Calculation of Earthquake Actions on Building Structures in Australia

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ABSTRACT: This paper presents from first principles methods of evaluating the seismic performance of a building using the method of inertial forces, method of maximum energy and method of maximum displacement. The introduction of these methods forms the main thrust of the paper. Importantly, the building can be deemed safe should this be indicated by any one of the three methods none of which requires the natural period of the building nor structural response factors to be estimated. Whilst these methods are very simple and consume little time to apply, the accuracies of the results are comparable with those from response spectrum methods. It is noted that the fundamental basis of each of these methods is very consistent with the new response spectrum model stipulated by the new Australian standard for seismic actions. A succinct and insightful account of the development of the seismic hazard model for Australia is also provided followed by a commentary on the use of dynamic analysis methods in practice.

1 INTRODUCTION

The *Equivalent Static Analysis* method commonly used in the seismic design and assessment of buildings structures enables a complex dynamic problem to be solved by the considerations of static seismic design forces. The characteristic reduction in the amplitude of the design force with increasing natural period of the building is taken into account in most codes of practices by means of a response spectrum. Whilst the method appears straightforward and is well known, it is also problematic in practical applications as the natural period of the building is often very difficult to determine. Simple algebraic expressions have been recommended for the estimation of the natural period of building structures. Some of these expressions are based on ambient conditions and hence could grossly understate the natural period of the building in an earthquake. Consequently, the calculated required base shear resistance of the building could be significantly higher than what is actually necessary for the satisfactory seismic performance of the building. A more fundamental issue with the equivalent static force method is the absence of an explicit approach to the displacement and energy absorption capacity of the structure in the evaluation of seismic actions on the building.

This paper presents from first principles methods of evaluating the seismic performance of a building using the method of inertial forces (section 2.1), method of maximum energy (section 2.2) and method of maximum displacement (section 2.3). The fundamental basis of each of these methods is very consistent with the new response spectrum model stipulated by the new edition to AS/NZS 1170.4. It is noted that these methods are outside the *Equivalent Static Analysis* provisions in the Standard (and can be described collectively as *Non-linear Static Analysis* which is also permitted by the Standard). The introduction of these methods forms the main thrust of the paper. Whilst these methods are very simple and consume little time to apply, the accuracy of the results are comparable with those from response spectrum analyses except when higher mode effects are significant (which is unlikely for buildings up to 25 m in height). Importantly, the building can be deemed safe should this be indicated by any one of the three methods (section 2.4). A succinct and insightful account of the development of the seismic hazard model for Australia is next provided (section 3) followed by a commentary on the use of dynamic analysis methods (section 4).

2 SIMPLIFIED METHODS FOR ESTIMATING SEISMIC ACTIONS

2.1 Method of Inertial Forces

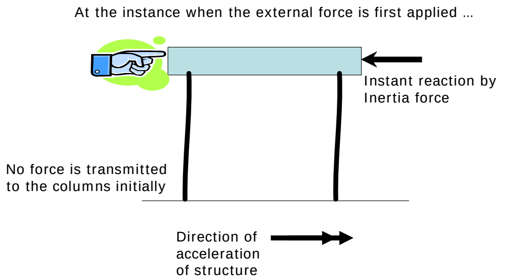
The most commonly used simplified method for calculating seismic actions on a building is based on representing those actions by a set of equivalent horizontal static design forces expressed as a percentage of the gravitational loading on the building. Many codes and regulations in countries of low and moderate seismicity, like Australia, employ the simplest form of this method in which the inertial force is expressed as a constant percentage of the gravitational loading. For example, the new edition to the Standard AS/NZS 1170.4 stipulates horizontal seismic design forces to be 10 % of gravitational loading for buildings not exceeding 12 m in height. This form of provision for horizontal loading is not necessarily exclusive to seismic loading and has been applied in a much broader context. For example, the robustness provisions (clause 6.2.2) in AS1170.0 : 2002 stipulates a minimum horizontal loading of 2.5 % gravitational loading to ensure a minimum level of robustness in the building.

The type of provision described above is a low tier method of specifying seismic design forces which has the advantage of simplicity as no dynamic analysis is involved and the natural period of the building need not be estimated. It is noted that the estimated actions which have not allowed for the variation in intensity of the horizontal design forces with the natural period of the building could become very conservative when applied to high period (tall) building structures. A qualitative description of how seismic actions are affected by the natural period of the building is provided in below using a simple single-storey case study building.

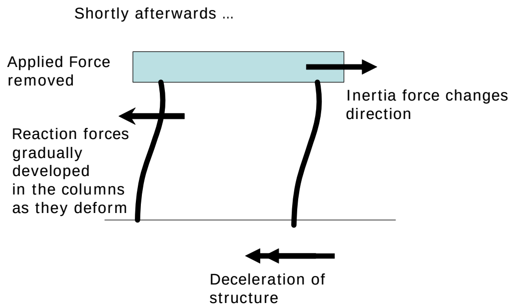
The application of a transient force (F_t) to a single-storey structure results in an inertial force (F_I) generated by the accelerating storey-mass to resist the applied force as shown in Figure 1a. In an earthquake, the applied transient force is associated with the acceleration of the ground, $\ddot{x}_g$ (ie $F_t = M\ddot{x}_g$) whereas the inertial force is generated by the acceleration of the storey relative to the ground, $\ddot{x}$ (ie $\boldsymbol{F_I} = \boldsymbol{M}\ddot{x}$). Thus, the inertial force can be described as the initial "defence" for countering the applied forces. Meanwhile, reactions from the columns (F_R) are developed with a delay, given that these reactions are proportional to the sway of the columns (assuming linear elastic behaviour) and hence take time to develop, as shown in Figure 1b.

First, consider the hypothetical case of a single-storey building with a heavy roof mass (ie. large M and natural period T is as high as 2 seconds). The building is subject to a single ground acceleration pulse, of about 0.5 seconds in duration (t_d) as shown in Figure 2a.

The application of ground accelerations to the building is like applying a transient force to the roof as depicted in Figure 1a. The time-histories of both the ground accelerations ($\ddot{x}_g$) and the reaction forces from the responding columns as obtained from a dynamic analysis, assuming linear elastic behaviour, are shown in Figure 2a. The column forces presented in the figure have been normalized with respect to the storey-mass (thus, the y-axis is in units of acceleration (ie. m/sec^2) for each line shown). It is shown in Figure 2a that the ground accelerations have already subsided by the time the columns experience significant sway and develop reactions. Consequently, the column reaction forces generated are much lower than $M\ddot{x}_g$ (or F_R/M is much lower than $\ddot{x}_g$).

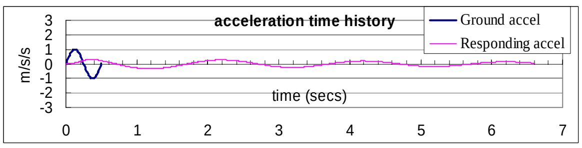


(a) Initial condition

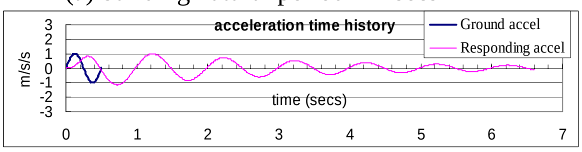


(b) Moments later

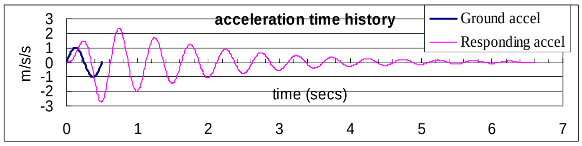
Figure 1 Reactions to transient force



(a) building natural period = 2 secs



(b) building natural period = 1 sec



(c) building natural period = 0.5 secs

Figure 2 Acceleration time-histories of building subject to single ground pulse

In other words, the inertial resistance generated by the self-weight of the building contributed mostly to its defence against the applied ground accelerations. In contrast, the columns were subject to very small deformations and consequently developed very small reaction forces.

Second, consider another case in which the storey-mass was reduced considerably so that the natural period T as defined by equation (1) has been shortened to about 1.0 second, and then to 0.5 seconds (refer Figures 2b and 2c respectively).

$$\boldsymbol{T} = 2\pi\sqrt{\frac{\boldsymbol{M}}{\boldsymbol{K}}} \qquad (1)$$

where M = storey-mass and K = storey-stiffness.

With a natural period of 0.5 seconds, the columns deform and develop reaction forces much faster than before. The inertial resistance, F_I , of the building soon changes sign as the storey stops accelerating and starts to decelerate (as depicted in Figure 1b). The inertial force generated by the storey-mass might then superpose with the applied transient forces, adding to the severity of the overall forces on the building. Consequently, the reaction forces from the columns normalized with respect to M (ie. F_R / M) became much higher than before, as demonstrated by comparing Figure 2a with Figures 2b and 2c.

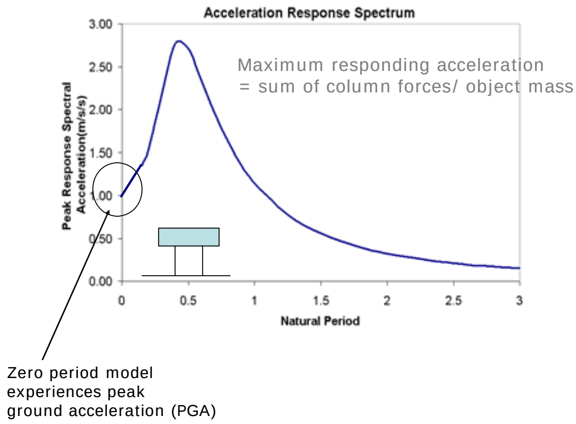
From the foregoing description, it is clear that the response of the storey to an acceleration pulse, or train of acceleration pulses, depends very much on the duration of the individual pulse (or the overall dominant period of the applied ground excitations) in relation to the natural period of the building (T). The sensitivity of the normalized column forces to the value of T as obtained from dynamic analyses is best represented by an acceleration response spectrum as shown in Figure 3a (which is associated specifically with the ground acceleration pulse as shown in Figures 2a and 2b). The highest normalized column forces analysed for a given natural period is represented by the ordinate of the response spectrum. For example, a maximum amplification factor of 2.8 at the natural period of 0.5 seconds is featured in the response spectrum of Figure 3a. The second response spectrum shown in Figure 3b was calculated from the acceleration time-histories recorded at El Centro, Southern California in the well known 1940 Imperial Valley earthquake of Richter Magnitude 6.6. A peak ground acceleration of approx. 3 m/sec^2 and an amplification factor of about 2.5 is noted. Many response spectrum models stipulated in seismic codes of practices around the world have been based on the normalized response spectrum of the El Centro motion as shown in Figure 3c.

A common feature of the response spectra is the decrease in the spectrum ordinate with increasing natural period beyond the corner period T_1 (which is typically of the order of 0.1 – 0.3 seconds on rock or stiff soil sites and can be considerably higher on soft soil sites). The response spectrum can be represented by the linear – hyperbolic relationship of equations 2a and 2b.

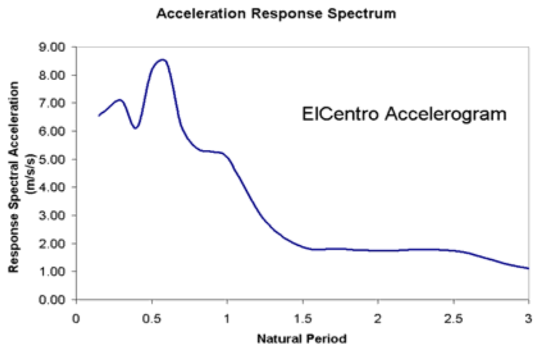
$$RSA = RSA_{max} = C_1 \, F_a \, R_p Z \quad (T \le T_1) \quad (2a)$$

$$RSA = \frac{C_2 R_p Z F_V}{T^n} \quad (T > T_1) \quad (2b)$$

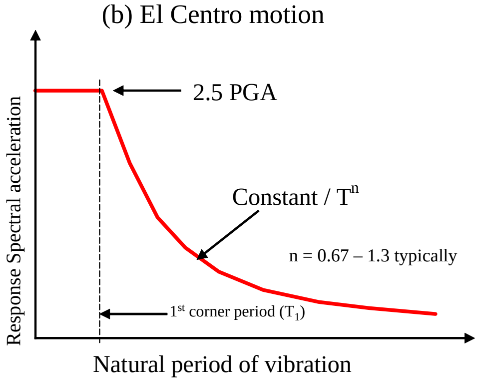
where R_p is the return period factor, Z is the seismic coefficient, C_1 is a constant which is typically taken as 2.5; C_2 is another constant; *Fa* and *Fv* are site factors (listed in Table 1 according to provisions in the new edition to AS1170.4); and exponent n is typically in the range 0.67 - 1.3.



(a) single ground pulse



(b) El Centro motion



(c) code model (schematic)

Figure 3 Acceleration response spectra

Table 1 Site Factors stipulated by new Standard AS/NZS 1170.4

Site Class	A	B	C	D	E
F_a	0.80	1.00	1.25	1.25	1.25
F_v	0.80	1.00	1.40	2.25	3.50

The flat part of the response spectrum (for conditions $T < T_1$) represents acceleration controlled conditions in which the maximum design acceleration of the building is correlated directly with the peak ground acceleration (*PGA*). In such conditions, the seismic forces are not sensitive to both the natural period of the building nor the lateral stiffness. In contrast, the hyperbolic relationship of equation (2b) (for periods $T > T_1$) mean that a taller structure, with a higher natural period, tends to develop lower design accelerations. Conversely, stiffer structures with lower natural period tend to develop higher design accelerations and inertial forces.

Consequently, increasing the size of the lateral resistance members in the building (with the intention of increasing the lateral load resistance) attracts higher seismic design forces. The *Equivalent Static Analysis* (force-based) method, although well known and easy to understand, can be very problematic to apply if the natural period of the building is uncertain (which is the case with most building structures, particularly reinforced concrete structures). The effective stiffness of reinforced concrete columns and beams in the cracked state could be very sensitive to the longitudinal reinforcement content and the level of pre-compression even before the onset of notional yielding in the lateral resisting elements of the building as highlighted by Priestley (1998). Ignoring these factors in modeling the structure could result in a gross mis-representation of the effective stiffness of the member and hence the natural period properties of the building as a whole. In summary, the dynamic properties of a building are much more complex than is typically assumed with conventional modeling approaches, even if the effects of interaction of the structure with non-structural components (such as partitions and facades) and with the foundation have been included. Difficulties in predicting the natural period of the building directly translate into difficulties in accurately ascertaining the seismic design forces in the force-based analysis.

Furthermore, when structural drifts are required to be checked to satisfy stability requirements and other performance requirements, the stiffness values must be used twice : (i) for the determination of the natural period which is, in turn, required for the seismic design forces, and (ii) for the calculation of drifts when the building is subject to the applied forces. The estimated stiffness for '(i)' is often implicitly defined by simple code rules (which typically expresses natural period as a function of the height of the building), whereas the estimated stiffness for '(ii)' is as specified in the structural (finite-element) model of the building. It is noted that these two estimates could be very inconsistent and hence could result in significant errors.

Notwithstanding problems with this force based method as described above, it is nevertheless convenient to check the normalized lateral strength of low period buildings against the maximum acceleration demand (RSA_{max}) as defined by equation 2a, which does not require the natural period of the building to be estimated.

The calculated acceleration demand (RSA_{max}) can be used for comparison with the ultimate normalised lateral strength of the building (strength normalized with respect to the mass of the building) in order that the adequacy of the seismic resistant capacity of the building can be ascertained. As noted earlier, the assessment method described is outside the *Equivalent Static Analysis* provision in the Standard. Consequently, prescriptive structural response factors including the *Structural Ductility Factor* (μ) and the *Structural Performance Factor* (S_p) do not apply. Instead, non-linear static (often described as "push-over") analyses can be undertaken to determine the ultimate lateral resistance of the building at the threshold of collapse. The ultimate resistance so determined by this approach can be considerably higher than that calculated by the conventional approach (wherein inelastic behaviour is only considered at the element level, and the structure as a whole is analysed assuming elastic behaviour). Obviously, it is conservative to take the design ultimate strength derived from conventional calculations as the ultimate strength in a non-linear static analysis.