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POSTER SESSION 9

Structural Engineering in Earthquake Zones

Structures de génie civil en zones sismiques

Konstruktiver Ingenieurbau in Erdbebengebieten

Coordinator: R.S. Stilwell, Canada

Retrofit System to Enhance Earthquake Resistance

Siegfried F. STIEMER
Assoc. Prof. Univ.
of British Columbia
Vancouver, BC, Canada

Experimental and analytical investigations of a retrofit-system to enhance the earthquake resistance in existing buildings and structures have been conducted in the University of British Columbia, Department of Civil Engineering, Earthquake Simulator Laboratory. The system consists of a combination of well known base-isolation techniques, a new base storey design, and newly developed miltidirectional energy absorber made of mild steel. The research results indicated the usefulness of this system and proved, that it can be implemented in existing buildings without reduction of the structural integrity of the building and without major modifications of the original design.

The key idea of the system is to separate the building from the existing ground motion by roller bearings or sliding pads and restricting it from excessive displacements by solid state steel energy absorbers. Extensive experimental tests have been conducted to prove the principles, to correlate analytical programmes, and to devise an engineering approach to the design of the system as a whole and the energy absorbers in particular. A transparent design procedure is now available, which enables the engineer to design this retrofit-system for different earthquake regions and for different structures.

One of the focal points of the research project was to show, how a retrofit base-isolation system could be implemented in steel buildings without additional blind-storeys or double foundations as used in recent proposals by others. This results in considerable cost savings, which make the new approach very attractive for a broad spectrum of buildings for which until now an update in respect to earthquake resistance was believed to be too expensive.

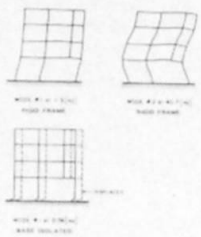
High importance was placed on the investigations and development of reliable, inexpensive, and replaceable energy absorbers. They have to restrain the building against wind loads and, in case being subjected to major earthquake loadings, have to prevent the building from excessive displacements. These devices are designed to deform elastically under minor loads (such as wind) or displacements (such as thermal expansion) and to deform plastically when severe loads (such as earthquakes) are imposed on the building or bridge. The energy absorbers allow displacements in all directions, however work most efficiently for displacements in the horizontal plane. They consist of curved plates of hot-rolled mild steel plates. An engineering method was introduced by the author for the practical design of the steel energy absorbers and for predetermining the number of cycles to failure.

In the experimental studies steel roller bearings were used because they had a very low friction coefficient without becoming unstable for large displacements. However, the design approach is valid for other low friction sliding devices, too, e.g. for neoprene bearings often used in bridge applications.

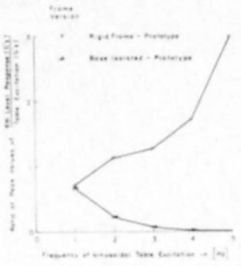
A wide range of applications can be foreseen for the solid state steel energy absorbers, particularly in the area where currently viscous or friction dampers are used (bridges, nuclear power plants), which can add considerable costs, constant maintenance, and can cause problems of implementation because of bulkiness.

RETROFIT-SYSTEM TO ENHANCE EARTHQUAKE RESISTANCE

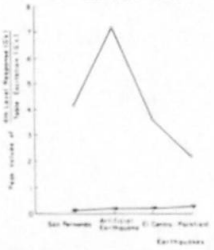
ANALYTICAL RESULTS
OF TESTED MODEL

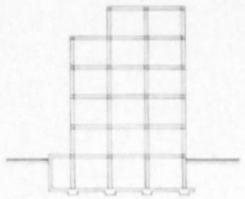


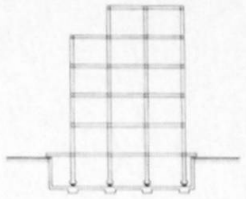
REDUCTION
OF SINUSOIDAL EXCITATION

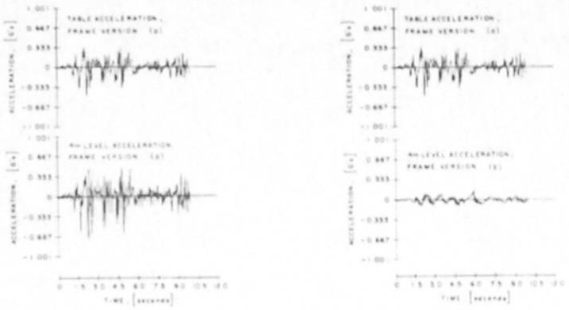


REDUCTION
OF EARTHQUAKE EXCITATION

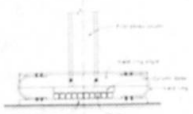




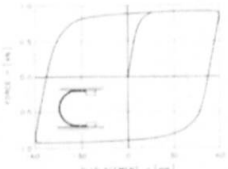




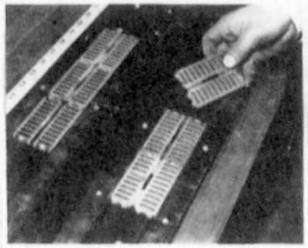
TIME HISTORY OF EXCITATION AND RESPONSE TO EL CENTRO EARTHQUAKE
FOR RIGID AND BASE-ISOLATED FRAME



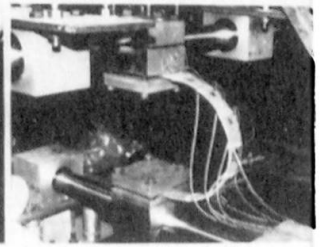
SCHEMATIC VIEW OF PROPOSED
ISOLATION PAD WITH STEEL ROLLER
BEARING AND STEEL YIELD RINGS



HYSTERESIS OF THE YIELD RINGS
USED IN MODEL TESTS



ROLLER BEARING PADS
FOR BASE ISOLATION RETROFIT



YIELD RING
IN DYNAMIC TESTING

EXPERIMENTAL INVESTIGATION
OF MODEL BUILDING ON EARTHQUAKE SIMULATOR

MULTIDIRECTIONAL YIELD RINGS
IN LOW CYCLE FATIGUE TESTING

Application of Diagonal Reinforcement to Reinforced Concrete and Masonry Short Columns

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The paper describes the use of diagonal reinforcement in concrete members to prevent brittle shear failure and to ensure ductile behavior in earthquake resistant structures. The use of diagonal reinforcement was proposed by T. Paulay to provide ductility in coupling beams and has been used in practical buildings. However, past earthquakes have revealed that brittle shear failure takes place more frequently in short columns than in beams, and can lead to overall structural failure. The authors have used the diagonally reinforcing system in short reinforced concrete columns and in wall-columns of reinforced concrete-grouted masonry, and have developed design formulas to predict the maximum load carrying capacity and ductility.

Figure 1 shows the hysteresis loops obtained from the preliminary tests of diagonally reinforced short columns. It can be clearly seen that the behavior of the diagonally reinforced column is far better than that of an ordinary reinforced concrete column, with respect not only to maximum load capacity but also to deformability without load degradation, shape of hysteresis loops, and energy dissipation. The superiority of the diagonal reinforcement has been confirmed by a series of experiments using full scale specimens, as indicated in Figs. 2 - 4. The physical model used to predict the ultimate load carrying capacity consists of two basic resisting mechanisms, a beam mechanism and an arch mechanism. This is shown in Fig. 5. Ultimate load capacity is obtained by the summation of the load capacities of the basic mechanisms according to the generalized addition theorem. Correlation between the experimental and predicted load capacities can be seen in Fig. 6. The proposed diagonal arrangement of main reinforcement for reinforced concrete column has been used in apartment buildings designed by the National Housing Cooperation of Japan, as shown in Figs. 7 and 8. In addition to its use in ordinary short columns, the diagonal system is very effective in the case of columns which are shortened in their clear height by wing wall, as in the school buildings shown in Fig. 9. The system is also applicable to concrete-grouted brick masonry wall. Ductile behavior can be anticipated even in brick masonry structures, as shown in Fig. 10.

The following conclusion can be drawn from the experimental and theoretical work:

(1) The diagonal reinforcement system is very effective in increasing the shear capacity and in improving the earthquake-resistant characteristics, such as ductility and energy dissipation, of reinforced concrete columns and masonry wall-columns.
(2) The maximum shear capacity, ductility and energy dissipation can be controlled by the amount of diagonal reinforcement.
(3) At least sixty percent of the main reinforcement should be arranged diagonally in columns to guarantee ductile performance during earthquakes.
(4) The maximum load carrying capacity of diagonally reinforced members can be predicted with reasonable accuracy by the proposed analytical method.

APPLICATION OF DIAGONAL REINFORCEMENT TO REINFORCED CONCRETE AND MASONRY SHORT COLUMNS

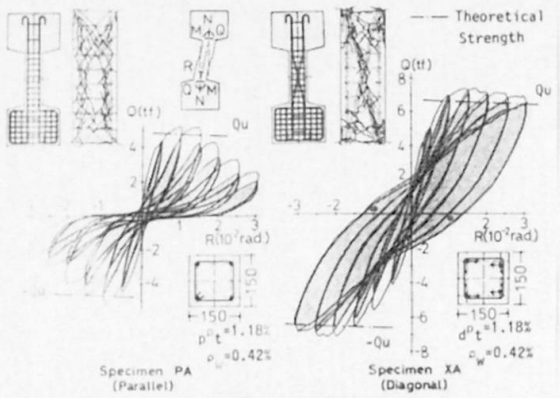

Fig.1 PRELIMINARY TEST

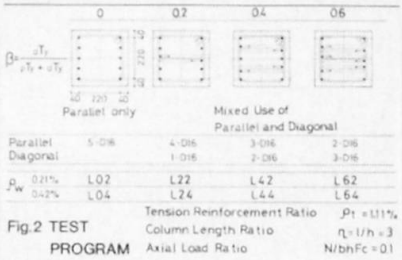

Fig.2 TEST PROGRAM

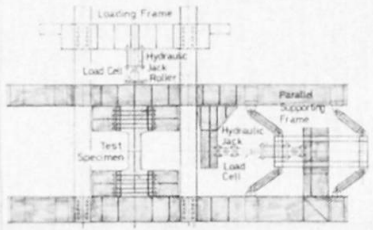

Fig.3 LOADING SET UP

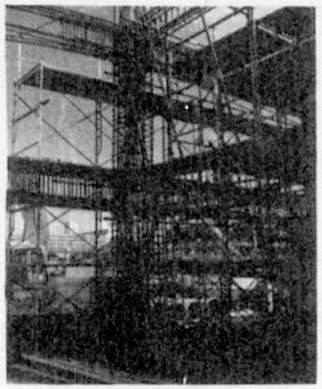

Fig.7 CONSTRUCTION SITE

Fig.8 REINFORCING ARRANGEMENT

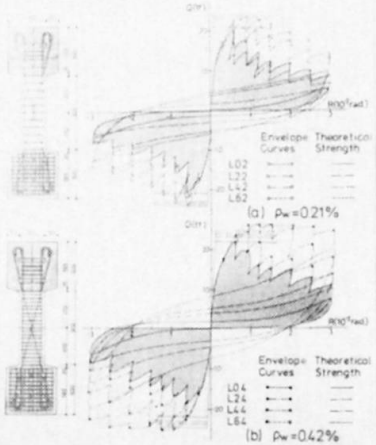

Fig.4 ENVELOPE CURVES

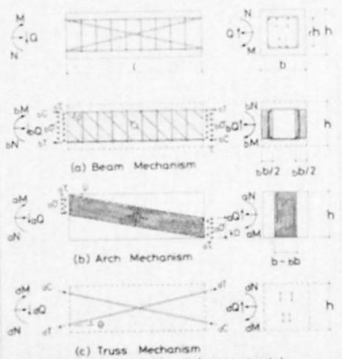

Fig.5 RESISTANT MECHANISM

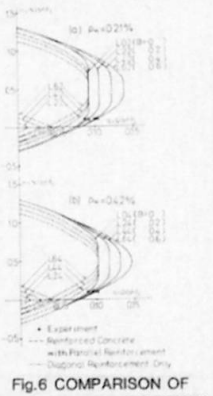

Fig.6 COMPARISON OF TEST AND THEORY

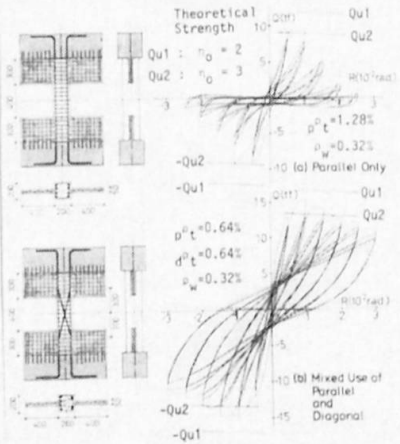

Fig.9 APPLICATION TO SHORT COLUMNS

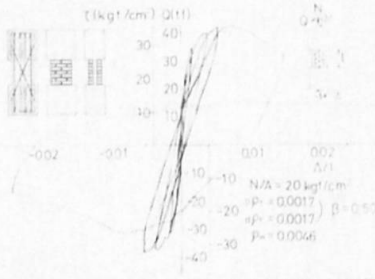

Fig.10 APPLICATION TO A MASONRY COLUMN

Inelastic Aseismic Design of Reinforced Concrete Bridges

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Professor
Kyoto University
Kyoto, Japan

Hirokazu IEMURA
Assoc. Prof.
Kyoto University
Kyoto, Japan

As stated in the introductory report by Professor Tassios, it is an essential approach for earthquake resistant design of most structures to produce a structure capable of responding to moderate shaking (more than a few times expected intensity of excitation in its life time) without damage, and capable of resisting to unlikely event of very strong shaking without seriously endangering the occupants. In the first case, it is satisfactory to adopt the "allowable stress" design method for the specified intensity of earthquake motion. However, in the second case, it is neccesary to propose reasonable design methods based on earthquake response properties of structures beyond yielding limit approaching to failure.

INELASTIC DESIGN CODES

In the first part of this study, present two inelastic earthquake resistant design codes of reinforced concrete (RC) bridges by Japan Road Association are explained and some problems in application are pointed out. One inelastic design is a static method by which it should be checked whether sectional forces due to 30% increased earthquake loads (130% of the intensity for elastic design) are less than ultimate strength of those forces. The other is a dynamic method with use of equivalent linearization technique. By this method, it should be checked whether ductility factor response due to 30% increased earthquake dynamic loads is less than the allowable ductility factor which is defined as one third of the ultimate ductility of members. In appliction of these two inelastic design methods to RC bridge structures (especially bridge structures), it was found that the second ductility requirement is generally hard to be satisfied even though several problems relating to values of equivalent damping factor and spectral intensity for dynamic response analysis and definition of the allowable ductility factor have been pointed out. Research efforts are needed to answer these problems.

HYBRID EXPERIMENTS OF EARTHQUAKE RESPONSE

In the second part of this study, results of the newly developed online hybrid experiment related to above mentioned problems are described. In the experiment, earthquake response is calculated by a digital computer adopting the real hysteretic restoring force of a RC bending structural element directly measured from a loading actuator. Therefore, accurate estimation of not only earthquake response but also deterioration process of structural properties has become possible. Effects of reinforcement ratio, axial stress, kinds and amount of tie-hoops and strength of concrete to inelastic earthquake response are examined. Process of partitioning of earthquake input energy to kinetic, potential and absorbed energy by hysteresis loops is also investigated as a measure for deterioration of structural properties. From the experiments, it is found that the ductility requirement by the present code is so conservative that new design codes based on earthquake input energy shall be developed.

INELASTIC ASEISMIC DESIGN OF REINFORCED CONCRETE BRIDGES

INELASTIC ASEISMIC DESIGN FORMATS

PRESENT CODES

1) DUCTILITY REQUIREMENTS

$$\mu_{max} \leq \mu_a$$

IN THIS REQUIREMENT, DUCTILITY FACTOR RESPONSE IS CALCULATED BY AN EQUIVALENT LINEARIZATION TECHNIQUE. IT SHOULD BE CHECKED WHETHER DUCTILITY FACTOR RESPONSE DUE TO 30% INCREASED EARTHQUAKE DYNAMIC LOADS IS LESS THAN THE ALLOWAELE DUCTILITY FACTOR WHICH SHOULD BE DEFINED FROM THE EXPERIMENTS.

2) ULTIMATE STRENGTH REQUIREMENTS

$$M_{max} \leq M_u$$

THIS REQUIREMENT IS A STATIC METHOD BY WHICH IT SHOULD BE CHECKED WHETHER SECTIONAL FORCES DUE TO 30% INCREASED EARTHQUAKE LOADS ARE LESS THAN ULTIMATE STRENGTH OF THOSE SECTIONS.

RECOMMENDED ENERGY REQUIREMENTS

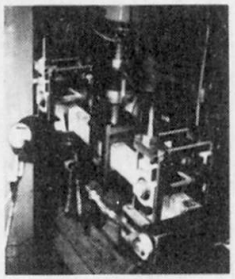
$$E_{total} \leq E_a$$

FROM ON-LINE HYBRID EXPERIMENTS, IT IS VERIFIED THAT HYSTERETIC ABSORBED ENERGY IS THE BEST PARAMETER TO REPRESENT DEGREE OF DAMAGE OF RC STRUCTURES. HENCE IT IS RECOMMENDED TO CHECK WHETHER TOTAL ABSORBED HYSTERETIC ENERGY IS LESS THAN THE ALLOWABLE VALUE WHICH SHOULD BE DEFINED FROM THE EXPERIMENTS.

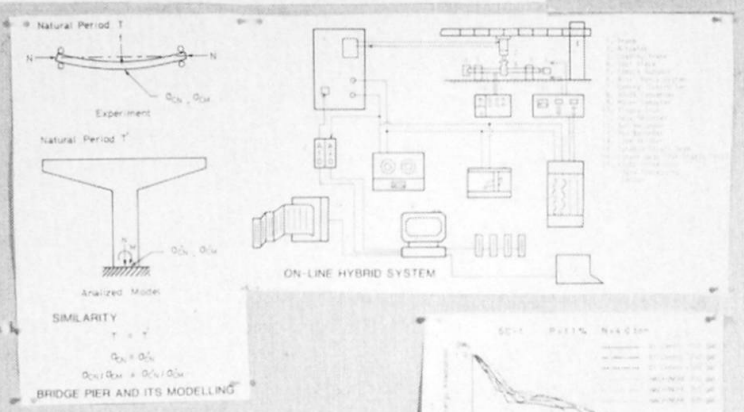
ON-LINE HYBRID EXPERIMENTS FOR HYSTERETIC EARTHQUAKE RESPONSE OF RC BRIDGE PIERS

SYSTEM EARTHQUAKE RESPONSE IS CALCULATED BY A DIGITAL COMPUTER ADOPTING THE REAL HYSTERETIC RESTORING FORCE OF A REINFORCED CONCRETE BENDING STRUCTURAL ELEMENTS DIRECTLY MEASURED FROM A LOADING ACTUATOR

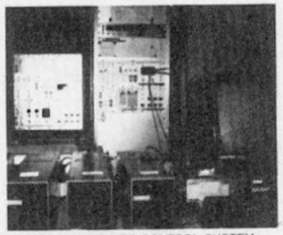
OBJECTS INVESTIGATION OF CONVENTIONAL AND NEW FAILURE CRITERIA TO DETERMINE INELASTIC ASEISMIC DESIGN FORMATS



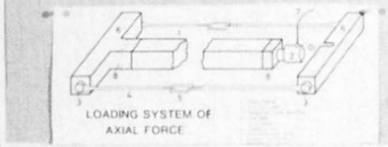
LOADING SYSTEM WITH ACTUATOR



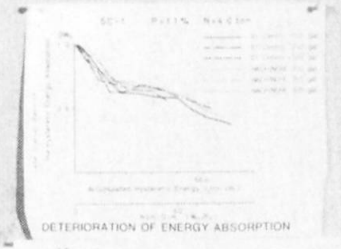
BRIDGE PIER AND ITS MODELLING



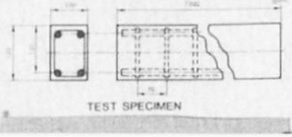
MICRO COMPUTER CONTROL SYSTEM
A TO D AND D TO A CONVERTOR



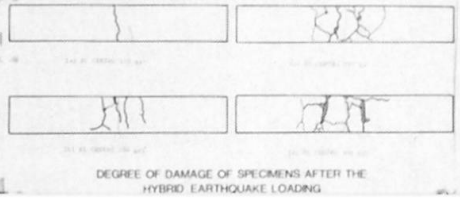
LOADING SYSTEM OF AXIAL FORCE



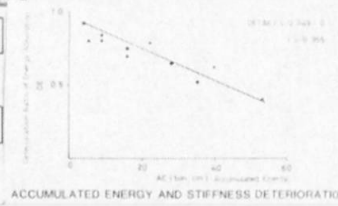
DETERIORATION OF ENERGY ABSORPTION



TEST SPECIMEN



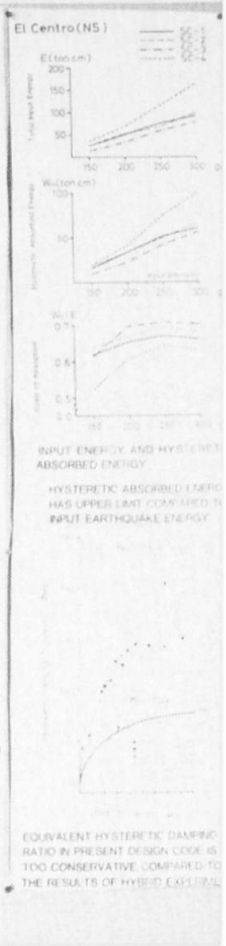
DEGREE OF DAMAGE OF SPECIMENS AFTER THE HYBRID EARTHQUAKE LOADING



ACCUMULATED ENERGY AND STIFFNESS DETERIORATION

HYSTERESIS LOOPS WITH AND WITHOUT AXIAL FORCE

PARTITIONING OF INPUT EARTHQUAKE ENERGY TO
W_K KINETIC ENERGY
W_C ABSOBED ENERGY BY VISCOUS DAMPING
W_H HYSTERETICALLY ABSORBED AND POTENTIAL ENERGY



INPUT ENERGY AND HYSTERETIC ABSORBED ENERGY

HYSTERETIC ABSORBED ENERGY HAS UPPER LIMIT COMPARED TO INPUT EARTHQUAKE ENERGY

EQUIVALENT HYSTERETIC DAMPING RATIO IN PRESENT DESIGN CODE IS TOO CONSERVATIVE COMPARED TO THE RESULTS OF HYBRID EXPERIMENTS

Cable Damper of Meiko-Nishi Cable-Stayed Bridge

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Nagoya, Japan

Michio TAKAHASHI
Nihon Doro Kodan
Hiroshima, Japan

Nobukazu KAJITA
Shin-Nippon-Giken Eng.
Tokyo, Japan

1. GENERAL

Meiko-Nishi Bridge, which is under construction in Nagoya Harbor, Japan, is a steel cable-stayed bridge with three spans of (175+405+175m) . The box-girder type with an orthotropic deck is a trapezoidal 3-cell box-sections with the depth of 2.8 m. Pylons are A-frame fixed to piers. The longitudinal cable configuration is the fan type of twelve stay cables. This bridge was closed on July 16, 1984, and will be completed on April, 1985.

2. EARTHQUAKE RESISTANCE DESIGN

The construction site which is located in the strong seismic zone is in rather poor geotechnical condition. In the earthquake resistance design, following earthquakes are considered. The one is expected to occur 2～3 times for 100 years with M=8.0 for the epicentral distance of 100 km, and the other is expected to occur 4～5 times with M=7.0 for the epicentral distance of 50 km, in which M is the magnitude. Therefore, the design of pylons and substructures needs special considerations to the longitudinal earthquake force. The degree of horizontal connection between pylon and girder is one of the most important influence factors against the earthquake response, because of the large mass of the stiffened girders. Meiko-Nishi Bridge has been installed horizontal cables, called cable dampers, in the connection of pylon and girder to reduce the horizontal forces to the pylon from the stiffened girders. The earthquake force will be reduced, because the cable dampers give lower natural frequency and larger system damping to the structure.

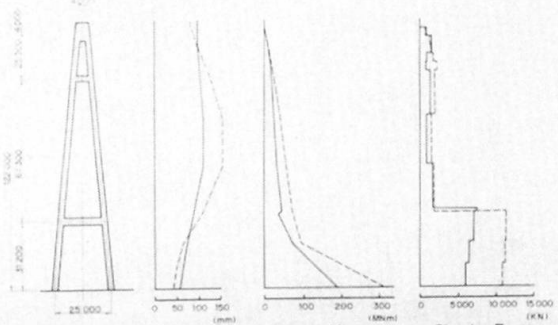
3. DIMENSION OF CABLE DAMPER

Material	2 bandles of 61 galvanized 7-wire strands
Length (L)	43.2m
Area (A)	$62.6\ cm^2 \times 2 = 125.2\ cm^2$
Modulus of Elasticity (E)	$1.9 \times 10^7\ t/m^2$
EA/L	$5.5 \times 10^3\ t/m$
Prestretching	Center Span ——— 440 t
	Side Span ——— 415 t

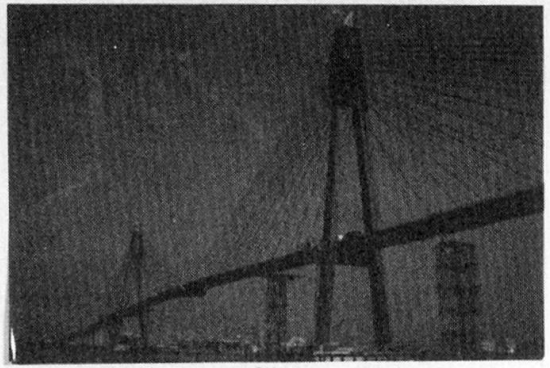
CABLE DAMPER OF MEIKO-NISHI CABLE STAYED BRIDGE

Meiko-Nishi Bridge, which is under construction in Nagoya Harbor, Japan, is a steel cable-stayed bridge with three spans of (175 + 405 + 175m). The box-girder type with an orthotropic deck is a trapezoidal 3-cell box-sections with the depth of 2.8m. Pylons are A-frame fixed to piers. The longitudinal cable configuration is the fan type of twelve stay cables.

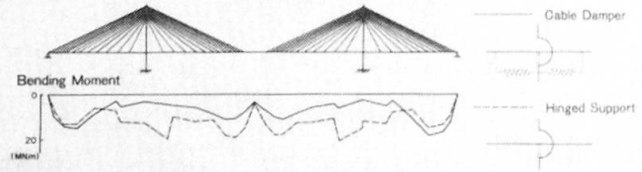
Meiko-Nishi Bridge has been installed horizontal cables (2 bandles of 61 galvanized 7-wire strands) with the length of 43.2 m, called cable dampers, in the connection of pylon and girder to reduce the horizontal forces to the pylon from the stiffened girders. The earthquake force will be reduced, because the cable dampers give lower natural frequency and larger system damping to the structure.



PYLON

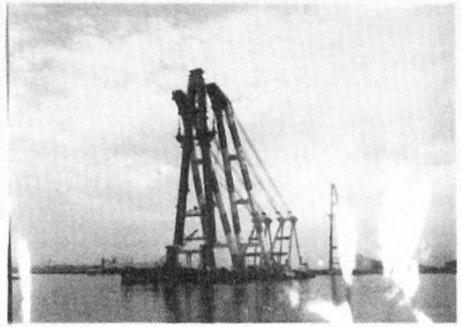


GENERAL VIEW

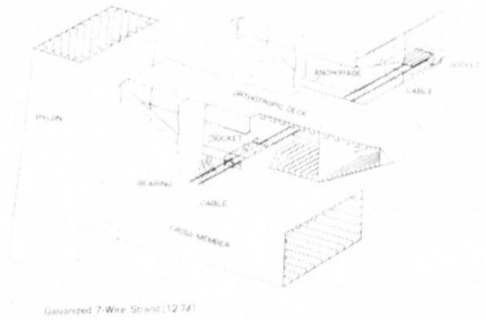


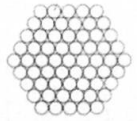
GIRDER

DYNAMIC RESPONSES OF THE BRIDGE SYSTEM CONNECTED BETWEEN PYLON AND GIRDER BY CABLE DAMPER AND BY HINGED SUPPORT



A-frame towers, the height of 104 m and the weight of 1250 ton, were hoisted into place by a 3000-ton floating crane in July, 1983





SECTION OF CABLE DAMPER

DESIGN CONDITION OF CABLE DAMPER

Area		62.59 cm²
Length		43.2 m
Design Load	Temperature	3557 KN
	Earthquake	10564 KN
Ultimate Load		10858 KN

Natural Frequencies and Mode Shapes of the Bridge over the Kocher-Valley

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The measuring method used to detect natural frequencies and modes is based on the excitation of the considered building by traffic, wind and natural microtremors. Any motion of a building can be considered as a sum of a large number of natural modes, vibrating with distinct amplitudes and at different frequencies, the natural frequencies. It is possible to detect these frequencies and modes by recording time-history-signals of the points of interest in the building for a sufficient period of time. These signals have to be processed by a spectrum analyzer, which computes transfer functions and spectral densities of the signals in the frequency domain and makes it possible to determine natural frequencies and to calculate corresponding mode shapes.

The advantage of this method is that no artificial excitation is necessary, so that measurements can be performed on completed buildings in full use without any damage and without having to interrupt work, as well as on structures in any stage of completion. It is of utmost importance to be able to check calculations of the vibration behaviour in order to ensure the earthquake resistance of the structure.

The mechanical model for the calculation of vibration characteristics concentrates masses and moments of inertia at the tops of the 8 pillars. Considering the different bearings, there remain 31 degrees of freedom for the whole system. The mass matrix was gained by a proper estimation of the contributing parts of pillars and girder. The stiffness matrix for the chosen degrees of freedom was calculated accurately by methods common in building statics. Three versions were calculated: Version 1 considered only the mass of the girder, version 2 in addition the pillar masses and the moments of inertia, version 3 contained the influence of axial forces in the pillars.

The measuring method presented here was found reliable not only in the presented example of a large bridge which is a comparatively rigid structure with very low natural frequencies, but also in several highrise buildings and in the very rigid structure of a nuclear power plant. This is of main interest for the earthquake resistance of buildings. Another application of this method could be to monitor prestressed concrete bridges. An eventual loss of stiffness would influence the natural frequencies as well as the mode shapes of the structure; see the inserted beam example. This loss of stiffness can be detected by using periodical measurements with the presented method which allows the determination of the natural frequencies with great accuracy.

The authors wish to express their appreciation to the Autobahnamt Baden-Württemberg, Stuttgart, for providing assistance with measurements and calculations, as well as to: Mr. A. Barske, Mr. K. H. Beyer, Mr. S. K. Chen, Mrs. C. Gurr-Beyer, Mr. L. Roth and Mr. W. Stöcklin, who assisted with the measurements. Many thanks to Hewlett-Packard, Richmond B.C., for lending us a HP 5423A Modal Analyzer.

1. Luz, E., Gurr, S.,"Measurements and Calculations of Natural Frequencies and Coupled Bending-Torsion Modes of Highrise Buildings", Proc. 7th World Conference on Earthquake Engineering, Istanbul 1980, Vol. 7, P. 437-440.

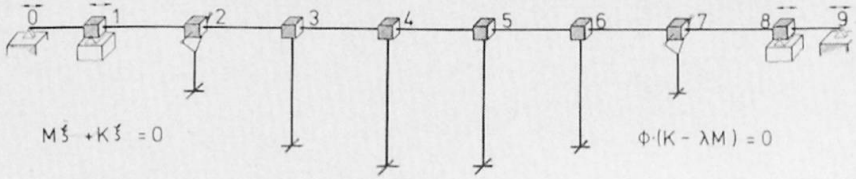
2. Luz, E., Gurr-Beyer, C., Stöcklin, W.,"Experimental Investigations of Natural Frequencies and Modes of the HDR Nuclear Power Plant by Means of Microtremor Excitation", Proc. 8th World Conference on Earthquake Engineering, San Francisco 1984, Vol. VI, P. 977-984.

MEASURED AND CALCULATED MODE SHAPES OF THE KOCHER-BRIDGE

FREQUENCIES [HZ]

MEASURED		CALCULATED		
MODE NO.		Version 1	Version 2	Version 3
1	0,078	0,1384	0,1291	0,1240
1A	0,117			
2	0,172	0,2261	0,2050	0,2019
2A	0,203			
3	0,281	0,3083	0,2868	0,2847
5	0,453	0,3993	0,3812	0,3798
8	0,672	0,5424	0,5208	0,5200
11	0,938	0,7693	0,7359	0,7353
14	1,22		1,4063	1,4063
15	1,37		1,5631	1,5631
16	1,58	1,2931	1,2750	1,2747
18	1,72		1,9823	1,9823
19	1,97	1,4248	1,3602	1,3599
20	2,31		3,0819	3,0819

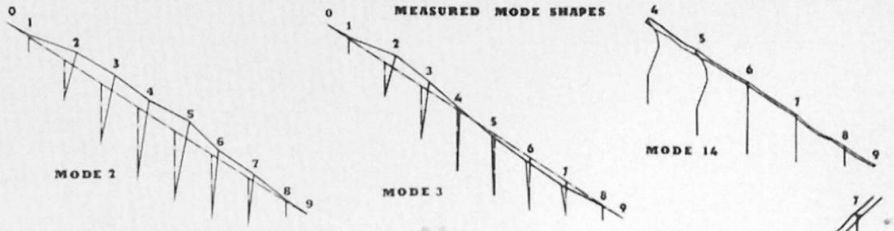
LUMPED-MASS METHOD

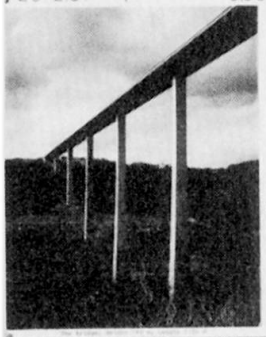


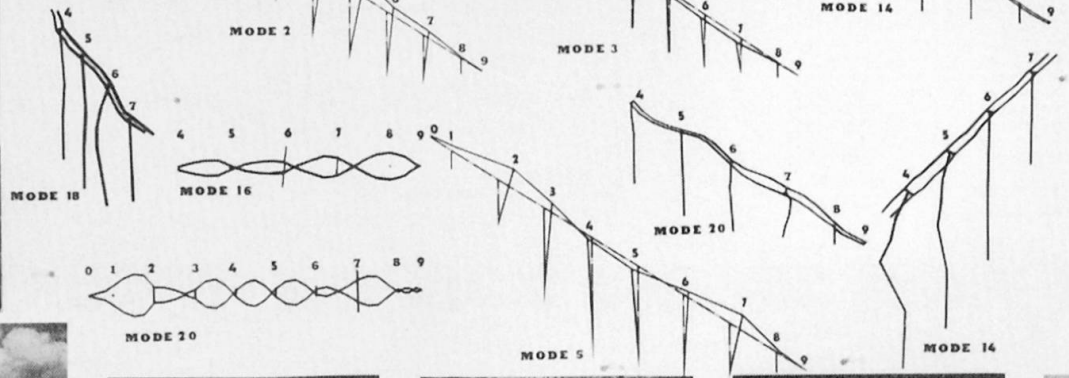
$M\ddot{\delta} + K\delta = 0$

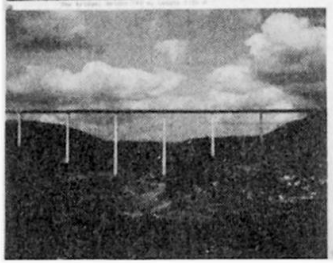
$\Phi \cdot (K - \lambda M) = 0$

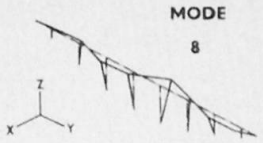
MEASURED MODE SHAPES

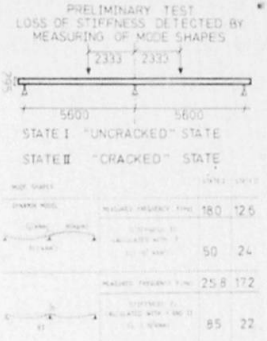


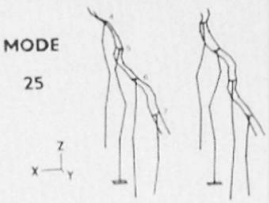


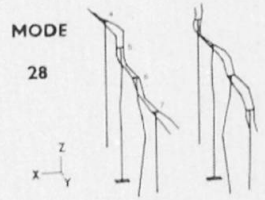


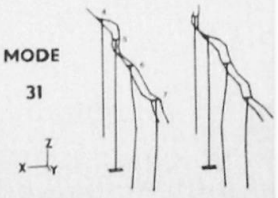












Shih-Sheng Paul LAI

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ON CONVENTIONAL AND ATC-6 ASEISMIC BRIDGE DESIGN

THE SEISMIC DESIGN OF BRIDGES IN TAIWAN HAS BEEN BASED ON THE STATIC SEISMIC COEFFICIENT METHOD. ALTHOUGH SIMPLE, BUT THE METHOD DOES NOT CONSIDER THE EFFECTS OF BRIDGE DYNAMIC BEHAVIOR AND LOCAL SOIL CONDITION. THE MAIN OBJECTIVE OF THIS PAPER IS TO ASSESS THE ADEQUACY OF THE BRIDGE DESIGN METHOD.

BASED ON THE NEW ATC-6 SEISMIC DESIGN PROVISION PROPOSED BY THE U.S. APPLIED TECHNOLOGY COUNCIL, SIX SELECTED BOX-GIRDER R.C. BRIDGES HAVE BEEN ANALYZED. THE RESULTING MEMBER FORCES AND THE REQUIRED PIER REINFORCEMENTS ARE COMPARED WITH THOSE OBTAINED BY USING THE CONVENTIONAL DESIGN METHOD. THE RESULTS INDICATE THAT THE DEGREE OF SEISMIC SAFETY (EVALUATED IN TERMS OF THE ATC-6 COMPATIBLE EQUIVALENT ACCELERATION COEFFICIENT) PROVIDED BY THE TRADITIONAL STATIC SEISMIC DESIGN METHOD VARIES SIGNIFICANTLY W.R.T. THE BRIDGE TYPES AND SOIL CHARACTERISTICS. THIS TENDS TO SUGGEST THAT THE CONVENTIONAL METHOD USED IN TAIWAN IS INADEQUATE FOR ASEISMIC BRIDGE DESIGN.

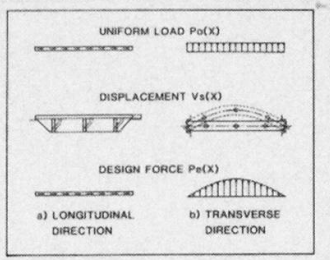


FIG 2 COMPUTATION ILLUSTRATION OF ATC6 SEISMIC DESIGN FORCE

CASE NO.		1	2	3	4	5	6
DESCRIPTION		180x1 column	130x2 columns	360x60 wall	100x2 columns	140x2 columns	180x2 columns
LONGITUDINAL	α (cm)	1957	1787	2292	30643	8516	5134
	δ (t-cm)	2024	2323	2979	43735	1107	6674
	γ (t-cm)	232	305	502	10819	6902	252
	PERIOD(sec)	0774	0829	0939	3599	1811	1406
	Cs(xAS)	1423	1369	1251	0511	0808	0999
	Pe(t/m,xAS)	185	1787	1626	664	105	1243
TRANSVERSE	α (cm)	1014	1145	114	7512	3981	2733
	δ (t-cm)	1319	1489	148	9766	5149	3553
	γ (t-cm)	119	152	01	693	1838	873
	PERIOD(sec)	0688	0731	0227	1886	1367	1135
	Cs(xAS)	154	1478	3229	0786	0974	1103
	Pe(t/m,xAS)	2468	2377	3354	1268	16	1804

TABLE 1 COMPUTATION OF ATC6 SEISMIC DESIGN FORCES FOR DIFFERENT CASES

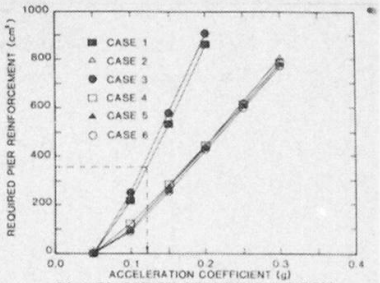


FIG 6 ATC6 REQUIRED PIER REINFORCEMENT FOR DIFFERENT DESIGN ACCELERATIONS (S=1.5)

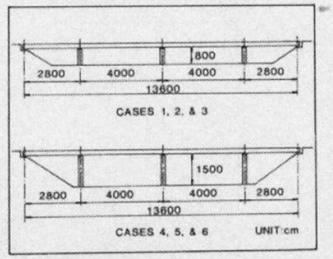


FIG 3 ELEVATIONS OF EXAMPLE FOUR-SPAN BOX-GIRDER BRIDGES

CASE NO.		1	2	3	4	5	6
LONGIT.	Mx(t-m)	—	—	—	—	—	—
	Mz(t-m)	20128	9813	17695	6775	1071	12578
	RESP. MODIF.	3	3	3	3	3	3
TRANS.	Mx(t-m)	13267	6273	24853	2202	5514	7338
	Mz(t-m)	—	303	—	101	256	343
	RESP MODIF.	3	3	2	3	3	3
LOAD 1	Mx(t-m)	442.2	209.1	1228.2	734	1838	2446
	Mz(t-m)	2013	106.2	177	71.1	1156	1382
	V(t)	5479	280.1	5715	282.9	3101	327
LOAD 2	Mx(t-m)	132.7	62.7	3884	22	551	734
	Mz(t-m)	6709	3235	5898	2235	3596	426
	V(t)	5479	280.1	571	282.9	3101	327
REINFORCE.(cm²)		884	451	904	432	423	432

TABLE 2 COMPUTATION OF REQUIRED PIER REINFORCEMENTS UNDER 0.3g ACCELERATION (S 1.0)

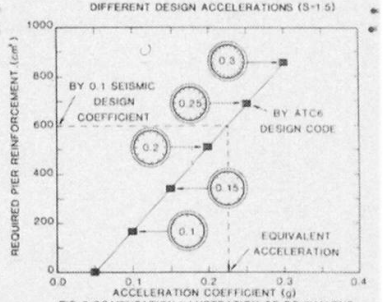


FIG 7 COMPUTATION ILLUSTRATION OF EQUIVALENT ACCELERATION COEFFICIENT

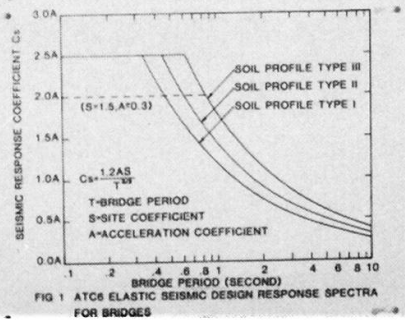


FIG 1 ATC6 ELASTIC SEISMIC DESIGN RESPONSE SPECTRA FOR BRIDGES

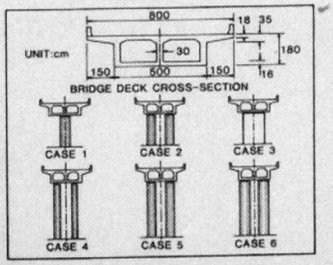


FIG 4 CROSS-SECTIONS OF EXAMPLE FOUR-SPAN BOX-GIRDER BRIDGES

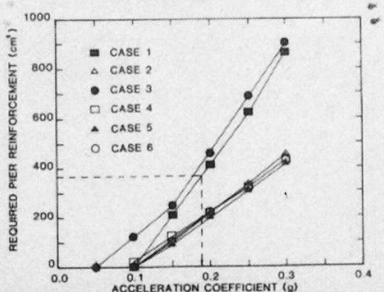


FIG 5 ATC6 REQUIRED PIER REINFORCEMENT FOR DIFFERENT DESIGN ACCELERATIONS (S=1.0)

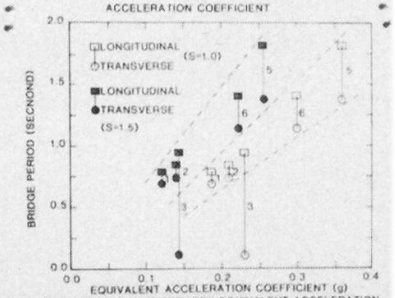
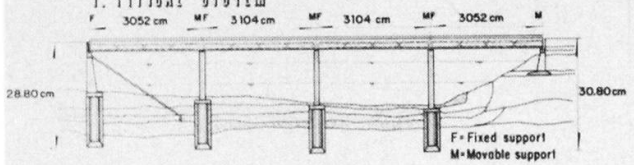


FIG 8 RELATIONSHIP BETWEEN EQUIVALENT ACCELERATION COEFFICIENT AND BRIDGE NATURAL PERIOD

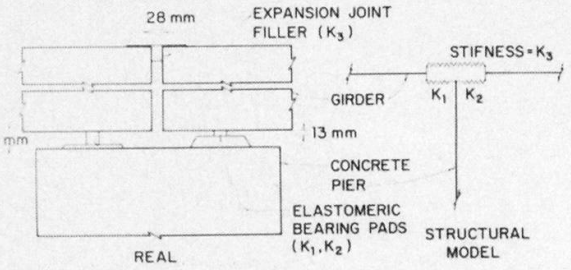
S.E. RUIZ — J.A. GARCIA — R. PAREDES
Universidad Nacional Autonoma de Mexico, Mexico

SEISMIC SAFETY OF BRIDGES

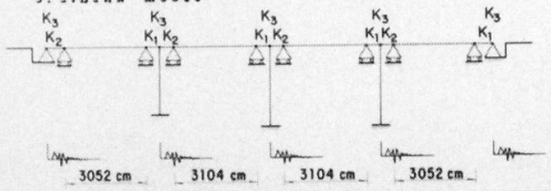
1. TYPICAL SYSTEM



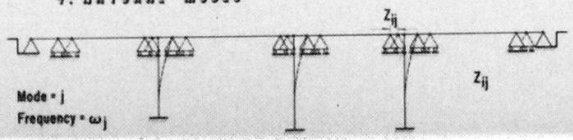
2. JOINT DETAILS



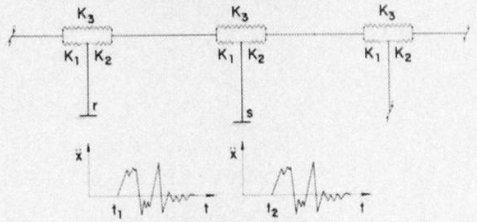
3. LINEAR MODEL



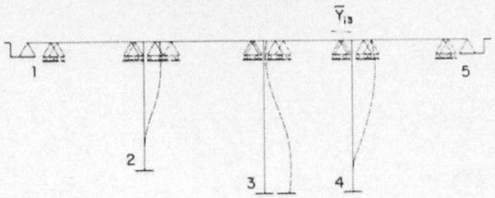
4. NATURAL MODES



5. OUT OF PHASE EXCITATION



6. SINGLE SUPPORT DISPLACEMENT



7. SUPERPOSITION

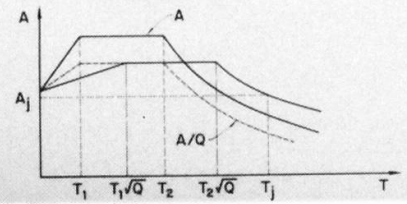
$$Q_i^2 = \sum_s \sum_r \beta_{is} \beta_{ir} \alpha_{sr} D_0^2 + 2 \sum_s \sum_r \sum_k \beta_{is} a_{kr} Z_{ik} \alpha'_{ksr} D_0 \frac{A_{kr}}{\omega_k^2} + \sum_s \sum_r \sum_j \sum_k a_{js} a_{kr} Z_{ij} Z_{ik} \alpha^*_{jkrs} \frac{A_{js}}{\omega_j^2} \frac{A_{kr}}{\omega_k^2}$$

a_{kr} = Participation factor

D_0 = Peak ground displacement

α's = Correlation coefficients

8. NON-LINEAR BEHAVIOR



9. RELIABILITY ANALYSIS

1. Seismicity

Poisson process with mean rate function $v_Y(y)$

y = intensity

$v_Y(y)$ = rate of occurrence of intensity values Y greater than y

2. Structural response for intensity Y

$S = \phi Y$

$F_\phi(u) = P[\phi \le u]$

3. Structural strength R

$F_R(u) = P[R \le u]$

4. Distribution of maximum intensity in t years

$$F_{Y_{max}}(y,t) = e^{-v_Y(y)t}$$

1. Rate of occurrence of response values S greater than s

$$v_Y(y) = -\frac{\partial v_Y(y)}{\partial y} dy \qquad P[S>s|Y=y] = P[\phi>s/y]$$

$$v_S(s) = \int_s^\infty -\frac{\partial v_Y(y)}{\partial y} P[\phi>s/y]\, dy$$

2. Failure probability (p_f)

A) Deterministic system

$$p_f = 1 - e^{-v_S(M_R)t}$$

B) System with uncertain properties

$v_S(M_R)$ is function of system properties f'_c and f_y (two-point probabilistic estimates)

$$p_{R_i} = 0.25 \qquad p_f = 1 - \sum_{i=1}^{4} e^{-v_S(M_{R_i})t} \, p_{R_i}$$

10. RESULTS

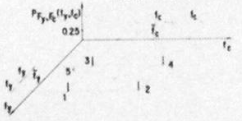
1) Accounting for support stiffnesses

Response	Wave velocity (m/s) ∞	2500	1500	1000	500	100
ΔL_0	0.52	0.53	0.56	0.61	0.71	0.66
ΔL_1	0.08	0.08	0.08	0.08	0.08	0.09
ΔL_2	4.92	4.83	4.76	4.64	3.89	2.70
M_{1L} (t-m)	398	394	386	375	311	206
M_{2L} (t-m)	351	350	345	336	282	186

2) Neglecting support stiffnesses

Response	Wave velocity (m/s) ∞	2500	1500	1000	500	100
$\Delta L_0+\Delta L_1$	5.8	6.35	6.7	7.1	8.5	11.9
ΔL_2	7.2	7.2	7.2	7.2	7.3	7.78

(Discrete distributions)



Case	f'_c (kg/cm²)	f_y (kg/cm²)	Resisting moment M_R (t-m)
1	226.4	4915	4700
2	346.8	4915	4900
3	226.4	3941	4000
4	346.8	3941	4200
5*	170	4000	3950

* Deterministic case (nominal values of material properties)

Case	P_f, 50 years	P_f, 1 year
A	0.044	9.0×10^{-4}
B	0.035	8.0×10^{-4}

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