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NONLINEAR SEISMIC RESPONSE AND COLLAPSE ANALYSIS OF A STAGGERED FLOOR BRICK BUILDING WITH RC MEMBERS

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ABSTRACT

The staggered floor brick residential building confined by reinforced concrete members is a new type of building structure. We designed a nine floor residential brick building with six staggered floors confined by reinforced concrete beams and columns in 1993, and the building was built in Shenyang City, China. The seismic response of the building is studied in this paper. Firstly, the aseismic experimental research of brick walls in China is reviewed. Secondly, a dynamic analytic model of this type of building is presented. By using this model, the dynamic characteristic and nonlinear response of the building subjected to earthquake ground motions are analyzed. Finally, the aseismic capability of the building is evaluated.

INTRODUCTION

In the developing economy of China, more and more residential buildings have been designed for reasonable, comfortable living in China in recent years. In this paper, the building studied is this type of residential building as designed by the authors in 1993. This building has been built and occupied in Shenyang City, China. It is a nine floor brick structure with six staggered floors confined by reinforced concrete columns and perimeter beams. Its aseismic designing intensity is VII. The usable floor area of the

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building increases 30% beyond than that of a building without staggered floors. Therefore, the land occupied by the building is economized and the price of per unit usable floor area is reduced. For this reason, this type of residential building has a bright future in the building market. However, can this type of building be widely built in the earthquake zone? How is its aseismic capability? We do not know! In this paper, by means of the step-by-step integral method, we will study the elastic, elasto-plastic and collapse responses of the building subjected to earthquake ground motions, give the evaluation of its aseismic capability, and propose suggestions for improving the aseismic capability of the building.

REVIEW OF EXPERIMENTAL RESEARCH

In the 1980's, a great many experiments on brick walls were made for studying the properties of strength, deformation and consumed energy. The experimental results can be used in nonlinear seismic response analysis. Here, some results are reviewed. Xian J.Q. (1986) got the load-deformation curve of brick walls as shown in Fig. 1. The control values are $P_A/P_u = 0.78$, $\Delta_D/\Delta_u = 7$, $K_1/K_0 = 0.152$, $\Delta_c/\Delta_u = 2$. In Fig. 1, A is the cracking point, B is the limiting point, C is the damage point, and D is the collapse point. The yield strength of a brick wall is $R_\tau = R_j + 0.4\sigma_0$. The elastic stiffness is $K_\sigma = (1 + \sigma_0/4)K_0$, $K_0 = Et/((H/b)^3 + 3H/b)$.

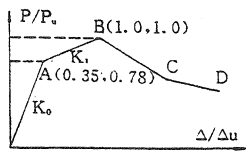

Fig. 1

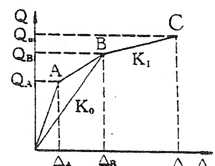

Fig. 2

The typical load-deformation curve presented by Huo (Huo, et al. 1986) is shown in Fig. 2. In which, A is the initial cracking point, B is the cracking point, and C is the limiting point. $Q_B/Q_u = 0.89$, $\Delta_B/\Delta_u = 0.51$, $Q_A/Q_u = 0.82$, $\Delta_A/\Delta_u = 0.23$, $K_1/K_0 = 0.111$.

The influence of the reinforced concrete columns on the strength and stiffness of brick walls was studied by Wu (Wu, 1982). The results indicated that the deformation capability of the wall increased by adding reinforced concrete structural columns, but the stiffness of the wall did not increase, and the yield shear force of the wall increased 15-20 percent. The control points of the load-deformation curve are as follows: cracking load $Q_c = R_\tau A_\tau/\xi$, in which $R_\tau = R_j\sqrt{1 + \sigma_0/R_j}$, $A_T = \beta A_H + \eta A_C G_C/G_W$, $G_C = E_C/2(1 + \gamma_C)$, $G_W = E_W/2(1 + \gamma_W)$. Limiting load $Q_u = \gamma f \sigma_0 A_H + \alpha A_g \sigma_t f(B) \times 0.75/h$. Cracking displacement $\Delta_C = Q_C h\xi/(G_W A_T \lambda\theta)$. Limiting displacement $\Delta_u = Q_u \delta_j/(\lambda\theta)$. In Q_C, Q_u, Δ_c and Δ_u, the η, γ_c, γ_W, γ, α, f, β, λ, ξ, f(B), θ, δ_j, see (Wu 1982). The elastic and elasto-plastic stiffness of the wall are $K_0 = Q_c/\Delta_c = G_W A_T \lambda\theta/(h\xi)$, $K_1 = (Q_u - Q_c)/(\Delta_u - \Delta_c)$. According to the experimental results of three groups of walls, the average values are $Q_c/Q_u = 0.7027$, and $\Delta_c/\Delta_u = 0.220$.

In Ref. 7 (Yang, et al. 1986), the limiting load was given as $Q_u = R_\tau A/\xi + 0.2 R_g A_g$. The limiting load of brick wall presented by the China Aseismic Design Code for Building (1989) is $Q_u = f_{ve} A/\gamma_{RE}$, in which $f_{ve} = \zeta_N \cdot f_v$, $\gamma_{RE} = 0.9$, and $\gamma_{RE} = 1.0$ (no structural columns). The stiffness of a brick wall with openings (windows or doors) used in CADCB is (a) small opening, i.e., $\alpha = \sqrt{b'h'/bh} \leq 0.4$, and $h'/h \leq 0.35$, $K_e = (1 - 1.2\alpha) K_{eo}$, (b) large opening, i.e., $\alpha > 0.4$ or $h'/h > 0.35$, $K_e = 1/\Sigma \delta_i$, in which, $K_{eo} = Et/((H/b)^3 + 3H/b)$ is the elastic stiffness of the wall and δ_i is the flexibility of the *i*th sub-wall.

CALCULATING MODEL

The building studied is a structure of brick walls with reinforced concrete structural columns and perimeter beams. The building has two story heights: the higher story height is 3.45 meters and lower story height is 2.30 meters. Two of the higher stories are staggered just with three lower stories and the whole building is formed by six higher stories staggered with nine lower stories. The calculating model cannot be simplified as for the story shear model because the floors are staggered with respect to each other. Using the dynamic characteristic of real measurement, and knowing that the torsional effect is small, we can neglect the torsional freedom and only consider the translational freedom. For the above reasons, we propose a simplified analytic model which has twelve degrees of freedom in both the transverse and longitudinal directions, as shown in Figs. 6 and 7.

Mass

The mass of every story is concentrated at the corresponding floor at which the translational displacement is defined. The results are as follows: $m_1 = 237124.4$ kg, $m_2 = 204514.2$ kg, $m_3 = 237124.4$ kg, $m_4 = 422314.4$ kg, $m_5 = 236698.0$ kg, $m_6 = 204230.6$ kg, $m_7 = 236698.0$ kg, $m_8 = 422314.2$ kg, $m_9 = 236698.0$ kg, $m_{10} = 204230.6$ kg, $m_{11} = 236698.0$ kg, $m_{12} = 341320.4$ kg.

Elastic Drift Stiffness

When calculating the elastic drift stiffness, we consider both shear deformation and bending deformation. For the walls with small openings, i.e., $\alpha = \sqrt{b'h'/hb} \leq 0.4$, and $h'/h = 0.35$), the elastic drift stiffness is

$$K = (1 - 1.2\alpha) \frac{Et}{\left(\frac{H}{b}\right)^3 + 3\left(\frac{H}{b}\right)} \tag{1}$$

For the wall with large hollow, i.e., $\alpha > 0.4$, or $h'/h > 0.35$, the elastic drift stiffness is

$$K = \frac{1}{\sum \delta_i} \tag{2}$$

in which δ_i is the flexibility of the *i*th sub-wall.

Limiting Shear Force
In the analysis, the limiting shear force for each story is determined from the following formula:

$$Q_u = \zeta_N \cdot f_v \cdot A/\gamma_{RE} \tag{3}$$

in which ζ_N is the effect factor about the normal stress in the wall. It is given in CAD-CB. f_v is the design value of shear strength. A is the effective sectional area of the wall. When there are structural columns in the walls, $\gamma_{RE} = 0.9$; when there are no structural columns in the walls, $\gamma_{RE} = 1.0$.

EQUATION OF MOTION

The general equation of motion of structures subjected to earthquake ground motion is

$$M\ddot{V} + F_D(\dot{V}) + F_K(V) = -MI\,\ddot{u}_g(t) \tag{4}$$

in which M is the mass matrix, F_D is the vector of damping force, F_K is the vector of elasto-plastic restoring force, and V, $\dot{V}$, $\ddot{V}$ are displacement, velocity and acceleration, respectively. In a short time, the damping force and elasto-plastic restoring force can be expressed by linear formula. Then the above equation of motion is expressed in incremental form as

$$M\Delta\ddot{V}(t) + C(t)\,\Delta\dot{V}(t) + K(t)\Delta V(t) = -MI\,\Delta\ddot{u}_g(t) \tag{5}$$

We adopt the step-by-step integral method to solve this equation and obtain the time history response of the structure. Since the calculating model adopted is in lumped-mass form, the mass matrix has a diagonal form $M=\mathrm{diag}[m_1, m_2, \ldots, m_i, \ldots, m_{12}]$. The Rayleigh damping form is used. That is $C = \alpha M + \beta K_e$ in which $\alpha = 2(\xi_1\omega_2 - \xi_2\omega_1)\,\omega_1\omega_2/(\omega_2^2 - \omega_1^2)$, $\beta = 2(\xi_2\omega_2 - \xi_1\omega_1)/(\omega_2^2 - \omega_1^2)$, ξ_1, ξ_2 are the first and second mode shape damping ratios. They are obtained by experimental formula (Yang et al., 1986) where the formula is $\xi_1 = 0.008 + 0.55A/F$, in which A is the section area of walls of the building, and F is the area of the building. In the analysis, we adopt $\xi_2 = \xi_1$ approximately. The damping ratios of this building in the transverse direction are $\xi_1=\xi_2=0.078$. The damping ratios in the longitudinal direction are $\xi_1=\xi_2=0.53$. The transverse and longitudinal stiffness matrices are of the same form, that is

$$K(t)=\begin{bmatrix}
K_1+K_2+K_{1p} & & & & & & & \text{sym.} & & & & \\
-K_2 & K_2+K_3 & & & & & & & & & & \\
-K_{1p} & -K_3 & K_3+K_4+K_{1p} & & & & & & & & & \\
0 & 0 & -K_4 & K_4+K_5 & & & & & & & & \\
 & & & -K_5 & K_5+K_6+K_{2p} & & & & & & & \\
 & & & & -K_6 & K_6+K_7 & & & & & & \\
 & & & & -K_{2p} & -K_7 & K_7+K_8+K_{2p} & & & & & \\
 & & & & 0 & 0 & -K_8 & K_8+K_9 & & & & \\
 & & & & & & & -K_9 & K_9+K_{10}+K_{3p} & & & \\
 & 0 & & & & & & & -K_{10} & K_{10}+K_{11} & & \\
 & & & & & & & & -K_{3p} & -K_{11} & K_{11}+K_{12}+K_{3p} & \\
 & & & & & & & & 0 & 0 & -K_{12} & K_{12}
\end{bmatrix}$$

RESTORING FORCE MODEL

In the current dynamic analysis, the best restoring force model is trilinear with a descending stiffness branch model, as shown in Fig. 3. The restoring force curves for brick walls obtained in the experiments are also of this form, so that in the analysis, this model is adopted. In the restoring force model, the values of control points are as follows:

1) The limiting load is $Q_u = \zeta_N f_v A/\gamma_{RE}$
2) The cracking load is $Q_C = 0.8Q_u$
3) The initial elastic stiffness is as calculated formulas [1] or [2]
4) The elasto-plastic stiffness is $K_1 = 0.15K_0$
5) The descending stiffness is $K_2 = -0.02K_0$
6) The unloading stiffness is $K_3 = (X_y/X_m)^{\alpha}K_0$, $0.5<\alpha<0.7$
7) The opposite direction loading stiffness is $K_4 = |Q(X'_m)/(X_0-X'_m)|$ in which X_y is the yield displacement, X'_m is the maximum displacement in the opposite direction, X_0 is the loading displacement in the opposite direction.
8) The building collapse is defined when the response is on the descending stiffness branch and the displacement is 8 times the limiting displacement (Xia, 1986).

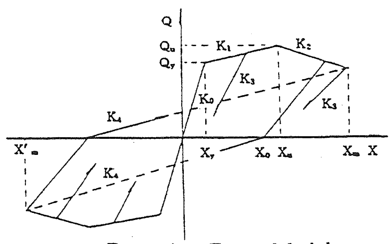

Fig. 3 Restoring Force Model

DYNAMIC CHARACTERISTICS

Solving the structural frequency equation $\|K - \omega^2 M\| = 0$, we have obtained all the frequencies and mode shapes of this building. The frequencies are listed in Table 1, and the first five modes of vibration are shown in Fig. 4 and Fig. 5. The basic frequencies obtained by calculation in the transverse and longitudinal directions are very close to the basic frequencies obtained by real measurement, see Table 1.

Frequencies (rad/s) Table 1

No.	1	2	3	4	5	6	7	8	9	10	11	12
Trans.	17. 14	47. 67	77. 77	106. 1	127. 3	148. 8	193. 5	220. 0	225. 0	230. 7	258. 2	274. 2
Longit.	19. 11	53. 02	86. 59	119. 7	143. 8	168. 3	221. 2	253. 0	268. 8	270. 6	311. 2	329. 4
Note	The basic real frequencies in transvers and longitudinal directions are 15. 07 rad/s and 19. 03 rad/s, respectively.											

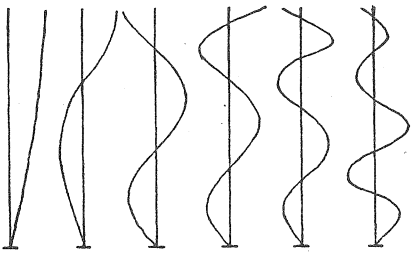

Fig. 4 First 5 Transverse Modeshapes

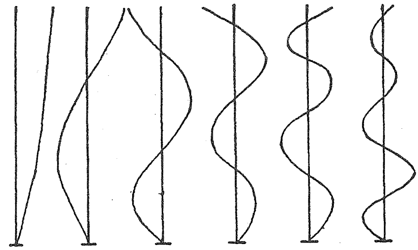

Fig. 5 First 5 Longitudinal Modeshapes

NONLINEAR SEISMIC RESPONSE

In the analysis, the basic earthquake ground motion input is the first 6 seconds of the El Centro acceleration record (U.S. 1940 NS), and it is multiplied by different coefficients, resulting in maximum acceleration values equal to different earthquake intensities in China. We also adjust the main frequency of the record and let it approach the basic frequency of the building. There are 13 cases of input loads in the transverse direction, and 6 cases of input loads in the longitudinal direction, as shown in Table 2 and Table 3.

Amplitude of Input Acceleration Records in Transverse Direction table 2

	intensity Ⅶ	intensity Ⅷ	intensity Ⅶ	intensity Ⅶ	intensity Ⅷ
small	A_{max}=0. 0356g	0. 0711g	0. 0356g	0. 0356g	
basic	0. 1070g	0. 2150g	0. 1070g	0. 1070g	
major	0. 2219g	0. 40g	0. 2219g	0. 2219g	0. 40g
Note			the main frequency is adjusted to 16. 32rad/s	neglecting the geo-metrical stiffness	

Amplitude of Input Acceleration Records in Longitudinal Direction table 3

	small	basic	major	Note
intensity Ⅶ	A_{max}=0. 0356g	0. 1070g	0. 2219g	
intensity Ⅶ	0. 0356g	0. 1070g	0. 2219g	the main frequency is adjusted to 16. 32rad/s.

Response in Transverse Direction

The maximum responses and the final state of the building under 13 input load cases are shown as the following table in which X_{max} is the maximum story shear force. From Table 4, we know that the maximum absolute displacements are always at the top floor and the maximum shear story forces are always at the base story. According to these phenomena, we know that the basic mode shape is the principal vibration form in the seismic response of the building. When the amplitudes of the input earthquake ground motions correspond to those of small intensity, basic intensity as well as major intensity, the states of the building are elastic, elasto-plastic (damage) and descending stiffness branch (strong damage), respectively. According to the China Aseismic Design Code for Building (1989), this building can be built safely in the zone of earthquake intensity VII. The floor relative displacement time histories and corresponding restoring force curves of earthquake intensity VII are shown in Fig. 8. When the amplitude of the input earthquake ground motion is equal to that of major earthquake intensity VII, the building has collapsed at 1.895 seconds.

Response in Transverse Direction Table 4

resp. / inte.	X_{max}(cm)	V_{max}(cm)	Q_{max}(KN)	danger story	final state of building	Note
small VII	0.03798(1)	0.2496(12)	1619(1)	No	elastic	(1) expresses the first story
basic VII	0.1139(S 2)	0.7492(12)	4845(1)	2nd, 6th. side 2nd, 5th	elasto-plastic	S 2 expresses the 2st story at side
major VII	0.6729(S 2)	1.721(12)	6623(1)	as above	descending branch	
small VIII	0.07586(1)	0.4985(12)	3232(1)	No	elastic	
basic VIII	0.5761(S 2)	1.713(12)	6596(1)	2nd, 6th side 2nd,5th	descending branch	
major VIII	1.843(2)	3.879(12)	7051(1)	as above	collapse t=1.895s.	
small VII	0.05416(1)	0.3488(12)	2308(1)	No	elastic	adjusting the main frequency
basic VII	0.2219(S 2)	1.054(12)	5915(1)	2nd, 6th, side 2nd,5th	elasto-plastic	as above
major VII	0.5277(S 5)	1.809(12)	6496(1)	as above	descending branch	as above
small VII	0.03789(1)	0.2491(12)	1614(1)	No	elastic	neglecting the geometrical stiffness
basic VII	0.1134(S 2)	0.7470(12)	6545(1)	6th, side, 2nd 5th	elasto-plastic	as above
major VII	0.6627(S 2)	1.725(12)	6545(1)	2nd, 6th, side 2nd.5th	descending branch	as above
major VIII	1.873(2)	3.972(12)	6984(1)	as above	collapse t=1.905s	as above

For studying the effect of the frequency spectra of input earthquake motion, we adjusted the main frequency of the input earthquake motion to 16.32 rad/s, which is close to the basic frequency of the building. The results of maximum story response are shown in Table 4. Comparing these results to the results of unadjusting input frequency from the record, we conclude that when the main frequency of the input earthquake motion is close to the elastic frequency of the building, the elastic response increases but sometimes the nonlinear response of the building does not increase. The reason is that when the building is in the nonlinear range, its frequency decreases and it is no longer close to the input frequency. Therefore, the response is relatively small. This conclusion indicates that a building is in more danger when the main input frequency of the earthquake is close to the elasto-plastic frequency of the building. From Table 4, it is seen that the effect of the geometrical nonlinearity is small in the seismic response of this type of building. The critical stories are the second and sixth stories and side stories 2 and 5 of the building in the transverse direction.

Response in Longitudinal Direction
There are 6 cases of earthquake ground motion input in the longitudinal direction of the building. The maximum response and the final states of the building in 6 cases are shown in Table 5.

Response in Longitudinal Direction — Table 5

resp. / inte.	X_{max}(cm)	V_{max}(cm)	Q_{max}(KN)	danger story	final state of building	Note
small VII	0.04204(1)	0.2393(12)	1921(1)	no	elastic	(1) expresses the first story
basic VII	0.4859(5)	0.9932(12)	3066(1)	1st, 5th stories	descending branch	
major VII	1.447(1)	2.141(12)	3149(1)	1st story	collapse t=1.9sec.	
small VII	0.0528(1)	0.2882(12)	2415(1)	no	elastic	adjusting the input earthquake main frequercy to 16.32 rad/s.
basic VII	0.2984(5)	0.8262(12)	3044(1)	1st, 5th stories	descending branch	
major VII	1.285(1)	2.467(12)	3288(1)	as above	descending branch	

In Table 5, the X_{max}, V_{max} and Q_{max} are maximum for the relative floor displacement, absolute floor displacement and story shear force, respectively. In the longitudinal direction, the basic mode shape of vibration is also the principal vibration form in the response of the building. The first and fifth stories are the critical stories. From results of adjusting the input earthquake frequency compared with those from the unadjusting frequency, we reach the same conclusion as in the transverse direction. The corresponding response time histories and restoring forces of the earthquake intensity VII are shown in Fig. 9.

Comparing the transverse response results with the longitudinal response results, we know that the aseismic capability in the longitudinal direction is weaker than that in the transverse direction of the building. The reason is that there are too many openings (windows) in the longitudinal walls. Reducing the number of openings in the walls, adding to the number of walls or increasing the thickness of the walls are methods to improve the aseismic capability in the longitudinal direction.

CONCLUSION

(1) The first mode shape is the principal vibration form in the seismic response of this building in both transverse and longitudinal directions, so that the base shear force method can be used to determine the seismic loads in the design of this type of building.

(2) The effect of geometrical nonlinearity is small and can be neglected in the seismic response analysis for this type of building.

(3) When the main frequency of input earthquake motion is close to the elasto-plastic frequency, the building is in more danger.

(4) As soon as the *i*th story of building gets into elasto-plastic range, the response of the *i*th story will increase quickly and the response of other stories will not increase relatively, therefore, avoiding having any one story becoming damaged earlier than others is necessary. If all stories of the building reach the elasto-plastic state at the same time, the building is the best structure for resisting earthquakes.

(5) The transverse aseismic capability of this building is better than the aseismic capability of the building in the longitudinal direction. The second and sixth stories as well as side stories 2 and 5 are critical stories in the transverse direction. The first and fifth stories are critical stories in the longitudinal direction.

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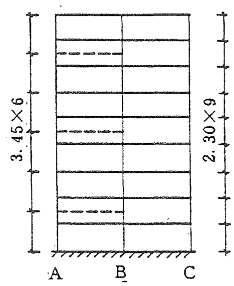

Fig. 6 Transverse Model

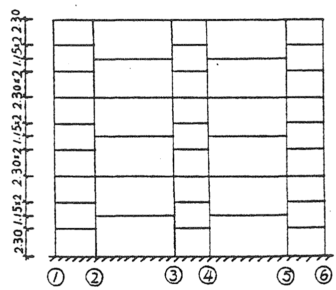

Fig. 7 Longitudinal Model

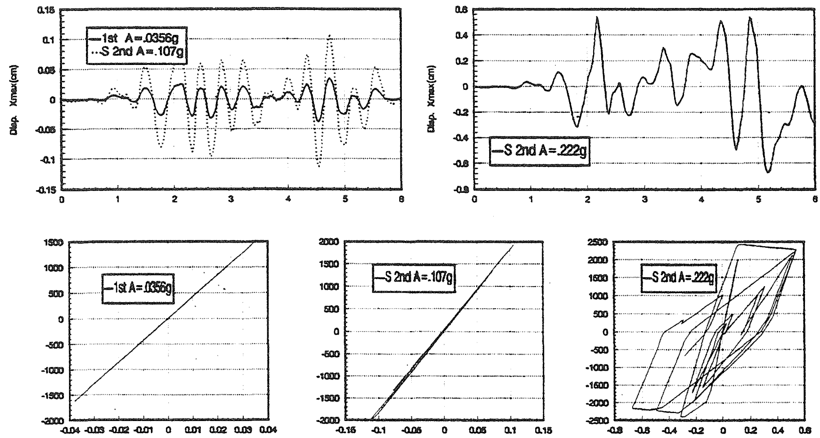

Fig. 8 Displacement Time Histories and Restoring Force

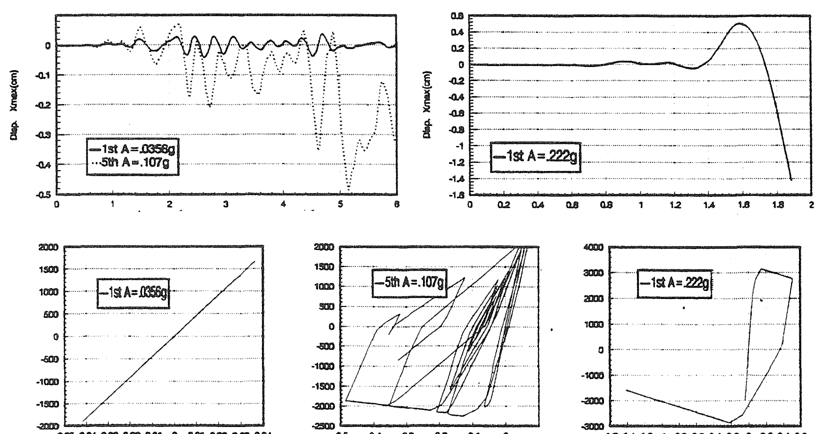

Fig. 9 Displacement Time Histories and Restoring Force