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Seismic response of low-rise buildings subjected to the 1988 Saguenay earthquake

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ABSTRACT

This paper presents elastic and inelastic response spectra of strong motion accelerograms recorded during the 1988 Saguenay earthquake. Comparisons are made with the NBCC 1990 lateral forces requirements for the seismic resistant design of short-period structures. The seismic response of a typical low-rise steel building designed according to the NBCC 1990 and CAN3-S16.1-M89 is then investigated in the elastic and inelastic range. The use of a period-dependent force modification factor is proposed to take advantage of the energy dissipation capacity of short-period structures on a more rational basis. It is also shown that to obtain a realistic picture of the ductility demand of low-rise buildings, the structural overstrength, that is the supplied strength in excess of the seismic design base shear, should be explicitly considered in the design process.

INTRODUCTION

The 1988 Saguenay earthquake, which registered peak ground acceleration in the order of 10% g near the epicentre was found to contain high energy in the frequency range of relatively stiff structural systems (with periods smaller than 0.3 sec) such as low-rise buildings, concrete dams or nuclear containment structures. After the earthquake a site visit team reported that the earthquake did not cause significant structural damages. However, the poor serviceability performance of low-rise "tension-only" cross-braced buildings, structures containing "soft-storeys" as well as unreinforced masonry walls and nonstructural elements were observed (Mitchell et al. 1990). Although the Saguenay earthquake did not cause extensive damages for the level of acceleration recorded, it has raised serious concerns for the seismic performance of existing or new structures during future events with larger return periods.

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This paper presents comparisons between the engineering characteristics of the 1988 Saguenay earthquake and the 1990 edition of the National Building Code of Canada (NBCC). The comparisons are first made in terms of elastic and inelastic response spectra for the lateral forces to be considered for the seismic analysis of short-period structures. The seismic response of a low-rise "tension-only" cross-braced steel building designed according to the NBCC 1990 and CAN3-S16.1-M89 for two sites with different levels of seismicity is then investigated. The acceleration records obtained at Chicoutimi, the nearest station to the epicentre, Baie St-Paul, corresponding to the largest PGA recorded, and Quebec City, a major city further away from the epicentre were scaled to the acceleration level specified by the NBCC to study the seismic behaviour of the structural systems. The peak ground motion parameters obtained from the selected sites are summarized in Table 1.

THE 1990 EDITION OF NBCC

In the 1990 edition of NBCC, the minimum lateral seismic force at the base of the structure is given by

$$V = (V_e/R)\ U \tag{1}$$

where U is a calibration factor equal to 0.6 to maintain a level of protection based on engineering experience and consistent with the previous code requirements, and R is the force modification factor that reflects the capability of a structure to dissipate energy through inelastic behaviour. The equivalent lateral force at the base of the structure representing the elastic response, V_e, is given by

$$V_e = v\ S\ I\ F\ W \tag{2}$$

where v is the zonal velocity ratio, S is the seismic response factor corresponding to an idealized 5% damped elastic response spectrum for unit velocity, "v" and weight "W". I is the importance factor, and F is the foundation factor to represent the local amplification of the ground motions due to soft surficial soil layers. In this formulation only the zonal velocity ratio, v, is specified explicitly whereas the zonal acceleration ratio, a, is used implicity in the seismic response factor, S.

RESPONSE SPECTRA

Many parameters such as the periods of vibration, ductility, overstrength, torsional coupling, and duration of ground motion will affect the value of the structural response modification factors. The NBCC 1990 defines the inelastic design response spectra by applying a force modification factor, R, to the elastic response spectra. The value of R is independent of the ground motion and vibration characteristics of the system. It ranges from 1, for nonductile structural systems expected to remain elastic under the design earthquake ground motion, to 4 for ductile moment resisting frames with good seismic detailing. In the case of a SDOF, the value of R is directly related to the displacement ductility demand of the system.

A more accurate procedure to obtain design loads is to develop inelastic design spectra from rigorous nonlinear dynamic analyses of structural models of specified initial stiffnesses (periods) and possessing realistic strengths, hysteretic behaviour, and damping properties, subjected to accelerograms that can be expected for the site under consideration. It is obvious that this procedure is highly impractical for preliminary design. However, the ground motion records obtain from the Saguenay earthquake provide an opportunity to investigate the adequacy of the NBCC 1990 in defining design loads for short-period structures located in Eastern North America.

The strength of the SDOF system is defined in terms of a non-dimensional strength parameter, n, expressed as the ratio of the design base shear at yield, V, to the maximum effective force applied during the earthquake:

$$n = V / M\, \ddot{v}_{gmax} = (vSIF)U / Ra \tag{3}$$

where M is the mass of the system and $\ddot{v}_{gmax}$ is the PGA expressed in consistent units with the mass.

Constant strength and constant ductility elastic (R=1) and inelastic design spectra were constructed for initial periods ranging from 0.05 sec to 0.75 sec. The constant strength spectra represents the ductility demand for systems with different yield levels. The constant ductility spectra reflects the strength demand imposed on the structure by the earthquake for a specified ductility level allowing for a direct comparison with the code formulation. An elastic-perfectly plastic (EPP) hysteresis model assuming 5% viscous damping was used. The EPP model was selected because it is representative of the interstorey hysteresis of low-rise steel structures with tension-only cross-bracing that were affected most significantly by the earthquake. Moreover, previous studies that have considered the effect of EPP, bilinear and stiffness degrading hysteresis models (SDM) on the ductility demand of SDOF systems have concluded that using an EPP model for inelastic design is generally on the conservative side (Mahin and Bertero 1981).

Strength Demand -vs- Capacity

A typical constant strength spectra is shown in Fig. 1 for Baie St-Paul. For systems with initial periods greater than 0.3 sec, there is generally a reduction in the ductility demand as the period increases. For systems designed with $n > 1$ the ductility demand will remain bounded as the period is reduced and the system converges towards the static response. For systems designed with $n \leq 1$ the ductility demand will grow unbounded if the EPP model converges towards the static response. The ductility demand is thus very high for short-period structures designed with a substantial strength reduction factor.

The smoothing effect of nonlinear behaviour on the strength demand can be observed by comparing the elastic (μ = 1) and inelastic (constant ductility) response spectra as shown in Fig. 2. Note that a value of F=1.4 has been used to compute the NBCC values according to Mitchell et al. (1990). It is obvious that the design base shear will tend to be the same whether the structure is elastic or inelastic for very short-period structures since all the spectra curves converge to a strength coefficient of 1 for an infinitely rigid system with a zero period. The inelastic response spectra indicate that the required design strengths do not present significant variations to achieve prescribed ductility levels above μ = 3.0. In this case, for a given ductility, μ, the required strength of the structure, n, generally decreases with an increase in the period. For a prescribed low ductility level, say, μ = 1.5 , the required strength exceeds the ultimate value supplied by the code over a small period interval for all the records analyzed. As the ductility level is increased the required strength to maintain the design ductility exceeds the code supplied values over a wider period range which approximately correspond to the portion of the short-period (acceleration controlled) range of the code design spectra which exhibit constant amplification values.

This observation, that code supplied strength is sometimes below the demand computed from dynamic analyses, has also been found by other researchers. It is sometimes believed that actually constructed structures will possess larger damping and ductility than specified in the design, which may compensate for some of the potential deficit. However, nonlinear dynamic analyses indicate that the damping effect is uncertain because of the impulsive nature of earthquake ground motion and that increased ductility will have a very small impact on the strength demand of short-period systems with ductility levels greater than 2, as is shown by the close agreement of the required strength coefficient for these cases in Fig. 2. It must therefore be concluded that some buildings will be able to resist the seismic excitation because they are constructed with real strengths that are far in excess of the code required value. Buildings with poor construction will fail.

It should be noted that in NBCC 1990 the force modification factor can be interpreted as the product of a global ductility factor varying from 1 to 4 and a constant overstrength factor, R_s, with $U = 1/R_s$ (Fischinger and Fajfar 1990). However, post-earthquake investigations of structural damages have indicated that the overstrength depends strongly on the type of structural systems and the number of stories (periods) of the structures. An improved overstrength (calibration) factor should therefore be considered in the next editions of NBCC.

Period-Dependent Force Modification Factors (R)

For short-period structures designed to remain elastic, the strength demand exceeds the code supplied capacity over a small-period interval. In this case, it might be appropriate to increase the elastic equivalent lateral force, V_e. One approach would be to increase the seismic response factor, S, in the short period range to account for the observed high a/v ratios. To satisfy the strength demand in the short-period range for systems designed with substantial ductility levels, a period-dependent strength reduction factor, $\bar{R}$, could be used by the NBCC code. The following bi-linear variation has been adopted by many codes around the world:

$$\bar{R} = 1 + (\mu_G - 1)\, T/T_1 \quad ; T < T_1 \tag{4}$$
$$\bar{R} = \mu_G \quad ; T \geq T_1 \tag{5}$$

where μ_G is the global ductility of the structure, T is its fundamental period of vibration and T_1 is a parameter given as a function of the seismic zone and the type of soil. T_1 should also be related to the transition period between the short-period or acceleration controlled range and the medium-period or the velocity-controlled range. Using the dynamic procedure of NBCC with proper zonal amplification factor the following values can be identified for

$$Za/Zv > 1, \quad T_1 = 0.30 \text{ sec} \tag{6}$$
$$Za/Zv = 1, \quad T_1 = 0.42 \text{ sec} \tag{7}$$
$$Za/Zv < 1, \quad T_1 = 0.61 \text{ sec} \tag{8}$$

In the case of inelastic systems, increasingly smaller values of T_1 are generally observed in Fig. 2 with an increase in ductility which will correspond to a decrease in code supplied strength. The hysteresis model might also influence the value of T_1. A preliminary investigation of a SDM inelastic spectra of the transverse acceleration record at Baie St-Paul has indicated a slight shift of T_1 toward shorter periods as compared to EPP model.

In the context of NBCC, the value of μ_G in Eqs. 4,5 will correspond to R. Figure 3 shows a comparison of the strength coefficients computed using the usual R values and new coefficients using $\bar{R}$ as the strength reduction factor in the short-period range. If a period-dependent strength reduction factor is used, there is a significant increase in the strength to be supplied by the code for all originally selected R values which are greater than one. To control the ductility demand for systems designed with $n \leq 1$ a new value of $\bar{R}$ can be defined to obtain a better balance between the earthquake strength demand and the code supplied capacity. This value of $\bar{R}$ could be selected such that the value of n converges to 1 as the period is reduced from T_1 to zero.

The following procedure is suggested for the design of short-period structures with $T < T_1$: (i) select R from NBCC, (ii) compute V from Eq. 1, (iii) compute n from Eq. 3, (iv) compute $\bar{R}$ from

$$\bar{R} = 1 + (R-1)(T/T_1) \quad ; T < T_1,\ n > 1 \tag{9}$$

$$\bar{R} = \frac{n\,R}{1 + (n-1)(T/T_1)} \quad ; T < T_1,\ n \leq 1 \tag{10}$$

$$\bar{R} = R \quad ; T > T_1 \tag{11}$$

(v) obtain the adjusted equivalent lateral force at the base of the structure from

$$V = V \cdot (\bar{R}/R) \tag{12}$$

This procedure has also been implemented in Fig. 3 for comparison with the actual code. Figure 4 shows the new strength coefficients obtained by the application of Eqs. 9-11. A much better balance between the actual strength demand from the earthquake and the code supplied ultimate capacity is now observed for all values of R selected from NBCC to take advantage of nonlinear behaviour.

BUILDING ANALYZED

The two-storey steel office building shown in Fig. 5 has been designed according to CAN3-S16.1-M89 and NBCC 1990 for Baie St-Paul, Chicoutimi and Quebec City. The lateral load resisting system is made up of orthogonal "tension-only" X-braced frames. For this structural system which possesses very little lateral redundancy and for which no special detailing of connections should be required, an R value of 1.5 has been selected. The fundamental periods of vibration in the X-dir. have been computed has 0.25 sec for the building located in Baie St-Paul and 0.34 sec for the other sites.

The seismic response of short period structures positioned in the acceleration-bound region of the spectra have been shown to be strongly dominated by the first mode of vibration. Building structures which are fairly regular can thus be modelled as SDOF systems according to the procedure described by Fajfar and Fishinger (1988). The strength and the stiffness of the MDOF system are determined by applying monotonically increasing lateral loads proportional to the first mode of the structure. For example, the initial yielding of the Baie St-Paul lateral load resisting system occurred at a base shear value of 1664 kN. The frame has been designed for a maximum base shear of 1320 kN according to NBCC. The 25% overstrength can be attributed to the use of a material factor (ϕ=0.9) and the member selection process using sections available from the CISC database. The actual building overstrength is likely to be larger if non-structural components are to be considered in the design process.

To demonstrate the effect of the earthquake on the structures, the PGA of each record was increased to reach the design ductility level of 1.5. The nonlinear response indicators are summarized in Table 2. The value of μ_{acc} is computed as the ratio of the sum of the absolute displacement values of all yield excursions that occurred during the record, to the yield displacement. This quantity is significant for structures that are susceptible to low-cycle fatigue. The maximum roof displacements, δ_{max}, are also given.

A PGA of 0.76g was required to reach the design ductility level at Baie St-Paul. This value represents a "safety margin" of approximately 1.9 that can be mainly attributed to the difference between the actual supplied strength and the design value. The Chicoutimi record, on the other hand, demonstrates a very low energy content at the fundamental period of the structure resulting in a low excitation level. The response from the Quebec City record demonstrates characteristics which are similar to the Baie St-Paul response.

CONCLUSIONS

The 1988 Saguenay earthquake provided many accelerogram records of very good quality characterizing the seismic behaviour of a significant event typical of the conditions found in Eastern North America (ENA). It was found that for short-period systems designed to exhibit significant nonlinear behaviour, the NBCC 1990 does not provide a rational control of structural damage that can be expected from seismic excitation with high a/v ratio typical of ENA. The use of a period-dependent strength reduction factor applied to the elastic strength demand of short-period structures has been proposed to take advantage of the inelastic energy dissipation capacity this type of system on a more rational basis.

The performance evaluation of the low-rise buildings has indicated that the ductility demand on the structure depends on the real strength of the lateral load resisting system including overstrength. To obtain a realistic picture of the ductility demand and a rational control of the collapse threshold, explicit consideration should thus be given to overstrength during the design process.

REFERENCES

Fischinger, M. and Fajfar, P. 1990, On the response modification factors for reinforced concrete buildings, Proc. Fourth U.S. Nat. Conf. EE, (2), 249-258.

Fajfar, P. and Fischinger M. 1988, N2 - A method for non-linear analysis of regular buildings, Proc. Ninth WCEE, Tokyo, Japan, V, 111-116.

Mithchell, D., Tinawi, R. and Law, T. 1990, Damage caused by the November 25, 1988 Saguenay Earthquake, Can. J. Civ. Engng., 17(3), 338-365.

Mahin, S.A. and Bertero, V., 1981, An evaluation of inelastic seismic design spectra, J. Struct. Eng., ASCE, 107, 1777-1795.

Table 1. Peak ground motion parameters

Site	PGA (g)	PGV (m/s)	NBCC a	NBCC v
BSP.	0.174	0.053	0.40	0.40
Chc.	0.131	0.025	0.20	0.15
Qué.	0.051	0.022	0.19	0.15

Table 2. Nonlinear indicators for steel building

Site	a (g)	n	μmax	μacc	δmax (mm)
BSP.	0.40	3.48	0.83	-	21.8
	0.48	2.90	1.00	1.00	26.2
	0.76	1.83	1.50	1.85	39.3
Chc.	0.20	3.97	0.22	-	6.4
	0.91	0.87	1.00	1.00	29.0
	1.60	0.50	1.50	2.25	43.6
Qué.	0.19	4.18	0.41	-	11.7
	0.47	1.69	1.00	1.00	28.9
	0.73	1.09	1.50	1.69	43.2

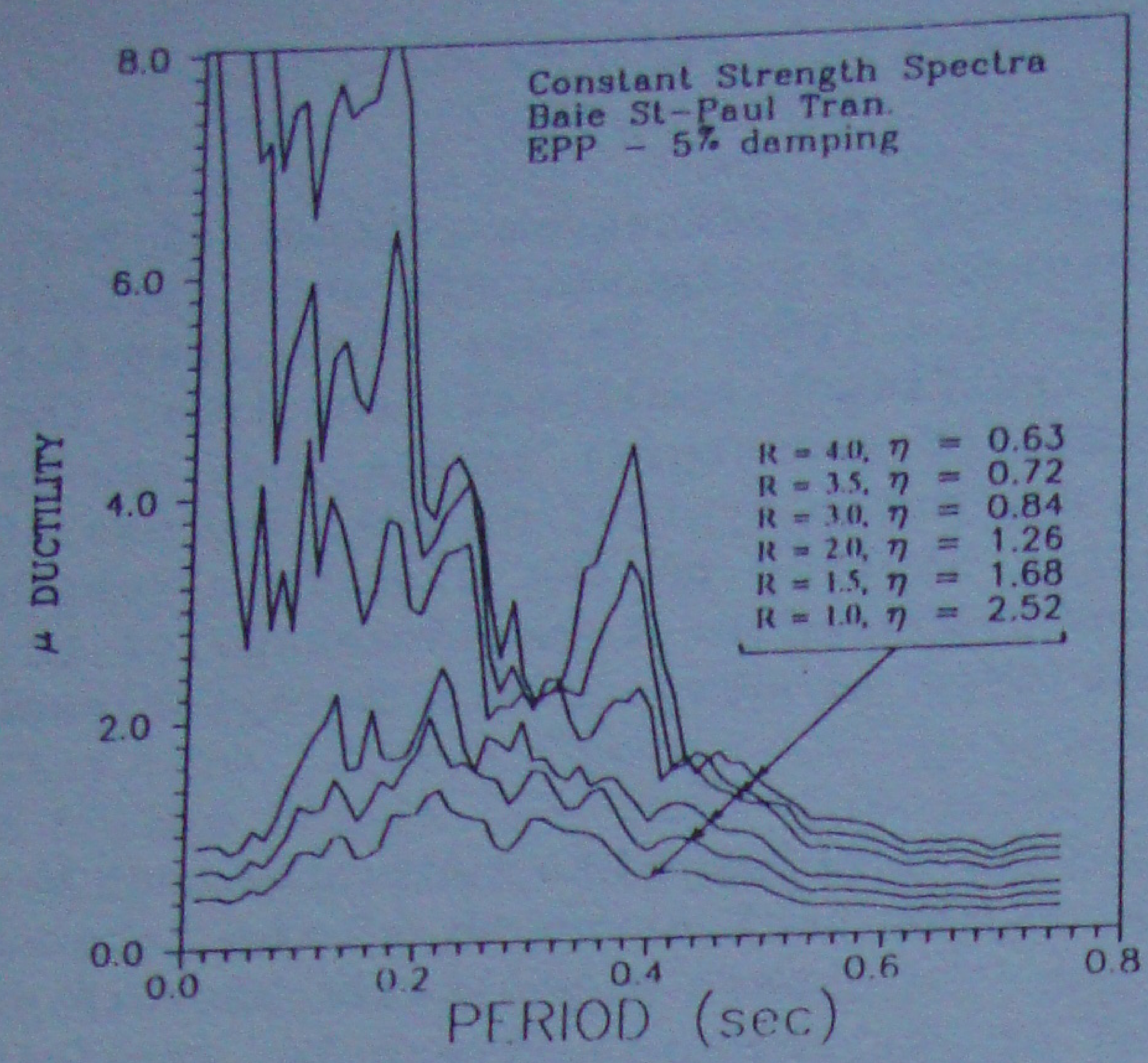

Figure 1. Constant strength spectra

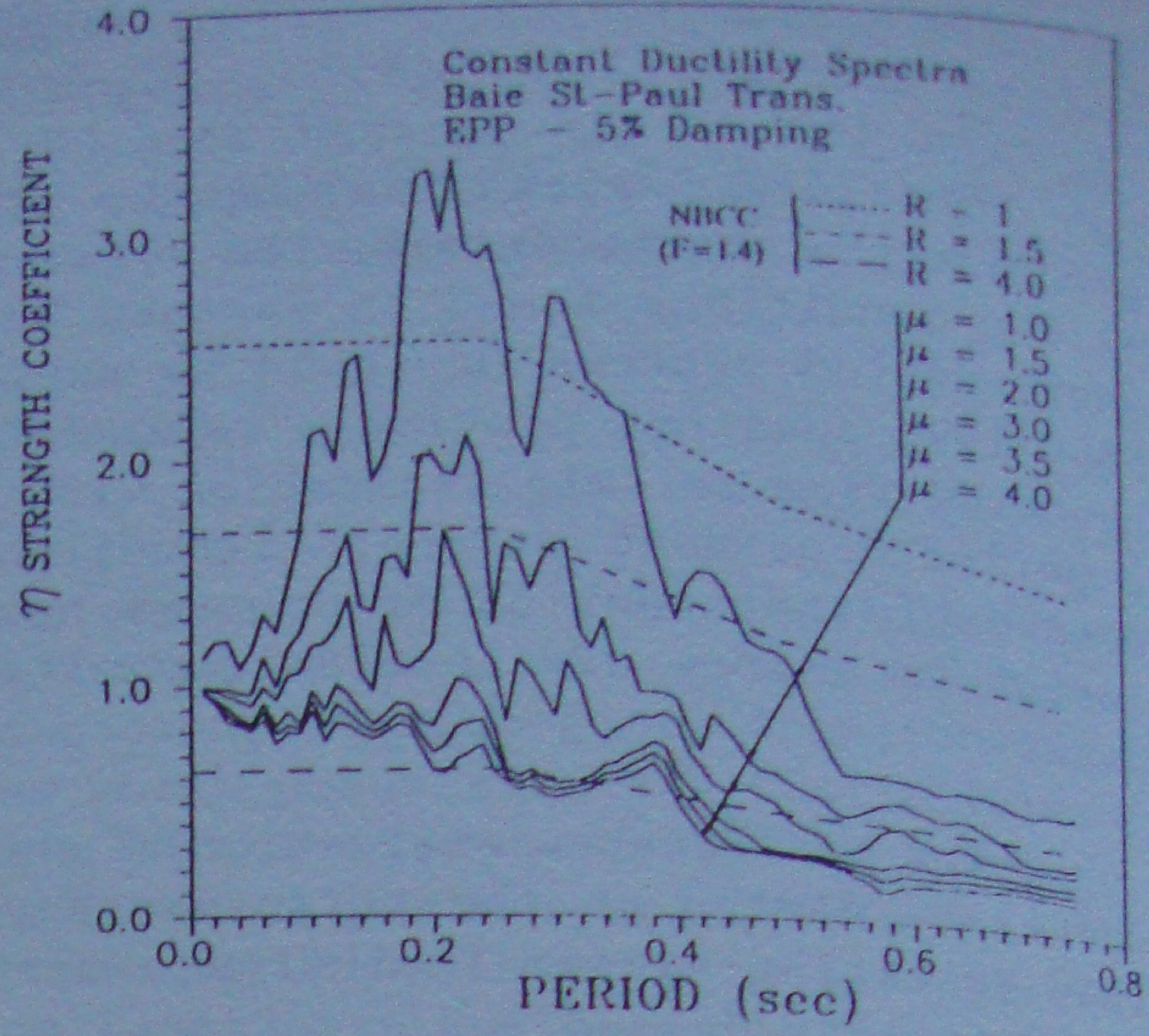

Figure 2. Constant ductilty spectra

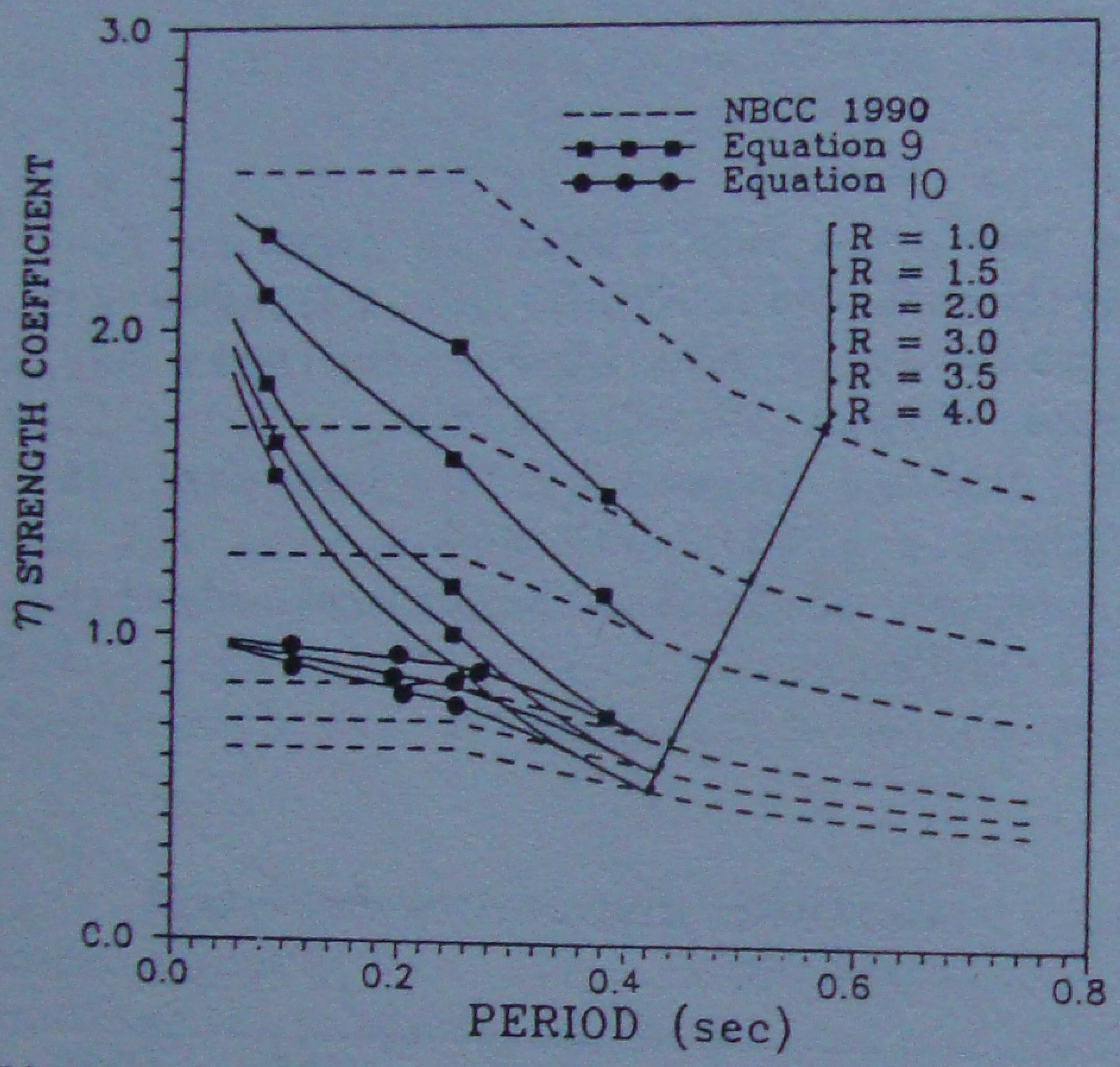

Figure 3. Ultimate strength capacity using R and $\overline{R}$.

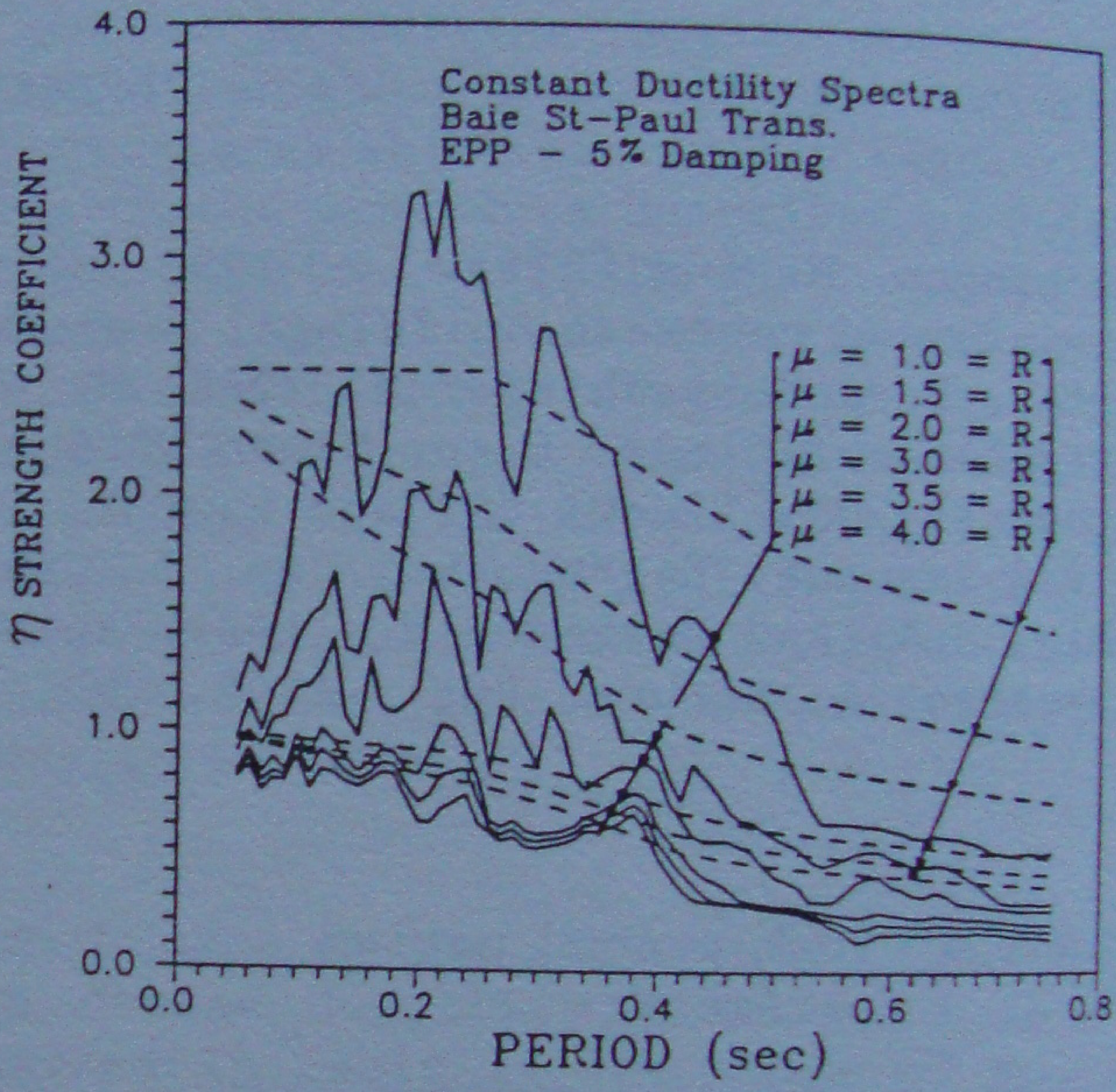

Figure 4. Strength demand -vs- capacity using $\overline{R}$.

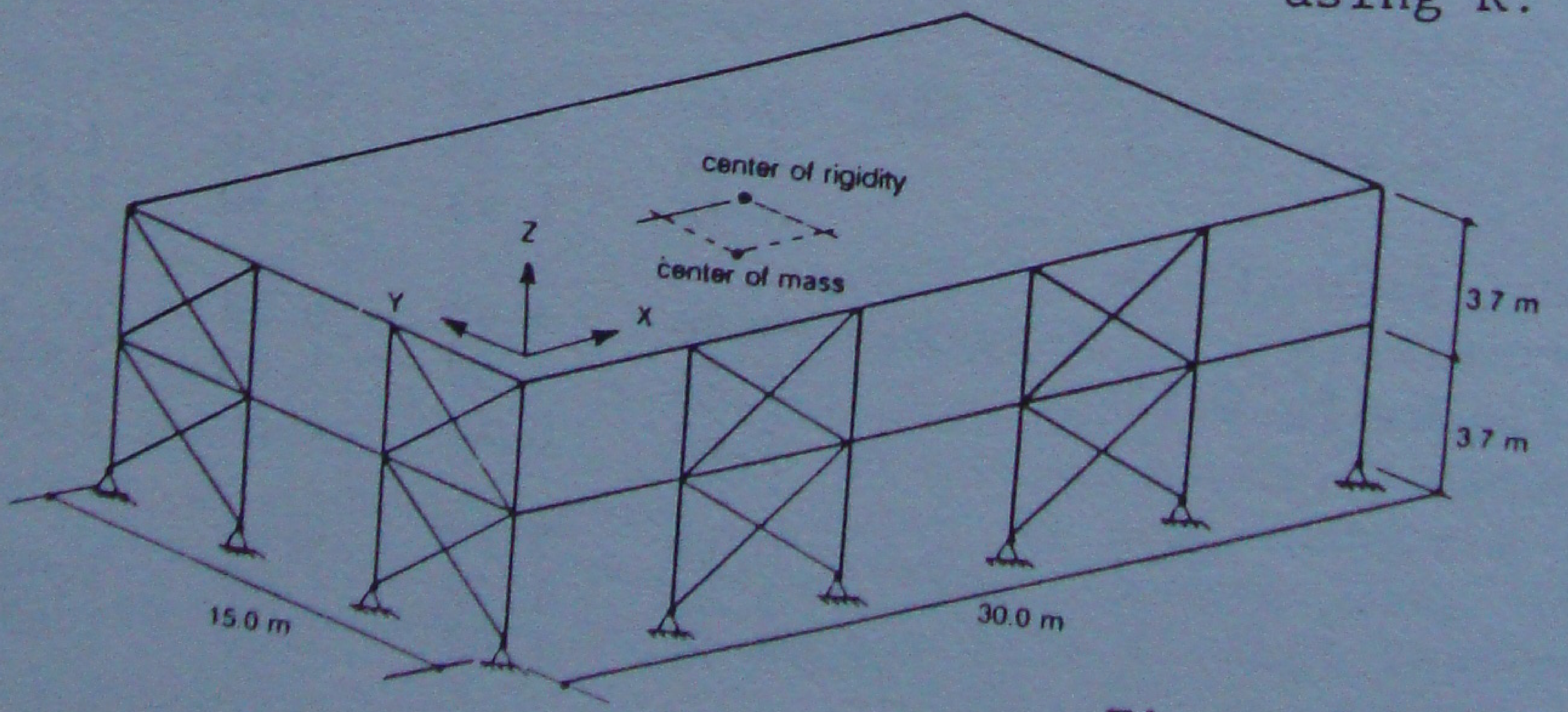

Figure 5. Building Analyzed