Journal of Sound and Vibration (2002) **254**(1), 51–68
doi:10.1006/jsvi.2001.4083, available online at http://www.idealibrary.com on IDEAL®

LOCAL-MODE RESONANCE AND ITS STRUCTURAL EFFECTS UNDER HORIZONTAL GROUND SHOCK EXCITATIONS

Y. Lu

School of Civil and Structural Engineering, Nanyang Technological University, Nanyang Avenue, Singapore 639798, Singapore. E-mail: cylu@ntu.edu.sg

H. Hao

Department of Civil and Resource Engineering, University of Western Australia, Nedlands, WA 6009, Australia. E-mail: hao@civil.uwa.edu.au

G. Ma

Centre for Composite Material, University of Delaware, Newark, DE 19716, U.S.A. E-mail: mag@ccm.udel.edu

AND

Y. Zhou

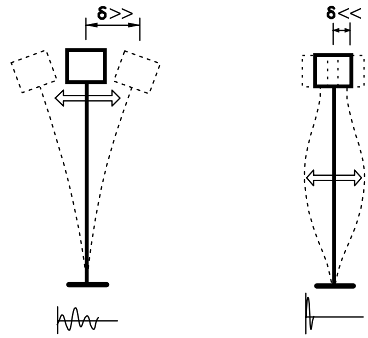
Defence Science & Technology Agency, Ministry of Defence, 1 Depot Road, Singapore 109679, Singapore

(*Received* 4 *May* 2001, *and in final form* 25 *September* 2001)

The dynamic response of building structures has been studied extensively for relatively low-frequency seismic actions, and it is established that the seismic response generally is governed by the global-mode vibration, i.e., the vibration in terms of the floor movement. Much less fundamental study has been done regarding the structural response to ground shock excitations with principal frequencies many times of the fundamental frequency of the structural system. Most of the existing code provisions on ground shock control have remained empirical. In this paper, it is demonstrated through numerical study and laboratory model testing that the structural response to high-frequency ground shocks have distinctive characteristics as compared to the seismic response, and most significant is the participation of the vibration at the local elemental level. Local-mode resonance could occur when the shock frequency is sufficiently high, and to a large extent it can be uncoupled from the global floor vibration. As a result, large force effects can develop at relatively small floor displacement, rendering the conventional displacement-based criteria inapplicable, while more focus on the stress–strain response is deemed necessary. The results pave a way for further development of more rational criteria for this category of the structural vibration problems.

1. INTRODUCTION

The dynamic response of structures depends on the frequency characteristics of the excitations. Over the years, the study of building response under damaging ground vibration conditions has been earthquake-oriented because of the great economic and social impact a major earthquake can produce. Another reason behind this concerns the nature of vibrations: earthquake excitations occur at relatively low frequencies, and



(a) Global-mode response (b) Local-mode response

Figure 1. Global- and local-mode controlled response scenarios.

low-frequency excitations have greater energy than high-frequency excitations, posing greater destructive effects.

While the above notion is true in a general sense, high-frequency ground motions (referred to as "ground shocks" hereinafter) of potentially damaging magnitude can occur as a result of construction blasting; piling; mining; and at the severest, of accidental detonation of surface or underground ammunition storage magazines. Typical construction vibrations can be of principal frequency around 30 Hz [1]; intense explosion-induced ground shocks have even higher frequency (50 Hz and above) in the near field, and close-in can present very high peak acceleration, as will be shown later in the case study. Despite short duration (often counted in milliseconds, hence of relatively small energy as compared to seismic motions) and rapid attenuation due to high-frequency contents [2], such ground shocks have the potential to cause damage to nearby structures due to high shock peak and localized effects.

Contrary to the low-frequency earthquake ground motions, where the severity is often well represented by the peak ground acceleration (PGA), field investigations have indicated that the effects of the ground shocks on structures are related less to the PGA but more closely to the peak particle velocity (PPV) [1–5]. Accordingly, most of the existing regulatory guides on the ground shock control concerning structural effects have been PPV based [6, 7] and they have evolved from empirical correlation between observed damage and recorded peak particle velocities during field tests. However, the exact response phenomenon and the underlying mechanisms for this special category of structural dynamic problems remain unclear.

In fact, with the high-frequency nature of the shock excitations, it is conceivable that the local-mode vibration could play a significant role in the structural response. Such a scenario and the likely effects can be understood by performing a simple experiment as follows. Use a flexible plastic ruler to represent the vertical element of a structure, and affix on top a rubber eraser to represent the heavy concentric floor mass, as shown in Figure 1. First, hold the bottom end of the ruler, align the "structure" vertically, and shake the base with a slow motion at a frequency around the fundamental frequency of the system to mimic an earthquake. The response will be seen of global nature with dominating movement (displacement) of the top concentrated mass, as depicted in Figure 1(a). Next, hold the specimen steadily and apply a brief punch to the base, analogous to a ground shock. What can be observed is a very different scenario, whereby the top mass remains almost motionless while the ruler body vibrates significantly on its own mode, as shown in Figure 1(b).

The dynamics background of the above experiment is elementary, but the phenomenological implications are significant from engineering perspective and they may be interpreted as follows. While the structural response to low-frequency excitations is dominated by the global vibration with large floor movements, the high-frequency shock response can involve significant local vibration due to the elemental mode resonance. Furthermore, as a result of the presence of the heavy floor mass, the local-mode vibration could be largely uncoupled from the global floor response, and hence differs fundamentally from the concept of higher global modes in case of multi-storey systems. Under this circumstance, an evaluation of the structural effects based only upon the consideration of the global-mode vibration, which is commonly adopted in the seismic analyses, could be misleading.

The dynamics of a simple global–local system like that shown in Figure 1 may be expressed in mathematical terms. However, a closed-form solution to the combined global and local vibration problem under arbitrary ground shock excitations will not be straightforward. In this paper, a comprehensive evaluation of the high-frequency response, with particular emphasise on the local-mode resonance effects, is carried out by means of numerical analyses in conjunction with the laboratory model testing. A pattern of the local-mode resonance phenomenon is established, based on which the underlying mechanisms are discussed. The unique model-testing scheme is recommended for future laboratory investigation of the high-frequency ground shock response problems.

2. NUMERICAL STUDY

2.1. ANALYTICAL MODEL AND VIBRATION MODES

In view of the fact that the basic difference in structural response to high- and low-frequency excitations is the degree to which the global (roof and floors) and local (elemental) modes are excited, a conceptual single-storey reinforced concrete (RC) frame, shown in Figure 2(a), is deemed appropriate for a pattern evaluation. The RC material properties are indicated in the figure (notation of symbols can be found in section 4).

The numerical representation of the system is shown in Figure 2(b), in which the column member is divided into several sub-elements with the actual mass of each column segment being lumped at the respective intermediate node points. It should be particularly pointed out that the importance of such distributed mass scheme in modelling high-frequency response is due to the effects of the elemental vibration. The concentrated mass on top of the column (3000 kg) approximates the reactive mass of the floor area that may be allocated to the particular column. The rotational freedom at the top of the model is restrained to represent a typical shear-type frame. For simplicity, a 5% damping coefficient is adopted for all modes involved in the analysis. Given the material properties shown in Figure 2(a), the system is computed to have the fundamental global-mode frequency $f_s = 5{\cdot}58$ Hz, and the first three local (column) mode frequencies $f_{c1} = 79{\cdot}0$ Hz, $f_{c2} = 205{\cdot}2$ Hz and $f_{c3} = 376{\cdot}8$ Hz respectively. Figure 2(c) shows the corresponding mode shapes.

For the purpose of demonstrating the response characteristics, a linear elastic analysis is sufficient. However, in order to facilitate later discussion on the likely damage mode, the RC column member is estimated, following a standard cross-section analysis, to have a yield moment resistance of 112 kN m. The cracking shear strength, on the other hand, is estimated to have an upper-bound limit as specified by ACI [8]: $V_c = v_c bd = (0{\cdot}3\sqrt{f_c'})bd = 110$ kN, where v_c denotes the concrete shear strength, f_c' is the concrete compressive strength, b and d are the width and effective depth of the cross-section respectively.

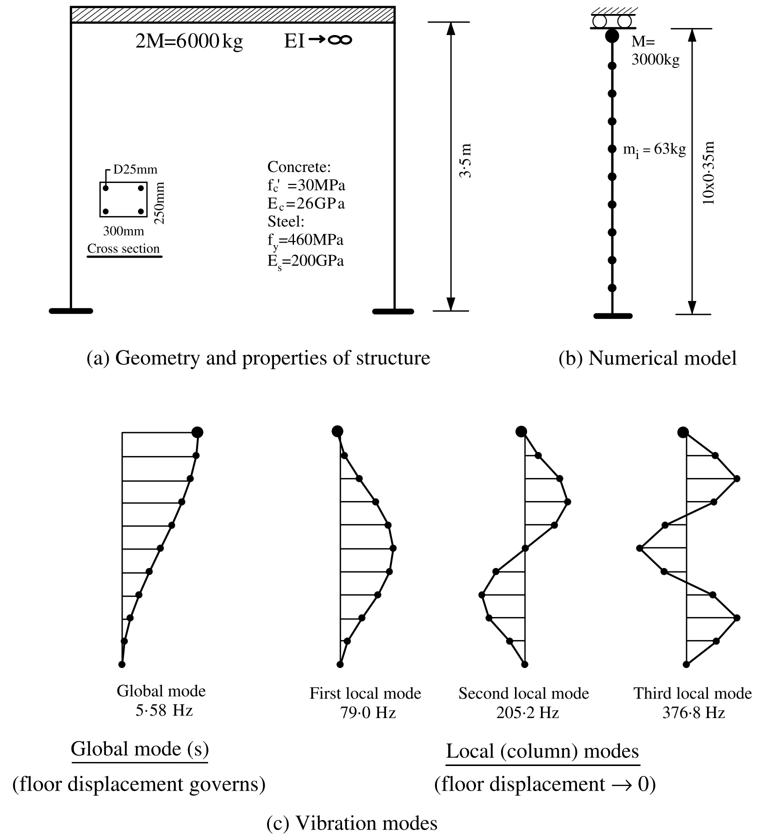


Figure 2. Simple frame model and its global and first three local vibration modes.

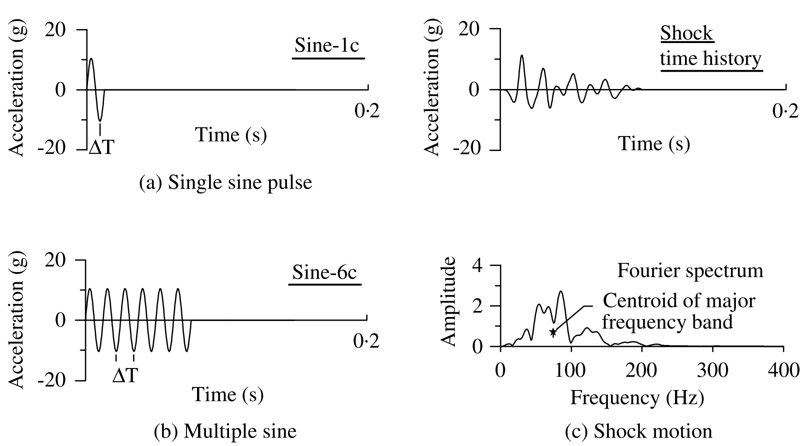


Figure 3. Three types of ground motions considered in the analysis.

2.2. INPUT MOTIONS

Three types of input motion waveforms (Figure 3) are considered to represent a variety of ground shock excitations. Namely, (1) a one-cycle sine pulse, designated as *Sine*-1*c*;

TABLE 1

Control parameters of input motions used in parametric analysis

Waveform	Principal frequency (Hz)	PGA (g)	PPV (m/s)
Shock	200	32·6	
	75	12·3	
	60	10·0	
	38	6·2	
	25	4·1	
	6·5	1·2	
			0·23
Sine-1*c* and *and* *Sine*-6*c*	200	27·6	
	80	11·5	
	50	7·2	
	25	3·6	
	10	1·4	
	5	0·72	

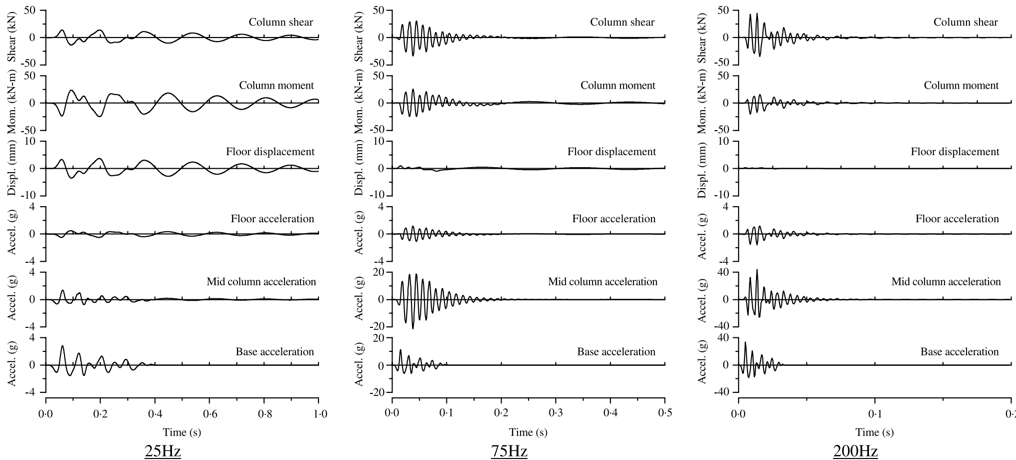
(2) a multiple-cycle sine wave (six cycles), designated as *Sine*-6*c*; and (3) a typical blasting ground shock motion [9], designated as *Shock*. The characteristic frequency for the sine inputs is taken to be $1/\Delta T$ with ΔT being the duration of a single cycle. The prototype *shock* motion has a characteristic frequency of around 75 Hz (Figure 3(c)).

In the parametric analysis, all these input motions are varied by scaling the time axis to achieve the desired principal frequencies while retaining the motion waveform. The acceleration of the motions is simultaneously scaled by the inverse of the time scale factor so that the peak particle velocity (PPV) remains constant at 0·23 m/s for all analysis runs. Table 1 summarizes the control parameters of the input motions considered in the analysis.

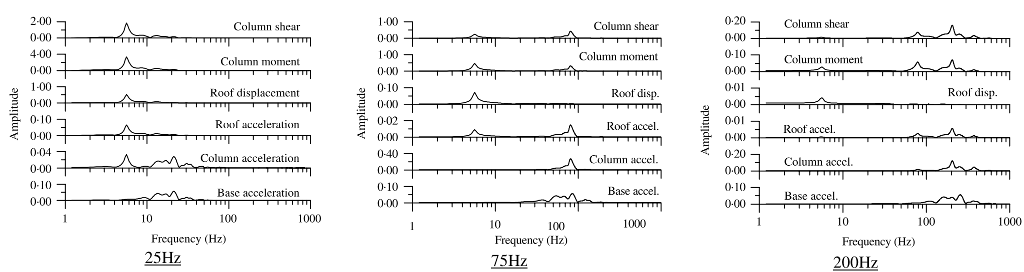
2.3. OUTLINE OF COMPUTED RESPONSES

The following key response parameters are examined: (1) the displacement and acceleration at the floor level (representing global response); (2) the acceleration at mid-height of the column (representing local vibration); and (3) the maximum column shear and moment (representing the structural effects). Figure 4 shows the response time histories and the corresponding Fourier spectra for *Shock* motions with the principal frequencies (f_{in}) being equal to 25 Hz ($\approx 5f_s$), 75 Hz ($\approx f_{c1}$) and 200 Hz ($\approx f_{c2}$) respectively. The primary response characteristics to the *Sine*-1*c* and *Sine*-6*c* excitations at comparable frequencies are similar to the *Shock* response, thus they are not presented in detail.

As can be seen, for excitations of principal frequency about $5f_{in}$ which also accounts for 35% f_{c1}, all the response quantities remain to be dominated by the global-mode response at $f_s = 5{\cdot}58$ Hz, i.e., the response is closely related to the movement of the concentrated floor mass. As such, the ratio between the maximum column moment and shear force is maintained approximately $0{\cdot}5H$ (H being the height of the column), as is expected for the global-mode response of the system.



(a) Time histories



(b) Fourier amplitude spectra

Figure 4. Computed response for *Shock* ground motion with principal frequencies of about 25, 75 and 200 Hz respectively (PPV $\equiv$ 0·23 m/s).

However, with the principal frequency of the ground motion increased to approach the column mode frequency, all response parameters except the floor displacement exhibit overwhelming high-frequency components. The Fourier spectra indicate that these high-frequency components can be attributed primarily to the local-mode resonance since the predominant spectral peak coincides precisely with the column first-mode frequency $f_{c1} = 79$ Hz. It is noted that the local-mode vibration affects the acceleration of the concentrated floor mass as well, and this in turn contributes to the force effects in the column member. As a combined output, the column shear force increases considerably even though the floor displacement drops to about 30% of that under 25-Hz excitation. The column moment, on the other hand, tends to become relatively less significant, such that the moment-to-shear ratio decreases from the normal 0·5H to approximately 0·2H. The above local-mode response features become even more pronounced when the principal input frequency approaches 200 Hz for which the second local-mode resonance is also excited.

The local-mode vibration contribution to the structural response can be visualized by the instantaneous distributions of responses along the column height. Figure 5 shows the typical response profiles at the time when the maximum column shear force occurs for the *Shock* motion with the principal frequency at three distinctive levels respectively. The displacement profiles that were conceptually illustrated in Figure 1 are very well resembled with the current frame system. As the local-mode resonance becomes significant, the shear force and moment distributions along the column height are also significantly altered.

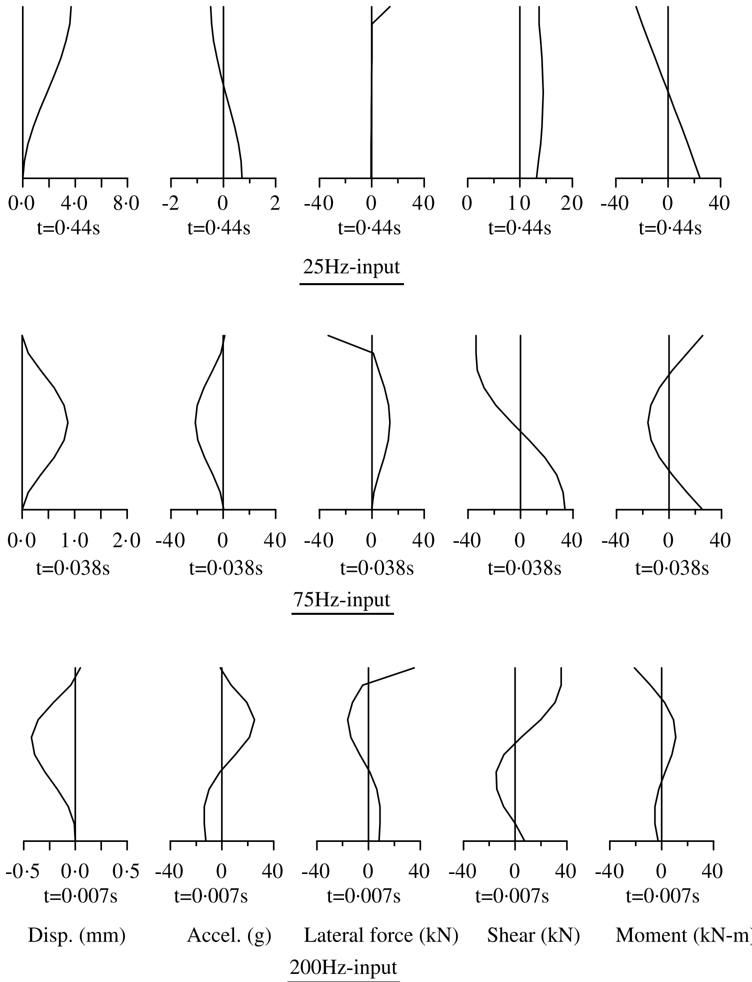


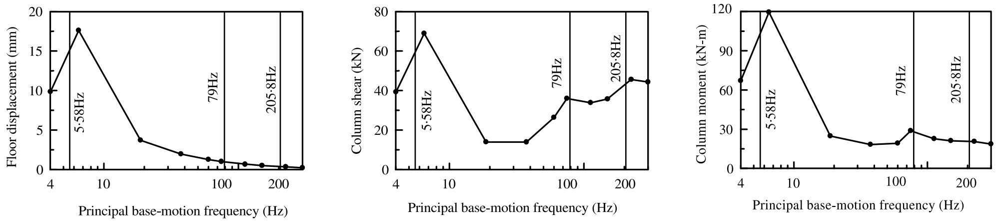
Figure 5. Response profiles at the time of maximum column (shear) force response.

2.4. VARIATION OF RESPONSE AMPLITUDES WITH INPUT FREQUENCY

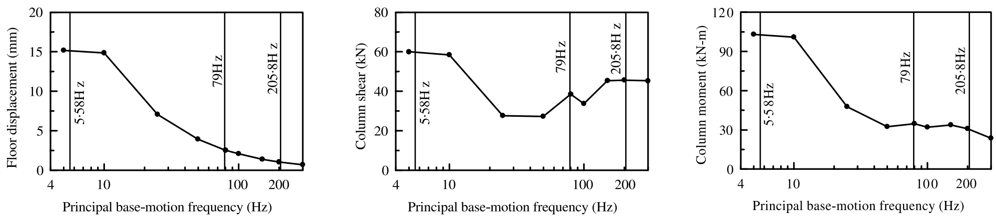
Having established the trend of shifting from global- to local-mode dominance in the structural response to the ground excitations with increasing frequency, it is necessary to examine the change of the response amplitudes so that the inherent structural effects can be better understood. Moreover, a threshold frequency that marks significant local vibration effects may also be identified. Figure 6 shows the variation curves of the maximum floor displacement, column shear force and moment under the three types of ground motions respectively. Note that the PPV is kept constant for all excitations to allow for comparison.

As can be seen, each respective response variation curve shows similar trend regardless of the specific ground motion waveform. With respect to the *Shock* response, the following may be observed:

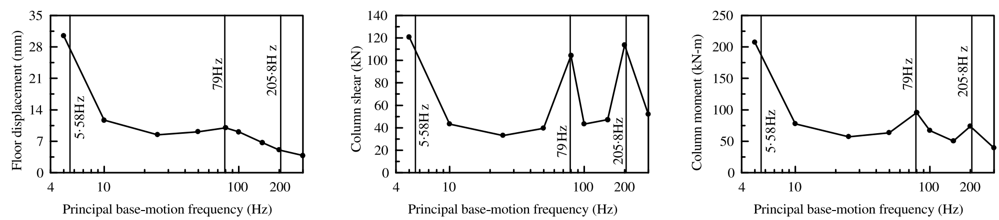
(1) The floor displacement, which represents the global-mode response, decreases progressively with increase of the base-motion frequency. At 75 Hz ($13f_s$ or approximately



(a) *Shock*: Floor displacement, column shear and column moment



(b) *Sine*-1*c*: Floor displacement, column shear and column moment



(c) *Sine*-6*c*: Floor displacement; column shear and column moment

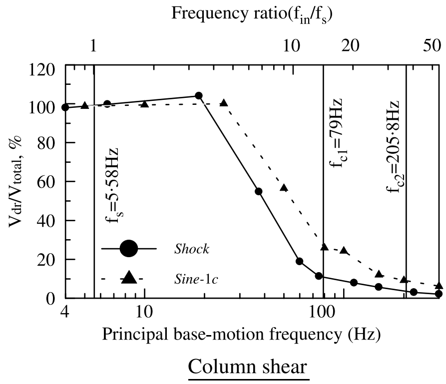
Figure 6. Variation of response amplitudes with principal base-motion frequency (PPV $\equiv$ 0·23 m/s).

$1{\cdot}0 f_{c1}$) level, the displacement drops to about 15% of that at near-global resonance. Further increase of the input frequency renders the floor displacement to virtually vanish.

(2) Contrary to the above, the maximum column shear force increases steadily towards the local-mode resonance after a "valley" next to the global resonance. The column moment appears to be less affected by the local-mode vibration though, and as a result, the shear-to-moment ratio increases with the increase of the input frequency. This signifies an increased risk for a brittle shear failure under high-frequency excitations.

(3) There exists an apparent transit frequency range, which in this case is between $4f_s$ and $0{\cdot}6 f_{c1}$, whereby the effects of the ground excitations are generally low because of the departure from the global resonance, while the local-mode vibration is yet to become significant. In fact, a good portion of construction vibrations may fall within this range [2], and this perhaps has contributed to the general adoption of increased allowable PPV for higher-frequency excitations. As is shown here, however, such treatment should be exercised with caution when the excitation frequency approaches the predominant local-mode resonance, because of the increased (shear) force effect. The current results indicate that an input frequency about 50–60% of the lowest column mode frequency may be set as a threshold for considering the local-mode resonance effects under horizontal ground excitations.

The apparent disassociation of the force effects from the floor displacement is seen to be as one of the most important features of the local-mode dominated response under



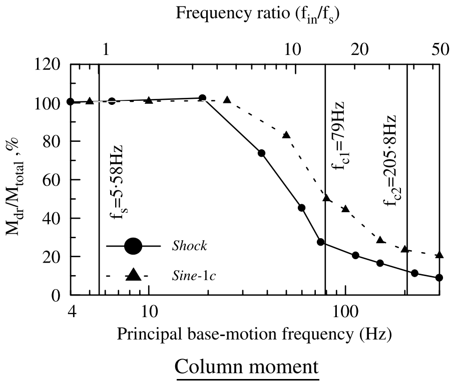


Figure 7. Percentage contribution of global response in column force effects.

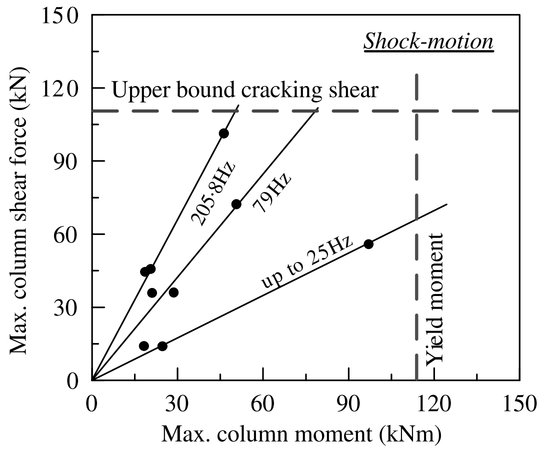


Figure 8. Relationship between maximum column shear and moment for base-motions of varying frequencies.

high-frequency excitations. For the simple frame model herein, the maximum shear and moment effects that may be attributed to the floor displacement, V_{dr} and M_{dr}, respectively, can be estimated from the maximum floor drift, δ, under the given boundary conditions as follows:

$$V_{dr} = \frac{12E_cI\delta}{H^3} \qquad M_{dr} = \tfrac{1}{2}V_{dr}H, \tag{1,2}$$

where I is the moment of inertia of the column cross-section, H is the storey height.

Figure 7 shows the variation of the percentage contribution of the above results in the total shear (V_{total}) and moment (M_{total}) effects, with the principal base-motion frequency. As can be seen, up to a frequency of 3–4 times that of the global-mode frequency, the column force effects can be attributed almost entirely to the floor displacement. With the local-mode vibration coming into picture, the floor-displacement contribution in the force effects decreases abruptly and becomes almost negligible during high-frequency excitations.

With regard to the tendency for premature shear failure to occur under high-frequency vibrations, Figure 8 shows the relationship between the maximum shear force and moment in the column member with respect to the corresponding strengths. It can be clearly observed that, as the local-mode vibration intensifies with increase of the ground-motion frequency, the path of the force development become increasingly prone to shear failure.

2.5. MULTI-STOREY SYSTEMS

Another important observation from the above single-storey structure analysis is that the acceleration response at the floor level decreases drastically as compared to the column acceleration during high-frequency horizontal ground motions. A similar phenomenon was also noticed in some previous investigations employing the wave propagation approach [10], and was attributed to the large wave reflection coefficient at the concentrated floor mass, resulting in a low-pass filter effect on the waves crossing the floor. From the dynamic response viewpoint, this can be explained as a result of the global-mode vibration being less excited under high-frequency ground motions. To further explore this phenomenon, a two-storey frame is analyzed with a simplified model shown in Figure 9(a). The frame's two global-mode shapes are depicted in Figure 9(b). The local modes for each individual column member are similar to that of the single-storey frame model shown in Figure 2.

Concerning the high-frequency response, Figure 10(a) shows the computed column moment and shear force time histories for the first and second storey, respectively, under the 75-Hz *Shock* excitation. The various response profiles at the time of maximum column shear force are plotted in Figure 10(b).

The "blocking" effects of the heavy floor mass on the propagation of high-frequency vibration can be readily observed from the sharp contrast between the first- and second-storey responses. It can also be observed that the response within the first-storey columns matches closely that of the single-storey system having the same column properties (compare Figure 4(a), 75 Hz). Although the extent of such floor "blocking" effects on high-frequency response may vary with the amount of the concentrated floor mass as well as the global and local stiffness characteristics, the general trends are expected to hold. Therefore, it may be concluded that (1) the local-mode resonance phenomenon can take

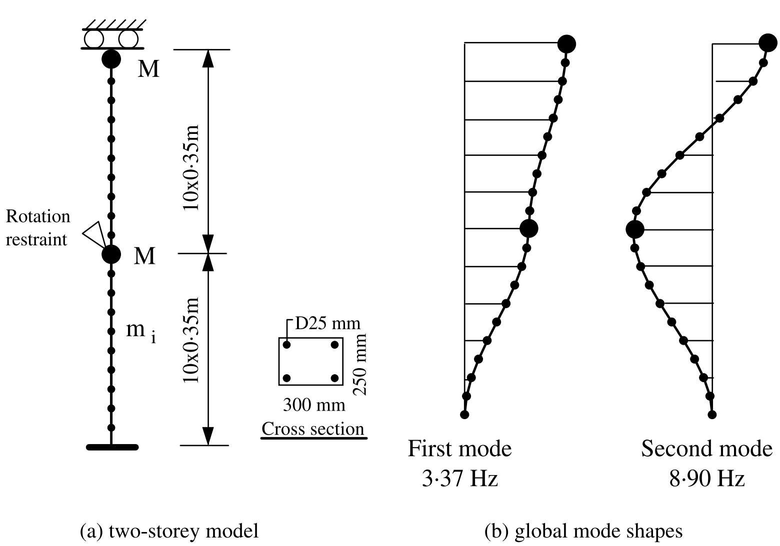


Figure 9. Two-storey frame model and global-mode shapes.

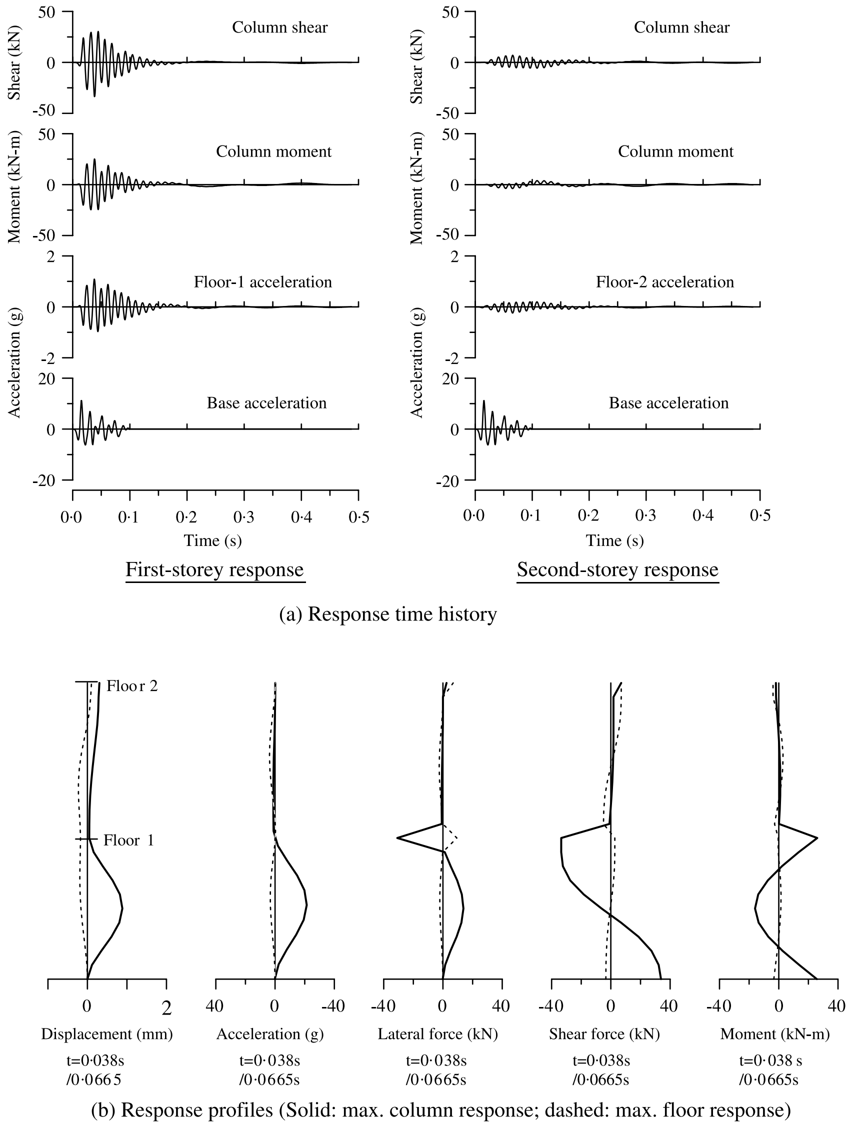


Figure 10. Computed responses of two-storey model under 75-Hz *Shock* motion.

place in single as well as multi-storey structures; (2) the critical impact of the local-mode resonance is likely to concentrate at the lowest storey(s) concerning horizontal ground shocks.

3. SEVERITY OF LOCAL-MODE VIBRATION PROBLEM IN PRACTICE

According to the analysis described above, the significance of the local-mode vibration in actual structures under horizontal ground shocks depends upon the likelihood that the natural frequencies of the (column) members fall into the predominant frequency range of the input shock motions. For illustrative purpose, take for example a typical 5-m long RC column with boundary conditions similar to that shown in Figure 2. Its first-mode

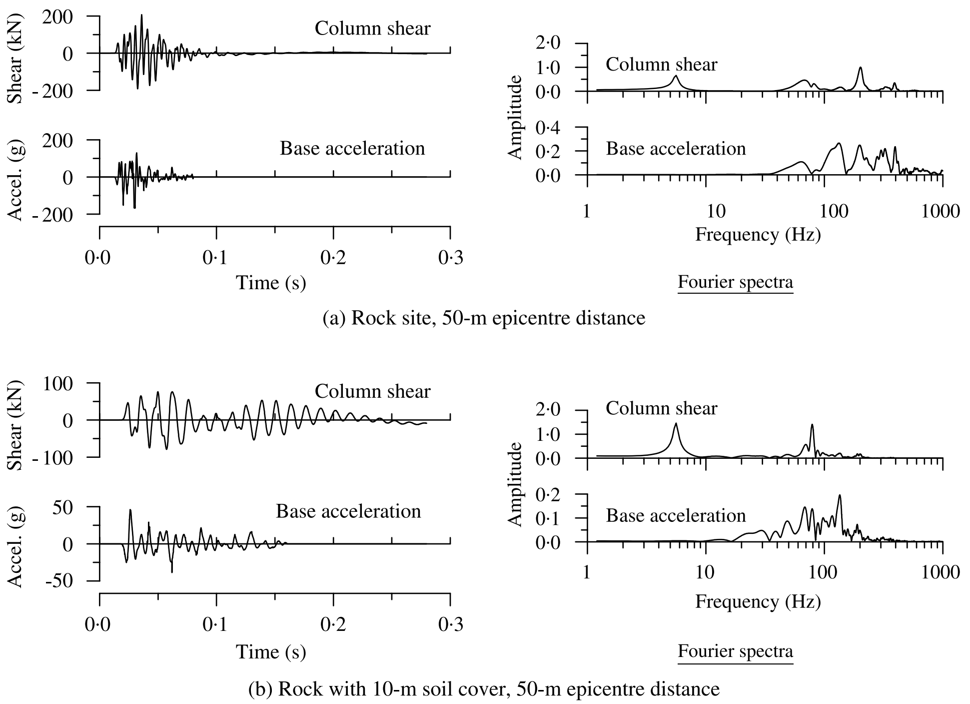


Figure 11. Case study: horizontal ground motions induced by underground explosion and column shear response.

frequency can be found to range from 40 to 80 Hz and second mode from 105 to 210 Hz when the length to cross-section depth ratio varies from 16 to 8. On the other hand, typical construction blasting ground motions can have a principal frequency ranging from 20 to 60 Hz [1], whereas the frequency range of the nearfield ground shocks in case of the detonation of a typical underground ammunition magazine, as will be described in the next paragraph, could spread over 50–300 Hz. Under these circumstances, the chances that the local-mode resonance gets excited are obvious.

As a case study, the frame shown in Figure 2 is subjected to the ground shocks generated by an intense underground explosion. The explosion and the subsequent stress wave propagation are simulated using a finite-difference computer code (AUTODYN). The profile of the explosion is as follows: charge weight 100 tons TNT equivalent; embedded depth 46·3 m below the rock surface; overburden soil none (case-A) or 10-m thick (case-B). The ground motions are sampled at an epicentre distance of 50 m on the ground surface. Figure 11 shows the horizontal ground motions at the above station for the two site conditions, respectively, together with the corresponding shear response in the column.

As can be seen, despite the varying waveform shapes and the spreading frequency contents of the ground motions, the computed response exhibits clearly a predominant contribution from the local-mode resonance. With the intensity of the ground motion for case-A (Figure 11(a)), i.e., PGA $= 175g$ and PPV $= 1{\cdot}02$ m/s, the maximum column shear response due to the horizontal excitation alone could reach 200 kN if the column were to remain elastic. This shear demand largely exceeds the estimated cracking shear strength of 110 kN for the column. The maximum moment response is 90 kN m which is below the estimated yielding moment of 112 kN m. Hence, a failure due to shear is likely to occur.

It is also interesting to note that with a thin layer (10 m) of soil cover (case-A), the PGA of the ground shock attenuates much faster than in the case with no soil cover (case-B), due to the enhanced filtering effect of the soil to high-frequency stress wave components [2, 11] as is evident from the Fourier spectra shown in Figure 11. Thus, the PGA in case-B is reduced to about 1/3 of the PGA in case-A, along with a marked change of the frequency spectrum, resulting in a much decreased column force effects.

4. EXPERIMENTAL INVESTIGATION

The analytical evaluation has been carried out under the following two assumptions: (1) a constant damping coefficient (5%); and (2) wave effect (phase lag) can be neglected. As

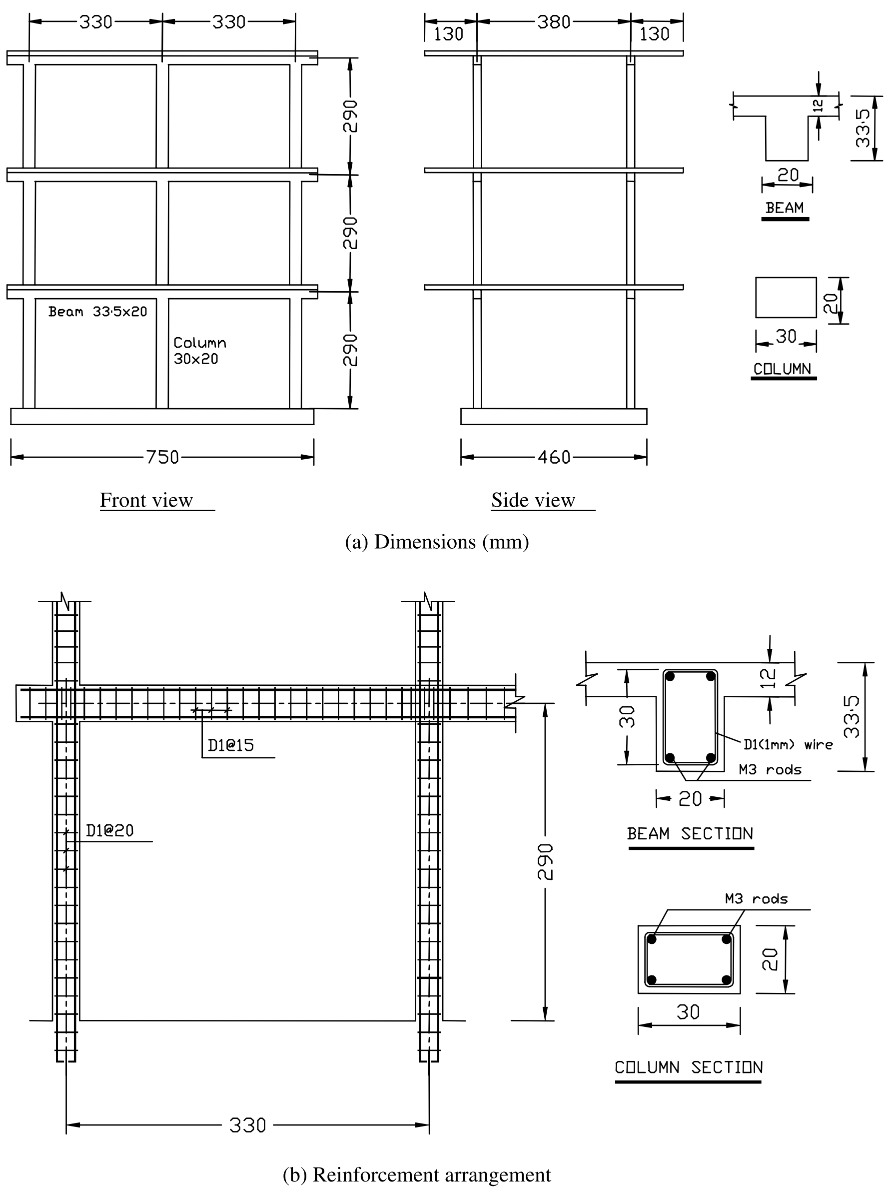


Figure 12. Test model dimensions and reinforcement arrangement (model scale 1:12).

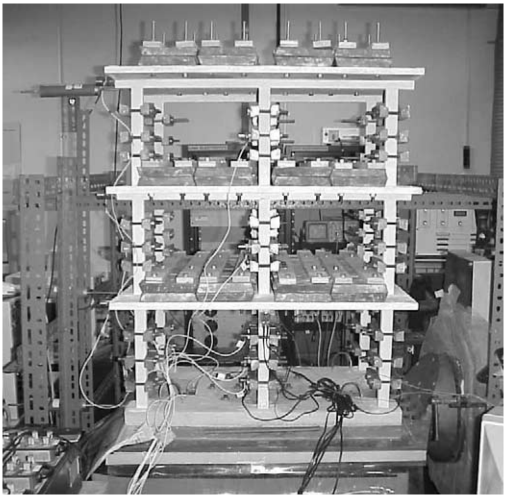


Figure 13. Test set-up.

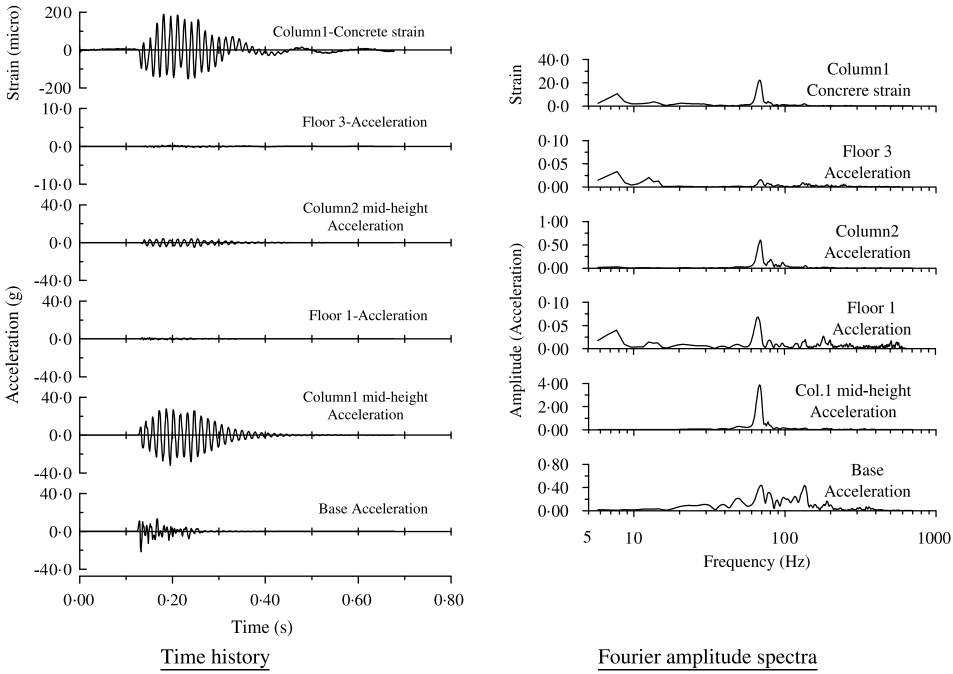
a matter of fact, the adequacy of these assumptions for high-frequency shock response predictions, even within the elastic stage, is not validated and relevant experimental data are very scarce from the literature. For this reason, an experimental investigation was conducted on a reinforced concrete model frame with an electromagnetic shaker. The experiment originally was also planned to demonstrate the anticipated damage process and failure mode pertaining to high-frequency local-mode vibration. This however was not actually achieved, due to the capacity limitation of the shaker, and the model was finally failed with lower frequency excitations which are not to be discussed in detail herein.

The test model represented a real three-storey two-bay prototype frame having a storey height of 3·5 m and span length 4 m. The model was fabricated at 1:12 reduced scale. Figure 12 shows the model dimensions and reinforcement details. To represent the local practice (non-seismic region), no special seismic design requirement was considered. Dedicated efforts were made to ensure the reliability of the model in representing the prototype frame response, such that the model frame was fabricated using microconcrete and deformed model reinforcement having mechanical properties similar to their prototype counterparts, as listed below:

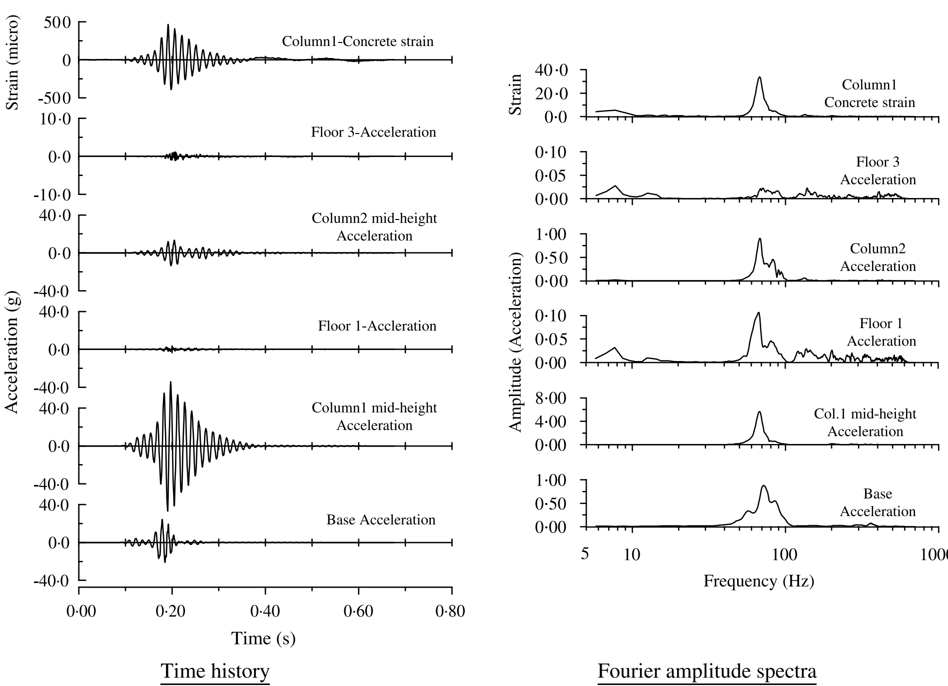
- *Model concrete*: Cylinder (100×50 mm) compressive strength $f'_c = 31$ MPa; splitting tensile strength $f_t = 2{\cdot}9$ MPa; modulus of elasticity $E_c = 23$ GPa.
- *Model reinforcing bars* (*M*3 *threaded rods*, *effective diameter* 2·36 mm): Yield strength $f_y = 345$ MPa; ultimate strength $f_u = 460$ MPa; modulus of elasticity $E_s = 200$ GPa.
- *Bond strength*: 13 MPa or $0{\cdot}45 f'_c$.

The electromagnetic shaker that was used to host the specimen was capable of producing high-frequency and high-acceleration (subject to the rated force limit) motions. Figure 13 shows the test set-up. The instrumentation was organized so that the accelerations at the floor levels and along the column height, as well as the concrete strains at various locations on the column face, could be measured.

It is worth noting that, in addition to the ballast placed on the floor slabs to satisfy the global similitude requirement [12], a proper amount of the artificial mass was distributed and attached to the columns such that the same similitude status is achieved at the local member level. Such distributed artificial mass scheme, although not usually required when



(a) Response to wide-band shock excitation (PPV=0·36 m/s)



(b) Response to narrow-band shock excitation (PPV=0·53m/s)

Figure 14. Measured responses to simulated ground shock excitations.

conducting low-frequency earthquake simulation tests, is apparently necessary for high-frequency response simulation because of the involvement of local-mode vibration.

To avoid referring to the scaling factors all the time, the measured responses reported here have already been converted into the actual (full-size) scale. Figure 14(a) shows the base excitation and the representative response time histories recorded during a typical shock simulation test, as well as the corresponding Fourier amplitude spectra.

As can be seen, despite that the base excitation had a wide frequency band over 40–130 Hz (similar to that shown in Figure 11), the measured accelerations exhibited apparently a predominant contribution from the first column mode at a frequency of about 70 Hz. The marked amplification of the acceleration on column indicated clearly a column resonance state, while the measured concrete strain on the column face confirmed that such local-mode resonance could indeed produce significant structural effects. With the PPV of the horizontal base motion being equal to 0·36 m/s, the measured maximum concrete strain was about $200\mu\varepsilon$.

Figure 14(b) shows the response recorded during a typical narrowband excitation having a principal frequency close to the column first-mode frequency. The column mode resonance effects become more significant, and at PPV = 0·530 m/s the maximum concrete strain exceeded $500\mu\varepsilon$.

For both tests, the accelerations measured at floor levels were very small as compared to the accelerations on the first-storey column. The acceleration of the second-storey column also decreased drastically from that of the first-storey column. These observations confirmed the analytical findings regarding the floor mass "blocking" effects due to the high-frequency vibration caused by the ground shocks.

A comparison between the predicted and measured accelerations at the first floor level and at the mid-height point of the first-storey columns is given in Figure 15. The distributed mass model described earlier was adopted for the analysis and a 5% damping was considered. A general agreement can be observed between the computed and measured

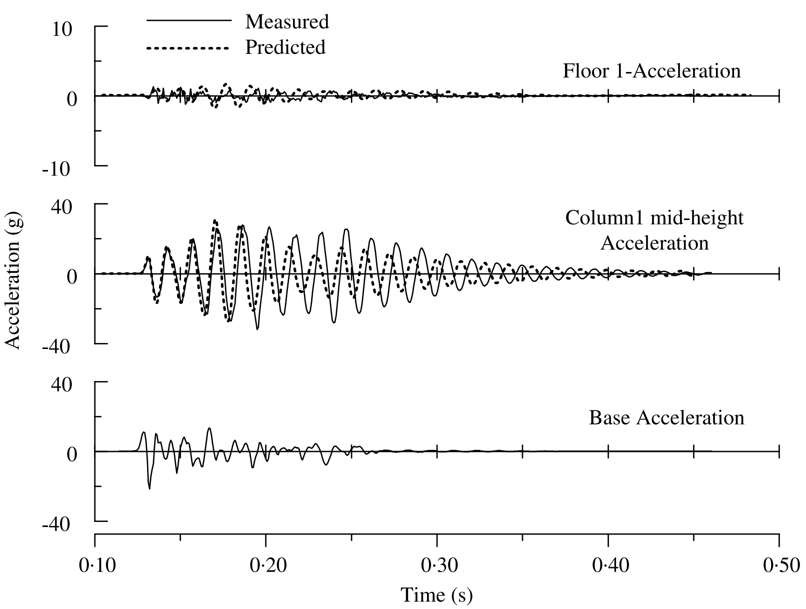
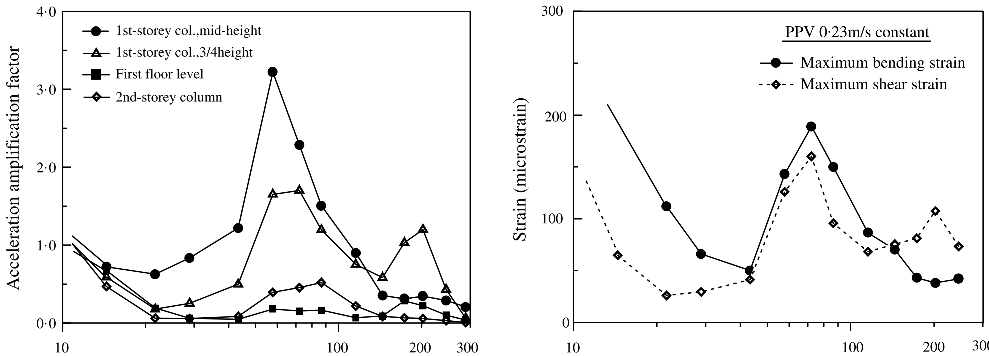


Figure 15. Comparison between measured and predicted acceleration response.



(a) Acceleration amplification factor (a/a_g)

(b) Measured concrete strain

Figure 16. Variation of measured response with principal base-motion frequency for PPV ≡ 0·23 m/s.

responses, and this suggests that the modelling scheme and the 5% damping coefficient are generally adequate for predicting the response of the frame subjected to high-frequency base excitations.

With regard to the effects of varying principal excitation frequency, Figure 16 shows the variation of the measured acceleration amplification factor and the maximum concrete strain with the principal frequency of the base motion having a waveform similar to that shown in Figure 14(b). The curves generally resemble the trends observed from the analytical predictions described earlier. The strain curves confirm that the relative significance of the shear effect as compared to bending effect does increase with increase of the input frequency. However, the absolute shear strain beyond the first local-mode resonance appears to be smaller than what would be anticipated from the analytical shear force curve as shown in Figure 6(a). This discrepancy may be attributed to increased wave effects at a frequency as high as 200 Hz. Apparently, for more accurate response prediction under such high-frequency excitations, more sophisticated analysis taking into account the wave effects may be necessary.

5. CONCLUSIONS

(1) The structural response to horizontal ground shock excitations has a pattern that differs significantly from the low-frequency seismic response. Contrary to the global-modes (floor movements) dominated seismic scenario, the high-frequency shock response is likely to involve significant local (elemental) mode vibration, which to a large extent can be uncoupled from the global floor response. Consequently, high force (stress) effects can develop at small floor displacements. As such, an indicator of the material level response (stress and strain), rather than the overall displacement or drift, becomes critical.

(2) An interesting phenomenon associated with the local-mode resonance is that the ratio between the maximum column shear and bending moment effects increases as compared to the low-frequency global mode response. This implies that there is an increased risk for a shear failure to occur when the excitation frequency becomes high, say above 70% of the column first-mode frequency. Although for equal ground motion PPV the low-frequency seismic response appears to govern both shear and

moment effects (Figure 6), a satisfactory seismic design does not automatically warrant a safe shear resistance against intense ground shocks because the shock PPV could be much higher. The trend of increased risk for a premature shear failure to columns under high-frequency horizontal excitations is confirmed by the experimental results.

(3) In contrast to the global-mode effects, the local-mode vibration is shown to have a localized feature under horizontal ground shocks, due to the "blocking" effect from the concentrated heavy floor mass. For typical frame structures, the local-mode effects are shown to concentrate at the lowest storey(s). This phenomenon is also confirmed by the observed response of the three-storey testing frame.

It has to be pointed out that the focus of the current investigation was to establish a pattern of the local-mode vibration response, as well as the corresponding structural effects, under high-frequency ground shock excitations. For this purpose, and particularly for a comparison with the established characteristics of seismic response, the consideration of the horizontal excitation alone seemed to be sufficient. However, in many practical situations, the ground shocks can involve significant vertical components which may also induce significant structural effects. In this respect, and concerning the derivation of more rational criteria for shock response and damage, further analytical and experimental studies will be needed.

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