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RESPONSE OF MODEL STRUCTURE UNDER SIMULATED BLAST-INDUCED GROUND EXCITATIONS

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SUMMARY

Little is known regarding the actual response characteristics and damage process of RC structures subjected to a close-in intense underground explosion. What seems to be clear is that, due to the high-frequency and high peak-acceleration nature of the excitations, the importance of overall response which is common under seismic conditions may give way to local vibration and the resultant high (shear) stress state. In this paper, an analytical evaluation of the problem is briefly described first, followed by the presentation of a unique experimental investigation. The experiment was conducted on a small-scale model structure using an electromagnetic shaker. Results indicate distinctive response characteristics under high-frequency ground motions as compared to that under low-frequency seismic conditions. In particular, the high shear force tends to dominate the response and the damage process.

INTRODUCTION

The ground motions induced by close-in intense underground explosions differ from the normal seismic ground motions mainly in the following two aspects: a) they are usually dominated by high frequency components (up to, say, 300Hz); and b) the peak ground acceleration could be very high. Because of these, the corresponding structural response are also expected to differ: Firstly, local vibration may effectively be excited as the result of regional resonance, introducing momentarily high stress state and subsequent damage; Secondly, and contrary to the above, overall structural response (featured by inter-story drifts or element rotations) is likely to be low, as is common under high-frequency vibration environment. These distinctive features render the conventional deformation-based damage criteria not applicable.

Concerning high frequency ground excitations, existing guidelines regarding the vibration limits are usually based on peak particle velocity of the ground motion (PPV) so that the criterion can be relatively independent of the actual frequency contents. Nevertheless, the nature of structural response to high-frequency ground excitations is not clearly identified. Furthermore, the existing provisions are of limitations because a) No clear distinction is made between frequently occurring vibrations (e.g., the construction vibrations) and extreme circumstances such as the accidental detonation of a underground ammunition magazine; and b) Most existing recommendations on vibration limits are crack-oriented and they have been based on observations of cracking over non-structural walls of relatively old houses, not typical for modern building structures. As the result, the recommended vibration criteria tend to be rather conservative, for instance, the “safe blasting” in terms of peak particle velocity of the ground soil ranges between 50.8 mm/s (2.0 in/s) [Duvall and Fogelso, 1962; Edwards and Northwood, 1960] and 83.8 mm/s (3.3 in/s) [Grandell, 1949], whereas the criteria for “minor” and “major” damage are 137 mm/s and 193~231 mm/s respectively [Duvall and Fogelso, 1962; Nicholls et al. 1971; Langefors et al. 1958]. It is noted that the published data are rather scattered, and in some cases the maximum “no damage” velocity reached 508 mm/s (20 in/s) [Nicholls et al. 1971].

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As part of the efforts in a broad research program towards damage assessment of surface RC structures subjected to intense underground explosions, an experimental program is being undertaken at the Nanyang Technological University of Singapore. The primary objective of the investigation is to acquire the badly-needed first-hand information regarding the response characteristics and damage process of structures under high-frequency ground shock excitations, and with this to calibrate the numerical models developed under the same project. This paper presents the investigation of a 1:5 scale model structure.

PRELIMINARY NUMERICAL ANALYSIS

Preliminary numerical analysis was carried out first to assess the main response features of the structure under the prescribed shock ground motions. For this analysis, the one-storey test structure is modelled using the global beam-column modelling scheme, as shown in Fig. 1. In order to capture the effects induced by the possible local (elemental) vibration, the column elements were divided into several segments connected by intermediate joint nodes, and the respective column mass was distributed to these nodes instead of being lumped to the floor level. For a comparison, the normal modelling scheme with which each column member is modelled by a single element (i.e., zero distributed mass in Fig. 1), as is common in seismic analysis, was also analysed. The latter model actually precludes the possibility for the elemental vibration to be excited, thus the comparison of the results between these two models would highlight the local vibration contribution to the structural response under high-frequency excitations.

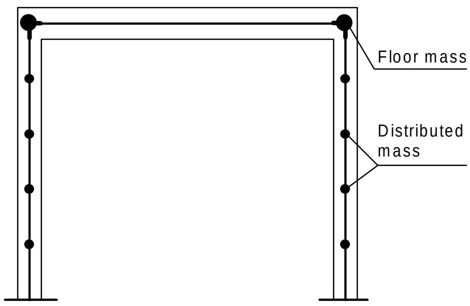
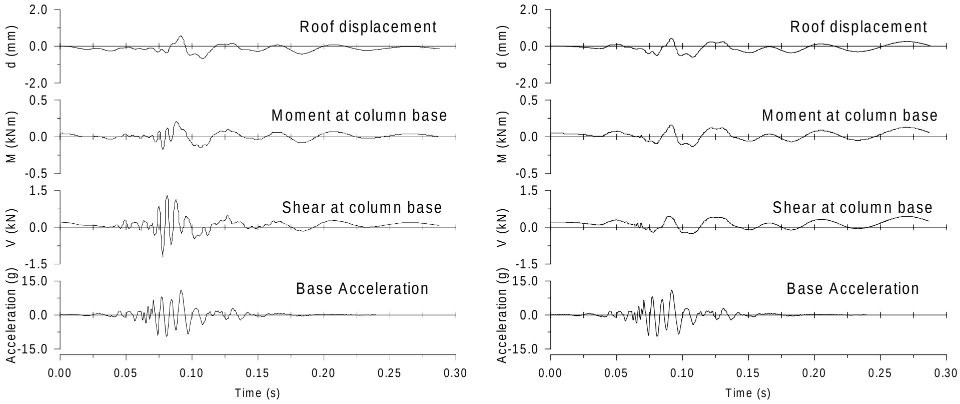


Fig. 1. Simplified model for analysis under high-frequency base excitations (dimensions see Fig. 4)

The base accelerogram was modelled after a numerical study of underground explosions [Hao et al. 1997]. The base accelerogram and the main computed response time histories (roof displacement, maximum column shear force and moment) are shown in Fig.2. Fig. 3 illustrates the respective Fourier amplitude spectra.

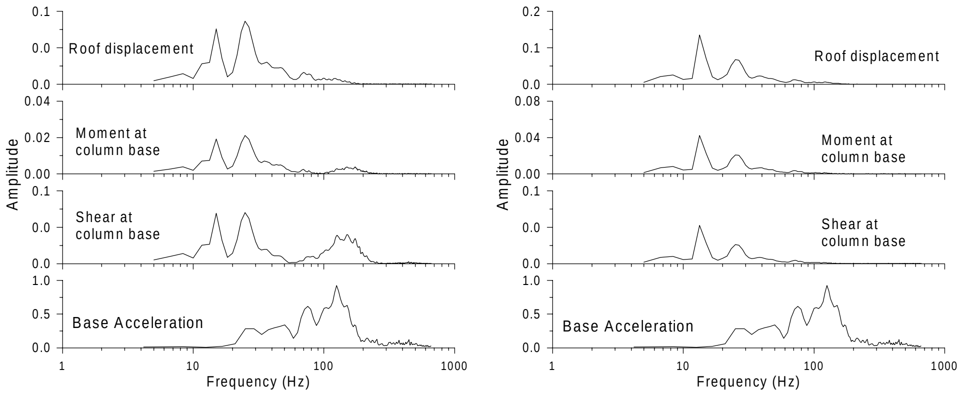
As can be seen from the figures, while the roof displacement and column moment time histories appear to be almost identical between the two cases (their absolute amplitudes are indeed small), the column shear force response, on the other hand, clearly demonstrates the local vibration effects. In fact, the high frequency contents shown in the Fourier spectrum of the column shear response, when distributed masses are considered, are at around the first-mode resonance of the column member (approximately 150Hz). More importantly, the increase of the shear response amplitude due to local vibration is enormous (about 300%), such that the corresponding shear stress could become a dominant parameter in the response and damage process of the structure.

The implications of the above results may be expressed as the following: 1) It can be very real that the local vibration be effectively excited under high-frequency ground shocks, introducing high-shear response; 2) Overall response items, such as the roof displacement and column moment, are likely to be small. Thus, the deformation-based damage assessment approaches seem not applicable; and 3) Concerning reduced-scale model tests, it is apparent that the similitude laws must be extended to the element level, or else the response features could be miscaptured.



a) With distributed masses on columns　　b) Without distributed masses on columns

Fig. 2. Predicted response time histories for the model structure with and without accounting for distributed masses along column members



a) With distributed masses on columns　　b) Without distributed masses on columns

Fig. 3. Fourier amplitude spectra of the computed responses

EXPERIMENTAL PROGRAM

Test structure

The test model was a rather conceptual single-storey framed structure, shown in Fig. 4. The dimensions of the model structure represented a prototype frame of storey height 3.5m and span length 4m, at a scale of 1:5. An enlarged base plate was provided for the anchorage of column longitudinal reinforcing bars, and to allow for a proper attachment of the test model to the shaker.

The model structure was constructed using microconcrete and model reinforcement. To better simulate the bond behaviour, use was made of threaded rods as model main reinforcing bars, after appropriately annealing treatment. The characteristic material properties were found to be satisfactory as listed below:

- Model concrete: Cylinder (100x50mm) compressive strength 31MPa, Splitting tensile strength 2.9MPa, Modulus of elasticity 27GPa
- Model reinforcing bars: Yield strength 465MPa, Ultimate strength 505MPa, Modulus of elasticity 200GPa, and maximum elongation above 10%
- Bond strength: 18MPa or 60% cylinder compressive strength

Test program

High frequency and high peak acceleration are two features of blast-induced ground shocks that actually hinder the laboratory reproduction of shock response using earthquake-oriented shaking-table facilities. In the current experiment, use was made of an electromagnetic shaker. This particular shaker is manufacturer-rated to the maximum acceleration of 120g and the maximum frequency of 3000Hz. In actual tests, however, the maximum achievable acceleration is governed by the shaker's force power (27 kN) and the overall test load. For the particular test model herein, finally a peak acceleration of about 10g was achieved. It is noted that only horizontal excitation was considered in the current tests. An overall view of the test setup is shown in Fig. 4.

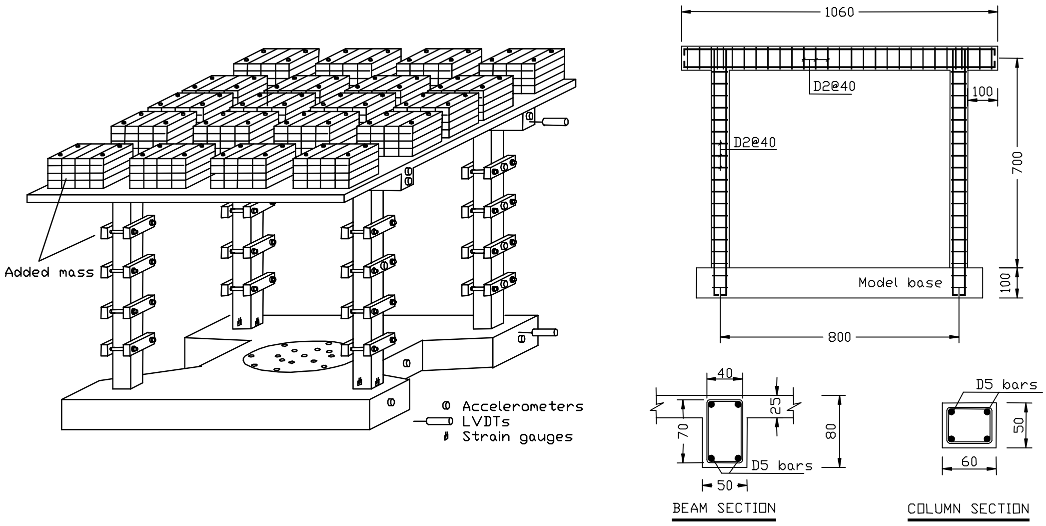


Fig. 4. Model structure configuration; test setup and reinforcement details

As mentioned earlier, because of the need for local vibration simulation, the way of mounting the additional mass becomes a particularly important issue. Unlike what is typically seen in low-frequency earthquake simulation tests, whereby the entire additional mass is allocated and attached to the test model at floor levels, part of the additional mass that is associated with the column members was distributed and attached to the columns of the model as shown in Fig. 4. This is a unique feature of the current test implementation and it is considered essential for high-frequency shock response simulation.

The instrumentation covered the following: 1) accelerations (measured by accelerometers) at base and roof levels, and along the height of column members; 2) displacements at base and roof levels, measured by LVDTs; and 3) reinforcement strains at selected locations in column and beam members. The entire data acquisition procedure was automated using a dynamic control program.

Totally some 30 tests were performed on the model, these tests were divided into the following three phases:

- First phase: the model was tested under simulated shock motions of which the principal frequency was about 100Hz, with the maximum peak acceleration to be nearly 10g, and the maximum PPV equal to 0.2m/s at the model scale;

- Second phase: the model was tested under modified shock motions, obtained basically by compressing the frequencies of the original input on a step-by-step basis (from 100 Hz to 15Hz). This allows a progressively increasing peak velocity while the peak acceleration of the motions was restricted to below 10g, it also allows for an evaluation of the relationship between the response amplitudes and the PPV with varying principal frequencies;

- Third phase: the model was subjected to base motions with further decreased frequencies (8~12 Hz at the model scale), with intent to generate overall resonance and test the model to failure.

Complementary low-amplitude random vibration tests were performed after each major test to determine the change of the dynamic parameters of the model.

TEST RESULTS AND DISCUSSION

General

Fig. 5 shows the typical measured base acceleration histories and their corresponding response spectra. The main characteristics of the base motions and the key response amplitudes are depicted in Fig. 6.

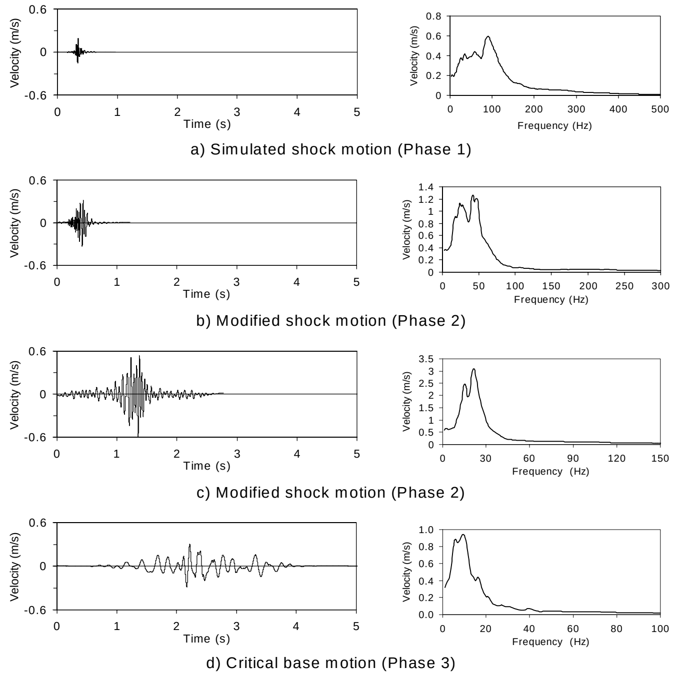


Fig. 5. Achieved base acceleration time histories and the corresponding response spectra

The gradual decreasing trend of the principal frequencies of the input motions for the three-phase tests can be readily seen from Fig. 6a). While the maximum peak base acceleration was restricted at below 10 g (Fig. 6b)), reducing the input frequencies enabled a gradual increase of the PPV from 0.2 to 0.6m/s for tests in Phases 1 and 2 (Fig. 6c)). Taking into account the scale factor, this corresponded to PPV at full-size scale from 0.45 to 1.3m/s, giving an extensive coverage of shock intensity in velocity terms.

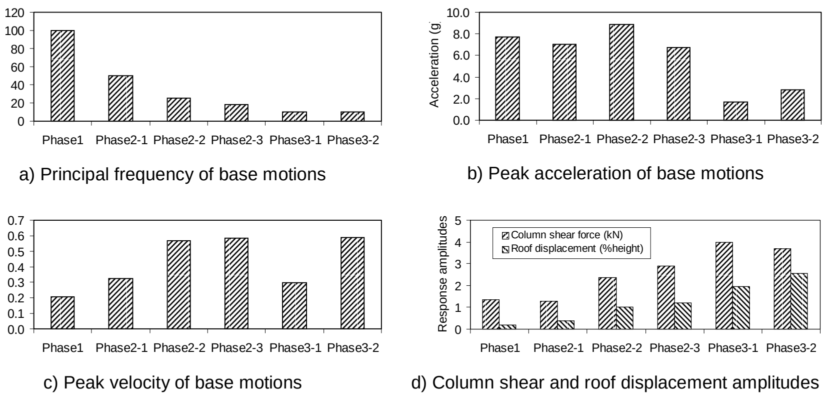
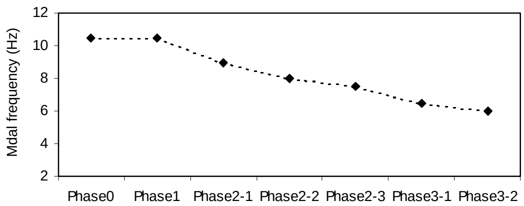


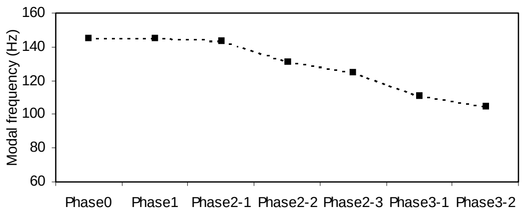
Fig. 6. Characteristic base-motion and response parameters

Shown in Fig.6d) are the measured column shear-force and roof displacement amplitudes. Note that the column shear force response was obtained according to the inertial forces induced by the reactive masses at all levels, and it is indicative to the magnitude of the local (column) vibration. The roof displacement represents the overall structural response. Two remarkable phenomena can be observed: a) Despite that the peak base accelerations and velocities were apparently low for the low-frequency tests in Phase 3, the structural responses (column shear and roof displacement alike) were considerably high; On the other side, concerning the high-frequency response (Phase 1), the column shear response appeared to be disproportionately large while overall response remained small. This clearly signifies the effects of the regional resonance, as indicated in the analytical predictions. The response during Phase 2 tests appeared to somehow correlate with the PPV in a stable manner.

The test model survived the Phase 1 and 2 tests without visible damage, although the maximum PPV reached 0.6m/s (equivalent to 1.3m/s at full-size scale). Nevertheless, the gradual decrease of the measured natural frequencies of the model, shown in Fig. 7, suggested that intrinsic damage within the structure progressively developed during the course of these tests.



a) Variation of first-mode frequency of structure



b) Variation of first-mode frequency of column

Fig. 7. Variation of measured modal frequency of test structure

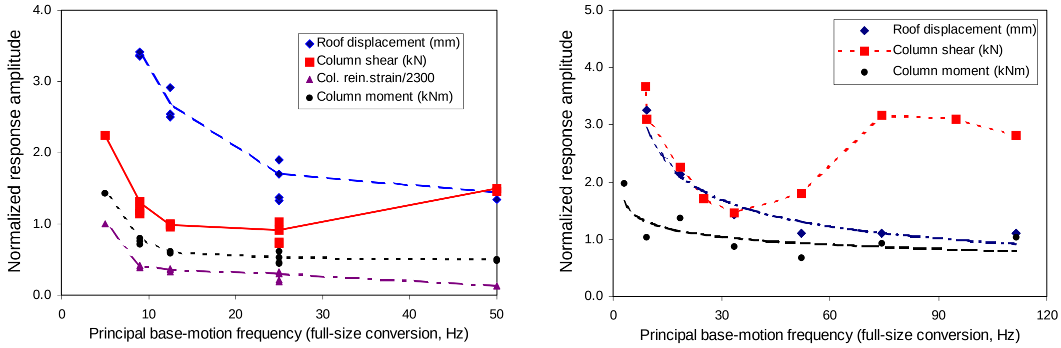
Major cracking occurred during the first test in Phase 3, and intensified thereafter. Most of the cracks took place at the top and bottom of the columns. After two consecutive tests, the maximum crack width was observed to be about 0.6mm, and the maximum strain of the column longitudinal reinforcement exceeded 5000 microstrains.

Correlation between structural response and base-motion frequency

The observations discussed in the previous section indicate that the relationship of the model response to PPV depends on the principal base motion frequency, especially in low- and high-frequency ranges. To further examine this dependency, the major response amplitudes (including the roof displacement, column shear force, column reinforcement strain and column moment) are normalised to equal PPV and plotted versus the principal frequency of the base motions in Figure 8a). Note that in the figure the frequency axis is already converted to the full-size scale. Besides, the column moment shown in the figure refers to the sum of the moments at the top and bottom ends of the column member. The following may be observed: a) While the general responses tended to decrease or remain stable with increase of the base-motion frequency, the column shear force, which signifies the local vibration effects, appears to abruptly increase when the input frequency becomes sufficiently high; b) The general response to low-frequency motions (Phase 3, <10Hz in Fig. 8) are significantly larger and tend to increase at a steep rate when the resonance of the global model is being approached. Quite obviously, the PPV does not seem to provide a unified representation of ground motion effects when the frequency characteristics vary significantly.

The computed results shown in Fig. 8b) indicate the similar tendency. Moreover, it can be seen that the increase of column shear force would continue until the resonance of vibration of the column member (at about 75 Hz of prototype scale) is reached. Further increase of the base-motion frequency tends to result in some decrease of the shear response while departing from the resonance.

Summarising the above observations, two levels of resonance can be identified: one at global structural level, which is likely to be excited under low-frequency seismic-type base motions; and one at regional (elemental) level and apparently is prone to high-frequency shock excitations. Whereas the response corresponding to the "global resonance" can be characterised by concurrently large response amplitudes in almost all terms, the response at "local resonance" is dominated by momentarily very high shear force, while the overall response remains small. Consequently, the damage process can be fundamentally different between the two cases.



a) Measured b) Computed

Fig. 8. Response amplitudes normalised to equal PPV vs. principal input frequencies

The tendency for damage to develop in a different ways under high-frequency shock excitations can also be examined by plotting the relationship between the maximum shear force and moment (sum of two ends) in the column members, as shown in Fig. 9. Tracing the development of the column forces along the line for high-frequency excitations, one may figure out that shear failure may precede the flexural failure. The situation can be further worsened if the regional resonance becomes more significant, as indicated by the dashed line.

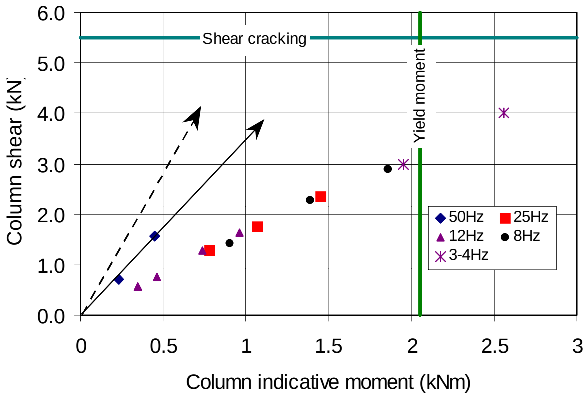
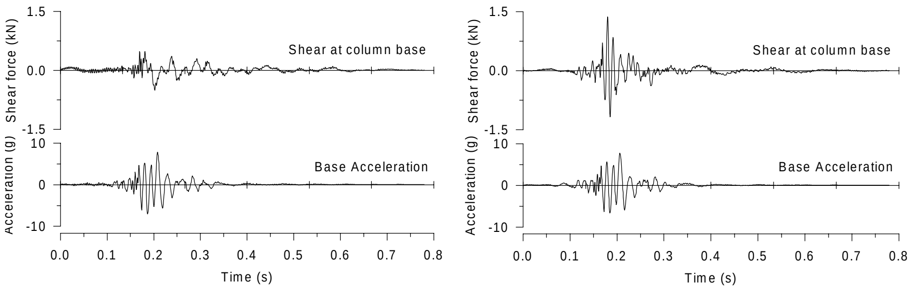


Fig. 9. Inter-relationship between column shear force and column moment for base motions of different principal frequencies (The frequencies indicated are full-size conversions)



a) Before attaching masses on columns b) After attaching masses on columns

Fig. 10. Comparison between measured column shear before and after attaching mass blocks on columns

Effectiveness of local mass similitude in simulating local vibration response

As mentioned earlier, in the current experiment the mass similitude was extended to the elemental level by attaching part of the additional mass along the height of column members. This was considered essential to the reproduction of the local vibration effects, and was proven effective by the experimental results. Fig. 10 shows a comparison between the measured column shear response histories before and after attaching the mass blocks on the columns, under almost identical shock excitations. As seen, after attaching masses on columns (thus maintaining similitude of vibration at elemental level), the column shear force exhibited pronounced high-frequency contents and the force amplitude reached about 3 times as high as that before attaching these masses. These observations agree well with the analytical predictions described in the preliminary analysis, and apparently are attributed to regional resonance effects which simply can not be reproduced in the case where the local mass similitude is ignored.

CONCLUSIONS

Based on the current test results, the following tentative conclusions may be drawn: 1) Structural response to high-frequency ground shocks can be fundamentally different from seismic response. In principle, the shock response is characterised by local vibration-induced high shear stress at relatively low overall (displacement) response. Consequently, the immediate shock damage is likely to be stress-oriented, whereby the conventional deformation-based damage criteria may no longer be applicable; 2) While the high-shear phenomenon requires particular care for response under high-frequency ground excitations, the current test results indicate that, for the particular RC model herein, the PPV equal to 0.7~0.8m/s (full-size conversion) seemed to be a threshold at which appreciable stiffness drop occurred, whereas PPV as high as 1.3m/s marked the onset of reinforcement yielding in the case where only horizontal excitations were considered; and 3) The exercise of extending the similitude in reactive mass to local (elemental) level is experimentally justified. In fact, it was proven to be essential to the reproduction of local vibration effects in the model structure under high-frequency shock motions. Further research efforts are required in the direction towards rational damage criteria for structural response under high-frequency, blasting type ground excitations.

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