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TITLE: COORDINATED EXPERIMENTAL/ANALYTICAL PROGRAM FOR INVESTIGATING MARGINS TO FAILURE OF CATEGORY I REINFORCED CONCRETE STRUCTURES

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COORDINATED EXPERIMENTAL/ANALYTICAL PROGRAM FOR INVESTIGATING MARGINS TO FAILURE OF CATEGORY I REINFORCED CONCRETE STRUCTURES

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ABSTRACT

The material presented in this paper deals with a coordinated experimental/analytical program designed to provide information needed for making margins to failure assessments of Category I reinforced concrete structures. The experimental program is emphasized and background information that lead to this particular experimental approach is presented. Analytical tools being developed to supplement the experimental program are discussed.

1. INTRODUCTION

The purpose of this Structural Margins to Failure Program is to obtain information needed to make more reliable assessments of the margins to failure of seismic Category I nuclear power plant structures constructed of reinforced concrete. This report includes information on areas that have been identified as needing additional study, current analysis methods, a proposed experimental plan designed to obtain needed data and to provide benchmark cases for code verification and development, and some results to date.

Seismic Category I structures are designed for specific loadings and load combinations and, because of the magnitudes of the load factors used in the design procedures, these Category I structures would not go beyond the effective elastic limit if subjected to the actual design loads. However, as plant operating experience increases, load definitions are changed or additional loadings may be prescribed. If the redefined loads were applied to the structure, the behavior may then be nonlinear. The amount of reserve strength between the elastic limit and the failure strength then becomes an important consideration. Since structural behavior beyond the elastic limit and up to failure is nonlinear, traditional structural analysis methods do not apply. The goal of this program is to obtain the information necessary to determine the margin to failure and the behavior near or at the ultimate strength of Category I structures. The proposed program includes analysis and experimental testing.

The Category I buildings at a nuclear power plant facility are the reactor containment building and the auxiliary building. The auxiliary building may be a single continuous structure or it may be the aggregate of several disjoined buildings. The auxiliary building or buildings include some or all of the following function units.

- Diesel Generator Building
- Control Room/Building
- Spent Fuel Pit
- Fuel Handling Building
- Safety-Valve Room
- Radioactive Waste Building and
- Waste Management Building

The information that follows pertains to the auxiliary buildings only. Several nuclear power plant building arrangements are shown in Figs. 1-3 to illustrate the relative locations of the auxiliary functional units with respect to the reactor containment building. The turbine buildings are not Category I structures; however, their potential impact on the adjacent Category I buildings must be considered.

The Category I buildings (exclusive of the containment building) are box shaped shear wall buildings as indicated in Fig. 4. These buildings are constructed of reinforced concrete, but they may include steel columns and steel beams that support floor slabs. The plan view dimensions of these Category I structures are about 90-120 m long, 15-30 m high, and 30-150 m wide.

The methods and procedures used by different Architectural and Engineering firms (A/E's) in the design and analysis of Category I structures are essentially the same. Category I buildings are designed to remain elastic for the safe shutdown loading combinations. For seismic loadings, the buildings are normally modeled as lumped mass systems; however, there is a trend toward the use of finite element models. In finite element models there is the advantage of easily including the effect of static loads, which allows the consideration of pressure loads using the same model. In the analysis of the buildings, the shear stiffness of the walls is based on uncracked sections. For seismic loadings, the response spectrum method is used for analyzing Category I buildings. The computer codes used in these analyses are generally available commercially or are in-house codes modified by the A/E from codes that are commercially available. Nonlinear analyses are usually used only for local regions such as missile impact areas.

In a survey of A/E's, several areas in which additional information is needed were brought forth. Many felt that the damping values allowed for shear walls are far too small. The rationale is that the ratio of cracked concrete volume to total concrete volume in shear walls is large; hence, the damping should be larger than for other concrete structural elements. Additional information regarding equipment-structure interaction is also needed. Theoretical methods for estimating equipment-structure interaction are available but these methods have not been experimentally verified. Safety margins on equipment are not known because the qualification of equipment by testing is to acceptance levels and not to failure.

Information on the stiffness of cracked shear walls is also needed. Shear wall stiffnesses are now calculated assuming an uncracked shear wall section. The degradation of shear stiffness as cracking progresses during load cycling also needs further study and quantification. The limit of stiffness degradation is also a topic that requires additional attention.

Because the shear wall is a primary structural element of a Category I structure and little is known concerning its post-elastic dynamic behavior, we have focused this program on the shear wall structure.

2. PROGRAM PLAN

Because of the limitations of analytical methods and because of the uncertainty of concrete material property data (damping and stiffness values) that are required for any of the analytical methods, it is neither feasible nor desirable to design an experiment to verify a particular analytical method applied to a specific plant design. Rather, our approach will be as follows.

- Identify those structural properties that will be essential if analytical methods are to have any reasonable chance of predicting structural behavior at load levels that are necessary to produce failure.
- Identify the preliminary experiments required to determine these structural properties.

- Identify a relatively simple structure for use in the preliminary experiments that meets the following conditions:
    - Typical of Category I reinforced concrete structures.
    - Structure sized so that it can be dynamically tested to failure under loading conditions similar to those postulated for Category I reinforced concrete structures, using existing test facilities.
    - Structure simple enough to permit nonlinear analysis. This requirement will make it possible to compare experimental results to the predicted structural response using both the current methods of analysis and appropriate nonlinear analysis.
- Identify reasonably simple structures that incorporate the three-dimensional effects associated with interconnected structural elements and other equipment items attached to the principal structure. The following points will receive considerable attention:
    - Material properties, element behavior, and analytical tools developed previously should be applicable to these relatively more complicated structures.
    - Structures must be sized so that experimental results may be taken as prototypical.
    - Because of the larger size of these structures, necessary test facilities must be located and/or planned and constructed.

Our discussions with the designers and builders of Category I reinforced concrete structures, our review of the literature, and our review of the currently used methods of analysis for the structures all point to the importance of and, hence, the need for, realistic values for damping and stiffness in understanding and predicting the behavior of Category I reinforced concrete structures subjected to loads that produce failure.

The determination of values for damping and stiffness of reinforced concrete structures over the entire loading range from elastic deformation to failure is a difficult task. Damping is usually understood to include all of the mechanisms of energy loss that reduce the response motion of a structure. However, in almost all analytical techniques, which take damping into account, the damping force is assumed to be "viscous" or "structural." Furthermore, since damping forces cannot be measured directly, most experimentally determined values of damping are actually computed from experimental results using relations between measured response and damping that involve a prior assumption concerning the nature of the damping. The three simplest experimental methods (free decay to find the logarithmic decrement, bandwidth at resonance, and amplification at resonance) all give values for equivalent viscous damping that are strictly applicable only for linear single degree of freedom systems. The so-called "response method" consists of modelling, mathematically, a given structure, solving for the response as a function of damping, and then finding the damping required to make this solution match experimental results. Unfortunately, when the "response method" is used to obtain structural properties from tests conducted on existing structures,* there are numerous difficulties. If the tests are restricted to the linear region, the data is not useful for predicting behavior in the nonlinear region. If the tests are carried into the nonlinear region the methodology for handling these more complicated structures is not well established and as a result the various possible types of damping (viscous, structural, Coulomb, etc.) may not be identified, the degradation of stiffness may not be separated

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* Including response data obtained when an existing structure is excited by an earthquake.

from damping effects, and the dependence of both damping and stiffness on amplitude of vibration may not be identified.

The preliminary experiments planned for this project are specifically designed to investigate the damping and stiffness of reinforced concrete in the fully cracked condition. Since reinforced concrete is known to behave as a "softening, hysteretic" system (see Fig. 5) all data will be analyzed using techniques appropriate to nonlinear systems. An undamped, single degree of freedom system containing a nonlinear spring element (a softening system) is well understood. [1] The "softening, hysteretic" system has received less attention; however, Jacobsen and Ayre [2] show how the response of such a system to a general ground motion input can be computed. Others have extended the methods of analysis that can be applied to this system. [3-5] Improved computational methods may make the solution of this problem easier.

The preliminary experiments will involve both quasi-static (load cycling) and harmonic vibration tests. To be of value in this program, these vibration tests must be carried to the point of structural failure. Both static and vibration tests are necessary if we are to distinguish damping associated with the static hysteretic mechanisms from that which is frequency dependent. Vibration tests are also important to determine if it is possible to predict resonant frequency and dynamic response from load cycling data alone.

The preliminary tests will involve simple one and two degree of freedom structures so that "exact" methods of analysis will be possible. Because vibration tests must be carried to failure the structure will of necessity be small. It is important to remember however, that the preliminary tests are not intended to qualify or predict the response of full sized structures.

Rather, the preliminary tests are designed to develop a test program that, when supported by verified theory, can most advantageously be applied to more nearly prototypical structures.

A reinforced concrete shear wall structure has been selected for the proposed experiments for two reasons: 1) the shear wall structure is the structure most often found in Category I nuclear power plant designs, 2) very few dynamic experiments have been conducted on shear wall structures, and to the authors' knowledge, none of these have been carried to failure.

Shear wall structures used in nuclear power plants are very large and massive structures. As a result, testing of full sized structures under carefully controlled conditions would be prohibitively expensive. Use of small scale models or smaller prototypical structures is the obvious alternative; however, the design of these small scale models and prototypical structures must be undertaken with great care.

If only elastic behavior were of interest, the use of scale models in an experimental program would be greatly simplified. Indeed, several very complex concrete structures have been designed for elastic behavior with the aid of scale model experiments and the techniques and methodology are well known. [6,7] However, since behavior at or near failure is the goal of this investigation, ultimate strength scale models must be considered. Ultimate strength models have also been constructed and tested to aid in the design of several complex concrete structures; [7,8] however, when ultimate strength models must be tested dynamically the model design and load conditions are most difficult. These difficulties are discussed in detail in Ref. [9]. The problem arising from the similitude requirements can be briefly stated as follows:

1. In reinforced concrete structures, both the system damping and the amplitude of external forces and/or input motions necessary to produce failure are affected by the gravitational forces. Hence, in a true scale model all forces, including gravitational forces, must be correctly scaled.

2. Since a scale model will, in most cases, be tested in the same gravitational field as the one in which the prototype will be used, it is difficult to scale gravity forces in the same manner as other forces. In theory it is possible to construct a model of materials other than the steel and concrete that will be used in the prototype (e.g., a plastic with fiber reinforcement) and satisfy the similitude requirements with a suitable length scale. In practice, however, if different materials are used in the model and prototype it is practically impossible to satisfy additional similarity requirements that relate to properties which govern reinforcement bond strength, crack development and growth, Poisson's ratio, etc. These properties are of great importance when testing to failure. As a result, when constructing ultimate strength models, the researcher is invariably forced to use the same material in the model as is used in the prototype, i.e., steel and concrete.

3. When the same materials are used in the model and prototype, placement of the required distributed mass so that the distribution of body forces (gravity forces) in the model will be the same as in the prototype is impossible in the strictest sense. However, in many cases an approximation of this distribution is adequate.

The problems just described are a result of the fundamental similitude requirements when the model and the prototype must be tested in the same gravitational field and gravity forces are important in system behavior. Some additional problems are:

1. Strict size and shape scaling of cement, aggregate, and reinforcing bar size is not possible.
2. Even when aggregate and reinforcing bar size are scaled, it cannot be assumed that such important properties as concrete strength and concrete/reinforcement bond strength are correctly scaled. As a result, these model properties are usually established by preliminary tests and adjusted as necessary.
3. The effect of curing time on concrete strength is different in the model and prototype. Drying and shrinkage effects are different in model and prototype. Different construction techniques may affect strength.

It is clear that all of these problems become more difficult to deal with as the model becomes smaller.

Reinforced concrete structures typical of Category I nuclear structures can, of course, be made in sizes smaller than used in the typical nuclear plant. Such structures might be dynamically tested at existing facilities and, without considering them to be scale models of particular plants, the test results could be used to benchmark analysis. In this approach it would be important to ensure that, in reducing the size, important response behavior was not modified or eliminated. This use of small prototypical structures was the approach taken in Ref. [10].

Yet another consideration in the selection of any test structure must be the method of dynamic testing and the capacity and availability of testing facilities. Three types of dynamic testing have been considered:

1. Sinusoidal vibration testing.
2. Non-periodic vibration testing; i.e., simulated earthquake.
3. Transient load testing; i.e., air blast or ground shock.

Although transient loading is of great importance in Category I nuclear power plant structures, this type of testing is probably more appropriate for a follow-on experimental program. The reason for this is that since analytical methods for predicting response of structures to transients are less well developed, there may be less to be learned from comparison of measured and predicted response. Analysis to predict response to sinusoidal vibration is undoubtedly the best developed; however, this problem is of the least practical interest except as a step toward the understanding of response to seismic excitation. Fortunately, a structure designed for testing on a shaker capable of producing simulated earthquake motions could be pretested using sinusoidal excitation as required to fully investigate structural response.

Facilities available for the seismic testing of either scale models or small prototypical structures are limited. An electrodynamic shaker with a stroke of $\pm$ 12 mm and a peak force of 88 KN is available at Los Alamos, and it is anticipated that this facility will be used in preliminary experiments. Structures to be tested in latter phases of this program will require a larger test facility, and as a result, the use of facilities outside of Los Alamos will be considered. Because of the proximity of the White Sands Missile Range (WSMR), their facility may be used for intermediate sized experiments. Larger and more advanced experiments will be planned using the capabilities at the University of California's Earthquake Engineering Research Center.

As a result of the above considerations the experiment program will be conducted in three phases

Phase I - These tests will be conducted on small reinforced, micro-concrete shear wall structures that can be statically and dynamically tested to failure on equipment available at Los Alamos.

Phase II - These experiments will involve scaled-up versions of the Phase I structures. As a result, the Phase II structures will be fabricated using realistic concrete (rather than the micro-concrete used in smaller models) and standard reinforcment. It is expected that these Phase II experiments will yield results that will be credible when applied to full sized concrete structures. Further, it is anticipated that Phase II experiments will confirm the applicability, to full sized structures, of any analytical methods developed from the Phase I experiments. These structures will be tested on a large servohydraulic seismic simulator.

Phase III - These experiments will be conducted on structures that incorporate the three-dimensional effects associated with interconnected elements and other equipment items attached to the principal structure. Phase III structures will involve the same basic structural element as that tested in Phases I and II (i.e., shear walls). Phase III structures will involve combinations of shear walls , slabs and other interior walls, etc. In addition, simulated equipment will be attached to the structure. Since this structure will of necessity be large, massive, and expensive, the final form of this structure will not be selected until data becomes available from Phase I tests. Candidate dynamic test methods and facilities must also be reviewed before the final form of the Phase III structure is selected. The hyraulic shaker designed by Smallwood and Hunter [11] and used by Chen, et al., [10] in the destructive test of a four-story concrete structure is being investigated for possible application in the Phase III experiments.

The Phase I experiments will be conducted on both one and two degree of freedom shear walls (see Fig. 6). These small structures will be proportioned so that their aspect ratios (h_w/ℓ_w) and wall thickness to wall height (h/h_w) will be typical of the shear walls used in Category I structures. The walls will be used in static and forced, sinusoidal, vibration tests to determine stiffness and damping as functions of both normal load (dead weight) and amount of shear deformation. The static tests will involve load cycling to progressively higher strain levels. From these tests, which are similar to tests conducted by others [12-16] on larger shear walls, we expect to evaluate degradation of stiffness and hysteretic energy loss.

The proposed forced vibration tests can best be outlined in connection with Fig. 7. Several walls, each with a different amount of normal load, will be subjected to a series of frequency sweeps. During each sweep the input acceleration at the base ($\ddot{X}$) will be held constant but the several sweeps will be made at progressively increasing values of base acceleration. The data obtained from these tests will be plotted in the form of amplification curves as shown in Fig. 7. From the data we expect to determine effective stiffness (K_e) from measured values of resonant frequency (ω_{n1}, ω_{n2}, etc.), i.e., $K_e = M\ \omega_n^2$. It may also be possible to determine a meaningful value for equivalent damping from the measured amplification factor, Q. However, for a highly nonlinear material (i.e., modulus undergoing a large decrease from zero to ultimate load) an equivalent viscous damping may be of little value.

Two-story shear wall structures (see Fig. 6b) will also be statically tested to determine the stiffness coefficients (K_{11}, K_{22}, K_{12}, K_{21}). This will permit comparison of this two degree of freedom test structure to the one degree of freedom structures previously tested. Two-story shear wall structures will also be subjected to sinusoidal vibration testing. Measured values of amplification (Q) and modal frequencies (f_n) will be compared to the values predicted from theory using the previously measured values of stiffness (K) and damping. Several computer programs which are discussed in the next section have been designed to reduce the data from these Phase I experiments. These programs are being designed to predict the response of one and two degree of freedom systems which are nonlinear (softening) and acted upon by any type of damping (viscous, structural, Coulomb, and hysteretic energy loss).

The final design and test procedures used in the Phase II experiments will await the results of the Phase I experiments. However, the need for these larger scale experiments is already apparent. By using larger test strucures, the reinforcement, the concrete, and the construction techniques can all be more typical of actual Category I structures. Because both the construction and testing of these larger test structures will be much more expensive, it is important that the number of Phase II tests be reduced to a minimum. Indeed, this is the purpose of the Phase I tests.

Phase II experiments will track the Phase I experiments in as much as preliminary static and sinusoidal shaking tests are planned; however, it is hoped that at least one Phase II structure can be subjected to simulated seismic excitation. With this in mind, the Phase II structure is being sized so that it could be tested on the seismic simulator at the University of California at Berkeley or a comparable facility.

Three-dimensional structures are required to investigate interconnection of structural elements and the effects of equipment mounted on the primary structure. On the other hand the 3-D structure for the Phase III experiments must be designed so that analysis is possible, given structural properties available from previous experiments and the required input data. Some preliminary small scale testing of 3-D structures may be undertaken to checkout data acquisition and reduction techniques and response prediction methods. However, it is intended that the principal Phase III experiment will involve a small size prototypical structure. As a minimum, the structure should involve orthogonal shear walls joined at the corners, and walls connected to floor slabs at top and bottom surfaces. It should be possible to vary the stiffness of the coupling between the simulated attached equipment and the main structure. All connecting forces and the relative motion between equipment and structural elements must be measured. This latter point is particularly important since failure may depend upon these forces and motions rather than upon the damage substained by the structure itself. It is anticipated that a detailed program plan will be prepared for the proposed Phase III experiments after results from the Phase I and II experiments become available.

3. RESULTS TO DATE

Two analytical efforts, which were undertaken to aid in the analysis of experimental results, are essentially complete.

One computer program has been written to investigate the importance of damping type (viscous, structural, Coulomb and combinations of these) on the higher mode response of multi-degree of freedom systems. When applied to a system such as the one shown in Fig. 8, this program computes response diagrams. Figure 9 shows the relative motion response of a three degree of freedom system with viscous damping. Figure 10 shows the relative motion response of the same system with structural damping, where the value of structural damping has been adjusted to make the first mode response identical to the viscously damped system. The difference in higher mode response is obvious and suggests how multi-degree of freedom test structures may be used to identify damping type.

The second analytical study has resulted in a computer program to compute the time-history response of multi-degree of freedom, nonlinear, hysteretic systems subjected to base motion excitation. The nonlinear hysteretic element of interest is, of course, the cracked shear wall for which a typical restoring force vs deflection curve is as shown in Fig. 5. To investigate the possible dynamic response of such an element the diagram shown in Fig. 5 has been linearized as shown in Fig. 11 and a parameter study has been conducted.

- As an example: If the single degree of freedom shear wall shown in Fig. 12 is characterized by the restoring force vs deflection diagrams shown in Fig. 11, we can compute how effective stiffness of the system varies with variation of system parameters. Effective stiffness (K_e), is computed as $K_e = M \omega_n^2$ in which M = system mass, and ω_n is the computed resonant frequency of the bilinear hysteretic system. Figure 13 shows how the effective stiffness can be expected to vary with changes in the ratios K_2/K_1 and U/Δ.

In the same manner, it is possible to study the variation in effective system damping (either equivalent viscous or equivalent structural). Figure 14 is an example of a response spectrum calculation for a bilinear, hysteretic system made using the same program.

The results presented in Figs. 15 and 16 illustrate the application of the program to two degree of freedom, bilinear, hysteretic systems. Figure 15 shows the absolute acceleration response of a two degree of freedom system (Fig. 6b, for example), having the bilinear hysteretic characteristics shown in Fig. 11, when it is subjected to sinusoidal base motion of peak amplitude $\overline{Y}$ equal to the element's linear displacement limit, Δ. Figure 16 is the result of analyzing the same two degree of freedom system after having assigned each of the two elements values of effective viscous damping and effective stiffness that were computed using the methods discussed in the preceding paragraph. In general, linearizing the system with effective stiffnesses for the restoring elements gives an adequate approximation of the modal frequencies; however, an 'equivalent effective damping' is more difficult to assign. For the case shown in Fig. 15 and 16, the assigned 'equivalent viscous damping' clearly does not result in equivalent response.

4.0 SUMMARY

The Program Plan for a coordinated experimental/analytical investigation has been presented. For the Phase I and Phase II experiments, the test structure is a reinforced concrete shear wall, a structural element found in most Category I structures. Both the Phase I and Phase II experiments will be conducted on two dimensional, single and two degree of freedom structures. The test structures in Phase III will be three dimensional, and will consist of interconnected elements. The Phase III experiments will also include simulated attached equipment.

Some analytical studies related to effective damping and stiffness of nonlinear elements have been completed. These preliminary studies indicate a possible method for identifying different damping types and give some insight into the difficulties encountered in using equivalent viscous damping to represent hysteretic energy loss.

5.0 ACKNOWLEDGMENT

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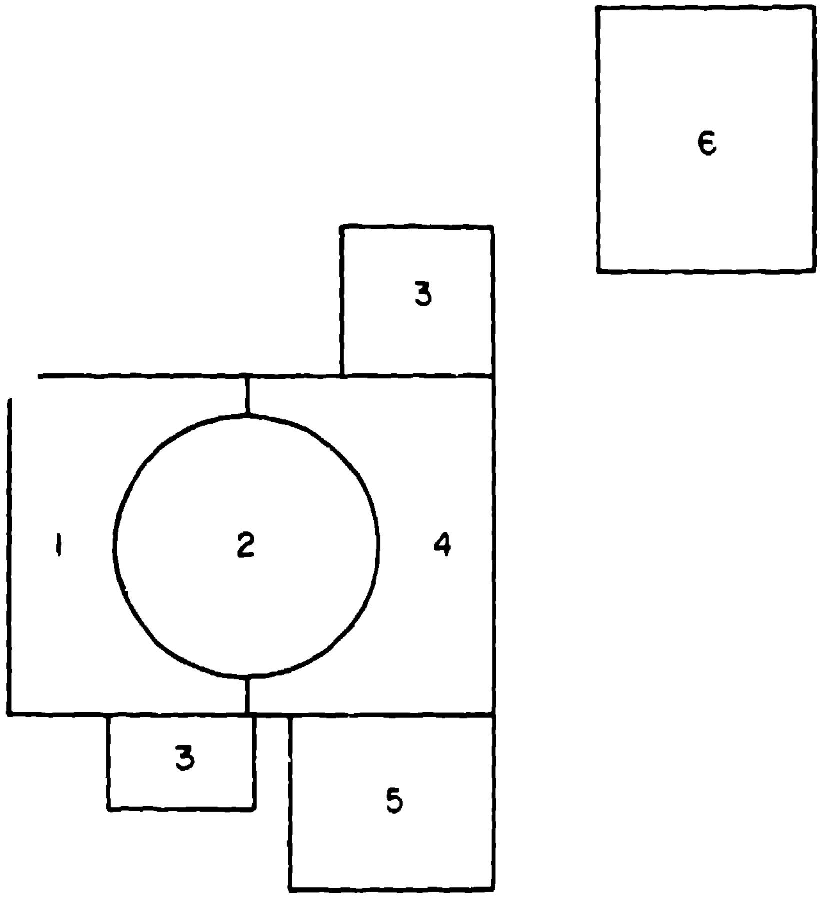
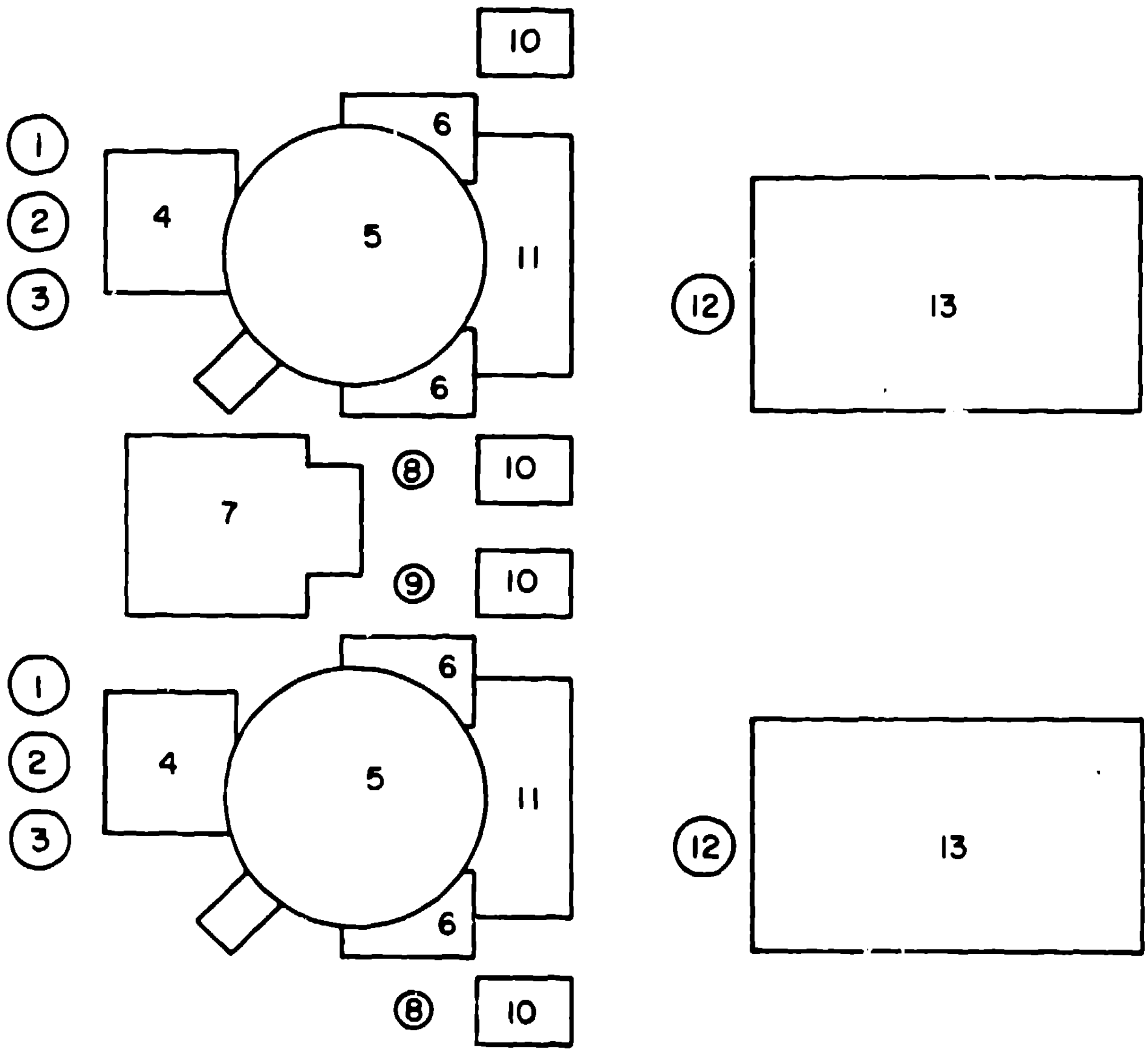
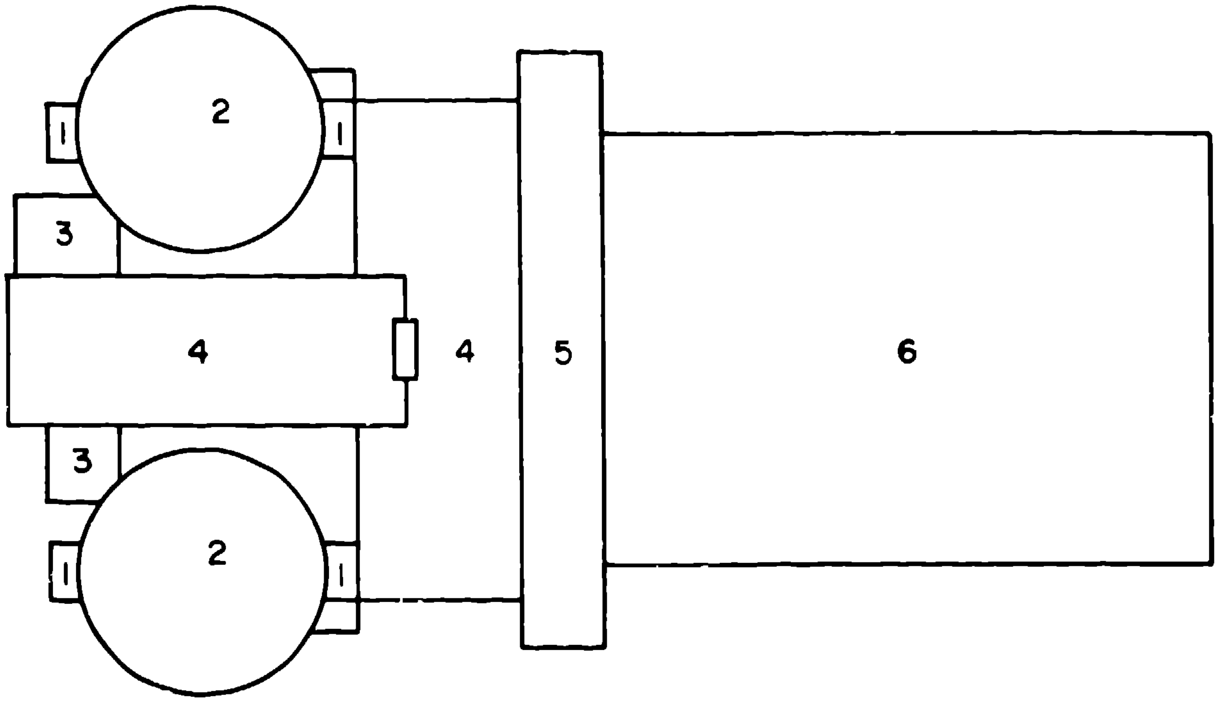
FIGURE CAPTIONS



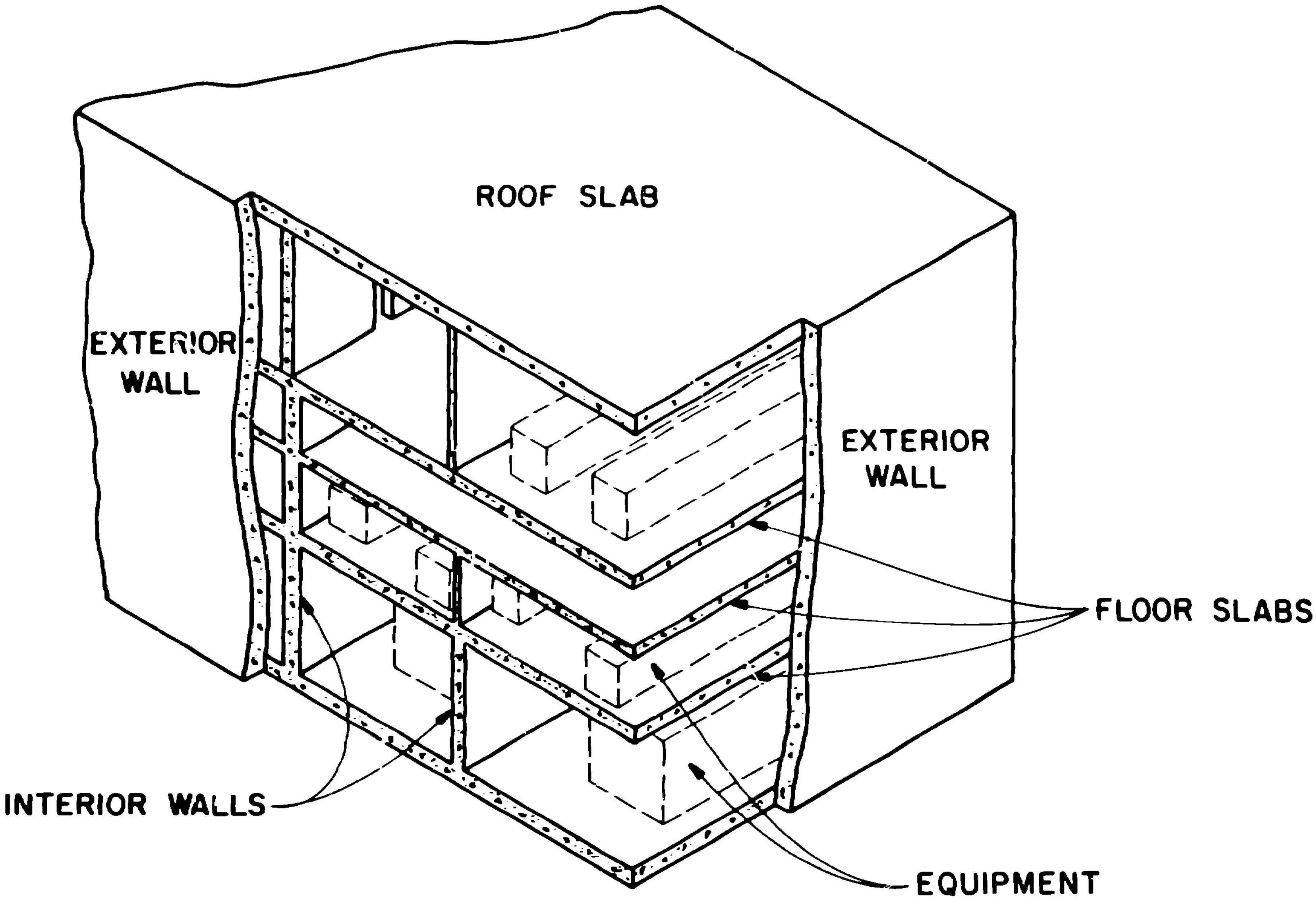
1 - FUEL BUILDING
2 - REACTOR BUILDING
3 - DIESEL GENERATOR BUILDING
4 - AUXILIARY BUILDING
5 - CONTROL BUILDING
6 - RADWASTE BUILDING



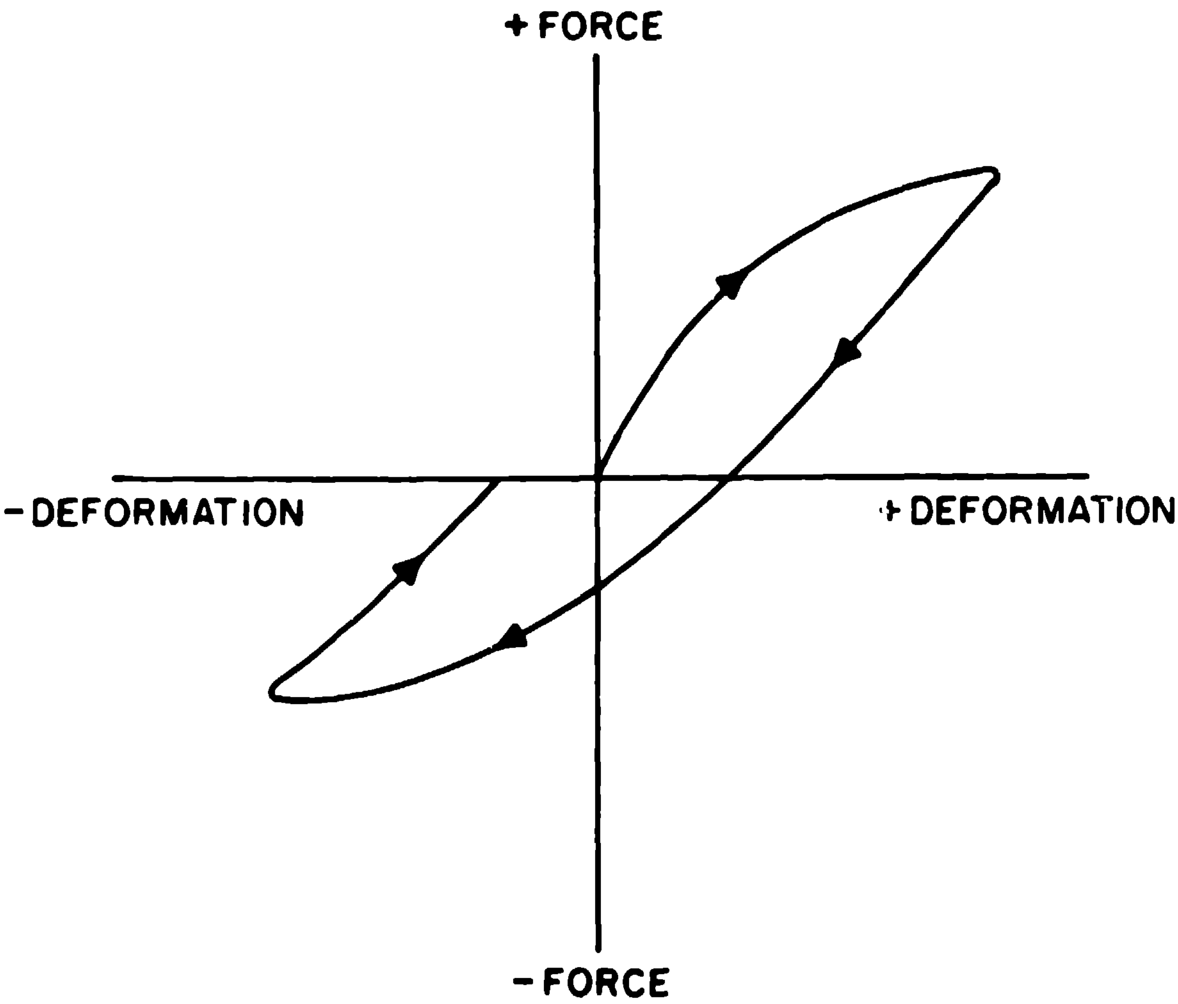
1 - HOLDUP TANK
2 - REACTOR MAKE-UP WATER TANK
3 - REFUELING WATER TANK
4 - FUEL BUILDING
5 - REACTOR BUILDING
6 - MAIN STEAM VALVE VAULT
7 - WASTE MANAGEMENT BUILDING
8 - EMERGENCY FEEDWATER TANK
9 - WATER REUSE TANK
10 - DIESEL GENERATOR BUILDING
11 - CONTROL BUILDING
12 - CONDENSATE STORAGE TANK
13 - TURBINE BUILDING

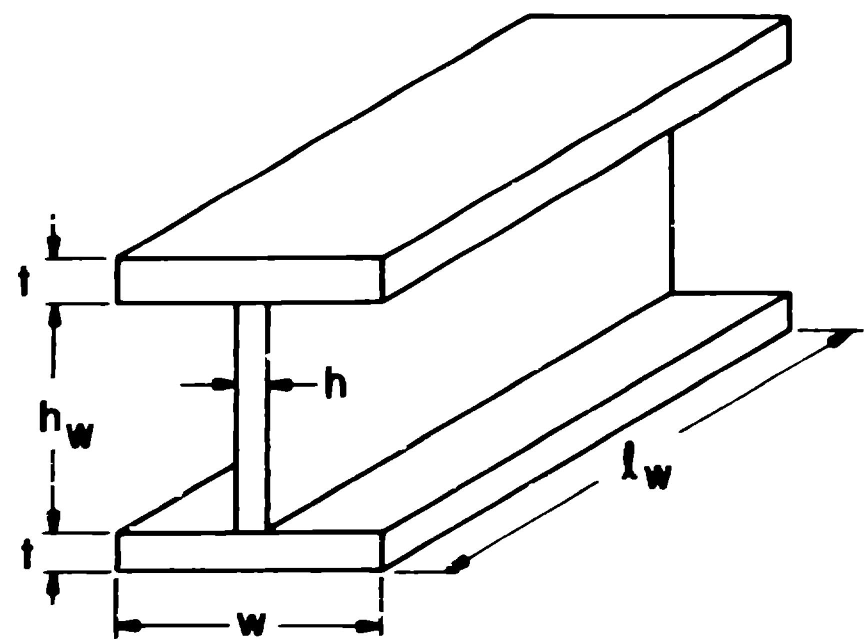


1 - STEAM VALVE VAULT
2 - REACTOR BUILDING
3 - ADDITIONAL EQUIPMENT BUILDING
4 - AUXILIARY BUILDING
5 - CONTROL BUILDING
6 - TURBINE BUILDING

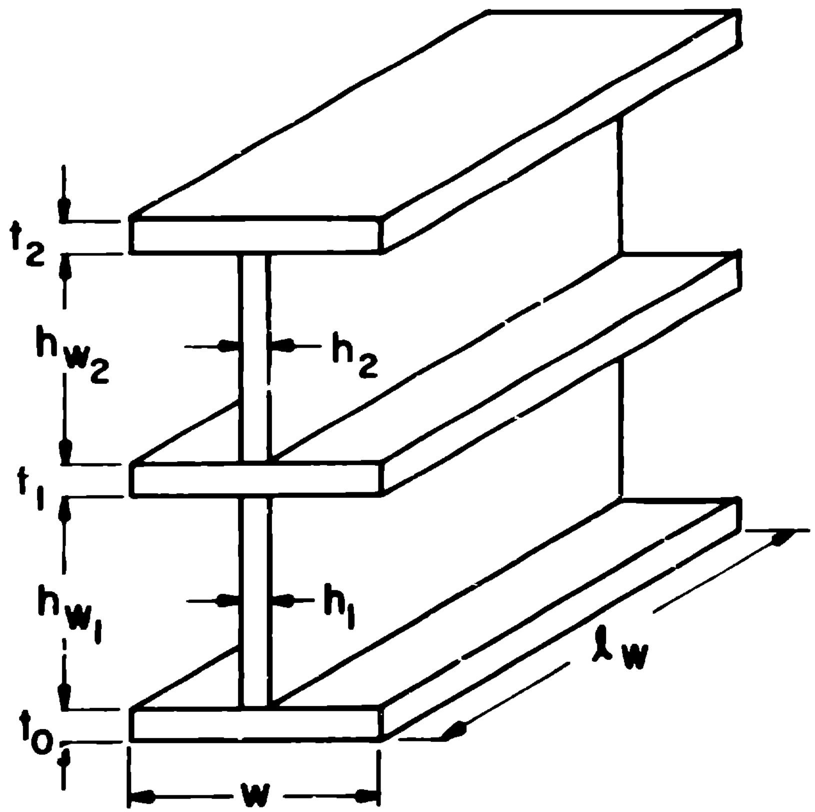


TYPICAL CATEGORY I STRUCTURE

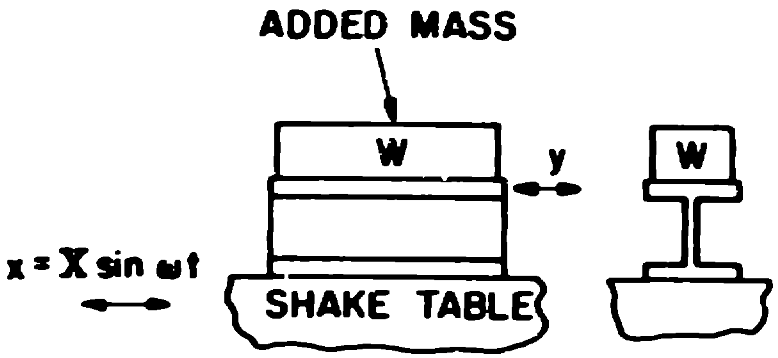

+ FORCE
- DEFORMATION
+ DEFORMATION
- FORCE



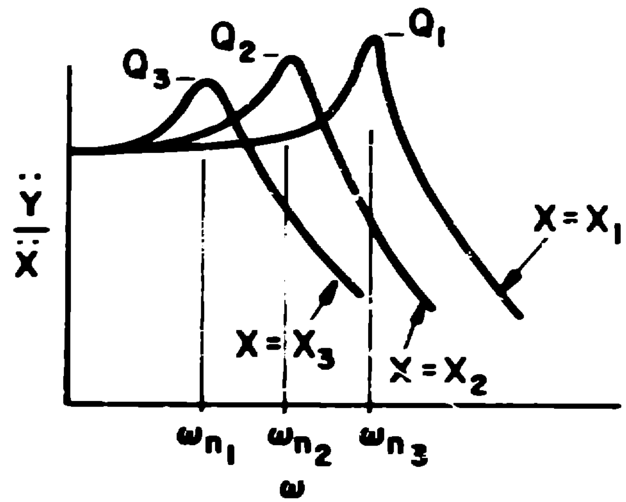
(a) ONE STORY SHEAR WALL



(b) TWO STORY SHEAR WALL

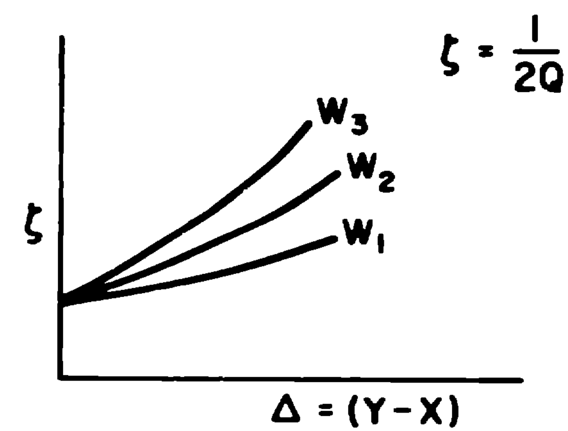


(a) METHOD OF LOADING



(b) MEASURED RESULTS

NOTE: SIMILAR CURVES FOR EACH VALUE OF W



(c) COMPUTED DAMPING

K = LINEAR SPRINGS

D = DAMPING ELEMENTS,

FOR VISCOUS DAMPING

$F_{D_v} \propto C\dot{u}$

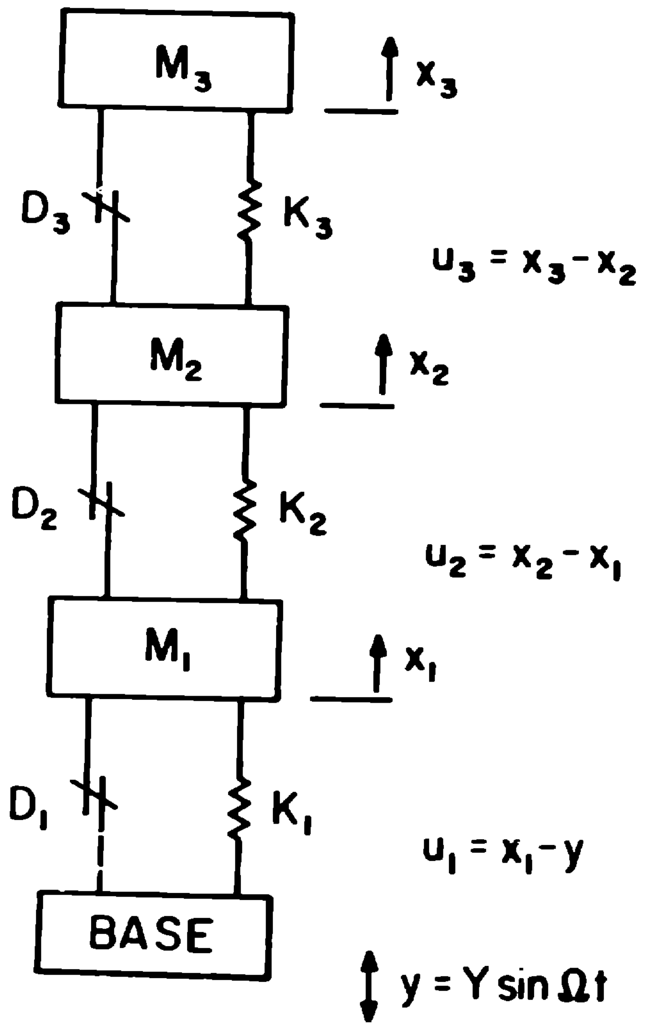
FOR STRUCTURAL DAMPING

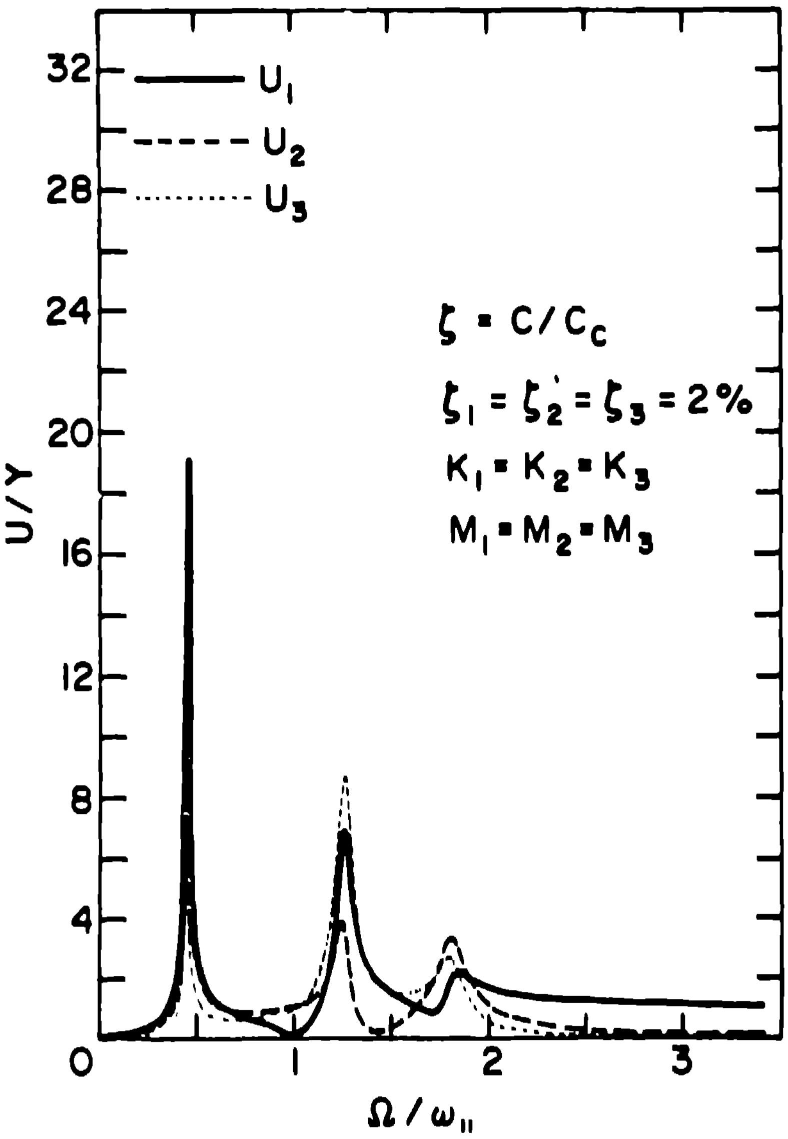
$F_{D_s} \propto hu$

FOR COULOMB DAMPING

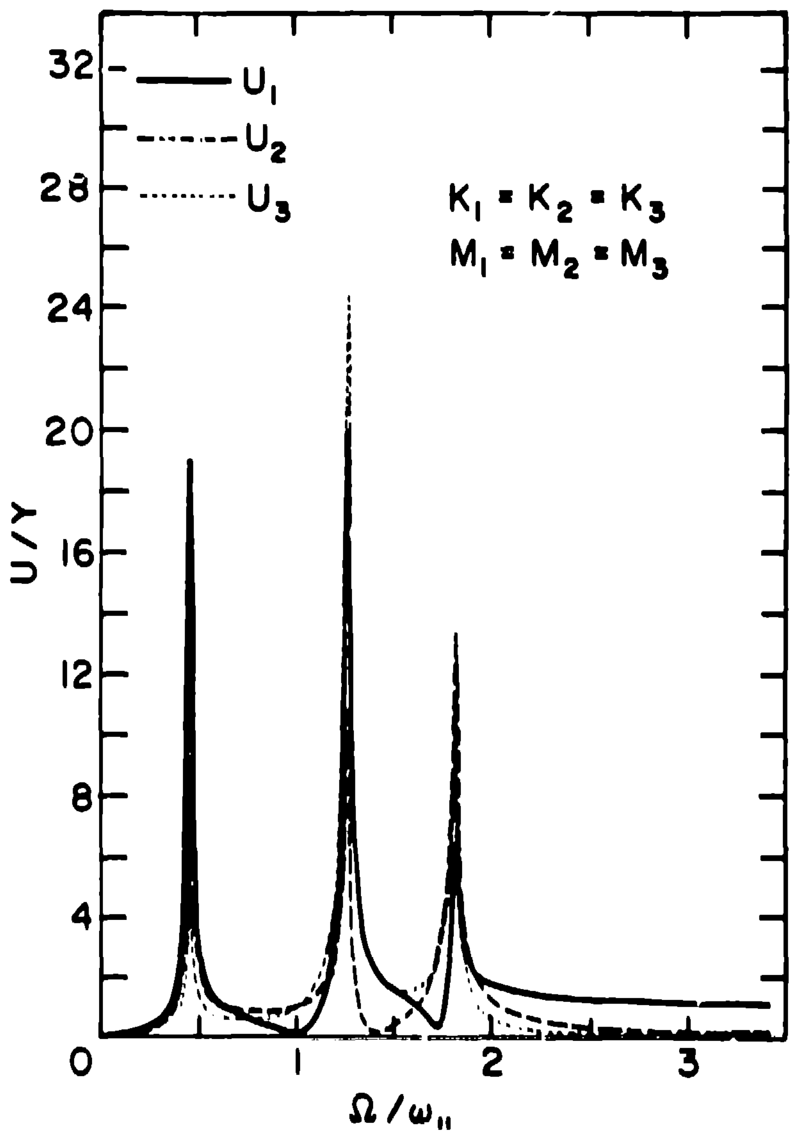
F_{D_c} = A CONSTANT

$\omega_n = \sqrt{K_1/M}$

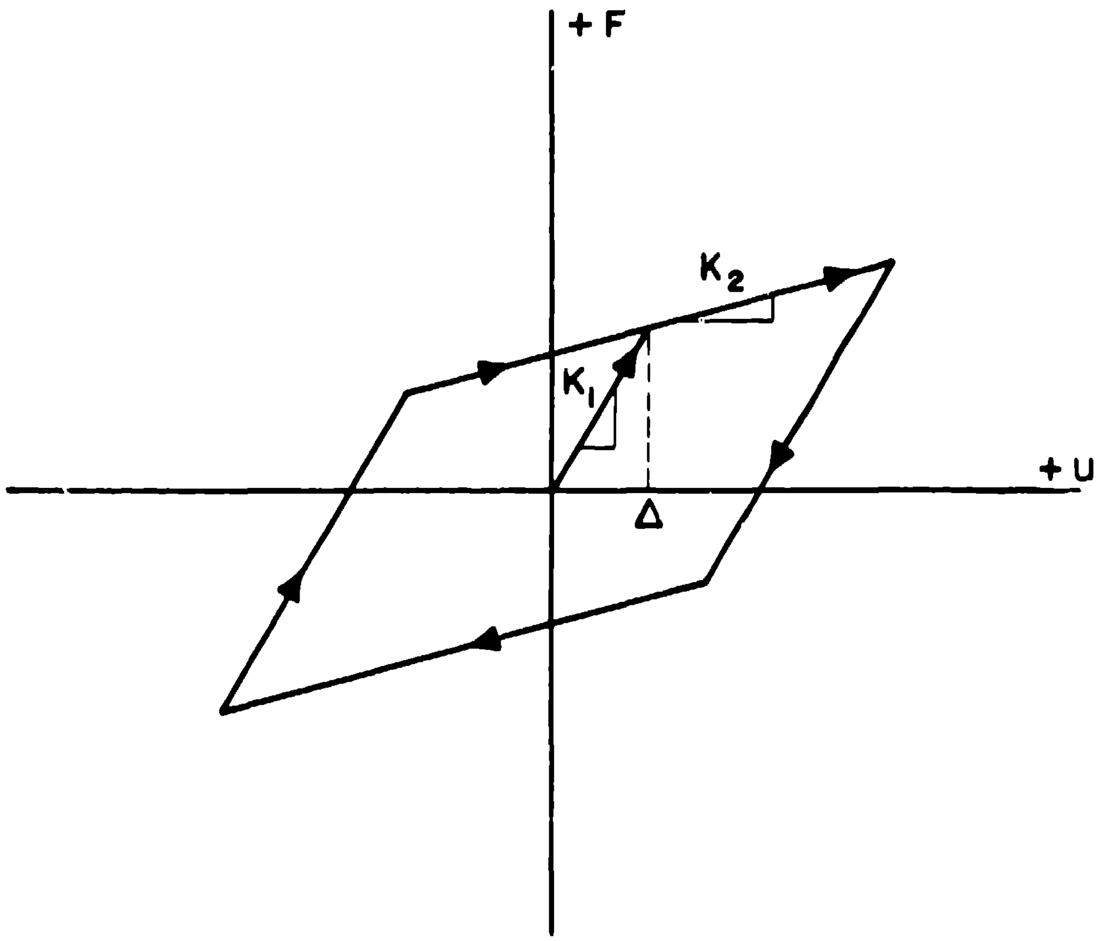


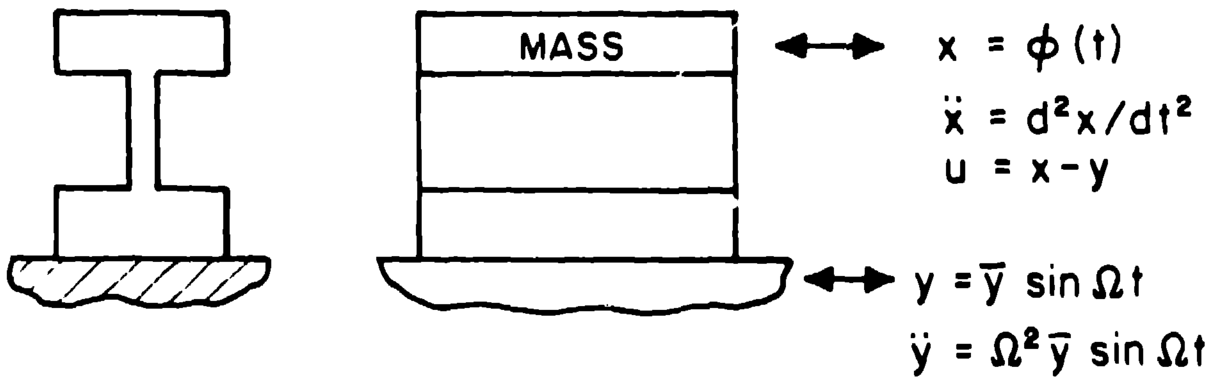


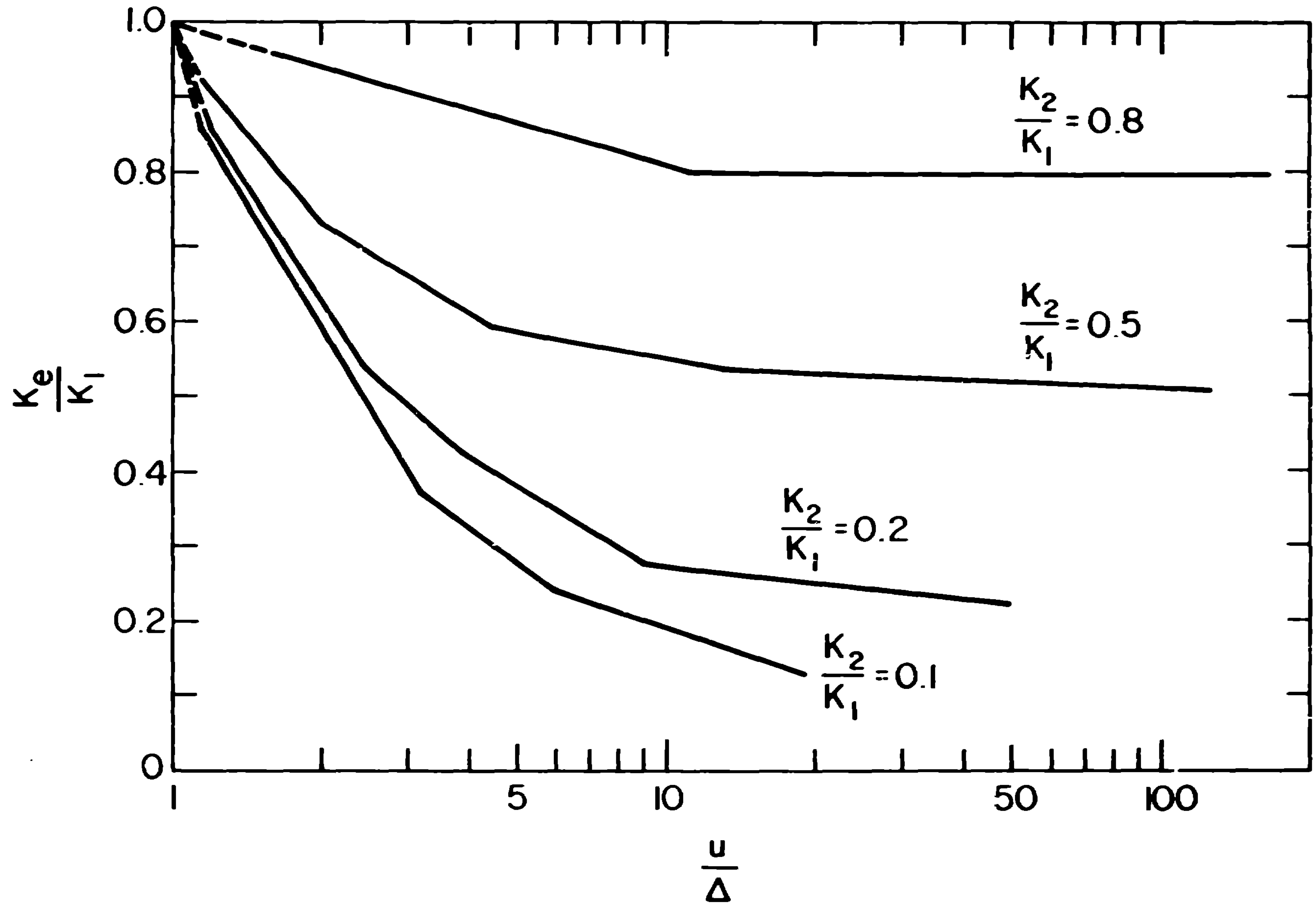
U_1
U_2
U_3
$\zeta = C/C_c$
$\zeta_1 = \zeta_2 = \zeta_3 = 2\%$
$K_1 = K_2 = K_3$
$M_1 = M_2 = M_3$
U/Y
Ω / ω_{11}
32
28
24
20
16
12
8
4
0
1
2
3



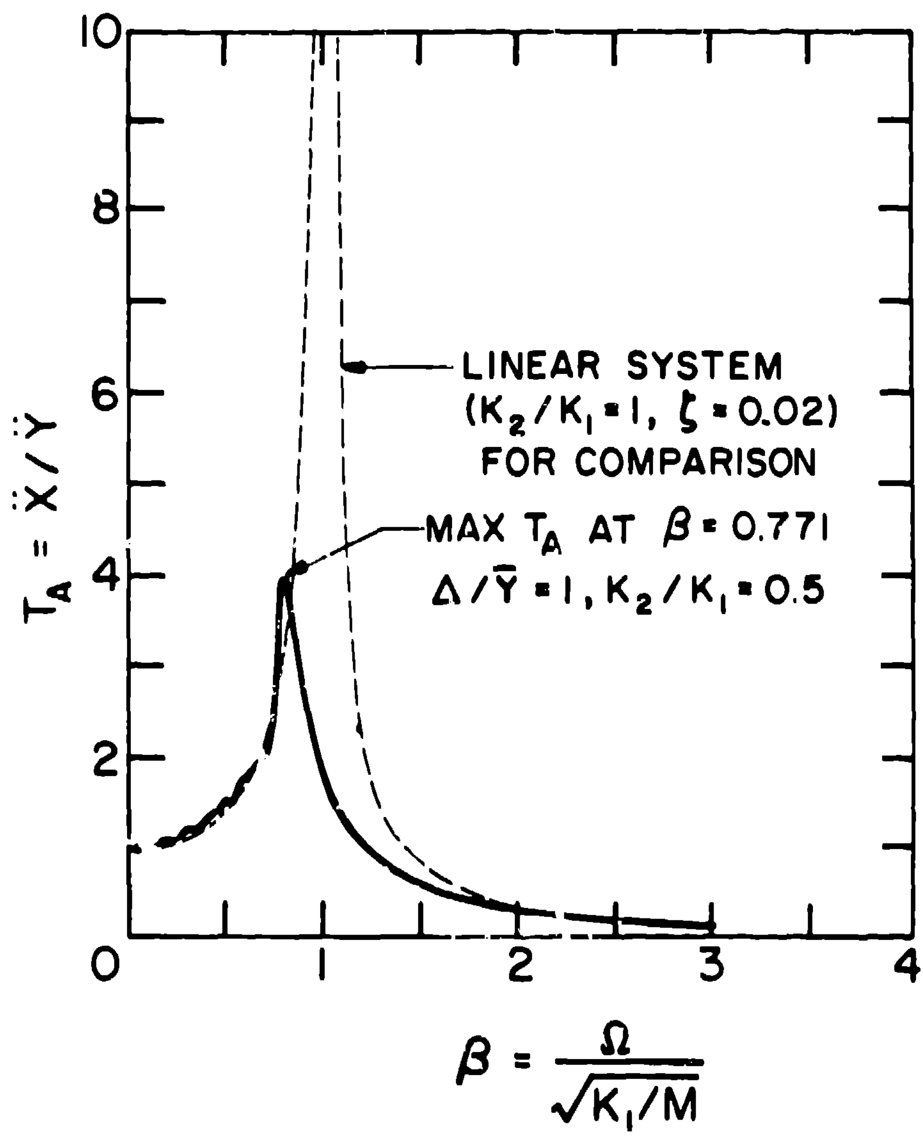
U₁
U₂
U₃
K₁ = K₂ = K₃
M₁ = M₂ = M₃
32
28
24
20
16
12
8
4
0
U/Y
1
2
3
Ω/ω₁₁


+ F
K2
K1
+ U
Δ

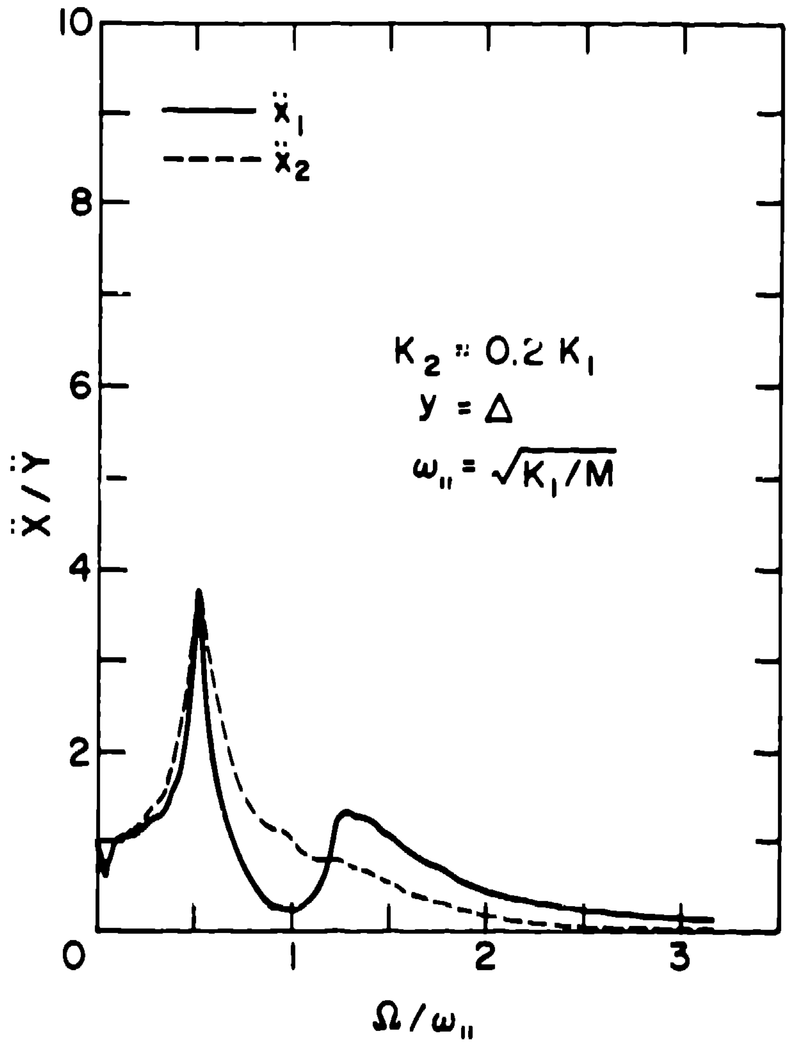

MASS
x = ϕ (t)
ẍ = d²x/dt²
u = x − y
y = ȳ sin Ωt
ÿ = Ω² ȳ sin Ωt



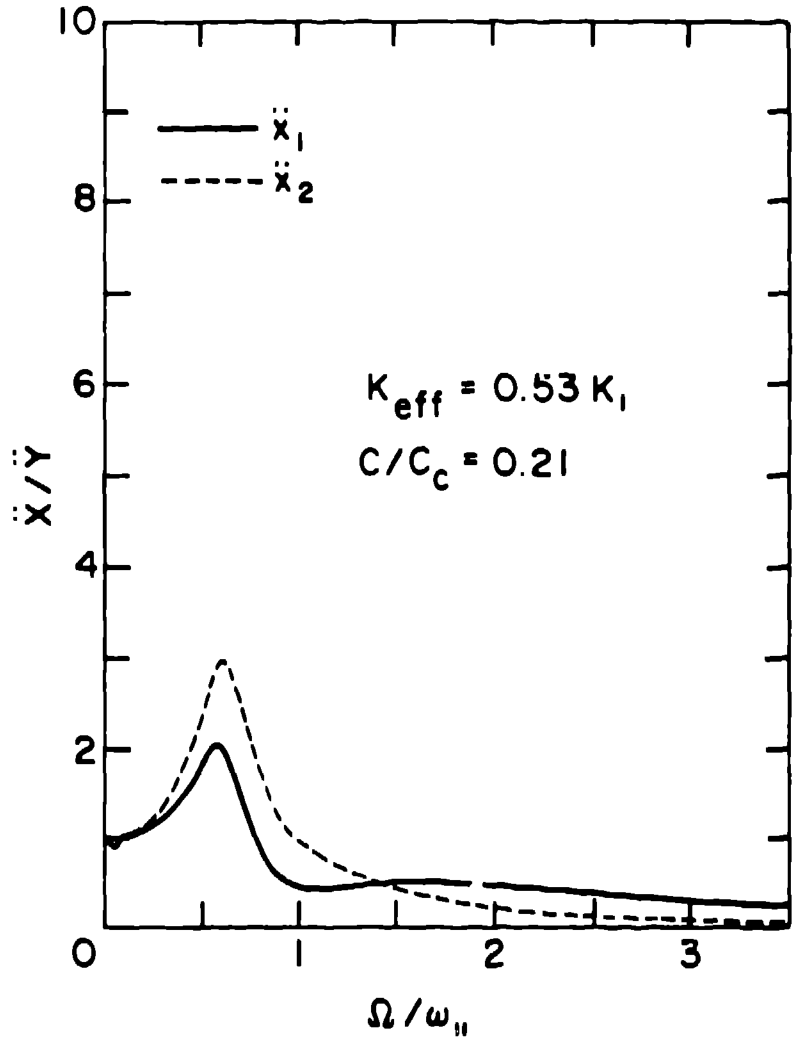
1.0
0.8
0.6
0.4
0.2
0
$\frac{K_e}{K_1}$
$\frac{K_2}{K_1} = 0.8$
$\frac{K_2}{K_1} = 0.5$
$\frac{K_2}{K_1} = 0.2$
$\frac{K_2}{K_1} = 0.1$
1
5
10
50
100
$\frac{u}{\Delta}$



10
8
6
4
2
0
1
2
3
4
$T_A = \ddot{X}/\ddot{Y}$
$\beta = \frac{\Omega}{\sqrt{K_1/M}}$
LINEAR SYSTEM
($K_2/K_1 = 1$, $\zeta = 0.02$)
FOR COMPARISON
MAX T_A AT $\beta = 0.771$
$\Delta/\bar{Y} = 1$, $K_2/K_1 = 0.5$



10
8
6
4
2
0
0
1
2
3
$\ddot{X}/\ddot{Y}$
Ω/ω_{11}
$\ddot{x}_1$
$\ddot{x}_2$
$K_2 = 0.2\ K_1$
$y = \Delta$
$\omega_{11} = \sqrt{K_1/M}$



10
8
6
4
2
0
1
2
3
$\ddot{x}_1$
$\ddot{x}_2$
$K_{eff} = 0.53 K_1$
$C/C_c = 0.21$
$\ddot{X}/\ddot{Y}$
Ω/ω_{11}