PB81-154593

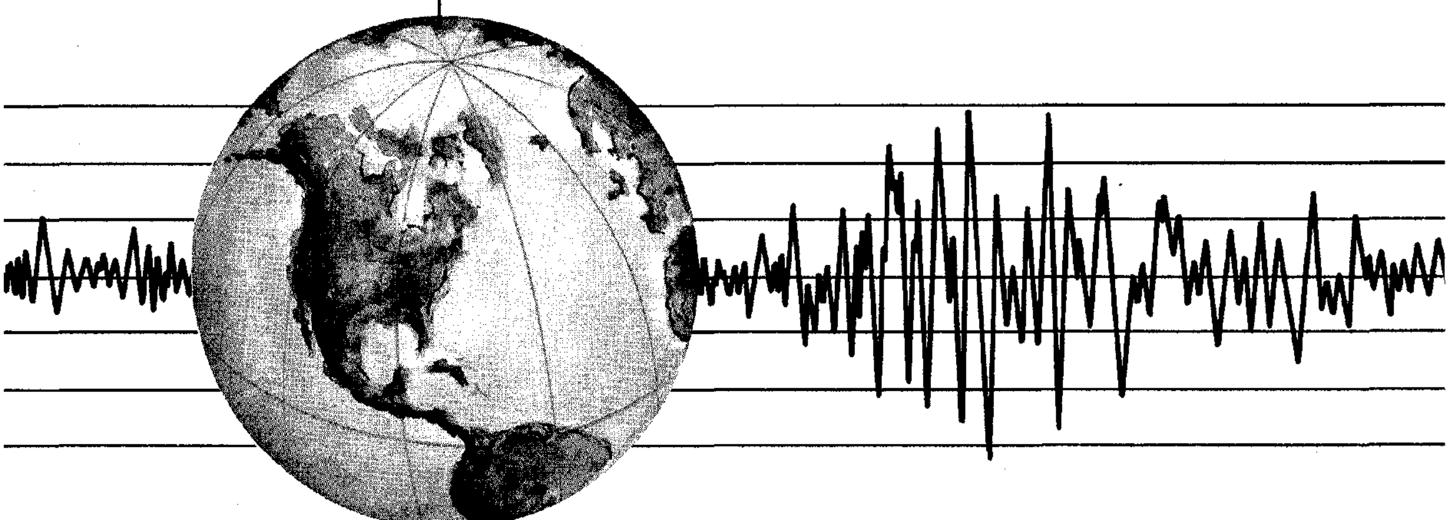
REPORT NO.
UCB/EERC-78/10
JUNE 1978

EARTHQUAKE ENGINEERING RESEARCH CENTER

SEISMIC PERFORMANCE OF NONSTRUCTURAL AND SECONDARY STRUCTURAL ELEMENTS

by
ISAO SAKAMOTO

Report to the National Science Foundation



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BIBLIOGRAPHIC DATA SHEET	1. Report No. NSF/RA-780770	2.	3. Recipient's Accession No. PB81 154593
4. Title and Subtitle Seismic Performance of Nonstructural and Secondary Structural Elements			5. Report Date June 1978 6.
7. Author(s) I. Sakamoto			8. Performing Organization Rept. No. UCB/EERC-78/10
9. Performing Organization Name and Address Earthquake Engineering Research Center University of Calif., Berkeley Richmond Field Station 47th Street & Hoffman Blvd., Richmond, Calif. 94804			10. Project/Task/Work Unit No. 11. Contract/Grant No.
12. Sponsoring Organization Name and Address National Science Foundation 1800 G Street, N.W. Washington, D.C. 20550			13. Type of Report & Period Covered 14.

15. Supplementary Notes

16. Abstracts

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18. Availability Statement	19. Security Class (This Report)	21. No. of Pages
Release Unlimited	UNCLASSIFIED	79
	20. Security Class (This Page) UNCLASSIFIED	22. Price

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ABSTRACT

A methodology is described for the evaluation of the damageability of nonstructural and secondary structural elements under seismic loading. In the first half of the report, problems associated with the assessment of the response of nonstructural elements to aseismic loading—including the classification of such elements and the damage to them, design methods, and methods of evaluation—are addressed. In the second half of the report, a general procedure is developed by which the damageability of secondary structural walls in existing buildings can be evaluated. Damageability is defined as a function of capacity to dissipate energy. A method whereby the mechanical behavior of nonstructural elements can be predicted is described. Acting models for the simulation of the mechanical behavior of assembly models—models of secondary wall systems comprised of unit models of elements within the system—are discussed and their implementation described and illustrated. Examples of the prediction of the mechanical behavior of secondary walls by the models mentioned above are described in the appendices to the report.

ACKNOWLEDGMENTS

The research reported herein was carried out at the Earthquake Engineering Research Center of the University of California, Berkeley, and was partially sponsored by the National Science Foundation. This support is gratefully acknowledged. The author wishes to express his appreciation for the encouragement, suggestions, and review of the manuscript by Professor Emeritus Boris Bresler of the Department of Civil Engineering, University of California, Berkeley, during the course of this work. The author was on leave from the University of Tokyo and was provided the opportunity to stay in the United States as a result of the long history of cooperation of the United States and Japan in the area of earthquake engineering. The work of M. C. Randall, who edited and typed the manuscript, is greatly appreciated.

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1. INTRODUCTION

Although much research has been carried out on the structural safety of buildings under earthquake loading, the seismic behavior of nonstructural elements has received little attention. Earthquake damage to nonstructural elements can have serious consequences, however, as demonstrated by the San Fernando (1971), Managua (1972), and other recent earthquakes. Concern for human safety as well as economic considerations dictate that an effort be made to reduce the potential for damage to nonstructural components of structures as part of the effort to reduce the overall hazard of structures in earthquakes.

Although nonstructural elements are strictly all elements of a building other than those considered to perform a primary structural function, they can nonetheless be classified according to the degree and type of structural action to which their presence gives rise if they interact with primary structural members in an unintended manner. Thus, only elements that are sufficiently strong and rigid to remain in place, but which are wholly unintegrated with the primary structural system will here be termed nonstructural elements. Such elements can affect structural behavior only through inertial force; they add no stiffness to the primary structure.

On the other hand, the rigidity and strength of certain nonstructural walls and/or partitions may be such that the seismic behavior of a primary structure in which they are present will be affected. Such elements, herein termed secondary elements, can, for instance, restrain primary structural elements in an unanticipated manner, as where a rigid infilled masonry spandrel acts on an adjacent column so as to reduce the effective shear span of the column. The stress distribution that results from this interaction will not be that predicted during the design process since the interaction was not considered, and may result in premature failure of the structural element. A relatively rigid secondary wall that is located eccentrically within a structure may give rise to structural eccentricities, such as torsional movement of the structural system, thereby diminishing structural performance.

Nonstructural elements are infrequently designed to resist seismic forces. In order to reduce the hazard posed by such elements in a seismic event, design methods and methods whereby the damageability of secondary elements can be evaluated must be developed. Such methods should be based on well-defined models of the mechanical behavior of secondary elements and should reflect a design philosophy such as that outlined below [6]:

1. Under minor earthquake ground motion, which type of motion may occur frequently during the service life of a structure, secondary elements should undergo no damage.

2. Under moderate ground motion, such as may occur occasionally during the service life of a structure, secondary elements should again suffer no damage.

3. Under severe ground shaking, however, such as may occur only rarely if at all during the service life of a structure, while damage to secondary elements of a sort that could lead to collapse of or serious damage to the primary structure must be prevented, limited damage may be accepted.

In order to implement such a design philosophy, the kind and degree of damage that secondary structural elements can be expected to sustain under varying intensities and durations of earthquake ground motion must be determined. Input loadings from seismic disturbances to nonstructural and secondary structural elements are, however, determined not only by the ground motion to which the primary structure is subjected, but are affected significantly by aspects of the response of the primary structure to the initial ground shock. The two most critical components of the response of a primary structure with respect to the response of nonstructural and secondary structural elements are absolute acceleration—the acceleration of a specified floor of a structure given as two horizontal components and a vertical component—and story drift—the relative displacement of two adjacent stories of a structure given as two horizontal components.

2. SEISMIC PERFORMANCE OF NONSTRUCTURAL ELEMENTS

In this study nonstructural elements are classified as shown in Figure 2.1. Fire protection is a common nonstructural element, especially in steel construction. Architectural elements are also common nonstructural elements. Exterior walls and partitions, some of which may also be defined as secondary elements or secondary walls, are the most commonly recognized type of nonstructural component and are discussed in Section 3 of this report. Mechanical and electrical equipment and the contents of a structure associated with occupancy are also nonstructural elements, but are not considered in detail in this report.

Each category indicated on Figure 2.1 has been further divided into groups, depending on material, building construction, local conditions, etc. For example, major construction types of exterior walls and partitions are discussed in Section 3 where such construction types have been classified both with respect to individual elements and the interface between a given element and a primary structure. This classification by construction type is later used when a mechanical model of nonstructural walls is developed.

2.1 Seismic Performance

Observations of damage to nonstructural elements due to seismic loading are scarce compared to those for damage to primary structures. However, such damage has been documented in some recent earthquakes. Observations from two North American earthquakes as well as observations from several Japanese earthquakes follow. The observations have been summarized by typical damage to given building elements.

The Niigata Earthquake (1964) [1]

Damage to nonstructural elements was often accompanied by damage to the primary structure. Both plastered ceiling and ceiling panels were damaged. Flooring was damaged, most often due to distortion of the diaphragm. Story drift and vibration induced wall finishes to fall. Many panes of glass in flexible structures, such as structures with wooden and steel skeletons, and many fixed glass panes were broken. Clay roof tiles on traditional Japanese housing often escaped dislocation.

The Tokachi-oki Earthquake (1968) [2,3]

Panel finishes on ceilings were not extensively damaged, although some plaster finishes fell. One person was killed when mortar finish fell from the eaves at the entrance to a building. Many fluorescent fixtures fell from ceilings. Mortar and plaster finishes on walls frequently spalled from the body or furring frames. The degree of damage was apparently a function of the thickness of the finish. Mortar and tile finishes spalled due to cracking of concrete walls. Of seventy-two masonry partitions investigated after the earthquake, only in ten structures had masonry partitions been damaged. In some cases, masonry partitions were damaged even where the primary structure remained otherwise intact. Of the same seventy-two structures investigated, glass panes in nine were broken. One hundred twenty fixed glass panes broke in a composite steel skeleton/reinforced concrete structure that suffered no other damage. Roofs were not significantly damaged during this earthquake.

The San Fernando Earthquake (1971) [4]

Ceiling panels were damaged due to hammering at perimeter walls and at moving light fixtures and sprinklers. Pendant lighting fixtures fell. Cracks on exterior structural walls resulted in the failure of exterior finishes, such as plaster. The degree of resiliency of the glazing material in the glass panes determined the extent of damage to window panes. Winches in elevators were dislocated and counterweights were derailed. Inadequately anchored machinery and other mechanical equipment was dislocated.

The Managua Earthquake (1972) [5]

Damage to primary structural elements was related to damage to nonstructural components. Rigid primary structures suffered serious damage while nonstructural elements of these structures escaped extensive damage. However, flexible primary structures suffered little damage while nonstructural elements were seriously damaged.

Ceilings with tee runners were extensively damaged. Pendant lighting fixtures were severely damaged, as were block masonry partitions with plaster finishes. Debris from partitions, block masonry walls, and ceilings blocked stairways that otherwise could have served as emergency exits. Furniture, wardrobes, and bookshelves in the upper stories of high-rise buildings were dislocated. Numerous mechanical components of elevators failed. Equipment installed on vibration absorbers was in some instances dislocated. Most emergency generators continued to function in the aftermath of the earthquake.

Summary

General conclusions to be drawn from these and other recent earthquakes with respect to the response of nonstructural elements during earthquakes and additional conclusions based on the results of experimental work by the author are described in Section 3.2 of this report.

2.2 Damage Criteria

In order to implement a rational design philosophy for earthquake resistance of nonstructural components, the kind and degree of damage that such components can be expected to sustain under varying intensities of earthquake ground motion must be determined. An outline of the mechanism of energy transmission to secondary and nonstructural elements is provided below.

The input loading from a seismic disturbance to a nonstructural element will be determined not so much by the ground motion to which the primary structure is subjected but by the response of the primary structure to that motion. The character of loadings to nonstructural components will depend strongly on the dynamic characteristics of the primary structure in which they are present. The stiffness of the primary structure is the single most important determinant of the seismic response of nonstructural elements. The seismic response of primary structures that must be considered as loading to secondary elements is absolute acceleration, which is usually specified as two horizontal components and a vertical component of the acceleration of a given floor. Story drift, the relative displacement of two adjacent stories of a structure, is also an important determinant of the response of secondary elements to ground motion.

Absolute acceleration induces a dynamic effect on nonstructural elements through inertial force. Horizontal components of absolute acceleration induce vibration of nonstructural components that is similar to the vibration of primary structures in response to ground motion. Fixtures on ceilings and walls, as well as machinery installed on floors, are subjected to horizontal acceleration, sometimes referred to as floor response. Heavy brittle walls, such as unreinforced masonry walls, are often seriously damaged due to forces induced by horizontal acceleration. The component of horizontal acceleration perpendicular to the plane of the wall is primarily responsible for such damage.

Suspended ceilings are vulnerable to damage due to horizontal acceleration. The movement of components that results when suspended ceilings are subjected to horizontal acceleration is complex, and will depend on the number of discrete elements in the diaphragm, the distribution of mass, and the restraint of movement at the perimeter of the zone. The vertical component of absolute acceleration contributes to the effects described above. The coupled effect of horizontal and vertical components of ground motion often causes furniture to overturn. Ceilings and ceiling fixtures subjected to such coupled motion often fall.

Story drift induces deformation of walls and service pipes. The component of story drift that is parallel to the plane of the walls significantly influences such seismic response. These effects are discussed in greater detail in Section 3 of this report.

2.3 Design and Evaluation

Most methods by which nonstructural elements are at present designed to resist seismic forces specify a seismic (force) coefficient that represents the seismic inertial force due to the acceleration of the element itself, and a seismic (story) drift coefficient that corresponds to unacceptable deformation of the element. Architectural procedures by which earthquake resistance is provided for nonstructural elements generally specify methods whereby story drift due to seismic loading can be accommodated [8-10].

The seismic performance of nonstructural elements can be evaluated either before or after construction. Before construction, the design documents for an element can be evaluated; this evaluation should be considered part of the design process, and modifications should, if necessary, be made as the result of such evaluation. After construction, the existing structure should be evaluated. A damageability index is used in such evaluations as discussed in Section 3. The results of such evaluations can then be used to determine whether an element should be strengthened, otherwise modified, or replaced.

The evaluation of existing buildings differs from that for design in several respects. In existing buildings architectural details and material characteristics are fixed, elements may have suffered from deterioration of materials (or the quality of construction materials may have been inferior or superior to that specified in the design documents), and samples can be cut from existing structural elements to gather data.

3. EVALUATION OF THE DAMAGEABILITY OF SECONDARY WALLS

3.1 Scope of the Study

In reports on the seismic response of secondary walls (exterior walls and partitions), damage is sometimes described in terms of mechanical behavior (e.g., breakage), and sometimes in terms of damage caused by mechanical behavior such as breakage (e.g., injuries). Thus, even when the behavior of secondary walls has been observed and perhaps even measured, evaluations are often stated from arbitrary and inconsistent points of view. A method by which the damageability of secondary walls can be evaluated should involve not only a logical procedure by which the mechanical behavior of such elements is ascertained, but also a definition of criteria by which projected levels of damage can be assessed.

When a secondary element of an existing building which has not undergone earthquake loading is to be evaluated, the first step is to determine the mechanical behavior of the element under probable levels of seismic loading. However, even in detailed evaluations of the seismic damageability of structural and nonstructural elements [1], the mechanical behavior of nonstructural elements is frequently left to the qualified judgment of experts; that is, behavior is not predicted by any consistent methodology and may be stated in qualitative terms only.

The mechanical behavior of a secondary wall should be determined by the same procedures used to determine the behavior of a primary structure. The following steps are necessary for an adequate evaluation:

1. a mechanical model of the secondary wall must be developed;
2. probable loadings must be determined and used as input to the model of the wall under consideration; and
3. the seismic behavior of the secondary wall must be simulated based on the above.

The following types of seismic response values should be determined:

1. story drift parallel to the plane of the wall; and
2. acceleration or inertial forces perpendicular to the plane of the wall.

In this study, the method by which the response of secondary walls is evaluated has been developed for secondary walls subjected to in-plane story drift. The interaction of the secondary wall and the primary structure should be considered. However, if the mechanical characteristics of the secondary wall are known, the effect of interaction can easily be evaluated by considering together the secondary wall and the primary structure as a single system.

In order to model the diverse types of secondary wall in structures, unit and assembly models, as described below, are used. Unit models may be either element or interface models; that is, they model an isolated wall or the interface between a primary structure and the element in place. Assembly models are comprised of unit models and represent the overall structural system considered in a given analysis.

In addition to mechanical behavior models, criteria for damageability must be defined. The following categories of damageability index must be considered:

I. Basic Damageability Index

II. Performance Damageability Index

   a. Cosmetic Damageability Index

   b. Structural Damageability Index

   c. Functional Damageability Index

III. Social Damageability Index

a. Life Safety Damageability Index

b. Economic Damageability Index

The basic damageability index, the prime index of damageability, is defined in terms of mechanical behavior prior to failure. The performance damageability indices are intended to indicate the social consequences of damage.

3.2 Mechanical Model

A mechanical model of a secondary wall is established based on information on damage accumulated in the aftermath of earthquakes and on experimental data.

From previous earthquakes the following patterns of response are apparent:

1. Most damage is caused by in-plane story drift.
2. Finishes such as boards, tiles, and plaster frequently peel off wall bodies to which they are attached due to story drift and distortion of walls.
3. Heavy masonry walls frequently deform out-of-plane due to inertial forces induced by acceleration.
4. Heavy finishes such as thick plaster and mortar fall from ceilings and floors due to inertial forces.
5. Unintended confinement results in unanticipated loading on elements and may induce damage to both secondary and primary structural systems.

The following additional information on patterns of response has been derived from experimental work performed by the author.

1. Patterns of damage are generally closely related to type of construction.
2. Geometric characteristics such as aspect ratio and clearance at boundaries affect the degree of damage.

3. The rotation of an element greatly contributes to the accommodation of story drift.
4. Story drift can be accommodated by cumulative slip or looseness at construction joints.
5. The mechanical characteristics of layered walls are governed by restoring forces caused by the differential movement of the body and the finish at the joint connecting them.

Unit Model

The unit model represents a basic component of a secondary wall, either exterior or partition. Unit models are categorized by their geometry and may be denominated as either layer types or in-frame types. The layer wall is generally a wall body and at least one overlay or finish layer. Each layer of such a unit model is classified as one of several types according to its construction. The linear behavior of a frame-panel wall, a type of layered wall system, is described in Appendix A.

In-frame walls contain openings such as windows and door frames. The unit model consists of an outer frame and an inner component that may be a sliding or a glass door or window. The layers of components are jointed at the interface between them.

The mechanical characteristics of a unit model depend on the conditions of confinement. If all four sides of the unit are surrounded by strong, rigid members and all four corners are hinged, the unit can only deform into the shape of a parallelogram. Such units are herein termed self confined; the confinement is such that the length of each side of the unit will remain constant and straight under loading (Figure 3.2). Accordingly, the shear-distortion characteristics of a self-confined element are fixed regardless of boundary conditions. If such confinement is imperfect, the element is termed unconfined.

The mechanical characteristics of unconfined elements vary depending on the effects of confinement at the interfaces of such elements, especially for elements such as concrete block walls. Therefore, when experimental data on the mechanical characteristics of such elements are gathered, the boundary conditions must be clearly noted for the data to be meaningful.

Interface Model

The response at the interface between a wall and a component of a primary structure is more complex than the response of the wall alone. Moreover, the character of the interface will significantly influence the behavior of secondary walls. Interface models are classified by boundary component and geometric relationship between the boundary and the element (see Figure 3.3), or by the material and/or mechanism used to create the interface and the layout of the interface.

(i) Boundary Component and Geometric Relationship Parameters

Boundary Components	Geometric Relationship
a. component of primary structure (column, beam)	a. in-plane or transverse
b. intermediate component (stud, runner)	b. parallel or nothing
c. similar element	
d. none	

Intermediate components are significant since the geometric relationship between the side or end of an element and a boundary component often confines the element, especially when the end of an element faces a boundary component. For case (e) in Figure 3.3, in which the end of the element faces the runner and the element is parallel to the beam, the runner may provide geometric confinement.

(ii) Material/Mechanism of Joint and Layout Parameters

Material/Mechanism	Layout
a. welding, adhesive	a. discrete—one, two or more
b. nail, bolt	b. continuous
c. special device	
d. contact pressure	

The materials and mechanisms of a joint govern the mechanical characteristics of the interface. Welding or adhesives form rigid joints, while nailed or bolted joints are somewhat more flexible. A fastener is a special device that allows free horizontal movement, for instance near a loose hole, to accommodate story drift. Even if no bearing joint is intended, horizontal racking forces are often transmitted to an element through contact of the element and the boundary component.

A pair of discrete joints, located opposite each other as in Figure A.3, A or B, will allow for free rotation of the element. In other layouts, joints will interact to produce a complex distribution of forces. In the plane of an element, an interface along one side of an element allows, at most, three degrees of freedom: along the interface, transverse to the interface, and rotational. Each degree of freedom is specified by the following three forms of mechanical behavior: free movement, material deformation, and geometric confinement. The material deformation is deformation governed by elastic behavior, plastic behavior, and ultimate failure. If intermediate components are present at the interface the mechanical characteristics of such components will influence the behavior of the assembly as well as the element model.

Assembly Models

Assembly models combine element and interface models. Numerous combinations, such as are illustrated in Figure 3.4, are possible. Two factors are considered: the combination of boundary components according to the interface model and the input side, the determination of which depends on the material and mechanism of joints at the interfaces of elements. An input side is that side in which a horizontal racking force due to story drift is transmitted from the primary structure to a secondary element. If there are effective joints along the top and bottom sides of a frame capable of carrying shear force, these sides are considered input sides. If there are effective joints along the left- and right-hand sides of a frame that can transfer horizontal racking force (that is, where both sides can transfer moment to an element), these sides are also considered input sides.

Reactions produced by horizontal racking forces transmitted to an element through input sides are considered to be confined as shown in Figure 3.5. Where no such confinement exists, no racking force is transmitted and the element is free to rotate.

Under certain circumstances horizontal racking forces will cause an element to deform in one of two possible modes. If an element is confined it will deform as a parallelogram. If unconfined, it may deform in a number of ways as illustrated in Figure 3.6. In the latter case the mechanical characteristics of the element vary with confinement conditions (such as direction and rigidity and strength of confinement), which characteristics are closely related to the mechanical characteristics of interface models. The confined and unconfined cases are illustrated in Figure 3.7. The top and bottom sides of the elements in this figure are assumed to be input sides. It is also assumed that no vertical movement will occur at the top and bottom sides and that these sides provide confinement. If there are studs and they are sufficiently rigid and strong, they will confine both the right- and left-hand sides of the element as shown in Figure 3.7(a). Such confinement results in deformation of the element as a parallelogram. If, however, there are no studs and both the right- and left-hand sides of the element are unconfined, the deformation pattern of the element may differ as shown in Figure 3.7(b).

Acting Model

Since nonstructural components are not necessarily designed according to computations of structural response, it is difficult to evaluate their behavior under seismic loading. Although the best method of determining the behavior of nonstructural elements would be experimental simulation of their response to seismic loading, testing of all types of elements is not practicable. The concept of an acting model by which the mechanical behavior of nonstructural components under seismic loading can be simulated is therefore required for adequate evaluations of behavior. If the force-displacement relationship for a given element is available, there is of course no necessity to resort to this acting model.

Acting models are used to predict the mechanical behavior of nonstructural elements that differ only in geometry from an element for which there are experimental data. The predictions are made through interpolation or extrapolation of values from experimental results. The accuracy of predictions from an acting model depends on the similarity of breakage patterns of the model when compared to those of the experimental specimen. Interpolation of results is preferred to extrapolation in order to ensure similarity in breakage pattern. An acting model can yield the force-displacement relationship for an element; this relationship is then used to compute the basic damageability index for the element under consideration. When secondary elements and the primary structure interact, the force-displacement relationship from an acting model can be used to compute the response of the integrated system to seismic loading. An acting model may be established in a number of ways, two examples of which—an extension model and an equivalent model—are provided in Appendices B and C.

3.3 Simulation of Mechanical Behavior

Story drift and inertial forces due to absolute acceleration are the most significant loadings to which nonstructural components are subjected as a result of input loading to primary structures. Story drift comprises two components of response, one parallel to the plane of a secondary wall and another perpendicular to the plane of the wall. When a secondary wall is subjected to story drift in its plane, the wall and/or its interface with the primary structure is forced to deform and may be damaged. Such damage is more probable than damage from drift perpendicular to the plane of the wall, since the latter is easily accommodated and may therefore be neglected in most cases.

Inertial force from absolute acceleration also comprises two components of response, one parallel to the plane and another perpendicular to the plane of the secondary wall. If a wall is sufficiently heavy to be affected by inertial forces, it will often fall out of the plane, suggesting that the effect of inertial force perpendicular to the plane is generally more significant than that parallel to the plane.

Both in-plane story drift and out-of-plane inertial force affect the behavior of secondary walls. However, one of two effects ordinarily dominates the response of secondary walls to earthquake loading. The first step in developing an analytical procedure is thus to establish a method of determining the magnitude of these effects. This report focuses on story drift; inertial force is not explicitly treated.

A force-displacement diagram may be obtained experimentally for a wall for a specified loading. However, to obtain force-displacement diagrams for a range of loadings, additional experimentation must be carried out. If an assembly model of a wall has been established and its mechanical behavior computed, the force-displacement diagram for the unit models of which the assembly model is comprised as well as the assembly model can be obtained. The force-displacement diagrams thus obtained for unit models provide detailed information on the behavior of each element of a secondary wall; simulation of behavior for any loading condition is possible.

Predictions of mechanical behavior are used to determine the critical condition or final phase for a specified performance. Such descriptions of behavior should include the behavior of the element as a whole as well as that of parts of the element. The element as a whole is represented by the assembly model, while each part is represented by a unit model. Even if damageability is evaluated based on experimental data rather than on analytical results, it is convenient to identify the parts of the system under consideration by the terms "interface" and "element."

Force-displacement diagrams obtained from experimental programs reflect both visible and invisible deformation and breakage of walls. While observations are limited to information on visible deformation and breakage, such observations can be used to determine cosmetic damageability indices. Observations may also provide information crucial to the determination of other indices. When an experiment is carried out, the following aspects of behavior should be noted:

1. story drift;
2. the part (or unit model) that undergoes damage;
3. the type of breakage;
4. the degree of breakage.

That part of a wall that undergoes damage is referred to as an interface or an element. The degree of breakage is not necessarily expressed in a standard form. Such damage may be stated qualitatively as "overall" or "partial," or numerically as "the crack width is XX mm." Assembly models can provide force-displacement diagrams which are similar to those obtained in experimental investigations. Simulation of behavior by the use of an assembly model can also provide information on the behavior of individual unit models of which the assembly model is comprised, such as kind and degree of deformation. From such information the type and degree of damage to a secondary wall can be estimated by analogy to experimental data. The analogy method is not yet well developed; elaborations with respect to nonstructural and secondary elements are needed.

3.4 Evaluation of Damageability

The damageability of secondary walls is evaluated according to the philosophy outlined in reference 12. Three indices—the basic damageability index (Category I), the performance damageability index (Category II), and the social damageability index (Category III)—are determined. The latter two indices are derivatives of the first.

When a force-displacement diagram is available for a secondary wall, the basic damageability index (Category I) is defined as a function of energy consumption due to breakage. The energy consumption is obtained from the force-displacement diagram for the assembly model. The basic damageability index (BDI) is obtained as follows:

$$(BDI) = \frac{AREA \ \ ABEF}{AREA \ \ ABCD}$$

where area ABEF (Figure 3.8) is the energy dissipated during loading, and area ABCD (Figure 3.8) is the energy dissipated if loading is continued until ultimate failure. The curve ABE in Figure 3.8 represents loading and is either a virgin curve or an envelope curve of cyclic loadings. The curve EF represents unloading and should correspond to observed behavior. The curve as a whole, curve ABCD in Figure 3.8, is the force-displacement diagram for behavior to ultimate failure.

In most reports of earthquake damage, damage is described in terms of cosmetic damage, structural damage, and so on. Such qualitative descriptions are predicated on the degree of social value attached to the damage. For instance, the presence of hairline cracks on a wall reflects cosmetic degradation that may or may not be viewed as damage. On the other hand, extensive cracking may pose a threat to life safety from mechanical failure of the wall and clearly should be viewed as significant damage. The emphasis placed on various types of damage must be consistent in order that a standard evaluation of performance damageability can be made.

The performance damageability indices (Category II) are designed to enable an evaluation of the damageability of nonstructural elements. Three indices are specified as typical performance damageability indices: a cosmetic damageability index, a structural damageability index, and a functional damageability index. The latter is not a single index, but several indices that differ one from another according to the function of the element under consideration (acoustical, insulating, etc.). A numerical scale for damage is determined from the basic damageability index; the mechanical behavior determines the critical conditions or final phase for a specified performance; the performance index becomes the maximum value. A performance damageability index can never be 100% since even after ultimate failure, an element will retain some strength or rigidity.

The structural damageability index will be used as an example of the performance damageability indices. This index represents the degree of threat from falling walls. To determine the final phase with respect to structural safety, the mechanical behavior should be examined in connection with the following:

(1) To determine the point at which the final phase is reached:

  (a) determine whether or not catastrophic sudden failure of the element is likely to occur (such as brittle failure of the element or the interface);

  (b) determine the structural stability of the element from the force-displacement diagram—a negative slope indicates loss of stability.

(2) To determine the percentage of the damageability at the final phase:

(a) determine whether the failure is partial or overall (degree of failure is clearly related to the maximum value of the damageability index);

(b) determine whether the wall is safe—the structural damageability index for a secondary wall could be 0% for any magnitude of story drift if the wall is a structural fail-safe system.

The social consequences of damage are evaluated by the social damageability index. The safety damageability and economic damageability indices may serve as examples of the process involved in determining the social damageability index. The safety damageability index represents human safety during and after an earthquake. The index represents not only such clear threats to safety as falling elements, but also such threats as the blockage of exits by fallen nonstructural elements. Economic damageability is difficult to define. Even if, for instance, nonstructural elements and a primary structure could be repaired economically, the loss of the use of the building during rehabilitation could result in considerable financial loss. All such costs must be considered.

Earthquake loads (story drift) and damage criteria must be specified in order to carry out an evaluation of seismic damageability. A mechanical model of the wall must be established. For this purpose one of the types of assembly model illustrated in Figure 3.4 must be specified for the secondary wall under consideration. The types of unit models (element and interface models) of which the assembly model consists must then be specified. After these have been specified, the assembly model, as a mechanical model, is completed by introducing the specific mechanical characteristics, such as stiffness and nonlinearities, of the element under consideration.

Since data available on the mechanical characteristics of secondary walls are scarce, experimentation is often necessary. If experimental data are not available and experimentation is impracticable, an acting model can be used to simulate the mechanical behavior of secondary walls. The simulation of such behavior is performed using the assembly model for the element. The behavior of each unit of which the assembly model is comprised is traced to ultimate failure. The evaluation of damageability is based on the results of such simulations. A basic damageability index is first computed based on resultant force-displacement diagrams. The evaluations of damageability for Categories II and III are derived from the basic damageability index.

4. CONCLUSIONS

A procedure has been described by which the damageability of nonstructural elements can be assessed. Two concepts, that of the assembly and unit models and that of the damageability index, have been introduced. The assembly model is comprised of unit models—either element or interface models. This method of establishing a mechanical model of a nonstructural element can also be used as a method of systematically documenting experimental results. The acting model discussed in the report is used to simulate the mechanical behavior of assembly or unit models for which experimental data are available.

From the results of computations using the above models, a damageability index for secondary elements is derived, providing a quantitative basis on which the damage of such elements can be assessed using specified criteria [12]. The performance and social damageability indices have also been discussed in the context of evaluating the probable response of nonstructural elements to seismic loading.

In order fully to realize the potential of the techniques described in this work, additional research is necessary, especially in the following areas:

1. Characteristics of Seismic Loading on Secondary Walls

   The magnitude of story drift and the number of cycles of loading during a seismic event must be better defined. Types of loading (acceleration, displacement/in-plane, out-of-plane) and the effects of the interaction of such loadings must be considered.

2. Documentation of Experimental Data on the Response of Secondary Walls

   Experimental data are at present either scarce or fragmentary. More experiments must be carried out and such data documented in a more consistent quantitative rather than qualitative manner, as discussed in this report.

3. Mechanical Behavior of Nonstructural Elements

   While the focus of this report has been on the response of secondary walls, the response of other nonstructural elements should be assessed better than is done at present. In particular, the effect both of story drift and acceleration and that of the mechanical characteristics of individual elements must be more accurately modeled.

4. Evaluation

   Detailed studies must be undertaken of the relationship between breakage and specified damage criteria, especially with respect to performance damageability indices.

5. Existing Buildings

   Construction details of nonstructural elements in existing structures and the condition of materials in-place should be documented. Materials inevitably deteriorate; the extent and rate of such deterioration should be evaluated to enable a more accurate assessment of the probable response of secondary elements in existing buildings.

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APPENDIX A

Theoretical Linear Behavior of Frame-Panel Walls

The two major types of secondary wall systems are layered and in-frame walls. A furring frame with an overlay panel is a common form of layered wall. The behavior of this type of wall is investigated below using the procedure outlined in this report and with the following assumptions:

(a) The geometry of the furring frame as well as that of the panel and the layout of the joints is symmetrical about the center of the element.

(b) The furring frame (without the overlay panel) is sufficiently flexible to allow the element to deform as a parallelogram without constraint.

(c) The panel is sufficiently rigid so that it does not deform in its plane.

(d) The behavior of the joint between the furring frame and the panel is linear-elastic.

When subjected to a horizontal racking force due to story drift, such a furring frame will tend to deform as a parallelogram and the panel will not deform in its plane but will rotate about its axis of symmetry (Figure A.1).

The differential movement of the furring frame and the overlay panel produces restoring forces due to the relative displacement at the joints. This relative movement occurs such that the rotational moment transmitted to the panel by the restoring force at the joint maintains equilibrium. Based on this theoretical scenario of response and the above assumptions, an analysis was performed of the response of a hypothetical secondary wall with respect to the geometric characteristics of aspect ratio (height to width) and number of subdivisions in the wall.

When a frame-panel wall is subjected to horizontal racking forces, the frame and panel move differentially as illustrated in Figure A.1. However, since the wall is symmetrical, the centers of the frame and the panel move identically; that is, they both shift horizontally. The distance between the original position O and the shifted position O_s is equal to half the story drift δ. The coordinates referred in the following discussion are based on reference axes in which the shifted center O_s is regarded as the point of origin. Movement of points on the furring frame and on the panel are indicated in Figure A.2.

A point on the furring frame is considered to move horizontally since if the story drift is small the change in position in the vertical direction will also be small and can be neglected. However, a point on the panel will move according to the rotation along the shifted center O_s. Story drift is indicated by the angle θ_F in radians, the ratio of story drift δ to the height of an element H. The movement of a point on the furring frame is indicated by $\overline{P_S P_F}$ on Figure A.2; its horizontal displacement $\overline{P_S P_F}$ is given by $\theta_F y$. The rotational angle of the panel is indicated by θ_P.

The resultant relative displacement $\overline{P_p P_F}$ between two points is given by:

$$\begin{aligned}\overline{P_p P_F} &= \sqrt{(\theta_p x)^2 + \{(\theta_F - \theta_p)y\}^2} \\ &= \theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2} \qquad \text{(A.1)}\end{aligned}$$

where $r = \theta_p / \theta_F$.

The restoring force at this joint is:

$$R\,\theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2} \qquad \text{(A.2)}$$

where R is the stiffness of the joint. Accordingly, the two components of this restoring force in the x- and y-directions are:

$$\begin{aligned}F_x &= R\,\theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2} \cdot \frac{(\theta_F - \theta_p)y}{\theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2}} = R\,\theta_F (1-r) y \\ F_y &= R\,\theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2} \cdot \frac{\theta_p x}{\theta_F \sqrt{r^2 x^2 + (1-r)^2 y^2}} = R\,\theta_F r x \qquad \text{(A.3)}\end{aligned}$$

Finally, the equilibrium of rotational moment around the shifted center O_s is expressed as follows:

$$\begin{aligned}M &= \sum (F_x (y - F_y) x) \\ &= \sum \{R\,\theta_F (1-r) y^2 - R\,\theta_F r x^2\} \\ &= R\,\theta_F \sum \{(1-r) y^2 - r x^2\} = 0 \qquad \text{(A.4)}\end{aligned}$$

The value of r is computed using this equation. The summation of the above equation need be carried out only for a quarter of the panel due to symmetry. The horizontal racking force carried by the frame-panel wall, which should be calculated for the upper half of the wall, is obtained using the value of r as calculated above in the following expression:

$$P = \sum F_x = \sum R\,\theta_F (1 - r) y$$

The aspect ratio is defined as follows:

$$\alpha = H/B \tag{A.5}$$

where H is the height of the element and B the width. If a panel is divided into two or more panels, the variation of the aspect ratio will depend on the direction of applied load as shown in Figure A.3 and defined by the following expressions for vertical and horizontal application of load, respectively:

$$\alpha = \frac{H}{B/n} = \frac{nH}{B} = \textit{vertical application}$$

and

$$\alpha = \frac{H/n}{B} = \frac{nH}{B} = \textit{horizontal application}$$

where n is the number of subdivisions.

In the examples described below, the rotation ratio r is given by θ_p/θ_F. The horizontal relative displacement along the horizontal boundary between panels HRD—that is, the displacement between the panel of the element under consideration and that of an adjacent element or between divided panels in an element (see Figure A.4)—and the vertical relative displacement along the vertical boundary between elements VRD are expressed, respectively, as in Equations (A.7) and (A.8) below:

$$\begin{aligned} &\theta_F H(1 - r)\text{: } \textit{single panel} \\ &\theta_F H(1 - r)\text{: } \textit{vertical application} \\ &\frac{\theta_F H}{h}(1 - r)\text{: } \textit{horizontal application} \end{aligned} \tag{A.7}$$

$$\begin{aligned} &B\,\theta_F r\text{: } \textit{single panel} \\ &\frac{B}{h}\theta_F r\text{: } \textit{vertical application} \end{aligned}$$

$B\,\theta\,r$: horizontal application (A.8)

The initial stiffness of the element K is defined as P/δ where P is horizontal racking force and δ story drift.

Joint Layout Type A

If there are only two joints in a panel, one at the top and the other at the bottom, no discrepancy between points at joints on the frame and on the panel will exist. The results of the necessary computations are then trivially simple.

Joint Layout Type B

If there are two joints only in a panel but one at each side rather than at the top and bottom as for type A, the computations are again trivially simple.

Joint Layout Type C

If there are joints at each of the four corners of the frame, the above-mentioned equilibrium equation for the rotational moment becomes:

$$r\frac{B}{2}^2 - (1-r)(\frac{H}{2})^2 = 0$$

$$r = \frac{(\frac{H}{2})^2}{(\frac{H}{2})^2 + 1} = \frac{\alpha^2}{\alpha^2+1}$$

Joint Layout Type D

If the joints are uniformly and closely distributed along the top and bottom edges of a panel, these joints can be assumed to form a continuous joint and the equilibrium equation is:

$$\int_0^{\frac{B}{2}} \{rx^2 - (\frac{H}{2})^2\}\,dx = 0$$

$$r = \frac{3\alpha^2}{3\alpha^2+1}$$

In this and the following cases the stiffness of the joints R represents stiffness per unit length.

Joint Layout Type E

If the joints are uniformly and closely distributed along edges on both sides of the frame, the joint can be assumed to be continuous and the equilibrium equation is expressed as follows:

$$\int_0^{\frac{H}{2}} \{r\left(\frac{B}{2}\right)^2 - (1-r)y^2\}\, dy = 0$$

$$r = \frac{\alpha^2}{\alpha^2+3}$$

Joint Layout Type F

If the joints are uniformly and closely distributed along all edges of the panel, the joint can be assumed to be continuous and the equilibrium equation is:

$$\int_0^{\frac{B}{2}} \{rx^2 - (1-r)\left(\frac{H}{2}\right)^2\}\, dx + \int_0^{\frac{H}{2}} \{r\left(\frac{B}{2}\right)^2 - (1-r)y^2\}\, dy = 0$$

$$r = \frac{\alpha^2(\alpha+3)}{(\alpha+1)^3}$$

Joint Layout Type G

If the joints are uniformly distributed over the panel as a whole and are close together, these joints can together be assumed to form a continuous joint and the equilibrium equation is:

$$\int_0^{\frac{H}{2}} \int_0^{\frac{B}{2}} \{rx^2 - (1-r)y^2\}\, dx\, dy = 0$$

$$r = \frac{\alpha^2}{\alpha^2+1}$$

This expression for r is identical to that for case C above. The stiffness of the joint R represents the stiffness of the element per unit area.

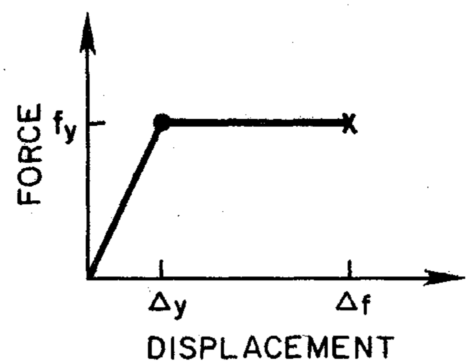
The results of a parametric study based on the formulation shown in Table A.1 are shown in Figure A.5. Graphs for $n = 1$ represent the fundamental solution for the single panel. Graphs for $H2$ and $H3$ and for $V2$ and $V3$ represent the effect of horizontal and vertical application of load for 2 and 3 divisions of the panel, respectively.

APPENDIX B

Nonlinear Behavior of Frame-Panel Walls

The nonlinear behavior of an assumed type of secondary wall is herein investigated using data from experiments carried out on a similar wall. The purpose is to demonstrate the implementation of the acting model described earlier. The particular type of acting model used here is an extension model. The specimen for which the data were obtained was a gypsum board wall with a wooden frame. The wall hypothesized for the analysis differed from the specimen only in width. The assumptions described in Appendix A, with the exception of linearity, are also made here.

The only difference between linear and nonlinear systems is with respect to restoring forces at the joints between the frame and the gypsum board. The nonlinearity considered in the analysis is assumed to be elasto-plastic as shown in the figure below.



The horizontal and vertical components of this restoring force are thus expressed as follows:

$$F_x = \frac{\theta_F y - \theta_p y}{\sqrt{(\theta_p x)^2 + \{(\theta_F - \theta_p) y\}^2}} f = \frac{(1-r)y}{\sqrt{r^2x^2 + (1-r)^2 y^2}} f$$

$$F_y = \frac{\theta_p x}{\sqrt{(\theta_p x)^2 + \{(\theta_F - \theta_p) y\}^2}} f = \frac{rx}{\sqrt{r^2x^2 + (1-r)^2 y^2}} f \tag{B.1}$$

where f equals $R\Delta$ for $\Delta < \Delta_y$, 1 for $\Delta_y < \Delta < \Delta_f$, and 0 for $\Delta_f < \Delta$; Δ is relative displacement as defined by the expression $\theta_F\{r^2x^2 + (1-r)^2y^2\}$; Δ_y is displacement at yielding; and Δ_f is displacement at failure. Compare this expression to equation (A.3) in Appendix A. The equilibrium of rotational moment about the shifted center O_s is:

$$
\begin{aligned}
M &= \sum (F_x \cdot y - F_y x) \\
&= \sum \{ \frac{(1-r)y^2}{r^2x^2 + (1-r)^2y^2} f - \frac{rx^2}{r^2x^2 + (1-r)^2y^2} f \} \\
&= \sum \{ \frac{(1-r)y^2 - rx^2}{r^2x^2 + (1-r)^2y^2} f \} = 0
\end{aligned}
\tag{B.2}
$$

where f is a function of r as expressed in equation (B.1). When a value for θ_F is specified, equation (B.2) should satisfy both r and f.

An iterative method should be used to solve equation (B.2). The value of r is assumed and that of f calculated; the value of M is then computed. If M is too large, the value of r is accordingly modified and the process repeated until an appropriately small value of M is attained.

After values of r and f have been obtained by the above-mentioned process, the horizontal racking force carried by the frame-panel wall is obtained as follows:

$$
P = \sum F_x = \sum \frac{(1-r)y}{\sqrt{r^2x^2 + (1-r)^2y^2}} f \tag{B.3}
$$

The summation should be carried out for the upper half of the wall.

The specimen described above is illustrated in Figure B.1; the force-displacement curves from horizontal racking force tests carried out on the specimen are shown in Figure B.2 [7]. Although the nonlinear characteristics of the joints in the specimen are not known, they can be estimated by the theory developed here. Results of this analysis are in fairly good agreement with experimental data (Figure B.2) when the following values, obtained by trial and error, are used as input: $\Delta y = 0.35$ cm; $\Delta f = 1.42$ cm; $fy = 15.2$ kg. The secondary wall assumed for the analysis is illustrated in Figure B.1(b). The wall was assumed to be constructed according to the specifications for the specimen, except that the width of the former was assumed to be twice that of the experimental wall. The nonlinear characteristics of the assumed wall were taken to be identical to those of the specimen.

The force-displacement relationship obtained for the assumed secondary wall is shown in Figure B.3. Force for this wall has been scaled by half so that the response of the two walls could be compared. The primary difference in response is in the accommodation of story drift, a difference attributable to the difference in width between the two elements. The basic damageability indices for these two secondary walls will differ accordingly.

APPENDIX C

Simulation of Behavior of a Block Masonry Wall

A concrete block masonry wall was modeled to demonstrate the use of the equivalent model, a model designed to simulate the behavior of a secondary wall for which experimental data are available [7]. The equivalent model is wholly based on experimental data. The specimen considered here is shown in Figure C.1.

The measured responses of the experimental element included elastoplasticity and slip. The equivalent model was therefore assumed to be comprised of the following elements (Figure C.2): a linear-elastic element, a rigid-plastic element, and a rigid-slip element. Although these elements could be combined in any number of ways, the equivalent model should be as simple as possible consistent with the need to model as many aspects of mechanical behavior as possible. In Figure C.3 a typical equivalent model in which the above elements have been combined is illustrated. The following phenomena can be simulated by this model: tri-linear force-displacement response, behavior for which rigid-plastic effects predominate for small story drift or *vice versa*, and degradation of stiffness and strength during the second cycle of loading before maximum displacement has occurred. The model cannot, however, simulate degradation during a third cycle of loading or degradation that results from response other than displacement. The trial-and-error method was again used; good agreement between the force-displacement diagrams for the experimental and equivalent models was achieved when the following parameters were used: $K_{E1} = 7000$ kg/cm; $K_{E2} = 625$ kg/cm; $K_{E3} = 500$ kg/cm; $F_s = 1759$ kg; and $F_p = 2000$ kg.

METHOD OF APPLICATION	SPECIFIC VALUES		JOINT LAYOUT TYPE A	B	C	D	E	F	G
SINGLE PANEL ($n = 1$)	ROTATION RATIO	r	1	0	$\frac{\alpha^2}{\alpha^2+1}$	$\frac{3\alpha^2}{3\alpha^2+1}$	$\frac{\alpha^2}{\alpha^2+3}$	$\frac{\alpha^2(\alpha+3)}{(\alpha+1)^3}$	$\frac{\alpha^2}{\alpha^2+1}$
	HORIZONTAL RELATIVE DISPLACEMENT	$(1-r)\theta H$	0	$1 \cdot \theta H$	$\frac{1}{\alpha^2+1}\theta H$	$\frac{1}{3\alpha^2+1}\theta H$	$\frac{3}{\alpha^2+3}\theta H$	$\frac{3\alpha+1}{(\alpha+1)^3}\theta H$	$\frac{1}{\alpha^2+1}\theta H$
	VERTICAL RELATIVE DISPLACEMENT	$\frac{r}{2}\theta H$	$\frac{1}{2}\theta H$	0	$\frac{\alpha}{\alpha^2+1}\theta H$	$\frac{3\alpha}{3\alpha^2+1}\theta H$	$\frac{\alpha}{\alpha^2+3}\theta H$	$\frac{\alpha(\alpha+3)}{(\alpha+1)^3}\theta H$	$\frac{\alpha}{\alpha^2+1}\theta H$
	STIFFNESS	k	0	0	$\frac{1}{\alpha^2+1}R$	$\frac{1}{\alpha(3\alpha^2+1)}\frac{RH}{2}$	$\frac{\alpha}{\alpha^2+3}\frac{RH}{2}$	$\frac{(\alpha+3)(3\alpha+1)}{\alpha(\alpha+1)^3}\frac{RH}{6}$	$\frac{1}{\alpha(\alpha^2+1)}\frac{RH^2}{36}$
VERTICAL APPLICATION ($n \geqslant 2$) $V2$ $V3$	ROTATION RATIO	r	1	0	$\frac{n^2\alpha^2}{n^2\alpha^2+1}$	$\frac{3n^2\alpha^2}{3n^2\alpha^2+1}$	$\frac{n^2\alpha^2}{n^2\alpha^2+3}$	$\frac{n^2\alpha^2(n\alpha+3)}{(n\alpha+1)^3}$	$\frac{n^2\alpha^2}{n^2\alpha^2+1}$
	HORIZONTAL RELATIVE DISPLACEMENT	$(1-r)\theta H$	0	$1 \cdot \theta H$	$\frac{1}{n^2\alpha^2+1}\theta H$	$\frac{1}{3n^2\alpha^2+1}\theta H$	$\frac{3}{n^2\alpha^2+3}\theta H$	$\frac{3n\alpha+1}{(n\alpha+1)^3}\theta H$	$\frac{1}{n^2\alpha^2+1}\theta H$
	VERTICAL RELATIVE DISPLACEMENT	$\frac{r}{n\alpha}\theta H$	$\frac{1}{n\alpha}\theta H$	0	$\frac{n\alpha}{n^2\alpha^2+1}\theta H$	$\frac{3n\alpha}{3n^2\alpha^2+1}\theta H$	$\frac{n\alpha}{n^2\alpha^2+3}\theta H$	$\frac{n\alpha(n\alpha+3)}{(n\alpha+1)^3}\theta H$	$\frac{n\alpha}{n^2\alpha^2+1}\theta H$
	STIFFNESS	k	0	0	$\frac{n}{n^2\alpha^2+1}R$	$\frac{1}{\alpha(3n^2\alpha^2+1)}\frac{RH}{2}$	$\frac{n}{n^2\alpha^2+3}\frac{RH}{2}$	$\frac{(n\alpha+3)(3n\alpha+1)}{\alpha(n\alpha+1)^3}\frac{RH}{6}$	$\frac{1}{\alpha(n^2\alpha^2+1)}\frac{RH^2}{36}$
HORIZONTAL APPLICATION ($n \geqslant 2$) $H2$ $H3$	ROTATION RATIO	r	1	0	$\frac{\alpha^2}{n^2+\alpha^2}$	$\frac{3\alpha^2}{n^2+3\alpha^2}$	$\frac{\alpha^2}{3n^2+\alpha^2}$	$\frac{\alpha^2(3n+\alpha)}{(n+\alpha)^3}$	$\frac{\alpha^2}{n^2\alpha^2}$
	HORIZONTAL RELATIVE DISPLACEMENT	$\frac{1-r}{n}\theta H$	0	$\frac{1}{n}\cdot\theta H$	$\frac{n}{n^2+\alpha^2}\theta H$	$\frac{n}{n^2+3\alpha^2}\theta H$	$\frac{3n}{3n^2+\alpha^2}\theta H$	$\frac{n(n+3\alpha)}{(n+\alpha)^3}\theta H$	$\frac{n}{n^2\alpha^2}\theta H$
	VERTICAL RELATIVE DISPLACEMENT	$\frac{r}{\alpha}\theta H$	$\frac{1}{\alpha}\theta H$	0	$\frac{\alpha}{n^2+\alpha^2}\theta H$	$\frac{3\alpha}{n^2+3\alpha^2}\theta H$	$\frac{\alpha}{3n^2+\alpha^2}\theta H$	$\frac{\alpha(3n+\alpha)}{(n+\alpha)^3}\theta H$	$\frac{\alpha}{n^2+\alpha^2}\theta H$
	STIFFNESS	k	0	0	$\frac{n}{n^2+\alpha^2}R$	$\frac{n}{\alpha(n^2+3\alpha^2)}\frac{RH}{2}$	$\frac{1}{3n^2+\alpha^2}\frac{RH}{2}$	$\frac{(3n+\alpha)(n+3\alpha)}{\alpha(n+\alpha)^3}\frac{RH}{6}$	$\frac{1}{\alpha(n^2+\alpha^2)}\frac{RH^2}{36}$

TABLE A.1 SUMMARY OF RESULTS

- NONSTRUCTURAL ELEMENTS
  - ARCHITECTURAL ELEMENTS
    - WALL
      - SECONDARY WALL
      - OTHERS (OPENINGS)
    - CEILING
      - SUSPENDED CEILING
      - OTHERS (LIGHTING FIXTURES)
    - APPENDAGE
      - CHIMNEY
      - ADVERTISING PANEL
      - ORNAMENTATION
  - MECHANICAL/ELECTRICAL ELEMENTS
    - SERVICE FACILITIES (GAS, WATER, AIR)
      - MACHINERY
      - PIPING
      - TERMINAL UNIT
    - TRANSFER FACILITIES
      - ELEVATOR
      - ESCALATOR
    - ELECTRICAL FACILITIES
      - GENERATOR
      - COMPUTER
      - TELEVISION
      - TELEPHONE
  - CONTENTS
    - FURNITURE
    - HOUSEWARES
    - STORAGE
    - FLAMMABLE
    - TOXIC
    - FILES
    - BOOKS
    - SUPPLIES
    - CLOTHING
      - RESIDENTIAL
      - COMMERCIAL
      - INSTITUTIONAL

FIGURE 2.1 CLASSIFICATION OF NONSTRUCTURAL ELEMENTS

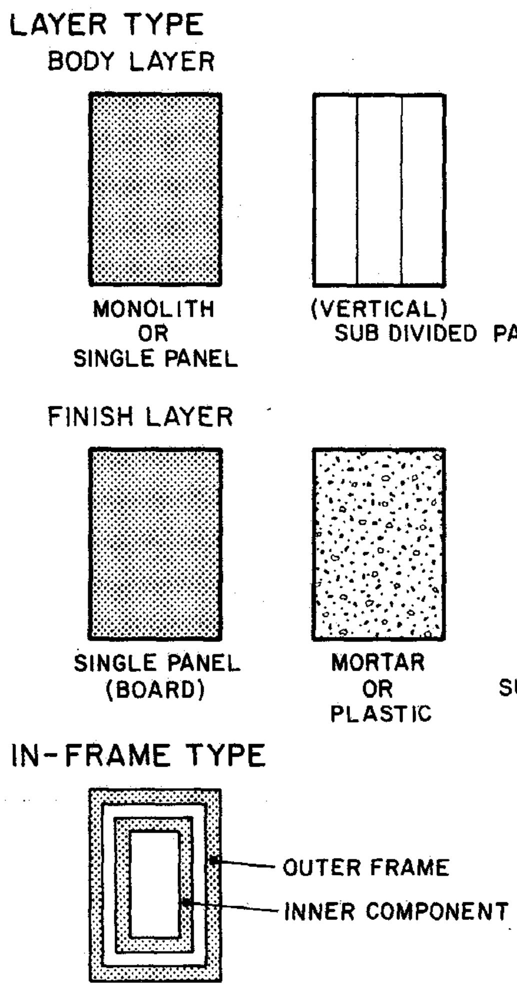
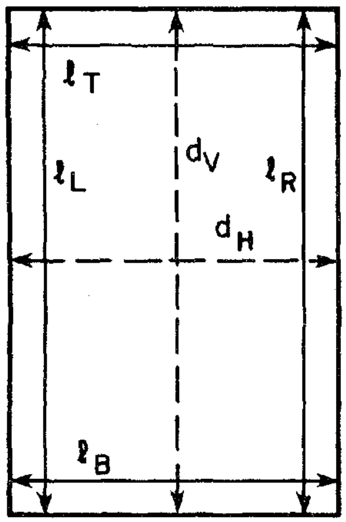


FIGURE 3.1 CLASSIFICATION OF ELEMENT MODELS



⟷ LENGTH OF EACH SIDE IS KEPT CONSTANT

⇠ ⇢ DISTANCE OF TWO OPPOSITE SIDES IS KEPT CONSTANT: THE TWO SIDES ARE KEPT STRAIGHT.

FIGURE 3.2 TYPES OF CONFINEMENT

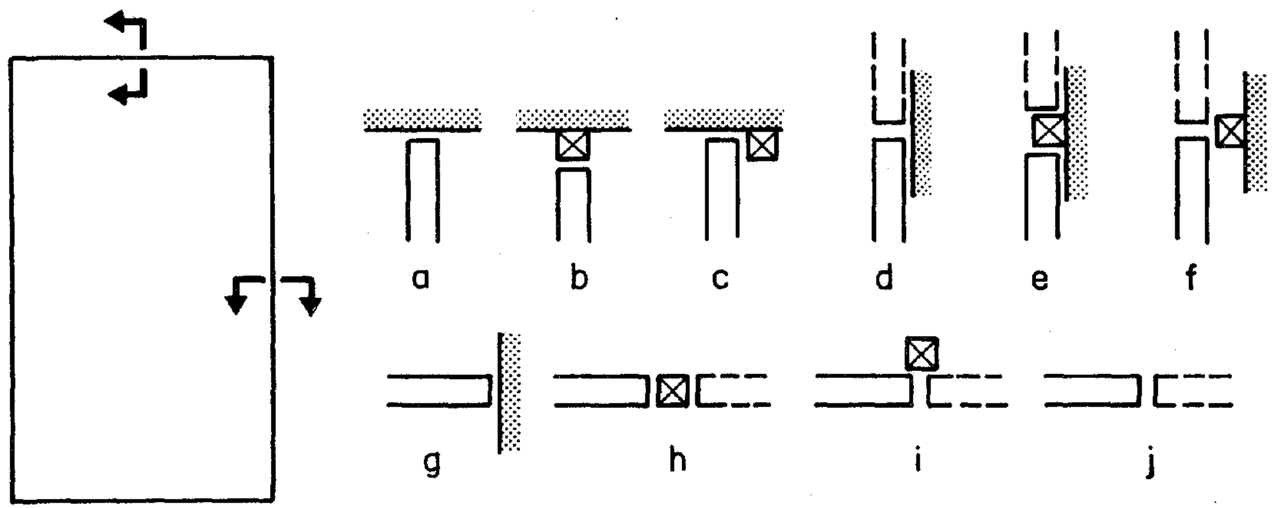


FIGURE 3.3 GEOMETRIC RELATIONSHIP AT INTERFACE

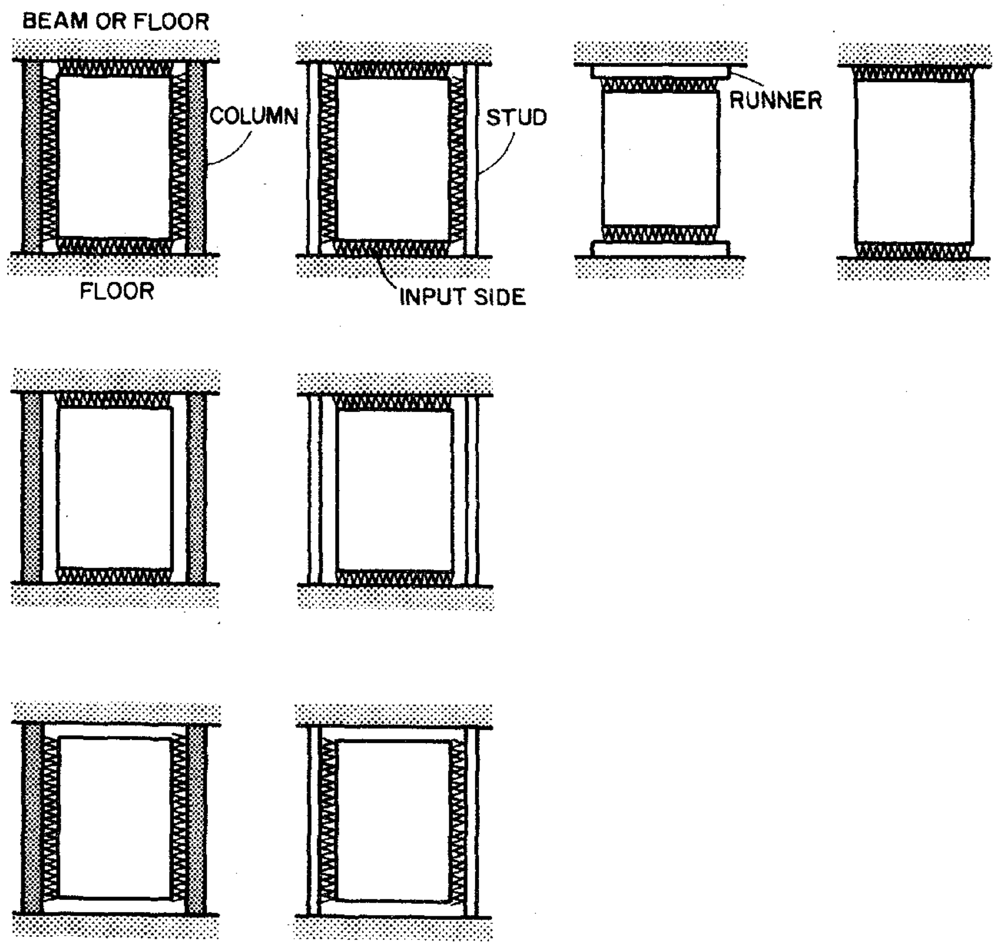


FIGURE 3.4 EXAMPLES OF ASSEMBLY MODELS

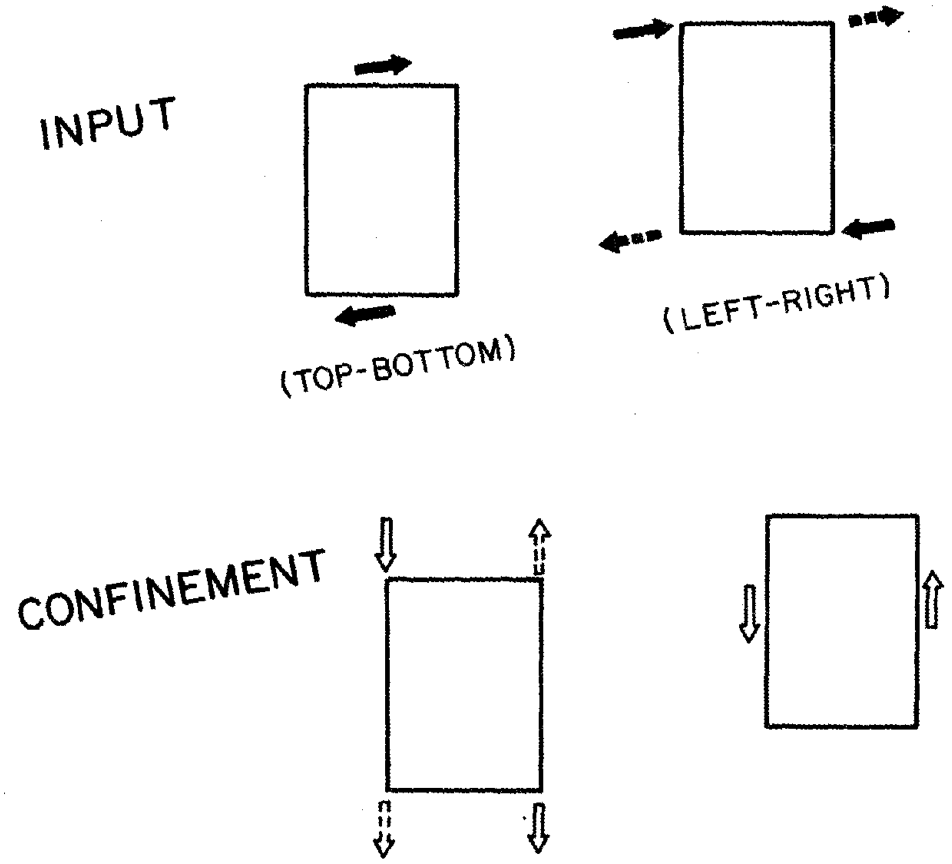


FIGURE 3.5 ARRANGEMENT OF INPUT AND CONFINEMENT FORCES

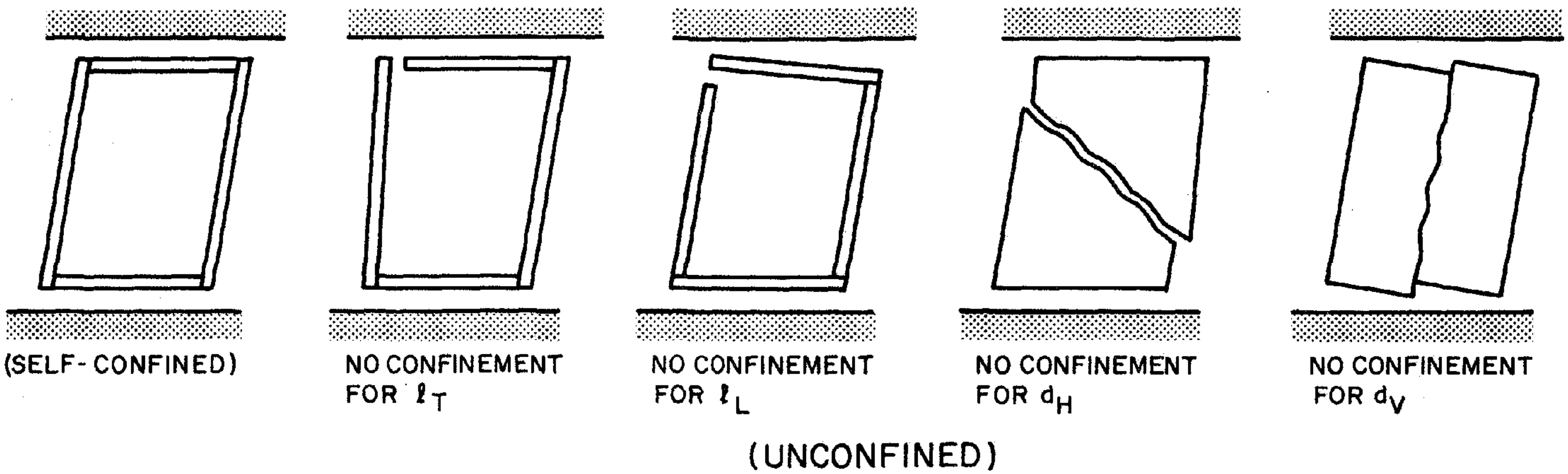


FIGURE 3.6 DEFORMATION AND BREAKAGE PATTERNS OF SELF-CONTAINED AND UNCONFINED TYPES

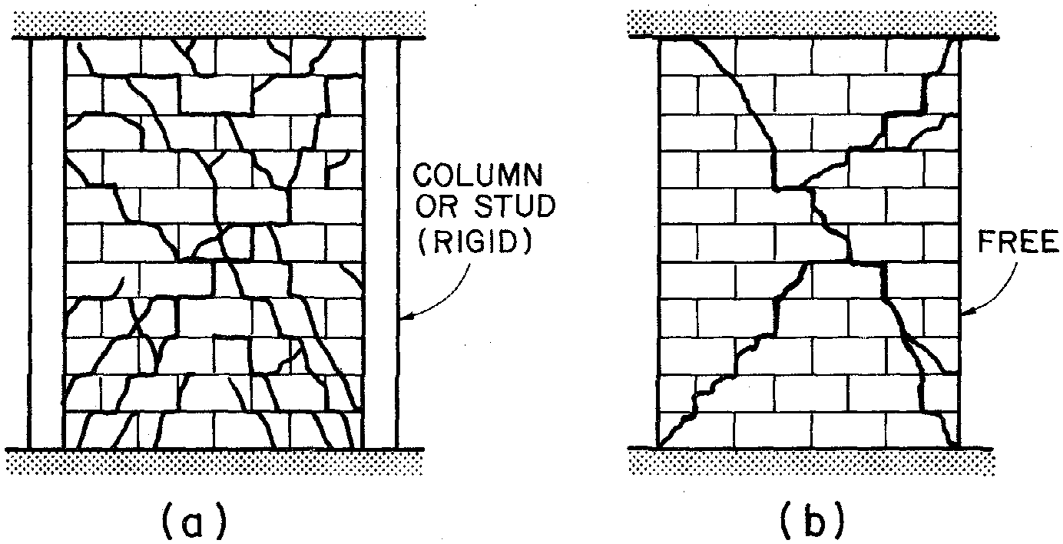


FIGURE 3.7 COMPARISON OF BREAKAGE PATTERNS

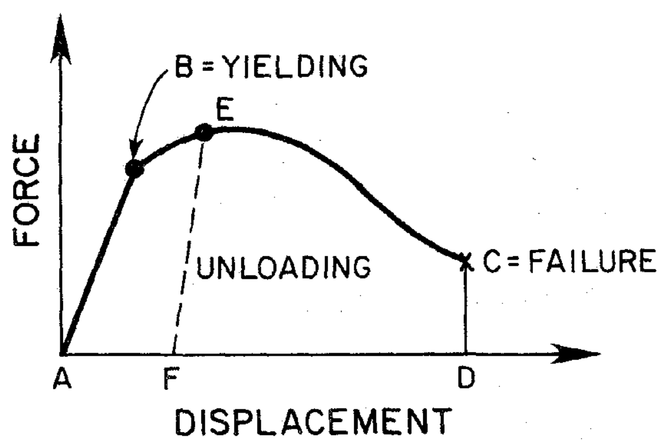


FIGURE 3.8 BASIC DAMAGEABILITY INDEX

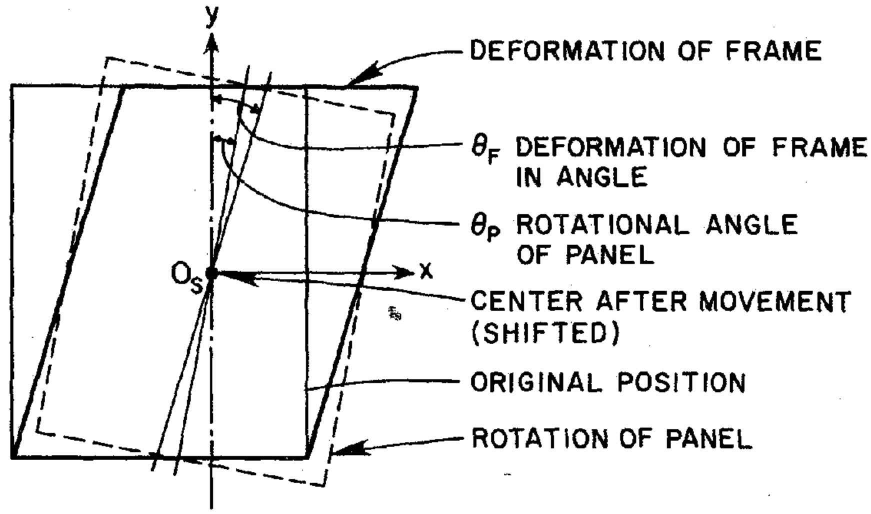


FIGURE A.1 OVERALL BEHAVIOR OF FRAME-PANEL WALL

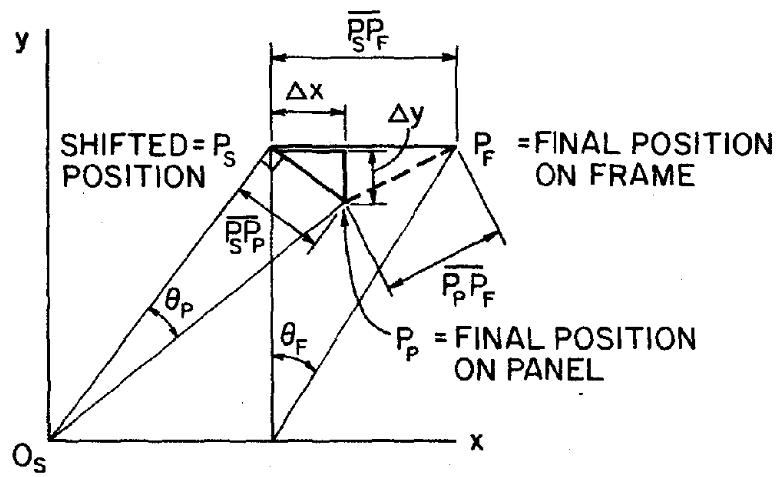


FIGURE A.2 MOVEMENT OF POINTS ON FRAME AND PANEL

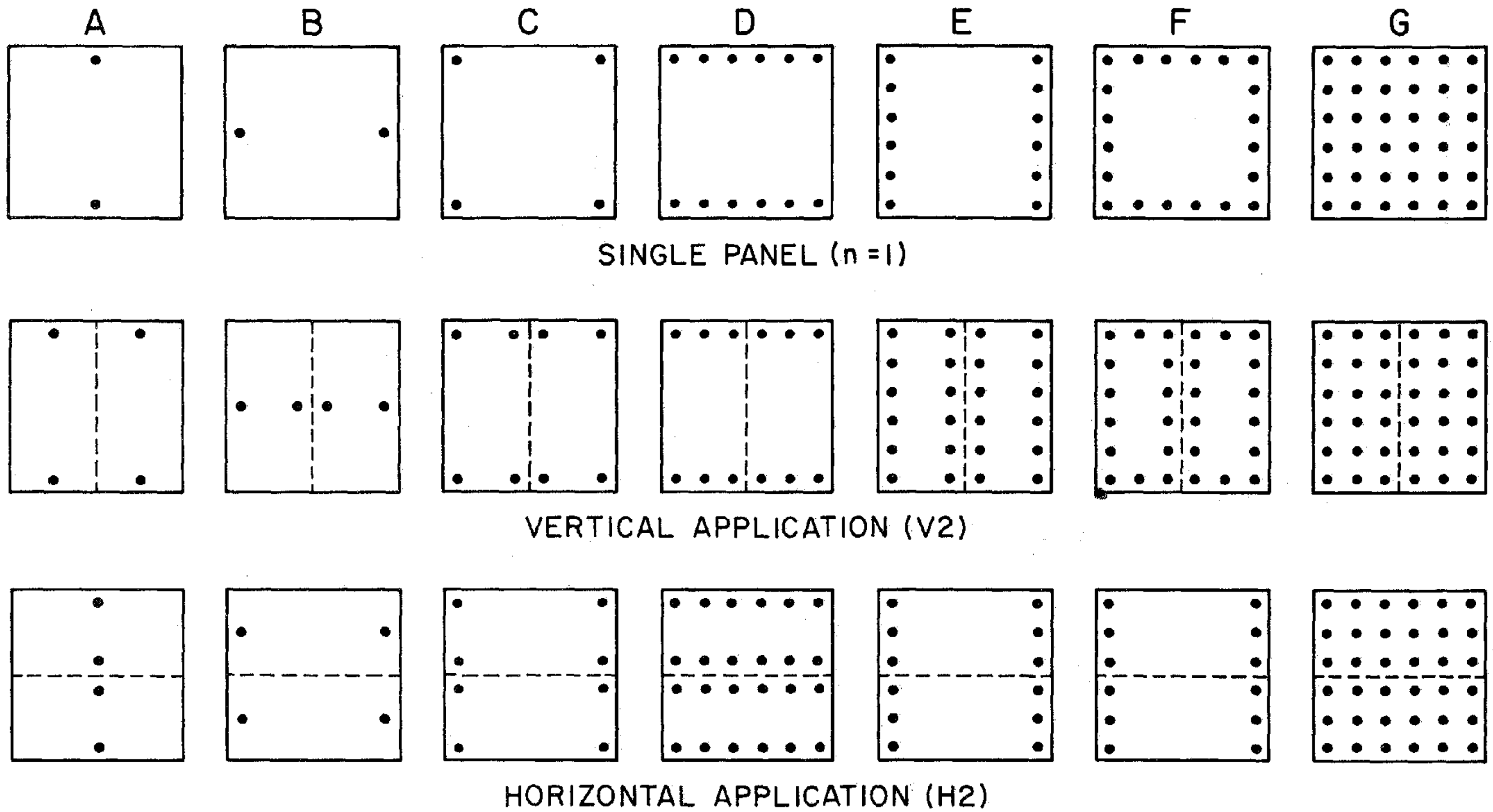


FIGURE A.3 ARRANGEMENT OF PANELS

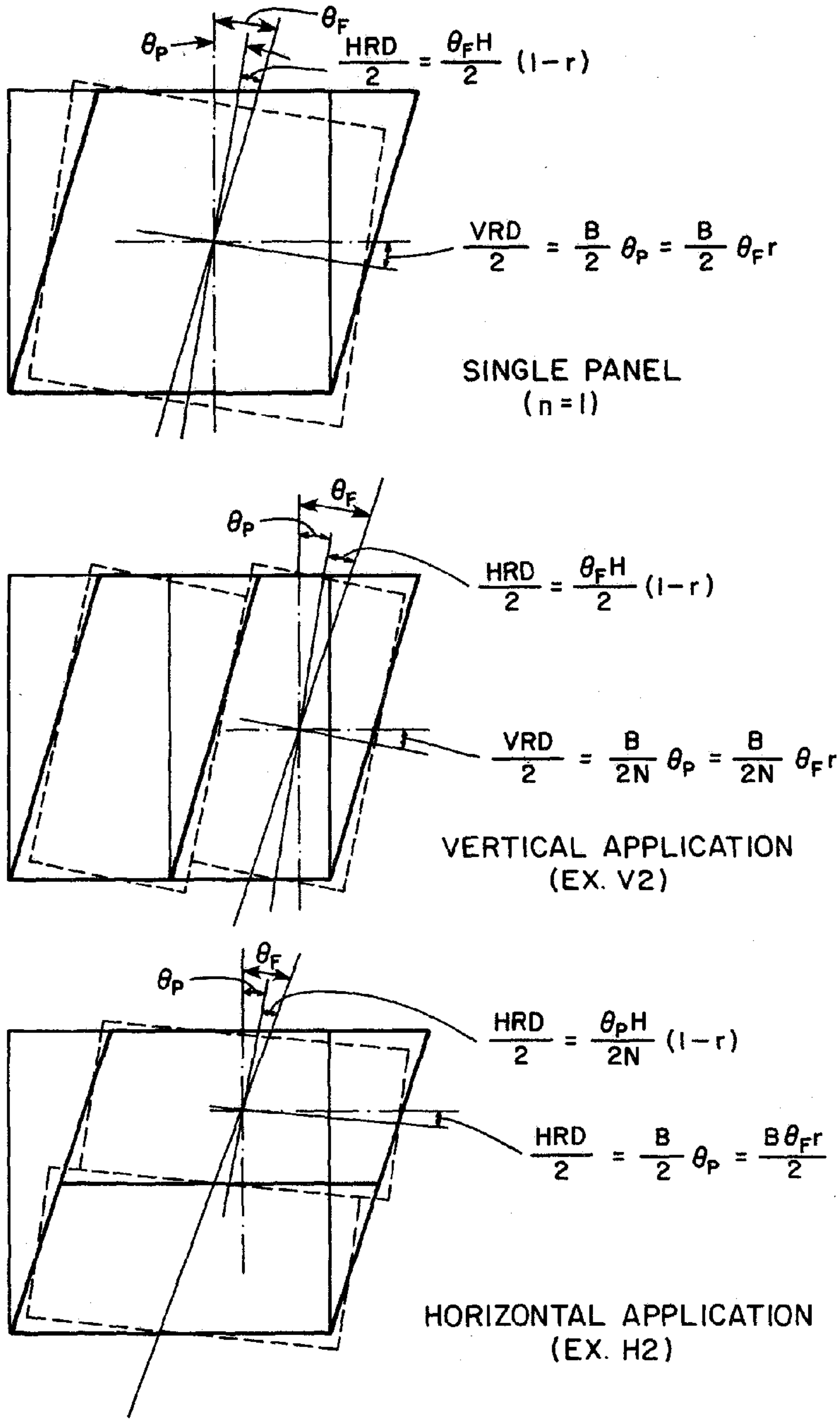


FIGURE A.4 RELATIVE HORIZONTAL AND VERTICAL DISPLACEMENTS

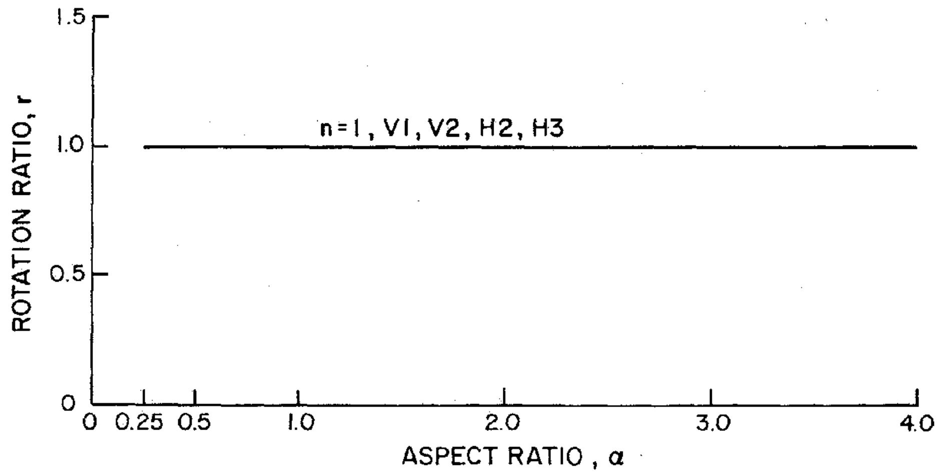


FIGURE A.5 (a1) ROTATION RATIO (TYPE A)

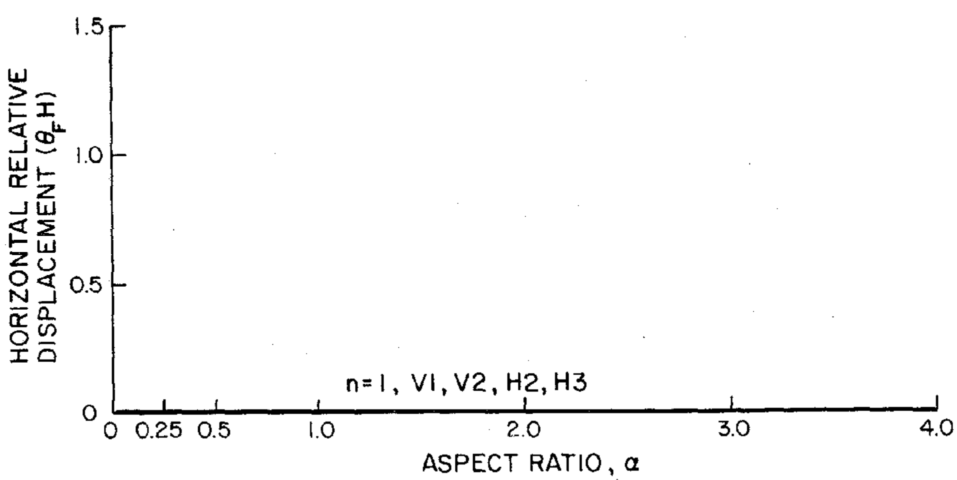


FIGURE A.5 (a2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE A)

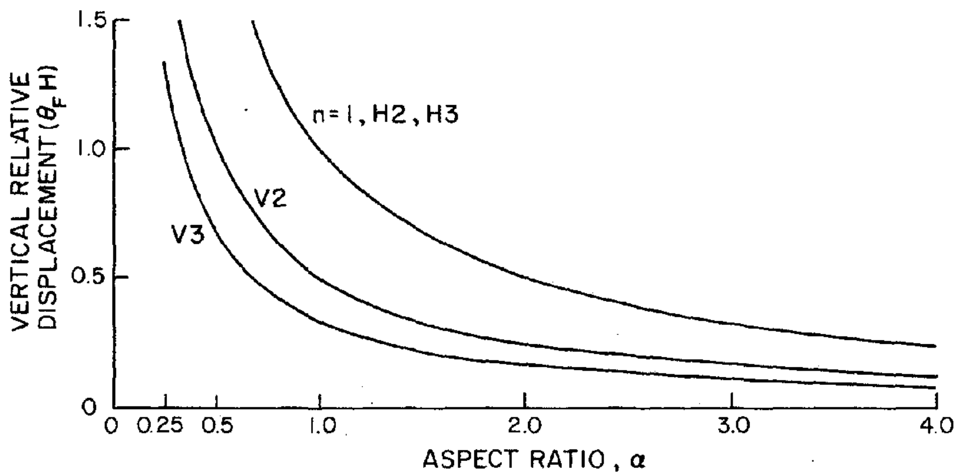


FIGURE A.5 (a3) RELATIVE VERTICAL DISPLACEMENT (TYPE A)

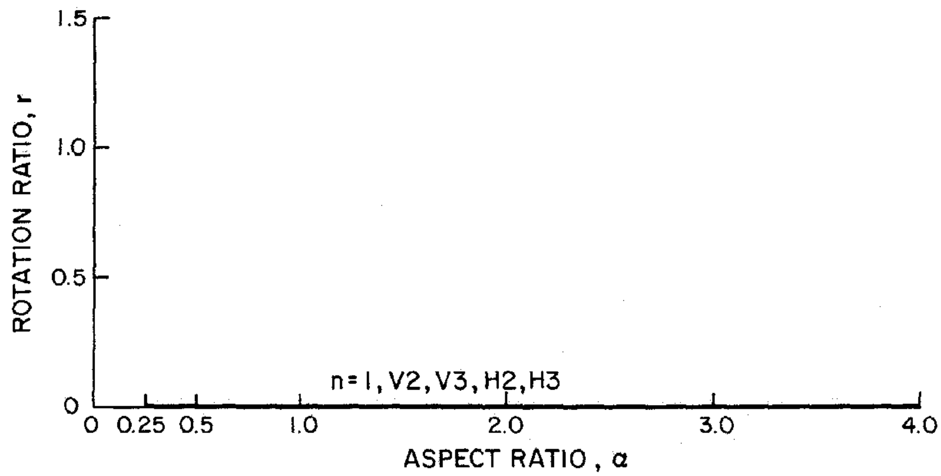


FIGURE A.5 (b1) ROTATION RATIO (TYPE B)

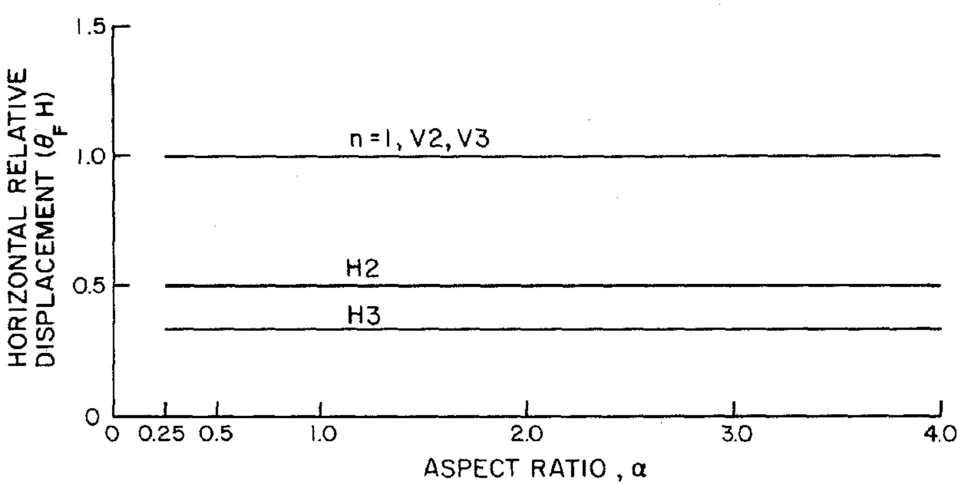


FIGURE A.5 (b2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE B)

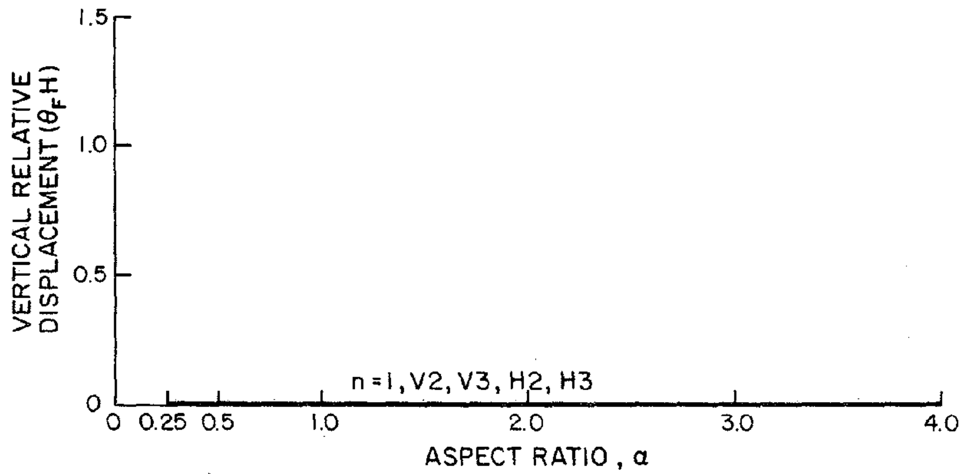


FIGURE A.5 (b3) RELATIVE VERTICAL DISPLACEMENT (TYPE B)

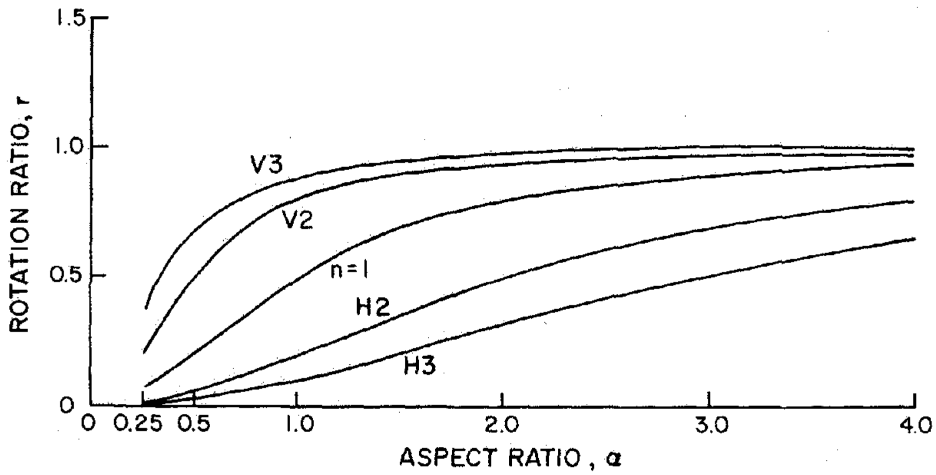


FIGURE A.5 (c1) ROTATION RATIO (TYPE C)

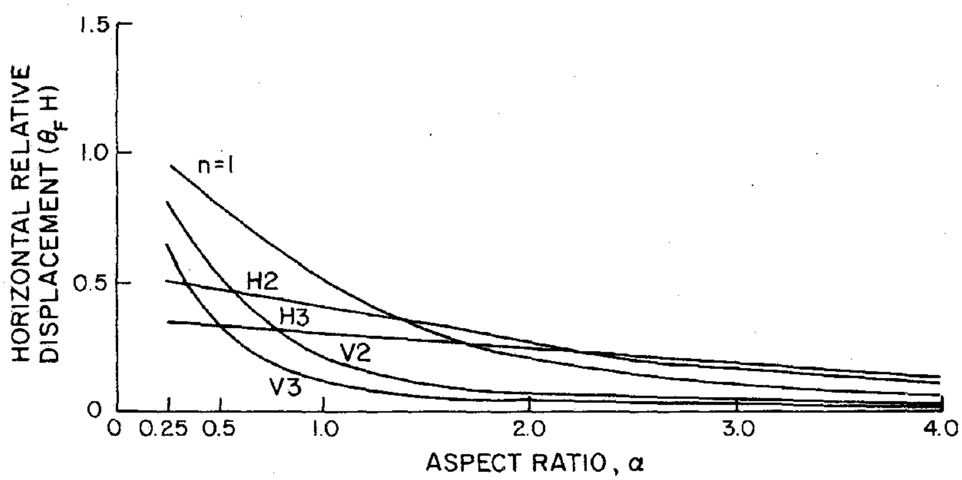


FIGURE A.5 (c2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE C)

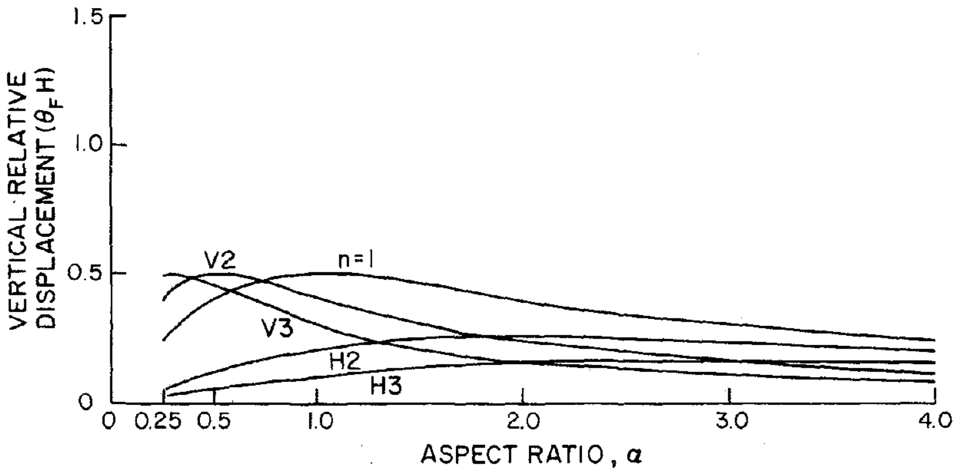


FIGURE A.5 (c3) RELATIVE VERTICAL DISPLACEMENT (TYPE C)

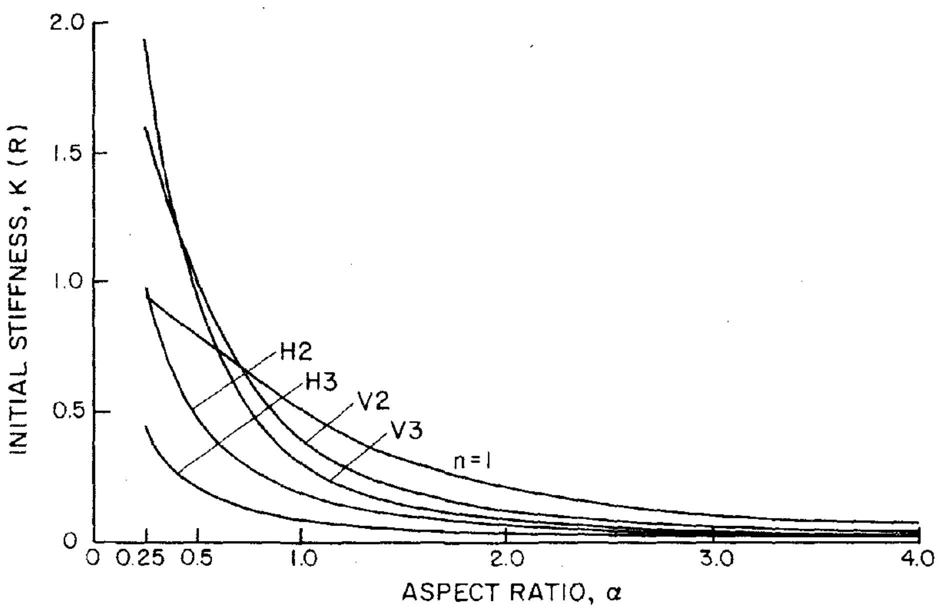


FIGURE A.5 (c4) INITIAL STIFFNESS (TYPE C)

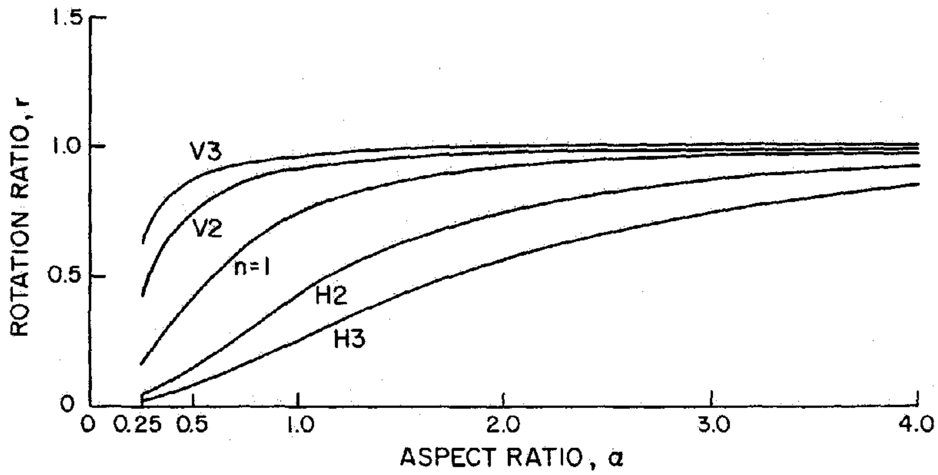


FIGURE A.5 (d1) ROTATION RATIO (TYPE D)

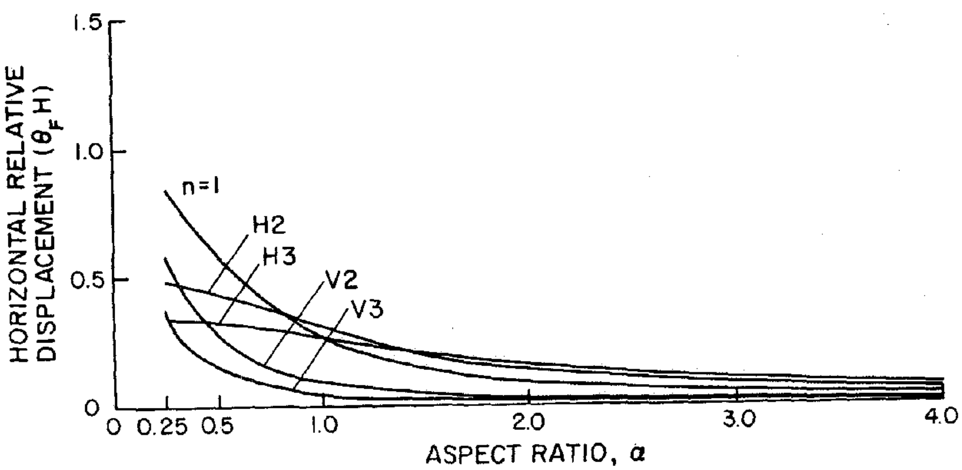


FIGURE A.5 (d2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE D)

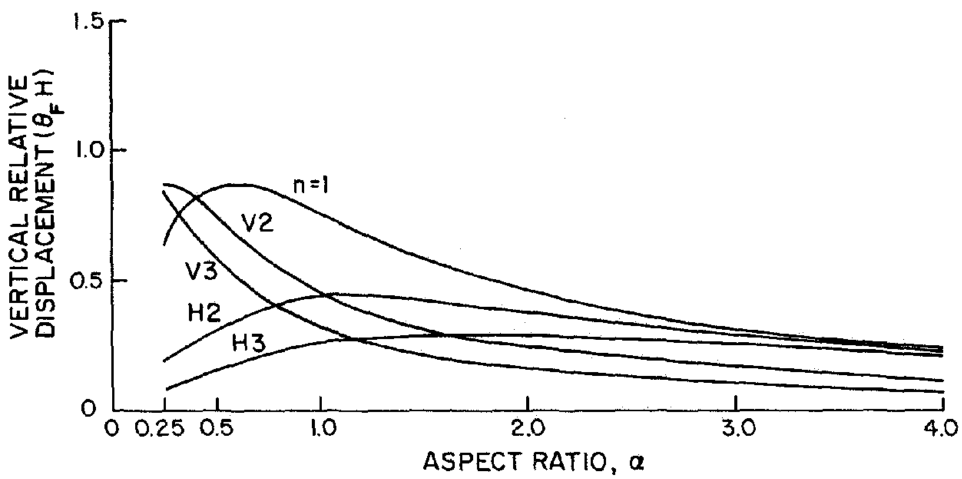


FIGURE A.5 (d3) RELATIVE VERTICAL DISPLACEMENT (TYPE D)

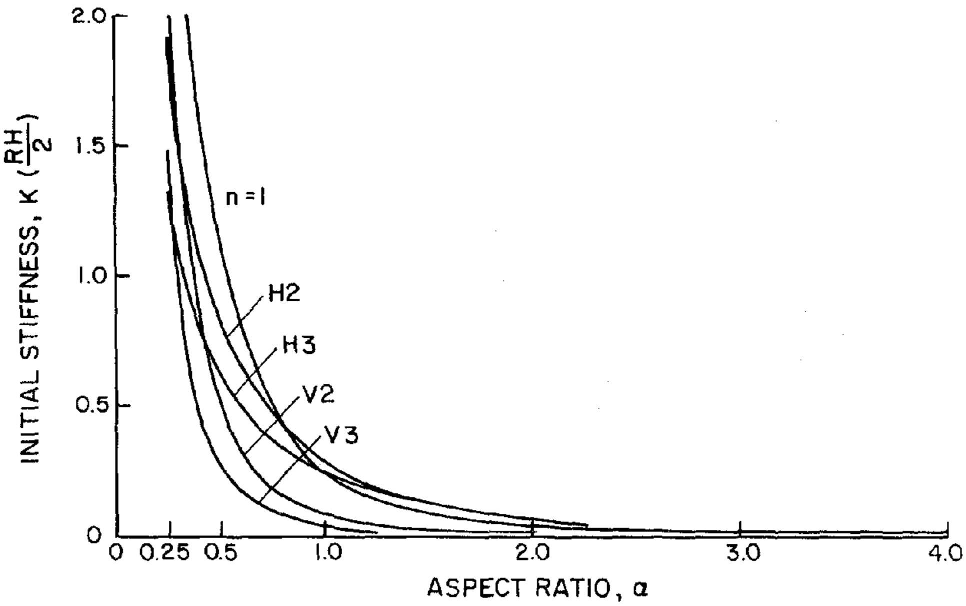


FIGURE A.5 (d4) INITIAL STIFFNESS (TYPE D)

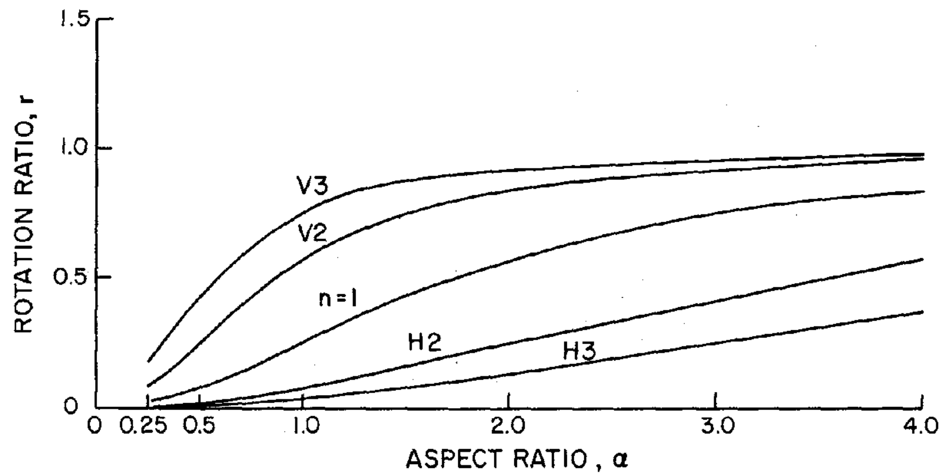


FIGURE A.5 (e1) ROTATION RATIO (TYPE E)

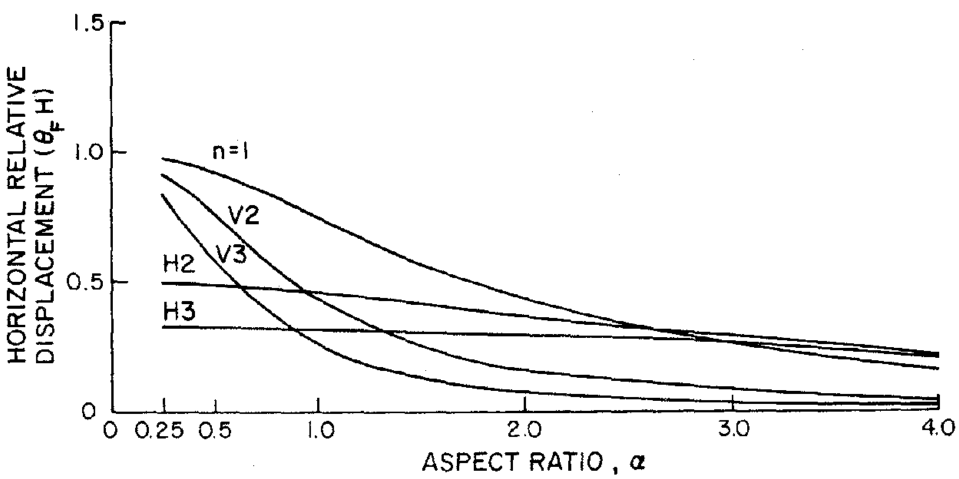


FIGURE A.5 (e2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE E)

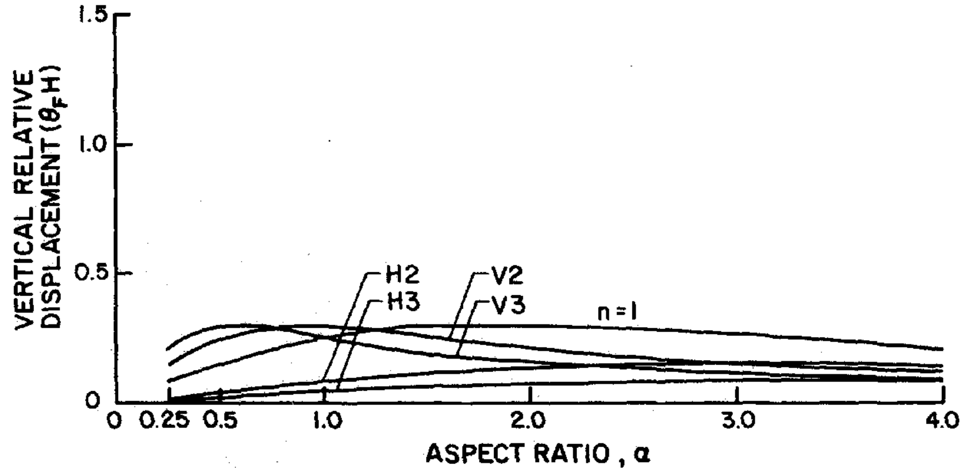


FIGURE A.5 (e3) RELATIVE VERTICAL DISPLACEMENT (TYPE E)

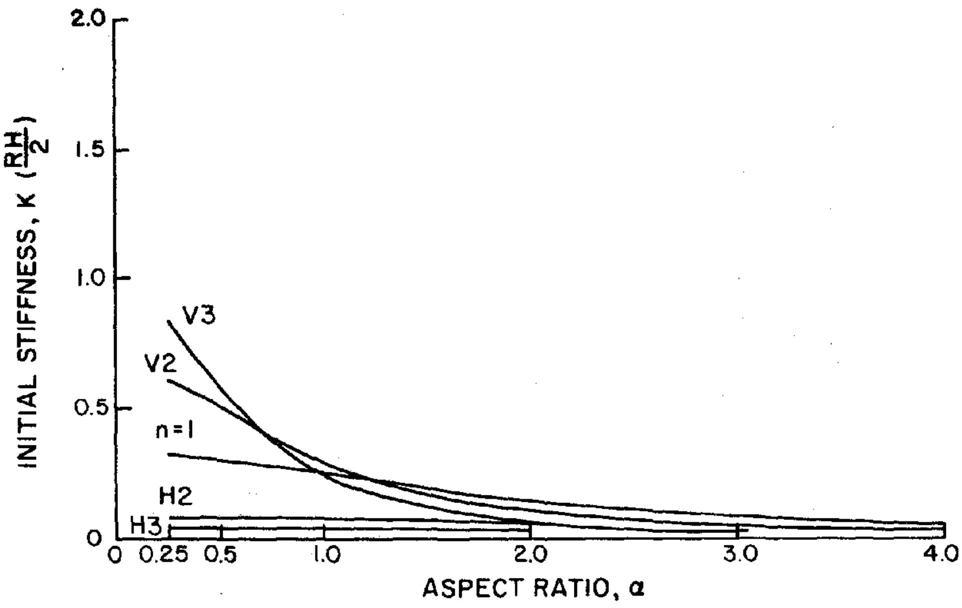


FIGURE A.5 (e4) INITIAL STIFFNESS (TYPE E)

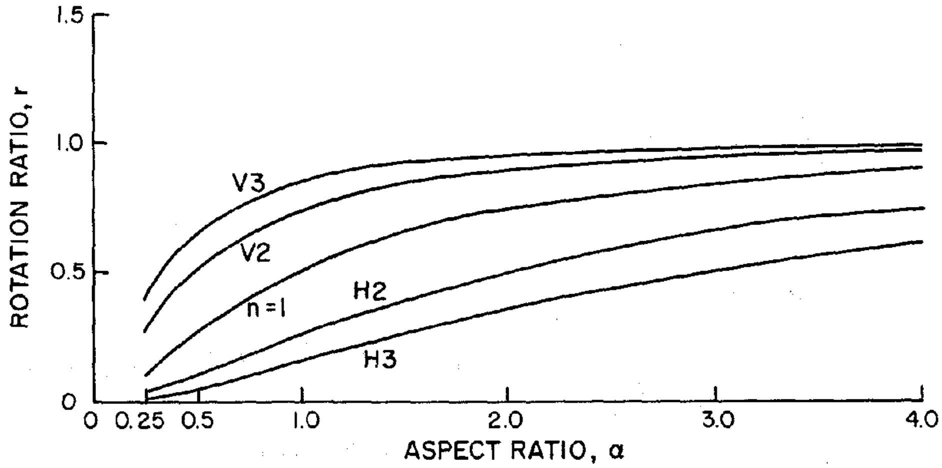


FIGURE A.5 (f1) ROTATION RATIO (TYPE F)

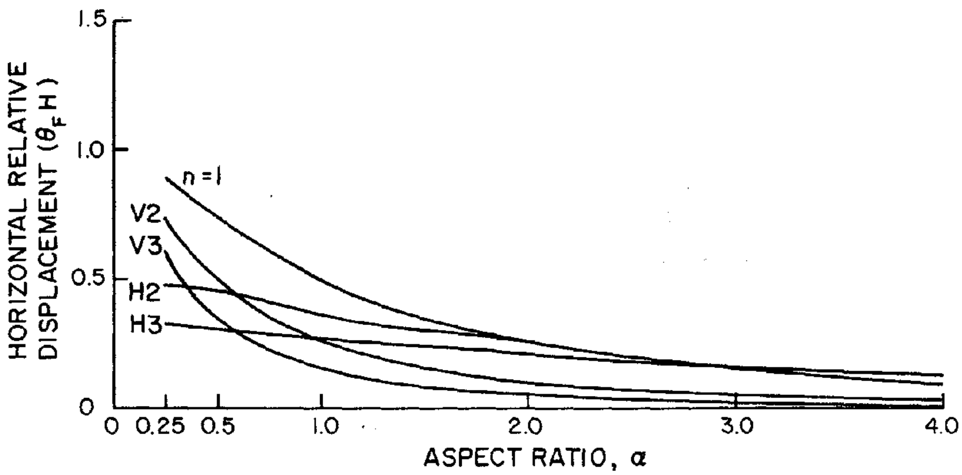


FIGURE A.5 (f2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE F)

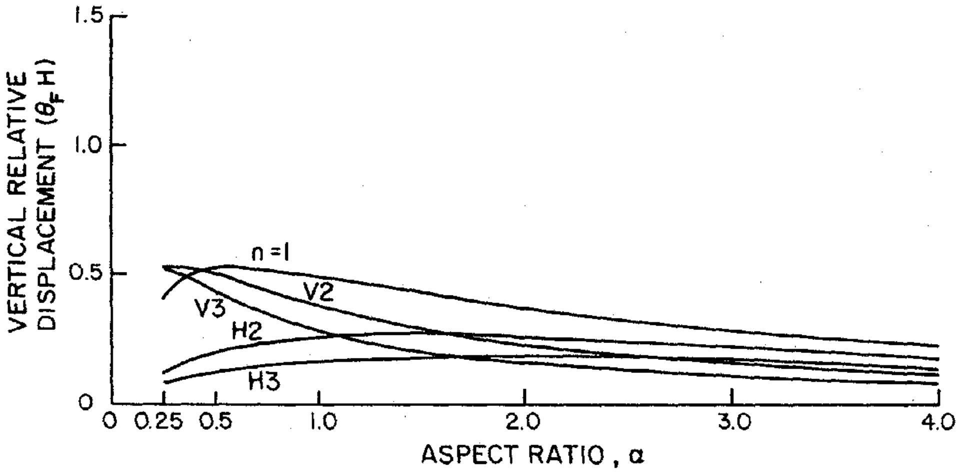


FIGURE A.5 (f3) RELATIVE VERTICAL DISPLACEMENT (TYPE F)

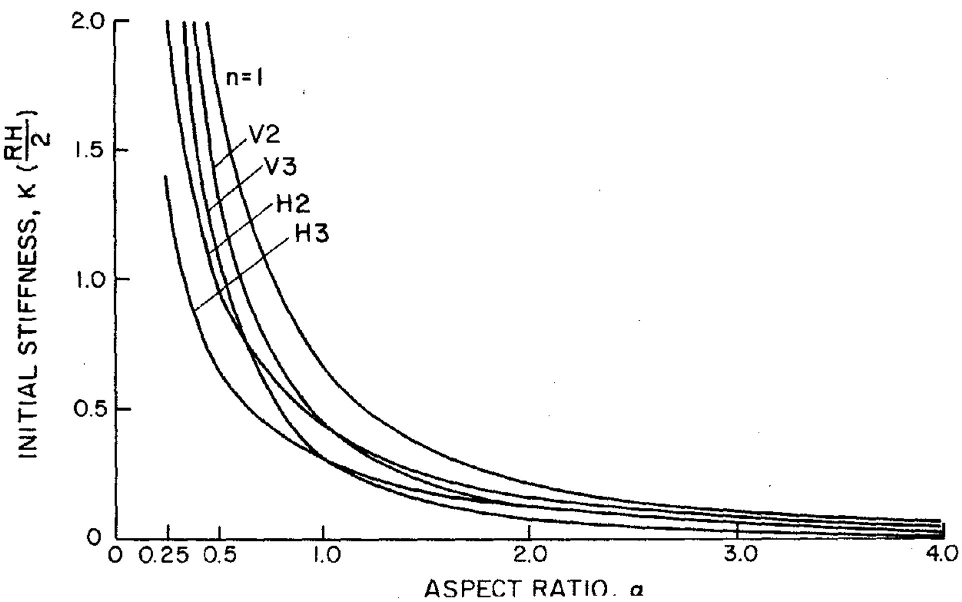


FIGURE A.5 (f4) INITIAL STIFFNESS (TYPE F)

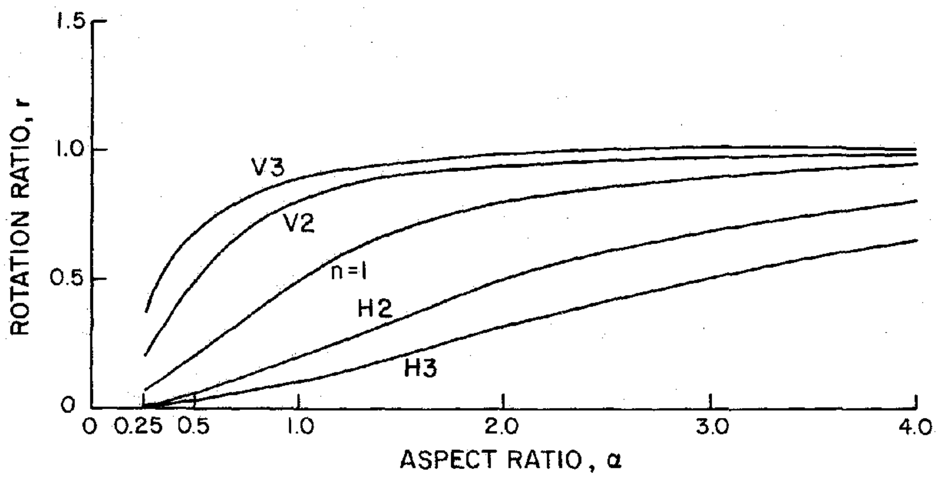


FIGURE A.5 (g1) ROTATION RATIO (TYPE G)

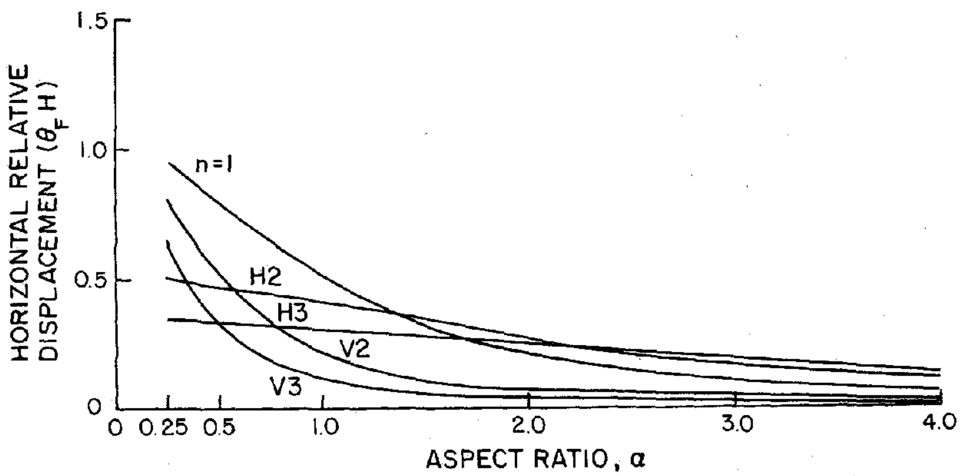


FIGURE A.5 (g2) RELATIVE HORIZONTAL DISPLACEMENT (TYPE G)

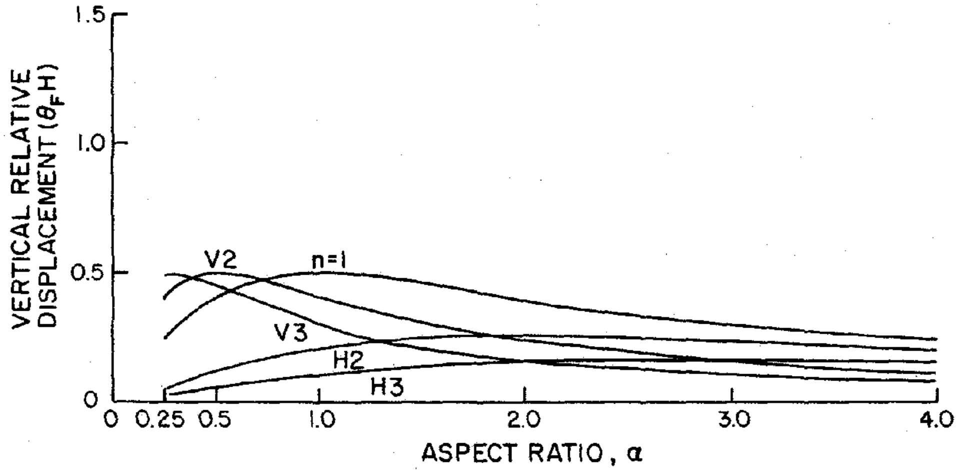


FIGURE A.5 (g3) RELATIVE VERTICAL DISPLACEMENT (TYPE G)

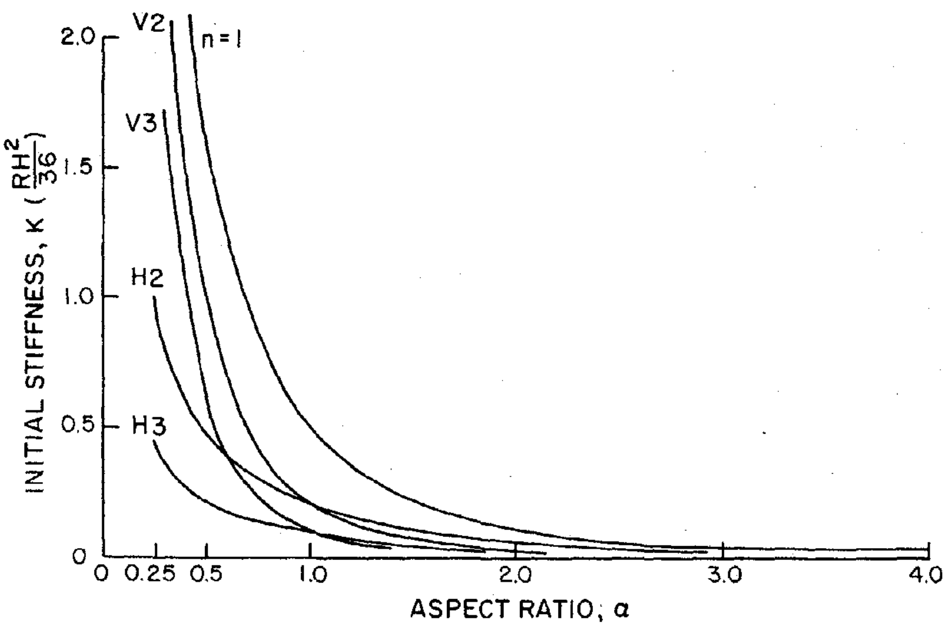


FIGURE A.5 (g4) INITIAL STIFFNESS (TYPE G)

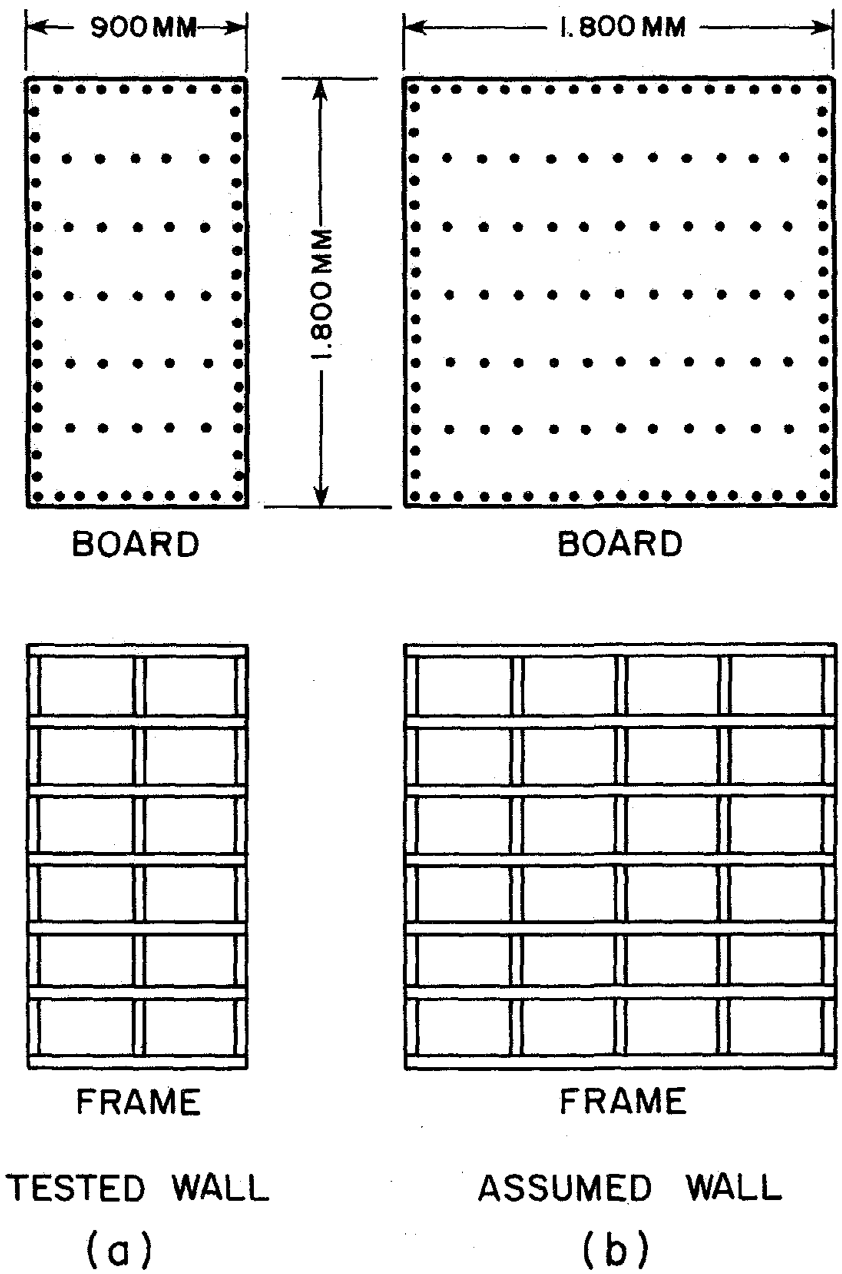


FIGURE B.1 GYPSUM BOARD WALL WITH WOOD FRAME

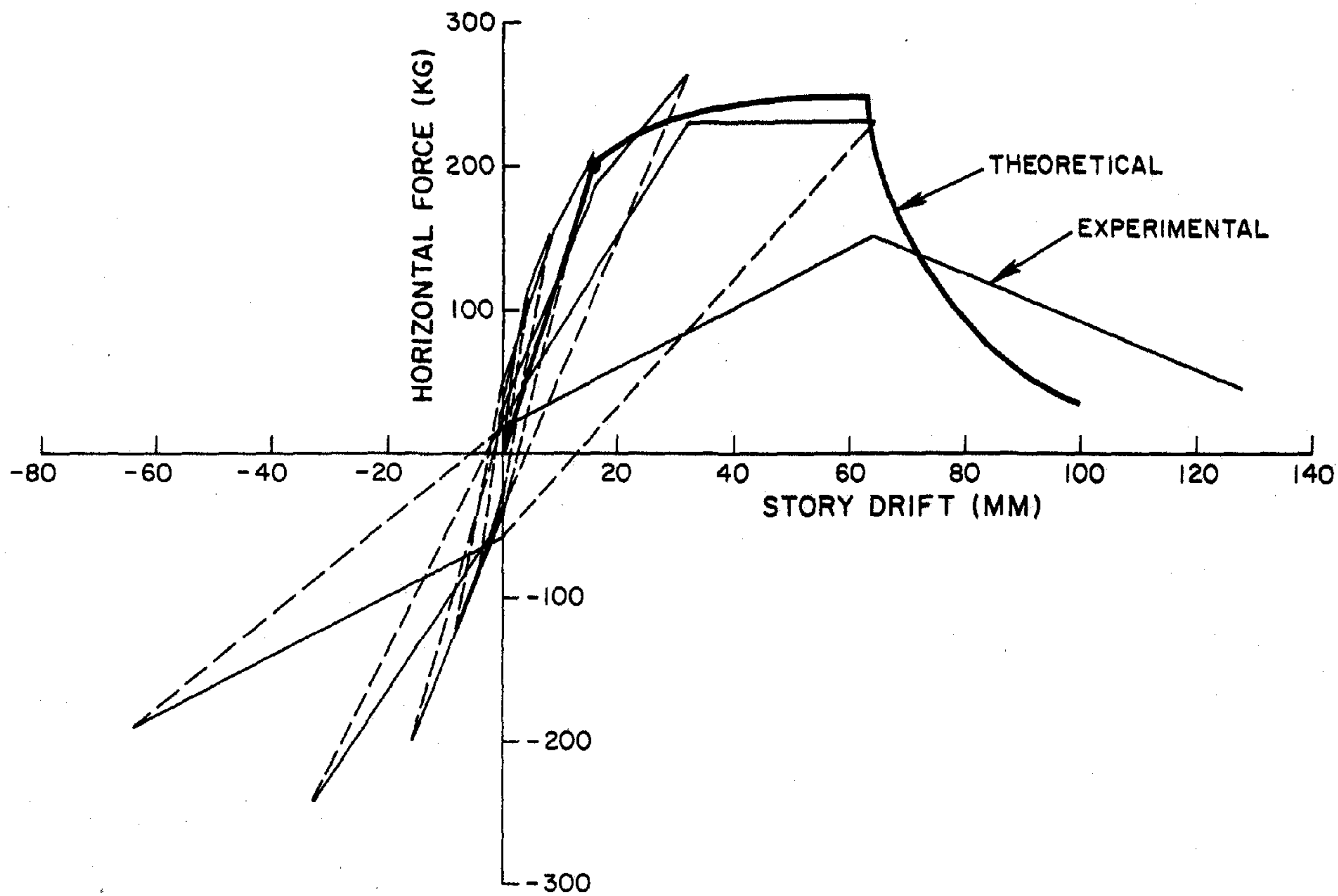


FIGURE B.2 COMPARISON OF THEORETICAL BEHAVIOR TO EXPERIMENTAL DATA

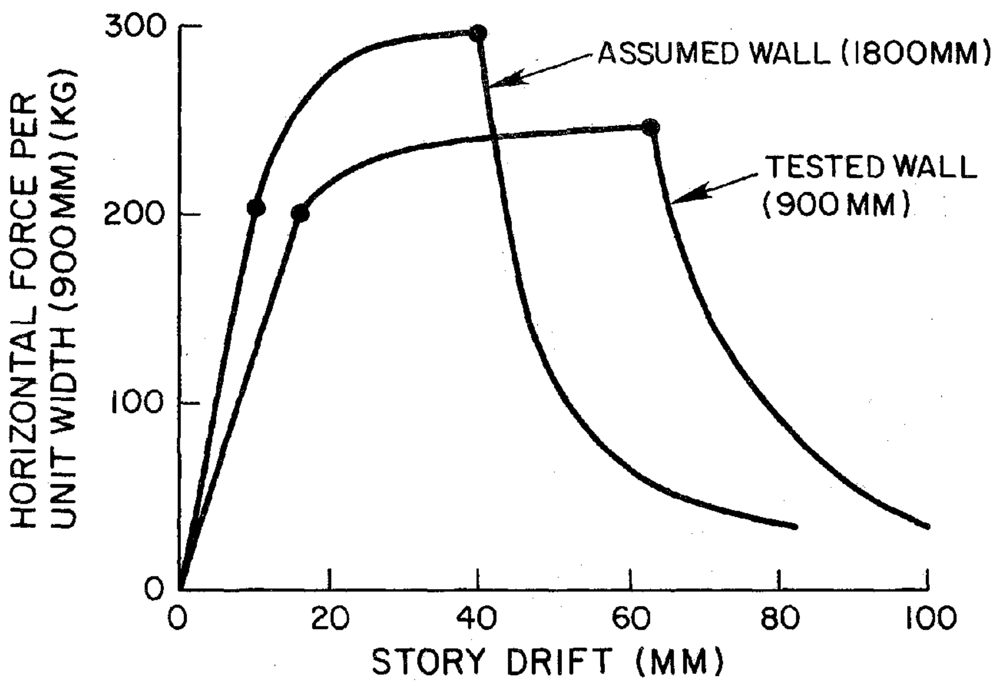


FIGURE B.3 COMPARISON OF THEORETICAL BEHAVIOR FOR DIFFERENT WALL WIDTHS

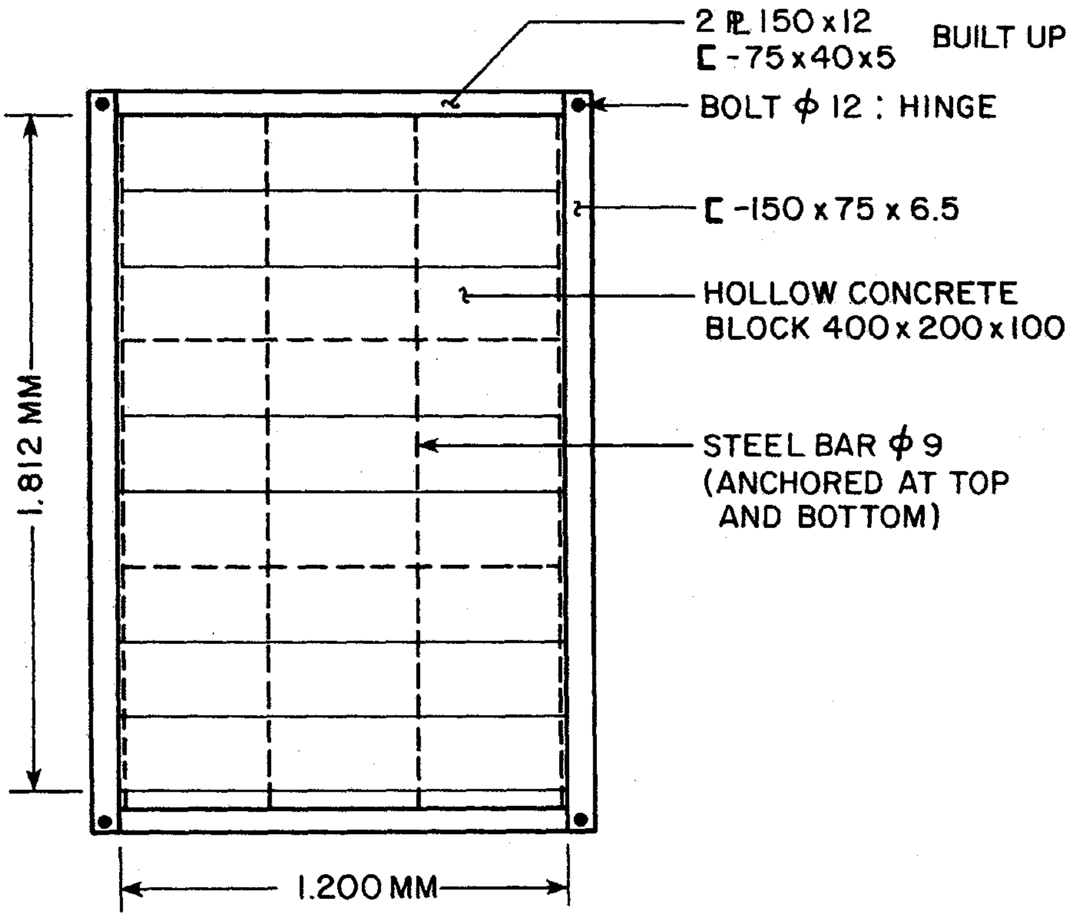


FIGURE C.1 EXPERIMENTAL CONCRETE BLOCK MASONRY WALL

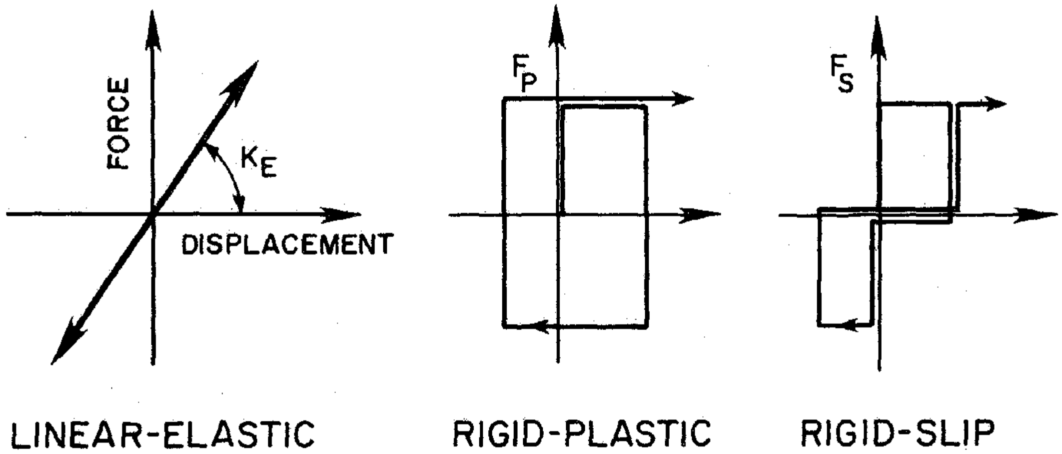


FIGURE C.2 CHARACTERISTICS OF ELEMENT

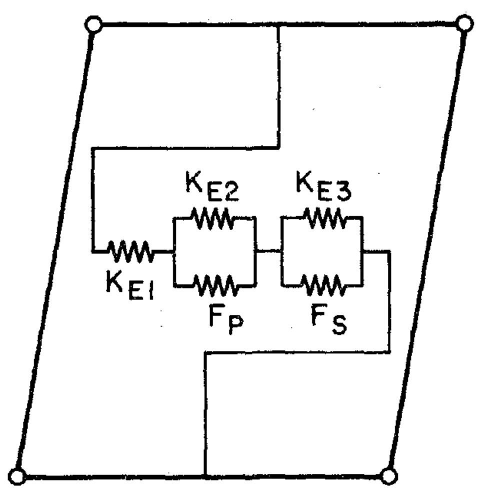


FIGURE C.3 COMBINATION OF ELEMENTS

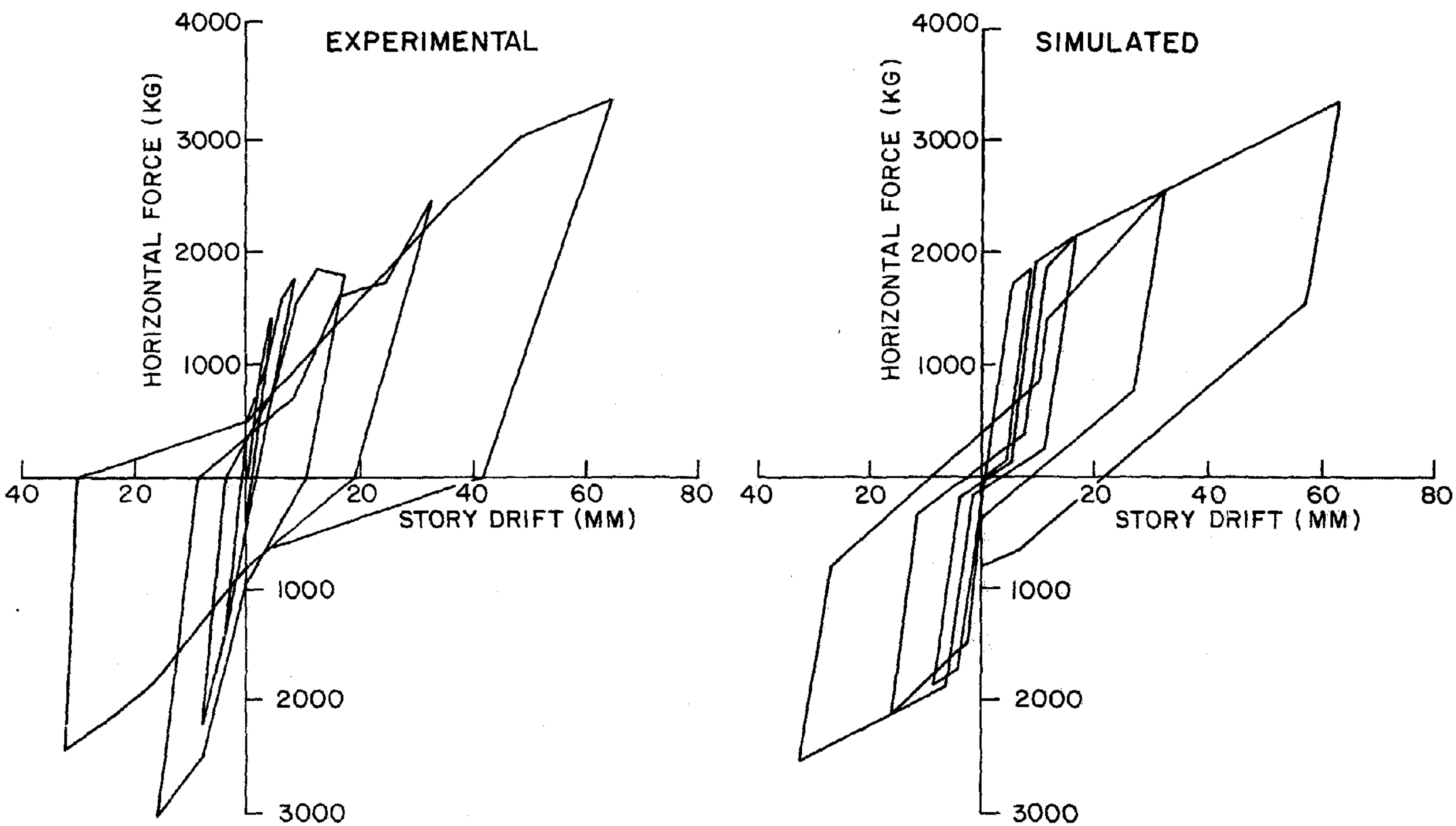


FIGURE C.4 COMPARISON OF SIMULATED BEHAVIOR TO EXPERIMENTAL DATA

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