ANALYSIS OF THE BEHAVIOR OF REINFORCED CONCRETE STRUCTURES DURING STRONG EARTHQUAKES BASED ON EMPIRICAL ESTIMATION OF INELASTIC RESTORING FORCE CHARACTERISTICS OF MEMBERS

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SYNOPSIS

Practical and rather rigorous method for inelastic dynamics of multi-story multi-bay reinforced concrete frames is developed for the purpose of the pursuit of collapse process during strong earthquakes. Based on empirical estimation of restoring force characteristics of all constituent members (girders, columns, beam-column connections and shear walls), increments of response values are successively evaluated at each time stage of response calculation using tangent stiffnesses of all members determined from the states of stress at the instant. Several examples of response analysis by this technique are shown, and comparison with those idealized by conventional shear model is discussed in detail.[(1)]

INTRODUCTION

Possibility of structural collapse of low-rise R.C. buildings during strong earthquakes is pointed out in view of experiences of recent earthquake damages (e.g., Tokachi-Oki 1968, San Fernando 1971). In order to discuss the aseismicity of such buildings, we must proceed one step from over-idealized and unrealistic response analysis and pursue analytically the collapse processes of theirs. In case of conventional inelastic earthquake response analysis, structures are modeled to pure shear systems where restoring force characteristics of all constituent members are synthesized and shear force and drift correspond one to one at each story, and hysteresis rule is simply idealized as bi-linear, normal tri-linear, degrading tri-linear and so on. This model extremely reduces the amount of required calculations and provides a fairly appropriate model for weak-column structures. But, in case of general structures, especially weak-girder ones, much unreasonableness or ambiguity arises in the process of modeling, and the reliability of result becomes questionable.

Rigorous treatment of restoring force characteristics of R.C. members is not necessarily impossible, but, for the pursuit of overall collapse processes of structures, excessively detailed treatment is both troublesome and unnecessary. For this purpose, one of the authors formerly proposed a general method to estimate, with practically sufficient accuracy, inelastic deflections of frames subjected to statical lateral load.[(2)(3)(4)] In this paper, with several modifications to this statical analysis technique, one method for inelastic dynamics of genaral R.C. structures is developed. This starts from the end moment - end rotational angle relation for anti-symmetrically deformed girders, columns and shear walls, and shear force - shear deformation relation for beam-column connections and shear walls, instead of stress - strain or moment - curvature relations, and the skeleton curves for these are approximated by tri-linear curves which are to be estimated from a set of empirical equations based on the

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review of many past experimental data. Two-component stiffness model for arbitrary moment distribution (i.e., not anti-symmetric deformation) is derived, and effect of shear and axial deformations, difference between positive and negative bending and influence of dead and live loads are also considered.

DETERMINATION OF INELASTIC CHARACTERISTICS OF CONSTITUENT MEMBERS

Inelastic characteristics of R.C. frames can be fundamentally determined if material (concrete, steel) properties under arbitrary tri-axial stress history and conditions to combine these (crack, bond slippage) are given, but this approach is impossible for the time being due to many unclarified characteristics and troublesome procedures. If we start from the conventional method following the cross-sectional properties of members, we cannot estimate influences of shear crack, opening at crack, bond slippage and so on. Keeping these circumstances in mind, one of the authors presented a method[2][3][4] in which the given structure is firstly divided into girders, columns, beam-column connections and shear walls (Fig.1), and after estimating the inelastic characteristics of each member independently from experimental results, these members are assembled from the compatibility of displacements. That is, parameters describing the relations between end moments and end rotational angles for girders, columns and flexures of shear walls, those between panel distorting moments and shear deformation angles and those between shear forces and shear deformations for shear walls must be given in the form of empirical equations (Fig.2). Under this guiding principle,[5] many available data of member tests were reviewed and one self-contained group of empirical equations for this purpose is now proposed.[5][6][7][8]

Employing these equations, all parameters necessary for tri-linear approximation of skeleton curves for all members can be evaluated. (Fig.3-1). As for the hysteresis rule, there is much room for investigations, but, in this paper, we employ, as merely a working hypothesis for later study, modified degrading tri-linear model the inelastic characteristics of which is different in two directions of loading (Fig.3-2).[1] In this model, after having exceeded the yield point, the stiffness for reversal load degrades aiming at the maximum point so far experienced, and under the repetition of reversal loads at the same deformation amplitude, the equivalent viscous damping coefficient becomes constant.[9]

It is substantially difficult to derive the stiffness matrix for arbitrary moment distribution given the empirical relation between end moment and end rotational angle for anti-symmetric deformation of members. But, as conveniences to solve this beam model problem, two-component model and one-component one are so far known.[2][3][10][11] The properties of these two models were discussed in detail by M.F.Giberson,[12] and he concluded the latter has more versatility compared to the former. The authors have different opinions on this matter which will be reported elsewhere. Here is proposed another model based on the assumption of parabolic distribution of flexural flexibility 1/EI along member axis (Fig.4).[1] (Shear stiffness is assumed to be proportional to the flexural one except for shear walls.) Under this assumption, the following formula is obtained after some simple integrations.

$$\begin{Bmatrix} dM_A' \\ dM_B' \end{Bmatrix} = \begin{bmatrix} f_{11}, f_{12} \\ f_{21}, f_{22} \end{bmatrix}^{-1} \begin{Bmatrix} d\tau_A' \\ d\tau_B' \end{Bmatrix}$$

$$f_{11} = (9\alpha+0.5)f_A + (-\alpha+0.5)f_B + (-8\alpha+\beta-0.5)f_0 + (-3\alpha-0.5)f_{AB}$$

$$f_{12} = f_{21} = (-6\alpha+0.5)(f_A+f_B) + (12\alpha-\beta-0.5)f_0 + (7\alpha-0.5)f_{AB}$$

$$f_{22} = (-\alpha+0.5)f_A + (9\alpha+0.5)f_B + (-8\alpha+\beta-0.5)f_0 + (-3\alpha-0.5)f_{AB}$$

$\alpha=1.5/(9+10\cdot g)$, $\beta=1.5/(1+2\cdot g)$, $g=6\kappa EI_0/(GA\ell'^2)$:parameter for shear deformation
$f_0=(1+2g)\ell'/(6EI_0)$:elastic fexibility
$f_{AB}=\pm\sqrt{(f_A-f_0)(f_B-f_0)}$; for $M_A'M_B'\geq 0$, upper sign ; for $M_A'M_B'\leq 0$, lower sign

In this formula, the symbols f_A, f_B represent the flexibilities at both ends in case of anti-symmetric deformation, and so, the new physical quantity - pseudo anti-symmetric rotation angle $\tilde{\tau}'$ is introduced by the next formulas. $d\tilde{\tau}_A'=f_A\cdot dM_A'$, $d\tilde{\tau}_B'=f_B\cdot dM_B'$ Then we have only to manipulate the relations $M_A'\sim\tilde{\tau}_A'$, $M_B'\sim\tilde{\tau}_B'$ as degrading tri-linear. In case of shear wall members, shear deformation is treated independently from the flexural one, that is, the stiffness matrices for these are obtained from the inversion of the sum of flexibility matrix of above equation (g=0) and that of shear deformation. The influence of dead and live loads is approximately taken into consideration by transferring the origins of restoring force characteristics diagrams. In these discussions, the skeleton curve for each member has been treated as fixed, but in case of columns, for example, the values Mc, My, αy change according to the fluctuation of axial forces. Consideration of this effect is possible but unemployed in this paper.

ESTIMATION OF STRUCTURAL STIFFNESS

Stiffness matrices in the preceding chapter are modified into the following ones in consideration of rigid zones at both ends. (Fig.5)

$$\begin{Bmatrix} dM_A \\ dM_B \end{Bmatrix}=\begin{bmatrix} & K & \end{bmatrix}\begin{Bmatrix} d\tau_A \\ d\tau_B \end{Bmatrix} \quad [K]=[L]^T[k][L] \quad [k]=\begin{bmatrix} f_{11}, f_{12} \\ f_{21}, f_{22} \end{bmatrix}^{-1} \quad [L]=(1-\lambda_A-\lambda_B)^{-1}\begin{bmatrix} 1-\lambda_B, & \lambda_B \\ \lambda_A, & 1-\lambda_A \end{bmatrix}$$

Then, defining rotation angle θ and distorsional angle γ for panel' points as Fig.6-1, stiffness matrices for girders [Bg] become, (Fig.6-2)

$$\begin{Bmatrix} dM_A \\ dM_B \\ dM_A' \\ dM_B' \\ dQ \end{Bmatrix}=[Bg]\begin{Bmatrix} d(\theta_A+\gamma_A) \\ d(\theta_B+\gamma_B) \\ -d\gamma_A \\ -d\gamma_B \\ d\delta \end{Bmatrix} \qquad [Bg]=[Ag]^T[Kg][Ag] \qquad [Ag]=\begin{bmatrix} 1, & 0, & 1-\lambda_A, & -\lambda_B, & -1/\ell \\ 0, & 1, & -\lambda_A, & 1-\lambda_B, & -1/\ell \end{bmatrix}$$

and those for columns and shear walls [Bc] become, (Fig.6-3)

$$\begin{Bmatrix} dM_A \\ dM_B \\ dM_A' \\ dM_B' \\ dQ \\ dN \end{Bmatrix}=[Bc]\begin{Bmatrix} d\theta_A \\ d\theta_B \\ d\gamma_A \\ d\gamma_B \\ d\eta \\ d\varepsilon \end{Bmatrix} \qquad [Bc]=[Ac]^T\begin{bmatrix} [Kc] & 0 \\ 0 & k_\varepsilon \end{bmatrix}[Ac] \qquad [Ac]=\begin{bmatrix} 1, & 0, & 1-\lambda_A, & -\lambda_B, & -1/h, & 0 \\ 0, & 1, & -\lambda_A, & 1-\lambda_B, & -1/h, & 0 \\ 0, & 0, & 0, & 0, & 0, & 1 \end{bmatrix}$$

(Axial stiffness k_ε is treated as elastic.)

Introducing the matrix $[\mathring{B}i]$ whose diagonal is an array of stiffness matrices [Bg], [Bc] for all girders, columns, and shear walls of i-th story, and displacement conversion matrix $[\mathring{A}i]$ of i-th story, the synthesized stiffness matrix for i-th story is represented in the form $[\mathring{A}i]^T[\mathring{B}i][\mathring{A}i]$. This matrix is divided into four submatrices as follows.

$$[\mathring{A}i]^T[\mathring{B}i][\mathring{A}i]=\begin{bmatrix} [\mathring{K}i^t] & [\mathring{C}i] \\ [\mathring{C}i]^T & [\mathring{K}i^b] \end{bmatrix}$$

Finally, the stiffness matrix for an overall structure is given by the following formula under new notation $[\mathring{K}i]=[\mathring{K}i\text{-}1^b]+[\mathring{K}i^t]+[\mathring{K}i^p]$ where the matrix $[\mathring{K}i^p]$ is the one which has each beam-column connection stiffness in the corresponding position.(3)(4)

M(i,j):panel rotating moment, $\overline{M}$(i,j):panel distorting moment
F(i):horizontal force, P(i,j):vertical force

$$\begin{Bmatrix}\{d\mathring{F}_1\}\\ \{d\mathring{F}_2\}\\ \vdots\\ \{d\mathring{F}_{n-1}\}\\ \{d\mathring{F}_n\}\end{Bmatrix} = \begin{bmatrix} [\mathring{K}_1],[\mathring{C}_1], & & & 0 \\ [\mathring{C}_1]^T,[\mathring{K}_2],[\mathring{C}_2] & & & \\ & \ddots & & \\ 0 & & [\mathring{C}_{n-2}]^T,[\mathring{K}_{n-1}],[\mathring{C}_{n-1}] & \\ & & & [\mathring{C}_{n-1}]^T,[\mathring{K}_n] \end{bmatrix} \begin{Bmatrix}\{d\mathring{D}_1\}\\ \{d\mathring{D}_2\}\\ \vdots\\ \{d\mathring{D}_{n-1}\}\\ \{d\mathring{D}_n\}\end{Bmatrix} \qquad \{\mathring{D}_i\}=\begin{Bmatrix}\theta(i,1)\\ \theta(i,2)\\ \vdots\\ \gamma(i,1)\\ \gamma(i,2)\\ u(i)\\ v(i,1)\\ \vdots\end{Bmatrix} \qquad \{\mathring{F}_i\}=\begin{Bmatrix}M(i,1)\\ M(i,2)\\ \vdots\\ \bar{M}(i,1)\\ \bar{M}(i,2)\\ F(i)\\ P(i,1)\\ \vdots\end{Bmatrix}$$

This formula is abbreviated as follows. $\{d\tilde{F}\}=[\tilde{K}]\{d\tilde{D}\}$. In this structural analysis technique originally proposed by R.W.Clough[13], the problem is formulated under the condition that foundations are fixed, and so, when these are pinned, connected by tie-beams or the vertical, horizontal and rotational motions of these are taken into consideration, an imaginary story must be added.

NUMERICAL INTEGRATION OF THE GOVERNING EQUATION

Using the instantaneous stiffness matrix of the structure at each stage of response calculation estimated by the preceding techniques based on the stress level of all constituent members at the instant, governing dynamic equation for (k+1)-th step is:

$$[\tilde{M}]\{d\ddot{\tilde{D}}\}_k^{k+1}+[\tilde{C}]_k^{k+1}\{d\dot{\tilde{D}}\}_k^{k+1}+[\tilde{K}]_k^{k+1}\{d\tilde{D}\}_k^{k+1}+\{\tilde{F}\}_k=-[\tilde{M}]\{e\}\ddot{y}_{k+1}$$

- $[\tilde{M}]$:lumped mass matrix of structure (Inertias only for lateral motions are considered.)
- $[\tilde{C}]_k^{k+1}$:instantaneous damping matrix between k-th step and (k+1)-th one
- $[\tilde{K}]_k^{k+1}$:instantaneous stiffness matrix between k-th step and (k+1)-th one
- $\{\tilde{F}\}_k$:restoring force at k-th step
- $\{e\}$:vector whose components are unit or zero
- $\ddot{y}_{k+1}$:input acceleration at (k+1)-th step

Basically, the stiffness matrix should be deflated into the one between the incremental displacements of stories and corresponding incremental external forces. As this practice is not advantageous from the numerical technique standpoint, the dynamical analysis is performed in the above form. This, however, makes numerical integration very unstable, because inertias for panel point rotations, distortions and vertical motions are very small, and some measures are required to cope with this numerical diversion. Here, this phenomenon is prevented by using β-scheme ($\beta=1/4$, constant acceleration). As the physical meaning is not clear in the form given by N.M.Newmark[14] or S.P.Chan[15], this scheme is modified into the form [pseudo stiffness matrix]X[incremental displacement vector]=[pseudo incremental force vector] following R.W.Clough[16], but, in this technique, the damping matrix enters into the force vector which is disadvantageous to numerical procedures, and so, the algorithm is further rewritten into the type of E.L.Wilson[17]. That is, employing Rayleigh type damping, we proceed in the following steps.

i) conversion into pseudo stiffness matrix
ii) evaluation of pseudo incremental external force vector
iii) calculation of pseudo incremental displacement vector solving simultaneous linear equation
iv) conversion into real incremental displacement vector
v) evaluation of incremental and total response values at (k+1)-th step

OVERALL FLOW OF COMPUTER PROGRAM FOR THE PRESENT ANALYSIS

Block flow chart of computer program for the analysis so far mentioned can be found in Ref.18. The stiffness range judgements of girders, columns and flexures of shear walls are performed by moments at critical sections, those of beam-column connections by panel distorting moments, and those of shears of shear walls by shear forces respectively. The solution of simultaneous linear equations at each step is very easy because of the block tri-diagonal form of coefficient matrix, and Choleski's method is applied to the inversion of symmetric submatrices that appear in the forward and backward steps of sweeping-out techniques.

EXAMPLES OF ANALYSIS

As an example of analysis, a typical R.C. building (three storied and without shear walls) designed in accordance with AIJ - Str. Stand.[19] is shown. An example of a building with shear walls can be found in Ref.1. The plan and the structural elevation are displayed in Fig.7, Fig.8. Longitudinal direction is treated as infinitely uniform, and, from the symmetry of configurations, a coupled system of only two frames (outer and inner) is analysed. For transverse direction, only No.2 frame (3 spans) is picked out. In application of empirical equations previously referred to, $j=(7/8)d$ (d:effective depth) is used for the determination of panel volumes and lengths of rigid zones, and increase of stiffness and strength and modification of secant stiffness reduction coefficient for girders with slabs are considered.

In addition to the rigorous analysis using data obtained in this manner, approximate ones by conventional pure shear system are conducted, and, for this practice, following three methods are employed. [Modeling-1] Each panel point is decided whether it is weak-column or weak-girder, comparing the sum of yield moments of girders and that of columns. In the latter case, it is transformed into pseudo weak-column by distributing the sum of yield moments of girders by elastic column stiffnesses, and, using these values, the yield shear force is evaluated. The crack shear force is calculated by regarding crackings as pseudo yieldings. Elastic and tangent semi-plastic stiffnesses are estimated by K.Muto's D-method using elastic stiffness ratios of members and secant plastic ones at the stage of pseudo collapse of each column respectively. Story characteristics is obtained from the sum of those of all columns at each story. [Modeling-2] As to the yield story shear forces, the collapse coupling among stories is attached much importance. That is, they are determined from equilibrium at overall collapse mechanism of structures. And crack story shear forces are likewise evaluated treating crackings as pseudo yieldings. Tangent semi-plastic stiffnesses of stories are estimated by D-method using those of columns and girders. [Modeling-3] By the technique submitted by one of the authors and T.Sugano,[2][3][4] story shear force - story drift relations are rigorously calculated by pursuing crackings and yieldings of all constituent members under monotonously increasing lateral loads. For the shear model analysis, these relations are optimally tri-linearized. Relations between story shear forces and story drifts by these methods are displayed in Fig.9. From this figure, it is concluded that Modeling-2 is much better than Modeling-1 for a simple and practical way to guess the rigorous result by Modeling-3.

Four dynamical models thus prepared are excited by base acceleration recorded at Hachinohe Harbor (NS component) in Tokachi-Oki Earthquake 1968 whose maximum acceleration 225 gal is enlarged to 500 gal. Results of response calculations are given in Fig.10 (collapse process), Table 1 (maximum responses) and Fig.11 (time histories of story drifts).

All shear modeling results are not a little different from the rigorous one (especially the story where largest story drift occurrs). On the other hand, the three shear modelings show roughly same trend. We cannot answer promptly whether shear modeling is essentially unjust for this kind of structures, or more appropriate shear modeling must be invented. At present, however, we suggest the former statement is probably true judging from the result of Modeling-3.

To investigate the influence of spectral characteristics of input excitation and the variation of response values due to its fluctuation, the outer frame in longitudinal direction (connected to ground by swaying spring and dashpot) is subjected to five sample artificial earthquake motions which are taken at random from each of two groups of waves generated by setting power spectrum roughly fixed and phase one random. This nonstationary process A(t) is assumed 'separable' or 'uniformly modulated'. i.e., $A(t)=F(t)Z(t)$

where $F(t)$: nonstationary deterministic modulating function

$0 \le t \le 2$, $F(t)=0.5t$; $2 \le t \le 10$, $F(t)=1$; $10 \le t$, $F(t)=\exp[-0.0693(t-10)]$

$Z(t)$: stationary random function i.e., $F(20)=0.5$

As a simple idealization of power spectrum of strong motions, K.Kanai and H.Tajimi suggested the next formula, which was supported by specral analyses (e.g., by S.C.Liu[20]) and adopted by many investigators.

$$S(\omega)=[1+4H_g^2(\omega/\omega_g)^2]/[\{1-(\omega/\omega_g)^2\}^2+4H_g^2(\omega/\omega_g)^2]S_0 \qquad (\omega_g=2\pi/T_g)$$

Here, two different ground conditions are examined.

$T_g=0.4$ sec., $H_g=0.50$, $\sigma_g=\sqrt{E[\{Z(t)\}^2]}=160$ gal

$T_g=0.8$ sec., $H_g=0.50$, $\sigma_g=\sqrt{E[\{Z(t)\}^2]}=120$ gal

The ratio of these two intensities of acceleration was tentatively determined consulting K.Kanai's empirical formula on the relation between maximum accelerations and predominant periods of ground. (Fig.12, 13) The variation, the mean values and standard deviations by sampled input excitations and groups of time histories of story drift responses are shown in Table 2, Fig.14. Larger variation of response values than that expected from elastic analysis is easily seen, especially in Tg=0.8 sec.. The mean values in Tg=0.8 sec. is larger than the one in Tg=0.4 sec..

CONCLUSIONS

In this paper, the outline of a technique for dynamical analysis based on inelastic characteristics of R.C. constituent members and a few typical examples have been presented, and discussions were mainly focused on the reliability of response calculation by conventional shear model. It may be concluded the shear model idealization brings about no reliable answer if fairly exact estimation of story drift responses are required. As pointed out already by some researchers, the variation of inelastic responses due to input fluctuation are larger than that expected from elastic analysis.

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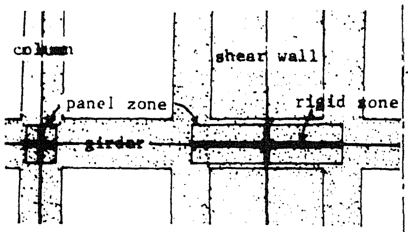


FIG.1 TYPICAL FRAMING MEMBERS

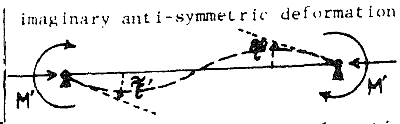


Fig.2-1 Flexural and Shear Deformation of Column & Girder, Flexural Deformation of Shear Wall

$\tilde{\tau}'$: pseudo anti-symmetric rotation angle

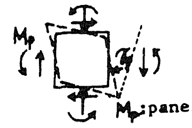


Fig.2-2 Shear Deformation of Beam-Column Connection

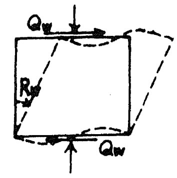


Fig.2-3 Shear Deformation of Shear Wall

FIG.2 DEFORMATIONS OF MEMBERS

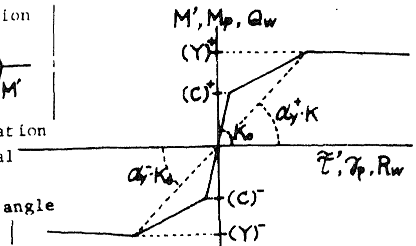


Fig.3-1 Skeleton Curve for Each Member

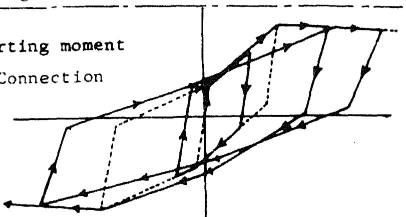


Fig.3-2 Hysteresis Rule for Each Member (Modified Degrading Tri-Linear Type)

FIG.3 SKELETON CURVES AND HYSTERESIS RULES

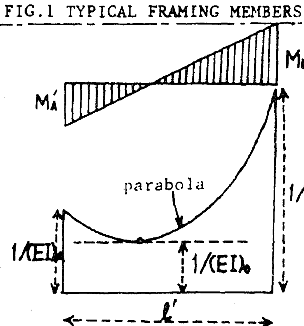


Fig.4-1 Case of $M_A' \cdot M_B' > 0$

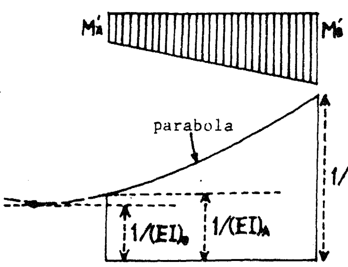
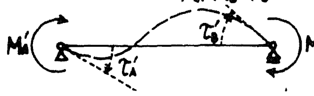


Fig.4-2 Case of $M_A' \cdot M_B' < 0$



$(EI)_0$: elastic flexural stiffness

FIG.4 ASSUMPTION OF FLEXURAL FLEXIBILITY DISTRIBUTION

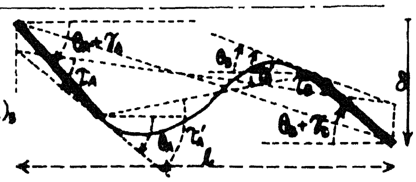


Fig.6-2 Deformation of Girder

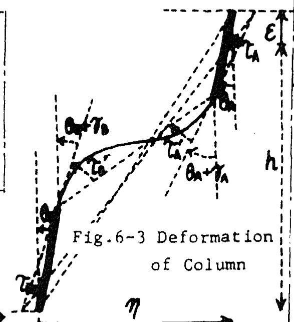


Fig.6-3 Deformation of Column

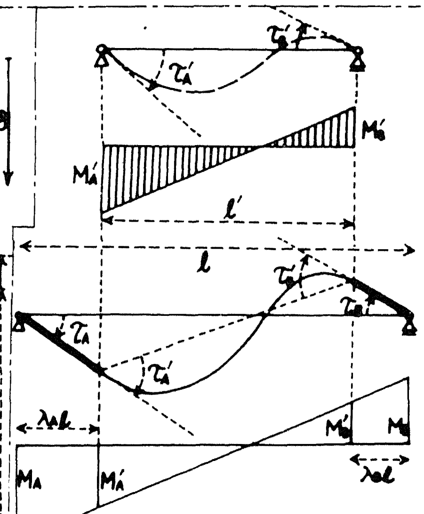


FIG.5 CONSIDERATION OF RIGID ZONES AT BOTH ENDS

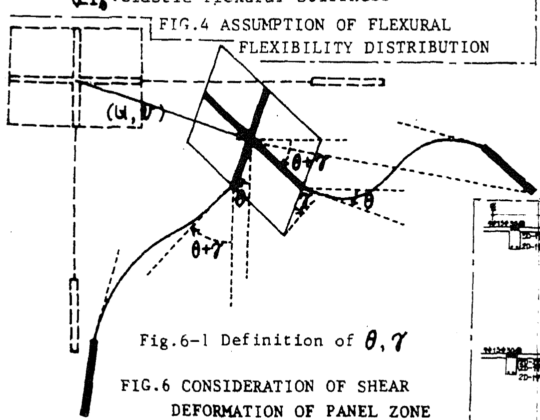


Fig.6-1 Definition of θ, γ

FIG.6 CONSIDERATION OF SHEAR DEFORMATION OF PANEL ZONE

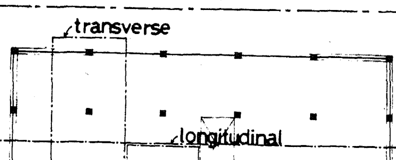


Fig.7 Plan

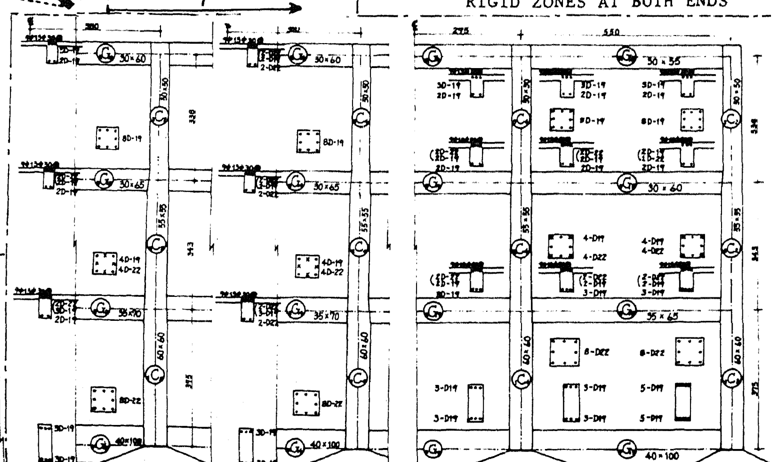
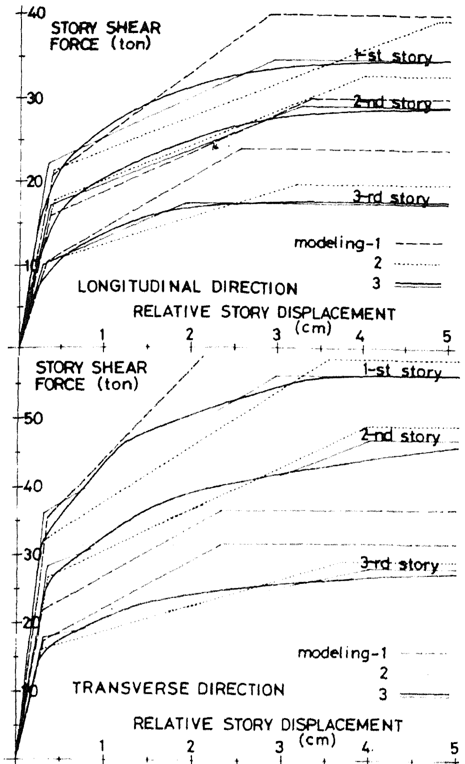
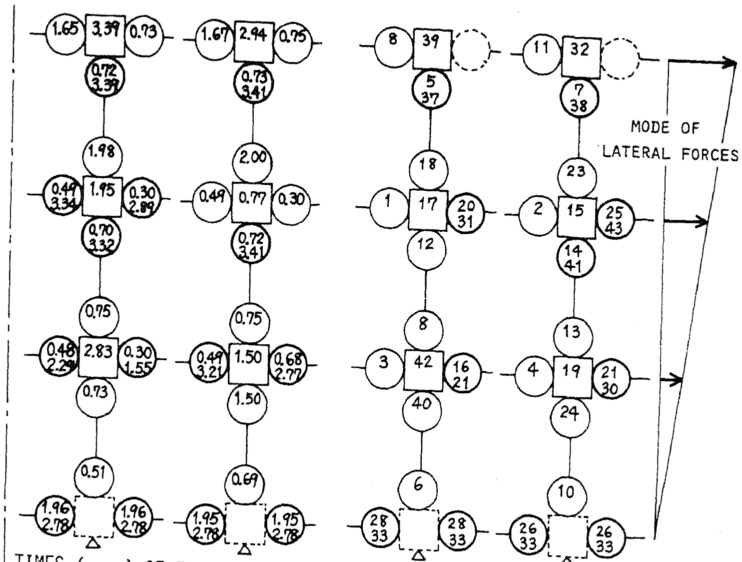


Fig.8 Structural Elevation



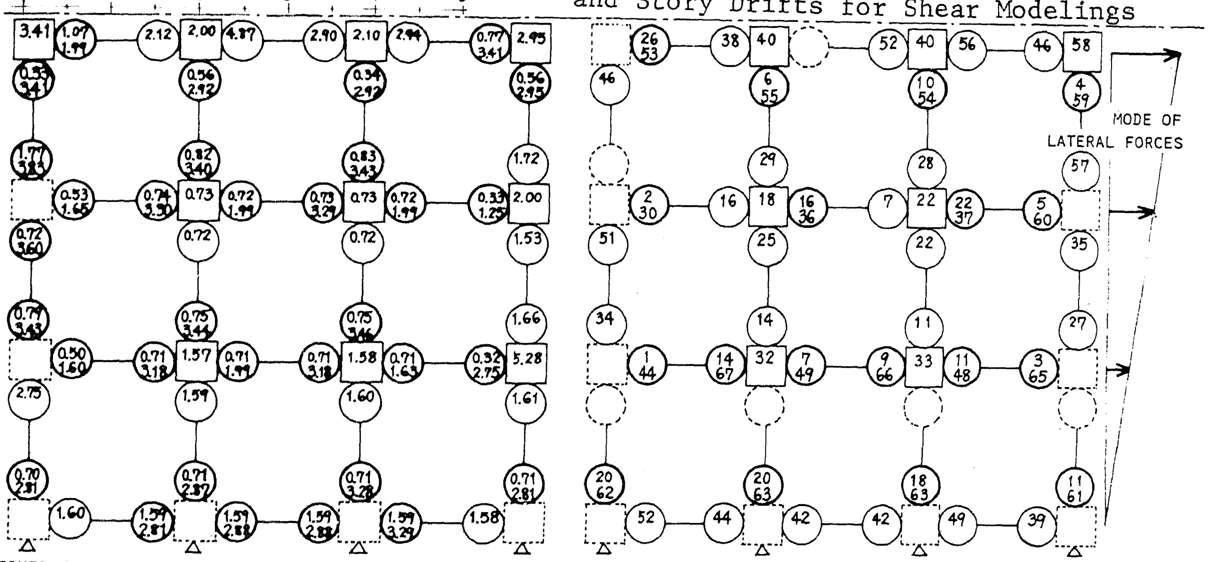


TIMES (SEC.) OF FIRST CRACKING AND YIELDING (DYNAMICS)
HACHINOHE HARBOR, NS, 500 GAL
UPPER NUMERALS:CRACKING

ORDERS OF FIRST CRACKING AND YIELDING (STATICS)
MONOTONOUSLY INCREASING LATERAL FORCES
LOWER NUMERALS:YIELDING

Fig.10-1 Collapse Process of Longitudinal Direction

Fig.9 Relations between Story Shear Forces and Story Drifts for Shear Modelings



TIMES (SEC.) OF FIRST CRACKING AND YIELDING (DYNAMICS)
HACHINOHE HARBOR, NS, 500 GAL

ORDERS OF FIRST CRACKING AND YIELDING (STATICS)
MONOTONOUSLY INCREASING LATERAL FORCES

UPPER NUMERALS:CRACKING LOWER NUMERALS:YIELDING

Fig.10-2 Collapse Process of Lateral Direction

Table 1 Maximum Responses

		LONGITUDINAL DIRECTION				TRANSVERSE DIRECTION			
		SHEAR MODEL - 1	SHEAR MODEL - 2	SHEAR MODEL - 3	RIGOROUS MODEL	SHEAR MODEL - 1	SHEAR MODEL - 2	SHEAR MODEL - 3	RIGOROUS MODEL
Maximum Relative Story Displacement - cm	δ_3	1.81 (3.49)	1.88 (3.51)	1.41 (2.82)	2.48 (3.47)	1.07 (2.12)	2.62 (3.65)	2.10 (2.87)	6.16 (3.71)
	δ_2	4.20 (3.46)	4.15 (3.45)	2.90 (3.46)	4.47 (3.44)	8.28 (3.74)	4.28 (3.68)	3.47 (3.68)	5.09 (3.67)
	δ_1	7.75 (3.41)	7.48 (3.39)	6.91 (3.41)	4.35 (3.41)	5.94 (3.66)	7.89 (3.64)	7.23 (3.64)	3.64 (2.88)
Ductility Factor	μ_3	0.71	0.59	0.73	-	0.46	0.72	0.53	-
	μ_2	1.24	1.11	0.89	-	3.57	1.08	0.87	-
	μ_1	2.69	1.54	2.35	-	2.13	2.26	2.50	-
Maximum Shear Force - ton	Q_3	17.8 (3.49)	15.2 (3.51)	14.8 (2.82)	17.6 (3.48)	22.6 (2.11)	25.1 (3.65)	22.9 (2.87)	35.2 (3.51)
	Q_2	29.6 (-)	32.3 (-)	27.2 (3.46)	28.8 (3.45)	36.3 (-)	48.5 (-)	44.2 (3.67)	53.4 (2.90)
	Q_1	39.4 (-)	38.8 (-)	34.2 (-)	36.5 (2.87)	67.4 (-)	58.2 (-)	56.0 (-)	70.8 (3.63)
Shear Coefficient	q_3	0.40	0.34	0.33	0.40	0.25	0.28	0.26	0.40
	q_2	0.32	0.35	0.29	0.31	0.20	0.26	0.24	0.29
	q_1	0.27	0.27	0.24	0.25	0.23	0.20	0.19	0.25

Numerals in the parenseses indicate occurrence time.

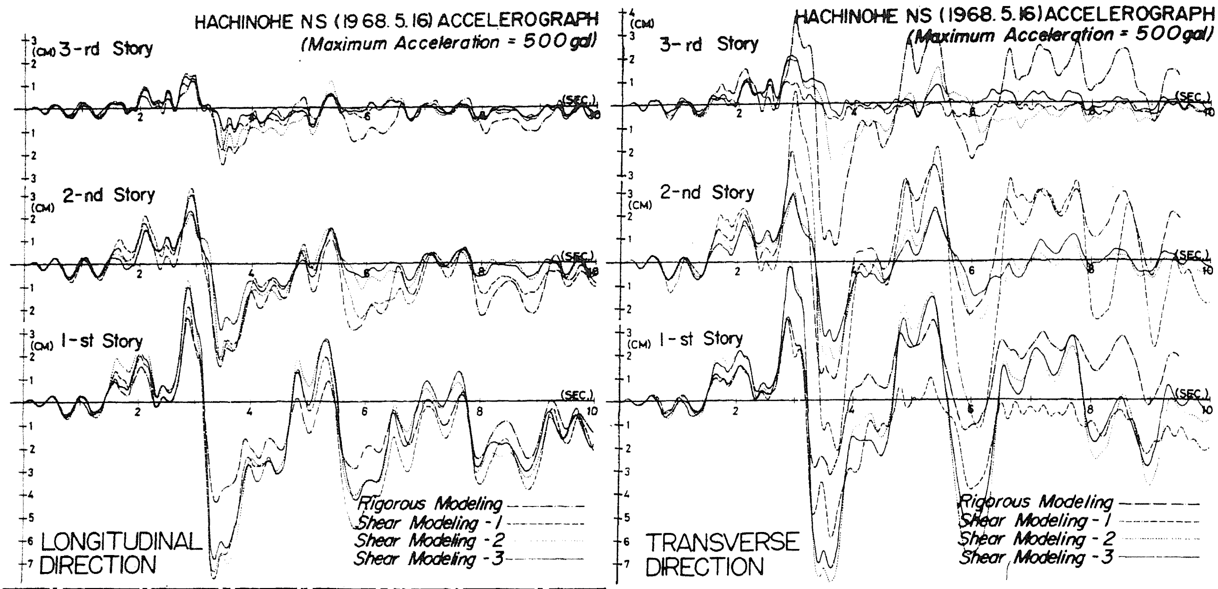


Fig.11 Time Histories of Story Drifts

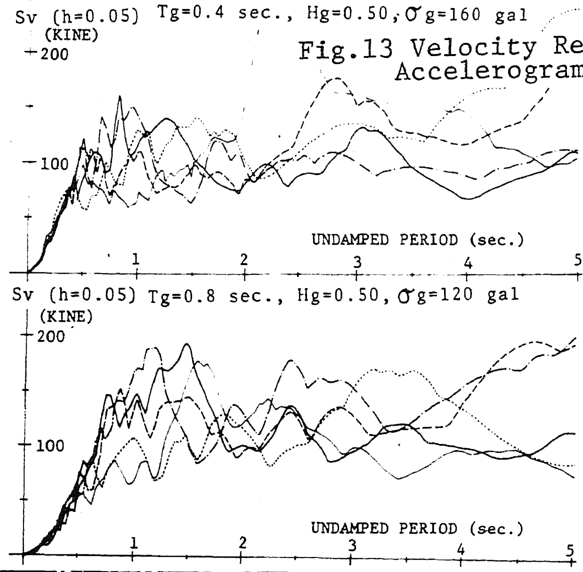


Fig.13 Velocity Response Spectra of Generated Artificial Accelerograms

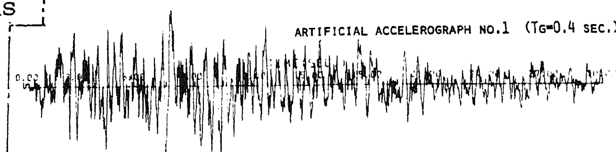


Fig.12 One Sample of Generated Artificial Accelerogram

Fig.14 Time Histories of Story Drifts due to Various Excitations

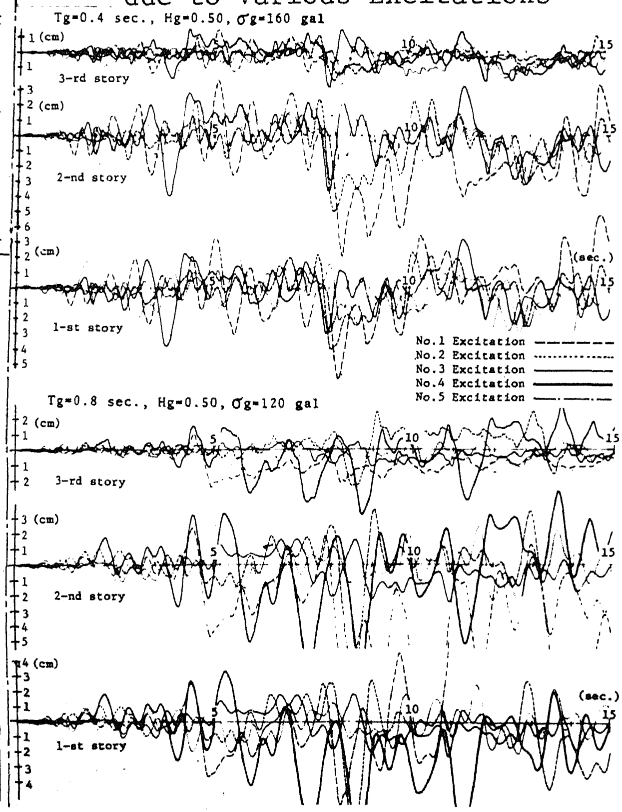


Table 2 Maximum Responses due to Various Excitations

Tg=0.4 sec., Hg=0.50, σg=160 gal

Identification of Excitation	Maximum Acceleration (gal)	Maximum Relative Story Displacement (cm)		
		1-st Story	2-nd Story	3-rd Story
No.1	437.76 (7.800sec.)	3.926 (8.070sec.)	4.948 (8.105sec.)	1.898 (8.135sec.)
No.2	487.09 (2.200sec.)	2.830 (8.000sec.)	3.034 (12.740sec.)	1.490 (12.760sec.)
No.3	529.68 (8.120sec.)	2.661 (12.480sec.)	3.092 (12.475sec.)	2.158 (8.325sec.)
No.4	492.67 (7.760sec.)	3.799 (3.880sec.)	3.927 (3.895sec.)	2.335 (8.010sec.)
No.5	523.65 (8.020sec.)	5.897 (8.220sec.)	7.900 (8.305sec.)	1.549 (5.125sec.)
Mean Value	494.18 gal	3.822 cm	4.580 cm	1.886 cm
Standard Deviation	36.62 gal	1.289 cm	2.012 cm	0.369 cm
Coefficient of Variation	0.074	0.337	0.439	0.196

Tg=0.8 sec., Hg=0.50, σg=120 gal

Identification of Excitation	Maximum Acceleration (gal)	Maximum Relative Story Displacement (cm)		
		1-st Story	2-nd Story	3-rd Story
No.1	364.99 (8.180sec.)	7.198 ([illegible].625sec.)	6.939 ([illegible].615sec.)	[illegible]78 (13.000sec.)
No.2	265.[illegible]8 (9.460sec.)	2.448 (11.680sec.)	2.467 (11.705sec.)	1.520 (12.060sec.)
No.3	281.15 (8.520sec.)	2.278 (10.960sec.)	2.493 (10.975sec.)	1.110 (11.015sec.)
No.4	368.21 (5.680sec.)	7.690 ([illegible].765sec.)	6.634 (8.730sec.)	4.306 (8.770sec.)
No.5	417.55 (8.020sec.)	15.444 (8.230sec.)	11.828 (14.965sec.)	6.548 (5.010sec.)
Mean Value	339.43 gal	7.012 cm	6.072 cm	3.192 cm
Standard Deviation	64.18 gal	3.306 cm	2.592 cm	1.490 cm
Coefficient of Variation	0.189	0.471	0.427	0.467