7$.

k_{eff} = 1.07 W/m (3554.5 kN/m)

T_{1eff} = 2.08 secs (c.f. from modal analysis: 2.04 secs)

λ_{eff} = 32.0% + 5% = 37.0% critical damping

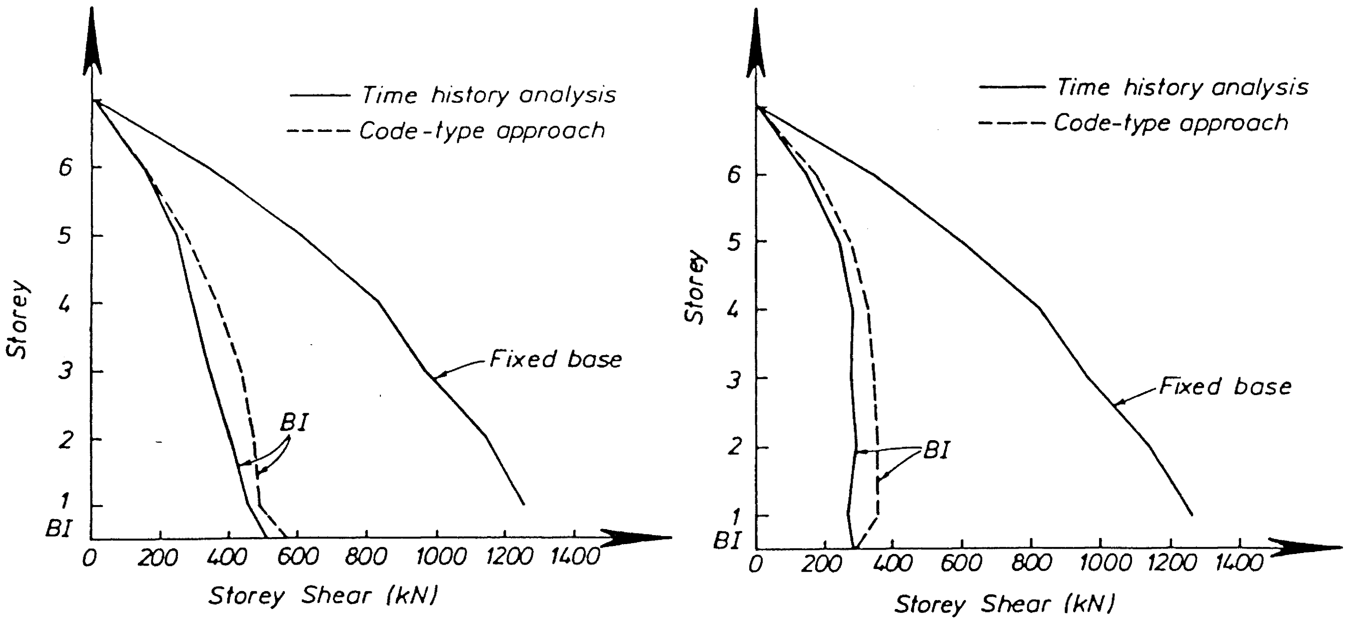
R = 0.50

The spectral acceleration, Sa = 0.0982 g
The maximum BI system shear force= 0.0892 x 332 = 296.3 kN
The maximum base shear = 1.2 x 296.3 = 355.6 kN

Storey i	h_i (m)	W_i (kN)	W_ih_iP (10^3kNm)	F_i (kN)	Storey Shear (kN)
6	20.10	440	.18.73	146.2	146.2
5	16.75	469	15.89	124.0	270.2
4	13.40	469	12.02	93.8	364.0
3	10.05	483	8.64	67.4	431.4
2	6.70	487	5.25	41.0	472.4
1	3.35	487	2.21	17.2	489.7
GR					(569.4)
		Σ	62.74		

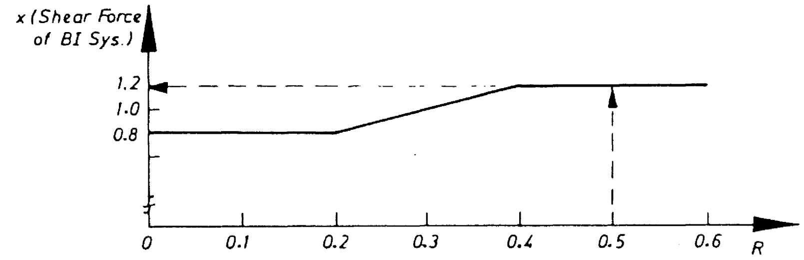
Note : ( ) denotes the BI system shear force.

Figure 8.a shows that the result of this approach shown above are in good agreement with the result obtained from the time history analysis conducted for comparison purposes. It can also be seen from this figure that the storey shears of the non-isolated fixed-base elastic structure are significantly reduced due to the inclusion of the BI system, i.e. by factors 2.7 and 2.3 at the first and top-storey respectively.



(a) BI System : $k_0 = 10.0W/m$, $\alpha = 0.15$, $F_y = 5\%W$ (b) BI System : $k_0 = 10.0W/m$, $\alpha = 0.05$, $F_y = 5\%W$

FIGURE 8: THE PREDICTED LATERAL STOREY SHEAR ENVELOPES BY THE CODE-TYPE APPROACH AND THE TIME HISTORY ANALYSES



For $T_{1(UI)} = 0.8$ secs; $\rho = \infty$, $p = -0.45 + 7.41 \times 0.50 = 3.25$

$\rho = 0.0$, $p = 0.16 + 5.90 \times 0.50 = 3.11$

$\rho \approx 0.7$, $p \approx 3.18$

Storey i	h_i (m)	W_i (kN)	$W_i h_i P$ (10^3kNm)	F_i (kN)	Storey Shear (kN)
6	20.10	440	6132.2	173.5	173.5
5	16.75	469	3660.5	103.6	277.1
4	13.40	469	1800.4	50.9	328.0
3	10.05	483	742.7	21.0	349.0
2	6.70	487	206.3	5.8	354.8
1	3.35	487	22.7	0.6	355.6
GR					(296.3)
		Σ	12564.8		

Note : ( ) denotes the BI system shear force.

Figure 8b shows that the result of the Code-Type approach shown above are in good agreement with the result obtained from the time history analysis conducted for comparison purposes. The top-storey and base shears of the non-isolated fixed-base elastic structure are now reduced by factors of 2.2 and 4.6, respectively.

5. SUMMARY

The step-by-step design procedure of the proposed Code-Type approach for use in practical design of BI multistorey structure has been presented. An illustrative design example was also given. In this example the predicted storey shears were shown to be in good agreement with the results obtained from the more rigorous inelastic time history analyses. The study also verified that this simple approach can be reliably used to predict the response of short-to medium-rise BI structures.

6. ACKNOWLEDGEMENTS

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APPENDIX

GENERATION OF ARTIFICIAL EARTHQUAKES

A series of artificial earthquake records were generated for research purposes in the University of Canterbury using the computer program SIMQKE [1] to match the normal and soft ground elastic spectral accelerations given in the draft of NZ Loadings Code (DZ4203:1986). These records may be used for any site in New Zealand or for any return period simply by scaling the records with appropriate zone and return period factors.

Typical input information to the computer program SIMQKE used to obtain the artificial records is given in the following :

(a) NORMAL SOILS: (after MacRae [2])

Smallest period in range of frequencies contributing to simulation = 0.2 secs

Largest period in range of frequencies contributing to simulation = 4.0 secs

Trapezoidal intensity function used
- Build up time = 2.0 secs
- Level time = 15.0 secs

Duration of generated accelerogram
= 20.0 secs

Discretization interval for generated accelerogram = 0.02secs

Desired maximum ground acceleration = 0.4 g

Seed number provided for selection of random phase angles = arbitrary

Number of iteration cycles of matching to target spectrum = 3

Number of points used to describe the target spectrum = 18

Percentage of critical damping appropriate to target spectrum = 5%

(b) SOFT SOILS: (after Whittaker et al [3]

Smallest period in range of frequencies contributing to simulation = 0.2 secs

Largest period in range of frequencies contributing to simulation = 8.0 secs

Trapezoidal intensity function used
- Build up time = 1.0 secs
- Level time = 15.0 secs

Duration of generated accelerogram
= 20.0 secs

Discretization interval for generated accelerogram = 0.02secs

Desired maximum ground acceleration = 0.4 g

Seed number provided for selection of random phase angles = arbitrary

Number of iteration cycles of matching to target spectrum = 3

Number of points used to describe the target spectrum = 25

Percentage of critical damping appropriate to target spectrum = 5%

Figures A-1 and A-2 show two generated acceleration records used in some of the time history analyses reported in Chapter 6. The acceleration response spectra of these two accelerograms and their target spectrum for SDOF oscillators with 5% of critical damping are shown in Figures A-3 and A-4.

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1 GASPARUNI, D. and VANMARCKE, E.H., Simulated Earthquake Motions Compatible with Prescribed Response Spectra, MIT, Dept. of Civil Eng., Research Report R76-4, 1976.

2 MacRAE, G.A., The Seismic Response of Steel Frames, PhD Thesis, Dept. of Civil Eng. University of Canterbury 1989.

3 WHITTAKER, D., PARK, R., and CARR, A.J., Seismic Performance of Offshore Concrete Gravity Platforms, Research Report 88-1, Dept. of Civil Eng., Univ. of Canterbury.

Figure A-1: Artificial Accelerogram Generated by SIMQKE to Match DZ4203:1986-Normal Soils Design Acceleration Response Spectrum

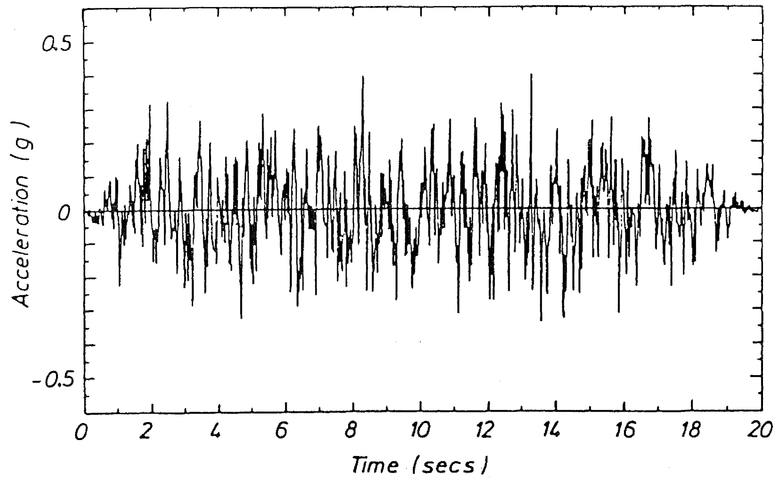


Figure A-2: Artificial Accelerogram Generated by SIMQKE to Match DZ4203:1986-Soft Soils Design Acceleration Response Spectrum

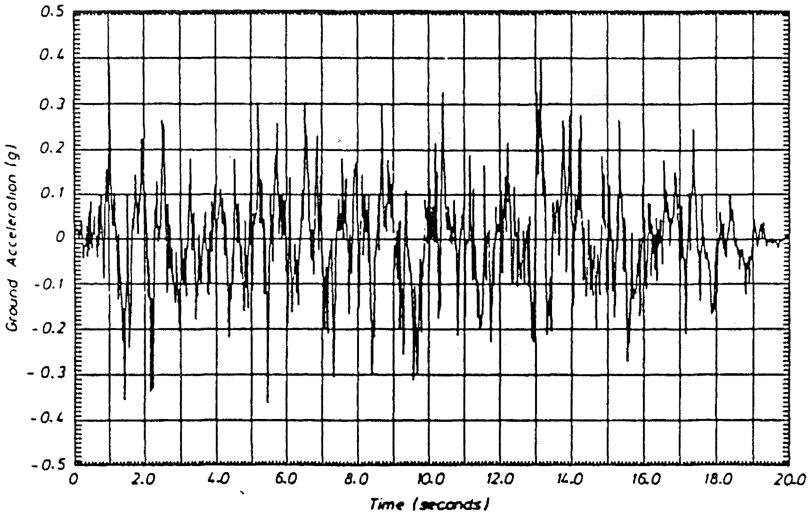
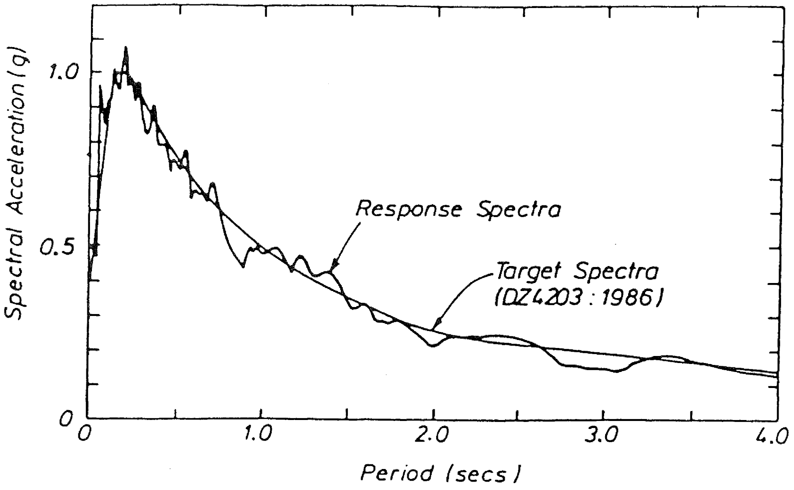


Figure A-3: Match of Acceleration Response Spectrum and Target Spectrum (DZ4203:1986-Normal Soils-Artificial Motion 1)



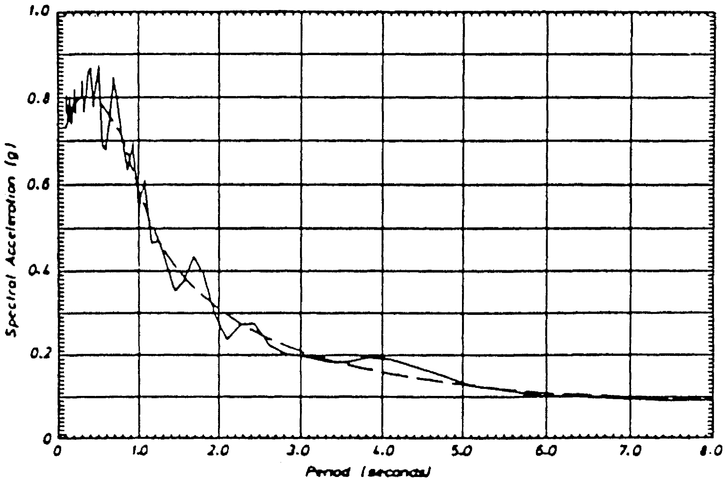


Figure A-4 : Match of Acceleration Response Spectrum and Target Spectrum (DZ4203:1986-Soft Soils-Artificial Motion 1)