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A Unified Code Model for Seismic and Impact Actions

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ABSTRACT

In Australia, separate standards have been developed to provide guidance on the estimation of dead load, live load, wind load, snow load and seismic load. All but the *Standards for Seismic Actions* and part of the standard for wind loading are based on static conditions. There is little written on how transient actions, and impact actions in particular, are to be estimated. This paper explains how the response spectrum model stipulated by the current (new) edition of the *Australian Standard for Seismic Actions* can be used for impact actions which can include the collision of a vehicle on a barrier or on the support of a (bridge or building) structure. The impact action of a fallen object, or a projectile, on a structural element can also be estimated using the response spectrum model which was originally developed for estimating seismic actions. It is proposed that in the future a range of transient actions in extreme conditions can be covered by the same standard. This paper is presented in a way that is intelligible to the average practicing structural engineer. Prior knowledge of impact dynamics is not required.

Keywords: earthquake; seismic, impact, collision, Standards

1. INTRODUCTION

Impact action is the second most common form of loading on structures next to gravity loading. Yet, it is difficult to find literature references or regulatory documents that provide general guidance to practicing engineers on the assessment of the effects of impact actions on structures. No part of the AS1170 loading standard series makes explicit references to impact actions.

With gravity dead loads, engineers would only need to have guidance on the density of the material and the partial safety factors. The training of engineers on statics and structural analysis would enable the gravity loads to be translated into internal forces for comparison with the strength of the structure. However, few engineers have the skills of analysing loading which involves the dropping of an object or other forms of impact. When specialized software is used to undertake the analysis, few engineers have the skills of evaluating and interpreting the computer generated results.

Isolated clauses can be found in bridge design standards on the design collision force for vehicular parapets or bridge supports that are situated close to a highway or railway track. Guidances on the berthing and mooring forces for the design of piers and dolphins can also be found. However, these clauses are typically prescriptive in nature and address specific types of loading without necessarily stating the basis of the recommendations and assumptions made. Engineers would not be able to make their own judgment on how to modify the provisions when conditions have changed. For example, few engineers would know how to make adjustment to the design collision force on a bridge pier to allow for an increase in the tonnage and design speed of vehicles. This has much to do with the ways civil engineers are educated. Whilst the teaching of statical equilibrium and free-body diagrams is the usual theme in the early (technical) part of a civil engineering degree program, the teaching of dynamics has been mostly in the context of analyzing mechanical/electrical systems. It is rare to see references being made to the basic principles of momentum transfer in the teaching of structural analysis. Ironically, most live loads experienced by a structure exposed to the environment are characterized by the motion of objects which make contact with the structure thus imposing a hazard.

The earthquake loading model adopted by the new standard for seismic actions for Australia (AS1170.4 – 2007) offers the opportunity of closing the knowledge gap. The response spectrum is of the tri-linear form and is constrained by parameters associated with the three elements of ground motion: acceleration, velocity and displacement as illustrated in Section 2 of the paper. It is demonstrated in this paper that the response spectrum calculated from a pulse generated by collision can also be represented by the same tri-linear model (refer Section 3). Results of idealized impact analyses are then presented in the rest of the paper for the development of a generalized model for impact actions. Finally, a unified model that can be used for representing both seismic actions and impact actions is presented.

2. THE CONSTRUCT OF THE EARTHQUAKE LOADING MODEL

The natural period dependable earthquake loading (response spectrum) model stipulated by the new standard for seismic actions for Australia (AS1170.4 – 2007) is made up of three piecewise continuous functions which can be written in the following form :

$$RSA\left(\text{in units of } g's\right) = C_1 \qquad \text{for } T \le T_1 \tag{1a}$$

$$RSA\left(\text{in units of } g's\right) = \frac{C_2}{T} \qquad \text{for } T_1 < T \le T_2 \tag{1b}$$

$$RSA\left(\text{in units of } g's\right) = \frac{C_3}{T^2} \qquad \text{for } T > T_2 \tag{1c}$$

where RSA is the response spectral acceleration (or base shear of a sdof system normalized with respect to its mass), T is the natural period of the sdof system, T_1 and T_2 are the first and second corner periods respectively, and C_1, C_2 and C_3 are coefficients that are dependent on the seismic hazard factor (Z) and the subsoil classification.

For a hazard factor (Z) of 0.08, which is the case for Melbourne, Sydney and Canberra for a return period of 500 years, the values of the coefficients C_1, C_2 and C_3 for subsoil class B (rock) and D (soft soil) are summarized in Table 1.

Table 1 Coefficient values for hazard factor of 0.08

Subsoil classification	C_1	C_2	C_3
B	0.24	0.069	0.104
D	0.30	0.242	0.363

The rationale behind this earthquake loading (response spectrum) model is much easier to comprehend if the response spectrum is presented in multiple formats. Response spectral co-ordinates can be expressed in terms of : (i) acceleration or normalized base shear (which engineers are most familiar with), (ii) velocity or normalized kinetic energy and (iii) displacement or relative drift demand. Examples of response spectra expressed in different formats are shown in Figures 1a – 1c. In the acceleration-displacement action diagram of Figure 1d, the displacement (or relative drift demand) is plotted against the acceleration (or normalized base shear). The transformations between acceleration, velocity and displacement are defined by the following expressions based on standard structural dynamics principles :

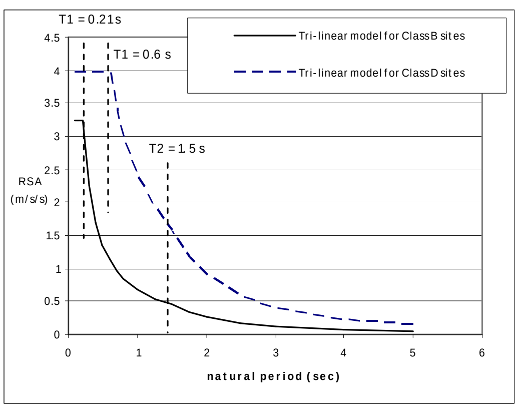
$$RSA\left(\text{in units of } g's\right) = \frac{2\pi}{T} RSV\ (\text{in } m/s) . \frac{1}{9.81} \tag{2a}$$

$$RSV\ \left(\text{in units of } m/\text{s}\right) = \frac{2\pi}{T} RSD\ (\text{in } m) \tag{2b}$$

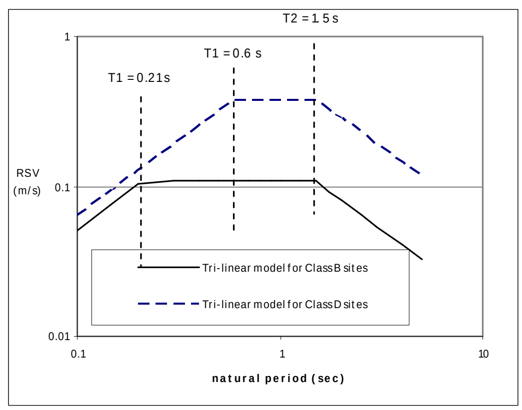
$$RSA\left(\text{in units of } g's\right) = \left(\frac{2\pi}{T}\right)^2 RSD\ (\text{in } m) . \frac{1}{9.81} \tag{2c}$$

The highest level in the acceleration response spectrum (Figure 1a) represents the highest acceleration demand (or highest normalized base shear) that a sdof system can be subjected. The highest level in the velocity response spectrum (Figure 1b) represents the highest velocity that can be developed in a sdof system during the course of the dynamic response. Likewise, the highest level in the displacement response spectrum (Figure 1c) represents the maximum displacement or relative drift demand that can be experienced by a sdof system within the natural period range of interests.

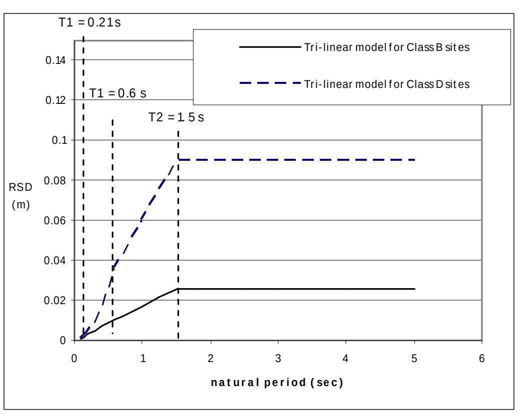
As is shown in Figures 1a – 1c, equation (1a) represents the constant (maximum) acceleration segment of the response spectrum, equation (2a) represents the constant velocity segment and equation (3a) represents the constant displacement segment. Consequently, sdof systems subject to earthquake excitations can be described as possessing acceleration, velocity or displacement controlled conditions depending on the region of the response spectrum which the natural period of the structure falls within. In the velocity response spectrum of Figure 1b which is shown in logarithmic scale, the three segments of the idealized spectrum are made of three straight lines, and hence the term *tri-linear model*.



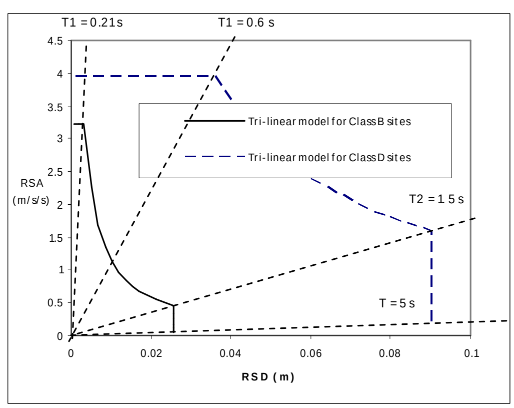
(a) Acceleration response spectrum



(b) Velocity response spectrum



(c) Displacement response spectrum



(d) Acceleration–Displacement Diagram

Figure 1 Response spectrum model of the Standard for Seismic Actions AS1170.4 (2007)

The acceleration-displacement action diagram of Figure 1d also presents very clearly the three segments of the response spectrum: the flat (horizontal) segment of constant acceleration, the hyperbolic segment of constant velocity and the vertical segment of constant displacement. The three segments are separated by radial (*broken*) lines representing the corner periods : T_1 and $T_{2.}$ Equation 3 which defines the hyperbolic segment was derived by substituting equation 2b into equation 2c (in order that variable T can be eliminated) and by holding *RSV* constant at the value of V_{max}.

$$RSA(\text{in units of } g's) = \frac{V_{\max}^2}{RSD} \cdot \frac{1}{9.81} \tag{3}$$

where RSD and $V_{\max}$ are in units of metres and seconds

The highest acceleration level (RSA_{max} or A_{max}) in units of *g's* is taken by AS1170.4 (2007) as $3.Z.F_a$ where F_a (site factor) is 1.0 for rock and 1.25 for other subsoil classes. The highest velocity level (RSV_{max} or V_{max}) is taken as 1.8 times the peak ground velocity (*PGV*) on a rock site times F_v where *PGV* (rock) in units of *mm/s* is taken as 750.*Z.* and F_v is 1.0 for rock and 3.5 for subsoil class D. The highest displacement (RSD_{max} or D_{max}) is then V_{max} times $T_2/2\pi$ where T_2 is taken as 1.5s for all subsoil classes in Australia. The coefficient values listed in Table 1 were calculated by substituting the values of A_{max}, V_{max} and D_{max} as defined above into equations 2a – 2c .

In summary, the construction of the tri-linear earthquake loading model of AS1170.4 (2007) is based on the following :

(i) $A_{max} = 3.Z.\ F_a$

(ii) $V_{max} = 1.8\ (750.Z)\ F_v$

(iii) $D_{max} = V_{max}\ (T_2/2\pi)$ where $T_2 = 1.5$ s

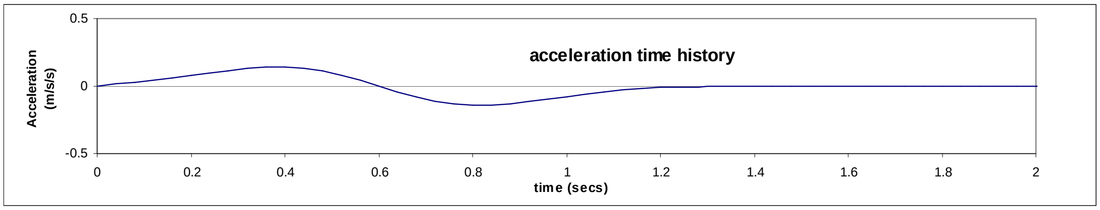
(iv) The transformation relationships of equations 2a – 2c and equation 3.

3. GENERALISATION OF MODEL FOR PULSE TYPE LOADING

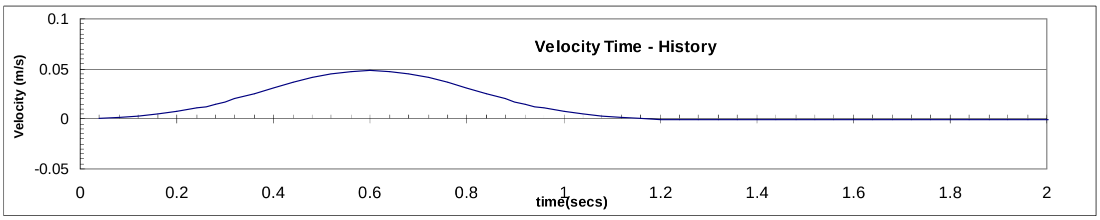
The earthquake loading model described in Section 2 was originally developed from regression analysis of response spectra calculated from both the real (recorded) and synthetic (computer generated) accelerograms for sites of different subsoil classes. In this section, the behaviour of the response spectrum for a single arbitrary pulse that is generated by a simple, and abrupt, translational motion of the ground is first studied. The second type of pulse to be studied is that delivered by the impact of a projectile on a lumped mass sdof system structure. The first type of pulse is denoted herein as "ground pulse" and the second type as "collision pulse".

First, consider that the ground translate by some 25 mm in approximately 1 second. The acceleration and velocity time-history of the ground motion, as shown in Figures 2a and

2b respectively, indicates a peak ground acceleration of 0.15 g and a peak ground velocity of 50 mm/s. Integration of the ground velocity function gives a permanent ground displacement in the order of 20 – 25 mm.

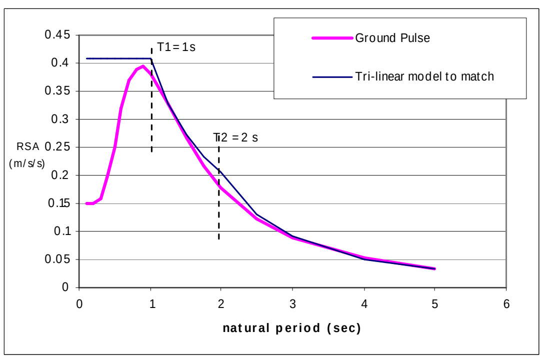


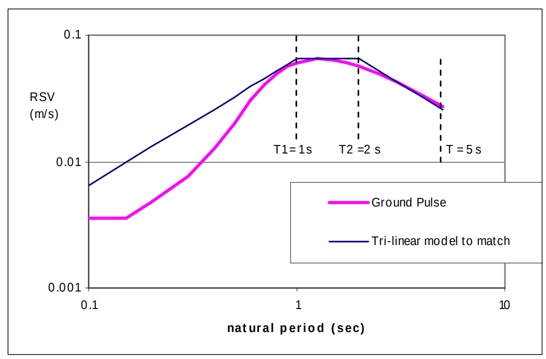
(a) Acceleration time-history



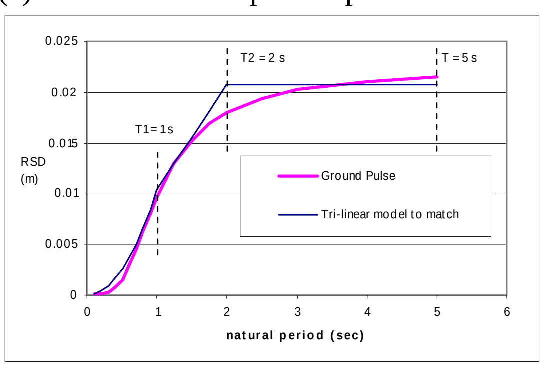
(b) Calculated velocity time-history

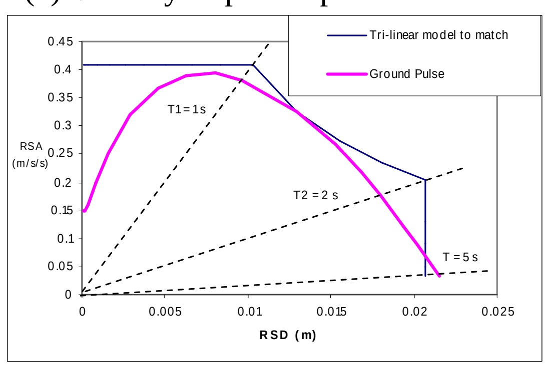
Figure 2 An arbitrary single ground pulse





(a) Acceleration response spectrum (b) Velocity response spectrum





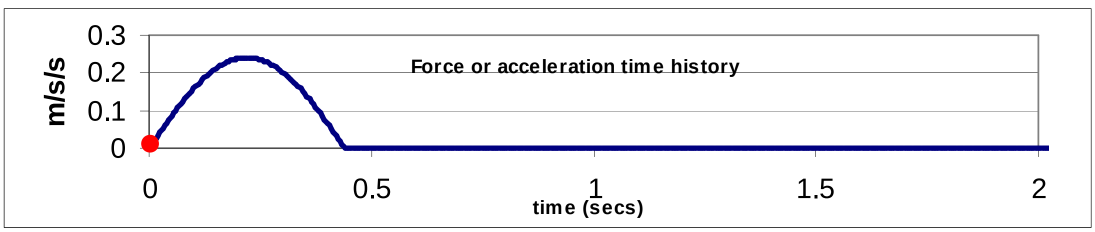
(c) Displacement response spectrum (d) Acceleration–Displacement Diagram

Figure 3 Response spectrum model of the arbitrary single ground pulse

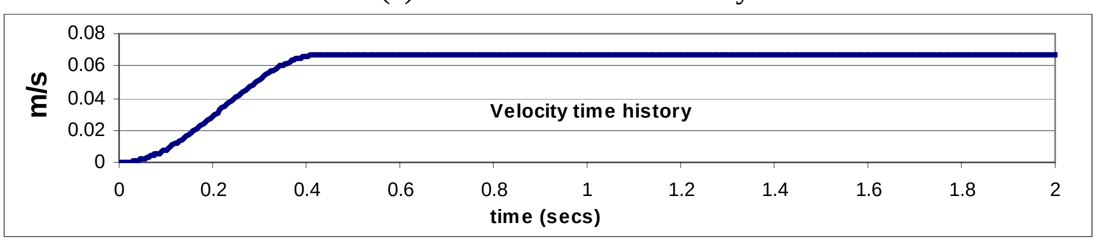
The response spectra calculated from the time-history based on 5% critical damping are shown in the different formats in Figures 3a – 3d. Clearly, the response spectra are well represented by the tri-linear model in a manner similar to that for earthquake loading. The three regions associated with acceleration, velocity and displacement controlled behaviour can be seen and they are separated by corner periods (T_1 and T_2) of 1 and 2 seconds respectively. It can be shown that the value of T_1 can be made lower by the introduction of higher frequency excitations (ie. noise). The value of T_2 can be made to change by varying the pulse duration. The value of A_{max} as shown in Figure 3a is not 3 times the peak ground acceleration but is of similar order of magnitude. The value of V_{max} (65 mm/s) as shown in Figure 3b is about 1.3 times the PGV (and not 1.8 PGV as in the case of the earthquake loading model). The value of D_{max} (22 mm) as shown in Figure 3c is consistent with the amplitude of the ground translation.

In summary, the response spectra calculated from the arbitrary ground pulse can be matched by the tri-linear model (although the corner periods and the ground motion multipliers are not the same as that defined by AS1170.4 – 2007).

Next, consider an arbitrary collision pulse. The acceleration values were calculated from the contact force generated by the impact and then normalized with respect to the mass of the target structure. The sinusoidal nature of the acceleration pulse as shown in Figure 4a is indicative of the elastic nature of the impact. Integration of the acceleration time-history gives the velocity time-history of Figure 4b. Unlike a ground pulse, a collision pulses has non-zero terminal velocity. In the illustrated example, the terminal velocity is about 60 - 65 mm/s. In reality, the duration of a pulse delivered by an impact of a solid projectile on a structure is typically of the order of milliseconds or tens of milliseconds. In the hypothetical case illustrated in Figure 4, an unusually long pulse duration of 0.5 s was chosen in order that direct comparison of the response spectra calculated from the collision pulse and the ground pulse could be made.

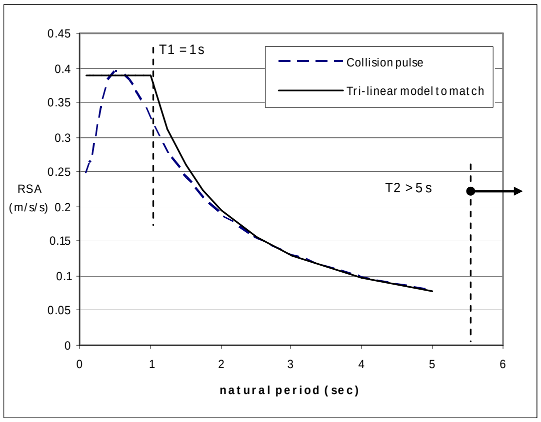


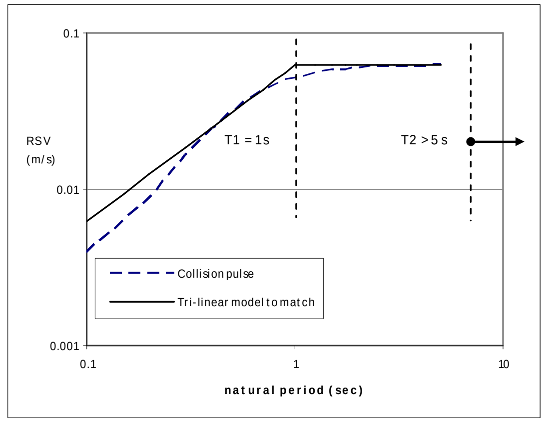
(a) Acceleration time-history



(b) Calculated velocity time-history

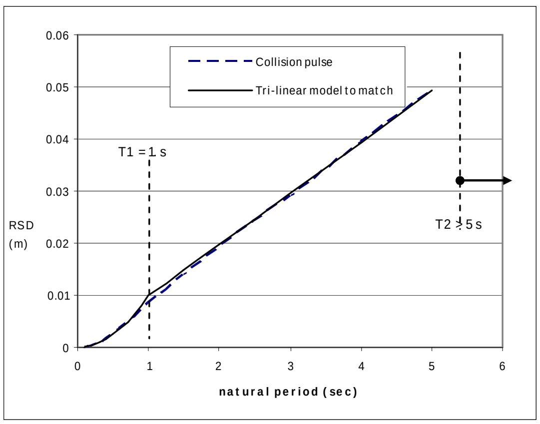
Figure 4 An arbitrary collision pulse

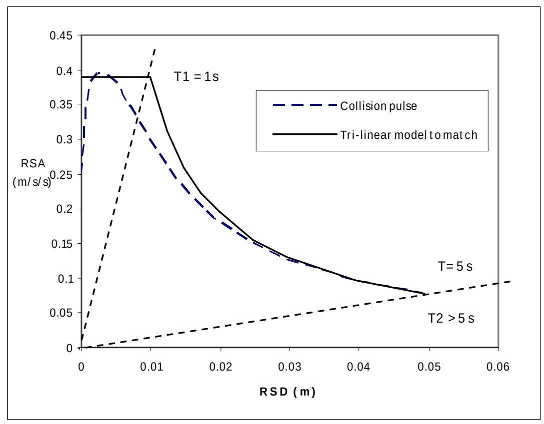




(a) Acceleration response spectrum

(b) Velocity response spectrum





(c) Displacement response spectrum

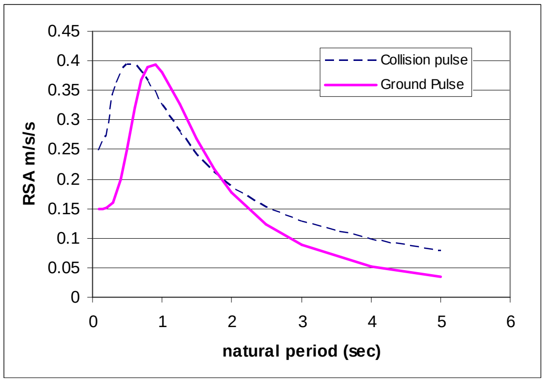
(d) Acceleration–Displacement Diagram

Figure 5 Response spectrum model of the arbitrary collision pulse

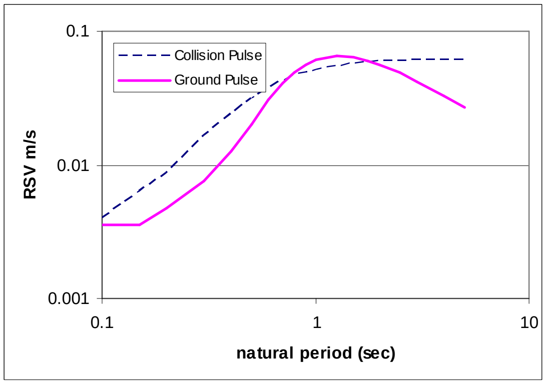
The overall appearance of the response spectra calculated from the collision pulse is generally similar to that of the ground pulse as shown in Figures 5a – 5d. Interestingly, the tri-linear model can be made to match with the response spectra for both types of pulses (despite the very different manner in which the pulses have been generated). The value of V_{max} is consistent with the value of the terminal velocity (60 – 65 mm/s) which can be calculated by integrating the pulse acceleration with respect to time (Figure 4b).

Comparison of the response spectra calculated for the ground pulse and the collision pulse reveals some interesting similarities (Figures 6a – 6d). However, the response spectra associated with the collision pulse does not have the displacement controlled segment. This is because the value of T_2 is equal to infinity or higher than the natural period range of interest (ie. $T_2 > 5$ s).

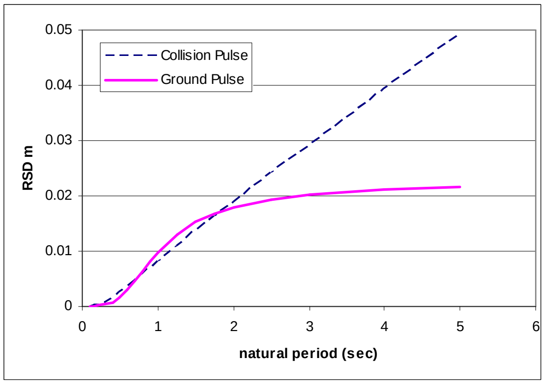
With both types of pulses, the hyperbolic segment representing velocity controlled conditions in the acceleration-displacement action diagram can be constructed by joining the apices of triangles of equal areas as shown in Figure 7. This feature of the action diagram can be explained by making reference to equation 3. It can be inferred from this equation that the area of a triangle representing elastic energy absorption (ie. ½ force x displacement = *M.RSA* x RSD) can be equated to the kinetic energy demand of the target structure that has been excited into motion (ie. ½ $M.V_{max}^2$). Thus, the hyperbolic segment of the action diagram can be constructed by drawing triangles of areas that are equated to the maximum kinetic energy delivered by the impact. The horizontal (flat) segment of the action diagram (Figure 7) represents another phenomenon to be explained later in the paper.



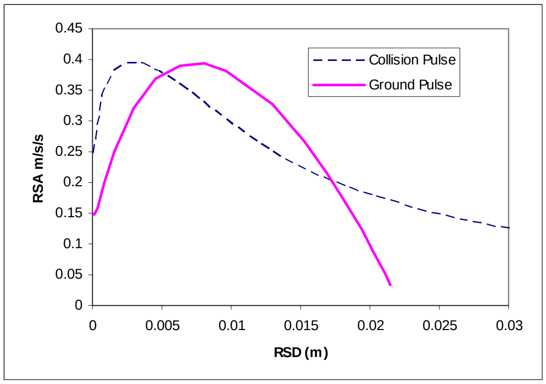
(a) Acceleration response spectrum



(b) Velocity response spectrum



(c) Displacement response spectrum



(d) Acceleration–Displacement Diagram

Figure 6 Response spectrum comparison of the ground and collision pulses

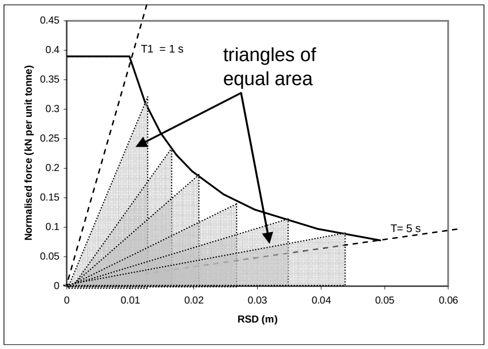


Figure 7 Basic construct of Acceleration-Displacement Action Diagram

The first corner period (T_1) which separates the flat and hyperbolic segments of the action diagram (as shown in Figure 7) is controlled by the duration of the collision pulse. Intuitively, the harder the impacting object, the shorter the pulse duration, the lower the value of T_1. In the extreme case of a very hard impacting object, T_1 tends to zero. In such a case, acceleration controlled conditions do not exist and the response spectrum is controlled fully by velocity. The acceleration (force) and the relative drift demand of the target structure can be determined by the graphical construction technique illustrated in Figure 7 once the value of V is known. The velocity developed in the target structure (V) is not to be confused with the velocity of the projectile (V_o) before it makes contact with the structure. The calculation of V from V_o is based on the principles of conservation of momentum as explained in Section 4.

4. BASIC MODEL FOR IMPACT ACTIONS (HARD PROJECTILES)

As a hard projectile of mass m and travelling at a velocity of V_o strikes a sdof lumped mass system (the "target") of Mass M the momentum transfer of the impact can be expressed as follows :

$$m(1+\upsilon)V_o = (m+M)V \quad \text{or} \quad V = \frac{m(1+\upsilon)V_o}{(m+M)} \tag{4a}$$

where ν (<1.0) is the coefficient of restitution which is dependent on the nature of the impact.

In situations where the mass of the projectile is an order of magnitude smaller than that of the target (and can be neglected) and there is no rebound of the projectile from the target, equation 4a can be simplified into equation 4b.

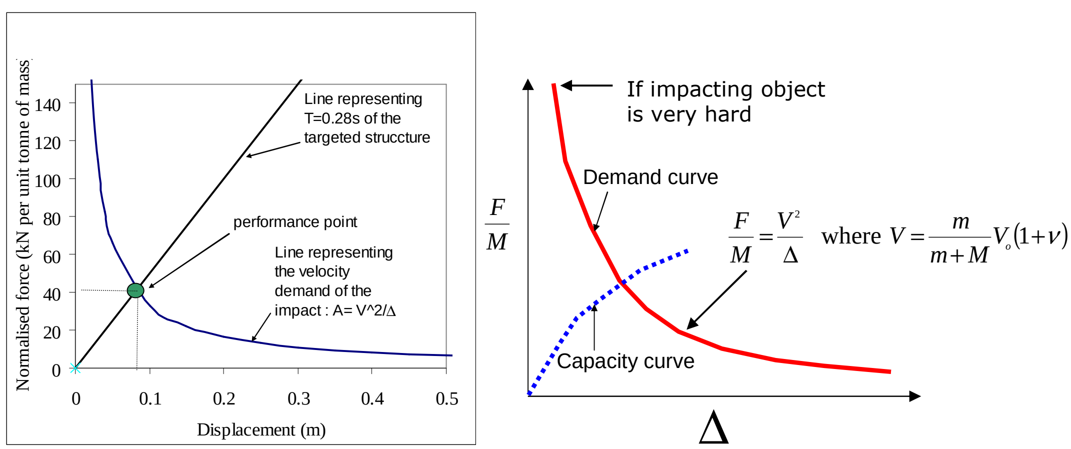
$$V = \frac{m}{M}.V_o \tag{4b}$$

Once the velocity of the target structure (V) has been calculated using equation 4a or 4b, the hyperbolic segment of the acceleration-displacement action diagram can be constructed using equation 3 or the graphical method illustrated in Figure 7. The normalized force – displacement (or acceleration – displacement) relationship of the target structure is then drawn to intercept with the constructed hyperbola.

The use of the Acceleration-Displacement Action Diagram is demonstrated herein with the example of a projectile (which has a mass (m) of 10 kg) impacting on a target structure, of 100 kg effective mass, and with an incipient velocity (V_o) of 10 m/s immediately prior to making contact with the structure. Principles of conservation of momentum are employed to estimate the response velocity of the target immediately following the impact. Assuming perfect rebound on impact (i.e. ν = 1.0) V is found to be equal to 1.8 m/s from equation 4a. Equation 3 could then be used to construct the hyperbolic segment of the action diagram as shown in Figure 8a. In this example, linear elastic behaviour is assumed of the target structure which possesses a natural period of T = 0.28 s. The (linear) capacity function representing the response behaviour of the structure, which has a gradient (A/Δ) equal to $(2\pi/T)^2$ intercepts with the hyperbolic (demand) function at an acceleration (A) of approximately 40 m/sec^2 and a displacement (Δ) of 0.08 m. The reaction force is accordingly estimated to be about 4000 N (= 40 m/sec^2 x 100 kg).

Whilst the use of the action diagram will provide accurate results for linearly elastic sdof lumped mass systems, the technique can be extended to cases of non-linear force-displacement relationships as shown in Figure 8b. The use of the action diagram for solving systems experiencing inelastic response behaviour based on linearisation is widely practiced in earthquake engineering.

The important assumption with this method of calculation is that the impacting object is non-deformable in order that the transfer of momentum from the projectile to the target structure occurs instantaneously. The use of the action diagram based on this assumption would give a conservative estimate of the impact action, when in reality a finite amount of time would be required for the transfer of momentum to take place from the (deformable) impacting object to the target structure (which is also deformable). If the effects of the softness of the projectile is to be taken into account, two-degree-of-freedom modelling has to be adopted, as discussed in Section 5.



(a) Linear elastic

(b) Inelastic (schematic)

Figure 8 Acceleration-Displacement Action Diagram for hard impact

5. GENERALISED MODEL FOR IMPACT ACTIONS (HARD AND SOFT PROJECTILES)

A schematic representation of the two-degree-of-freedom (2DOF) system model is shown in Figure 9. With the 2DOF model, the impacting object and the target structure is each represented by a single-degree-of-freedom system which is characterised by mass, m and M, and spring stiffness, k and K, respectively. The stiffness k associated with the impacting object is to model the deformation of the object as well as the indentation of the object into the surface of the target structure. The value of k can be calculated by dividing the contact force by the displacement of the centre of mass of the impacting object. Thus, a lower value of k refers to a softer impacting object. The natural period of the object (T_m) which controls the duration of application of the contact force can be estimated using equation 5 which is to be read in conjunction with Figure 9.

$$T_m = 2\pi\sqrt{\frac{m}{k}} \quad or \quad 2\pi\sqrt{\frac{m}{F_{contact}/\delta_m}} \tag{5}$$

It is evident in equation 5 that the softer the impacting object (the lower the value of k) the longer the duration of contact and hence the longer the delay in the full transfer of momentum from the impacting object onto the target structure. Details of the computational algorithm for implementation of the 2DOF analysis on EXCEL can be found in Lam & Tsang (2008).

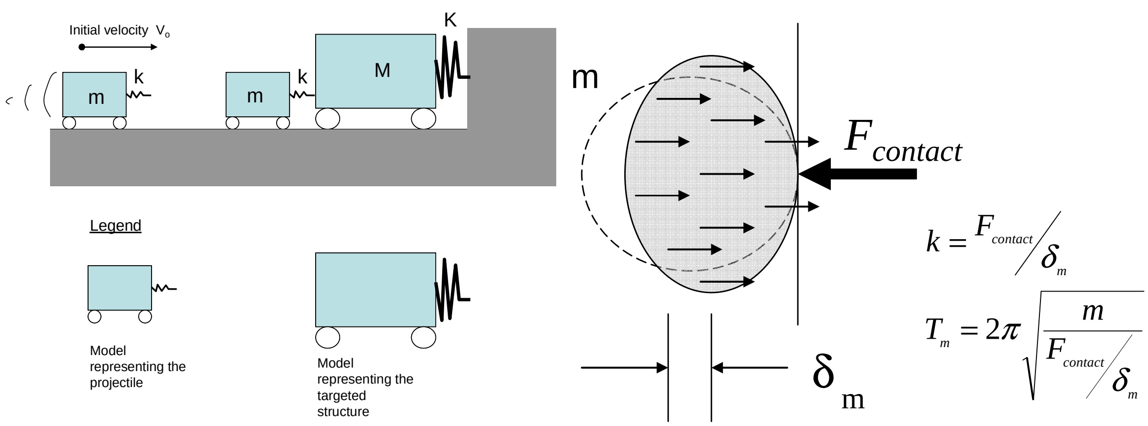
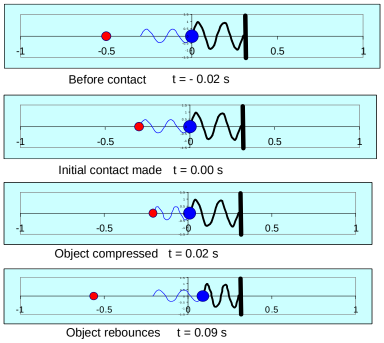


Figure 9. Two-degree-of-freedom model.

Analyses based on the example cited in Section 4 have been undertaken. As before, the m = 10 kg object is considered to impact the M = 100 kg target with a velocity (V_o) of 10 m/s. Modal damping of the system is assumed to be 0.5 %. Simulations of impact using the 2DOF model can be used to demonstrate the important phenomenon of delayed response of the target which is demonstrated in Figure 10. The simulation was based on impact parameters: k = 25,000 N/m (which is translated into T_m = $2\pi\sqrt{m/k} = 2\pi\sqrt{10/25000} = 0.12\,s$). The value of T_m is significantly lower than the natural period of the target structure (T = 0.28 s). At the instance when maximum contact force (of approximately 10000 N as shown in Figure 11) is developed on impact, only a very small reaction force is initially developed in the support to the target structure as shown in Figure 10 (at t = 0.02s). The compression of the spring which is connected to the structure occurs much later (at 0.09s). A maximum reaction force (of approximately 4000N) was eventually developed at 0.1s. The time-histories of the development of the contact force and reaction force are shown in Figure 11. The time-history trace of the contact force is sinusoidal in form and is consistent with that shown in Figure 4a. The much attenuated reaction force of the target and the delayed response is clearly shown.



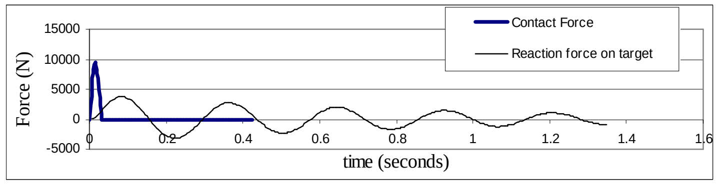
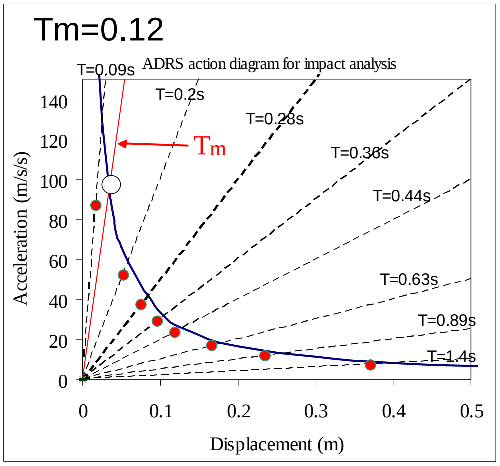


Figure 10 Simulation of impact

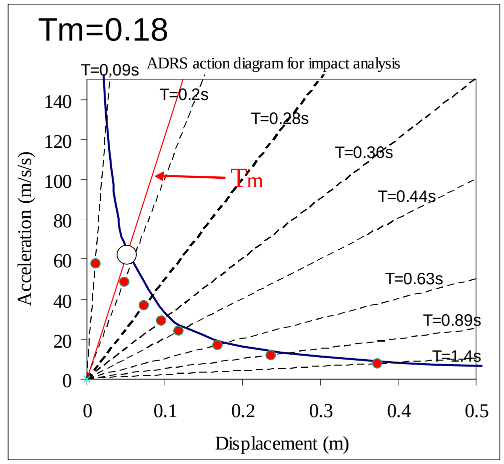
Figure 11 Time-histories of generated forces

A large number of analyses similar to the one shown above were undertaken with varying properties of both the projectile and the target structure. The maximum reaction force and the displacement of the target was then recorded from each of the analyses for comparison with results obtained from the simplified method introduced in Section 4. Both sets of results are presented in Figure 12 with different legends. The solid line represents estimates from the simplified method whereas the solid circular symbols represent estimates from the 2dof analyses. Good agreement between the two sets of results are generally observed with cases where the natural period of the structure was significantly higher than that of the impacting projectile (when $T >> T_m$). However, predictions by the simplified method is shown to be very conservative when the target structure is of a lower natural period (when T < T_m). The conservatism can be explained by the significant delays in the momentum transfer and the effects of such delay could not be accounted for by the simplified method. It is observed from each action diagram of Figure 12 that the reaction forces calculated from the 2dof analyses cannot exceed a certain limit. This upper limit is observed to be very close to the point where the radial line associated with T_m intercepts with the hyperbola (refer hollow circular symbol in the figure). Clearly, the acceleration level of this upper limit decreases with increasing value of $T_{m.}$ A generalised model for impact actions is proposed herein based on these observations.

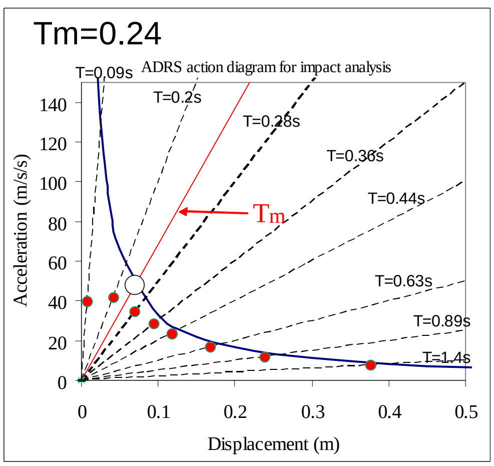
It is shown in the generalised model (Figure 13) that estimates of the force and displacement demand of the target structure be based on the intercept of the capacity line with the hyperbola defined by equation 3 when $T > T_m$. With lower values of T, the hyperbolic segment is replaced by a flat segment which has $F = F_{peak}$ where $\mathrm{F_{peak}}$ is obtained by intercepting the radial line for T_m with the hyperbola. Results from analyses of the 2dof models could be presented in terms of the acceleration (normalized force) demand as shown in Figure 14a. It is shown clearly in the figure that the reaction force of the target structure is very sensitive to the value of T_m. The response spectra are shown to have very consistent characteristics when the response spectral ordinates have been made dimensionless as shown in Figure 14b. In the dimensionless presentation, the force ratio is the reaction force of the target structure divided by the contact force and the natural period ratio is T divided by T_m . An interesting, and significant, observation to make of Figure 14b is that it is not dissimilar to a response spectrum model for earthquake loading.



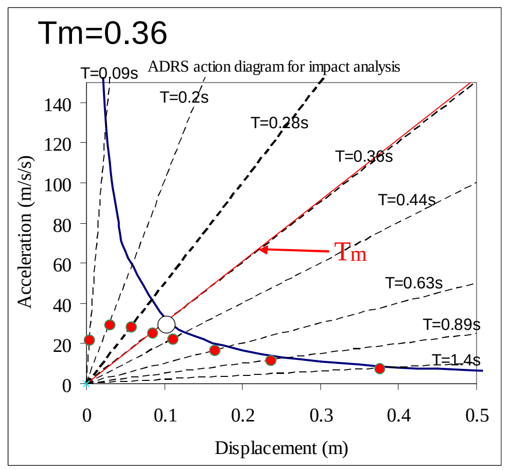
(a) $T_m = 0.12s$



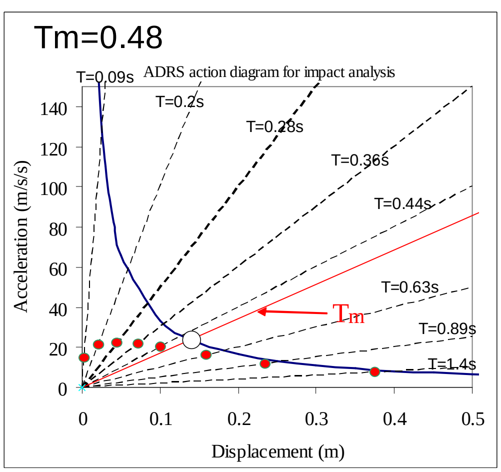
(b) $T_m = 0.18s$



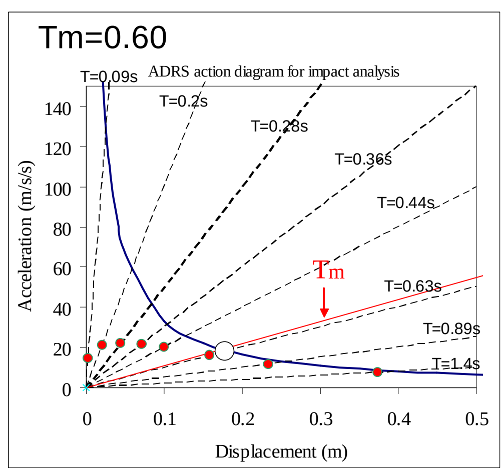
(b) $T_m = 0.24s$



(c) $T_m = 0.36s$



(c) $T_m = 0.48s$



(d) $T_m = 0.60s$

Figure 12 Comparison of results from 2DOF model and Acceleration-Displacement Action Diagram
(m=10 kg M=100kg V_o=10m/sec ν=1.0)

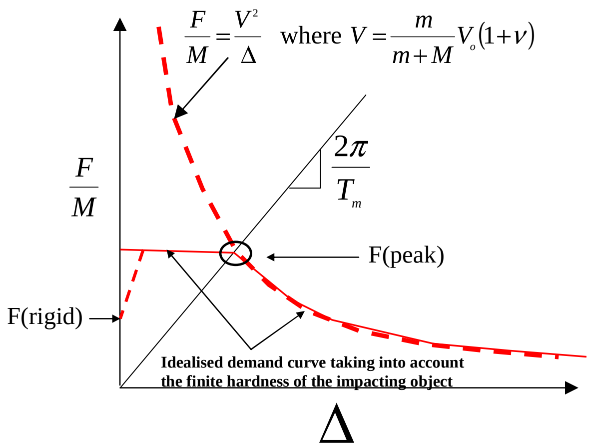
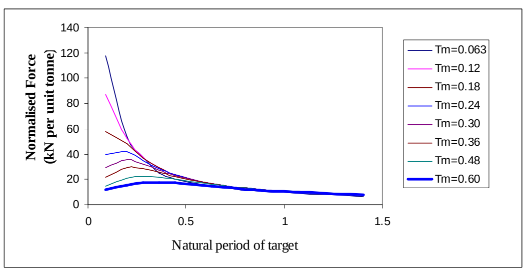


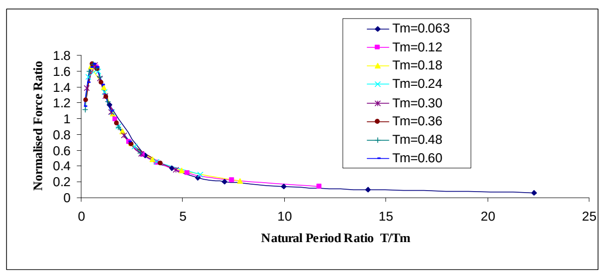
Figure 13 Action diagram shown the generalised model for impact

Results could also be presented in terms of the displacement demand as opposed to the acceleration (force) demand. Such information could be presented in the very compact format of a displacement response spectrum which shows the displacement demand behaviour associated with different values of T_m (refer Fig. 15). The displacement demand of hard objects (which are characterised by low values of T_m) pertains to the linear relationship of equation 5 which provides a conservative benchmark for estimating the displacement demand of the target structure.

$$\Delta = V.\frac{T}{2\pi} \tag{5}$$



(a) original form of spectrum



(b) response spectrum in dimensionless form

Figure 14 Acceleration response spectra from 2DOF analysis

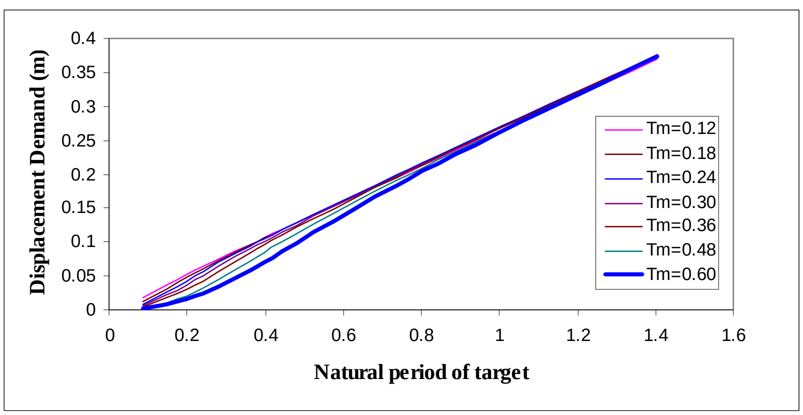


Figure 15 Displacement demand of target from 2DOF analysis

6. SUMMARY AND CLOSING REMARKS

A proposed generalized model for impact actions has been presented along with the newly implemented model for earthquake actions of AS1170.4 (2007). The two models are summarized in Table 2 which is to be read in conjunction with the schematic diagram of Figure 16.

The generalized model enables the reaction force of a lumped mass sdof system to be estimated reasonably accurately without the need to resort to the use of a specialized software. The generalized model can be applied readily to situations of an impacting object striking the edge of a slab or the soffit of a bridge deck. Solutions can be conveniently obtained for target structures which weigh much less than the projectile. In situations where an object is striking a beam, a column, or a canopy, the effective mass surrounding the point of contact has to be determined. It is common practice to take the effective mass of impact to be equal to the effective modal mass of the fundamental mode of vibration of the element (which can be a beam or a slab). For example, the effective mass of a simply supported reinforced concrete beam has been assumed to be 4/5 of the total mass of the beam in the expression proposed by Simms (1945) for the calculation of the energy absorption of the beam expressed as a fraction of the kinetic energy of the impacting object. Methods of calculating the effective modal mass of a member, or a structure, is explained in standard structural dynamics text books. It is cautioned herein that neglecting the contributions to higher modes of vibration can result in significant modelling errors. Examples of illustration of this can be found in Lam *et al.* (2008).

Table 2 Summary of unified model for earthquake and impact actions

Response parameters	Model for seismic actions AS1170.4 (2007)	Generalised model for impact (this paper)
A_{max} (g's)	$3.Z.F_a$	$V\left(\frac{2\pi}{T_m}\right)\Big/9.81$
V_{max} (mm/s)	$1.8\ (750.Z.)\ F_v$	$\frac{m}{m+M}(1+\nu)V_o$ or V
D_{max} (mm)	$V_{max.}\ (1.5/2\pi)$	$V.(T/2\pi)$
T_1 (s) T_2 (s)	$0.2s$ - $0.6s$ $1.5s$	T_m infinity

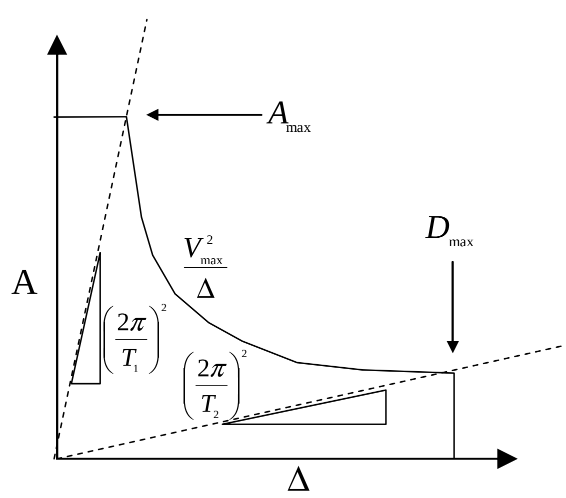


Figure 16 Unified model for earthquake and impact actions

The unified model presented in this paper is a viable starting point for civil engineers to broaden their perspective on how structures respond to dynamic actions that are associated with hazards such as earthquakes, explosions and other forms of impact loading, by considering the acceleration, velocity and displacement system demands. Design criteria and values of factors to be used in design could be developed using this model.

7. ACKNOWLEDGEMENTS

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8. REFERENCES

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