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STATISTICAL INVESTIGATION OF DYNAMIC BEHAVIOR OF BRACED FRAME-FOUNDATION SYSTEMS DURING SEVERE EARTHQUAKES

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ABSTRACT

This paper addresses the statistical investigation of the effects of foundation interaction upon the elastic-plastic behavior of braced frames during severe earthquakes. The dynamic behavior of braced frames can be strongly affected by the soil-foundation condition. In order to investigate this topic, the response sensitivity factor(R.S.F.) which presents the ratio of the responses of the structure-foundation system to those of the rigid based system is employed. On the assumption of the log-normal distribution for the R.S.F., the expectations of R.S.F. are estimated through a series of numerical analyses for the planar braced steel frame-foundation systems, and the statistical tendency of the dynamic behavior of these systems is discussed with reference to the influence of the dynamic characteristics of earthquakes, the fundamental period of superstructures, and the sort of soil-foundations.

KEYWORDS

Structure-foundation interaction; foundation spring model; response sensitivity factor; braced frame; energy-absorption; statistical investigation; log-normal distribution; dynamic differential settlement

INTRODUCTION

In the rigid-based braced frame during severe earthquakes, the large axial forces occur in the brace-members and concurrently most of the horizontal load induced by the ground motion is shared by the brace-members of the frame. However, in reality it may be very rare that the foundation of buildings is rigid. Subsequently, the bending moment in the beams and columns adjacent to the braced substructure can be generally underestimated. Considering the effect of the soil-foundation on the behavior of the super-structure, the dynamic differential settlement and the re-distribution of the forces of the members consisting of the frame can be sufficiently expected to occur, and the foundation springs bound with the braced substructure may be overloaded by the axial forces of the columns and braces due to the ground motion. Also, the behavior of the braced frame-foundation systems is significantly influenced by the characteristics of input ground motions, the fundamental period of superstructures, and the sort of soil-foundations.(Jennings *et al.*,1973) Therefore in order to further discuss the effects of foundation interaction, it may be more preferable to use the statistical procedure for the analytical results of structure-foundation systems. The object of this investigation is to elucidate the statistical tendency in the elastic-plastic behavior of the braced steel frame-foundation systems during severe earthquakes. For that purpose, the response sensitivity factor(R.S.F.) which presents the ratio of the structural response with foundation interaction to that of the rigid-based structure is defined, and a series of elastic-plastic dynamic analyses for the fundamental period of superstructures, and the sort of soil-foundations is performed. And then, the frequency distribution and expected value of R.S.F. are evaluated from these results, and the statistical tendency of R.S.F. is discussed.

ANALYTICAL METHOD AND DEFINITION OF R.S.F.

In this analytical method, the flexural members of a superstructure are assumed to have tri-linear flexural springs at their both ends as given in Fig.1. Based upon the assumption that brace-members can resist only against tensile force, the hysteresis rule of the brace members is presented as shown in Fig.2. And also, the soil-foundation of a system is modeled as two nonlinear springs which are independent each other and present tri-linear hysteric behavior for the horizontal direction and poli-linear one for vertical direction as shown in Fig.3, respectively. By using the general stiffness matrix for the superstructure and the stiffness coefficient of the foundation springs, the governing equation of motion for the structure-foundation system can be obtained.(Kuroda, 1986)

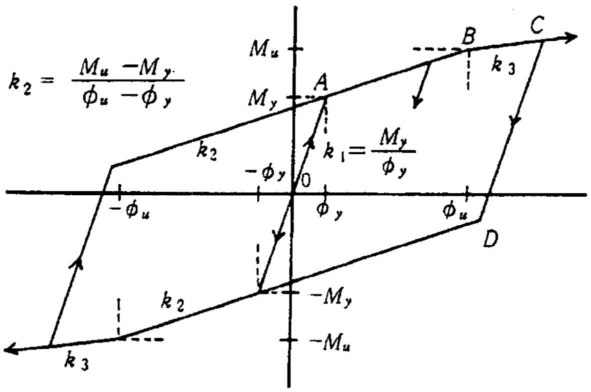


Fig. 1 Moment-Rotation Relation of Flexural Member

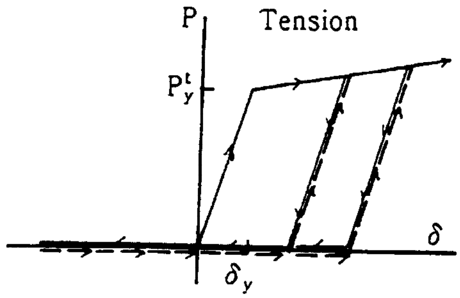
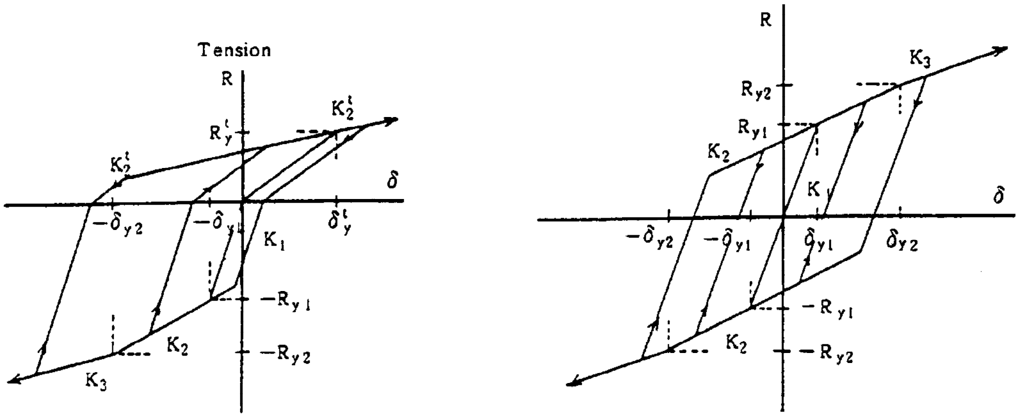


Fig. 2 Force-Deformation Relation of Brace Member

Denoting the structural response of the structure-foundation system and the rigid-based system as Rf and Ro, respectively, the response sensitivity factor Ar can be defined by Eq.(1).

$$Ar = Rf / Ro \qquad (1)$$

where R presents displacement, velocity, acceleration, energy-absorption, and ductility in an analyzed system. Taking into account the specified conditions, a series of deterministic analyses is performed for the ground motions with different dynamic characteristics. From the results of these analyses, the R.S.F. for the structural response is evaluated by Eq.(1). Denoting the incremental interval of Ar as Δ Ar and counting the number of Ar which lies on the incremental interval Δ Ar between Ar,i and Ar,i+1, the frequency distribution and expected values are obtained under the assumption that the log-normal distribution for the R.S.F. Ar can be postulated(Ang *et al.*, 1975).



(a) Vertical Spring (b) Horizontal Spring

Fig. 3 Reaction-Displacement Relation of Foundation

NUMERICAL ANALYSES AND DISCUSSIONS

Premises and parameters of numerical analyses

In order to statistically investigate the dynamic behavior of braced frame-foundation systems, the model system shown in Fig.4 is analyzed.

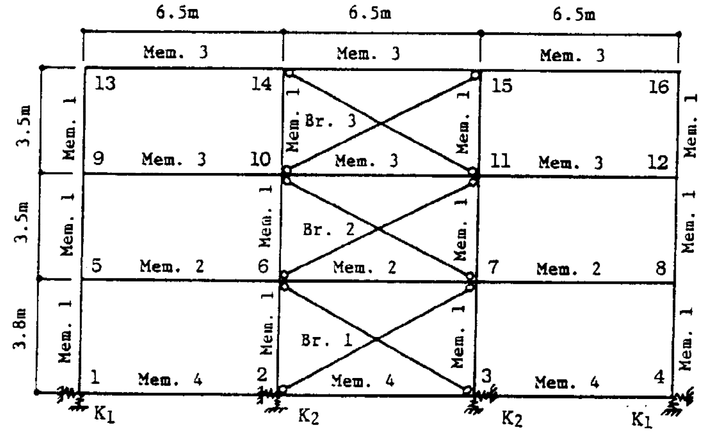


Fig. 4 Analytical Model

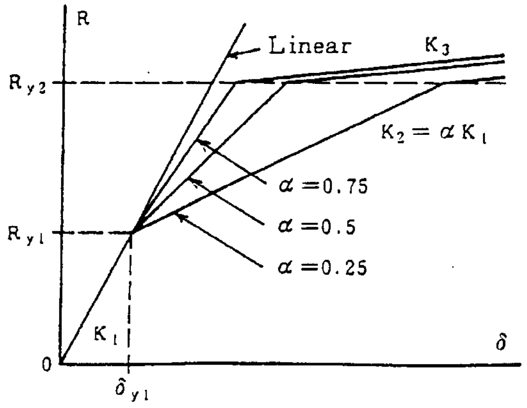


Fig. 5 Initial foundation Property

Table 1 Properties of Flexural Member

Mem. No.	1	2	3	4
My (tf•m)	23.1	22.2	18.0	30.0
Mu (tf•m)	25.4	24.2	19.8	33.0
Φ y ($\times 10^{-3}$)	0.1313	0.1644	0.1761	0.0173
Φ u ($\times 10^{-3}$)	0.1183	0.1481	0.1586	0.0156
k_3	36.26	27.84	21.08	367.4

The mechanical properties of the flexural members and brace members of the superstructure which are constant independently to the fundamental period are listed in Table 1 and 2. The fundamental periods of the initial elastic rigid-based superstructures are adjusted to 0.25, 0.5, 0.75, and 1.0

seconds by modifying the elastic stiffness of a superstructure. Three soil-foundation conditions accounting for (A)soft, (B)medium, and (C)stiff soils are considered. Their mechanical characteristics are presented in Table 3. Moreover, the four cases in which the ratio of the first-order plastic stiffness to the elastic stiffness of a foundation spring lies on 0.25, 0.5, 0.75, and 1.0(linear) for each type of foundation springs are considered regarding to their nonlinearity as they are shown in Fig. 5.

Table 2 Properties of brace Member

Mem. No.	Br-1	Br-2	Br-3
Ny (tf)	45.6	39.6	22.37
δ y (cm)	0.86	0.84	0.84

Table 3 Properties of Foundation Spring

Spring No.	1			2		
Spring Type	A	B	C	A	B	C
Horizontal						
K1 (tf/cm)	27.0	45.0	72.0	48.6	81.0	129.6
K3 (tf/cm)	1.35	2.25	3.60	2.43	4.05	6.48
Ry1 (tf)	24.5	24.5	24.5	44.1	44.1	44.1
Ry2 (tf)	52.6	52.6	52.6	94.6	94.6	94.6
Vertical						
K1 (tf/cm)	90.0	150.0	240.0	162.0	270.0	432.0
K3 (tf/cm)	4.5	7.5	12.0	8.1	13.5	21.6
Ry1 (tf)	81.9	81.9	81.9	147.2	147.2	147.2
Ry2 (tf)	175.5	175.5	175.5	315.5	315.5	315.5
K1 (tf/cm)	0.45	0.75	1.20	0.81	1.35	2.16
K3 (tf/cm)	0.09	0.15	0.24	0.162	0.27	0.432
Ry (tf)	8.2	8.2	8.2	14.7	14.7	14.7

The joint masses of braced frame are constant irrespective of time and are given as follows.
$m_1 = m_4 = 0.0153$ tonf·sec^2 /cm, $m_2 = m_3 = 0.0275$ tonf·sec^2 /cm
$m_i = 0.0148$ tonf·sec^2 /cm (i=6,7,11,12,14,15), $m_i = 0.0266$ tonf·sec^2 /cm (i=5,8,9,12,13,16)
As the input ground motions, the following six earthquakes are employed.

1. El Centro NS May 18, 1940	2. Taft EW July 21, 1952
3. Ferndale EW December 21, 1954	4. Olympia EW April 13, 1949
5. Hachinohe NS May 16, 1968	6. Miyagiken-Oki EW 12, June, 1978

The peak ground accelerations are enlarged so that the velocities of applied earthquakes become equal to 50 kines that is specified as the level II ground motion in the aseismic design of high-rise buildings in Japan, and the first 30 seconds components of them are imposed on the systems at the bottom of the foundation. In the numerical integration of the differential equation of motion by the Newmark's algorithm, the time-step is chosen as Δ t =0.005 seconds. And the damping ratio is also assumed to be equal to 2 %. By employing the step-by-step procedure for nonlinear equation of motion, the dynamic analyses for rigid-based systems and flexibly supported systems are performed under the above-mentioned conditions.

Results of Rigid-Based Systems From the analyses according to the above-described premises, the representative results for the rigid-based systems are given in Table 4. In the table, each parameter presents the following maximum response of the structure.

Dmax = Displacement of the top-floor level

Est=Energy of structural force in the equation of motion
Emem = Total energy of the flexural and brace members of a superstructure
Ebri=Energy absorption of individual brace-member, respectively.

For all earthquake employed in this paper, the maximum displacement extensively increases as the fundamental period of the rigid-based system becomes longer. The results of Emem show the maximum value for the period of 0.25 seconds in the Olympia and for the period of 0.5 seconds in the El Centro and Taft earthquakes. Also, the maximum response of energy absorption in the Miyagiken-Oki earthquake occurs for 0.75 seconds. On the contrary, those for the Ferndale and Hachinohe earthquakes increase with longer fundamental period.

Table 4 Max. Displacement and Absorbed Energy in Fixed-Based Frames

(Dmax in cm, Energy in tonf•m)

El Centro

Period	0.25	0.50	0.75	1.00
Dmax	3.67	12.78	19.38	33.01
Est	5.38	15.89	16.08	28.83
Emem	3.32	5.64	3.86	4.47
Ebr1	0.809	1.055	0.845	0.895
Ebr2	0.671	1.036	0.720	0.758
Ebr3	0.358	0.631	0.462	0.539

Taft

Period	0.25	0.50	0.75	1.00
Dmax	2.64	8.43	13.44	18.87
Est	2.87	7.16	8.00	9.28
Emem	1.59	3.42	2.86	2.46
Ebr1	0.399	0.739	0.632	0.531
Ebr2	0.285	0.601	0.465	0.423
Ebr3	0.142	0.379	0.326	0.239

Ferndale

Period	0.25	0.50	0.75	1.00
Dmax	1.46	6.52	15.38	25.75
Est	0.87	4.42	10.82	16.30
Emem	0.51	2.19	3.11	3.16
Ebr1	0.125	0.571	0.693	0.672
Ebr2	0.096	0.424	0.570	0.577
Ebr3	0.051	0.212	0.271	0.342

Olympia

Period	0.25	0.50	0.75	1.00
Dmax	4.72	13.25	15.32	25.73
Est	10.38	17.90	10.01	16.42
Emem	7.22	6.41	3.32	3.32
Ebr1	1.163	1.197	0.622	0.709
Ebr2	0.966	1.059	0.556	0.563
Ebr3	0.517	0.593	0.416	0.361

Hachinohe

Period	0.25	0.50	0.75	1.00
Dmax	1.77	4.70	10.45	24.61
Est	1.30	2.20	5.19	15.53
Emem	0.77	1.38	2.04	3.06
Ebr1	0.207	0.321	0.515	0.709
Ebr2	0.149	0.258	0.352	0.544
Ebr3	0.071	0.160	0.210	0.299

Miyagiken-Oki

Period	0.25	0.50	0.75	1.00
Dmax	2.87	8.38	23.32	28.92
Est	3.39	6.94	23.02	19.80
Emem	2.01	3.20	4.79	3.86
Ebr1	0.508	0.689	1.018	0.825
Ebr2	0.356	0.587	0.841	0.631
Ebr3	0.200	0.313	0.478	0.444

Frequency Distribution of R.S.F. The R.S.F. for the given conditions is evaluated from the analytical results by using Eq.(1). The frequency distributions of the R.S.F. of the energy absorption of superstructures, Est are presented in Fig. 6 relating to the kind of a foundation spring. And the regression equations approximated under the assumption of the log-normal distribution for the R.S.F. are also described in the figure.

As shown in Fig.6, the R.S.F. of Est ranges from 0.4 to 10.2 for the soft foundation A, 0.4 to 13.4 for the medium foundation B, and 0.4 to 11.8 for the stiff foundation C. Some of the R.S.F. for the medium and stiff soils show larger response than for the soft soil due to extreme values under particular conditions. However, the R.S.F.s for stiffer soils have a remarkable peak between 0.8 and 1.4. The standard deviation in ln Ar becomes larger with softer soil that is 2.93 for soft soil, 2.52 for medium soil, and 2.02 for stiff soil. Subsequently, the frequency distribution curve is more gentle as the soil becomes softer.

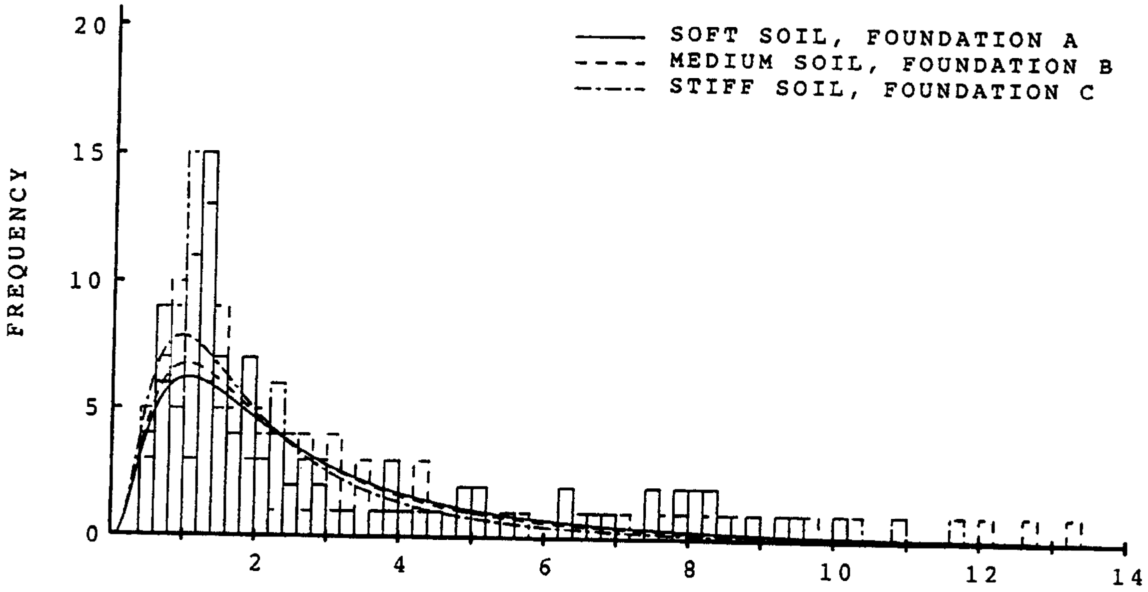


Fig. 6 Frequency Distribution of Response Sensitivity Factor in Energy by Structural Inner Force

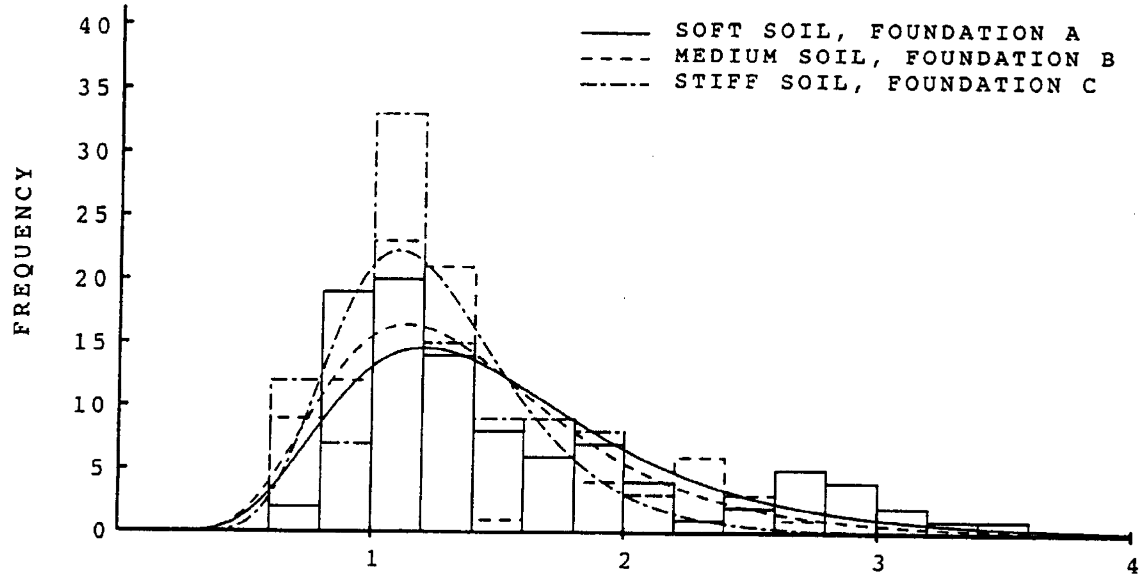


Fig. 7 Frequency Distribution of Response Sensitivity Factor in Top-Floor Displacement

In Fig.7, the frequency distribution of R.S.F. of top-floor displacement, Dmax is presented. The tendency above-mentioned in the energy by structural inner force is apparently observed. In general, it is clear from these frequency distributions of the R.S.F. of Emem and Dmax that its range becomes wider with softer foundation condition.

Expectations of R.S.F. In Table 5, 6, 7, and 8, the representative expectations of the R.S.F. for the log-normal distribution are given. In general, the expected values of the maximum displacement show a tendency to become larger than 1.0 because the braced frame-foundation system should be longer in period and more flexible than the rigid-based system. In regard to the input ground motions as shown in Table 5, the expectations of R.S.F. of Est and Emem show prominently large values for the Taft and Miyagiken-Oki earthquakes. Because the energy absorption in brace members decreases due to foundation interaction, the energy absorption of flexural members should increase more seriously than those for another earthquakes.

Table 5 Expectation of Response Sensitivity Factor as to Earthquakes

Earthquake	El Centro	Taft	Ferndale	Olympia	Hachinohe	Miyagiken-Oki
Dmax	1.16	1.60	1.50	1.17	1.57	1.43
Est	1.93	3.83	1.82	2.17	3.11	3.55
Emem	2.56	4.38	1.47	2.38	3.12	4.76
Ebr1	0.44	0.66	0.69	0.55	0.88	0.59
Ebr2	0.47	0.69	0.77	0.59	0.88	0.61
Ebr3	0.45	0.93	0.85	0.82	0.98	0.63

As to the effect of the fundamental period of a superstructure, the expectations of Dmax, Est, Emem become prominently larger as the period of a superstructure becomes shorter. On the contrary, the expectations of R.S.F. in the energy absorption of brace members decrease. Therefore, it can be pronounced that the braced frames of shorter periods should be significantly affected by the foundation interaction.

Table 6 Expectation of Response Sensitivity Factor as to Period of Fixed Based Frame

T1(Second)	0.25	0.50	0.75	1.00
Dmax	2.12	1.36	1.19	0.95
Est	6.42	2.08	1.52	0.91
Emem	7.22	2.46	1.66	1.10
Ebr1	0.43	0.69	0.72	0.70
Ebr2	0.36	0.72	0.83	0.76
Ebr3	0.43	0.89	0.89	0.89

Table 7 Expectation of Response Sensitivity Factor as to Soils

Foundation	A	B	C
Dmax	1.54	1.42	1.26
Est	2.98	2.80	2.42
Emem	3.52	3.21	2.60
Ebr1	0.61	0.64	0.65
Ebr2	0.62	0.69	0.69
Ebr3	0.72	0.79	0.81

Table 8 Expectation of Response Sensitivity Factor as to Foundation- α

FND.- α	Linear	0.75	0.50	0.25
Dmax	1.42	1.39	1.38	1.38
Est	2.39	2.72	3.11	3.11
Emem	3.64	3.10	2.58	2.58
Ebr1	0.68	0.64	0.59	0.59
Ebr2	0.71	0.68	0.61	0.61
Ebr3	0.84	0.79	0.71	0.71

In Table 7, the expectations of R.S.F. as to the kind of soils are presented. The large axial forces of the column and brace members cause the dynamic differential settlement in a system. Subsequently, the energy absorption in flexural springs of members abruptly increases. Because the dynamic differential settlement becomes larger with softer soils, the expectations of R.S.F. of Dmax, Est, and Emem increase as the foundation springs become more flexible. Especially, Emem which represents the index of the structural damage comes to 3.52 for soft soil, 3.21 for medium soil, and 2.60 for stiff soil. Conversely, the expected values of R.S.F. of the energy absorption in brace members decrease prominently with softer soils because the braced substructure becomes more flexible due to the foundation interaction.

The expectations of R.S.F. as to the foundation spring - α which represents the plastic characteristics of foundation spring are listed in Table 8. As the α-value become smaller, the Expectations of R.S.F. of Dmax, Emem, and Ebri decrease and the efects of the foundation interaction can be mitigated.

Increase of energy absorption in flexural members As above-described, it is strongly pronounced that the re-distribution of the energy absorption in the system should occur due to the foundation interaction. The changes of energy absorption of representative members for the El Centro earthquake are shown in Table 9. As the kind of soils becomes softer, the energy absorption remarkably increases in the flexural springs of members. The energy absorption in lower beams shows prominently large value and that of $\mathrm{Mem}_{1\text{-}2}$ at 2-end comes to 6.601 tonf•m. Therefore, the capacity of energy absorption in such members should be particularly required. Furthermore, it is clearly indicated that the small second stiffness of foundation spring can relieve the drastic increase in flexural springs of members.

Table 9 Energy of Flexural Spring of Member (tonf•m)
El Centro NS 1940 Period = 0.25 sec.

Found. Cond.	Rigid	A=LIN	B=LIN	C=LIN	A=0.25
$\mathrm{Mem}_{1\text{-}5}$ 1-end	0.007	1.902	0.705	0.746	0.908
$\mathrm{Mem}_{5\text{-}6}$ 5-end	0.016	1.750	0.588	0.709	0.779
$\mathrm{Mem}_{5\text{-}6}$ 6-end	0.002	1.524	0.442	0.589	0.590
$\mathrm{Mem}_{9\text{-}10}$ 9-end	0.040	1.352	0.443	0.656	0.539
$\mathrm{Mem}_{1\text{-}2}$ 2-end	0.000	6.601	3.679	2.946	5.025

CONCLUSION

In this paper, in order to investigate statistically the dynamic behavior of the braced frame-foundation systems, the frequency distribution and expected value of R.S.F. under the assumption of the log-normal distribution have been evaluated through a series of analyses. And by using them, the dynamic behavior of the braced frame-foundation systems are discussed. Consequently, the following conclusions can be summarized from these analyses and discussions.

(1) The frequency distribution curve is more gentle as the soil becomes softer. Therefore, it can be indicated that the R.S.F. of Emem and Dmax ranges more widely with softer foundation condition.

(2) The expectations of R.S.F. in the energy absorption of brace members generally decrease by the effect of the foundation interaction.

(3) Because the dynamic differential settlement becomes larger with softer soils, the expectations of R.S.F. of Dmax, Est, and Emem increase as the foundation springs become more flexible.

(4) It is strongly pronounced that the re-distribution of the energy absorption in the system should occur due to the foundation interaction. The energy absorption in brace members always decreases, and contrarily that in flexural members becomes seriously large by the effect of foundation interaction.

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