REPORT NO.
UCB/EERC-79/28
SEPTEMBER 1979

EARTHQUAKE ENGINEERING RESEARCH CENTER

INFILL PANELS: THEIR INFLUENCE ON SEISMIC RESPONSE OF BUILDINGS

by

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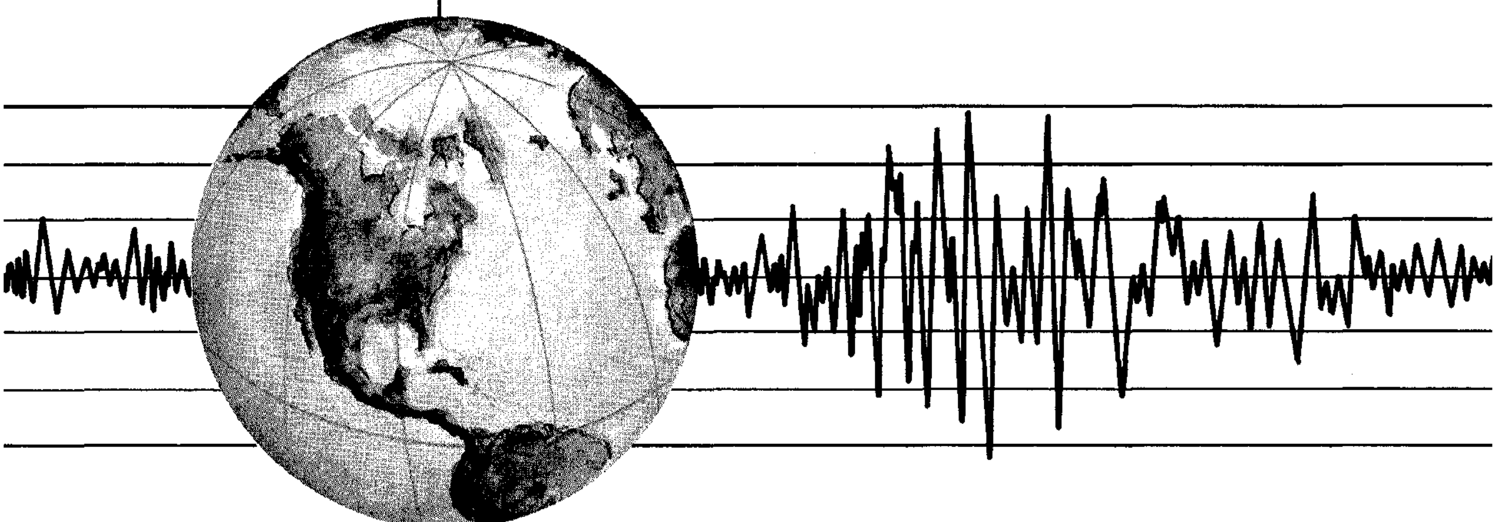
Report to Sponsor:
National Science Foundation



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A report on research sponsored by
the National Science Foundation

Report No. UCB/EERC-79/28
Earthquake Engineering Center
College of Engineering
University of California
Berkeley, California

September 1979

ia

BIBLIOGRAPHIC DATA SHEET	1. Report No. NSF/RA-790352	2.	3. Recipient's Accession No. PB 80 163371
4. Title and Subtitle Infill Panels: Their Influence on Seismic Response of Buildings			5. Report Date September 1979
			6.
7. Author(s) J.W. Axley and V.V. Bertero			8. Performing Organization Rept. No. UCB/EERC-79/28
9. Performing Organization Name and Address Earthquake Engineering Research Center University of California, Richmond Field Station 47th and Hoffman Blvd. Richmond, California 94804			10. Project/Task/Work Unit No. 0-22099
			11. Contract/Grant No. ENV76-04262
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

Infilled frame structural systems, wherein conventional frames are filled, in their plane, with construction, have consistently performed poorly in past earthquakes yet frame-infill systems continue to be used throughout the world as they provide an economic means to enclose and partition space that suits many local building traditions. There is, therefore, a clear need to develop methods to predict the behavior of frame-infill systems to anticipate undesirable behavior and take measures to avoid it. This study poses an answer to this need.

The problem of modeling the stiffness contribution of infill panels to elastic frame-infill systems is discussed. A set of dimensionless parameters is developed that is sufficient to define the nature of this stiffness contribution.

A means to model the structural behavior of frame-infill systems is proposed wherein it is assumed that the primary structural system (the frame) constrains the form of the deformation of secondary structural elements (the infill panels). It is suggested that such a constraint approach may be considered to be generally useful in modeling the behavior of certain classes of secondary structural elements.

This constraint approach, as developed here, is an approximate finite element substructuring technique that has the effect of reducing the analytical complexity of frame-infill systems and leads naturally to the development of a group of computationally attractive 12 degree of freedom infill elements that may simply be "plugged" into conventional frame analysis programs. Four infill elements are presented corresponding to completely and partially infilled frames with complete and partial constraint assumptions considered. Other possible elements are discussed briefly. The suitability and accuracy of the constraint approach is evaluated.

These infill elements are then utilized in a detailed three dimensional elastic analysis of a building that suffered extensive damage during the February 1976 Guatemalan earthquake. The nature of the response of this building to seismic excitation is considered and the influence of the infill upon this response is discussed in detail.

18. Availability Statement	19. Security Class (This Report) UNCLASSIFIED	21. No. of Pages 205
Release Unlimited	20. Security Class (This Page) UNCLASSIFIED	22. Price

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ABSTRACT

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Acknowledgments

The project on the elastic dynamic behavior of frame-infill systems reported herein was conducted as part of a larger project - **The Post Earthquake Damage Analysis Project** - with financial support provided by the National Science Foundation under Grant No. ENV76-04262, Subproject No. O-22099.

Dr. Steven Mahin, Professor of Civil Engineering, U.C. Berkeley and Dr. Stan Zagajeski, Professor of Civil Engineering, U. Colo., Fort Collins, provided informal, yet critical, assistance during the investigation that was greatly appreciated. Dr. Graham H. Powell, Professor of Civil Engineering, U.C., Berkeley and Dr. Keith Miller, Professor of Math, U.C., Berkeley, generously offered to review a final draft of the report, providing invaluable criticism.

Much of the work reported here was undertaken by James Axley, under the direction of Dr. V. V. Bertero, as a study leading to a dissertation for a Doctor of Philosophy degree in engineering.

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INTRODUCTION

Seismic structural research is directed toward the goal of the development of analytical techniques to reliably predict the mechanical behavior of structures under the combined excitations produced by gravity and ground motion generated by earthquakes as well as other possible excitations so that;

1. potential hazards may be anticipated

and

2. means to avoid or minimize these hazards, through improved design and construction practices, may be evaluated.

Experimental investigations and direct field observations of post earthquake damage serve to achieve this goal allowing comparison of observed and predicted behavior as well as providing direct evidence of the physical mechanisms responsible for the behavior upon which analytical models may effectively be developed.

Infilled frame structural systems, wherein conventional frames of reinforced concrete or steel are filled, in their plane, with construction usually of masonry, have resisted analytical modeling, although they have been studied experimentally for many years [A5]*. Indeed;

* Numbers in square brackets, [ ], refer to references found at the end of this section.

"At the present time no practical computer program capable of such an (frame-infill system) analysis has been published ..." 1977 [A3]

Yet, or perhaps because of this, buildings utilizing frame-infill systems have consistently performed poorly in past earthquakes [A10,A17]. Inspite of this, frame-infill systems continue to be used throughout the world as they provide an economic and direct means to enclose and partition space that suits many local building traditions.

In principal;

"The composite behavior of an infilled frame subject to racking combines the desirable characteristics, and attentuates the undesirable ones, of the separate wall and frame. Separately the wall is stiff but brittle and the frame is ductile but relatively flexible. Acting as an infilled frame, the combination is stiff, strong and tough. Consequently it has wide potential use for bracing buildings whether single storey, low-rise or high-rise." [A16]

In fact, experience suggests this potential may seldom be realized. Indeed, brittle failures of apparently ductile frames due to the unanticipated restraint offered by partial height infill panels as well as partially destroyed infill panels (eg. "short" or "captive" column behavior) has beccome characteristic of the seismic response of frame-infill systems.

The experience of the 1976 Guatemalan earthquake provided an especially interesting example of a building, the Escuela De Niñeras*, with a frame-infill structural system that suffered extensive damage but not complete collapse. This building is typical of a large number of buildings that utilize frame-infill systems in that it is a moderate sized building with a reinforced concrete frame and masonry infill. For this reason it was selected for a detailed analytical study.

This study is one of several studies of buildings damaged in recent earthquakes undertaken as part of a larger parent project, *The Post Earthquake Damage Analysis Project.* This

* Escuela De Niñeras - Nursery School

parent project is directed toward the general goal presented above and uses both experimental and direct field observation of seismic phenomena coupled with analytical studies to achieve this goal, with an emphasis placed upon characterizing damage response.

In an effort to model the seismic response of this building a means to model the stiffness contribution of infill panels to elastic frame-infill systems was developed. The "infill model" was then used in the dynamic analysis of the Escuela De Niñeras with some success. (The objectives, methodology and scope of these studies are discussed in section 1.2. of Part B of this study.) As expected the influence of the infill upon the dynamic character of the building and consequently its response to seismic excitation was seen to be significant, even dramatic. The infill model allowed an apparently reasonable estimation of local member forces that were seen to be reasonably well correlated to the oserved damage of both the structural frame and the infill.

The infill-model was developed using an approach that may be considered to be a general approach, for certain classes of secondary structural elements, wherein it is assumed that the primary structural system (eg. the frame) constrains the **form,** but not the degree, of the deformation of the secondary structural system (eg. the infill). The approach is classical in the sense that a simplifying kinematic assumption (eg. the constraint assumption) is made to achieve a solution but here the finite element method is used to solve the resulting problem numerically rather than analytically. This constraint method leads naturally to the development of computationaly attractive 12 degree of freedom infill elements that may simply be "plugged" into conventional frame analysis programs. Four infill elements were developed corresponding to completely and partially infilled frames with complete and partial constraint assumptions considered, other infill elements may also be developed by a straightforward application of the method as well (eg. infill panels with openings or of anisotropic material).

It is believed that the infill element developed in this study provides a practical means to predict the structural response of frame-infill systems and thereby allows full utilization of the potential added strength and stiffness offered by infill as well as the construction and use economies it affords.

The development of the infill element is considered in the first part of this report, Part A, and the use of this infill element in the dynamic analysis of the Escuela De Niñeras is discussed in the second part of the report, Part B. The Fortran source code for an infill element overlay, compatible with SAP IV (a structural analysis program [A1]), and its use is discussed in the Appendix.

Part A : ELASTIC STIFFNESS OF FRAME-INFILL SYSTEMS

1. Introduction

To the builder, especially in Latin American countries, infill panels provide an economic, simple and direct means to enclose and partition space. To the structural analyst, on the other hand, infill panels introduce unwanted analytical complexities and as a result they have often been ignored. The influence of stiff infill panels, such as masonry infill panels, is, however, of primary importance when considering lateral loading and may, conceivably, be important in gravity loading as well. In a typical situation an infill panel may stiffen a frame laterally by an order of magnitude and increase its ultimate strength by a factor of four. Such ignored sources of strength and stiffness can result in structural behavior completely unanticipated by the structural designer that may well be catastrophic.

To date, frame-infill systems have been modeled by either an "equivalent" strut approach or refined finite element discretization. (Klingner [A5] provides a thorough review of the literature on the subject.) The former method is simple and computationally attractive but is theoretically weak while the latter approach is complex and computationally prohibitive albeit theoretically sound.

Polyakov [A13], in his rather early experimental studies of simple isolated frame-infill systems, noted the typical separation of the infill panel and surrounding frame under moderate loading and suggested the infill panel could be modeled as an "equivalent" strut. This approach was intuitively satisfying to other investigators and was a clearly attractive approach from a practical computational point of view. As a result, much of the subsequent experimental work in this area was directed toward the goal of defining governing relations between characteristics of the frame-infill system and the "equivalent" strut. That such relations existed was, apparently, taken for granted.

Unfortunately, "equivalency" was considered in only a very limited sense. Apparently "equivalent" struts were selected so that the modeled system would have the same translational stiffness (racking stiffness) as the experimental specimen. Usually, only simple isolated frame-infill systems were considered, often a single infill panel with surrounding frame, and these were supported and loaded (ie allowed to have certain specific degrees of freedom) in a manner that generally would not correspond to practical situations.

In effect, then, the "equivalent" strut concept was developed upon an implicit assumption that the infill stiffness contribution could be related to a single parameter, or degree of freedom, the translational stiffness of the system. It is in this limited sense that equivalency was sought. Furthermore, there seems to have been little recognition that the support condition and loading of the experimental specimen could influence the apparent stiffness contribution of the infill by modifying the possible mode of deformation of the system. As a result, it was assumed that once the governing empirical relations had been developed, relating the frame-infill characteristics to the "equivalent" strut, for the simple isolated frame-infill systems then these struts could be used in more complex systems. There was no attempt to model other stiffness contributions of the infill, such as the rotational constraint to column-beam joints or local restraint to the frame members. These effects seemed of secondary importance in the simple frame-infill system studied. Unfortunately, it is not difficult to identify more complex frame-infill systems wherein these other stiffness contributions and the importance of different support conditions may be equally important.

In addition to the limited sense of equivalency inherent in the "equivalent" strut method, the approach is presently limited to infill panels that completely infill their surrounding frames. Although these struts may allow the prediction of global response they cannot be expected to correctly model response local to the infill panel and consequently leave the meaning of member forces, in this area, poorly defined [A3]. Furthermore, the many experimental investigations that have been directed ,in part, to develop this approach have yet to yield good agreement on the empirical relations to be used to size the "equivalent" struts. This uncertainty is often attributed to the uncertain material properties and quality of typical infill construction.

It is generally agreed that the "equivalent" struts should be placed diagonally within the frame, with thickness and material stiffness equal to that of the infill, yet there is some disagreement concerning suitable strut widths. Suggested widths for the same infill, may vary from one third to one tenth of the panel diagonal and are generally related to some measure of the relative stiffness of the infill to the frame and some measure of the system's geometry.

At the other end of the analytical spectrum from the "equivalent" strut approach is the refined finite element idealization of frame-infill systems. Linear elastic, nonlinear elastic and nonlinear inelastic models have been considered [A3,A6,A14]. Single infill panels have been modeled with as many as 15 plane stress elements having as many as 15 degrees of freedom each. The computational effort necessary for such refined models limits this approach to research studies, but we may be confident that this modeling approach will produce accurate and complete results if material properties and the sources of nonlinearity are carefully identified. The success of the finite element idealization lies in the complete sense of its equivalency to the real system. The finite element method achieves nearly complete equivalency by modeling the entire displacement field of the system rather than attempting to model the system based on one single displacement parameter.

This report presents a modeling approach that lies between the "equivalent" strut approach and the refined finite element models. A model of the infill is presented based upon the assumption that the frame constrains the form or shape (but not the degree) of the displacement of the infill. Such an approach may be thought to be a generalization of an idealization suggested by Newmark [A11] wherein the frame and the infill (constrained by a rigid linkage) are imagined to deform so that the corners of the infill remain compatible with the frame joints, but here compatibility is sought along the entire frame-infill boundary.

To those familiar with finite element procedures the use of constraint conditions to reduce the number of degrees of freedom of a system is not new or unusal, but its specific application and adaptation here is particularly suitable. Here, equivalency is sought in terms of an approximate displacement field whose validity must be demonstrated analytically and experimentally. The approach is justified in that it allows the development of a large variety of infill elements including elements suitable to model completely as well as partially infill frames, to model different degrees of constraint between the infill and surrounding frame and to model special cases of infill of unusual geometry, possibly with openings, or unusual material properties. Other elements that may prove useful to model certain nonlinear aspects of frame-infill panel behavior in a sense analogus to secant stiffness approaches presently used for one dimensional structural elements will be discussed subsequently.

It is worthwhile first, however, to consider the simplest frame-infill system, a single infill panel with surrounding frame, using dimensional analysis. In this way some special insight into the complexity of the problem, its phenomenological regimes and limiting cases of behavior will be realized.

2. Dimensional Analysis of A Simple Frame-Infill System

Consider the following simple problem;

A simple frame-infill system consisting of a single infill panel and surrounding frame is isolated and is to be idealized by a 12 degree of freedom model as indicated schematically in figure A2.1. The system is to be considered to be linearly elastic with the infill well bonded to the surrounding frame.

The frame alone may be modeled in the usual way by a 12 degree of freedom "frame stiffness" that may be directly subtracted from the proposed model stiffness leaving only the "infill stiffness" to be considered. This infill stiffness will, in general, have 144 terms, 22 of which will be independent, (if we limit ourselves to an isotropic elastic infill).

We may identify 12 system parameters (see Fig. A2.1) that will completely define the system and hence the infill stiffness. These include;

E_i = *The infill material stiffness*

t = *The infill thickness*

μ = *Poisson's ratio of the infill material*

E = *The frame material stiffness*

I_c = *The moment of inertia of the column sections*

A_c = *The area of the column sections*

I_b = *The moment of inertia of the beam sections*

A_b = *The area of the beam sections*

l' = *The length of the system*

l'' = *The length of the infill*

$h' = \text{The heigth of the system}$

$h'' = \text{The heigth of the infill}$

This set of parameters constitues a complete description of the system, although not a very concise one. Formally, then, the infill stiffness may be represented functionally as;

$$K_{ij}=f(E_i,t,E,I_c,I_b,A_c,A_b,l',l'',h',h'',\mu) \tag{2.1}$$

where K_{ij} corresponds to the i,jth term of the infill stiffness. We may attempt the heroic task of relating each term of the infill stiffness to these 12 parameters by appropriate analytical or experimental studies. Alternatively we can attempt to find a more concise description of the system using judgement and dimensional analysis.

By judgement we may reasonably replace the independent parameters of;

$$(E_i,\ t,\ E,\ I_c,\ I_b,\ A_c,\ A_b)$$

by the composite parameters of;

$$(E_i t,\ EI_c,\ EI_b,\ EA_c,\ EA_b)$$

thereby reducing the number of parameters to consider by two. These composite parameters were selected to reflect the characteristic stiffnesses of the infill, the columns and the beams, respectively, and as such they may be considered to be what Becker [A2] identifies as eigenmeasures. It also seems reasonable to replace the geometric parameters of l', l'', h', & h'' by the centerline geometric parameters of l & h as an approximation appropriate for the present discussion. (A correction for the error introduced by centerline geometry is suggested in the subsequent discussion of the development of an infill element.)

The description of the system has now been reduced from one of 12 parameters to one of 8 (including Poisson's ratio). A further reduction to 6 nondimensionless parameters (Becker's eigenratios) is possible by considering appropriate ratios of suitable eigenmeasures, (ie. by dimensional analysis). The 22 independent infill stiffness terms may thus be expected to be related, individually, to this set of 6 nondimensionless parameters. The eigenmeasures selected are;

(EI_c/h^3) = *A measure of the columns flexural stiffness*

(EI_b/l^3) = *A measure of the beams flexural stiffness*

(EA_c/h) = *A measure of the columns shear stiffness*

(EA_b/l) = *A measure of the beams shear stiffness*

(Eit) = *A measure of the infill in-plane stiffness*

h = *The centerline heigth of the system*

l = *The centerline length of the system*

μ = *Poisson's ratio of the infill material*

and the nondimensional functional relation is;

$$K_{ij}=f\left\{\frac{(EI_c/h^3)}{(Eit)},\frac{(EI_b/l^3)}{(Eit)},\frac{(EA_c/h)}{(Eit)},\frac{(EA_b/l)}{(Eit)},\frac{l}{h},\mu\right\} \tag{2.2}$$

It will be noted that the first four terms of this nondimensional relation are ratios of eigenmeasures of stiffnesses. Separate terms are seen to be necessary to account for both the relative flexural stiffness and the relative shear stiffness of the framing members. In typical construction, however, where one may expect to find similar cross section proportions for a range of section sizes, the use of only the dimensionless parameters relating flexural stiffness to the infill stiffness is warranted.

It is interesting to note that other investigators have identified similar dimensionless parameters. Mainstone [A7,A8,A9] has related the single degree of freedom stiffness corresponding to the racking stiffness of this simple isolated frame-infill system to dimensionless parameters essentially identical to the two flexural terms and the aspect ratio (h/l). He concludes, based upon experimental observation, that the relative flexural stiffness parameter for the framing columns is of principal importance and the aspect ratio is of little importance, "for all likely values of l'/h'." It will be noted that Mainstone, as well as, most other investigators were interested in only the racking stiffness of the system and a more complete description of the system stiffness may require the inclusion of these seemingly unimportant terms. There seems to be no recognition of the shear stiffness terms and Poisson's ratio, the importance of these parameters has yet to be disclosed.

Riddington [A14], using finite element analysis of frame-infill systems including frame-infill separation, considered, in effect, the importance of three of these dimensionless parameters; the relative flexural stiffness parameter for the framing columns as well as the framing beams and the aspect ratio of the infill. His study was apparently limited to static lateral loads of selected magnitudes. He concludes;

1. with regard to the relative flexural stiffness parameter for the beams;

   "... variation in the stiffness of the beam and its end connections do not significantly affect the behavior of the structure ..."

2. with regard to the aspect ratio;

"Stresses at the centre of an infill are influenced strongly by the height to length ratio but are almost independent of the frame stiffness."

3. with regard to the relative flexural stiffness parameter for the columns;

   "... changes in the stiffness parameter ... affect most significantly the infill corner stresses and the effective width of the infill ..."

and

4. with regard to estimating the stiffness of frame-infill systems;

   "The lateral deflection cannot be estimated accurately. A conservative value can be obtained from the static analysis ... with the infills replaced by diagonal bracing struts assigned to have effective width equal to one tenth of their length."

It should be kept in mind that Riddington's work as well as the work of most other investigators of frame-infill systems was directed toward sensitivity studies of a few of the parameters that govern the behavior of frame-infill systems. There seems to have been no (formal) attempt to identify the complete set of parameters that may affect the behavior of these systems nor to attempt to reduce the complexity of this set of parameters through the formal use of dimensional analysis. Although it was recognized that frame-infill system behavior is complex the degree of complexity was not clearly defined. Recognizing the complexity of characterizing the nature of frame-infill systems these investigators limited their studies to special conditions of loading, usually (equivalent) lateral static loads, and special conditions of support.

This report presents a different approach to the problem wherein a complete set of dimensionless parameters that are sufficient to define the behavior of the system is first identified, simplifying theoretical assumptions are sought to reduce this set to manageable size and then using this (reduced) set of dimensionless parameters the elastic force-deformation behavior of general frame-infill systems is considered for general conditions of loading and support. In an attempt to define parameter studies, on a more or less intuitive basis, that were of manageable scale, many of these earlier investigators limited their studies too severely and attempted to draw conclusions from practically limited but, nevertheless, insuffient amount of data. The success or failure of a parameter study approach lies in the scale of complexity of the problem under consideration. It behoves the researcher to attempt to define the scale of complexity and dimensional analysis provides one means to this end.

With the functional nondimensional relation in hand we are in a position to attempt to seek a governing relation between each of the 22 independent stiffness terms and the six dimensionless parameters. Such a task is certainly ambitious and yet, if successfull, we would have only modeled the simplest infill system and may not be able to use the results of such a study for modeling more complex systems. By considering a simple limiting case of the dimensionless parameters an alternative approach of modeling the system may be found.

If we consider a system that has a very stiff surrounding frame, in both the flexural and shear senses, and a relatively soft infill then it would be clear that the frame would constrain the deformation of the infill. Since we may determine the frame deformation from elementary principals we may use this field of deformation to determine the infill stiffness contribution. We may go one step further and ascert that if the infill is sufficiently soft, relative to the frame, then the **form** of deformation of the frame will determine the form of deformation of the infill. This begs the question; "How soft is sufficiently soft?" . This report attempts to answer this question and to do so the presented dimensionless parameters are used as measures of "softness".

A final note must be made, to facilitate subsequent discussion, however. It is worthwhile to also nondimensionalize the infill stiffness terms as;

i. Stiffness terms relating translational degrees of freedom to translational degrees of freedom (ie.displacements to forces) may be nondimensionalized by division by (Eit) the eigenmeasure of the infill stiffness.

ii. Stiffness terms relating translational degrees of freedom to rotational degrees of freedom (ie. displacements to moments or rotations to forces) may be nondimensionalized by division by $(Eit)(l^2+h^2)^{1/2}$. The expression $(l^2+h^2)^{1/2}$, was selected as a potentially better eigenmeasure of a characteristic length than either l or h alone and corresponds to the diagonal length of the system.

iii. Stiffness terms relating rotational degrees of freedom to rotational degrees of freedom (ie. rotations to moments) may be nondimensionalized by division by $(Eit)(lh)$ The product (lh) may be considered as an eigenmeasure of the systems characteristic area.

3. Constrained Infill Stiffness

An approximate approach to modeling the infill stiffness contribution is suggested by the discussion presented above. The infill stiffness may be determined if it is assumed that the frame constrains the (form of the) deformation of the infill. This then suggests a general approach of modeling the structural behavior of certain classes of secondary structural elements that are constrained to deform to the form of the deformation of the primary structure (eg. infill panels, stairways and, perhaps, floor slabs).

It is useful to apply this constraint approach to the simple isolated frame-infill system, discussed above, and compare the constraint approach to a more exact evaluation of the infill stiffness possible by a standard condensation procedure of a mesh of plane stress elements. Both approaches are presented schematically in figure A3.1.

In the more exact procedure the real system is first modeled by a sufficiently refined assemblage of beam and plane stress elements. The stiffness of this refined model is formed and then condensed to the desired 12 degree of freedom system stiffness. The frame stiffness is substracted from this reduced system stiffness leaving what may be considered to be the infill stiffness contribution to the system.

In the constraint approach the system is modeled by separate assemblages of finite elements for the frame and infill. The separate stiffnesses are formed and the stiffness of the infill alone is reduced, by condensation, to the boundary degrees of freedom. A constraint relation is assumed between the 12 frame degrees of freedom and the infill boundary degrees of freedom thereby allowing a congruent transformation of the separate systems to a composite approximate frame-infill system with 12 degrees of freedom, as desired. The frame stiffness is subtracted leaving a constrained infill stiffness contribution to the system.

(These seemingly complex schemes were selected for pedagogical reasons. The motivation for considering this difficult constraint approach will become clearer subsequently.)

Two questions follow naturally from these proposed schemes;

i. What constraint relation is to be assumed?

ii. When and in what sense (eg. norm) will the constraint infill stiffness be, nearly, equal to the more exact infill stiffness?

These questions will be left unanswered, temporarily, to consider the algebra of both approaches in greater detail.

3.1. Algebra of the "Exact" Scheme

The system is modeled by a sufficiently refined mesh of finite elements as indicated schematically in figure A3.1. For the present discussion centerline geometry will be assumed to be sufficiently accurate.

The stiffness of this n degree of freedom system may be formed *analytically* , in principle, and may be represented as;

$$\mathbf{K}_{n\times n}=(EI_c/h^3)\mathbf{K}_1+(EI_b/l^3)\mathbf{K}_2+(EA_c/h)\mathbf{K}_3$$

$$+(EA_b/l)\mathbf{K}_4+(Eit)\mathbf{K}_5 \qquad (3.1)$$

where,

$$\mathbf{K}_1, \mathbf{K}_2, \mathbf{K}_3, \mathbf{K}_4, \& \mathbf{K}_5$$

are analytic functions of geometry and Poisson's ratio, μ, alone. It is seen, therefore, that the unreduced system stiffness is linear in the characteristic stiffness terms that have been previously identified as suitable eigenmeasures of the system.

One may, again in principle, condense this system stiffness to the desired 12DOF system stiffness (eg. by a systematic application of Gauss elimination) and then subtract the frame stiffness to obtain the 12DOF infill stiffness. It is clear that such an analytical condensation and subtraction would result in a complicated, but rational, expression nonlinear in the system's eigenmeasures. Although equation (3.1) lends support to the dimensional analysis, (see equation (2.2)) we are in no better position to deal with the problem practically.

3.2. Algebra of the Constraint Scheme

The system is modeled by a separate finite element discretization of the frame and the infill, see figure A3.1 again. The frame is modeled by the obvious 12DOF beam system and the infill panel by a refined mesh of plane stress elements.

The frame degrees of freedom may be identified as $\mathbf{U}_f$ and the associated frame stiffness as $\mathbf{K}_{ff}$.

The infill panel degrees of freedom and the panel stiffness may be partitioned to distinguish those degrees of freedom that are to be constrained (eg. the boundary degrees of freedom), $\mathbf{U}_c$, and those degrees of freedom that are dependent on the constrained degrees of freedom, (eg. internal degrees of freedom), $\mathbf{U}_d$.

The stiffness relation for the unconstrained system, then, may be expressed as;

$$\begin{Bmatrix} \mathbf{F}_d \\ \mathbf{F}_c \\ \mathbf{F}_f \end{Bmatrix} = \begin{bmatrix} \mathbf{K}_{dd} & \mathbf{K}_{dc} & 0 \\ \mathbf{K}_{cd} & \mathbf{K}_{cc} & 0 \\ 0 & 0 & \mathbf{K}_{ff} \end{bmatrix} \begin{Bmatrix} \mathbf{U}_d \\ \mathbf{U}_c \\ \mathbf{U}_f \end{Bmatrix} \tag{3.2}$$

The system of equations are uncoupled as the constraint has yet to be imposed.

The dependent panel degrees of freedom may be "condensed-out" in the usual way, leaving;

$$\begin{Bmatrix} \mathbf{F}_c \\ \mathbf{F}_f \end{Bmatrix} = \begin{bmatrix} \hat{\mathbf{K}}_{cc} & 0 \\ 0 & \mathbf{K}_{ff} \end{bmatrix} \begin{Bmatrix} \mathbf{U}_c \\ \mathbf{U}_f \end{Bmatrix} \tag{3.3}$$

where,

$$\hat{\mathbf{K}}_{cc} = \mathbf{K}_{cc} + (-\mathbf{K}_{cd}\mathbf{K}_{dd}^{-1}\mathbf{K}_{dc})$$

The uncoupled system may now be coupled to form the combined frame-infill system. To achieve this coupling a constraint relation, **G**, is assumed between the constrained infill degrees of freedom and the system degrees of freedom, **U**;

$$\mathbf{U}_c = \mathbf{G}\mathbf{U} \tag{3.4a}$$

In as much as the frame-infill systems degrees of freedom are identical to the now-coupled frame degrees of freedom (ie. $\mathbf{U}_f = \mathbf{I}\mathbf{U}$) the constraint relation may be written as;

$$\begin{Bmatrix} \mathbf{U}_c \\ \mathbf{U}_f \end{Bmatrix} = \begin{bmatrix} \mathbf{G} \\ \mathbf{I} \end{bmatrix} \mathbf{U} \tag{3.4b}$$

The unconstrained system, equation (3.3), may then be constrained (coupled) by a congruent coordinate transformation as;

$$\mathbf{F} = \begin{bmatrix} \mathbf{G}^T \mathbf{I} \end{bmatrix} \begin{bmatrix} \hat{\mathbf{K}}_{cc} & 0 \\ 0 & \mathbf{K}_{ff} \end{bmatrix} \begin{bmatrix} \mathbf{G} \\ \mathbf{I} \end{bmatrix} \mathbf{U} \tag{3.5a}$$

or,

$$\mathbf{F} = \begin{bmatrix} \mathbf{G}^T \hat{\mathbf{K}}_{cc} \mathbf{G} + \mathbf{K}_{ff} \end{bmatrix} \mathbf{U} \tag{3.5b}$$

where **F** are the (coupled) system force degrees of freedom.

It is thus seen that the infill stiffness contribution;

$$\mathbf{K}_{infill} = \begin{bmatrix} \mathbf{G}^T \hat{\mathbf{K}}_{cc} \mathbf{G} \end{bmatrix} \tag{3.6}$$

is distinct and is simply added (in a direct stiffness assembly sense) to the frame stiffness, $\mathbf{K}_{ff}$, to obtain the combined frame-infill system stiffness.

To those initiated to the mysteries of matrix structural analysis this result is an obvious result of the assumption of the constraint. There are, however, other benefits of the constraint approach that may not be as obvious.

The algebra presented is not limited to the simple frame-infill system considered thus far. Complex frame-infill systems may be treated in the same manner, but in such a case the infill stiffness contribution of each individual infill panel will be given by individual terms of the form of equation (3.6) with, possibly, different constraint conditions, as;

$$\mathbf{F}=\Big[\mathbf{G}_1^T\hat{\mathbf{K}}_{cc1}\mathbf{G}_1 + \mathbf{G}_2^T\hat{\mathbf{K}}_{cc2}\mathbf{G}_2 + \mathbf{G}_3^T\hat{\mathbf{K}}_{cc3}\mathbf{G}_3 + \cdots$$

$$+ \mathbf{K}_{ff}\Big]\mathbf{U} \tag{3.7}$$

in other words, the constraint scheme provides a convenient means to substructure the infill panels or, equivalently, allows the generation of an infill macro element. A complex frame with completely infilled frames may then be modeled with the same number of degrees of freedom as the frame alone. Partially infilled frames may also be considered but extra nodes will generally be needed, yet the resultant increase in size of the system's stiffness may still be practically reasonable.

The uncoupled (unconstrained) infill stiffness is linearly related to the eigenmeasure of the infill, (Eit) . The reduced uncoupled infill stiffness, $\hat{\mathbf{K}}_{cc}$, is also, therefore, linearly related to this eigenmeasure, so we may write;

$$\hat{\mathbf{K}}_{cc} = (Eit)\hat{\mathbf{K}}^*_{cc} \tag{3.8}$$

where $\hat{\mathbf{K}}^*_{cc}$ is dependent upon the system's geometry and Poisson's ratio, μ, alone.

It is natural to define the constraint relation, $\mathbf{G}$, kinematically so it too will be dependent upon the system's geometry. The constrained infill stiffness, equation (3.6), may then be expressed as;

$$\mathbf{K}_{infill} = (Eit)\Big[\mathbf{G}^T\hat{\mathbf{K}}^*_{cc}\mathbf{G}\Big] \tag{3.9}$$

The product $\left[\mathbf{G}^T\hat{\mathbf{K}}^*_{cc}\mathbf{G}\right]$ is, then, dependent upon the system's geometry and Poisson's ratio, μ, alone.

The constraint approach, if reasonably accurate, then suggests the configuration of the simple, as well as, more complex frame-infill systems may be defined by three eigenmeasures rather than six as dimensional analysis indicates. These include the characteristic infill stiffness, (Eit), the system's geometry and Poisson's ratio. Furthermore the dependance on the first eigenmeasure is linear. For rectangular systems, then,

$$K_{ij} = (Eit)\ f(\frac{l}{h}, \mu) \tag{3.10}$$

(Compare this to equations 2.1 and 2.2 above.)

This last equation appears to contradict the conclusions drawn by earlier investigators. Here, equation 3.10, we see a linear dependency upon a simple measure of the infill in-plane stiffness, (Eit), and an (unknown) dependency upon the system's aspect ratio. Earlier studies of the BRE sought a (unknown) dependency upon a relative measure of infill in-plane stiffness (eg. relative to column flexural stiffness) and concluded there was little dependency upon aspect ratio, although their relative stiffness measure included some measure of the system's geometry. A closer examination of some of the experimental results and, indeed, the derived empirical relations given, relating the racking degree of freedom stiffness to the system's characteristics, seem, however, to support this result. For example, Mainstone [A8], reports the empirical relation;

$$\frac{k_R}{(Eit)\sin(2\theta)} = C\left(\frac{h^4(Eit)\sin(2\theta)}{EI_c h'}\right)^{(-0.01)}$$

where,

k_R=*SDOF racking stiffness*

$\theta = arctan(h''/l'')$

$C = a\ constant$

The constant, C, varies from 0.180 to 0.200 for brickwork and 0.120 to 0.133 for concrete.

The right hand side of this empirical equation is nearly constant, however, over a wide range of frame-infill combinations. One may as reasonably fit a "constant" curve to the experimental results as a power function as the experimental scatter was considerable. If this is done then;

$$\frac{k_R}{(Eit)\sin(2\theta)} = (Constant)$$

or,

$$k_R = (Constant)(Eit)\sin(2\theta)$$

a linear relationship with (Eit) and a nonlinear dependancy upon geometry, $\sin(2\theta)$, is revealed, encouraging further investigation of this approach.

3.3. Constraint Conditions

The constraint approach pivots upon the constraint assumption, yet the form of this constraint has been left ambiguous. Two classes of constraints have been considered;

A. Conforming constraints that assure the deformation of the infill will be contained within that of the frame.

B. Nonconforming constraints that may not offer this assurance.

3.3.1. Conforming Constraints

Two conforming constraints have been considered, to date, and a third may yet be investigated. These include (see Fig. A3.2)

1A. Cubic hermitian polynomials (the exact shape functions for end loaded flexural elements) have been used to define the constraint of infill- frame boundary degrees of freedom transverse to the framing member axis.

2A. Cubic hermitian for transverse boundary degrees of freedom with linear constraints (the exact shape functions for end loaded truss elements) used to define the additional constraint of infill-frame boundary degrees of freedom longitudinal to the framing member axis.

3A. Constraint 1A. or 2A. may, possibly, be considered in conjunction with a linear constraint assumption along the panel diagonal, utilizing two separate plane stress element meshes on either side of the diagonal to attempt to model a cracked panel.

(When flexural-shear beam elements are used to model the frame the cubic hermitian constraint will not, strictly, assure conformation.)

It will be convenient to refer to the components of the conforming constraints as either "cubic-transverse" or "linear-longitudinal". The second conforming constraint, 2A., is then the combination of cubic-transverse and linear-longitudinal constraints and corresponds to a frame-infill system with a continuously bonded panel, what Kost [A6], refers to as a "monolithic" panel. The "exact" scheme is inherently "monolithic", as presented, and may, therefore, be compared only to the constraint scheme utilizing the constraint 2A. This comparison has been made for a range of frame-infill proportions and is reported below.

Constraint 1A., cubic-transverse alone, corresponds, physically, to an infill panel that is not bonded to the frame yet may sustain compressive as well as *tensile* strains across the frame-panel boundary. This constraint will therefore produce an infill element softer than the "monolithic" element and may be suitable for modeling reinforced panels that have been deformed sufficiently to break the panel-frame bond (this often occurs at low load levels) yet tensile boundary strains may still be developed through the anchorage of the reinforcement.

Thus far, completely infilled frames have been considered and the appropriate constraints would therefore be applied to the four attached boundaries of the panel. The extension of the method to partially infilled frames is obvious; constraints would be applied to only the attached boundaries of the panel, the free boundaries would be unconstrained. (If the free boundary-frame gap is extremely small then the nonlinear contact-release problem may be important.) An additional extension of the method to non-rectangular geometries, unusual partial infilling and complete or partial infill panels with openings or infill of anisotropic material nature is straightforward, but will not be considered here.

Each combination of constraints and geometry produces a separate group of infill macro elements. In the present report only rectangular 12DOF elements will be considered. It is tempting to believe the 12DOF macro element corresponding to the completely constrained "monolithic" panel would be identical to a 12DOF plane stress element based on cubic-linear shape functions. This finite element has, in fact, been developed, analytically, by Oakberg and Weaver, [A12]. Both the proposed macro element and the Oakberg-Weaver element have the same boundary constraints (shape functions) that would be compatible with adjacent framing elements. The interior degrees of freedom of the Oakberg-Weaver element are, however, over-constrained by the specified shape functions, while the interior degrees of freedom of the "monolithic" macro element are free to find their minimum potential and should, therefore be expected to produce a more accurate element.

In the present study it was concluded that 16 constant strain rectangular plane stress elements were sufficient to achieve reasonable convergence to the 12DOF "monolithic" element stiffness. This element proved to provide much greater accuracy than the corresponding Oakberg-Weaver element in modeling the simple frame-infill system's behavior (as compared to the results of the "exact" scheme).

It appears, therefore, that the constraint scheme provides a means to "bootstrap" lower order accuracy elements up to higher order accuracy elements. In effect, the finite element method is used to find optimum interior shape functions, numerically, for given boundary shape functions. Although the implications here are fascinating there was not time to consider this aspect further.

3.3.2. Nonconforming Constraints

Nonconforming constraints may violate the important requisite of the finite element method of compatibility which assures convergence of the method. These constraints must, therefore, be considered as an ad hoc approach, but with care they may prove useful. The number of nonconforming constraints that may be considered is limited only by one's imagination, but here only two nonconforming constraints are considered.

1B. Experimental studies have repeatedly shown the tendency of the frame and infill to separate under moderate loading. As a result the infill is placed under diagonal compression. With this observed behavior in mind an infill stiffness was developed assuming only nodes at and adjacent to diagonally opposed corners of the panel were constrained by the frame, in the cubic-transverse sense (see Fig. A3.2). This infill stiffness presupposes a racking deformation in only one direction and may not conform under other system deformations. This nonconforming approach is compared below to one experimental test.

2B. To achieve a first order correction to the error introduced by using centerline geometry rather than true panel geometry one may, conceivably, use an infill plane stress mesh and constraint relations based upon true panel dimensions. Such an approach will not, strictly, conform with the centerline frame model yet would conform with a rigid joint model (beam-column joint) model. The additional computational difficulties necessary for the rigid joint model seem unwarranted here while the use of true panel geometry infill element may be accomplished with no additional effort. Therefore, it seems reasonable to use such an infill model with the centerline frame model to achieve a more nearly correct model of the system (see Appendix).

3.4. Accuracy of the Constraint Scheme

The practical suitability of the proposed method depends largely upon its accuracy. It has been shown, heuristically, that the method will be accurate for frame-infill systems with relatively soft infill panels. One may compare the 12DOF infill stiffness contribution generated by the constraint scheme to that generated by the "exact" scheme to answer the question posed earlier;

> When and in what sense (eg. norm) will the constraint infill stiffness be nearly equal to the more exact infill stiffness?

Alternatively, one may compare observed experimental behavior with modeled behavior to assess the accuracy of the constraint method.

3.4.1. Analytical Evaluation of Accuracy

Constrained and "exact" 12DOF infill stiffness matrices were generated for two different series of simple isolated frame-infill systems (Fig. A3.4.1);

Parameter Study Series

The first series was selected to allow a parameter study of the influence of panel thickness and system length on the accuracy of the constraint method. Both completely infilled frames and partially infilled frames were considered. A conventional parameter study approach was considered to have intuitive merit over a nondimensional parameter study, albeit less generality.

Nondimensional Parameter Study Series

The second series was selected to allow a less conventional but more general nondimensional parameter study. In as much as the "exact" system is determined by its configuration (ie. the set of nondimensional parameters) the error inherent in the constraint method of modeling of the "exact" system will be determined by the same configuration. The nondimensional parameters are, therefore, the logical parameters to relate to this error and provide a convenient means to define, quantitatively, the relative softness of the infill.

Within this study series column section and beam section proportions were kept constant at 1:1 and 4:5 respectively. These values were thought to be reasonably typical of actual construction and allowed the reduction of the systems configuration by two dimensional parameters. Poisson's ratio, μ, was not varied. A constant value of 0.15 was assumed. The influence of three of the original six nondimensional parameters were, then, considered;

$$\frac{(EI_c/h^3)}{(Eit)} = \textit{The relative stiffness of the beams}$$

$$\frac{(EI_b/l^3)}{(Eit)} = \textit{The relative stiffness of the columns}$$

$$(l/h) = \textit{The aspect ratio}$$

The cubic-transverse plus linear-longitudinal constraint, (constraint 2A., section 3.3.1. above), was used in the constraint scheme to assure a consistent comparison between the "exact" and the constraint approach results (this aspect was discussed earlier). As this constraint will result in the stiffest possible infill contribution we may expect the modeling inaccuracy associated with this type of constraint to place an upper bound upon the modeling inaccuracy of infills constrained otherwise. That is to say, any other constraint would lead to softer infill behavior which would therefore satisfy, more exactly, the basic theoretical assumption that the frame constrains the deformation of the infill.

The generated 12DOF infill stiffnesses were characteristically well coupled, completely filled stiffness matrices, as might be expected. (It is worthwhile to digress to note that the stiffness contribution of an "equivalent" strut can not provide this same richness - it quite simply would not even "look right".)

The exact and constrained infill stiffnesses were first compared term by term. This comparison, which is, in effect, the maximum error norm;

$$||\underline{E}||_\infty = ||\frac{\mathbf{K}_{approx}}{\mathbf{K}_{exact}}||_\infty \tag{3.11}$$

where;

$$||\underline{E}||_\infty = \textit{the maximum error norm}$$

$$||\frac{\mathbf{K}_{approx}}{\mathbf{K}_{exact}}||_\infty = \max|\frac{k_{ij_{approx}}}{k_{ij_{exact}}}| \textit{for all terms of the stiffness matrices}$$

verified the conclusion that the constrained infill stiffness would approach the exact infill stiffness as the infill became, relatively, more and more soft. The error in individual stiffness terms was however seen to be large for typical construction and yet the constrained infill provided reasonably accurate modeling of system behavior.

It was felt that this norm (equation 3.11) was not an appropriate measure of accuracy for the modeling of system behavior. From perturbation analysis we know that for (stable) stiff systems a relatively large error in individual stiffness terms will produce smaller relative errors in the system's behavior (ie. displacements). As frame-infill systems are characteristically stiff the maximum error norm may be expected to provide an overly conservative estimate of error. As the principal objective of this study was aimed to accurately modeling system behavior and not correctly estimating individual infill stiffness terms another measure of accuracy was sought.

As the frame-infill system's behavior was to be modeled two additional norms were selected that used measures of the system's stiffness rather than the infill stiffness alone. The first of these measures corresponds to the SDOF translational stiffness of the stably supported system shown in figures A3.4.2, A3.4.3 and A3.4.4. The second measure is the SDOF rotational stiffness also shown in these figures. These measures were selected for their obvious physical significance and were determined by forming the system stiffness, restraining the 1st, 2nd and 10th degrees of freedom and condensing to either the 11th or 12th degree of freedom. The norms were then simply taken as the ratio of the constraint method SDOF stiffness to the "exact" SDOF stiffness for the translational DOF and for the rotational DOF selected.

The results of the conventional parameter study are shown in figure A3.4.2 for completely infilled frames and figure A3.4.3 for partially infilled frames. The results of the nondimensional parameter study, for one aspect ratio (l/h) = (6.5/3.0) are shown in figure A3.4.4. On all of these figures a typical as-built condition (ie. of realistic size and materials) is indicated as the range of parameters considered extended well beyond realistic ranges.

Again, it is seen that the constraint approach provides greatest accuracy for softer infill in terms of;

i. thinner infill panels,

ii. less infill panel (ie. partially infilled frames)

iii. lower infill stiffness, E_i, relative to the beam stiffness,

iv. lower infill stiffness, E_i, relative to the column stiffness,

and probably,

v. softer constraints.

The parameters were varied well beyond practical limits, as, for example, the panel thickness varied above and below typical values by an order of magnitude. The constraint method SDOF stiffnesses were consistently greater than the "exact" values as is to be expected from a conforming yet less complete finite element idealization.

The inaccuracy associated with the selected SDOF translational stiffness is relatively small for the range of parameters considered, while the SDOF rotational stiffness error is not only always greater but is more sensitive to these parameters. The accuracy provided in modeling the SDOF translational stiffness is well within the uncertainity of the infill stiffness, E_i, of typical construction. The inaccuracy in modeling the SDOF rotational stiffness is relatively large, however, with very stiff infills. In typical situations the infill may increase the SDOF translational stiffness of the frame by an order of magnitude while only doubling the SDOF rotational stiffness, so this error may not be very significant in many cases.

3.4.2. Experimental Evaluation of Accuracy

Three different models were made of a full scale specimen tested by the British Research Establishment (BRE) [A8]. The details of the specimen, its support conditions and load-deformation behavior under monotonic loading are presented in figure A3.4.5. This particular test was selected principally because the details of the test and the specimen were well reported. The BRE measured the infill material stiffness by direct compression tests. The specimen was loaded to low load levels and unloaded three times then monotonically loaded to failure. The early low level load histories were carefully recorded (see Fig. A3.4.6(a)).

The specimen was especially suitable for modeling by the proposed method as the frame was particularly stiff. The three constraint method models were based upon each of three different constraints;

Model A

A cubic-transverse plus linear-longitudinal (ie. "monolithic") constraint was assumed. This corresponds to the conforming constraint 2A, presented in section 3.3.1. (Fig. A3.2).

Model B

The cubic-transverse constraint alone was assumed. This corresponds to the conforming constraint 1A, (Fig. A3.2).

Model C

The nonconforming constraint discussed above, 1B, was assumed, (Fig. A3.2).

The load-deformation behavior of each of these three models (ie. straight line elastic behavior) is plotted with the observed behavior in figure A3.4.6. The elastic frame behavior and the "exact" elastic model behavior are also shown. The early, low level, loading results shown in the figure (A3.4.6(a)) are particularly interesting.

The initial stiffness of the uncracked specimen is very well modeled by the "monolithic" Model A while the stiffness attained after the appearance of the first diagonal cracks and possibly bond failure at the frame-infill interface (ie. the 4th loading) appear to be well modeled by Model B (Fig. A3.4.6(a)). Model C appeared to provide a reasonable "secant-stiffness" approximation to the state of initiation of large inelastic deformation (Fig. A3.4.6(b)).

All three models produced very similar results in relation to the complete load-deformation behavior shown in figure A3.4.6(b).. This one study, although encouraging, does not verify the validity of the constraint method. One may expect, however, that the softer constraint condition of Model B would in some sense model an unbonded infill panel and it appears from this study that it may suit this purpose.

4. Approximate Infill Stiffness

It has been shown that the constraint method allows unusual flexibility in modeling a range of different infill panels as substructures (or macro-elements). The accuracy of the method appears to be reasonably good, but the computational effort necessary to generate these elements directly, as proposed, is substantial, requiring;

1. generation of a suitably refined mesh of plane stress elements,

2. formation of the stiffness matrix of this mesh,

3. reduction to the constrained degrees of freedom,

4. formation of the constraint relation,

and,

5. congruent transformation to the governing degrees of freedom.

This direct approach may nevertheless be useful for modeling structures with many identical infill panels or possibly when the use of unusal constraints seems particularly suitable.

The constraint method may, alternatively, be used to generate approximate relations that may then be be used to form the constrained element stiffness avoiding direct generation, thereby offering an attractive computational economy, albeit, an additional loss of accuracy.

It has been shown (equation 3.10) that the infill element stiffness, generated by the constraint method, will be dependent upon the characteristic infill stiffness, (E_it) , the infill geometry and Poisson's ratio. Furthermore, it will be recalled that the dependency on the first parameter, (E_it), is linear. If the generated infill stiffness is nondimensionalized as outlined in section 2.0 and only a single value of Poisson's ratio considered (μ=0.15 was assumed here) then each of the independent terms of the nondimensional stiffness may be related, via approximation, to the system's geometry. For rectangular panels the aspect ratio provides a sufficient and convenient measure of system geometry.

Using this approximate extension of the constraint method four approximate infill elements have been developed (Fig. A4.1) corresponding to the four conforming elements discussed above. Those elements based upon the combined cubic hermitian transverse constraint and linear longitudinal constraint will be referred to as "stiff" elements while those based upon the cubic transverse constraint alone will be referred to as "soft" elements. We may then identify the four infill elements as either;

1. stiff complete,
2. soft complete,
3. stiff partial,

or,

4. soft partial infill elements (see Fig. A4.1).

Nondimensional infill stiffnesses were generated for a range of rectangular infill aspect ratios. Each of the independent nondimensional infill stiffness terms, S_{ij}, were related to the infill aspect ratio using a third degree polynomial least squares fit, eg.,

$$S_{ij} \approx S_{ij}^{*} = C_{1_{ij}} + C_{2_{ij}}(l/h) + C_{3_{ij}}(l/h)^2 + C_{4_{ij}}(l/h)^3 \tag{4.1}$$

where,

S_{ij} - is the i,jth term of the nondimensionalized infill stiffness generated directly by the constraint approach. Each individual stiffness term was nondimensionalized as; (see section 2.)

$$S_{ij} = \begin{cases} \dfrac{K_{ij}}{(Eit)} & \textit{trans. to trans. DOF} \\ \dfrac{K_{ij}}{(Eit)(l^2+h^2)^{1/2}} & \textit{trans. to rot. DOF} \\ \dfrac{K_{ij}}{(Eit)(lh)} & \textit{rot. to rot. DOF} \end{cases} \tag{4.2}$$

S^{*}_{ij} - is the approximated i,jth term of the nondimensional infill stiffness from the polynomial fit to the computed values S_{ij}.

and,

$C_{1_{ij}}, C_{2_{ij}}, C_{3_{ij}}, C_{4_{ij}}$ - are the polynomial coefficients corresponding to the i,jth stiffness term determined by least squares fit to the data obtained using the constraint approach.

4.1. Accuracy of the Approximate Infill Elements

The suitability and accuracy of the approximate infill elements was evaluated by comparison with analytical studies and experimental studies of frame-infill systems. Two example frame-infill systems studied analytically by Kost [A6] and one experimental specimen studied by Vallenas [A18] were modeled using the approximate infill elements presented above. Equivalent strut models were also considered, using the BRE method for sizing these "equivalent" elements [A7,A8,A9].

4.1.1. Analytical Evaluation of Accuracy

Kost's One Story Infilled Frame

A single story infilled frame supported on a rigid base was analyzed in detail by Kost [A6] using as many as 15 plane stress elements with up to 15 degrees of freedom each. First mode periods for the frame alone and the infilled frame were reported and are presented in table A4.1.

The results of eigenanalyses of systems using the indicated approximate infill elements, the BRE "equivalent" strut and an "equivalent" strut system ten times as stiff are also presented in this table. The BRE single equivalent strut was replaced by a double strut system, each strut with half the require BRE stiffness, to enable a reasonable (ie. symetric) model to be made for eigenanalysis yet maintaining practically the same racking stiffness.

The stiff approximate element produced a first mode period that was in good agreement with Kost's results without the enormous computational effort required to obtain Kost's results. The BRE "equivalent" strut stiffness had to be increased by an order of magnitude to achieve the same agreement. The rigid support used here allows a greater participation of the infill than that possible with the infills considered by the BRE in the development of their "equivalent" strut.

Kost's Four Story Example

A four story example studied by Kost was also modeled with approximate infill elements and "equivalent" struts. The results of Kost's eigenanalysis are compared to those obtained using stiff approximate infill elements, soft approximate infill elements and BRE "equivalent" struts in table A4.2 and figure A4.2. In all of these models only lateral dynamic degrees of freedom were considered (ie. only masses affecting horizontal translational degrees of freedom were modeled) and only the lateral components of the mode shapes are presented.

The mode shapes and periods of the model employing the stiff approximate infill elements are seen to be in excellent agreement with Kost's results while the BRE "equivalent" struts result in estimates of natural periods over double the values reported by Kost. It may be argued that the "equivalent" strut model should be expected to result in longer natural periods as these struts were developed from experimental data of test specimens that were, most certainly, less than monolithic and as such may better model real system behavior. Although a real system may be expected to be less than monolithic it is believed that the BRE struts result in a significant overestimate of the real system natural periods and the soft approximate infill element may provide a better modeling of real system behavior. It may be seen that the model using these soft infill elements resulted in periods somewhat longer than the monolithic models.

Ironically, it appears that the BRE "equivalent" struts capture the first four mode shapes accurately (in comparison to both Kost's result and those obtained using the stiff infill element). A more complete modeling of the system including vertical as well as lateral dynamic degrees of freedom reveals the shortcomings of the use of "equivalent" struts in dynamic analysis, however. The results of such a more complete mass modeling are presented in figure A4.3 (Kost did not report such a case). The "equivalent" struts were developed principally to model the lateral stiffness contribution of infill panels to their surrounding frames. For this reason, perhaps, these struts were able to capture the lateral components of the mode shapes when only lateral dynamic degrees of freedom were allowed. When, however, both vertical and lateral dynamic degrees of freedom were allowed the struts not only fail to provide good estimates of the natural periods but fail to capture the mode shapes well (Fig. A4.3). If it is assumed that the stiff approximate infill modeling provides an accurate modeling it is evident, from a comparison of these mode shapes (Fig. A4.3), that the "equivalent" struts fail to model the vertical stiffness contribution of the infill to the frame. Again this evidence points to the limited sense of equivalency offered by "equivalent" struts.

4.1.2. Experimental Evaluation of Accuracy

The three story concrete frame-wall specimen sketched in figure A4.4 was among several specimens studied by Vallenas [A18]. This specimen was modeled using both stiff and soft approximate infill elements. True panel dimensions for the infill, gross section geometry for the framing members and material properties reported by Vallenas were used in the modeling. The fundamental frequency of the actual specimen was found to be 45 cps in a first release test and 41 cps in a second release test. The analytical results for the fundamental mode were very close to the measured results; 49 cps using stiff infill elements and 38 cps using soft infill elements.

The measured load-deformation behavior for the indicated loading pattern was well estimated by the infill models in the early stages of loading, as to be expected (Fig. A4.3). Again (see section 3.4.2) the results of the stiff and soft modelings seem to bracket the actual initial behavior of the specimen and may possibly provide upper and lower bounds on the initial elastic behavior of other frame-infill systems as well.

4.2. Rigid Body Modes

The approximate extension of the constraint approach offers computational econcomy at the expense of a loss of accuracy. In the representation of the elastic stiffness contribution of infill panels to frame-infill systems, however, this loss of accuracy appears to be well within the likely uncertainty of the mechanical characteristics of common infill construction. In the representation of rigid body modes of deformation, on the other hand, the error produced by this approximate extension of the constraint approach may prove to be problematic.

A successful infill element should, properly, be able to translate and rotate in its plane rigidly (ie. without deformation) thus without straining. The infill element stiffness generated directly by the constraint approach satisfies this condition (to machine accuracy). The approximate extension of this approach introduces sufficient error to significantly compromise this rigid body mode representation. In extreme cases this error may result in a system stiffness with negative eigenvalues (ie. an indefinite system) that will inhibit rational analysis. A scheme was, therefore, devised to improve the computed approximate infill stiffness matrix by, essentially, shifting the eigenvalues corresponding to the rigid body modes closer to zero. This technique is discussed in the Appendix. In the future it will be desireable to devise more effective means, if possible, to provide better rigid body mode representation or alternatively apply the constraint approach directly to avoid this problem.

5. Member Force Evaluation

The infill element was developed to answer an interim objective of modeling the stiffness contribution of infill panels to elastic frame-infill systems and to this end the infill element has proven useful. The structural designer, however, needs not only an estimation of stiffness contribution but also a reasonable estimation of member forces to make rational design decisions. It is natural, then, to consider the accuracy of member force evaluation offered by the use of the infill elements presented above.

The four approximate infill elements presented above are based upon displacement field assumptions that are compatible with the surrounding framing members modeled with conventional beam elements*. From basic finite element theory we know that a compatible displacement field assumption offers assurance of a least energy best fit estimation of member forces. The use of approximation to avoid direct generation of the infill elements introduces an error, however, that is yet of uncertain importance. For this reason member force evaluation was considered briefly using two simple example cases.

5.1. Frame

Framing member forces obtained from the refined "exact" finite element scheme discussed above were compared to those results obtained using approximate infill elements (Fig. A5.1). In as much as the "exact" scheme corresponds to a monolithic frame-infill system only the stiff approximate infill elements were considered as they correspond to a monolithic system and are therefore comparable.

It is seen that the 12 degree of freedom infill elements allow only constant value evaluation of adjacent framing member axial and shear forces and straight line variation of internal bending moment as the beam elements used to model these framing members are conventional beam elements. Yet the least energy best fit nature of this evaluation (ie. a straight line

* Compatibility will not strictly be attained if beam elements are used that include shearing deformation although the resulting error in typical applications should be expected to be small (see section 3.3.1).

approximation based upon the minimization of elastic strain energy) appears to be uncompromised by the error resulting from approximation, in these examples.

It appears that the approximate infill element allows the nature of framing member force variation to be captured as well as may be expected from an element with such a limited number of degrees of freedom. The member force evaluation for the partially infilled frame is particularly encouraging. Eventhough a practically accurate straight line estimation of member force variation has been achieved in these examples the deviation from the straight line is significant. The axial force, internal bending moment and, especially, shear force of the framing members are significantly underestimated at the end regions of the framing members. Yet these regions are inevitably the critical regions of the system. The inability of the proposed method to capture the detailed nature of member force variation is an important limitation of the method and must be kept in mind. It is of some consolation to recognize that the exact shear force variation presented here represents a conservative estimate as the real system behavior will be less monolithic and may be expected to have significant inelastic behavior (and hence force redistribution) at even low load levels.

It is interesting to note that the tension of the "windward" column, in these examples, is of greater magnitude than the compression of the "leeward" column. With the infill removed this tension and compression would be of nearly equal magnitude. A more detailed examination reveals that the infill has the affect of increasing the tensile force in the windward column and decreasing the compressive foce in the leeward column. This may be characteristic of infilled frame response, the "equivalent" strut idealization of frame-infill behavior would support such a conclusion. Such behavior will tend to aggrevate problems of shear behavior in these columns.

5.2. Infill

The member forces associated with the infill are twelve in number and correspond to the twelve degrees of freedom of the infill elements. These member forces have little physical meaning alone yet one may determine more familiar panel force quantities (eg. average shear stresses, average axial stresses and internal resisting moment) from simple static analysis of appropriate free bodies using these member forces. Average horizontal shear stress is perhaps the most interesting single measure of infill member force response available from these elements (see section 5.5.2 of Part B of the report).

6. Conclusion

A means to approximately substructure infill panels has been presented. One may conceivably apply the basic approach to other secondary elements as well, (eg. floor slabs). The method may then be considered to be a general approach of substructuring certain classes of secondary structural and "nonstructural" elements that are, more or less, constrained to deform by a primary structural system. The method is a particularly attractive approach in that it allows the consideration of complex primary-secondary structural systems without substantially increasing the number of degrees of freedom to be considered above that of the primary system alone.

Although approximate, the method is theoretically consistant allowing considerable flexibility in the type of infill panel that may be considered including;

i. Completely infilled panels

ii. Partially infilled panels

iii. Possibly, "outfilled" panels (ie. panels adjacent to columns but placed outside of the frame)

iv. Rectangular and nonrectangular geometries

v. Panels with openings

vi. Panels of anisotropic materials

and

vii. Panels constrained to deform under various different constraint assumptions.

The preliminary study of error, presented here, indicates that the constraint approximation may provide a degree of accuracy well within the inevitable uncertainty of the infill material stiffness, homogenaity, and continuity, when conforming constraints are assumed. Although nonconforming constraints may not offer this assurance it appears, from one case studied, that they may also be of use if carefully applied.

The form of the constrained infill stiffness is simple enough to allow the generation of approximate infill stiffnesses and hence approximate infill elements. Four such infill elements have been developed that are intended to be used with existing general purpose frame analysis programs. The three examples studied using these approximate elements suggest that the additional error produced by this approximate extension of an already simplified method may be tolerable while the computational savings is substantial.

Although the constraint approach, and the approximate infill elements, appear to provide a computationally efficient yet practically accurate means of estimating the initial elastic stiffness of frame-infill systems this approach has some important limitations with regard to member force evaluation. The estimation of member forces and stresses, local to an infill panel and surrounding frame, is limited to mean value estimates (ie. least energy mean variation in framing member forces and average shear stress levels in infill panels) that may significantly underestimate the extreme values of member force and panel stress. Unfortunately, 12 DOF infill elements and single beam elements can offer no more.

In actual building response damage will be initiated at those regions where the extreme values occur. The designer, or research engineer, may not, then, be able to pinpoint these locations nor estimate the magnitude of these extreme values using the constraint approach alone. The constraint approach will, however, provide some indication of probable critical panels and framing members and their distortion that may be used as a basis for more refined finite element analysis.

Existing general purpose structural analysis programs would require moderate modifications to form the approximate substructures as proposed, however, programs have recently been developed with this capability, [A4,A15]. For this reason the approach seems worthy of additional study and as its theoretical basis is clear and consistant, (to the extent that the constraint approximation is consistant), it may prove useful to extend the method to model nonlinearities resulting from nonlinear constraint conditions (eg. conditional constraints) as well as nonlinear material behavior and possibly cracking.

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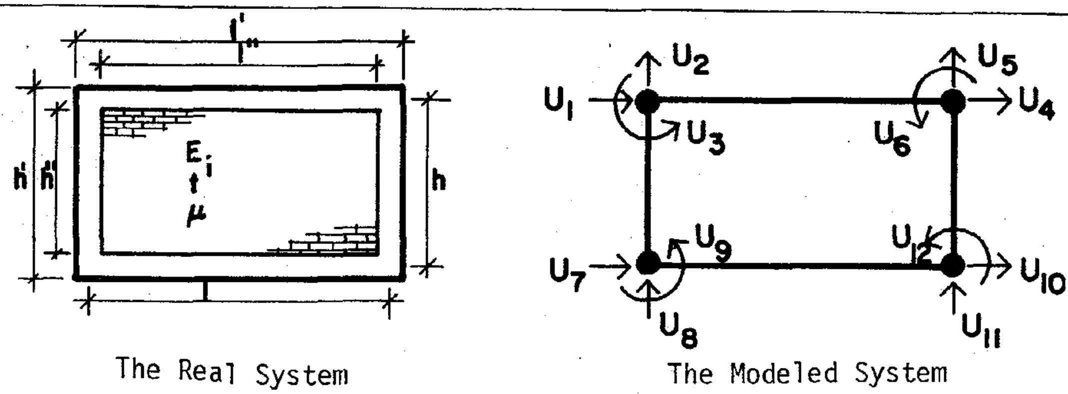


FIG. A2.1 A SIMPLE FRAME-INFILL SYSTEM

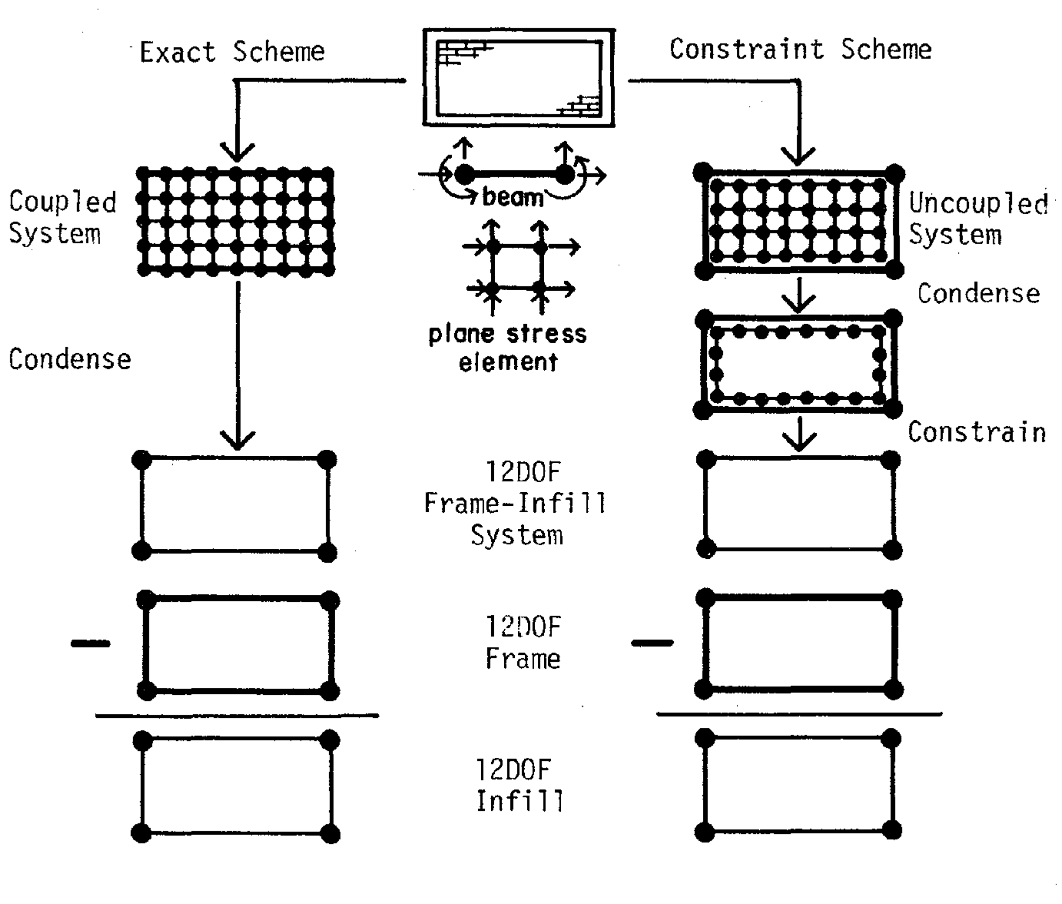


FIG. A3.1 MODEL SCHEMES

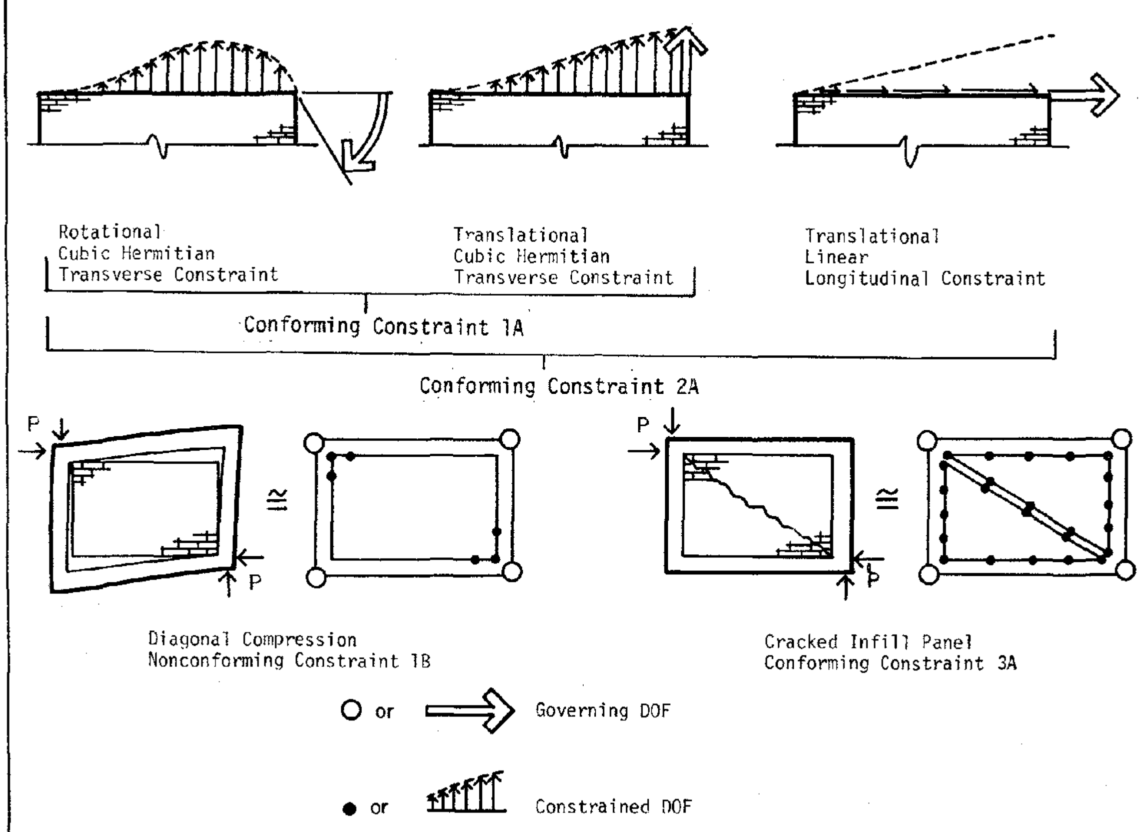


FIG. A3.2 CONSTRAINT CONDITIONS

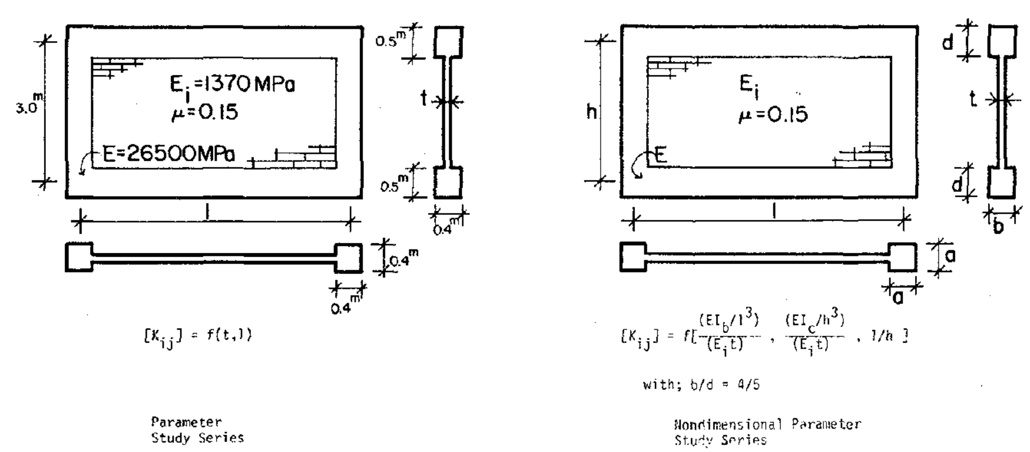


FIG. A3.4.1 FRAME-INFILL SYSTEMS STUDIED

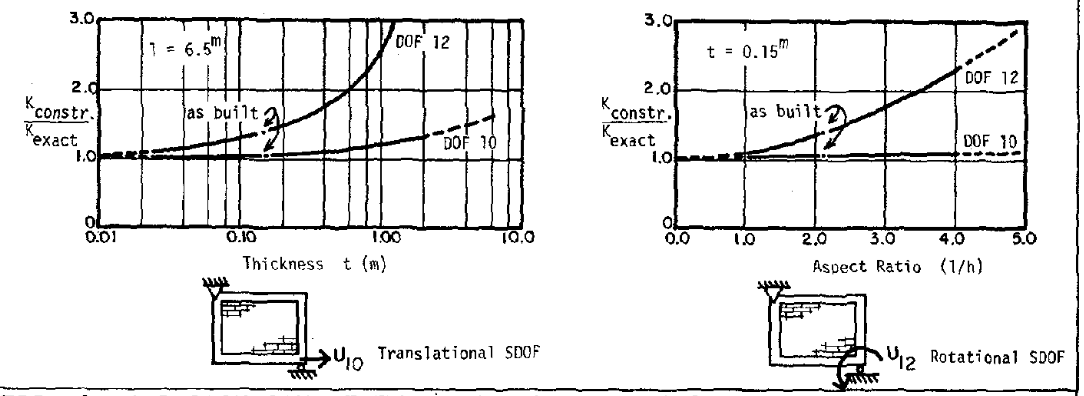


FIG. A3.4.2 ACCURACY OF THE CONSTRAINT APPROACH
PARAMETER STUDY OF COMPLETELY INFILLED FRAMES

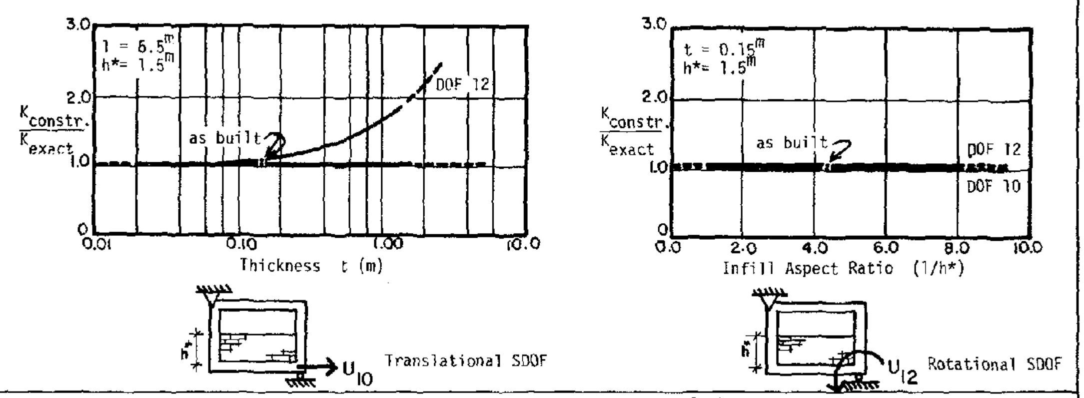


FIG. A3.4.3 ACCURACY OF THE CONSTRAINT APPROACH
PARAMETER STUDY OF PARTIALLY INFILLED FRAMES

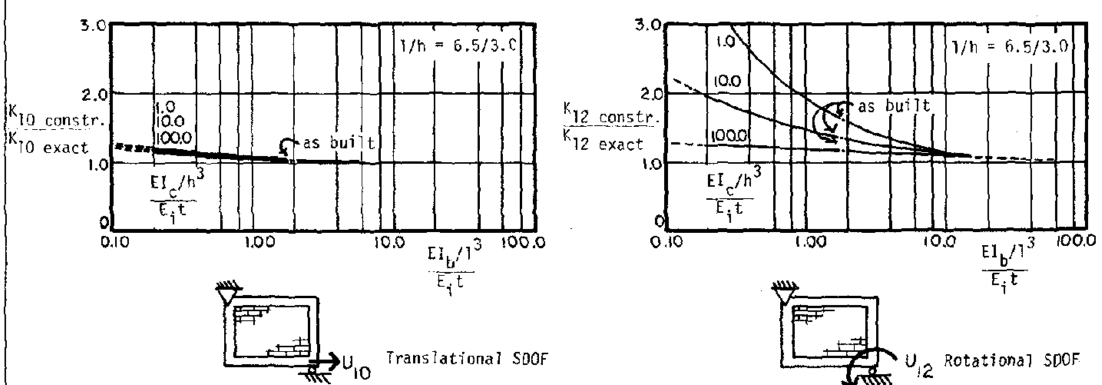


FIG. A3.4.4 ACCURACY OF THE CONSTRAINT APPROACH
NONDIMENSIONAL PARAMETER STUDY OF COMPLETELY INFILLED

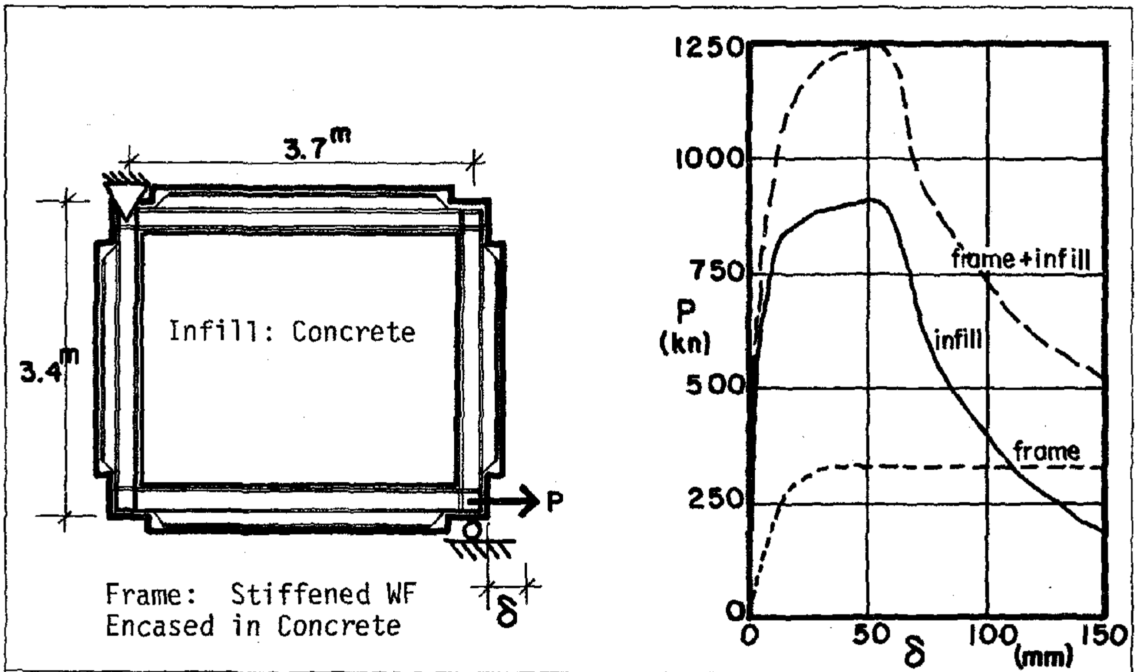


FIG. A3.4.5 BRE FULL SCALE TEST RESULTS

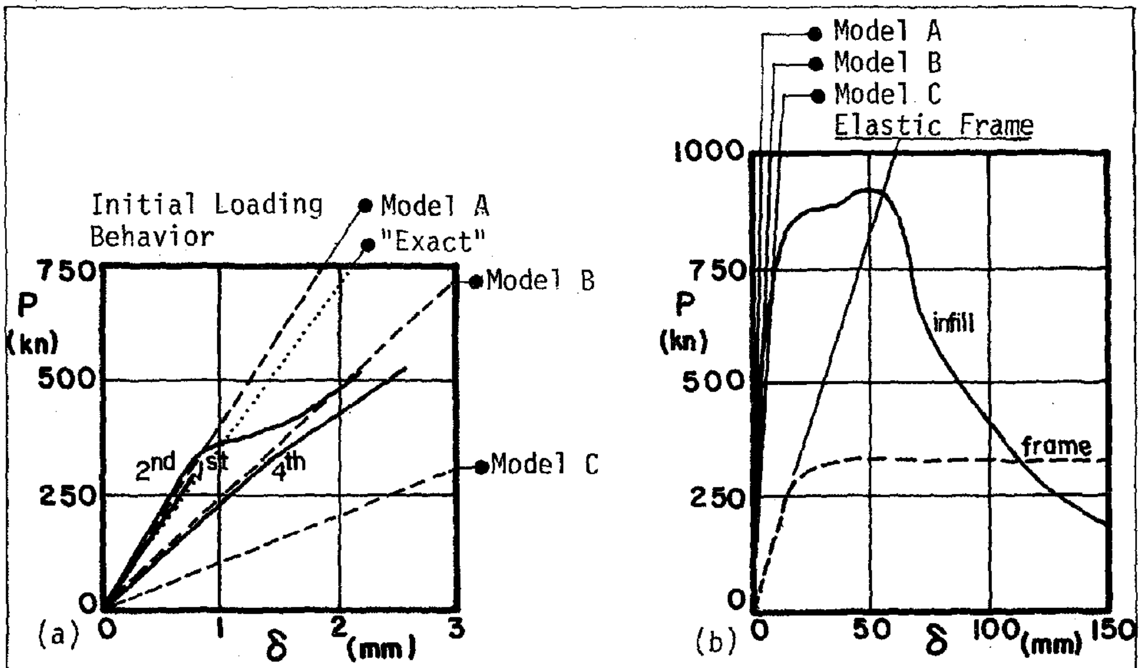


FIG. A3.4.6 ACCURACY OF CONSTRAINT APPROACH
COMPARISON WITH EXPERIMENTAL RESULTS

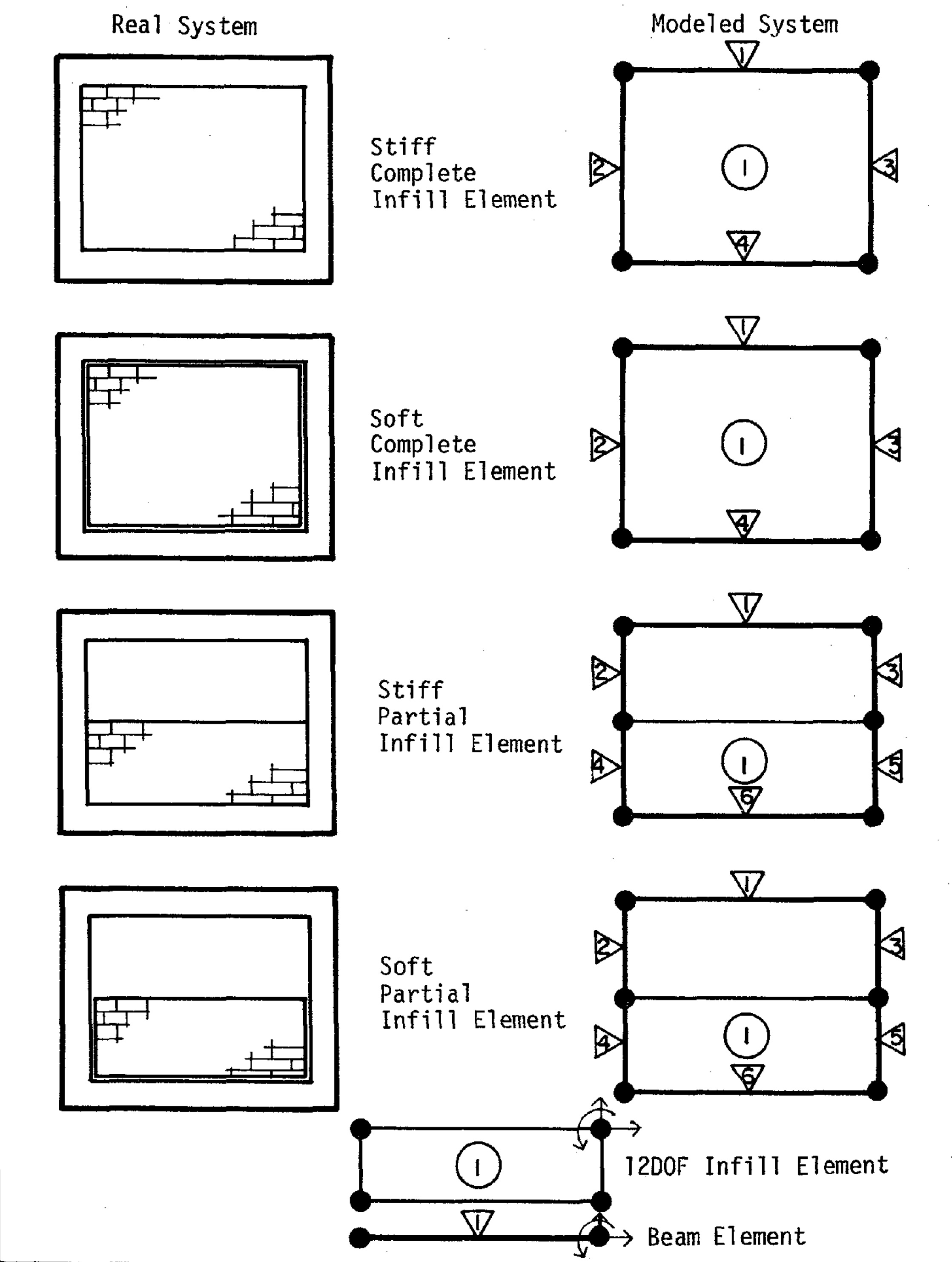


FIG. A4.1 INFILL ELEMENTS DEVELOPED TO DATE

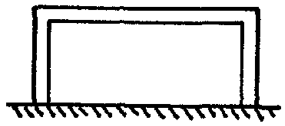
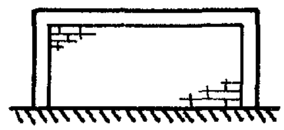
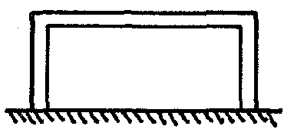
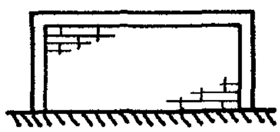
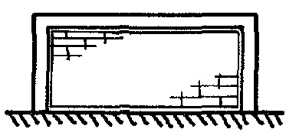
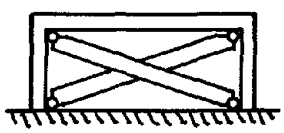
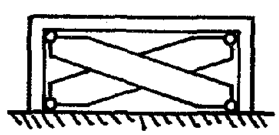
Kost's Result	Model	Periods (sec.) Mode 1	Mode 2	Mode 3
	Kost's Frame	0.113	-----	-----
	Kost's Frame + "Monolithic" Infill	0.0213	-----	-----
Present Study				
	Frame	0.111	0.0116	0.0116
	Frame + Stiff Infill	0.0206	0.0110	0.0105
	Frame + Soft Infill	0.0324	0.0110	0.0109
	Frame + BRE "Equivalent" Struts	0.0578	0.0123	0.0123
	Frame + 10 x BRE "Equivalent" Struts	0.0209	0.0121	0.0117

TABLE A4.1 ACCURACY OF THE APPROXIMATE INFILL ELEMENTS COMPARISON WITH KOST'S ONE STORY EXAMPLE

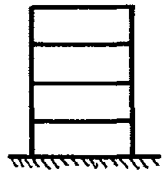
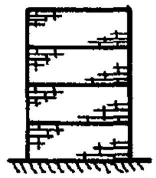
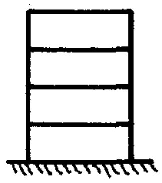
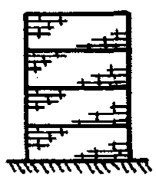
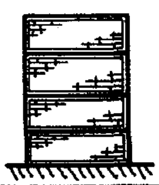
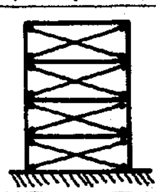
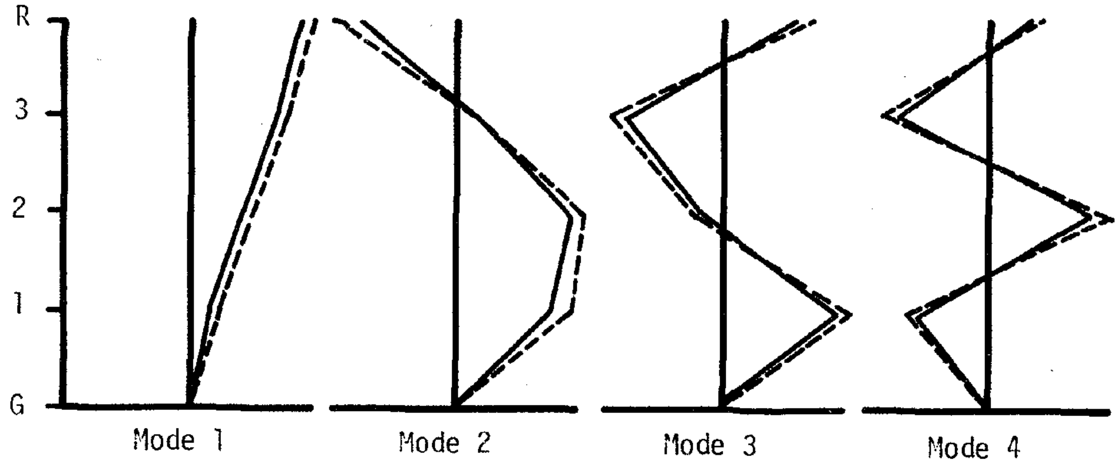
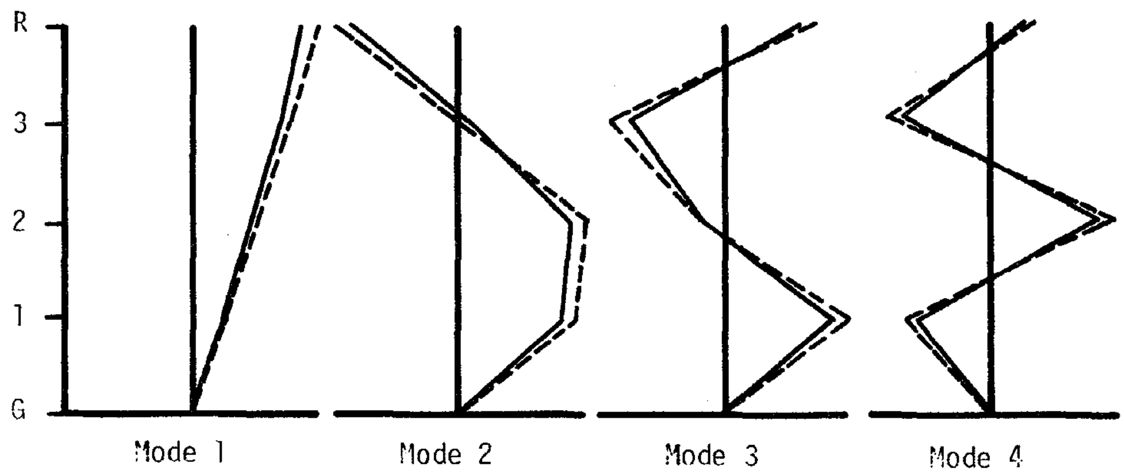
Kost's Results	Model		Periods (sec.)		
		Mode 1	Mode 2	Mode 3	Mode 4
	Kost's Frame	0.449	0.140	0.0773	0.0546
	Kost's Frame + "Monolithic" Infill	0.0848	0.0265	0.0153	0.0122
Present Study					
	Frame	0.443	0.138	0.0757	0.0532
	Frame + Stiff Infill	0.0857	0.0259	0.0146	0.0116
	Frame + Soft Infill	0.120	0.0392	0.0231	0.0180
	Frame + BRE "Equivalent" Struts	0.199	0.0674	0.0414	0.0325

TABLE A4.2 ACCURACY OF THE APPROXIMATE INFILL ELEMENTS COMPARISON WITH KOST'S FOUR STORY EXAMPLE



——— Mode Shapes From Frame + Stiff Infill Model

- - - - Mode Shapes Reported By Kost



——— Mode Shapes From Frame + "Equivalent" Strut Model

- - - - Mode Shapes Reported By Kost

FIG. A4.2 ACCURACY OF THE APPROXIMATE INFILL ELEMENTS
COMPARISON WITH KOST'S FOUR STORY EXAMPLE

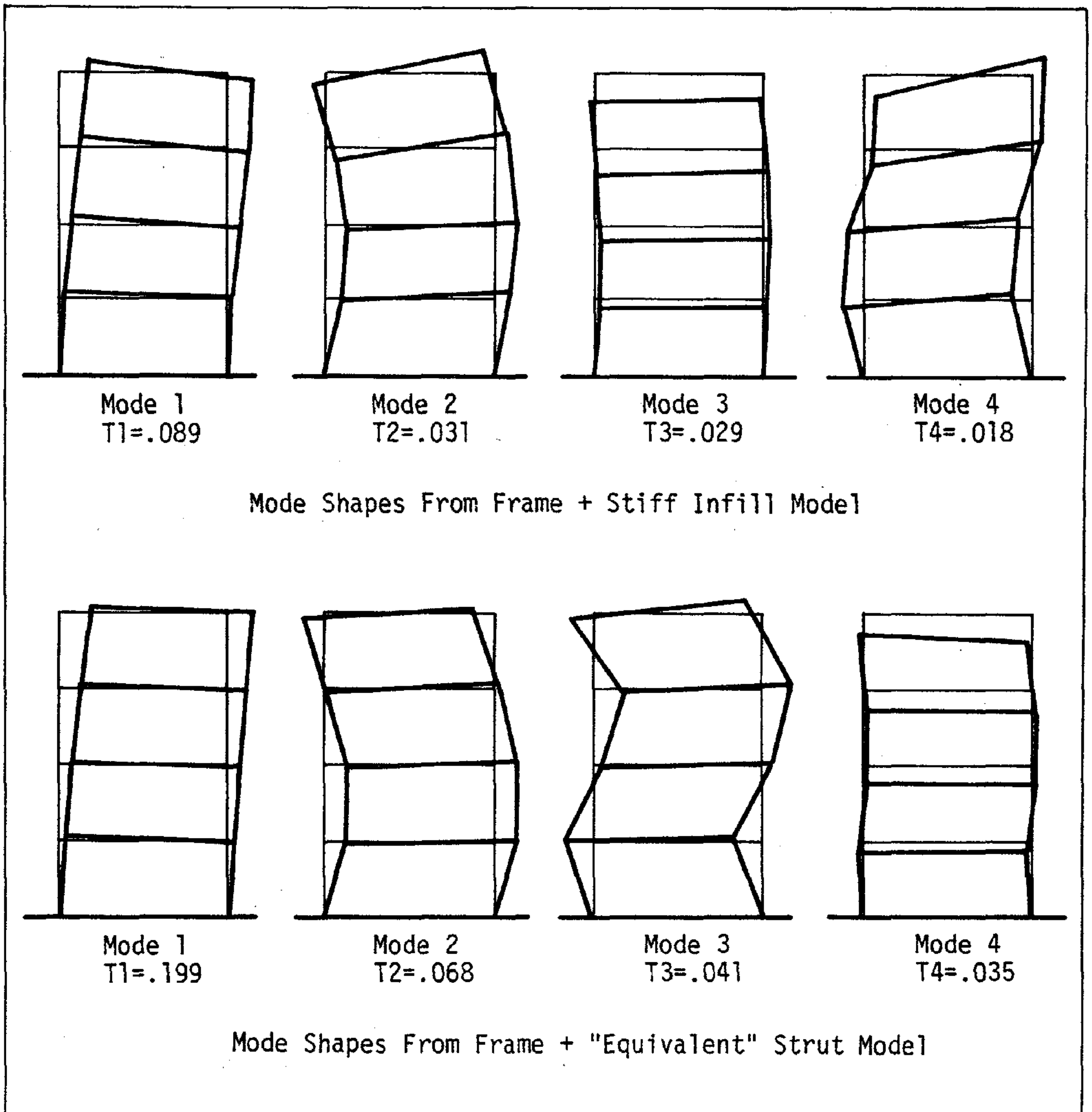


FIG. A4.3 COMPARISON OF MODE SHAPES & PERIODS FOR KOST'S FOUR STORY EXAMPLE INCLUDING VERTICAL DYNAMIC DOF

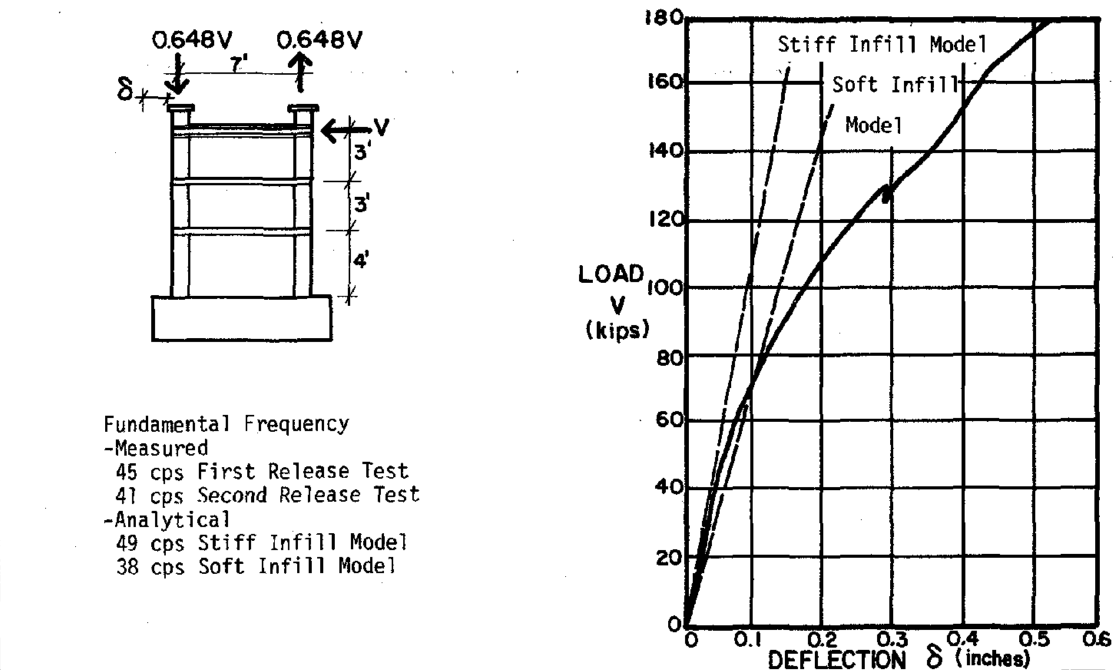


FIG. A4.4 ACCURACY OF THE APPROXIMATE INFILL ELEMENTS COMPARISON TO VALLENAS' STUDY

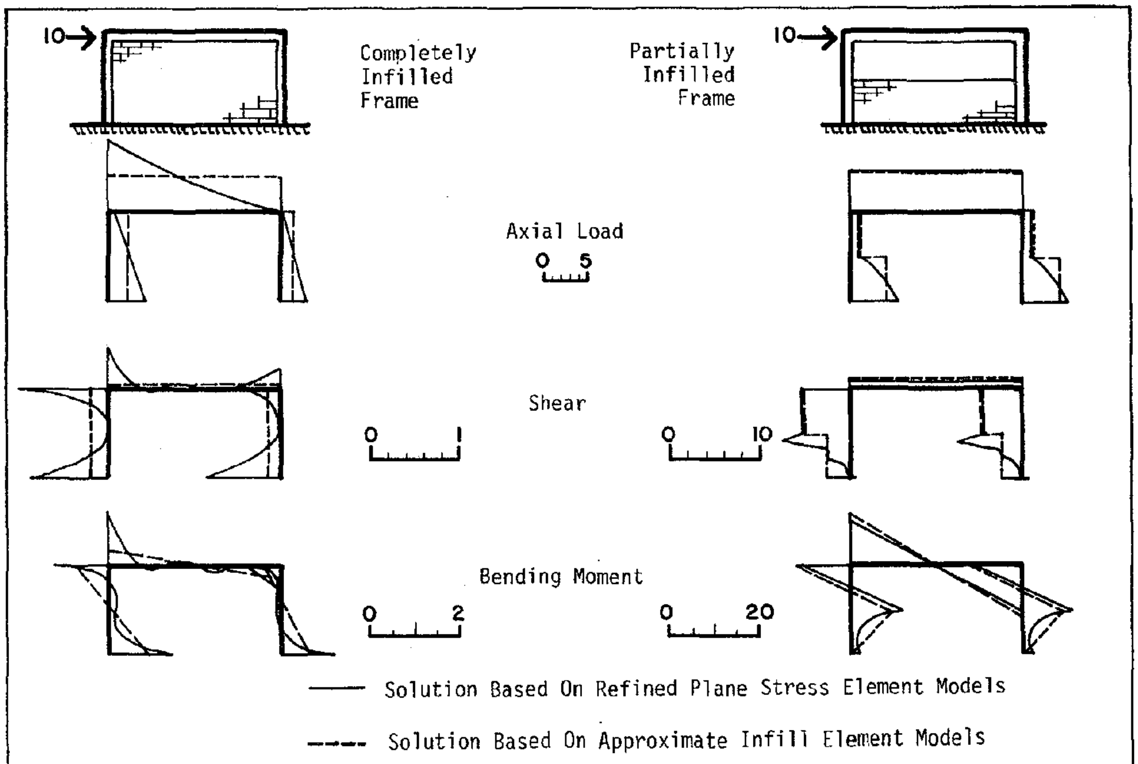


FIG. A5.1 ACCURACY OF THE APPROXIMATE INFILL ELEMENTS MEMBER FORCE EVALUATION

Part B : THE ANALYSIS OF THE ESCUELA DE NIÑERAS

1. Introduction

1.1. General

In Feburary, 1976 Guatemala experienced a major earthquake, two minor but damaging aftershocks and over a thousand lesser aftershocks. These seismic events caused over 24,000 deaths, injuries to over 77,000 people, and widespread, often complete, property damage over an area of some 100,000 square kilometers leaving over one million people homeless and over one billion dollars worth of damage.

The major quake, reportly introduced a day earlier by a single small foreshock [B26], occurred at 3:03 a.m. (local time) Feburary 4. With a surface wave magnitude, M_s, of 7.5 this event ranks among the three most destructive Guatemalan quakes of the past century. Its epicenter has been located some 170 kilometers northeast of Guatemala City on the principal Guatemalan fault, the Motagua Fault (Fig. B1.1) and its focal depth has been determined to be a shallow 5 kilometers. The Motagua Fault, a fault tectonically similar to the San Andreas Fault in California, apparently generated the quake through a left-lateral slip of roughly one meter along its length evidenced by an impressive 230 kilometers of continuous ground breakage. Not since the 1906 San Francisco quake has the western hemisphere experienced such an extensive ground rupture [B4,B8,B9].

The Motagua Fault is the principal fault of the Motagua Fault zone associated with the Motagua Valley that sweeps through the southern part of the country in a long shallow arc southwestward from the Gulf of Honduras turning westward and passing Guatemala City to the north by only 25 kilometers. The Motagua Fault zone is one of three parallel transcurrent fault zones in this area that apparently define the active boundary between the North American and the Caribbean plates. It appears likely that the Motagua Fault may have been responsible for many of the Guatemalan earthquakes of the past two centuries but the evidence is not yet sufficient to free the other transcurrent faults from responsibility [B22,B23,B25].

Several short normal faults are associated with the Motagua Fault that have special importance due to their proximity to the urbanized areas in and about Guatemala City. A zone of these secondary faults located between Guatemala City and Mixco 10 kilometers to the west suffered movement that apparently accommodated part of the lateral movement demanded by the movement of the Motagua Fault during the February events. Of these secondary fault movements the Mixco fault, a 35 kilometer long series of en echelon fault segments aligned in a general northeast-southwest direction, ruptured most dramatically with ruptures over an area 20 kilometers long and up to 8 kilometers wide. The extensive ground breakage associated with the Mixco fault contributed to the damage caused by the principal destructive phenomena, the ground shaking due to the Motagua Fault ruptures [B3,B4,B22].

The two minor quakes were first reported to be a single event centered on the Motagua Fault [B26] but later reports [B21] identified the occurrence of two sequential events centered near Guatemala City. The first and smaller event occurred at 12:11 p.m. (local time) centered 31 kilometers southeast of the city at a depth of 5 kilometers while the second event occurred eight minutes later at 12:19 centered 19 kilometers northwest of the city. Although both events may be defined to be minor in terms of magnitude, 5.0 for the first and about 5.5 for the second, their consequences were not. These quakes were effective in causing additional damage to structures weakened and softened by the major quake of Feburary 4th.

Damage, and hence ground shaking intensity, was greatest along the western end of the Motagua Fault (away from the epicenter of the Feburary 4th quake) immediately northwest of Guatemala City (Fig. B1.2 [B9]). In these areas adobe was the principal building material and as a result building damage was nearly complete. Modified Mercali intensities in these areas appeared to reach IX. Plafker notes;

"Virtually all of the area of major shakng is within 40 kilometers of the Motagua Fault trace and is predominately in areas of thick plestocene pumiceous ash flow deposits that may have amplified ground motions." [B22]

Ground shaking intensities in Guatemal City varied from VI to IX with consderable local variations. The intensity variaton as reported by Espinosa [B9] is reproduced here (Fig. B1.3). Most of the damage reported in Guatemala City resulted from ground shaking, but there was also some damage associated with secondary fault movement and landsliding (in the several deeply cut ravines common to the area) that may not be well correlated with the intensity distribution [B7].

Reconnaisance teams of scientists and engineers arrived soon after the major quake to survey the consequences of the quakes. Although the major portion of building damage was to adobe buildings the reconnaisance teams apparently felt little could be learned from this all too familiar type of damage and proceeded to, instead, survey in detail the damage to engineered structures. In as much as Guatemala City is the country's largest city with a population of over a million that has grown very rapidly over the last twenty five years many of Guatemala's modern engineered structures are to be found in Guatemala City. Most of these structures were designed upon the basis of state-of-the-art seismic desgn practices current to their time, although no earthquake resistant design code was mandatory under Guatemalan law. In recent years the Structural Engineers Association of California (SEAOC) code provided the basic guideline for design and the American Concrete Institute code, ACI 318-63, provided the details needed for the design of reinforced concrete construction.

One of the buildings surveyed at this time, the Escuela De Niñeras*, was selected for the analytical study presented in this section. This building is situated in the northern section of Guatemala City and is used for both classrooms and dormitories for young children. The building suffered extensive damage to windows, doors and masonry walls and partial damage to the

* Escuela De Niñeras - Nursery School

structural reinforced concrete frame. The building is a moderate sized building utilizing a reinforced concrete frame with masonry infill and partition walls. This type of building is common to Central and South America, and not uncommon elsewhere, yet the behavior of this common type of mixed masonry-reinforced concrete system is not yet well understood.

Of the buildings surveyed this building, The Escuela De Niñeras, was selected for further study principally because;

1. It is representative of a generic type of building commonly used throughout the world; a mixed masonry-reinforced concrete frame building of moderate size.

2. It suffered seismic damage that has become characteristic of this genera of building system (eg. "captive" or short column failures and shear failures of masonry infill panels).

3. The building suffered extensive damage yet not total collapse, that is to say it suffered near-collapse leaving sufficient evidence of its behavior to allow the formulation of hypotheses of behavior that could be critically evaluated by analytical studies. The reconnaisance teams were able to document the nature and extent of damage in the building and obtain structural drawings and material properties for the building.

and secondarily, because;

4. The building utilizes generous cantilevers, also common to Central America building design, that were suspected to play an important part in the response of the building to the seismic excitation.

It was felt that this building would satisfy some of the objectives set by the parent study of this project, The Post Earthquake Damage Analysis Project, underwhich this study has been defined.

1.2. Objectives, Methodology, and Scope of the Study

The present study, The Analysis of The Escuela De Niñeras, is one of several in-depth analytical studies undertaken as part of a larger parent project, The Post-Earthquake Damage Analysis Project. This parent project has several objectives;

1. Identification, classification and categorization of characteristic types of seismic damage to both structural and nonstructural building components with an aim to set priorities for further study.

2. Identification of mechanisms responsible for these characteristic types of damage and, when possible, quantification of limit states for the damage.

   The need to extend limit state design for strength and serviceability to include damageability has become more and more apparent since the 1974 SEAOC criteria stipulated that a building should be able to;

   i. resist minor earthquakes without damage,

   ii. resist moderate earthquakes without structural damage but with some nonstructural damage,

   and,

   iii. resist major earthquakes without structural collapse but some structural as well as nonstructural damage.

   To include consideration of damageability in limit state design, damage limit states must necessarily be identified. Practically speaking this will require the identification of response parameters that are well correlated with the type of damage of concern and determination of acceptable limits of these measures of response for appropriate levels of loading. Such response parameters may, most reasonably, be identified through an understanding of the mechanisms responsible for each type of damage of concern.

3. Assessment of the reliability and suitability of use of existing analytical models and techniques for predicting stuctural response, with a particular emphasis on damage response, to earthquake ground motion. This shall include the evaluation of methods to predict gross (global) as well as detailed (local) behavior of the structure.

4. Development of practical analytical models and techniques for the prediction of structural response, including damage response, to earthquake ground motions when deemed necessary. A special emphasis shall be placed on the characterization of the damageability of the building systems.

5. Assessment and or development of means to avoid these characteristic damage hazards through improved design and construction practices.

6. Assessment and/or development of means to strengthen, stiffen or modify existing damaged or undamaged buildings to minimize the risk of damage during future earthquakes.

The methodology used to achieve these objectives involves;

1. Post-earthquake reconnaisance of damaged buildings including detailed documentation of damage to selected buildings and detailed damage, design and construction documentation of buildings deemed worthy of further analytical study.

2. In-depth analysis of selected buildings damaged during recent earthquakes using conventional as well as avant-garde analytical methods.

3. Experimental studies of subassemblages, members and details that have been identified to play an important role in certain characteristic types of damage response.

4. Experimental studies of improved and/or repaired subassemblages, members and details that may serve to abate characteristic types of damage hazards.

As it was clear that the building, The Escuela De Niñeras, would be particularly suitable for future analytical studies the reconnaisance team from U. C. Berkeley's Division of Structural Engineering and Structural Mechanics* carefully documented the location and the nature of the observed damage in the building and discussed and recorded hypotheses of the probable building behavior during the Guatemalan earthquake. Other reconnaisance teams were also to record the damage to this building [B12,B18,B26].

Structural drawings of the building were obtained, reconnaisance reports and photographs of the damage were reviewed and from these objectives were formulated for an in-depth analytical study of the building, the Escuela De Niñeras. The objectives included;

1. A study of the variation of the dynamic characteristics and seismic response of the building due to the effect of the masonry infill used. To this end complete three-dimensional analyses of the building with and without the stiffness contribution of the infill were compared.

2. A study of the importance of vertical motion of beams in the response of the building and on the damage response of the infill panels. (It was recognized that the heavily weighted cantilevered balconies of the upper floors would produce a significant vertical motion response that would be, most certainly, coupled to horizontal ground motion (see Fig. B1.4). A question was therefore raised; Could this coupled vertical response be held responsible for any part of the wall damage or is wall damage due principally to lateral motions?)

* The team included V. Bertero, S. Mahin, S. Sugano, & R. Mayes

3. A critical evaluation of the preliminary hypotheses of building response posed by the reconnaisance teams' reports. An emphasis was placed on damage response.

   These objectives were formulated to provide focus for a general objective;

4. The identification of mechanisms responsible for the observed damage and development of techniques for predicting this damage, possibly through the identification and quantification of damage limit states.

Early on in the study it was decided that available methods used to model the stiffness contribution of masonry infill panels were inadequate for this study and an interim objective was set;

5. The development of a computationally efficient infill element to model the stiffness contribution of complete as well as partial masonry infill panels.

The first section of this study answers this interim objective with the presentation of the development of these needed infill models.

— — — — —

In as much as the infill elements developed for this study are linear elastic elements this study was limited to linear elastic analysis. Complete three-dimensional analysis of models of building were considered in an effort to satisfy the stated objectives. Eigenanalysis and response history analysis for the existing 1976 Guatemala ground motion record as well as the more familiar 1940 El Centro and 1971 Pacoima Dam records were considered.

2. Buiding Description

2.1. Introduction

The Escuela De Niñeras is one of several buildings in a complex of buildings, the Casa Del Niño #1 *, located in the northern part of Guatemala City. The Casa Del Niño #1 provides facilities for some 500 children with roughly 100 children living in and 400 children recieving day care. Of the buildings on the site the Escuela De Niñeras sustained the most significant damage during the Feburary 1976 quakes. Other buildings on the site, including a one story masonry structure, a one story reinforced concrete framed hospital-clinic, and an adobe laundry-kitchen, suffered little damage.

2.2. Physical Description

The Escuela De Niñeras is a three story building with a partial basement. It was designed in 1964, constructed in 1968 and modified in 1975. It is rectangular in plan with a long east-west dimension of 27.5 meters and a short north-south dimension of 11.5 meters (Fig. B2.1, Fig. B2.2). The partial basement is located under the eastern third of the building with direct access to the public streets (the corner of 2a Avenida and 9a Calle). The ground floor level is one story above the street level, the difference of level is provided by massive retaining walls that abut the building frame (Fig. B2.3). Access to the upper two floors is provided by a separate stair tower placed adjacent to the building on its northern side.

The upper two floors are cantilevered out over the ground floor by 2.5 meters on the north and south and 1.25 meters on the east and west providing shading for the ground floor. Large, massive, vertical, reinforced concrete planks are used on the southern facade of the upper floors to provide shading (Fig. B2.1). A great number of masonry walls, many of partial height, are distributed throughout the building in an irregular, nonsymetrical manner. Only

* Casa Del Niño - Children's Home

three nonmasonry walls are found in the building, a small wood wall at the ground floor entry and two reinforced concrete walls in the partial basement area. At the upper levels the eastern and western end walls have been surfaced with a cement plaster and, as a result, have the appearance of concrete, yet they too are masonry.

2.2.1. Structural System

The principal structural system is a reinforced concrete frame supporting two-way slabs and supported on spread footings. There are two longitudinal frames and seven transverse frames (Fig. B2.4 thru B2.8). All columns are 40 by 40 cm in section while all principal beams are 40 cm wide and 50 cm deep. The structural sections (Fig. B2.9) reveal that the floor slabs are placed flush with the tops of the supporting beams in the interior floor areas while they are placed flush with the bottoms of the supporting beams in the cantilever and roof areas. There is no structural floor slab in the western area of the ground floor area, instead a terrazzo finish was placed directly on compacted fill in this area. Under the partial basement the spread footings are linked by tie beams while there are no tie beams used to link the other spread footings of the western end of the building (Fig. B2.5).

The stair tower, designed to be structurally separate from the building, consists of two full height reinforced concrete walls linked by beams and stair landings. A small separation gap was provided between the stair tower and the building to assure structural independence.

In addition to these principal structural elements there are;

1. Reinforced concrete sun shades on the southern facade (Fig. B2.1) that link the second, third and roof slabs together. From the structural drawings it appears that the linkages were intended to be hinged, while inspection of the building suggests that the necessary separation to achieve hinge action was not provided and the linkages may better be considered to be rigid connections. (The sun shades are considered to be structural here simply because they were included in the structural drawings. They will contribute significantly to the stiffness of the stuctural system in that they couple the upper slabs

vertical motion and in this sense may be considered to be structural. Clearly the separation between nonstructural and structural elements is not always obvious.)

2. Two reinforced concrete walls are found in the basement, a north-south wall filling transverse frame 3 and an east-west wall partially filling the first bay of longitudinal frame A. These walls retain earth fill and appear to be well anchored to the beams above and below (ie. tie beams) yet were designed to be separated from adjacent columns by a 4 centimeter gap filled with a rubber gasket.

3. Three principal reinforced concrete retaining walls located at the eastern end of the building in the area of the partial basement were designed to be separated from the structural columns by a 4 centimeter gap filled with a rubber gasket. Their connection to ground floor beams is, however, uncertain but it appears likely that they effectively restrain the structure at the points of contact. These retaining walls are extremely massive and their influence on the structural behavior is believed to be important.

2.2.2. Nonstructural Components

The distinction between nonstructural elements and structural elements in this study is based upon the distinction thought to be made by the designers of the building. The author prefers to distinguish between those components that contribute significantly to the stiffness of the structure and those that do not, the former being, then, nominally structural components and the latter nonstructural. Such a distinction is believed to be more reasonable, yet it is realized that a distinction based upon stiffness contribution is not commonly used nor is it easy to implement as the stiffness contribution of many components is yet uncertain, therefore the more subjective and conventional distinction will be used here.

The following nonstructural components were identified;

Architectural Components

1. Exterior Walls & Interior Partitions - Virtually all walls were constructed of the "tubular" type of brick, a brick with dimensions of 23 x 14 x 6.5 cm. having three tubular perforations in the shortest dimension. These walls were apparently reinforced with single rebars, possibly #3 or #4, placed vertically at the extreme edges of the wall and occaisonally in the center. The walls were either left unfinished, painted white or finished with a "blanqueado" or stucco-like finish. The walls of the northern facade were constructed of two cell, tubular fired clay blocks placed so as to form a screen. One wall at the ground floor entry was constructed of wood.

2. Ceilings - Ceilings appear to have been simply the underside of the above slab finished with a paint.

3. Windows and Doors - Windows were made of aluminum sash and were of three different types; fixed, horizontal casement and jalousie. Door jambs appeared to be steel while most doors were apparently hollow core wooden doors. A wrought iron metal gate was used to separate the upper floors from the ground level.

4. Stairs - The stairs were constructed of reinforced concrete with granite treads. Railings were of wood supported by steel standards.

5. Floors - The floor finish throughout was terrazzo. The cantilevered areas, which had structural slabs flush with the bottoms of the supporting beams, were filled with a local gravel, compacted and the terrazzo was placed on this compacted fill.

6. Finishes - Stucco-like finishes were used on some walls and columns, "martelinado"-a hammered finished stucco on columns and "blanqueado"- a normal finished stucco on walls. Tile was used to finish the exposed vertical faces of the cantilevers.

7. Parapets - Parapets appear to have been used at the edge of the roof but their details are unknown.

Mechanical & Electrical Components

Details of the mechanical and electrical components were not available. Light fixtures appeared to be "bare-bulb" flourescent fixtures at all levels except the newer top level where flush light fixtures were used. Electrical outlets and switches appear to be enclosed in steel boxes connected by steel conduit.

Furnishings

Unfortunately there is no information available on the furnishings that were in the structure at the time of the quakes nor on the behavior and/or damage to these furnishings.

2.3. Design Criteria

The structural design was based upon the provisions of the 1963 American Concrete Institute Code, ACI 318-63, utilizing ultimate strength design procedures. Dead loads and the following live loads were accounted for.

Roof live load	----- 960 Pa	(20 psf)
Room live load	----- 1910 Pa	(40 psf)
Corridor live load	----- 2890 Pa	(60 psf)

It is not known if any seismic loading was considered.

2.4. Material Properties and Quality of Construction

The building materials consisted of concrete, reinforcing steel, and fired clay bricks & mortar.

The design specifications recorded in 1964 on the structural drawings called for concrete having a 28 day cylinder strength of 27.6 MPa, (4,000 psi) and reinforcing steel with a minimum yield strength of 228 MPa (33,000 psi). No material tests were performed to confirm these specifications but examination of the damaged columns revealed deformed round reinforcing bars that may have been of greater strength than specified. The relatively low strength rebar specification must be considered suspect as such bars are relatively rare presently and were probably uncommon at the time of construction, furthermore, such low strength bars were commonly supplied as square undeformed bars. It was concluded, then, that the presence of deformed round bars in the damaged columns indicated a more typical reinforcement may have actually been used in the construction of the building, grade 40 with a yield strength of 276 Pa (40 ksi) is the likely choice.

The material properties of the masonry were not specified on the 1964 structral drawings. It is likely that the masonry was considered to be nonstructural and simply not considered at all structurally. Tests of local Guatemalan masonry walls have, however, been reported, [B10], that indicate the likely material properties of the masonry used in the Escuela De Niñeras. Direct compression tests of full scale walls constructed of 23 x 14 x 6.5 cm "tubular" bricks indicate;

1. ultimate crushing strengths of from 1.75 to 2.25 MPa (250 to 325 psi)
2. initial elastic stiffnesses of from 680 to 3140 MPa (98.6 to 455 ksi)

   and,

3. elastic limits of from 0.40 to 0.78 MPa (58 to 114 psi)

based upon the gross cross section of the walls tested. The lower values correspond to walls constructed with low strength mortars and the higher values to walls constructed of higher strength mortar.

The strength and stiffness uncertainties indicated here is typical of masonry construction. An estimation of the stiffness contribution of masonry infill to a reinforced concrete frame must, necessarily, be limited by this uncertainty. It is unreasonable, then, to attempt to define a very exact model of the infill stiffness contribution as the uncertainty of the material does not allow exactness. It is believed that the infill element presented in the first section of this report provides a reasonable degree of accuracy in the modeling of this stiffness contribution by capturing, albeit approximately, the complete form of the displacement field of the infill-frame system.

Although the building appeared to be well crafted the details of the construction were not well designed from a seismic point of view. Column steel was tied with widely spaced (40 cm) lightweight rebar that could not be expected to provide much additional shear strength and confinement of the concrete core. The masonry walls were essentially unreinforced and could only be expected to behave brittly if loaded to capacity. The cantilever portions of the building were heavily loaded by masonry walls and sun shading devices in addition to the exceptionally massive floor construction of compacted gravel and terrazzo. In short there appeared to be no attempt to minimize the mass of the building nor to provide construction details that would develop the potential strength and ductility of the structure.

2.5. Site Conditions

Design soil pressures were noted on the stuctural drawings that indicated nonuniform soil conditions across the site with a low of 120 kPa (2,500 psf) to a high of 144 kPa (3,000 psf). The details of the soil at the site were not available but Sozen has characterized the local condition;

> "Guatemala City is located on a plateau surrounded by mountains, including four active volcanoes, and serrated by very deep ravines. The basic soil is made of volcanic ash and pumice, down to at least 100m. The top 8 to 15 m are the result of weathering of the volcanic ash, with the expected gradual transition from clay (of a brownish color) and sometimes organic silts (dark) to silts (yellowish to red) silty sands (yellowish, light brown or white) and sands (gray, pink or beige).
>
> While this profile is typical, the actual soil properties, degree of plasticity and water content change appreciably in different parts of the town. Depth to dense sand may vary from 1 or 2 m to 8 or 9 m ..."[B26]

The nonuniform design bearing pressures apparently reflect the variability of the upper soil layer typical of the city.

2.6. Concluding Remarks

The Escuela De Niñeras is very much like many school buildings used throughout the world. It is a moderately sized building with a reasonably straightforward and typical building system. It is built of 'better' materials, with care and attention to durability. Enclosure and partition walls are built of masonry that is used in a direct manner (from a construction point of view) providing economy as well as the acoustical insulation necessary in schools. Yet this sensibly designed, and not inexpensive, building sustained more damage than other buildings at the site that would normally be considered to be of lower quality.

It is likely that the designers of the building were very conscientous in their attempt to produce a well designed building yet they apparently did not fully appreciate the seismic consequences of their design choices. Indeed, the seismic behavior of this common type of mixed masonry-concrete building system is not yet well understood. This study attempts to provide some of this understanding so that designers, working in this system, may better anticipate the seismic consequences of their design choices and thereby achieve maximum utility of this building system that offers construction and use advantages and economies.

3. Damage

3.1. Introduction

Although damage is of principal concern here, the structural designer should properly consider life hazard to be of greater priority than damage. The damage to the Escuela De Niñeras was extensive, dramatic, and conspicous yet an account of the events of the night of Feburary 4, 1976 reveals the pivotal importance that one building component can have in affecting life safety.

At the time of the principal shock, 3:00 am Feburary 4th, the children who live-in were sleeping in the dormitory areas of the Escuela De Niñeras on the upper floors. A steel gate isolating these upper floors from the ground level became jambed at this time and as a result the children could not be immediately evacuated from the building.

It is likely that most of the damage to the building resulted from this principal shock bringing the building nearly to collapse. The lesson is clear; the importance of a seemly minor building component, the steel gate, can be seen to be potentially very great in terms of life safety while relatively insignificant in terms of cost and structural importance.

3.2. Structural Damage

Structural damage was limited, principally to the first floor columns, especially in the area of the transverse frame lines 2 and 3. The column damage was of two types;

1. Shear-type failures, especially in short columns ("captive" columns) characterized by diagonal cracks, spalling, buckled reinforcement and in one case a ruptured tie (Fig. B3.1).
2. Bending-type failures resulting in spalling and crushing of concrete near the top of columns (Fig. B3.2).

At the eastern end of the building beams and columns suffered superficial damage - spalling of the "martelinado" stucco surface finish. There was also some evidence of pounding between the building and the stair tower. The resulting damage was superficial and the residual separation slight. No damage was observed in the floor system.

3.3. Nonstructural Damage

Much of the damage found in the building was, nominally, nonstructural damage or due to the constraint offered by nonstructural components. By far the most conspicuous damage was the damage to masonry walls. Damage to these walls was chacterized by;

1. diagonal shear cracks,
2. horizontal cracking,
3. horizontal crushing,

and,

4. complete collapse/explosion of the wall (Fig. B3.3).

Although the walls failures were most dramatic other nonstructural damage was apparent;

1. Door jambs and doors were damaged by movements of the walls.
2. Many windows were broken.
3. The stairway connecting the basement with the ground floor level was only superficially damaged but was blocked by debris from collapsing walls.
4. The steel gate isolating the upperfloors from the ground level was jambed during the principal February 4th quake.
5. The presence of electrical conduit and outlet boxes appeared to have weakened at least one of the walls, evidenced by cracking.
6. The damage to flourescent lights was most interesting. It appears that flourescent tubes fell from their fixtures in the area, on the ground floor, of greatest damage. In this area one complete light fixture fell, while tubes and fixtures appeared to be intact in other areas of the building. In all instances the light fixtures were attached to overhead slabs.

7. Some secondary column-like members used to stiffen and tie brick walls to the structural frame suffered short column type failures (eg. X cracking).

3.4. Concluding Remarks

The damage of principal interest here - damage to walls and the structural frame - was carefully surveyed by the U.C. Berkeley's reconnaisance team. They documented the location and degree of damage to columns and masonry walls (Fig. B3.4) and formulated preliminary hypotheses of the response of the building to the shock. The degree of damage to each damaged element was assessed subjectively and assigned qualitative values of;

N.D. - no damage,

S.D. - slight damage,

P.D. - partial damage, and

B.D. - badly damaged.

At this time the reconnaisance team discussed possible mechanisms of failure and recorded;

"The main reason for the damage was;

1. Nonuniform distribution of masonry walls with plan and elevation and,
2. the inadequate shear reinforcement of the columns."

The eccentric distribution of walls, especially on the first floor, was thought to introduce significant torsional response resulting in the observed failures. It was hypothesized that the failures of the masonry infill preceeded and lead to the shear failures of the columns, this conclusion seems to be supported by the elastic analyses that follow.

In terms of quantity and cost the damage to the masonry walls was clearly most significant. In terms of life hazard, however, the jambed steel gate was most important. The debris on the lower stairway, while not very important in this building, has become recognized as a typical type of life hazard in buildings of this type. The damage to the structural elements was not very extensive but was, nevertheless, very serious. It is not an exaggeration to conclude that the building came very close to the collapse of the ground floor level.

4. Ground Motion

4.1. Introduction

The damage to the Escuela De Niñeras resulted from the seismic events of February 1976 and their interaction with the building system. The certainty with which the response of this building may be predicted is thus limited by both the knowledge of the building's dynamic character and the knowledge of the February seismic events. Conversely, the uncertainty in predicting the building response results from both the uncertainty of modeling the structure and of modeling the loads. The damage survey allows an observational, rather than experimental, point of view against which the success of modeling the building's response may be judged. The principal focus of this study relates to the modeling of the structure, yet the success with which this modeling is achieved may only be evaluated in the context of the load models used.

The general nature of the February seismic events and their relationship to the geology and seismology of the area have been discussed, briefly, above (section 1.1). The single available ground motion record of these events, its uncertainty and suitability to the analysis of the Escuela De Niñeras will be considered in this section. The dynamic character of the building, its uncertainty and modeling will be considered in the subsequent section setting the stage, finally, for a comparison of the predicted response and the observed damage in the context of the uncertainties of the loading, of the dynamic character of the building and of the damage recorded.

4.2. Characteristics of the Feburary 4, 1976 Record

Guatemala City is located in a region of high seismicity with a recorded history of earthquakes dating back to 1530. Destructive earthquakes have been a familiar part of Guatemala City's history, yet no strong motion accelograph was operative at the time of the Feburary 4, 1976 shock. One seismoscope record was, however, obtained and ground motion intensities were estimated from the observed damage. The seismoscope, apparently a Wilmot instrument, was located on the ground floor of the Administration Building of the San Carlos University - a technical institution. This building is situated in the southern part of the city some 18 km southwest of the Escuela De Niñeras site. The ground shaking intensities in this area were, apparently, less severe and possibly of a different nature than those at the site, nevertheless this record was used to characterize the ground motion at the site as it was felt that the frequency content, the intensity, and the duration of the two sites would not be unreasonably dissimiliar. Furthermore, no better record was available.

The original seismoscope record (S/N 189), the trace of a stylus scratching on smoked glass, is reproduced here (Fig. B4.1) to indicate the nature of such a record and consequently, by implication, the uncertainty in the ground motion time histories obtained from it. Ground acceleration time histories were recovered from this record (Fig. B4.2) for both north-south and east-west components [B19]. Unfortunately the seismoscope trace was indistinct and could not be followed continuously. Two separate continuous traces could be read and thus two separate ground acceleration time histories were recovered from the record. Although the trace between sequences could not be followed it was apparently clear that sequence I was first in time.

It can be seen that several peak values of the ground motion record used fall in the 0.20g to 0.30g range, as may be expected from the reported ground shaking intensity of VIII. There is, however, a single spike reaching 0.60g in sequence II that may or maynot be "real" but in any event this peak has very little energy associated with it. Ground acceleration time histories of the 1940 El Centro record and the 1971 Pacoima Dam record (the measured, not the derived) scaled to have peak accelerations of 0.50g are also presented for comparison (Fig. B4.2).

Single degree of freedom elastic response spectra were generated using sequence I alone (0% damping), sequence II alone (0% damping), and sequence I followed immediately by sequence II (0% and 5% damping) for both the north-south and east-west components (Fig. B4.3). In as much as the Guatemala records were incomplete and, hence, rather short (sequence I had a duration of 1.8 seconds and sequence II 6.9 seconds) the longer period, lower frequency, end of the response spectra would be expected to be misrepresentative of the actual event. The periods of concern here for the rather stiff Escuela De Niñeras will be less than 1.0 second and consequently attention should be directed toward the more meaningful shorter period, higher frequency, end of the response spectra.

It can be seen that the SDOF response spectra for these three cases - sequence I, sequence II & sequence I+II - are significanly different in amplitude although similar in form, with the exception of the condition at a period of 0.40 second. There is, then, some uncertainty in the ground motion characteristics due to the uncertainty of interpretation of the seismoscope record in addition to the uncertainty due to the differing locations of the seismoscope and the building under study. The former uncertainty may be put into perspective by noting that it is reasonably well bounded by the 0% and 5% damped spectra for the combined sequence I+II. The uncertainty in the ground motion resulting from the indistinct seismoscope trace is roughly equivalent to an uncertainty in structural damping of + or - 2.5% (in the shorter period range of interest here).

A comparison of the SDOF elastic response spectra for the Guatemalan record to the 0.50g El Centro record and 0.50g Pacoima Dam record provides some additional insight into the severity and nature of the Guatemalan event (Fig. B4.4). In an approximate sense the Guatemalan quake is comparable to a 0.50g Pacoima Dam quake yet less severe than a 0.50g El Centro quake in the period ranges of 0.10 to 0.20 seconds and 0.60 to 1.00 seconds. In between these ranges the Guatemalan quake is less severe than both of the 0.50g Pacoima Dam and the 0.50g El Centro. It will be seen that the masonry infill used in the Escuela De Niñeras tended to shift the response of the building into the lower period range.

The three ground motion records were used in the analytical studies of the building to investigate both the sensitivity of the building to vertical ground motion (no vertical record exists for the Guatemalan quake) and to ground motions of differing frequency content. The El Centro and Pacoima Dam ground acceleration records were scaled so that the SDOF elastic response spectra of these two records and the Guatemala record would be of, roughly, similar amplitude in the frequency range of interest here. To achieve this equivalency the El Centro and Pacoima Dam records were scaled to have peak accelerations of 0.5g while it will be recalled that the Guatemala event had a peak of 0.6g. Only the first 10 seconds of the El Centro and Pacoima Dam record were used to obtain similar duration to the Guatemala record, leaving, hopefully, a comparison of only the difference of frequency content and vertical accelerations.

5. Elastic Analyses

5.1. Introduction

The general dynamic characteristics of the Escuela De Niñeras and its response to the three ground motion records discussed above will be considered in this chapter utilizing four different elastic models of the building. The models were selected to study the variation of the dynamic characteristics of the building due, principally, to the influence of the infill panels and also due to the influence of the floor slabs. The limitations of elastic analyses are well known yet the results of elastic analyses present a reasonably good estimation of the initial dynamic response of buildings and thereby indicate probable areas of early inelastic response. Combined with the observed damage patterns, the elastic analyses of the Escuela De Niñeras suggest possible modes of failure of the critical columns and infill panels. From an understanding of the mechanisms responsible for this damage, means to mitigate or avoid these damage hazards may be developed.

The elastic analyses of the Escuela De Niñeras also provided a means to investigate the suitability of the infill elements, presented in the first part of this report, to model the stiffness contribution of infill and hence to predict damage. The developed elements were incorporated into an existing elastic analysis program, SAP IV, (developed at U.C. Berkeley by Bathe, Wilson and Peterson [B2]), and the modified program was used for all of the analyses. Some mild limitations of the infill elements were identified, related to their inherent numerical approximation error (see Appendix), and their value in predicting the degree of damage to infill panels was evaluated.

The accuracy of an elastic analysis of the response of a building that clearly suffered significant inelastic behavior is limited by the accuracy of the theoretical assumptions of the theory underlying the elastic analysis, this much is obvious. The accuracy is also limited by uncertainties in material properties and construction, that lead to uncertainties in modeling the stiffness, mass and damping of the system, as well as uncertainties in the nature of the ground excitation. The uncertainty in the material properties, especially the masonry material stiffness characteristics, and the uncertainty of the Guatemalan ground motion have been discussed above. Additional uncertainties in the estimation of the mass and stiffness of the structure will be noted subsequently. The superposition of all these sources of uncertainty do not, however, render the analysis meaningless, rather, they indicate that exact analysis is not possible in this case and therefore any analysis, elastic or inelastic, can at best provide only a sense of scale to the problem. That is to say, any analysis can only be expected to give a general sense of the building's real behavior and point to probable deficiencies of the buiding system. With the sources of uncertainty in mind it is natural to consider the sensitivity of the elastic system's behavior to a perturbation (ie. an uncertainty) of the system's characteristics, of ground motion characteristics or a variation of theoretical assumption. The computational effort necessary to conduct such sensitivity studies (parameter studies) will often be prohibitive and the analyst may then seek to limit inquiry to limiting values of the system's characteristics, the load's characteristics or theoretical assumptions. That is to say the analyst may seek to "bound" the behavior of the real system by considering limiting cases of behavior of the modeled system.

For example, if it is known that the uncertainty in a given system response parameter, δr_1, (eg. maximum roof displacement) is dependent upon the uncertainty in one parameter that relates to the character of the building system, δb_1, and another uncertainty of another parameter that relates to the character of the load, δl_1, while all other system parameters and load parameters are certain. Then, in general, to completely define the sensitivity of the uncertain response parameter to the variation (perturbation) of the system parameter and load parameter one must seek a governing relation that would define a surface*;

$$\delta r_1 = f(\delta b_1, \delta l_1).$$

* In some instances one may be able to define the degree of this uncertainty by other means. For example, perturbation analysis of the general structural stiffness system of equations may be used to show that the uncertainty in a single displacement response parameter will be relatively smaller than the uncertainty in the stiffness parameters if the system is relatively stiff (ie. the stiffness relation is a convergent mapping from loads to displacements when the stiffness matrix is stiff).

The analytical effort needed to determine this relation, empirically or theoretically, will most often be prohibitive and the analyst will opt to consider a few limiting cases of the system and load parameters, (eg. $b_1 \pm \max\delta b_1$ and $l_1 \pm \max\delta l_1$) that will hopefully result in limiting cases of the system response parameter (ie $r_1 \pm \max\delta r_1$).

In as much as the sources of uncertainty are multiple only the most pertinent bound studies may practically be considered. The selection of the studies to be considered and the assignment of limiting values requires sound judgement. It must be recognized that the consideration of extreme values of a given parameter may not lead to limiting cases of system behavior [B11].

The limiting cases considered in this study were;

1. Comparisons between models of the building with and without the infill contribution and models of the building with and without the slab contribution were considered to evaluate the system's sensitivity to these structural model variables.

2. A comparison between models using the stiff infill constraint assumption and the soft infill constraint assumption was considered to evaluate the system's sensitivity to the nature of the basic theoretical assumption of constraint conditions.

and,

3. Comparisons of the several models' response to base excitation of different frequency content and to base excitation with and without vertical components included were considered to evaluate the system's sensitivity to load modeling (ie. input excitation uncertainty).

5.2. The Computer Program

An existing and well known general purpose program, SAP IV, was modified and used for all elastic analyses. SAP IV is a structural analysis program designed for both static and dynamic analyses of linear structural systems.

The version of SAP IV used in this study has a finite element library of nine elements including;

1. three dimensional truss elements,

2. three dimensional beam elements (with provision for the inclusion of shear deformations),

3. plane stress and plane strain elements,

4. two dimensional axisymetric solid elements,

5. three dimensional solid elements,

6. variable-number-nodes thick shell and three dimensional elements,

7. thin plate or thin shell elements,

8. boundary spring elements,

and

9. pipe elements.

To this library was added the;

10. infill elements

presented in the first section of this report. All of these elements may be used in static or dynamic anayses. Each nodal point in the system may have from zero to six displacement degrees of freedom allowing complete three dimensional analysis capabilities.

The structural matrices are stored efficiently in condensed form, using blocked storage for larger systems so that out of core storage may be used effectively. In static analysis the equations of equillibrium are solved and then element forces (stresses) are determined. In dynamic analysis there exist a choice between;

1. eigenanalysis only,

2. eigenanalysis followed by response history analysis by direct integration of the uncoupled system equations,

3. eigenanalysis followed by response spectrum analysis,

or

4. response history analysis by direct integration of the system equations.

Selected response histories of nodal displacements and member forces (stresses) will then be computed upon request. The program was modified slightly to obtain other response parameter histories including story drifts and nominal infill panel shear stresses.

For the larger systems both static and dynamic solutions are carried out on a block by block basis sequentially removing and placing blocks on out-of-core memory thereby avoiding the need for large in-core memory. The eigenanalysis for smaller systems is achieved by determinate search while for larger systems subspace iteration is used. The models of the Escuela De Niñeras were all larger systems.

There are no special assumptions or limitations beyond the usual assumptions and limitation of linear elastic finite element analysis imposed by the program, although a lumped mass matrix (diagonal mass matrix) is assumed and damping is assumed to be identical in all modes in analysis by mode superposition while in direct integration analysis Rayleigh damping is employed. The program has no special provision for the consideration of rigid finite sized joints (non-centerline geometry), although it would be possible to simulate rigid joints (or semi-rigid joints) with some difficulty and loss of computational efficiency. Complete three dimensional ground motion accelerations may be used.

Rayleigh damping allows easy generation of an orthogonal damping matrix (ie. a modeling of damping that allows the system's equations to be uncoupled via eigenanalysis) useful for direct integration procedures, but does not allow constant damping in all modes. Ordinarily damping is similar in all modes of building systems [B20] and therefore the damping provided by SAP IV, in the modal superposition option, may normally be preferred. In exceptional structures the damping may increase or decrease with higher modes and in these cases Raleigh damping may be better suited. (Infact, damping may usually vary with the strain level experienced and the loading history, but these nonlinear affects will not be considered here.)

Raleigh damping also offers the advantage of the possibility of "artificially" damping (filtering) out the higher modes that are less likely to be accurately modeled than the lower modes that often do not participate significantly in the response of the system.

5.3. Finite Element Idealization of the Structure

A series of four three-dimensional models of the Escuela De Niñeras was considered to investigate the influence of both the infill masonry panels and the floor slabs upon, first, the dynamic characteristics of the system and, secondly, the response of the building to the ground motion records discussed above. The formulation of these models involved;

1. Identification of building components thought to contribute significantly to the stiffness/resistance of the building. This set of components may be considered to constitute the "real" stuctural system as opposed to the nominal stuctural system that may be found in the structural drawings of the building.
2. Idealization of the "real" structural system by an assembly of finite elements, thought to represent the behavior of the "real" elements, connected at selected structural nodes.
3. Evaluation of the distribution of mass throughout the structure and idealization of this mass distribution in terms of lumped masses placed at the nodes and estimation of energy dissipation potential, nominally, through damping.

Building components thought to contribute significantly to the stiffness/resistance of the structure included columns, beams, floor slabs, masonry infill walls and other masonry walls, retaining walls, stairways and the massive southern sun shades. Although not normally considered to be a building component, soil may be included in this list.

Finite elements were selected to represent all of these "real" system components but two, the masonry walls that did not infill frames and stairways, as their contribution was thought to be relatively unimportant and no practically reasonable elements are available to model these two components. The finite element idealization was based upon centerline geometry with structural nodes located at the intersection of the real system beam-column joints and the ends of cantilevered beams (Fig. B5.1). Additional nodes were introduced, as necessary, to define partially infilled frames. Three-dimensional beam elements, plate elements, boundary elements (springs), and infill elements were used. Some of the details, uncertainties and assumption made in the idealization of individual components will be discussed below.

5.3.1. Columns and Beams

Column and beam components were idealized by the three-dimensional homogeneous, isotropic, linear elastic beam elements of uniform prismatic section. Flexural, shear and torsional deformations were accounted for. Section properties for each component were based upon gross section geometry and element stiffnesses were formulated based upon centerline geometry assuming rigid connections of beam and column line elements at the (inifinitesimally small) joints or nodes. The material stiffness (Young's modulus) of these elements was taken equal to the material stiffness of the concrete alone.

Clearly there are many simplifications made in this idealization. Reinforced concrete beams and columns are non-homogeneous (ie. composite), non-isotropic components whose effective section properties depend upon the loading (eg. degree of cracking), the loading history, and the amount and distribution of steel at each section and therefore, in general, vary along the length of the member in a nonuniform manner. The relatively greater rigidity of the beam-column joint (the panel zone) can significantly affect the behavior of the component. Nevertheless the simplifications assumed here are often a practical necessity that result in a reasonable distribution of beam and column stiffnesses, in a relative sense, that provide a practically accurate evaluation of the initial stiffness and member forces of reinforced concrete frame systems. It also appears, from the preliminary studies presented in the first part of this report, that such simplifications will also provide practically accurate evaluation of the initial stiffness and member forces of reinforced concrete frame-infill systems.

One of the objectives of this study was aimed at the development of a practical means to model the behavior of frame-infill systems. Practicality necessarily involves a compromise between accuracy on the one hand and convenience on the other and is, therefore, clearly subjective. What may be practical to one engineer may be too theoretical to another, yet too approximate to a third.

5.3.2. Floor Slabs

Floor slabs were idealized by the thin plate elements available in SAP IV. The plate element is kinematically compatible with beam elements and should, therefore, be able to capture, in a least-energy-best-fit sense, the complete three dimensional behavior of the slab. The plate element has five degrees of freedom per node, three translational and two rotational, (the rotational degree of freedom normal to a flat plate is undefined) or twenty degrees of freedom per element.

In practice floor slabs are most often modeled as rigid diaphragms, less often modeled as flexible diaphragms and occaisonally modeled as elastic diaphragms in an attempt to more accurately model the floor slabs' contribution to the system behavior. The elastic modeling is conventionally achieved through the use of either "equivalent" beams or plate elements, the latter limited to three dimensional analysis. This more accurate modeling option is often avoided in practice as it usually involves a greater computational cost than either the rigid or flexible diaphragm approximations and it is commonly believed that the real behavior of a given building system will be bounded by the behavior of these approximate, but computationally attractive idealizations. These approximate idealizations may not, infact, bound the behavior of the real system [B11]. Furthermore, the use of the more exact plate elements need not lead to unreasonable computational expense.

Plate elements were used in lieu of "equivalent" beam elements or the slightly more generalized "equivalent" beams offered by the Stiffness Matrix Method [B6]. "Equivalent" beams (ie. floor beams stiffened in relation to some measure of slab stiffness and geometry) ignore the coupling of diagonally opposed degrees of freedom of slab panels altogether and are based upon limited definition of equivalency. These equivalent members were originally developed, by parameter studies, to replace slab-beam systems so that the "equivalent" beam system would have a lateral stiffness equal to that of the slab-beam system. Consideration of the more general three-dimensional deformation of slab-beam system (eg. torsion, warping and in-plane distortion of the slab) was apparently not considered. Such consideration would have added unwanted complexity to the already difficult task of defining "equivalent" beams by parameter studies. (The Stiffness Matrix Method seeks equivalency in a slightly different sense by attempting to match the added rotational stiffness offered by the slab to the beam column joints. Again, however, the equivalency is limited (eg. no diagonal coupling considered) eventhough it is based upon a comprehensive but nevertheless limited parameter study of 122 different slab-frame systems.)

The shortcomings of these "equivalent" beam approaches are multiple. First, being based upon parameter studies it is difficult to critically review their developement and to critically assess their suitability. Parameter studies do not offer the generality nor lend themselves to critical evaluation that a more theoretical approach can offer; one will always wonder if one would draw the same conclusion from a given parameter study that another investigator would. Secondly the sense of equivalency sought was necessarily limited. Thirdly, the "equivalent" beam approach seeks to replace a three dimensional element by a set of one dimensional elements and in so doing looses much of the nature of the more complex element (eg. the number and the coupling of the degrees of freedom considered and the meaning of member force evaluation). The plate element seeks equivalency in the more general sense (with a theoretical rather than empirical basis) of approximating the displacement field and in so doing achieves a modeling of stiffness contribution and member force evaluation as well. Finally, it is simply more convenient to use plate elements (in 3D elastic analysis) to gain, at least, a first order estimate of the influence of floor slabs upon building response.

Each floor slab was subdivided into a mesh of plate elements defined by the structural grid of the floor beams (ie. with single plate elements of either 5.0 x 6.5 m or 5.0 x 2.5 m). The use of such large plate elements would normally be considered unacceptable in plate and thin shell analysis yet in building analysis they offer a more exact representation of the slabs stiffness contribution than that to be expected from "equivalent" beams, without increasing the number of degrees of freedom necessary to model the system beyond that of the frame alone. If the analyst is interested in the detailed behavior of the beam-slab system a finer mesh would be required. If, however, the principal interest is in building behavior, in the global sense, with some reasonable estimation of beam and column member forces then the use of single-panel plate elements offers a practically attractive compromise.

The plate element used was a quadrilateral of arbritrary geometry formed from four compatible triangular plate elements. The element stiffness is formed by the superposition of the stiffnesses of separate membrane and plate bending elements. Again centerline geometry and homogeneous, isotropic, linear elastic behavior was assumed. The material stiffness was taken to be equal to the concrete material stiffness. (It will be recalled that the slabs were placed eccentric to the beam centerlines (Fig. B2.9) but the complexity and cost of modeling this eccentricity was thought to be unwarranted here.)

5.3.3. Infill Walls

Masonry infill walls were modeled with the infill elements presented in the first part of this study. These infill elements are, essentially, specialized rectangular plane stress elements defined by four nodes, one at each corner. Each node has three degrees of freedom, two translational and one rotational, corresponding to in-plane deformations of the infill. The out-of-plane stiffness of the infill is not modeled.

Typically, infill that completely fills the surrounding frame will be modeled by a single "complete" infill element using nodes defined by the centerline geometry of the surrounding frame. In this way a complete infill element may simply be "plugged" into a conventional frame finite element idealization. To model infill that partially infills a frame two additional nodes must be introduced into the surrounding frame. As a result of the introduction of some of these additional nodes displacement compatibility between some floor beam elements and plate elements (used for the floor slabs) could only be assured by further subdivision of the floor slab finite element mesh. It was decided, instead, to accept this incompatability and the plate element mesh was not subdivided further. Practically speaking, assurance of convergence to the exact solution of the idealized problem, wherein both equillibrium and kinematics are satisfied, is compromised by accepting this incompatibility. One may, however, achieve a reasonably accurate solution if the incompatibility is not unreasonable.

Two variants of both complete and partial infill elements were considered corresponding to either a "soft" constraint or "stiff" constraint relation assumed between the infill and the surrounding frame (see section 3.3 of Part A of this report). Separate models of the building were formulated for each of the two types of contraints in order to evaluate the suitability of the constraint assumptions. It appears that these two constraint assumptions may bound the constraint that may actually exist, initially, between the frame and the infill panel.

The element stiffness generation algorthm for these infill elements allows an ad hoc means to gain some correction for the lack of consideration of beam, column, and infill true dimensions (ie. for the shortcoming of using centerline geometry). This is achieved by using the true dimensions of the infill panel to generate the infill stiffness which will then be added directly to the centerline geometry model without modification by transformation due to the implied rigid joint. This ad hoc procedure has proven to be effective in two examples presented in the first part of this report and was therefore used here with the recognition that its suitability must be considered further.

In all, 13 complete infill elements and 28 partial infill elements were used in the study. The use of infill elements carries with it an additional computational cost beyond that of the frame analysis alone that is due to three effects;

1. An increase in the size of the system (ie. the number of equations used to represent the structural system behavior) resulting from the introduction of extra nodes necessary for partial infill elements. No additional nodes are needed for complete infill elements.

2. The additional computation necessary for the additional element input, stiffness formation, and stiffness addition tasks.

3. The additional member force evaluation needed to obtain the desired infill forces.

The computational effort necessary for the infill element input, stiffness formation and assembly, and member force evaluation is very similar to that needed for beam elements and as such is reasonably insignificant. The computational effort outlined in 1. above will, in most cases, be by far the most significant yet in typical applications should not be unreasonable.

In a sense, then, the use of infill elements is comparable to the use of beam elements. One may add the stiffness contribution of complete infill panels to a conventional frame model with no significant cost penalty. To include the stiffness contribution of partial infill panels necessitates an increase in the size of the system and consequently a significant computational cost increase, but this increase should not be expected to be prohibitive in most practical applications. The use of conventional finite element idealizations (eg. plane stress elements) will generally be cost prohibitive if the same degree of accuracy is sought.

5.3.4. Footings and Retaining Walls

Footings were treated as fixed supports, there was no attempt to model the stiffness of the foundation media or soil-structure interaction. A parameter study of the importance of these effects was rejected as it was not central to the objectives of the study and the details of the soil properties at the site were not available.

Retaining walls that were known to abut the structure were thought to play an important part in stiffening the structure and were modeled with simple linear springs placed at the points of abutment, providing restraint in a direction parallel to the retaining wall. As the retaining walls were massive a large spring stiffness was assumed for these springs. Again the sensitivity of the structural response to these parameters (ie. the spring stiffnesses) was not considered central to the objectives of the study and as such was not considered.

5.3.5. Mass and Damping

The distribution of the mass in the building was modeled by a "lumped" mass idealization wherein mass was lumped at each structural node in proportion to the mass that was tributary to that node. The mass was computed upon the basis of the product of the volumne of each building material and its density within the region thought to be tributary to a given node. The tributary regions were determined upon the basis of judgement alone and every node was considered separately as the distribution of mass was irregular. No part of the occupant load or furnishings were included as the former was very small (no more than 100 small children) and the later was unknown.

As a primary objective of this investigation was to study the variation of dynamic characteristics and seismic response of the building due to the stiffness contribution of the infill the mass distribution used in all models was identical and reflected the actual distribution in the building found at the time of the Guatemalan quakes. Some details of the mass distribution are summarized below (Table B5.1).

There are sources of uncertainty in the mass modeling that warrant discussion. The modeling error introduced by the use of a lumped mass idealization must, properly, be identified but this familiar theoretical simplification should introduce no significant error here. The source of greatest error lies in the evaluation of the mass of each building component and the determination of tributary regions. The former became especially problematic as the details of the construction were often uncertain.

Table B5.1 Mass Distribution For The Escuela De Niñeras

Mass Distribution Statistics					
Level	Area	Structural Mass	Partition Wall Mass	Total Mass	
	(m2)	(kgm)	(kgm)	(kgm)	(kgm/m2)
Roof	366	195,000	0	195,000	533
3rd	366	273,500	55,900	329,400	900
2nd	366	264,600	71,500	336,100	918
1st	195	141,200	40,400	181,600	931
Total	992*	874,300	167,800	1,042,100	767

* Less roof area but including basement area (ie. useable floor area).

In particular, the details of the roof construction were unknown. It was known that in the cantilevered areas, where the structural slab was placed flush with the underside of the supporting beams, a gravel fill was used to fill the void above the slab so that a level floor could be constructed. The entire roof slab was placed flush with the underside of the supporting roof beams thereby producing a waffle-like upper surface. It was not known if this upper waffle-like surface was also filled (with gravel?) to produce a flat roof surface or left unfilled and drainage ports provided. The difference in mass for these two options is great. It was assumed that the waffle-like upper surface was not filled.

Again, to maintain focus upon the central objective of investigating the influence of the stiffness contribution of the infill walls, energy dissipation, nominally, through damping was modeled identically in all models. A viscous damping of 5% of critical was assumed in all modes considered in the time history analyses.

5.3.6. Summary of Models Studied

In all, four models were considered to investigate the influence of the stiffness contribution of both the masonry infill and floor slabs. All of the building mass was included in each model and viscous damping of 5% of critical was assumed in all modes considered for each model. The models included;

Frame

In this model the stiffness of only the frame and the retaining walls were included.

Frame + Slabs

In this model the stiffness of the frame, the floor slabs and the retaining walls were included.

Frame + Slabs + Soft Infill

In this model the stiffness of the frame, the floor slabs, the masonry infill walls, and the retaining walls were included. The infill was modeled using the soft infill elements (ie. based upon the soft constraint assumption).

Frame + Slabs + Stiff Infill

In this model the stiffness of the frame, the floor slabs, the masonry infill walls, and the retaining walls were included. The infill was modeled using the stiff infill elements (ie. based upon the stiff constraint assumption).

It will be convenient to use the second model - Frame + Slabs - as the standard against which other models will be compared as it is representative of the familiar model of frame and rigid diaphragm (floor slab) that is commonly used in practice. The comparison between the first model - Frame - and the standard - Frame + Slabs - is presented, pedagogically, to demonstrate the well known importance of the influence of the (almost rigid) slab upon the dynamic character and the seismic response of a building. Although this comparison is not central to the objectives of this study the familiar results obtained point out the suitability of using plate elements to achieve a reasonable modeling of the stiffness contribution of the floor slabs.

The comparison between the standard model - Frame + Slabs - and the two models including the stiffness contribution of the infill is central to this study and will be discussed in detail subsequently.

Sources of uncertainty that affect the modeling of the stiffness, the mass, and the damping of the structure have been identified. These uncertainties are not unique to this problem, rather it is suspected that these types of uncertainties are common to most building analyses. Some important sources of uncertainty in the modeling of the building under study are;

Stiffness Uncertainties

* The uncertain material mechanical characteristics of the concrete, steel and especially the masonry used in the building.
* The uncertain modeling of beam section properties.
* The uncertain consequences of the use of centerline geometry ignoring beam-column joints and slab eccentricities.
* The uncertain constraint condition between the infill and surrounding frames.
* The uncertain soil stiffness and soil-structure interaction.
* The uncertain connectivity of the massive, stiff upper story sun shades.
* The uncertain restraint offered by the retaining walls and stair towers.

Mass Uncertainties

* The uncertain construction of the roof.
* The uncertain evaluation of tributary regions used to determine the lumped masses.
* The uncertain mass coupling of the stair tower and soil.

Damping Uncertainties

* The (usual) uncertain use of viscous damping and the assignment of the magnitude of the damping.
* The uncertain importance of additional energy dissipation through soil-structure interaction.

In addition to modeling uncertainties there are uncertainties in the loading, discussed above, and uncertainties that result from practical considerations of analysis (eg. the number of modes chosen to capture the behavior of the building to ground excitation). Much of this uncertainty may well be minimized through the good judgement of the analyst but only with an appreciation of the uncertainties and the approximations inherent in an analysis may one draw the full meaning, and no more, from an analytical study.

5.4. Natural Periods, Mode Shapes and Participation Factors

The dynamic character of each of the four models may be completely defined by;

1. the complete set of natural periods and mode shapes (ie. eigenpairs) obtained from an eigenanalysis of the undamped system of equations defining each model,

2. the complete set of participation factors derived from these mode shapes and the mass distribution (ie. mass matrix) of each model,

and,

3. the assumed damping (5% of critical in the present study) used in all modes of each model.

Not only are periods, mode shapes, participation factors, and damping ratios sufficient to define the dynamic character of a linear damped system but they appeal to the intuition and hence a physical understanding of the dynamic nature of the real system. For this reason the periods, mode shapes and participation factors will be considered in detail in this section (damping was discussed above).

In the present study there were 1086 such eigenpairs, for each model, and consequently 3 x 1086 modal participation factors, one for each base degree of freedom considered (ie. two horizontal and one vertical). It was therefore unreasonable to consider the complete set of eigenpairs and participation factors. A compromise between computational expense and accuracy was made and only the first 18 eigenpairs were determined for each model. The natural periods found and some of the associated participation factors are tabulated below (Tables B5.2 & B5.3).

Table B5.2 Natural Periods For Four Models Of The Escuela De Niñeras

NATURAL PERIODS (seconds)				
Mode	Frame	Frame + Slabs	Frame + Slabs + Soft Infill	Frame + Slabs + Stiff Infill
1	0.650	0.604	0.413	0.362
2	0.623	0.586	0.372	0.302
3	0.562	0.540	0.331	0.246
4	0.356	0.204	0.159	0.152
5	0.350	0.200	0.152	0.150
6	0.257	0.177	0.142	0.140
7	0.218	0.150	0.138	0.129
8	0.211	0.148	0.132	0.128
9	0.206	0.142	0.130	0.120
10	0.192	0.142	0.119	0.118
11	0.189	0.128	0.118	0.112
12	0.176	0.125	0.115	0.107
13	0.173	0.122	0.113	0.105
14	0.166	0.119	0.107	0.097
15	0.156	0.110	0.095	0.093
16	0.153	0.100	0.094	0.093
17	0.150	0.099	0.093	0.092
18	0.148	0.098	0.090	0.089

Participation factors were computed, for each mode and each of the three ground motion components. For the x component of ground motion;

$$c_{nx}=\frac{\mathbf{\Phi}_n{}^T\mathbf{m}\mathbf{r}_x}{\mathbf{\Phi}_n{}^T\mathbf{m}\mathbf{\Phi}_n}$$

where;

c_{nx}= the participation factor for mode n considering ground excitation in the x direction,

Φ_n= the eigenvector (mode shape coordinates) of the nth mode,

m= the mass matrix of the system,

and

$\mathbf{r}_x$= a vector of zeros and ones with those elements equal to one corresponding to x translational degrees of freedom of the system.

Participation factors for the y and z components of ground motion, c_{ny} and c_{nz}, are defined in a similar manner.

Table B5.3 Participation Factors For Four Models Of The Escuela De Niñeras

PARTICIPATION FACTORS				
Mode	Frame	Frame + Slabs	Frame + Slabs + Soft Infill	Frame + Slabs + Stiff Infill
Longitudinal Ground Motion				
1	.0018	.0003	.8177	.9102
2	.0170	-.0101	-.4607	-.2721
3	.9255	-.9238	.1228	-.0840
4	.0010	-.0700	-.0574	-.0302
5	-.0167	-.0446	.0087	-.0048
6	.0041	.2597	-.0413	-.0191
7	-.0041	.0006	.0347	.0171
8	-.0469	.0208	-.0327	.0607
9	-.0250	.0004	.0008	.0166
10	.2234	.0012	-.0091	.0040
11	-.0122	.0007	-.1668	.0238
12	.0751	-.0012	.0310	.0134
Transverse Ground Motion				
1	.8874	-.8358	.1203	.0711
2	-.0444	.2898	.4147	.4517
3	-.0009	-.0032	.7796	-.7773
4	-.0295	-.3220	-.0228	.0073
5	-.0018	.0744	-.1467	.0881
6	.0221	.0024	-.0688	-.0438

PARTICIPATION FACTORS				
Mode	Frame	Frame + Slabs	Frame + Slabs + Soft Infill	Frame + Slabs + Stiff Infill
7	.3234	.0729	-.0242	-.0648
8	-.0404	-.0015	.2974	-.0137
9	.0781	.0549	-.0167	-.1770
10	.0089	-.0023	.0015	-.0258
11	.0088	.0499	-.0080	-.0765
12	-.0049	.0019	.1101	-.0793
Vertical Ground Motion				
1	-.0127	.0133	-.0001	.0012
2	.0012	-.0045	-.0049	-.0020
3	.0006	-.0006	-.0136	.0093
4	-.0031	-.0521	.0204	.0586
5	.0002	.0129	.1315	-.1410
6	.0024	-.0015	.0549	.1261
7	.0498	.1307	.1176	.1541
8	-.0079	.0129	.0402	.0188
9	.0140	-.2527	-.1575	.1638
10	-.0007	.0058	.1632	-.0402
11	-.0255	.0063	.0194	.0622
12	.0002	.2726	.2273	-.0965

These natural periods clearly indicate that the infill (and the floor slabs) has a very significant effect upon the dynamic character of the building, shifting the period of the lower three modes most significantly as well as altering their participation. The third mode is shifted most dramatically by the infill from the results obtained using the Frame + Slabs model; by 39% when using soft infill modeling and 55% when using stiff infill modeling. The periods alone tell only part of the story, though, and one must consider the mode shapes and participation factors as well. Perspective projections of the first 12 modes of each model were prepared and compared, the first six mode shapes are reproduced here (Fig. B5.2 to Fig. B5.7). (These drawings are straight-line approximations to the true mode shapes calculated so the reader is encouraged to imagine some smoothing.)

An examination of the participation factors, with the mode shapes in mind, suggest the longitudinal motion of the Frame + Slabs model is largely uncoupled from its transverse motion. That is to say longitudinal (east-west) ground motion seeks the participation of the principal longitudinal modes (ie. modes 3 and 6) primarilly and transverse ground motion seeks the participation of the principal transvers modes (ie. modes 1, 2 and 4). the infill appears to have the effect of coupling the transverse response to the longitudinal response as the mode shapes are not distinctly longitudinal or transverse, indeed they have a torsional nature, and the participation factors of the lower three modes for both longitudinal and transverse ground motion are of significant magnitude.

It also appears that vertical motion excites principally the higher modes (see discussion below of component modes and cantilever response) as may be expected. The infill appears to have the effect, though, of introducing these higher modes earlier in the modal sequence. It is likely, then, that the set of the first 18 modes will prove insufficient to completely capture the response to vertical excitation. The secondary objective posed above (section 1.2) to;

"Study the importance of vertical motion of beams in the response of the building ..."

may thus be compromised.

From these drawings, the tabulated natural periods and participation factors it is clear that the dynamic character of each model is significantly different from the others. The two infill models do have similar lower modes though. In essence the Frame model, the Frame + Slabs model, and the two Frame + Slabs + Infill models constitute three completely distinct structural systems, from a dynamic point of view, and must, therefore, be expected to have completely different seismic behavior. One may not, reasonably, use a model like the Frame + Slabs model to model the behavior of a building that makes extensive use of infill.

The comparison of the Frame model and the Frame + Slabs model indicate the importance of the slabs in coupling the response of the building's frames. This aspect of behavior is well known and is often modeled by constraining degrees of freedom within the plane of the floor slabs to be rigidly coupled (ie. rigid floor diaphragm simplification). When employing a general three-dimensional analysis program one must take special care to model this important aspect of behavior. It is believed that the use of so called 'equivalent' beams in three-dimensional analysis may not provide sufficient in-plane stiffness to accurately model the in-plane rigidity of the slabs (see section 3.3.2).

The comparison between the Frame + Slabs model and the two Frame + Slabs + Infill models is of principal interest here. To this end it will be convenient to distinguish between;

Body Modes

Modes that may be characterized as involving the deformation of the structure as a whole.

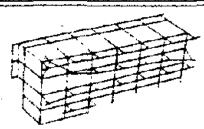
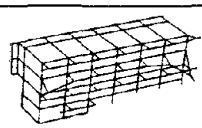
Component Modes

Modes that may be characterized as involving, principally, the deformation of a component part of the structure (eg. cantilevers).

In a general sense it may, then, be said that the infill tends to;

1. Suppress the development of some of the lower body modes (eg. modes 4, 5, and 6 of the Frame + Slabs model).
2. Alter the form of the important lower fundamental modes (ie. modes 1, 2, and 3), coupling longitudinal to transverse motion.
3. Introduce component modes (eg. cantilever "flopping") earlier in the modal sequence. For example consider and compare the two groups of similar mode shapes below.

Table B5.4 Similar Mode Shapes For Different Models Of The Escuela De Niñeras

SIMILAR MODE SHAPES (period in secs.)			
Mode Shapes (cantilevers)	**Frame + Slabs**	**Frame + Slabs + Soft Infill**	**Frame + Slabs + Stiff Infill**
	**Mode 8 (0.148)**	**Mode 4 (0.159)**	**Mode 4 (0.152)**
	**Mode 7 (0.150)**	**Mode 5 (0.152)**	**Mode 5 (0.150)**

and, of course,

4. Shift the periods of similar mode shapes and fosters the development of completely dissimilar mode shapes.

The first three modes are particularly interesting. In the the Frame + Slabs model we see the familiar lower modes; a transverse lateral body mode (with some slight torsional component), a torsional body mode, and a longitudinal lateral body mode. The first three modes of the models including the infill contribution, on the other hand, show significant torsional components in all the lower three modes.

It is appropriate to reconsider the response spectra presented in section 4.2 in light of these findings. The response spectra of the Guatemalan event indicates a relatively lower response (in terms of pseudoacceleration and consequently member forces) in the 0.3 to 0.7 second period range than in the lower 0.1 to 0.3 second period range (Fig. B4.3). Such increased response at lower periods is a characteristic of the Guatemalan quake not shared by the El Centro and Pacoima Dam events (Fig. B4.4) or of proposed design response spectra [B20]. The infill has the effect of shifting the lowest modes out of this response trough toward the lower period range of greater response. In as much as the lower modes will, most likely, dominate the response of the structure to the Guatemalan excitation this effect may account for the relatively greater damage suffered by the Escuela De Niñeras than other buildings of similar size but without a frame-infill system. In other words, the infill may have had the effect of tuning the building to the Guatemalan quake.

5.5. Response History Analysis

5.5.1. Introduction

Complete three dimensional dynamic analyses of the Frame + Slabs model and the two Frame + Slabs + Infill models were computed by the superposition of the response of each of the 18 modes discussed above to each of several ground motion excitations. The ground motion excitations considered included;

1. 1976 Guatemala/189 - horizontal components.
2. 1940 El Centro (0.5g) - horizontal components.
3. 1940 El Centro (0.5g) - horizontal and vertical components.
4. 1971 Pacoima Dam (0.5g) - horizontal components.
5. 1971 Pacoima Dam (0.5g) - horizontal and vertical components.

(The vertical components of the 76 Guatemala/189 are not available. The measured Pacoima Dam record rather than the Derived Pacoima Dam record was used in this study.)

The response parameters of principal interest in all these studies included;

1. Selected floor displacement components.
2. Selected interstory drift components.

3. Column member forces - shears, axial force, bending moments and torsion.

4. Infill member forces and infill nominal shear stresses.

5.5.2. Guatemalan Ground Motion Response

The available ground motion record for the Feburary 4, 1976 Guatemalan quake was generated from a seismoscope trace of the event. As such the record is rather uncertain (see section 4 above) and only the horizontal components of the ground motion could be obtained, the nature of the vertical component is simply unknown. Furthermore, these horizontal components have been generated for two separate sequences in time, I and II, with some certainty that sequence I was first in time.

The Frame + Slabs and the two infill models were each subjected to the ground excitation (acceleration) of both components simultaneously of the combined record of sequence I plus sequence II. As the reported components were colinear with the principal axes of the building and the reference axes of the models, the N-S excitation was applied directly to the N-S base degrees of freedom and the E-W excitation to the E-W base degrees of freedom. The duration of the combined sequences was 6.9 seconds. The details of the building response are discussed below.

Displacement Response

It is difficult to characterize the complete three dimensional response time history of the models of the building as the number of displacement parameters considered is so great (1086 degrees of freedom). Yet with a knowledge of the mode shapes, presented above, the roof displacement envelopes presented in figure B5.8 suggest the important features of this displacement response.

It is seen that the infill has a significant effect on the displacement response of this building, as anticipated from inspection of the modes of vibration alone. In a general sense both the soft and the stiff infill idealizations tend to decrease maximum displacement response, when compared to the model ignoring the infill contribution, but certain local maximum displacements are seen to be practically equal to those maximum displacements obtained when the infill contribution is ignored. We are lead to the paradoxical conclusion, then, that the infill will tend to stiffen the building in a global sense but may not reduce all displacements.

This curious conclusion results as a consequence of the increased importance of the torsional response so evident in the lower mode shapes and again reflected in the roof displacement envelopes of the infill models. On the otherhand, the envelopes for the Frame + Slabs model indicate that lateral deformations play the dominant role and torsion plays a small role in the response of this model.

The response of the two infill models is not very different. The stiff infill model does tend to reduce displacements in a general sense but again local displacements are seen to be greater than in the soft infill model. It is expected that the response of the real system may be bounded by these two extremes.

Maximum story drift indices were computed for each frame in both the N-S and the E-W directions for each of the three models. Some of these results are reported below.

It is recognized that the serviceablity of a building may be compromised if the building sustains large drifts during a seismic event (eg. damage to furnishings and partitions) even though the structural system may suffer no distress. Consequently, it is reasonable to design buildings so that they are stiff enough to minimize drift. Unfortunately there is very little reliable data to establish these drift limit states, indeed, there is not sufficient data available to even establish a suitable measure of drift (ie. a measure well correlated with damage) [B15].

Recognizing the need to define drift limit states tentative proposals have been made, based largely on judgement. A drift limit state must be defined in terms of both a limit to a suitable measure of drift and a load level underwhich this limit is to be considered.

In the United States the Uniform Building Code (UBC 76) stipulates;

Table B5.5 N-S Drift Of The Escuela De Niñeras Due To The Guatemala 76 Excitation

MAXIMUM NORTH-SOUTH DRIFT INDICES (Guatemala 1976)				
Floor	Transverse Frame No.	Frame + Slabs	Frame + Slabs + Soft Infill	Frame + Slabs + Stiff Infill
	1	0.0036	0.0024	0.0017
3rd	4*	0.0037	0.0011	0.0009
	7	0.0037	0.0027	0.0020
	1	0.0041	0.0023	0.0013
2nd	4*	0.0042	0.0011	0.0008
	7	0.0042	0.0027	0.0016
	1	0.0037	0.0036	0.0026
1st	4	0.0045	0.0025	0.0016
	7*	0.0045	0.0018	0.0007

* Corresponds to an infilled frame.

Table B5.6 E-W Drift Of The Escuela De Niñeras Due To The Guatemala 76 Excitation

MAXIMUM EAST-WEST DRIFT INDICES (Guatemala 1976)				
Floor	Transverse Frame No.	Frame + Slabs	Frame + Slabs + Soft Infill	Frame + Slabs + Stiff Infill
3rd	A	0.0019	0.0010	0.0010
2nd	A	0.0041	0.0016	0.0013
1st	A	0.0054	0.0038	0.0030

"Lateral deflections of drift of a story relative to its adjacent stories shall not exceed 0.0050 times the story height unless it can be demonstrated that greater drift can be tolerated. The displacement calculated from the application of the required lateral forces (for strength design) shall be multiplied by (1.0/K) to obtain the drift. The ratio (1.0/K) shall not be less than 1.0. ..." (section 2312h)

(It will be recalled that K is an inverse measure of a buildings probable ductility ranging from 0.67, for the most ductile structures, to 1.33, for the least ductile structures.)

This proposal is similar to others in that;

1. the measure of drift is taken as the "horizontal drift" (ie. the ratio of interstory lateral displacement to story height).

and

2. the load level underwhich the limit is set is different from the load level used for strength design (strength limit states), usually a larger load level is considered appropriate.

In Canada the National Building Code limits the horizontal drift to 0.005 also, but at a load level three times the equivalent static seismic loading used for strength design (this NBC static loading is consistently lower than the UBC static loading, though) [B27]. In Mexico City a drift limit of 0.008 is set under a seismic loading defined by a design response spectra for deformation that is substantially greater than that used for strength design [B24]. Although the Japanese Building Standards Law has no drift provision Aoyama's [B1] current proposal for a seismic code sets a limit of 0.010 under a seismic loading based upon a simplified inelastic analytical procedure using elastic design response spectra.

New Zealand takes a different approach to the problem. A first drift limit of 0.006 is set for a static equivalent load twice that load used for strength design above which brittle elements (eg. glass) or elements that have sufficient stiffness to alter the structural behavior of the building as a whole (eg. masonry infill panels) are to be separated from the principal structural elements. A second drift limit is also set, for the same loading, that must not be exceeded in any case.

It is generally, but not universally [B15], agreed that horizontal drift, as defined above, is a suitable measure of drift yet there appears to be little evidence to support this. Although one must certainly expect greater levels of damage with larger horizontal drift any level of damage does not correlate well with this measure. For example Newmark [B20] reports that bond failure between masonry infill and its surrounding frame may occur at horizontal drifts of from .0001 to .0030. Such a wide range indicates a practically useless correlation. It is also generally agreed that strength design seismic load levels intended for equivalent static elastic analysis are unreasonably low for deformation analysis needed to set drift limits. Rosenblueth notes further;

> "The question, however, is not so clear-cut. We should limit drifts essentially to control serviceablity. We should therefore be more interested in shorter return periods than when we are concerned with collapse. We should even adopt different design spectral shapes for design against collapse than for drift limitations. ..."[B24]

Possibly the horse is in front of the cart; before considering the appropriate loading to be considered for drift limitations one must first find a measure that is well correlated with the serviceability problem of concern; damage to partitions.

Due in large part to these drift limit proposals, that were introduced as tentative suggestions only, horizontal drifts of from 0.005 to 0.010 have become informally associated with the initiation of damage to nonstructural partitions. A growing faith in these numbers and an apriori belief that horizontal drift is well correlated with damage has resulted.

Horizontal drift, as defined above, was originally intended to provide some nominal measure of the state of shearing strain (distortion) in building partitions. Although it is now recognized that horizontal drift is not a good measure of distortion* and other measures have been proposed (eg. tangential story drift) it is not reasonable to expect a simple relation between a single nominal measure of strain and damage to exist for all wall materials and/or construction. The assumption that such a limit state exists is contrary to basic structural mechanics in that a structural element's resistance to stress/strain (ie. an element's behavior) is generally

* For example a rigid rotation of a building about a horizontal axis through its base results in horizontal drift without distortion.

dependent upon the element's construction, support conditions and material behavior.

There also appears to be some uncertainty in the proper use of the conventional limit state of horizontal drift; should it be considered as a limit for the analysis of a structural idealization including the stiffness contribution of the partitions or an idealization ignoring this contribution?

In the present study the drift indices of the idealization ignoring the stiffness contribution of the partitions approached this popular limit state of 0.0050. We know that the damage was very great to the partitions of this building. We can not, however, conclude that this observation justifies the use of the number 0.0050 as it has been shown that the structural idealization of the building ignoring the contribution of the infill represents a structure that is altogether different from the real structure and as such may not be correctly used to predict the behavior of the real building. We must conclude that the correspondance between the popular limit state of 0.0050 and the observed behavior of this unrealistic modeling is merely fortuitous.

The infill models are expected to provide a more realistic prediction of the behavior of the response of the building. The drift indices predicted from this more accurate modeling are well below the limit state of 0.0050 and yet we know the damage to the partitions was extensive and severe. Indeed the smallest drift indices are associated with the infill partitions, the very partitions that suffered the greatest damage. We know that infill may very effectively stiffen a frame and as such limit local drift. This is, of course, accomplished at the cost of the tendency of these stiffened frames to transmit higher shear forces and thus, typically, infill partitions may be expected to suffer higher shear stress levels while still limiting drift.

The attempt to find a single number to characterize a drift limit state for partition damage must be expected to be a fruitless venture. The materials and method of partition construction must surely come into play. Taking a more traditional approach, the author attempted to correlate a nominal measure of maximum shear stress in the infill panels with the known, albeit uncertain, material behavior of the infill wall material. This will be discussed subsequently.

Member Force Response - Columns

Column member force histories were computed for all columns of the Frame + Slabs model and the two infill models for the combined effect of gravity and seismic loading. Maximum member force values and their time of occurrencewere recovered from this computation. As the analysis was three dimensional, six force parameters were considered for each column end including two shears, two bending moments, an axial force and a torque.

To investigate the influence of the infill upon column member force response a comparison was made between the maximum force quantities obtained for the Frame + Slabs model and each of the two Frame + Slabs + Infill models (ie. Soft Infill and Stiff Infill). A stress increase ratio was defined to facilitate this comparison as;

$$R_{ijk} = \left\{ \frac{F_{infill} - F_{frame}}{F_{frame}} \right\}_{ijk}$$

where;

R_{ijk} = stress increase for member i, end j considering member force k. With $k = 1$ *to* 6 corresponding to each of the six member force components.

F_{infill} = maximum member force computed including the stiffness contribution of the infill.

F_{frame} = maximum member force computed ignoring the stiffness contribution of the infill.

Stress increase ratios were computed for the stiffness contribution due to the stiff infill model as well as the soft infill model for each of some 69 columns. This produced two sets of 828 stress increase ratios (69 columns x 6 member forces x 2 ends per column). These values have been plotted in a manner similar to member force diagrams on perspective representations of all transverse frames of the building (Fig. B5.9 to Fig. B5.20). On these drawings the column axes correspond to a zero member force increase (ie. no change in member force), values plotted to the right (northward) correspond to a positive increase and to the left (southward) a negative increase (ie a decrease in member force). A scale is included showing a five-fold increase keyed to the length of the cantilevers for convenience.

These figures require some patience, at first, to read but with this initial effort provided they reveal a great deal about the nature of force redistribution in the complete three dimensional frame due to the influence of the infill.

Although the stress increase ratios provide a valuable relative comparison the significance of any relative increase is ultimately determined by the absolute value of the forces in question and the capacity of the member to resist this force. For this reason more conventional member force envelopes were plotted, again in perspective representation, for both infill models (Figs. B5.21 to B5.32). These member force envelopes may be used in conjunction with the stress increase envelopes to identify the significant member force increases.

The capacity of any column must be defined in terms of the interaction of all six member forces, in general. The capacity of any column of this building is typically determined by consideration of shear alone as the tie spacings were so great in the column and thus the shear capacities were very low. One interaction diagram is presented here (Fig. B5.33) for the column of principal interest in the study, the first floor column of transverse frame 3. (see Fig. B2.9). In as much as this column is an axysymmetric square column this one interaction diagram will define the capacity for the combination of axial load and either M_{xx} or M_{yy} and may be used to estimate the biaxial bending capacity of the column as well. Furthermore, as all columns in the building are 40 x 40 cm and are practically limited by their shear capacity one may reasonably use this one diagram to approximately estimate the shear capacity of any column in the building.

In this interaction diagram the ultimate flexural capacity was generated from first principals assuming an ultimate concrete compressive strain of 0.003. The ultimate shear capacity of the column (based upon the reccomendations of the American Concrete Institute code - ACI 318-71) is also indicated on this figure assuming a likely moment distribution over the height of the column, under lateral loading, as shown. Ultimate shear capacities of a full height column

(3.0 m) and a short column (0.9 m) are presented. This interaction diagram was generated using the program RCCOLA [B14].

It should be noted that this figure (Fig. B5.33) is included here solely to give scale to the results obtained in the response history analysis of the building. As all analyses were elastic and therefore did not account for inelastic dissipation of energy the computed member forces will be over estimated. Consequently, member forces in the critical columns are well beyond the capacities defined by this figure. Nevertheless, a careful examination of the critical column member forces reveal combinations of relatively low axial loads , with occaissonal tensile excursions at times, in combination with relatively large bending moments that indicate, in light of the interaction diagram, the likely failure mode of shorter columns must be expected to be a shear failure mode. It will be seen that partial height infill has the effect of producing short column behavior.

The force redistribution due to the contribution of the soft infill is very similar to that of the stiff infill, the latter showing greater exaggeration than the former in most respects. For this reason it is best to direct one's attention to the results obtained in the comparison of the stiff infill model to the model not including any infill contribution.

With the exception of column torsion the infill tended to reduce maximum member force response in the upper stories while increasing member forces in the lower stories. This redistribution from the upper to the lower story columns is generally significant with stress increases typically between 1.0 and 2.0. Shear and moment increases associated with short columns due to the constraint offered by partial infill or as found below grade in the western end of the building were most dramatic. In particular the short columns found in the basement level suffered moment increases as high as 26.0.

The increases for column torsion are not significant as the torsional moments are small in absolute sense. These increases result, however, from the greater torsional response of the building as a whole resulting from the infill influence discussed above.

Although the infill can effectively carry vertical loads thereby reducing axial loads on the adjacent columns the interaction between the frame and the infill may, nevertheless, result in larger axial loading of the columns under seismic excitation. Such an increase is evident in figures B5.9 & B5.10. In at least one instance the infill not only increased the maximum compressive force in the column but caused large tensile excursions that were not observed in the model without the infill contribution (see discussion below of critical column 54). Curiously, the columns that suffered the greatest axial load increase are located below grade in transverse frame 6 and as a result of their location these columns were not inspected by the U.C. reconnaisance team.

It is natural to consider the stress increase of the column shear, V_x, (east direction) and the column moment, M_{yy}, (about the north-south axes) together, as we may associate these member forces with longitudinal (east-west) distortion of the building (Figs. B5.13 & B5.15 or Figs. B5.14 & B5.16). These results are the most dramatic with relative stress increases associated with longitudinal partial infills as high as 26.0 for M_{yy} and 3.5 for V_x. These values, associated with the basement column of transverse frame 2, are, however, not as dramatic in the absolute sense (see Figs. B5.25 & B5.27 or Figs. B5.26 & B5.28) and as such are not practically significant. The reconnaisance team was unable to inspect these basement columns and thus their real behavior is unknown.

The increases in the northern, first floor columns of transverse frames 3 and 4 are seen to be especially important in light of the combined consideration of the relative stress increases and the absolute values of member forces. The computed infill force levels were also greatest in these areas. As the actual damage suffered by the building was concentrated in these areas it appears that the prediction of the behavior of the building and the observed behavior of the building were in accord in the general sense. To investigate the success of predicting local behavior the response of a specific column of this area will be discussed in detail. This column, the northern column of the first floor tranverse frame 3, (henceforth referred to as column 54) was selected for detailed consideration because it was among the two most severly damaged columns and lended itself to modeling more easily than the other critically damaged column.

The short stub columns located below grade in transverse frames 5, 6 and 7 showed only slight increases in shear, V_x, and moment, M_{yy}, yet the absolute value of these member forces were, nevertheless, significant. As noted the location of these columns did not allow inspection and their real behavior is unknown.

It is also natural to consider the shear, V_y, together with the moment, M_{xx}, as these forces may be associated with transverse deformations (Figs. B5.17 & B5.19 or Figs. B5.18 B5.20). Surprisingly these member forces are seen to decrease throughout most of the structure with only the short stub columns, found below grade, showing both a relative increase and significant force levels as a result of infill influence. Yet the observed damage clearly indicated distress in columns due to transverse deformations.

It appears, then, that the infill models' behavior captures the longitudinal behavior of the real building but not the transverse behavior. One may ask why? In the longitudinal direction partial height infill panels play a key role in the response of the building. These partial height infill panels constrain the adjacent columns to behave as short columns, that is to say the infill forces a relatively greater participation of these columns in resisting the lateral loads.

In the transverse direction, on the other hand, all the infill is of full height. Full height infill panels actually relieve adjacent columns of shear and moment due to lateral loading as long as the infill stress levels do not exceed the capacity of the infill material. If, however, the full height infill stress levels become excessive local or complete failure of the infill panels may result, demanding a concomitant force transfer to the columns. The elastic infill elements used to model the infill panels behavior will not, of course, "fail" and as a result these infill panels will continue to relieve the column demands at all loading levels.

In another sense the full height elastic infill models tend to protect the adjacent columns from distressing deformation in the plane of the infill while the partial height infill panels will not. One must, therefore, look to the infill stress levels rather than the column member forces to find areas of potential distress to both the infill and the adjacent columns when considering full height infill panels. An inspection of column member forces may be sufficient when considering partial height infill panels, but an examination of infill stress levels will provide some additional indication of potential problem areas.

The forgoing discussion suggests a failure mechanism for these full height infill-frame subsystems. Perhaps in the actual response of such infill-frame systems the full height infill panels may be stressed sufficiently to cause a partial failure of the panel thereby transforming the panel from a full height panel to a partial height panel. Such transformations were evident in the inspection of the building under study (see Fig. B3.3). Upon being transformed from a full height panel to a partial height panel the adjacent column is transformed from a protected column to a short column and consequently suffers a sudden increase in the lateral load it must resist (see Fig. B3.1). This failure mechanism is illustrated schematically in figure B5.34. It is likely that simply breaking the bond between the infill and the upper beam will effectively cause such a transformation. This bond failure is expected to be accomplished at relatively low stress levels in most instances and for this reason the protection offered by full height panels may be a theoretical possibility but practically unrealistic.

Member Force Response - Column 54

The damage to column 54 was dramatic (Fig. B3.1). The column was completely severed by a diagonal shear failure plane defined, roughly, by an outward normal with unit components along each of the three axes defining the structural geometry (ie. a 1,1,1 plane)(Fig. B5.35). Such a plane suggests the shear failure was due to a state of biaxial shear - combined V_x and V_y.

The member force response histories computed for this column indicate that the infill resulted in significant increases in the V_x shear, some decrease in the V_y shear and, importantly, several excursions into a tensile state. Such tensile excursions were not observed when the infill contribution was ignored. Furthermore, in the soft infill modeling the early tensile excursion was coincident in time with the maximum V_x shear while in the stiff infill modeling the early tensile excursions occurred simultaneously with significant V_x shears and occurred close in time to the maximum shear response. In both infill models the maximum shear at the top of the column had a positive x sense (an easterly sense) and was significant in magnitude while the shear in the y direction was not significant. The sense and significance of these shears would suggest a failure plane with a outward normal vector with (x,y,z) components of (1,0,1), different from the failure actually observed.

This column was restrained in the longitudinal (x or easternly) direction by a partial height infill panel and in the transverse (y or southernly) direction by a full height panel. As noted above, an elastic model of a full height partition may not "fail" and thus protects the adjacent column from distortion at all levels of loading. On the otherhand the partial height infill panel forced short column behavior and thus resulted in the increased and significant V_x shears observed.

The computed maximum nominal infill shear stress levels of the full height infill panels associated with this critically damaged column were found to be larger than all other panels and well above the probable strength of the panels. Two full height panels were associated with this column separated by a door opening (see Fig. B2.9), a northern wider panel and a southern narrower panel. The computed maximum nominal infill shear stresses for these panels were, respectively, 1.91 MPa (277 psi) and 2.60 MPa (377 psi) using the stiff infill modeling and 1.28 MPa (186 psi) and 1.66 MPa (241 psi) using the soft infill modeling. (It will be recalled that the direct compressive strength of the infill was likely to be in the range of 1.75 to 2.25 MPa and the shear strength would be expected to be a fraction of these values.) Furthermore these infill panels reached their maximum stress levels very close in time to the maximum response of the column to longitudinal deformation.

It appears, then, that the differences between the observed behavior and the predicted behavior may be reconciled if it is assumed that the transverse full height infill panels failed partially leaving partial height infill panels in both the longitudinal and transverse directions. The column would then be expected to suffer a shear failure along a failure plane defined by the outward normal of (1,1,1) as obsevered.

It is important to emphasize that this failure mechanism is postulated upon the combined evidence of the observed behavior and the predicted behavior. One may not expect to predict such a nonlinear sequence of events from just one elastic analysis alone, although a series of elastic analyses, removing or softing elements as they reach their capacity may offer some insight. A single elastic analysis may help to point to areas of potential initial distress, however, and in this limited sense such analyses are useful.

Member Force Response - Infill Panels

Response histories were computed for twelve member force components (corresponding to the twelve degrees of freedom) of each infill element for both the soft infill model and the stiff infill model. From these histories nominal panel shears stresses were computed. The nominal shear stress, at each time step, was taken equal to the total horizontal load resisted by the panel divided by the panel's horizontal cross sectional area. The stress distribution across the panel was not computed. From this computation maximum nominal shear stresses were recovered for each panel along with their time of occurence. These maximum values were compared to the observed infill panel damage.

Four damage levels were identified during the post earthquake survey;

N.D. - no damage

S.D. - slight damage

P.D. - partial damage

and,

B.D. - badly damaged.

Although only a few infill panels fall into each of these categories (only two suffered partial damage) a mean value of the computed maximum nominal shear stress within each category was evaluated. Many of the short partial height infill panels were not included in the post earthquake survey (eg. the southern walls at the ground, second and third floor levels, see Fig. B3.4) and consequently were not included in this evaluation. These mean values (along with an indication of the standard deviation about the mean) were plotted against the damage indices to investigate the correlation between this measure of infill distress and the observed damage (Fig. B5.36).

It is seen that the correlation is positive, albeit rather uncertain. The uncertainty is greater at the higher damage levels in part because there were fewer infill panels in these categories and in part because the elastic analysis cannot be expected to capture the inelastic force redistribution that most certainly occurred in the real response. Nevertheless it appears that the stress levels computed via an elastic analysis may effectively point to areas of potential initial failure.

It is interesting to compare these computed stress levels with the likely shear capacity of the infill material. It has become common practice to relate the shear strength of concrete and masonry to the square root of the direct compressive stress of the material, f'_m (ie. the prism strength). For this reason the data plotted in figure B5.36 is normalized with respect to $\sqrt{f'_m}$. The probable crushing strength of the material was 2.00 MPa +or- 0.25 MPa (see section 2.4 above) an uncertain number.

Newmark [B20] reports that infill panels will suffer initial cracking at a shearing stress of approximately $0.23\sqrt{f'_m}$, MPa, and may sustain a maximum shearing stress of $0.28\sqrt{f'_m}$, MPa, [B20]. The stress levels correlated with the initiation of damage (ie. between N.D. and S.D.) of approximately $0.5\sqrt{f'_m}$, MPa, and $0.4\sqrt{f'_m}$, MPa, for the stiff and soft models, respectively, are double Newmark's suggestions. This is to be expected as the elastic analysis does not account for energy dissipated inelastically that was clearly an important part of this building's response. As a result the elastic force levels that were developed must be expected to be overestimated.

The Uniform Building Code (UBC 1976) sets the maximum allowable shear stress in unreinforced masonry walls at $0.024\sqrt{f'_m}$, MPa ($0.3\sqrt{f'_m}$, psi). This reccomendation may be appropriate for unconfined shear strength but as infill panels may sustain some confinement, depending on the completeness of the infilling and the stiffness of the surrounding frame, this reccomendation may be too conservative. Nevertheless it seems reasonable to reccomend a limitation on panel shear to avoid panel damage and the reccomendation must necessarily be very conservative as the material properties of the infill are seldom well known. It may be reasonable to set separate reccomendations for panels of partially infilled frames (perhaps $0.024\sqrt{f'_m}$, MPa) and panels of completely infilled frames.

For completely infilled frames it would be reasonable to allow greater stress levels with more rigid surrounding frames. A suitable measure of the relative stiffness of the surrounding frame may be provided by the dimensionaless parameters discussed in part A of this study. Perhaps a reasonable range of allowable stress levels of from $0.024\sqrt{f'_m}$ to $0.100\sqrt{f'_m}$, MPa, would be appropriate for the corresponding range of frame stiffness from very soft frames to very stiff frames. This would provide a safety factor against initial cracking of 2.3 at the upper end if Newmark's suggestions are correct. The surrounding frame would, necessarily, have to be designed so that its stiffness could be utilized, that is, it must have sufficient strength to provide the confinement and should not fail when the infill fails (cracks) (see Klingner [B13]).

5.5.3. Response to the El Centro and Pacoima Dam Records

The response of the building to the 0.5g El Centro and 0.5g Pacoima Dam records was investigated, in part, to characterize the Guatemalan event by comparison to these more familiar quakes, in part, to study the sensitivity of the building, the Escuela De Niñeras, to records of different frequency content and, in part, to study the importance of vertical accelerations upon the response of the building in general and the infill in particular. The El Centro and Pacoima Dam records and their SDOF response spectra have been discussed above (section 4.2). The reader is reminded that the first 10 seconds of the original records were employed here, adjusted by direct proportion to have maximum peak accelerations of 0.5g. The actual Pacoima Dam record and not the commonly used "Derived Pacoima Dam" record was used here.

Displacement Response

The displacement response of the Frame + Slabs model and the Frame + Slabs + Soft Infill model to the horizontal components alone of the two additional records was computed and maximum response values were extracted from this computation. The displacement response of the Frame + Slabs + Soft Infill to the combined horizontal and vertical components of these two records was also determined and maximum values were recorded. The soft infill modeling alone was used to model the stiffness influence of the infill as it was thought to better characterize the behavior of the real structure.

Envelopes of the maximum roof displacements due to the horizontal components of the 0.5g El Centro and the 0.5g Pacoima Dam records are compared to the response to the Guatemalan event in figures B5.37 and B5.38. The responses of the Frame + Slabs + Soft Infill model to horizontal components alone and to the combined effect of horizontal and vertical components were very nearly identical for both records.

It is seen (Fig. B5.37) that the Frame + Slabs model of the building responded to all records in a manner apparently dominated by nontorsional translational behavior. This model, however, demonstrated a significantly greater sensitivity to the 0.5g El Centro event than the other two quakes. The response to the Guatemalan and the 0.5g Pacoima records was nearly identical. It will be recalled that the three fundamental modes of this model had periods of 0.604, 0.586 and 0.540 seconds. The SDOF response spectra considered above (Fig. B4.4) indicated a greater response to the El Centro record, in this period range, than to either the Pacoima Dam or Guatemalan records. In as much as it appears that the two fundamental translational modes (ie. modes 1 and 2) dominate the response of this model (having the largest participation factors) and these modes affect independent degrees of freedom one would expect the SDOF response spectra to provide a good estimate of the greater sensitivity to the El Centro event and such was the case.

One may expect, by the same reasoning, that the Frame + Slabs model would be somewhat more sensitive to the Pacoima event than the Guatemalan event yet the maximum responses to these two quakes is nearly identical. Upon closer examination it is seen that this model has significant body modes (modes 4 and 6) in the period range of 0.150 to 0.200 seconds, a range in which the Guatemalan record had a greater SDOF response than the Pacoima Dam record (yet still less than the El Centro).

The roof displacement envelopes for the second model - the Frame + Slabs + Soft Infill Model - on the other hand indicate the greatest sensitivity to the Pacoima Dam record, somewhat less sensitivity to the El Centro record and significantly less sensitivity to the Guatemalan record with all responses apparently dominated by torsional behavior. Physical arguments based upon SDOF response spectra become practically difficult as the torsional behavior couples the transverse and longitudinal displacement responses. In a very general sense, however, the infill has the affect of stiffening the structure shifting its response into a period range where the relative differences between the three records is not as great as it would be for the frame alone.

Member Force Response - Columns

The response of the critically damaged column, column 54, to the 0.5g El Centro and the 0.5g Pacoima Dam records was investigated. The Frame + Slabs model and the Frame + Slabs + Soft Infill model were considered to investigate the influence of the infill stiffness contribution on the behavior of this one column. The complete nature of the force redistribution throughout the entire structure due to the inclusion of the stiffness contribution of the infill was not investigated in detail (vis-a-vis the investigation of this force redistribution for the Guatemalan ground motion responses discussed in section 5.5.2).

As in the earlier studies the infill tended to have a significant affect upon the behavior of this column. Stress increase ratios (see section 5.5.2 above for the definition of this parameter) due to the influence of the infill for the five significant member forces considered are tabulated below for the Guatemala/189 record, the 0.5g El Centro record and the 0.5g Pacoima Dam record.

Table B5.7 Stress Increase Due to Infill - Column 54

	Ground Motion Record		
Member Force	**El Centro (0.5g)**	**Guatemala**	**Pacoima (0.5g)**
Axial	**-0.15**	**0.60**	**0.80**
Shear Y	**-0.75**	**-0.37**	**-0.04**
Moment XX	**-0.81**	**-0.48**	**-0.17**
Shear X	**0.49**	**1.01**	**1.41**
Moment YY	**0.95**	**1.69**	**2.15**

(It will be recalled that a zero ratio corresponds to no change in member force, a negative ratio corresponds to a decrease and a positive ratio an increase in member force.)

In all three cases the infill had a similar affect upon the column. The transverse full height infill panel adjacent to the column tended to inhibit distortion of the column in the transverse plane thereby reducing the shear, V_y, and moment, M_{xx}, associated with distortion in this plane. (Of course, these reduced member forces associated with the transverse distortion of the column are an artifice of elastic behavior in that failure of the infill was not allowed. The analyst must be careful in interpreting such results and look to the stress levels in the infill as well as the force levels in the columns; see section 5.5.2.) The longitudinal partial height panel adjacent to the column, on the otherhand, forces short column behavior making the column effectively stiffer (without, of course, increasing its strength) and consequently forcing it to carry greater lateral loads, that is higher shear, V_x, and higher moment M_{yy}.

The comparison offered by this table is especially relevant in that it provides a comparison of the relative sensitivity of the column to each of the three ground motion records. The table has been arranged to reflect a consistent trend in this relative sensitivity. It may be seen that the Pacoima Dam event is most effective in creating distress in this column in that it results in more column distortion in the plane of the full height panel (ie. less stress reduction, due to the infill, of V_y and M_{xx}) and causes greater stress increase due to the partial height longitudinal panel. The Guatemala record is less effective in causing distress in the column and the El Centro record is least effective.

In as much as the stress increase ratios are comparisons of responses of linear systems with and without the stiffness contribution of the infill panels they are independent of the intensity of the event and thus provide a relative measure of the sensitivity of the structural system to the frequency content (and duration) of the event.

The responses of the critical column to the horizontal ground excitation alone and the combined horizontal and vertical excitations were compared for both the 0.5g El Centro and the 0.5g Pacoima Dam records. The vertical component had an insignificant affect upon the response of this single column in both cases.

Member Force Response - Infill Panels

Again it is interesting to compare the responses to the two additional records with the Guatemalan response and also investigate the importance of vertical accelerations upon the infill panel force response. Attention was limited to four important infill panels that completely infilled transverse frames.

For these selected panels the maximum infill member forces due to both the 0.5g El Centro and the 0.5g Pacoima were, on the average, 75% greater than those experienced in the Guatemalan study with some maximum member forces two and one half times those from the earlier study. Unlike the column response discussed above the maximum member forces for the El Centro and the Pacoima records were very nearly identical. Again the vertical component of ground motion had an insignificant affect upon these maximum force responses.

Conclusion

It appears that this building was not sensitive to vertical accelerations, as far as the column and infill response is considered. It will be recalled that the analyses were based upon mode superposition using only the first 18 modes that appeared to be insufficient to capture the complete nature of response to vertical excitation (see section 5.4). Although an objective was set to study the importance of vertical deflections of the transverse, heavily loaded, cantilevered beam upon infill force response the details of such behavior were not investigated. (In as much as the constraint approach modeling will include this aspect of behavior this objective became of secondary concern.) The maximum displacement response and maximum infill force response were consistent showing, roughly, twice the sensitivity to the 0.5g El Centro and the 0.5g Pacoima Dam records as the Guatemala/189 record. The evidence of column force response indicates, on the otherhand, that the building was most sensitive to the Pacoima Dam record, somewhat less sensitive to the Guatemala/189 record and least sensitive to the El Centro record.

6. Conclusion

6.1. Summary

This study reviewed the construction and post earthquake evidence of the seismic behavior of a building, the Escuela De Niñeras, then investigated its dynamic character through a series of detailed linear elastic analytical studies. The building is typical of many moderate size buildings that utilize structural frames infilled with masonry (ie. frame-infill structural systems) and suffered seismic damage that has become characteristic of this type of system (eg. "captive" column and infill panel shear failures). The structural contribution of the infill to the frame was modeled, analytically, by a constraint approach developed as part of the study (see Part A of this report) and the influence of the infill upon the structural response to different earthquake excitations was considered in detail. The suitability of the proposed constraint approach to model frame-infill interaction was critically evaluated.

6.2. Conclusions

Infill, even of relatively light construction*, may have a primary influence upon the dynamic character and seismic response of a structural frame as the infill will tend to significantly stiffen and strengthen the frame. If ignored, such sources of strength and stiffness may lead to behavior completely unanticipated by the structural designer that may well be catastrophic. The response of a frame-infill system must be expected to be altogether different from that of the frame alone. Frame-infill system behavior may not, then, be reasonably modeled by a bare frame structural idealization, the structural contribution of the infill must be included.

Infilling provides construction economy, use advantages and an attractive means to stiffen and strengthen structural frames but, unfortunately, this potential is often compromized by the brittle strength characteristics of commonly used infill construction as well as the brittle (shear)

* A partition constructed of wood studs sheathed with decorative plywood forced a shear failure of a principle column of transverse frame 2 of the building studied.

column behavior induced by frame-infill interaction. Such potential may, however, be realized through improved infill construction methods (see Klingner's encouraging results [B13]) and through rational analysis of frame-infill system behavior.

One analytical technique, suitable for modeling the static as well as dynamic linear elastic behavior of complex frame-infill systems has been presented here. This approach, a constraint approach, is theoretically consistent and, apparently, effective yet nevertheless has limitations. Some of these limitations are common to linear elastic analytical methods in general and some are specific to the approach presented here. These limitations are related to;

i. uncertainties in modeling the structure,

ii. the completeness of the analytical model,

iii. uncertainties in modeling the load (excitation),

iv. theoretical approximations and simplifications,

and

v. problems in interpreting analytical results.

Uncertainties in modeling the structure (ie. in elastic analysis, mass, stiffness and damping uncertainties) must, properly, be identified in order to reasonably interpret analytical results. The nature of these uncertainties are, however, specific to the building under study and may or maynot be significant. Bound studies, if carefully formulated, may define their significance. Nondimensional analysis may serve to limit and generalize such studies.

The structural analyst is critically limited by the completeness of his model. At the global level he cannot expect to capture 3D response phenomena with a 2D model of the structure. At the local level he cannot expect to capture a two dimensional displacement field (eg. infill panel displacement field) with a single degree of freedom element (eg. an "equivalent" strut). Simplified models carry not only the penalty of loss of accuracy (usually) but may not be able to capture phenomena that may prove to be critical. The Escuela De Niñeras was sufficiently complex to demand a complete 3D modeling, it is believed that most buildings are as complex.

The uncertainty in modeling the **expected** seismic excitation for a proposed building design is always significant, in part, because of the uncertain nature of future events and, in part, because the experience of this study as well as others suggests that most buildings are sensitive to the characteristics of the seismic excitation. The structural analyst seeks to find the excitation, that has a reasonable probability of occurrence, that will drive the structure to its most critical response. It appears that there is presently no more reliable means to do this than to consider the response of the structure to several probable design excitations that cover the range of variations of the dynamic characteristics of the expected ground motions. Indeed there is no reason to expect that only one "most critical" response exists.

The analyst is limited by multiple theoretical approximations that must be kept in mind. Important here are the assumptions peculiar to the constraint approach and the infill elements developed from this approach. The approach is based upon the assumption that the infill panels are constrained to deform to the form of deformation of the surrounding frame (ie. to the form of the beam shape functions here). Although this approximate assumption appears to provide an accurate means to model the stiffness contribution of the infill to the frame it does not allow a **detailed** estimation of framing member force variation or infill stress field*. Rather it provides a mean estimation of member forces and infill stress local to the infilled frame that, albiet useful, may well underestimate the extreme values that can be expected to occurr (see section 6 of Part A of this report).

Linear elastic techniques do not include inelastic effects that are inevitably a part of strong motion seismic response of structures. Consequently, when interpreting the results of linear elastic analysis individual member forces (and displacements) may not be taken literally, rather they must be considered in terms of available member capacity, time of occurrence and in relation to other member force responses. In particular, when interpreting analytical results of frame-infill system response modeled with the proposed linear elastic infill element it is

* An approximation to the "exact" stress field may, however, be obtained through the development of a nodal-displacement-to-stress-field transformation matrix in a manner very similar to the development the infill stiffness matrix presented here.

necessary to consider column member force levels in terms of not only the column's capacity but also in relation to adjacent infill panel stress levels, capacities and time variation of both (see discussion of member force response section 5.5.2).

The possibility of a correlation between the predicted maximum nominal infill shear stress and the observed damage to infill panels for the building under study was considered. The correlation proved to be positive although the amount of data considered was too meager to provide confidence. Nevertheless, it is believed that damage limit states for infill panels may be established by limiting (nominal) infill shear stress (or strain, possibly) to appropriate allowable stress (or strain) levels. Possible allowable stress levels were discussed (see section 5.5.2). The load level(s) for which such limits should be considered depend on the type of analysis used (ie. linear or nonlinear) and upon the structural importance of the infill. If the infill is part of the primary structural system it should sustain moderate quakes without damage while if it is nonstructural in nature (ie. does not contribute significantly to the stiffness of the primary structural system) then it need sustain only minor quakes without damage.

6.3. Implications For Design

Infill must be considered by the structural designer of a building even if it is of apparently light construction. The structural designer may choose to;

1. separate the infill from the surrounding frame to inhibit its structural contribution to the lateral resistance of the structure and thereby avoid the problems so characteristic of conventional frame-infill system response,

2. stiffen the structural frame system sufficiently (eg. through the use of shear walls or diagonal bracing) to inhibit deformation and thus limit frame-infill interaction and thereby avoid the problems characteristic of conventional frame-infill system response,

or,

3. employ the infill directly to stiffen and strengthen the structural frame and thereby gain the construction and use advantages offered.

The first option demands careful attention to infill support details so that forces (inertial as well as others) normal to the panel are resisted and yet in-plane frame-infill interaction is inhibited. Clearly such details must remain effective throughout the life of the structure and therefore may require maintenance and inspection during this time to provide this assurance. Such details are likely to compromise the construction and acoustical advantages characteristic of the conventional direct use of infilling. This approach of separation does, however, avoid the analytical problems of frame-infill response analysis.

The second and third options are different only by degree. One may achieve the effect of the second option (ie. of designing a stiff primary system to avoid frame-infill interaction) by employing a "soft" yet ductile infill construction. The second option suggests, however, the possibility of utilizing infilling as a second line of resistance that may become effective should the primary structural system soften during an extreme seismic event. For both options it is important to;

1. use infilling rationally;
    i. infill in a regular and symetrical manner in plan,
    ii. infill in a regular and continuous manner in section, avoiding abrupt changes in infill/frame stiffness (eg. avoid a nonuniform distribution of openings in infill panels over several stories of a single frame),
    iii. avoid partial height infilling,
2. design the frame-infill subassemblage so that failure behavior will be ductile (see Klingner [B13]),

and,

3. include the stiffness contribution of the infill in the design analysis of the proposed structure and consider a set of design earthquakes of different, yet probable, characteristics as the nature of the critical design earthquake may not be obvious.

The designer may choose to separate specific infill panels from the surrounding frame (eg. partial height panels) to gain a more advantageous distribution of "active" infill panels.

The importance of infill contribution to frame-infill system response should alert the designer to consider other seemingly unimportant secondary structural and nonstructural elements in design considerations of the building as well as any modifications to the building in the future.

Drift Limits

The need to identify suitable damage limit states as design criteria to avoid nonstructural damage during minor earthquakes and to limit nonstructural damage while avoiding structural damage during moderate earthquakes has been recognized by the structural engineering profession. Drift limits have been proposed to answer this need, yet, if the stiffness contribution of infilling is included in the analysis of frame-infill system seismic response, predicted drift will, necessarily, be limited in those areas that are infilled -- the areas that may suffer the greatest damage*. Consequently, conventional drift indices have little meaning in such analyses.

Drift limit proposals have been postulated upon the belief that partitions do no significantly contribute to the stiffness of the primary structural system thus drift determined from the analysis of the primary structural system alone may be expected to be correlated with damage to partitions (and, perhaps other nonstructural elements as well). This study has attempted to show that infill partitions may, however, contribute significantly to the stiffness of the primary structural system and thus the analysis of the primary system alone may have little

* In the building studied infilling had the effect of limiting drift local to the infill, drift in other areas was greater (see Table B5.5), yet the damage was concentrated in the infilled areas. For this building, then, there was a negative correlation between damage and drift.

relation to the behavior of the combined primary-secondary (eg. frame-infill) system. When partitions significanly modify the system response, as infill partitions do, then conventional drift limits are no longer appropriate as they will not be well correlated with the damage to these partitions and as such can not provide a suitable criteria to limit this damage.

A damage limit state must be understood to include;

1. limiting values of response parameters that are well correlated with (specific types of) damage,

as well as,

2. the load level(s) and load characteristics underwhich these limits are to be considered.

Nominal infill shear stress has been proposed, in this report, as a suitable damage limit state response parameter for infill panels. The load level(s) under which nominal infill shear stress is to be limited depends upon the structural importance of the infill (ie. the significance of the stiffness contribution of the infill to the primary structural system). If the infill is of a non-structural nature it need sustain only minor quakes without damage, while if it is structural infill it should sustain higher load levels without damage. Limiting values of this parameter have been proposed (section 5.5.2).

6.4. Recomendations For Research And Development

Frame-infill system response behavior is not yet well understood. If infilling structural frames is to become a reliable construction technique additional experimental as well as analytical research is needed to develop a more complete understanding. With this understanding new construction techniques may be developed and better analytical methods for the prediction of frame-infill response may be formulated. There should be an emphasis on improving the inelastic response of these systems (ie. developing construction details to improve the ductility and hysteretic behavior of the frame-infill subassemblages) and developing analytical methods to model this inelastic response.

Experimental investigations should consider realistic frame-infill systems that are supported and loaded in a manner that simulates conditions that may be expected to occurr in actual structures. The hysteretic behavior of completely infilled frame systems, partially infilled frame systems, multiple and mixed partially and completely infilled frame systems should be considered. The reliability of construction details to separate the infill from the surrounding frame should be evaluated, in-plane as well as out-of-plane behavior will be important. The mechanical characteristics of commonly used infill materials and construction need to be determined for purposes of analysis and determination of the capacity of the infill to resist loads. Means to repair damaged infill should be investigated and suitable response parameters should be correlated to the nature and degree of observed damage.

Analytical capabilities may be expanded by a direct application of the proposed constraint method to infill panels of nonrectangular geometry, infill panels with openings, infill panels constructed of nonisotropic materials (eg. masonry), and other constraint assumptions not considered here. Analytical and experimental studies may be needed to evaluate the suitability of such developments. The constraint method may also be extended to consider aspects of inelastic behavior by consideration of nonlinear infill material behavior as well as nonlinear constraint conditions. A means to estimate, in greater detail, the member force variation and infill stress field local to a single frame-infill subassemblage is needed. Other means to model the inelastic behavior of frame-infill systems may be developed upon the basis of an improved understanding of the mechanisms of response.

The structural importance of other secondary structural and/or nonstructural elements should be investigated. The influence of non-infilled partitions, stairways and precast "architectural" elements (eg. fascia) among others may be important. The contribution of these elements may be considered, analytically, using a constraint approach as well, this should be investigated. The structural contribution of flat slabs, joist-slab systems and waffle slabs may be modeled also using a constraint approach, this, too, should be investigated.

As other secondary structural and nonstructural elements enter into structural considerations then their resistant capacities and damageabilities need quantification. Thus means to estimate these capacities for each new important class of elements need to be developed. Damage limit states for these elements need to be established and guidelines for the use (eg. distribution and details) of such secondary structural and nonstructural elements should be formulated.

The whole matter of drift limits to limit nonstructural damage should be reevaluated. What measures of drift, if any, are suitable and when are they applicable? What load level(s) and what types of analysis should be considered in their application? Are other response parameters more suitable?

If the structural designer is to use existing analysis programs to their capabilities (eg. 3D analysis and inelastic analysis) means to facilitate preparation of input as well as checking and interpreting output will have to be developed. "Preprocessors" and "postprocessors" serve this need, graphical capabilities are desirable in such processors, but this is not enough. The designer will need some guidance on which response parameters should be considered and how they may effectively be reviewed. A emphasis must be placed upon presenting a sense of structural behavior rather than simply listing maximum values of selected response parameters. The three dimensional representations of mode shapes and member force distribution, used in this study, provide a first attempt to do this, other means of presentation should be investigated.

If the structural designer is to consider the response of a structural idealization to several design earthquakes, as proposed here, guidelines for selecting sets of design excitations that will provide comparable response results need to be provided. In the present study ground motion records were scaled so as to have SDOF linear elastic response spectra of roughly similar amplitude. For linear elastic dynamic analysis it may prove useful to consider a set of different excitations scaled so as to produce identical maximum responses of a single key response parameter (eg. roof displacement or shear in a critical element) or, perhaps, a scalar norm of response such as maximum total base shear or maximum total strain energy. In areas with reliable and detailed seismic history it may be possible to establish a set of probable design excitations to be considered. There is a need;

1. to establish the importance of considering a set of design seismic excitations (possibly, an envelope of several response spectra for preliminary design and a set of ground motion records for design development and detailing),

2. to investigate the means to select a set of comparable design earthquakes (or an envelope of design earthquake response spectra for preliminary design purposes),

and,

3. to investigate the means to formulate meaningful comparative analyses using sets of design earthquakes and to interpret such analyses to achieve better design.

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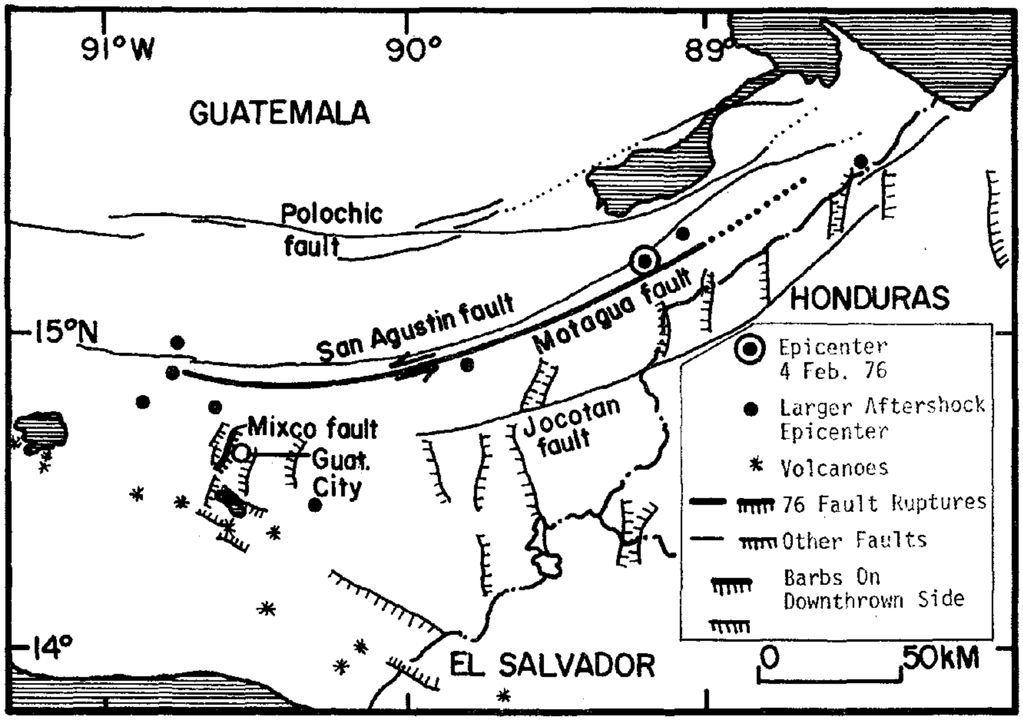


FIG. B1.1 GUATEMALAN FAULT SYSTEMS (after Plaker)

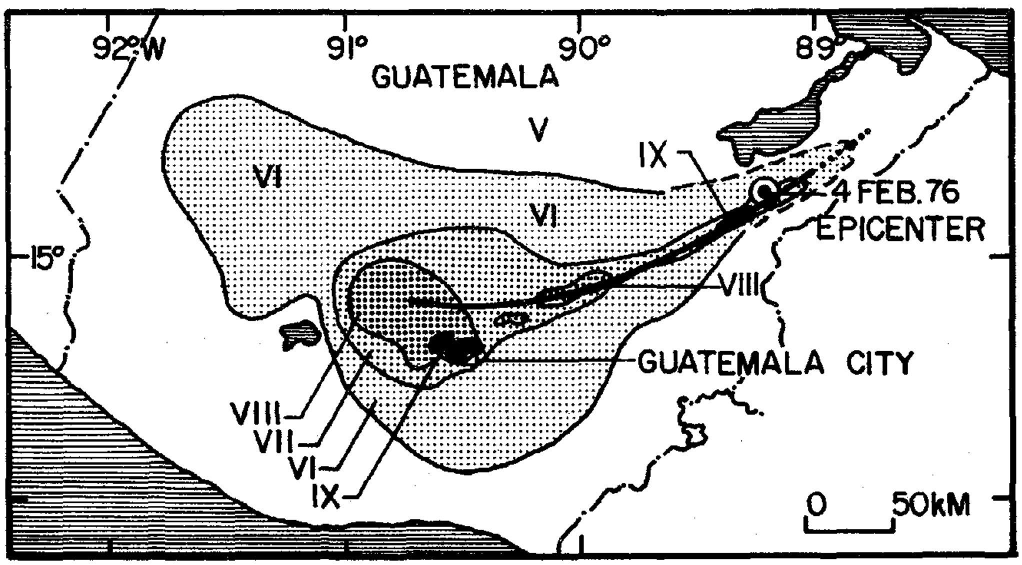


FIG. B1.2 DISTRIBUTION OF MMI INTENSITIES OF FEBRUARY 1976 GUATEMALAN EARTHQUAKE (after Espinosa)

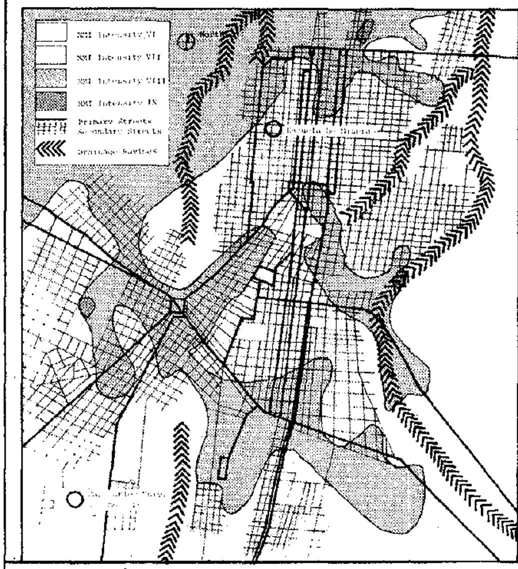


FIG. B1.3 DISTRIBUTION OF MMI IN GUATEMALA CITY - 76
(after Espinosa)

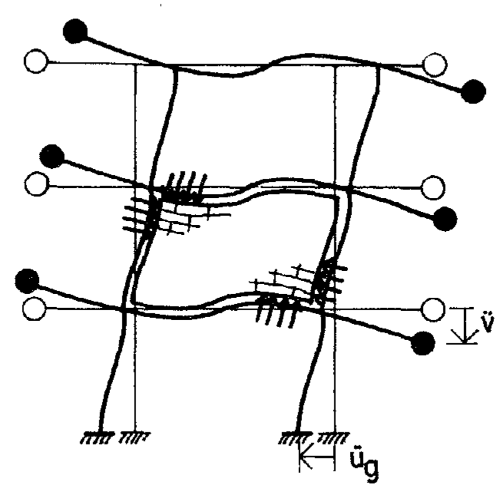


FIG. B1.4 PANEL DISTRESS AND COUPLED VERTICAL MOTION

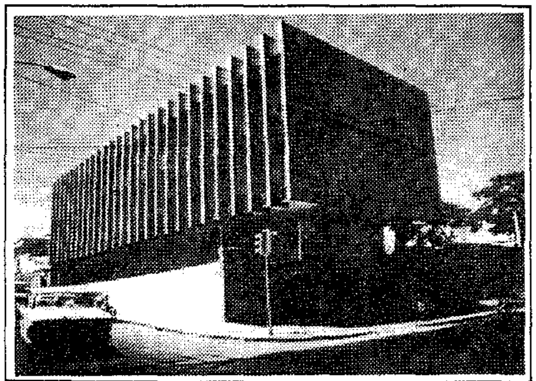


FIG. B2.1 SOUTHEASTERN VIEW OF ESCUELA DE NIÑERAS

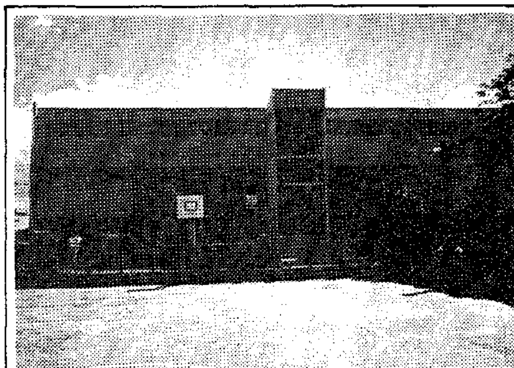


FIG. B2.2 NORTHERN VIEW OF ESCUELA DE NIÑERAS

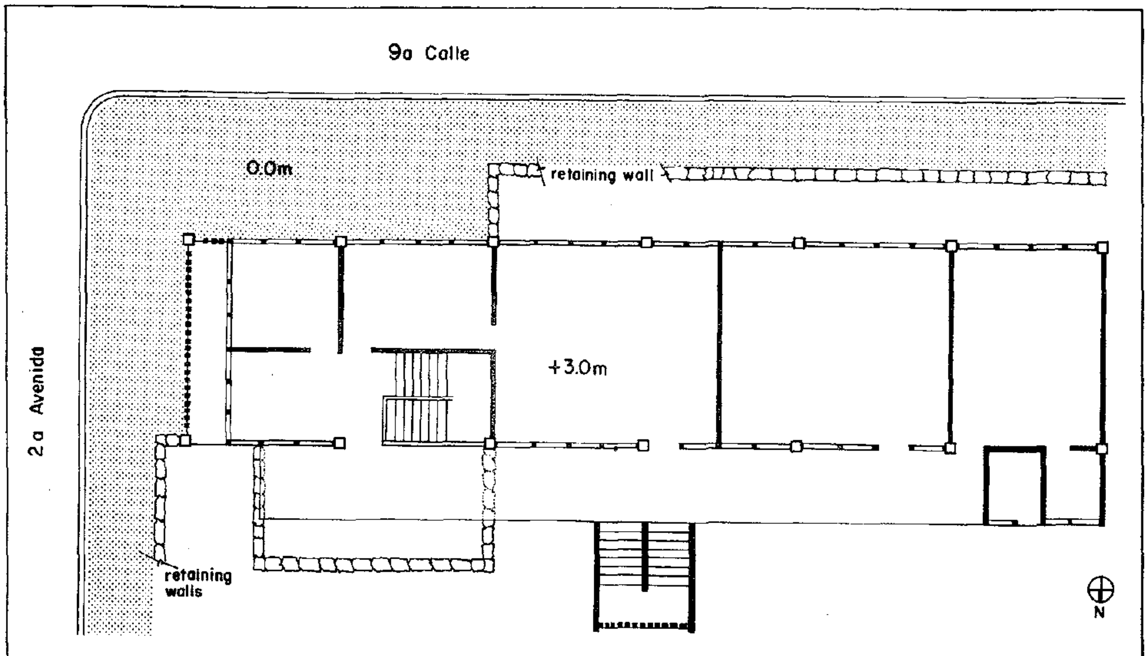


FIG. B2.3 ARCHITECTURAL PLAN OF THE ESCUELA DE NIÑERAS
GROUND FLOOR

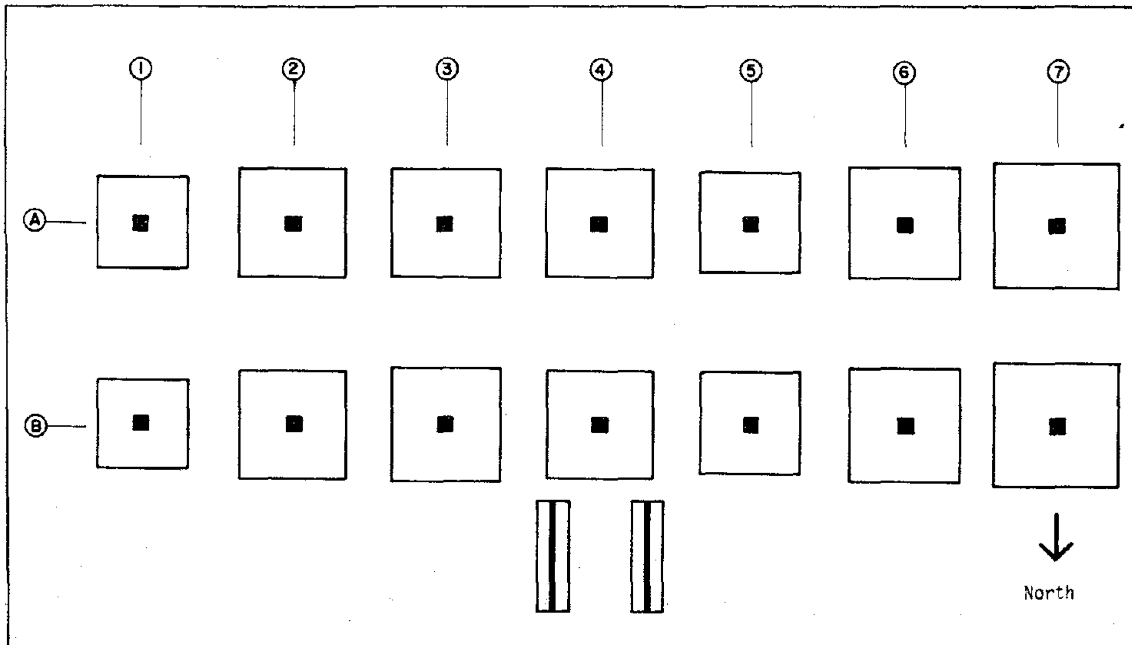


FIG. B2.4 STRUCTURAL PLAN OF THE ESCUELA DE NIÑERAS
FOUNDATION

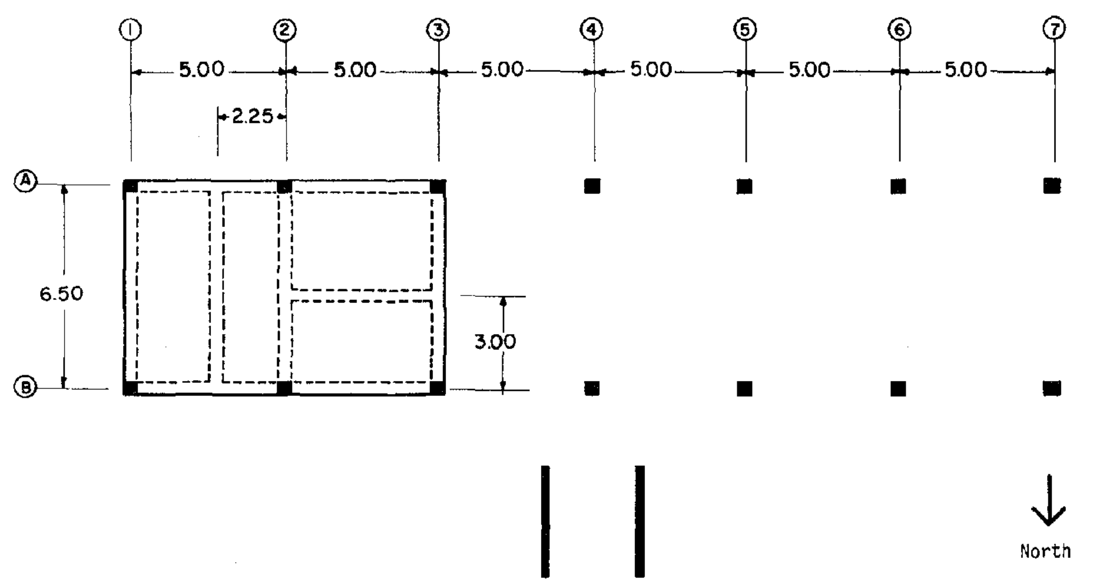


FIG. B2.5 STRUCTURAL PLAN OF THE ESCUELA DE NIÑERAS
BASEMENT

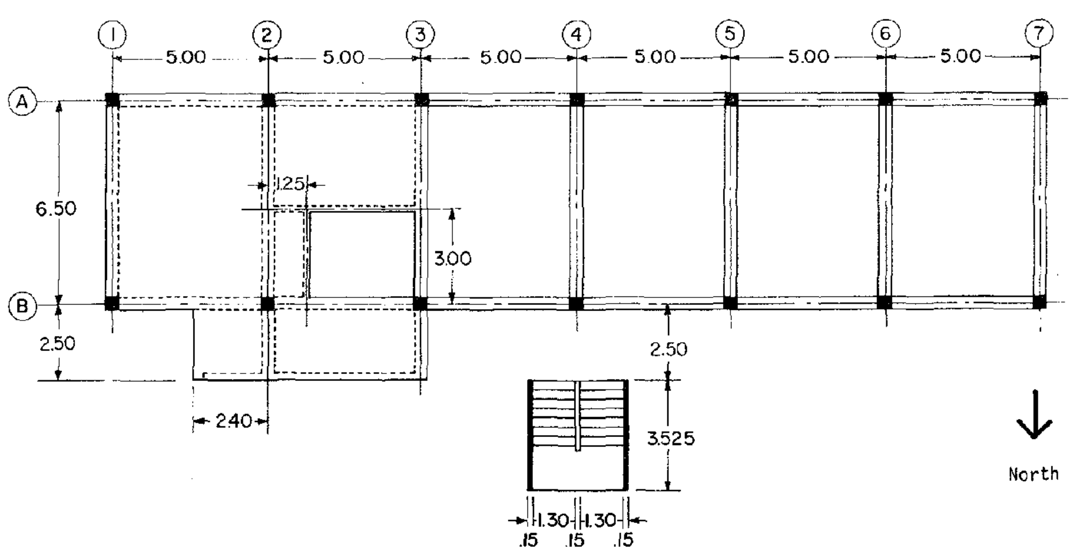


FIG. B2.6 STRUCTURAL PLAN OF THE ESCUELA DE NIÑERAS
GROUND FLOOR

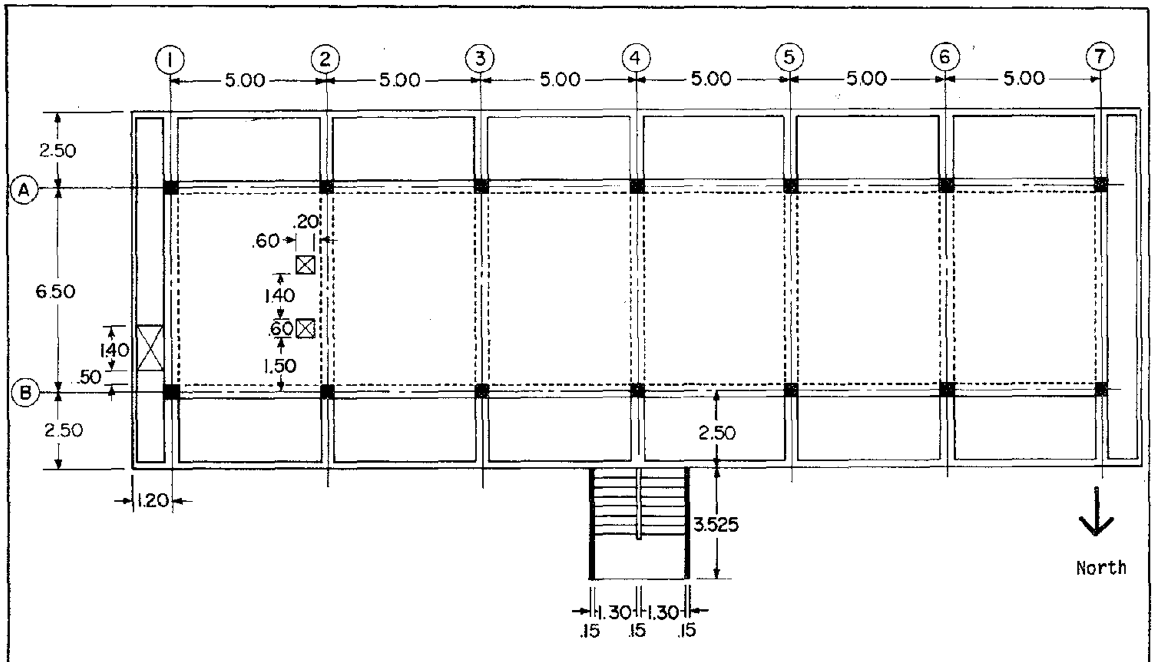


FIG. B2.7 STRUCTURAL PLAN OF THE ESCUELA DE NIÑERAS
SECOND AND THIRD FLOOR

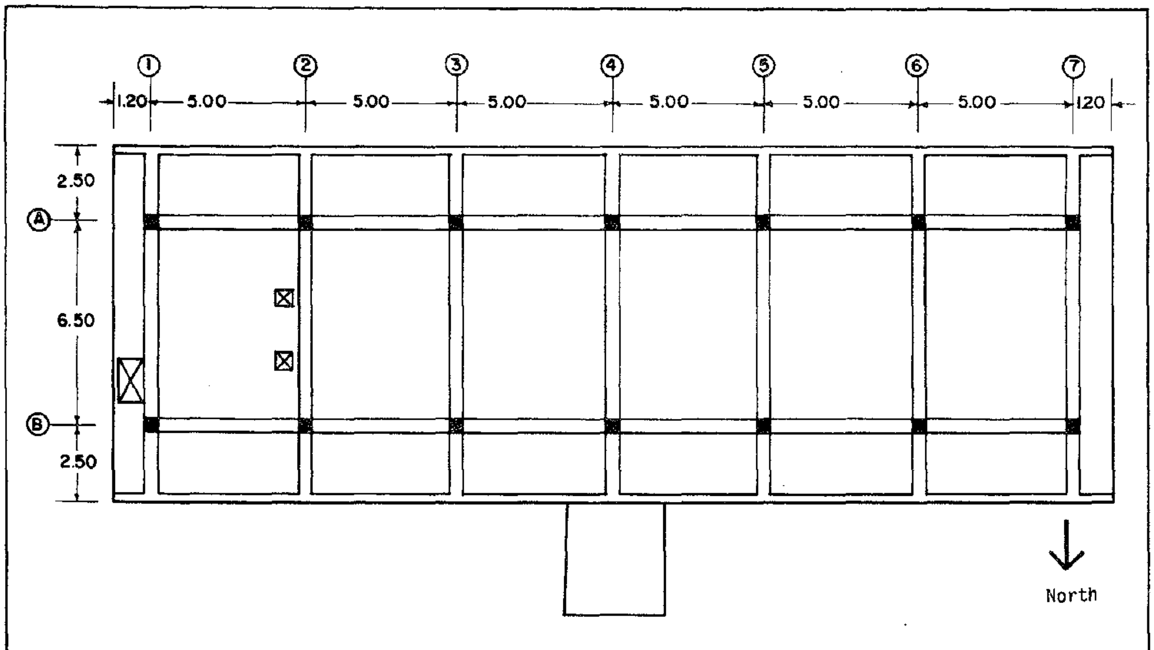


FIG. B2.8 STRUCTURAL PLAN OF THE ESCUELA DE NIÑERAS
ROOF

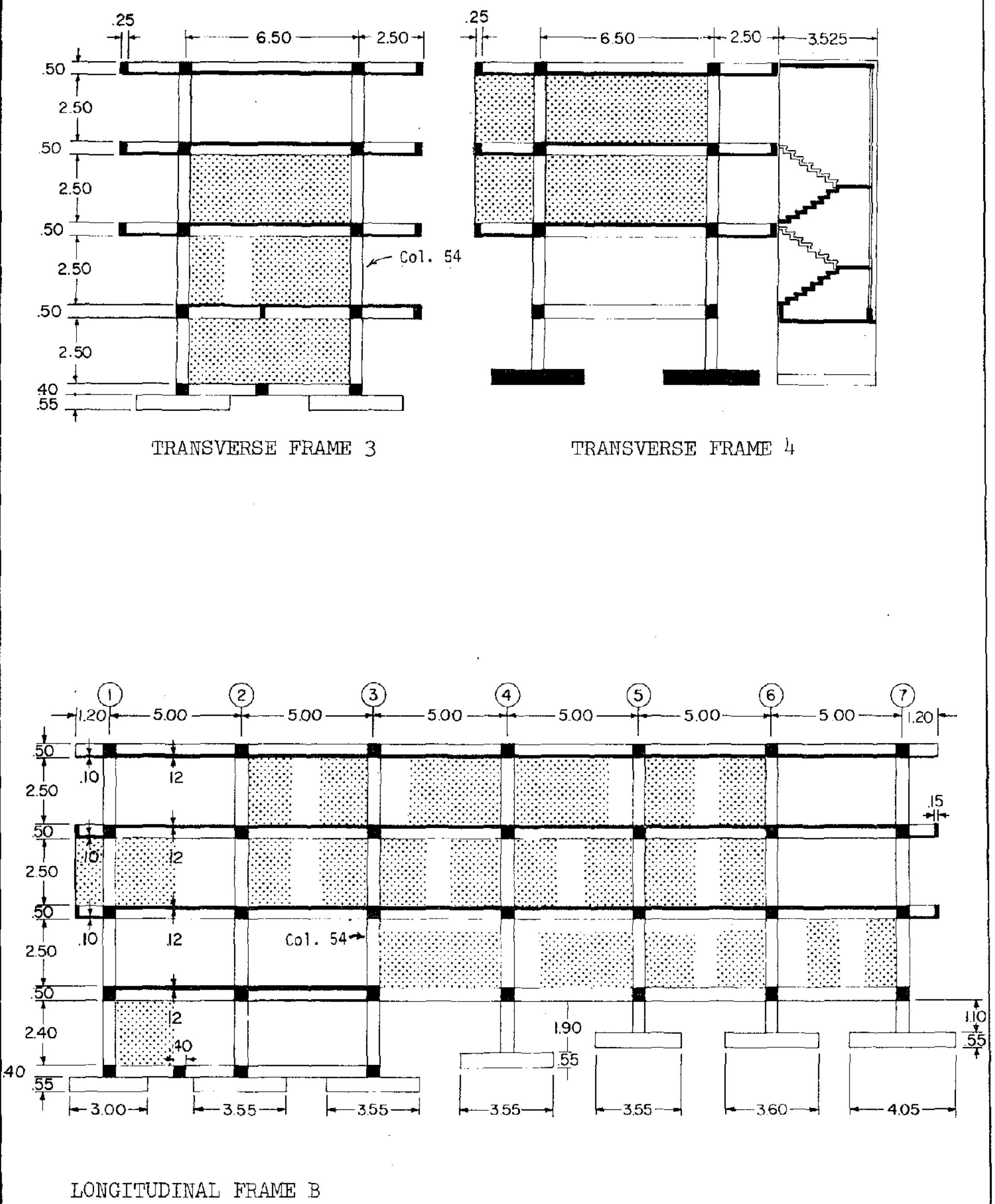


FIG. B2.9 STRUCTURAL SECTIONS OF THE ESCUELA DE NIÑERAS

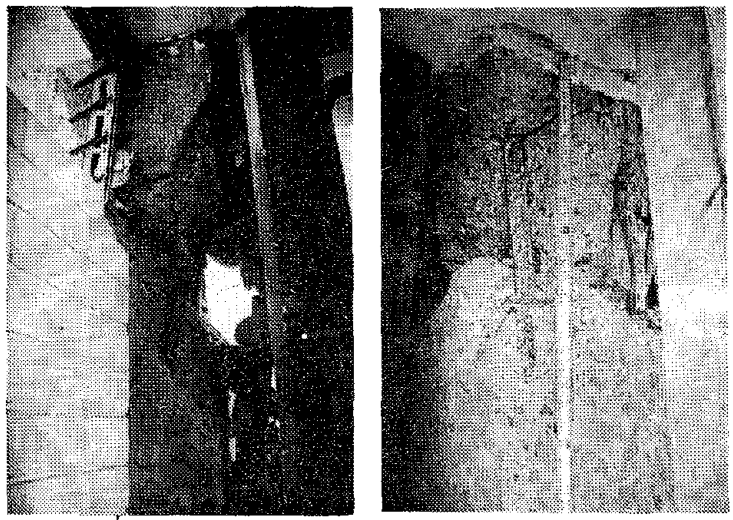


FIG. B3.1 CAPTIVE COLUMN SHEAR FAILURE
(Column "54")

FIG. B3.2 BENDING FAILURE AT COLUMN TOP

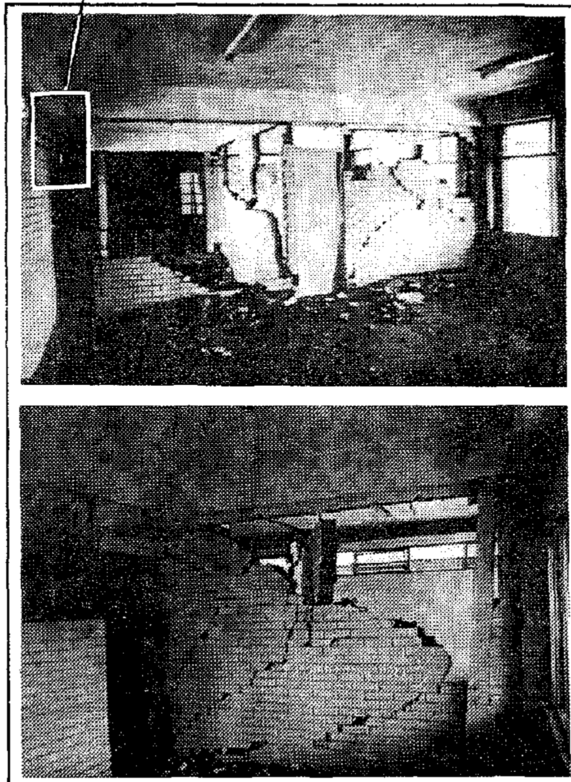


FIG. B3.3 WALL DAMAGE AT TRANSVERSE FRAMES 2 & 3

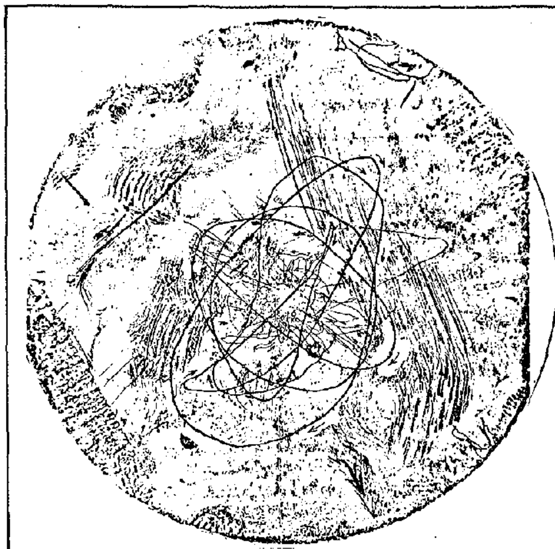


FIG. B4.1 ORIGINAL SEISMOSCOPE TRACE OF GUATEMALA 2/4/76 QUAKE

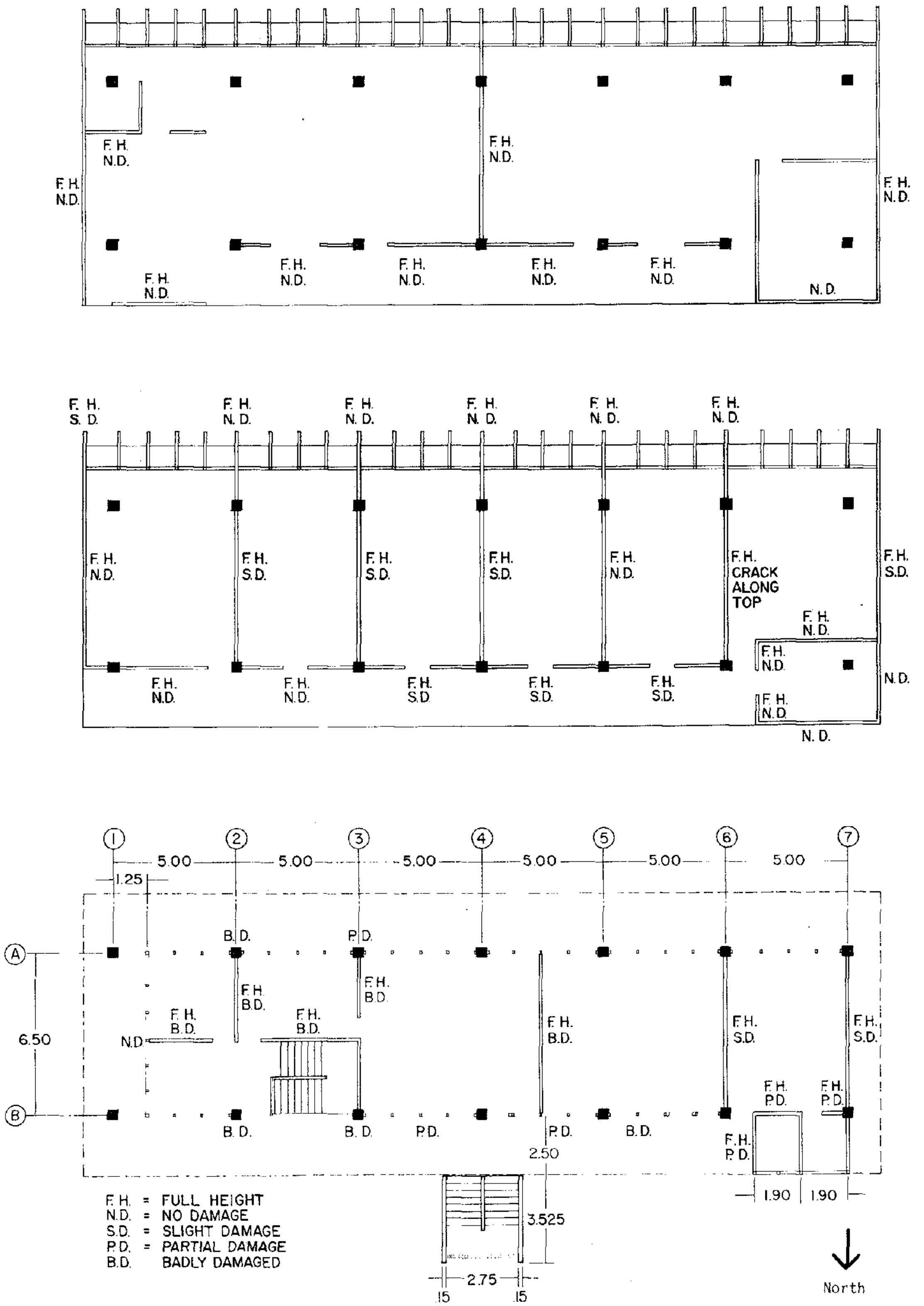


FIG. B3.4 ESCUELA DE NIÑERAS DAMAGE SURVEY

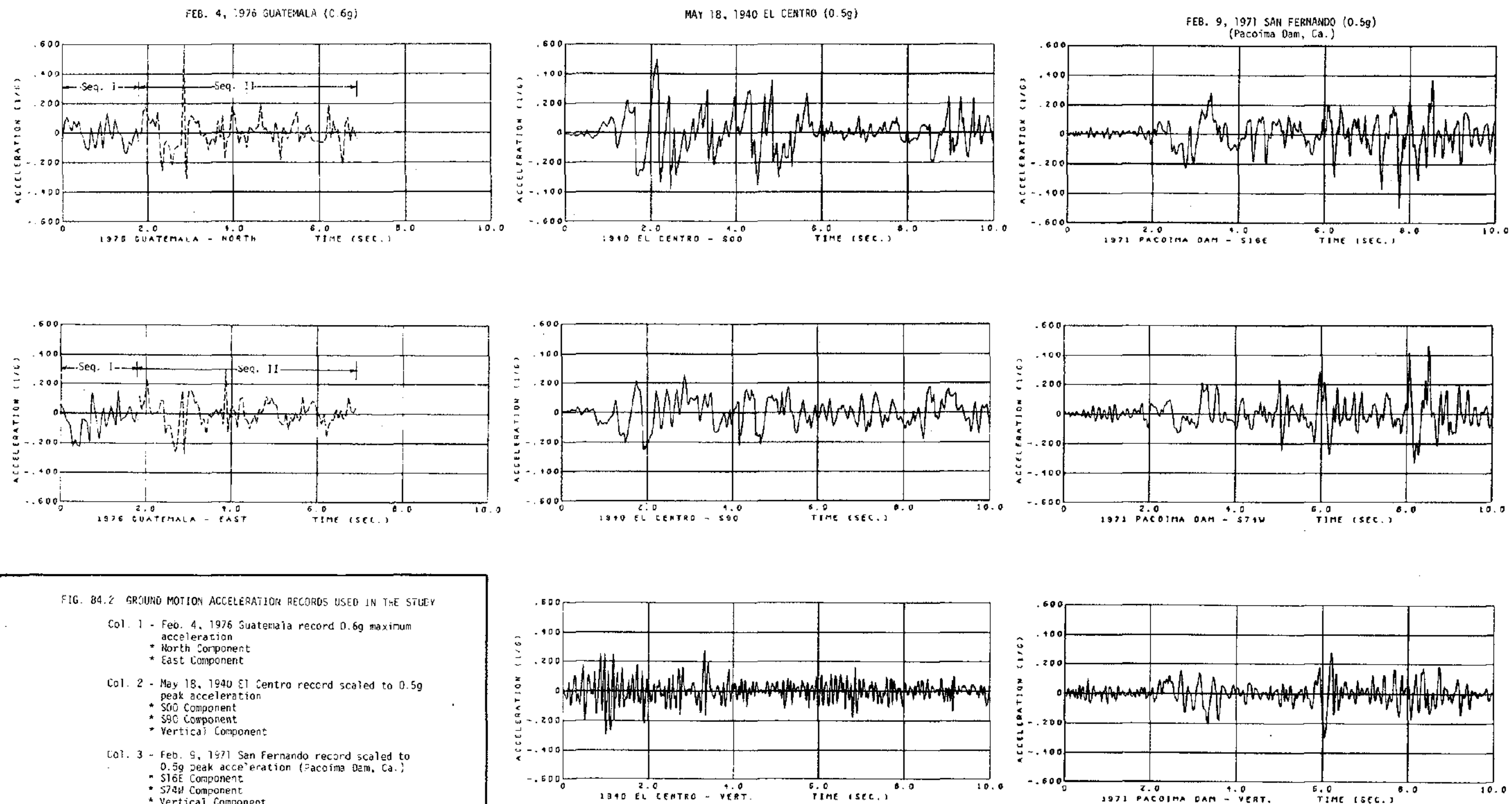
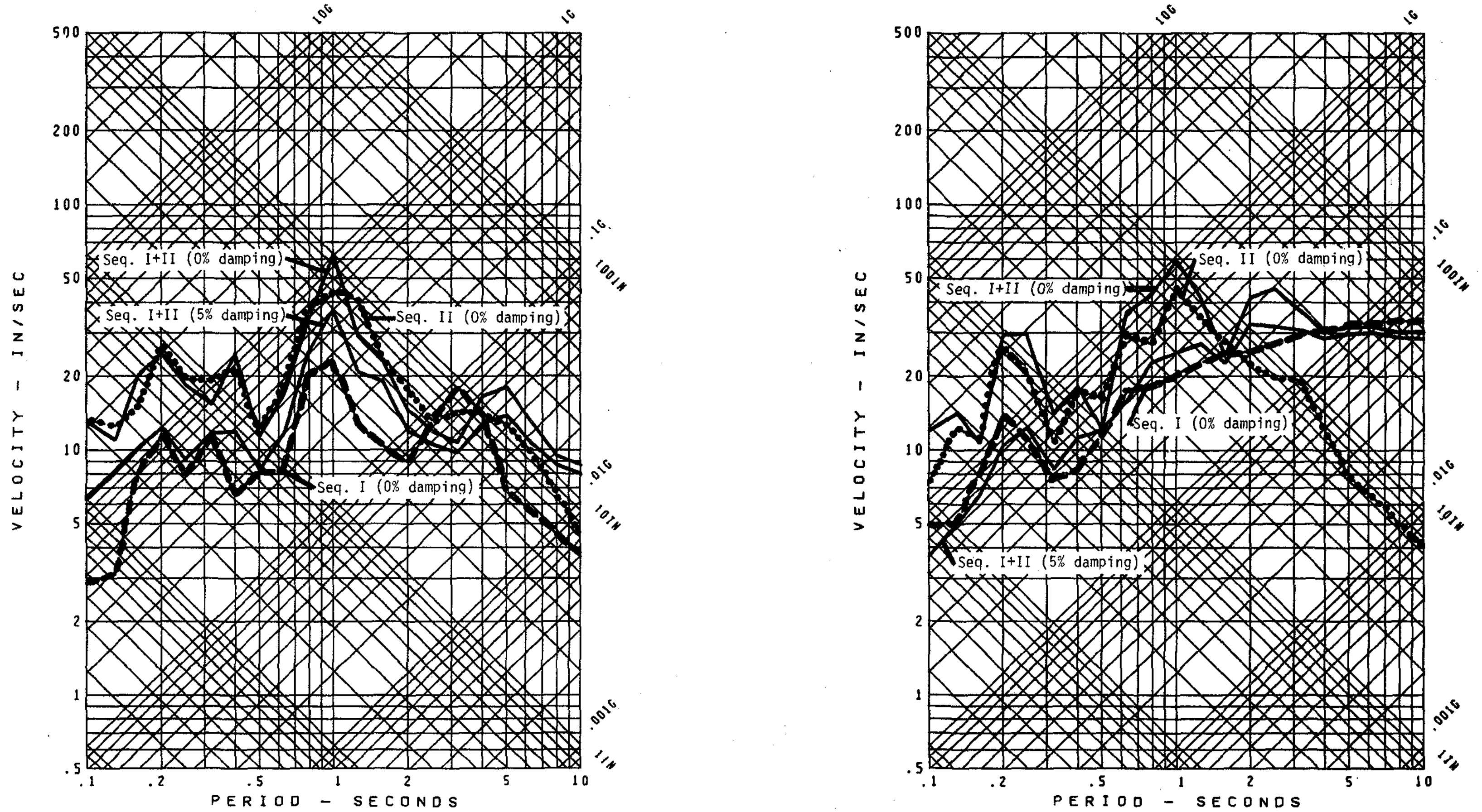


FIG. 84.2 GROUND MOTION ACCELERATION RECORDS USED IN THE STUDY

Col. 1 - Feb. 4, 1976 Guatemala record 0.6g maximum acceleration
* North Component
* East Component

Col. 2 - May 18, 1940 El Centro record scaled to 0.5g peak acceleration
* S00 Component
* S90 Component
* Vertical Component

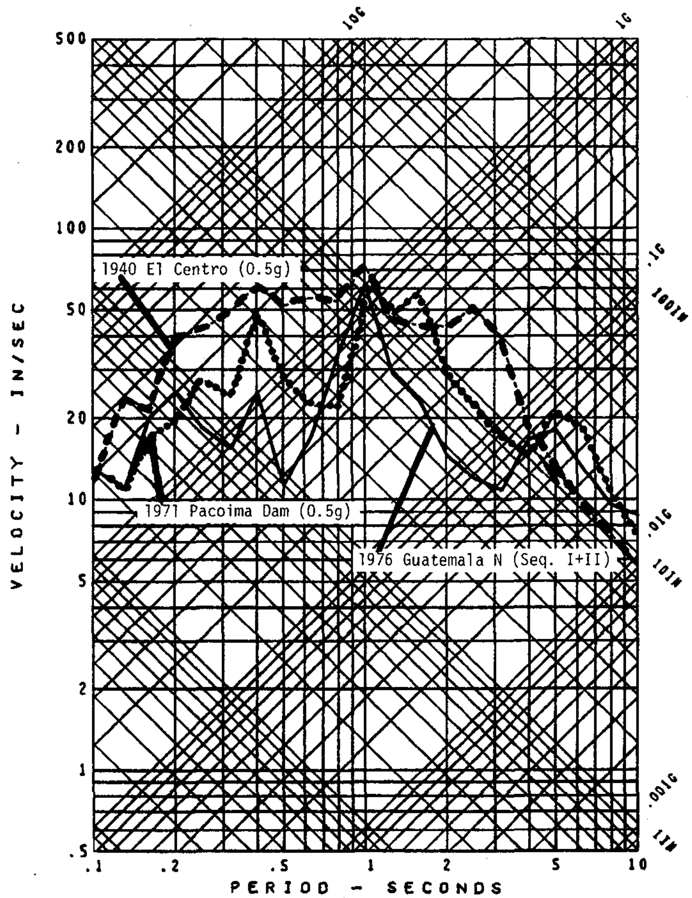
Col. 3 - Feb. 9, 1971 San Fernando record scaled to 0.5g peak acceleration (Pacoima Dam, Ca.)
* S16E Component
* S74W Component
* Vertical Component



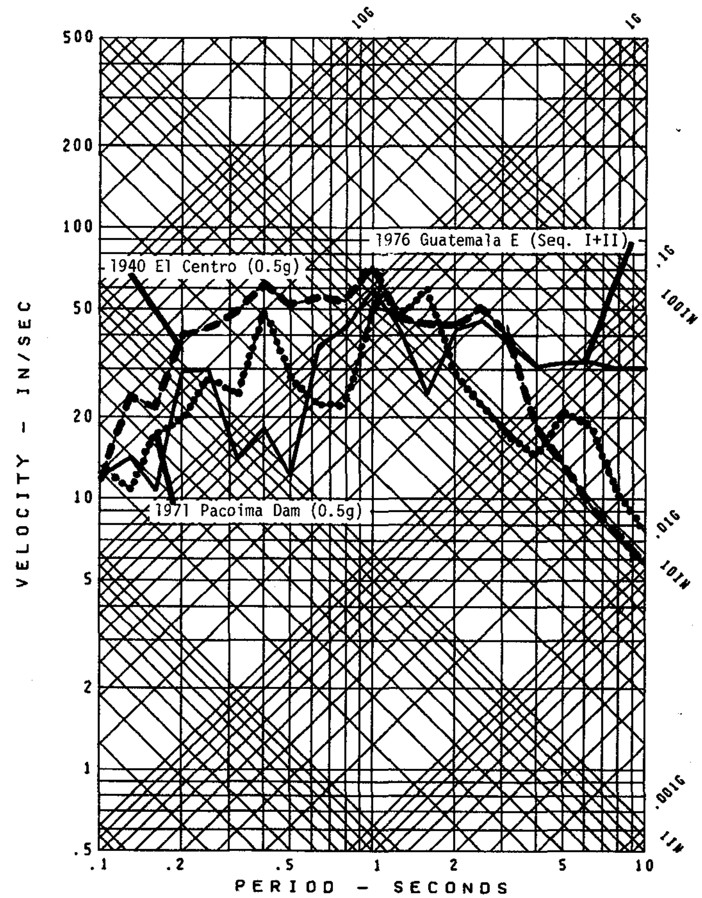
1976 GUATEMALA/189 NORTH
ELASTIC RESPONSE SPECTRA

1976 GUATEMALA/189 EAST
ELASTIC RESPONSE SPECTRA

FIG. B4.3 SDOF ELASTIC RESPONSE SPECTRA FOR GUATEMALA/189 GROUND MOTION RECORD



GUAT.-N, EL CENT.-S00, PACOIMA-S16W
ELASTIC RESPONSE SPECTRA



GUAT.-E, EL CENT.-S00, PACOIMA-S16W
ELASTIC RESPONSE SPECTRA

FIG. B4.4 COMPARISON OF GUATEMALA/189, EL CENTRO (0.5g), AND PACOIMA DAM (0.5g) RESPONSE SPECTRA

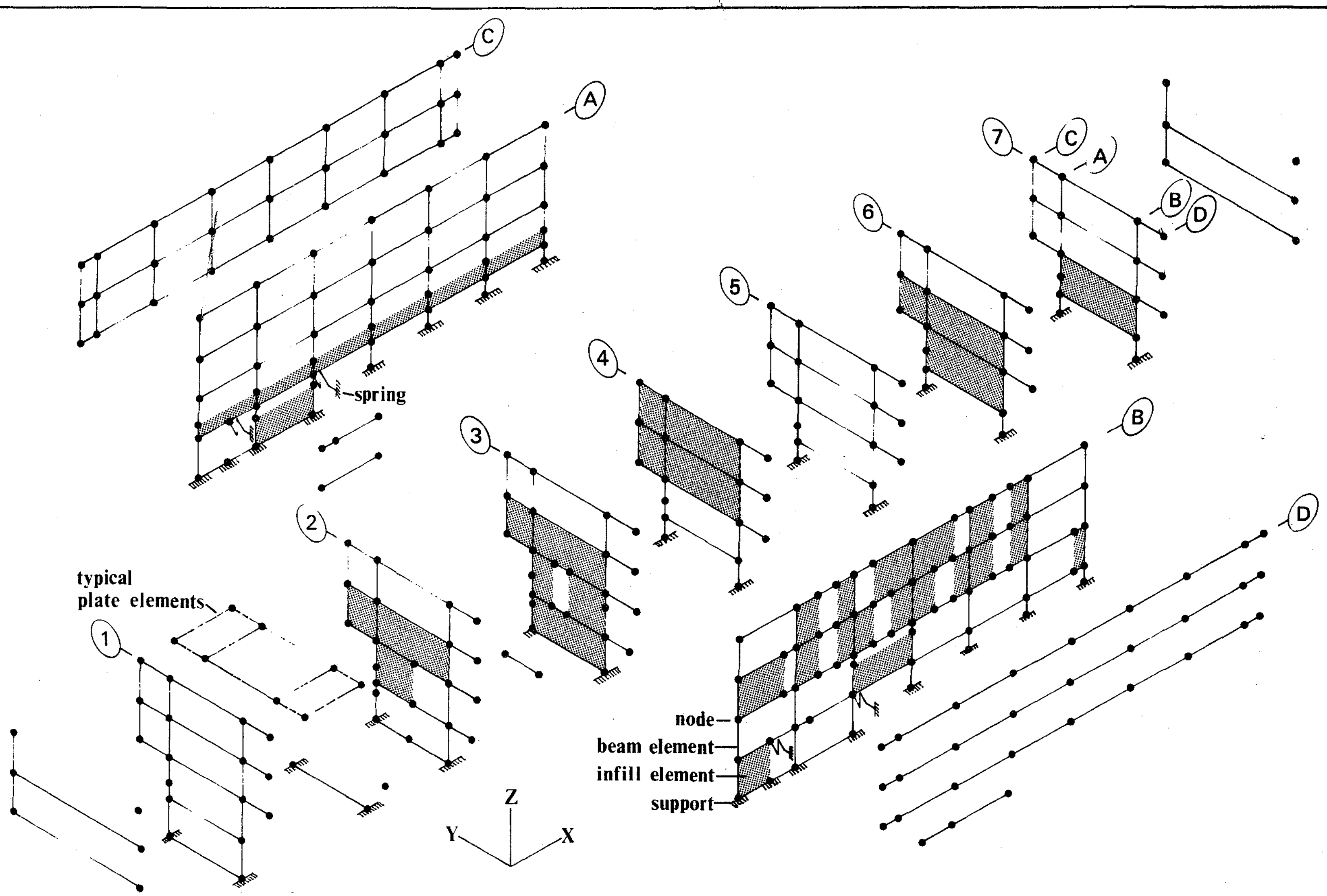


FIG. B5.1 FINITE ELEMENT MODEL OF THE ESCUELA DE NIÑERAS

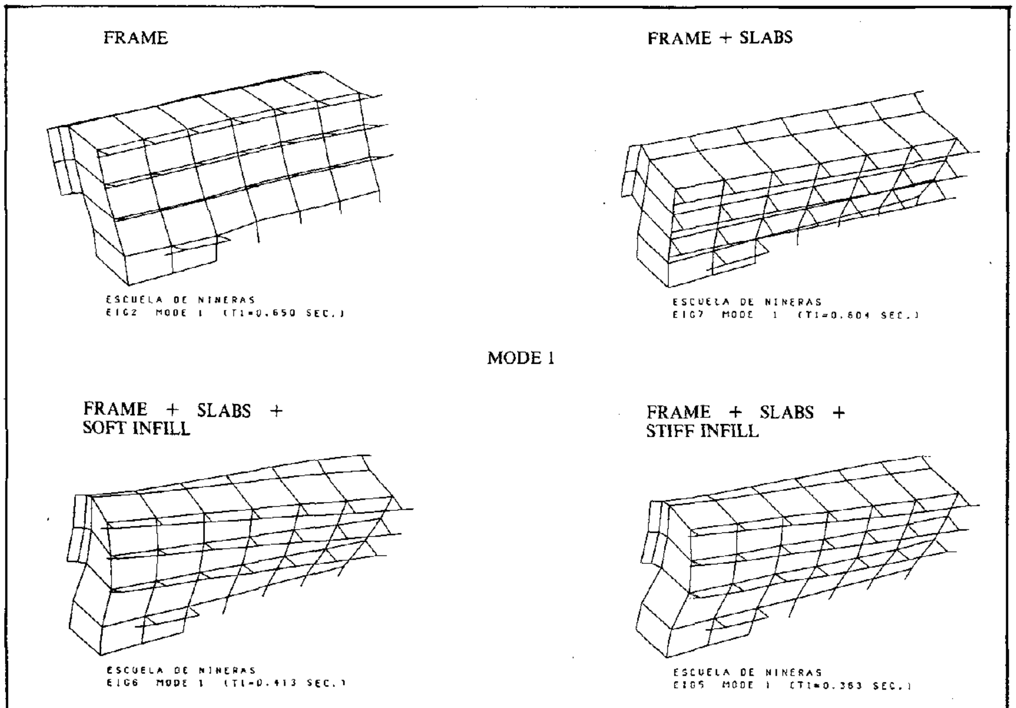


FIG. B5.2 MODE 1 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

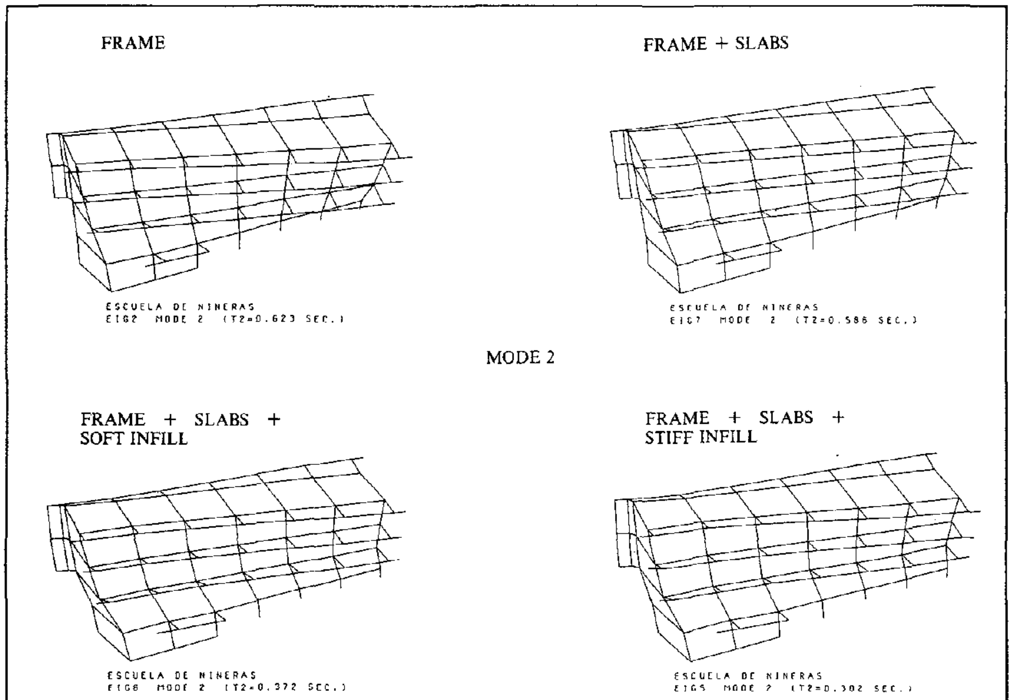


FIG. B5.3 MODE 2 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

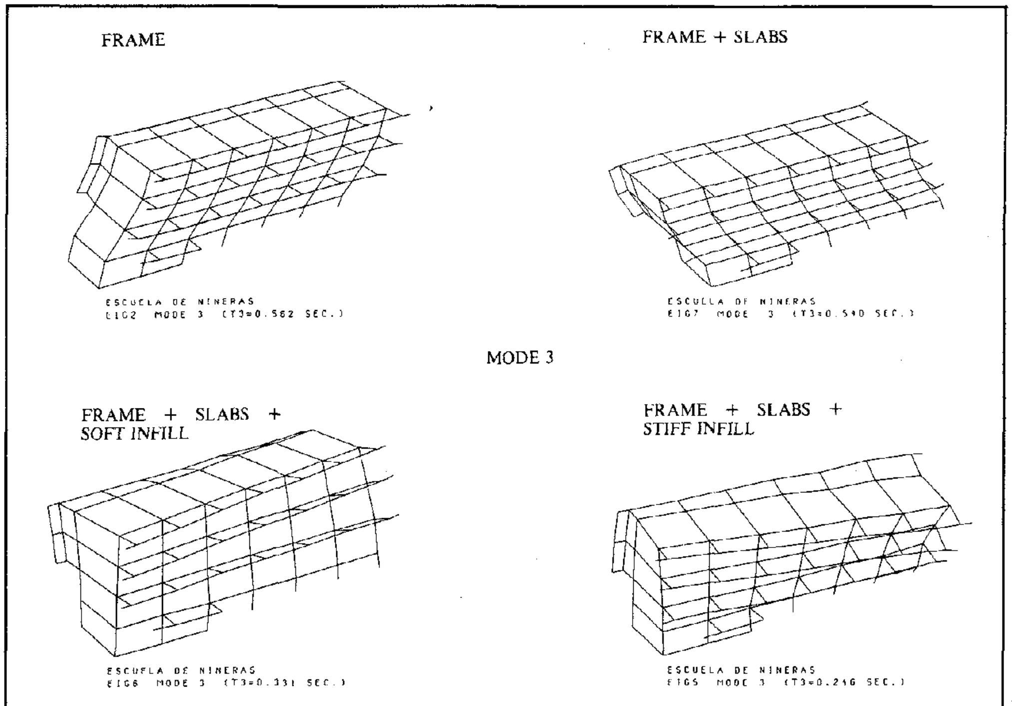


FIG. B5.4 MODE 3 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

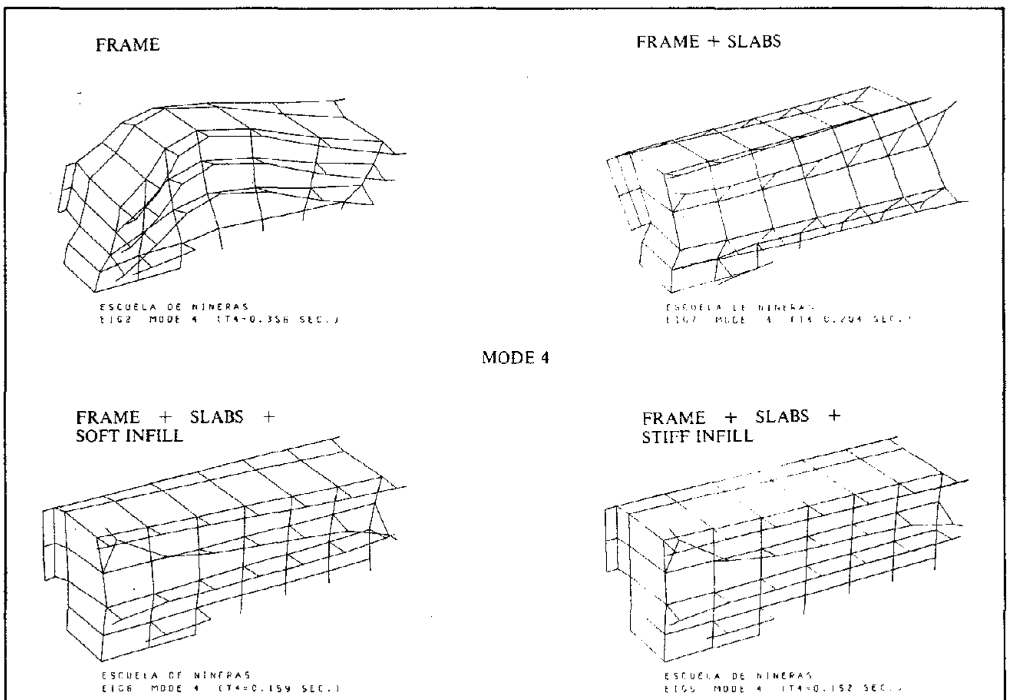


FIG. B5.5 MODE 4 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

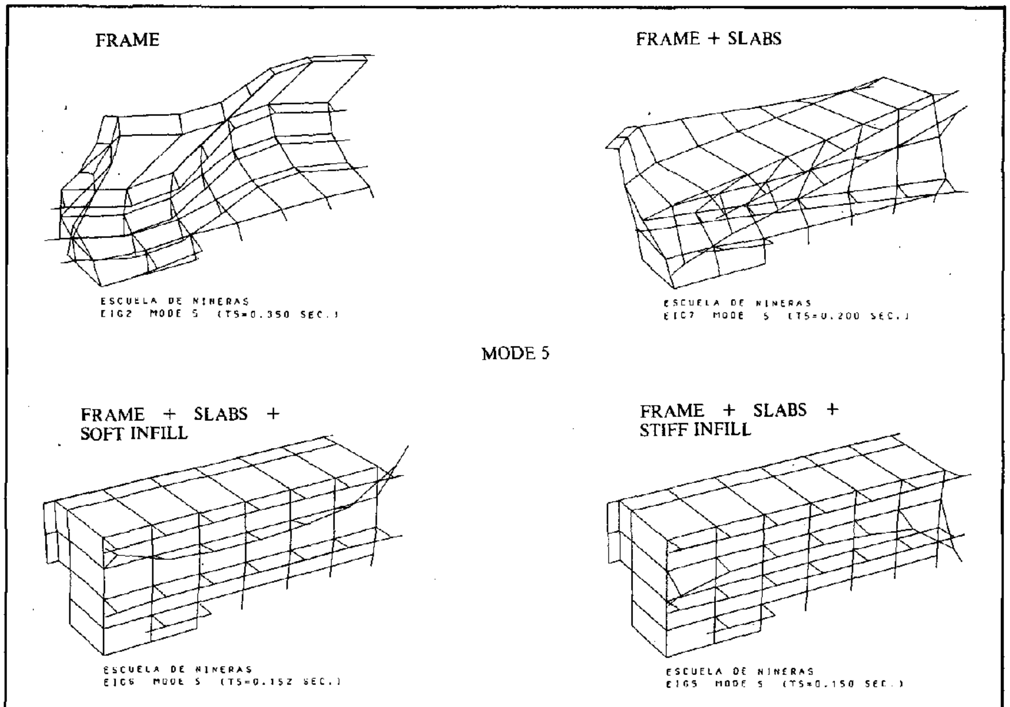


FIG. B5.6 MODE 5 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

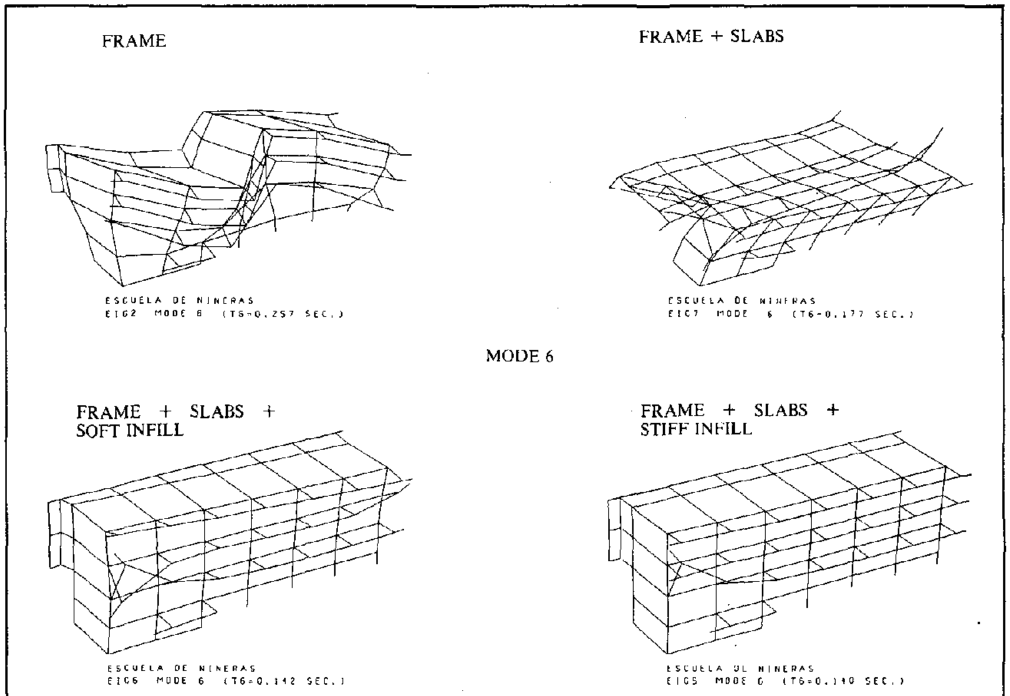


FIG. B5.7 MODE 6 -- MODE SHAPES AND PERIODS OF FOUR MODELS OF THE ESCUELA DE NIÑERAS

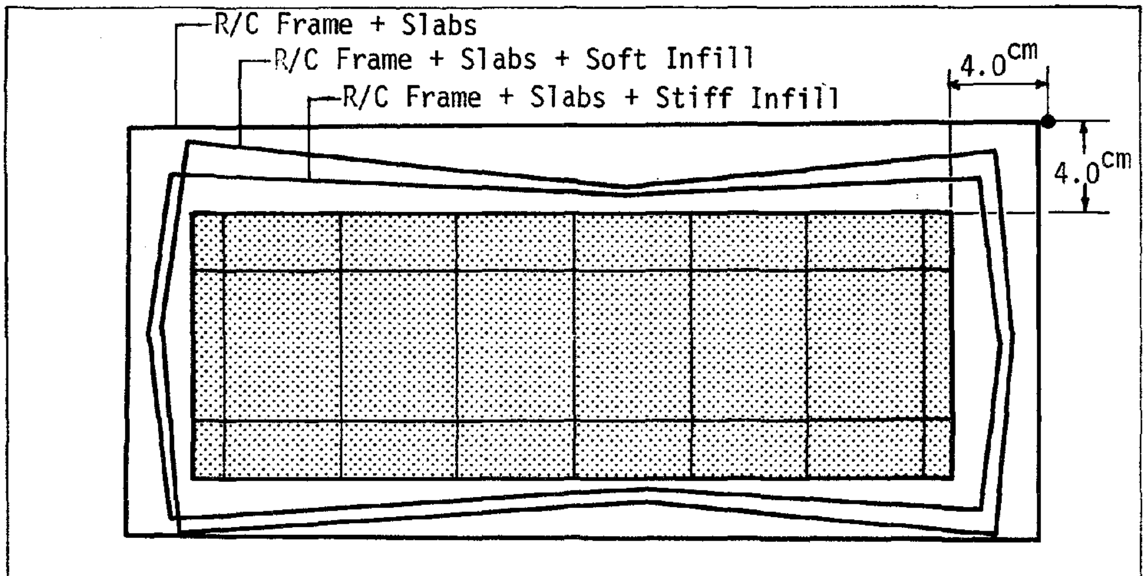


FIG. B5.8 ESCUELA DE NIÑERAS ROOF DISPLACEMENT ENVELOPES
GUATEMALA EARTHQUAKE, FEB. 4, 1976

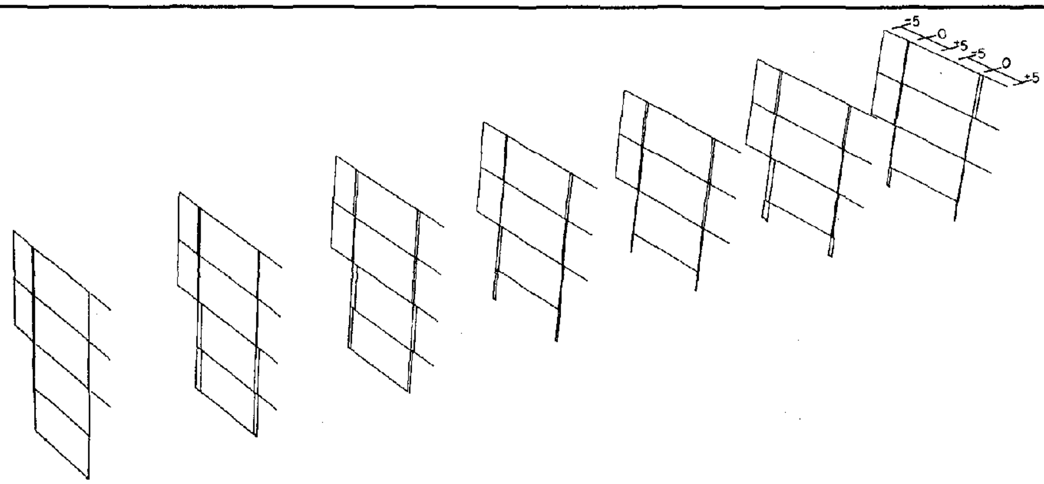


FIG. B5.9. STRESS INCREASE DUE TO INFILL
AXIAL SOFT INFILL

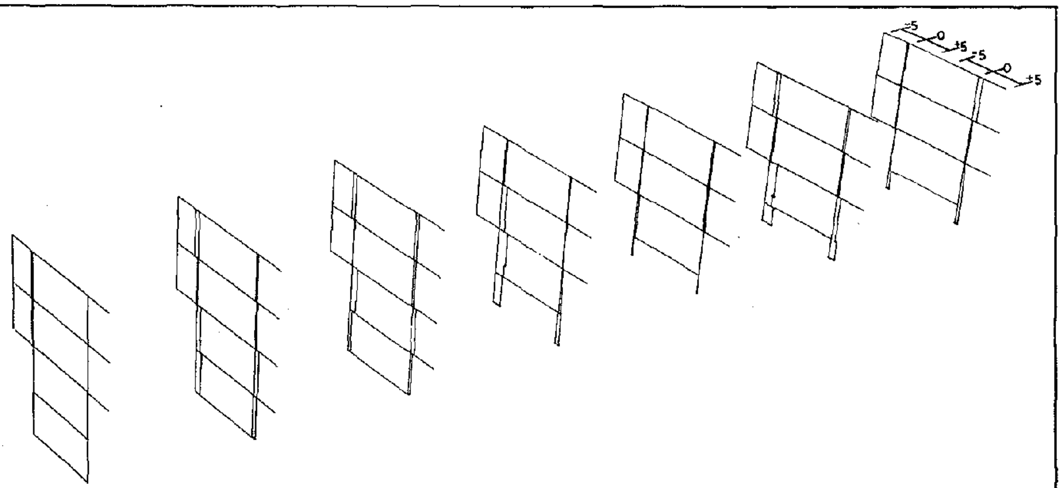


FIG. B5.10 STRESS INCREASE DUE TO INFILL
AXIAL STIFF INFILL

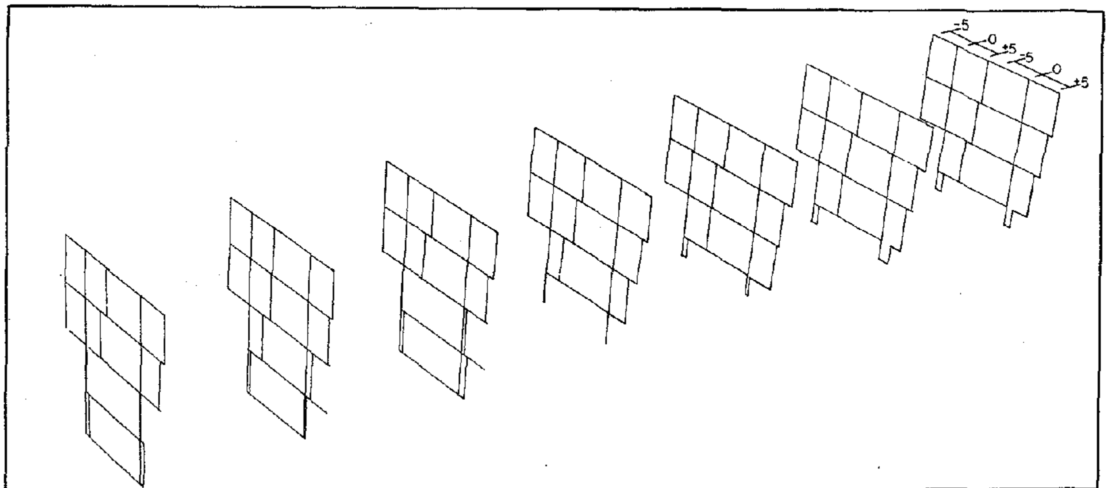


FIG. B5.11 STRESS INCREASE DUE TO INFILL
TORQUE SOFT INFILL

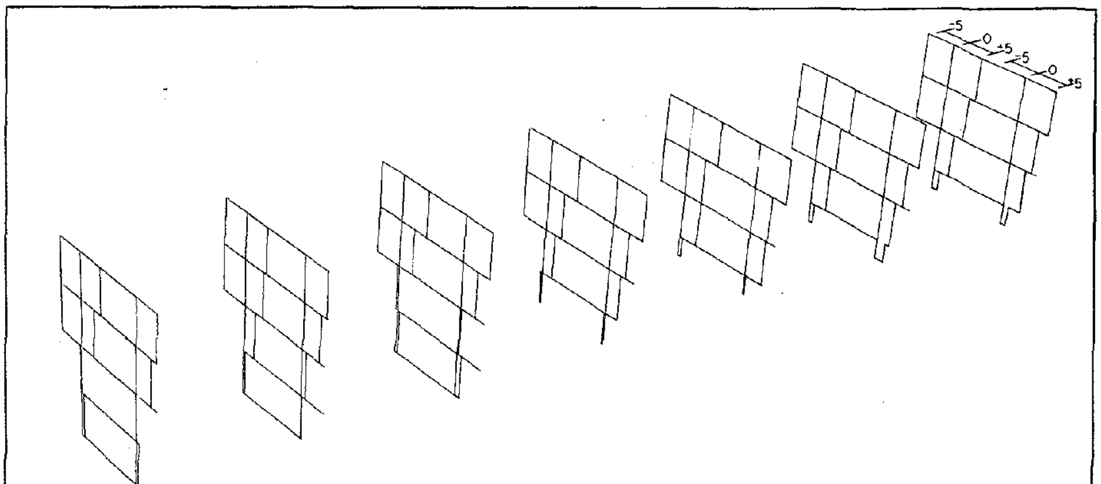


FIG. B5.12 STRESS INCREASE DUE TO INFILL
TORQUE STIFF INFILL

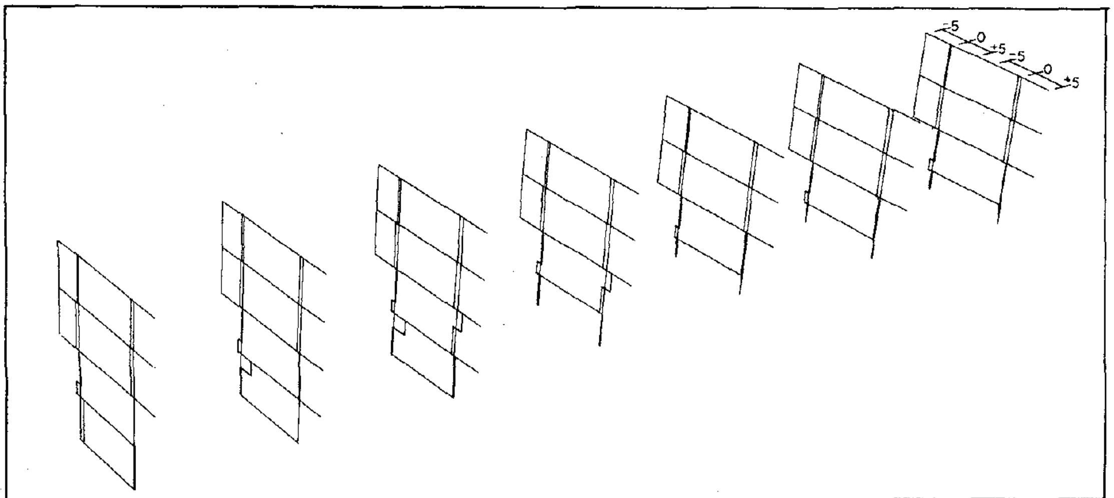


FIG. B5.13 STRESS INCREASE DUE TO INFILL
SHEAR X SOFT INFILL

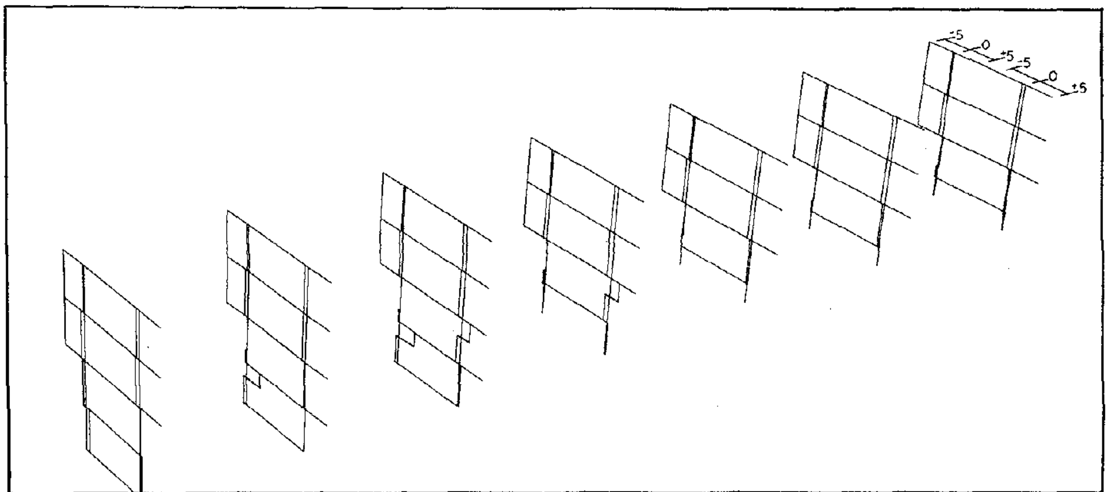


FIG. B5.14 STRESS INCREASE DUE TO INFILL
SHEAR X STIFF INFILL

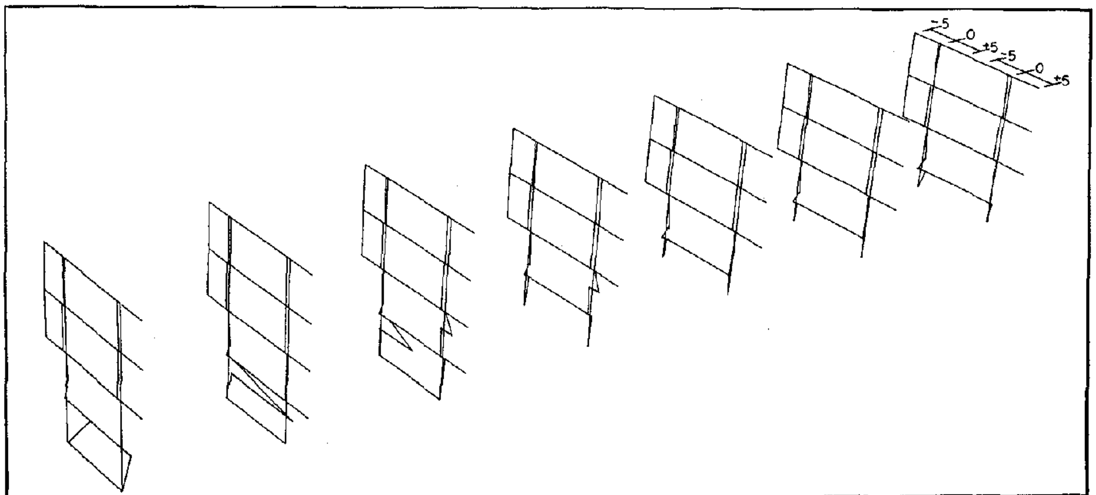


FIG. B5.15 STRESS INCREASE DUE TO INFILL
MOMENT YY SOFT INFILL

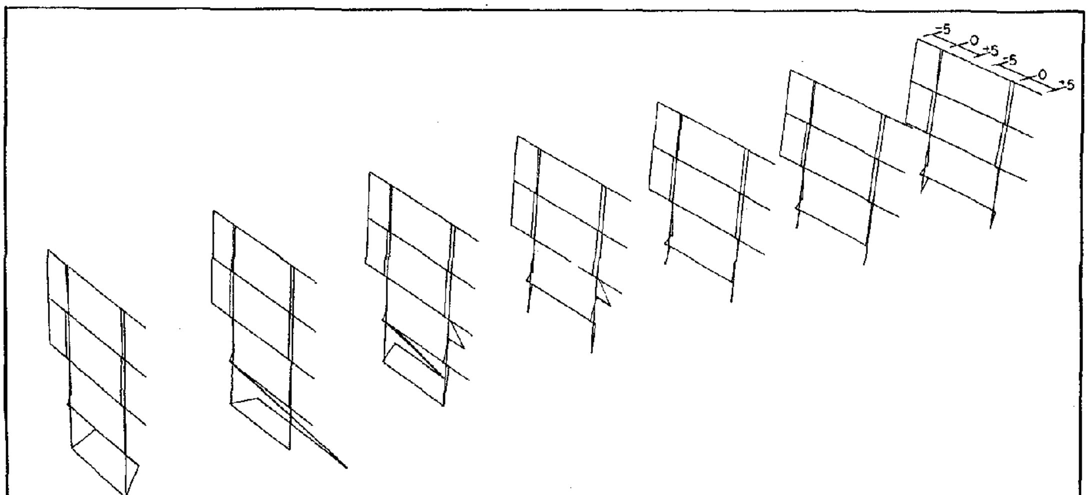


FIG. B5.16 STRESS INCREASE DUE TO INFILL
MOMENT YY STIFF INFILL

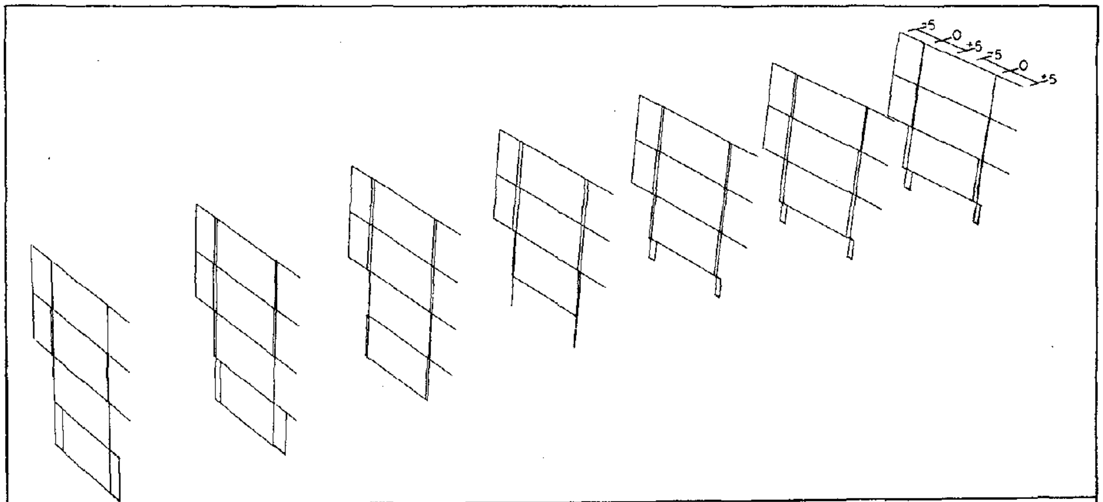


FIG. B5.17 STRESS INCREASE DUE TO INFILL
SHEAR Y SOFT INFILL

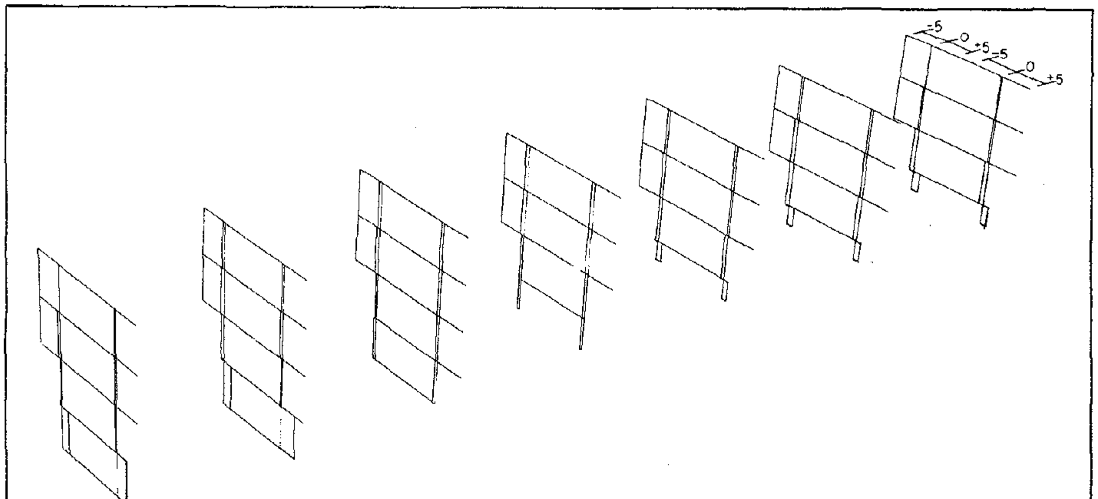


FIG. B5.18 STRESS INCREASE DUE TO INFILL
SHEAR Y STIFF INFILL

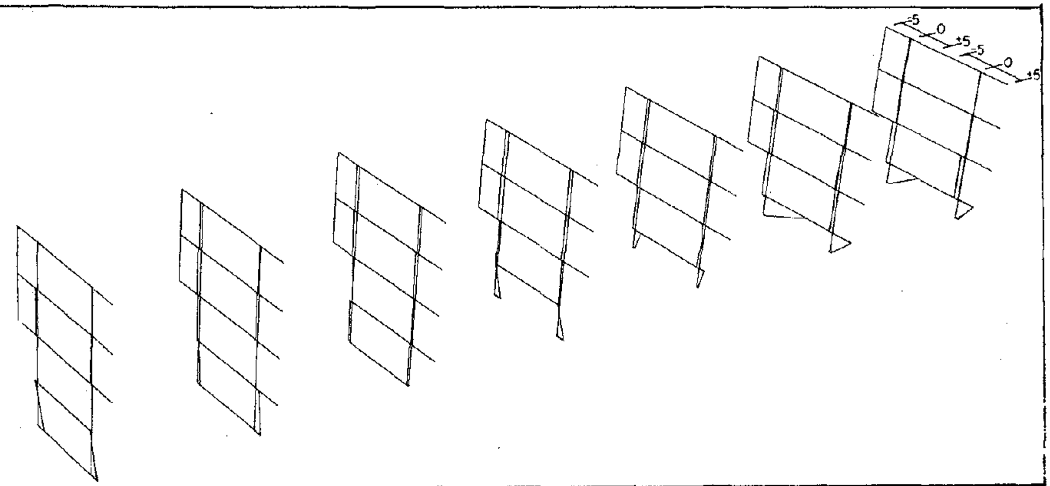


FIG. B5.19 STRESS INCREASE DUE TO INFILL
MOMENT XX SOFT INFILL

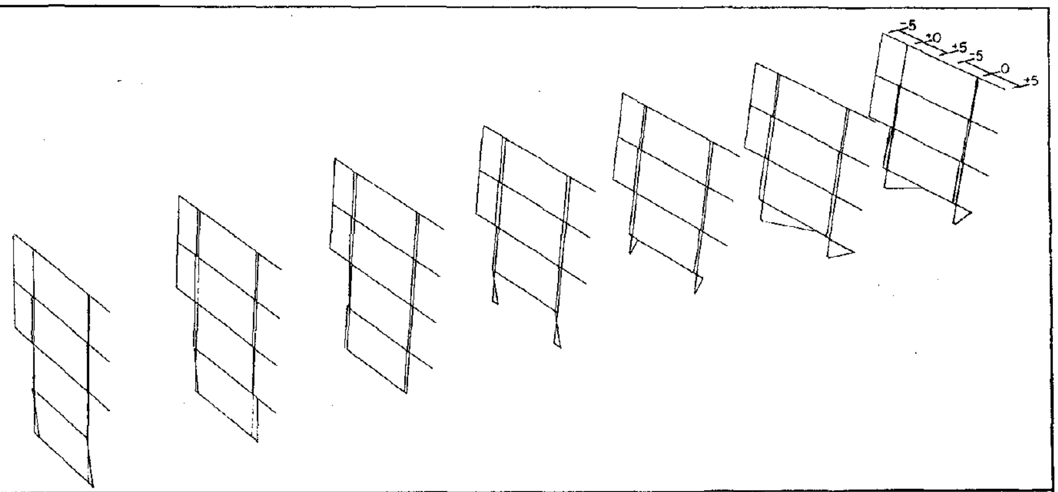


FIG. B5.20 STRESS INCREASE DUE TO INFILL
MOMENT XX STIFF INFILL

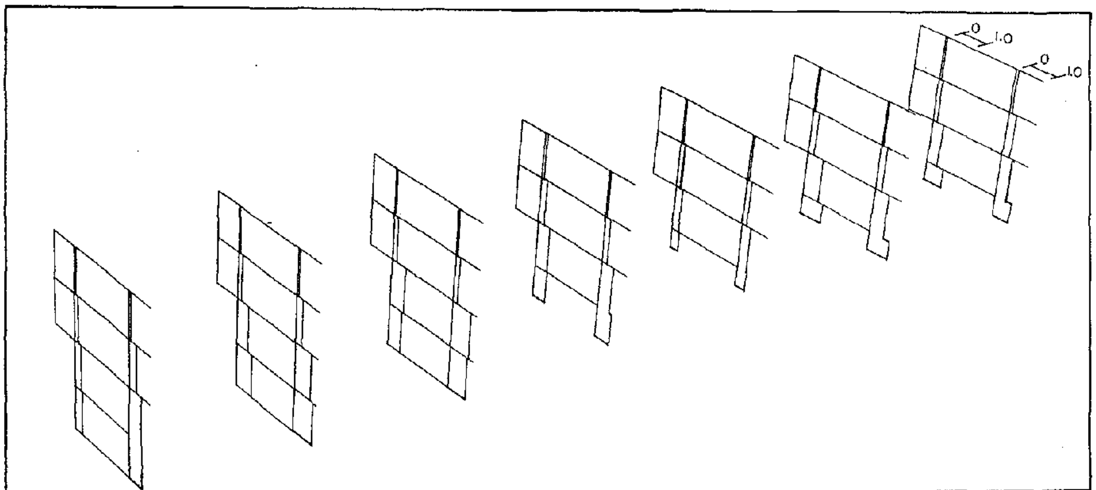


FIG. B5.21 MEMBER FORCE ENVELOPES - GUAT.76
AXIAL MN SOFT INFILL

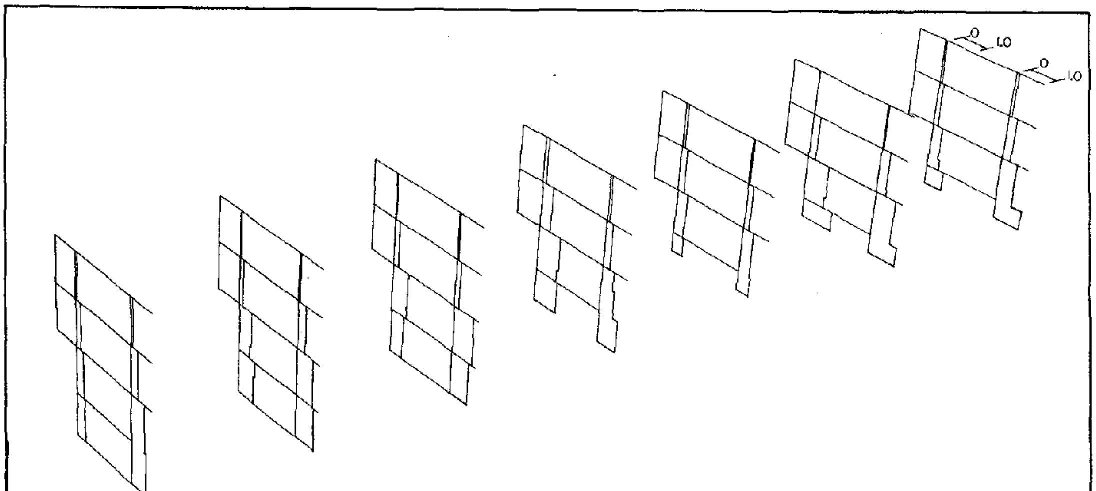


FIG. B5.22 MEMBER FORCE ENVELOPES - GUAT.76
AXIAL MN STIFF INFILL

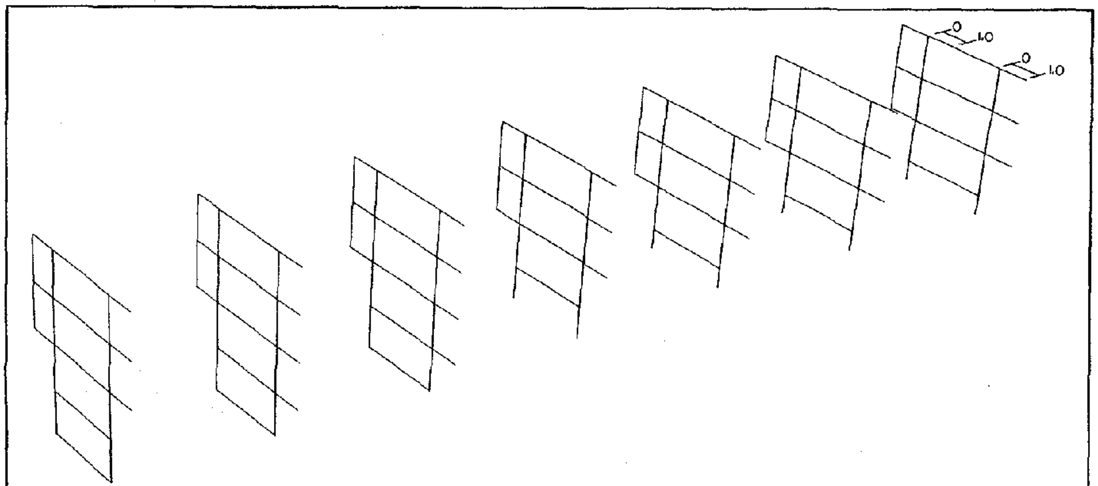


FIG. B5.23 MEMBER FORCE ENVELOPES - GUAT.76
TORQUE MN-M . SOFT INFILL

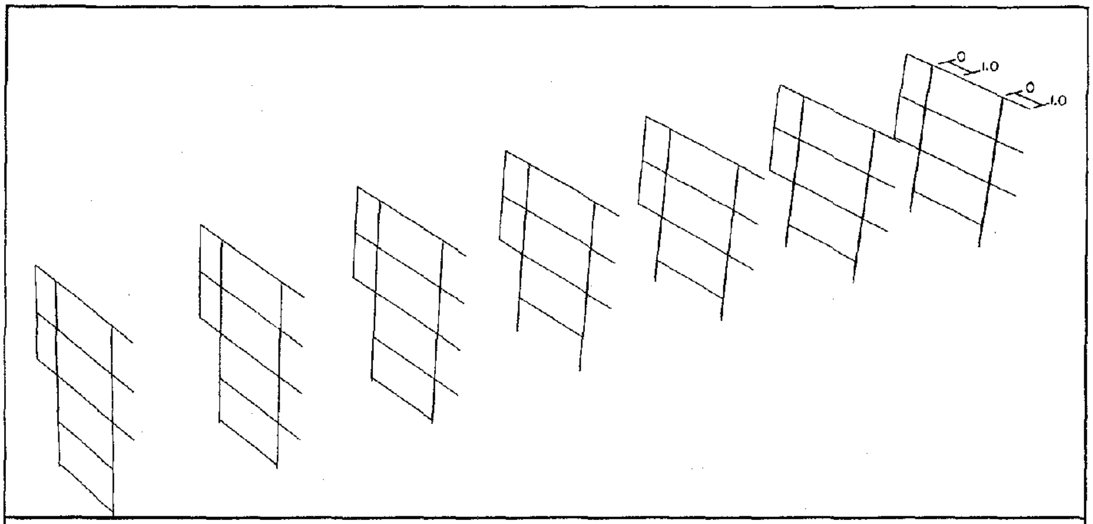


FIG. B5.24 MEMBER FORCE ENVELOPES - GUAT.76
TORQUE MN-M STIFF INFILL

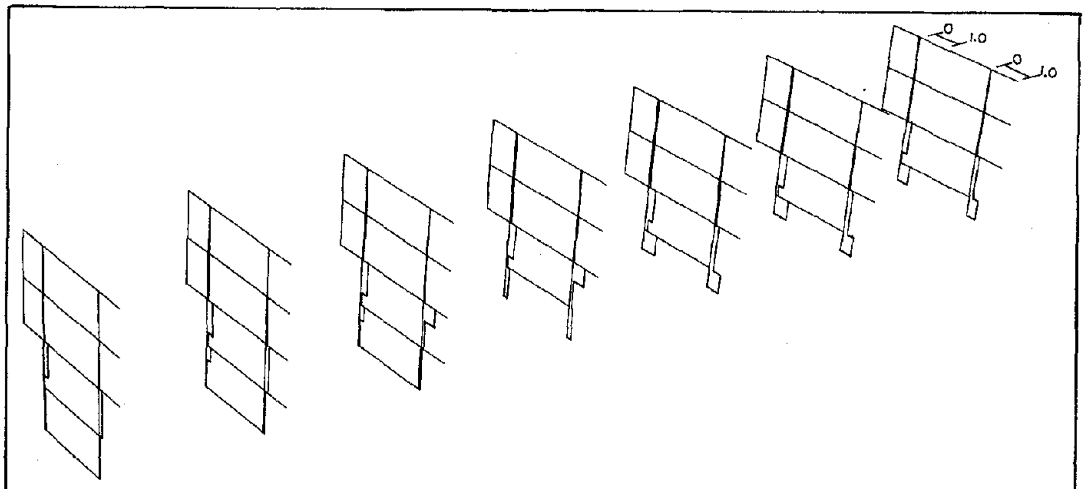


FIG. B5.25 MEMBER FORCE ENVELOPES - GUAT. 76
SHEAR X MN SOFT INFILL

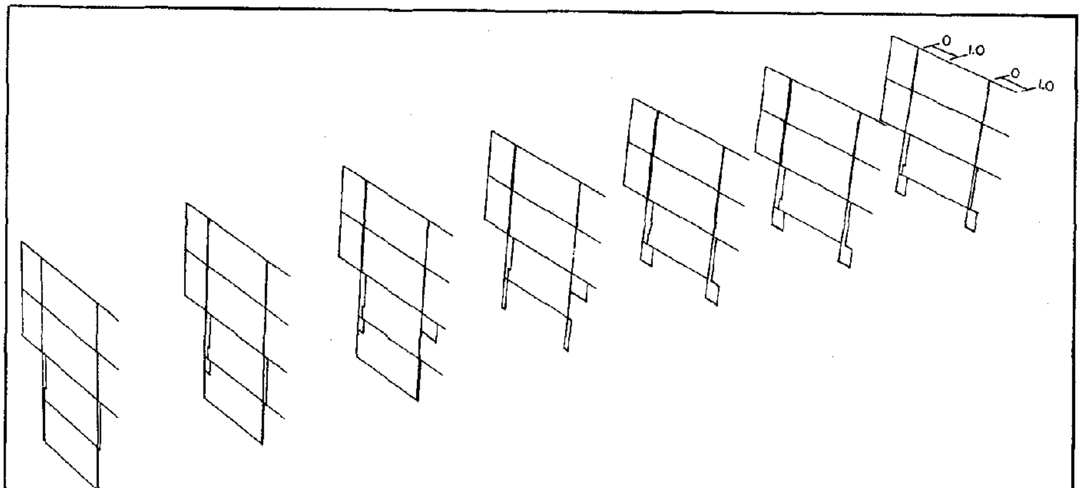


FIG. B5.26 MEMBER FORCE ENVELOPES - GUAT. 76
SHEAR X MN STIFF INFILL

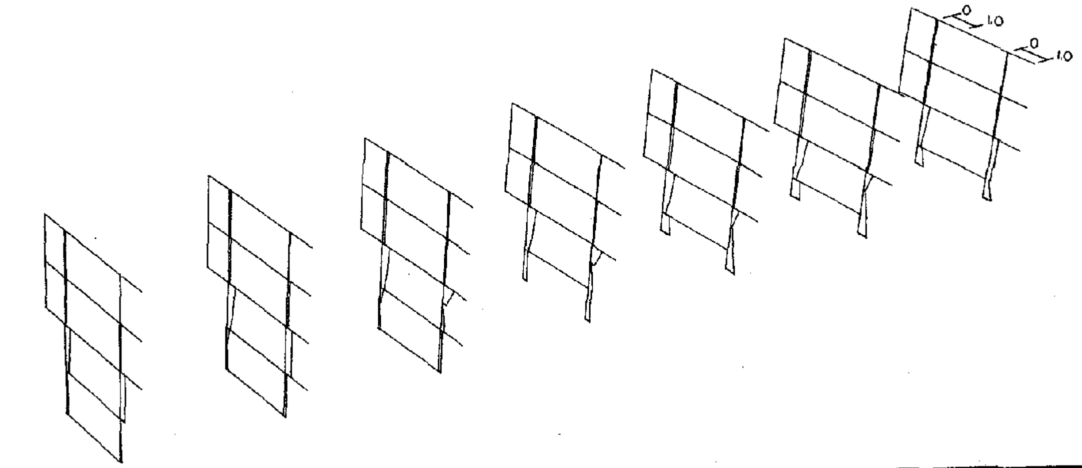


FIG. B5.27 MEMBER FORCE ENVELOPES - GUAT.76
MOMENT YY MN-M SOFT INFILL

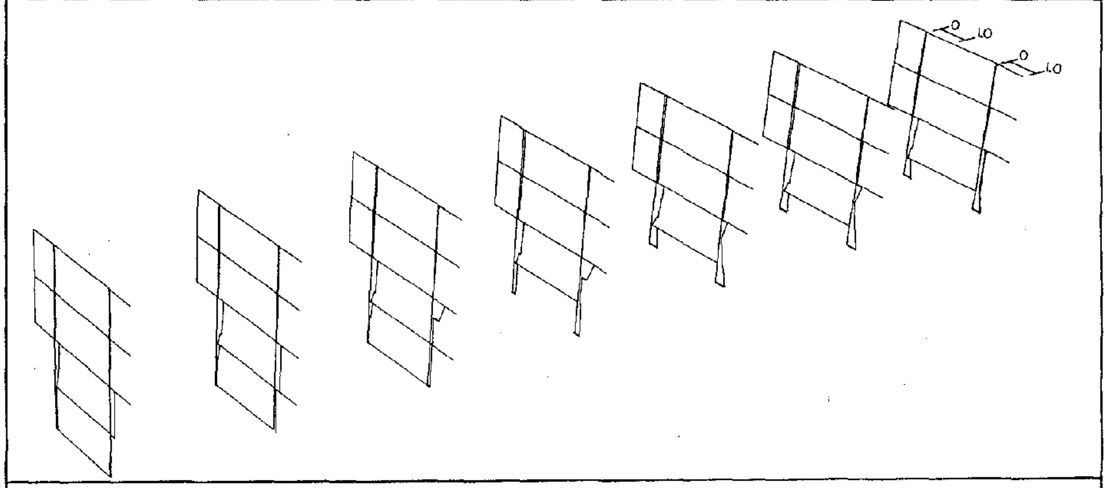


FIG. B5.28 MEMBER FORCE ENVELOPES - GUAT.76
MOMENT YY MN-M STIFF INFILL

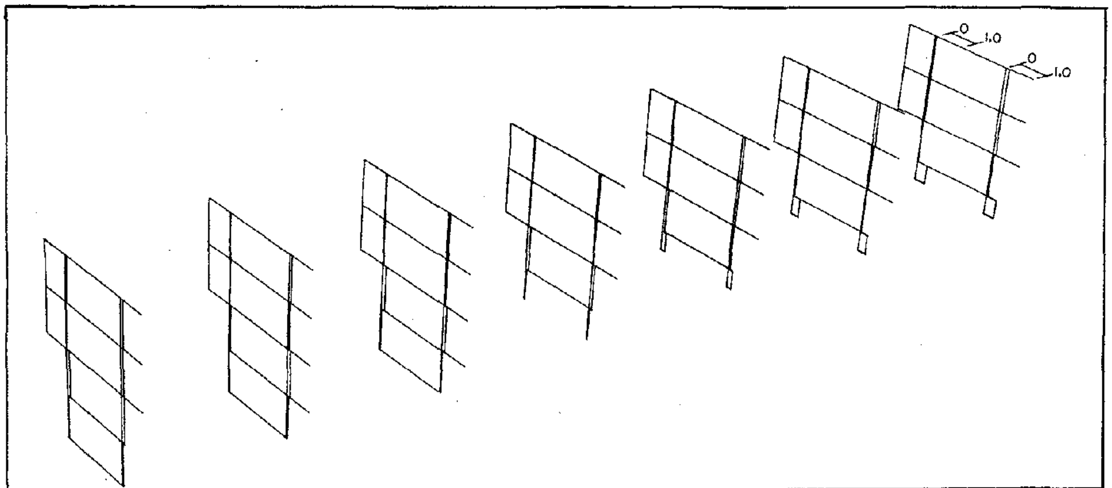


FIG. B5.29 MEMBER FORCE ENVELOPES - GUAT.76
SHEAR Y MN SOFT INFILL

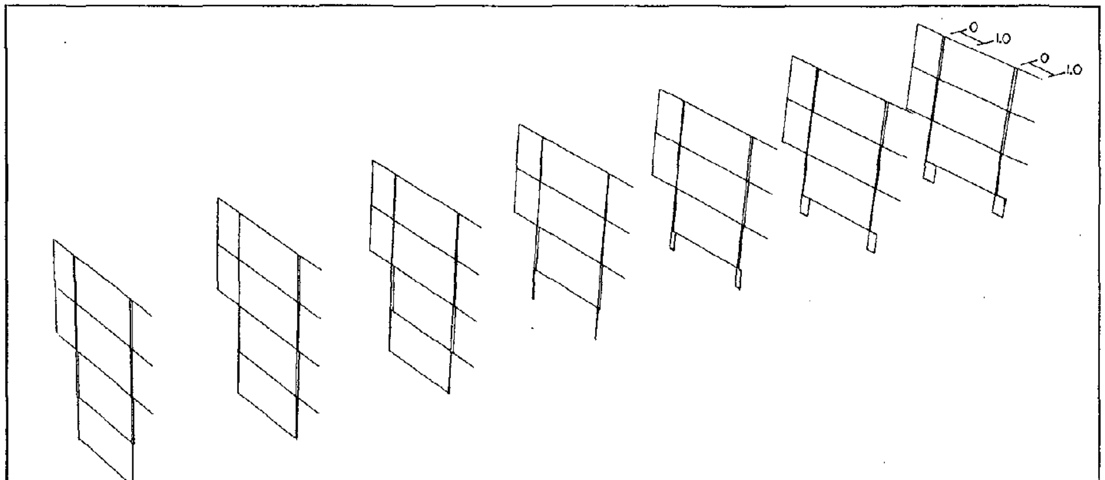


FIG. B5.30 MEMBER FORCE ENVELOPES - GUAT.76
SHEAR Y MN STIFF INFILL

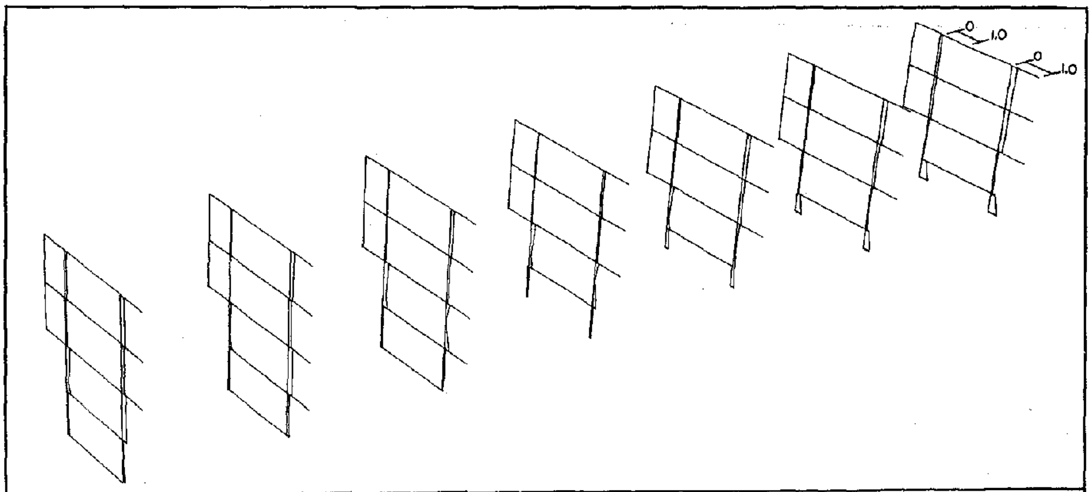


FIG. B5.31 MEMBER FORCE ENVELOPES - GUAT. 76
MOMENT XX MN-M SOFT INFILL

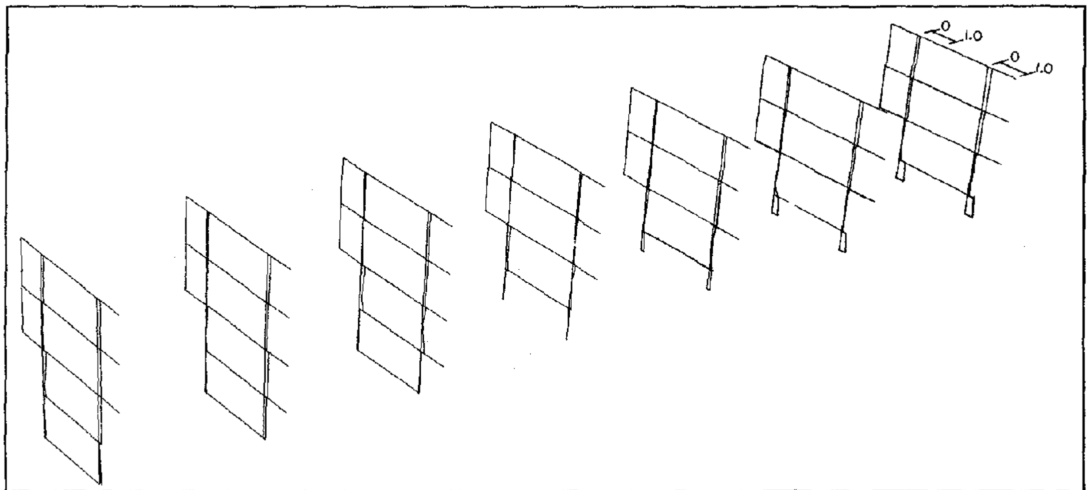


FIG. B5.32 MEMBER FORCE ENVELOPES - GUAT. 76
MOMENT XX MN-M STIFF INFILL

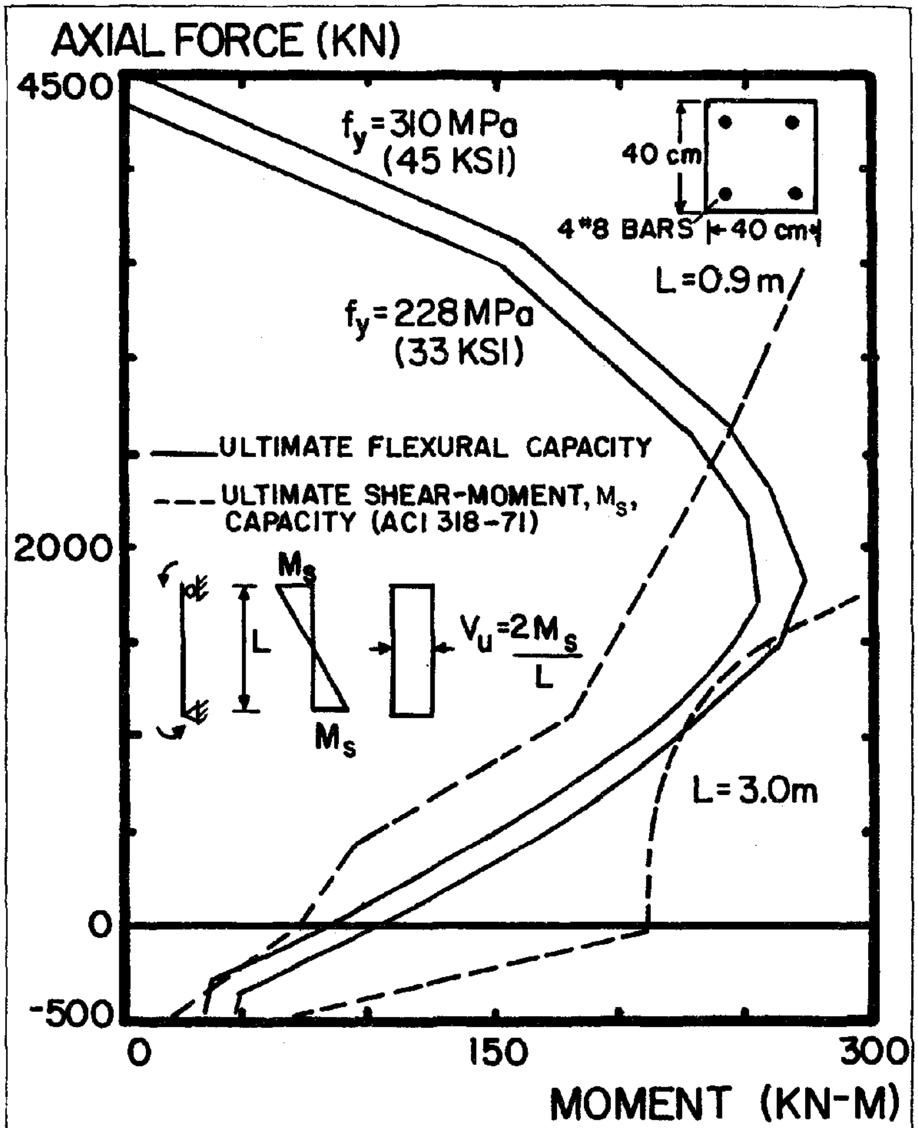


FIG. B5.33 ULTIMATE CAPACITY OF A TYPICAL FIRST FLOOR COLUMN OF THE ESCUELA DE NIÑERAS (eg. Col.54)

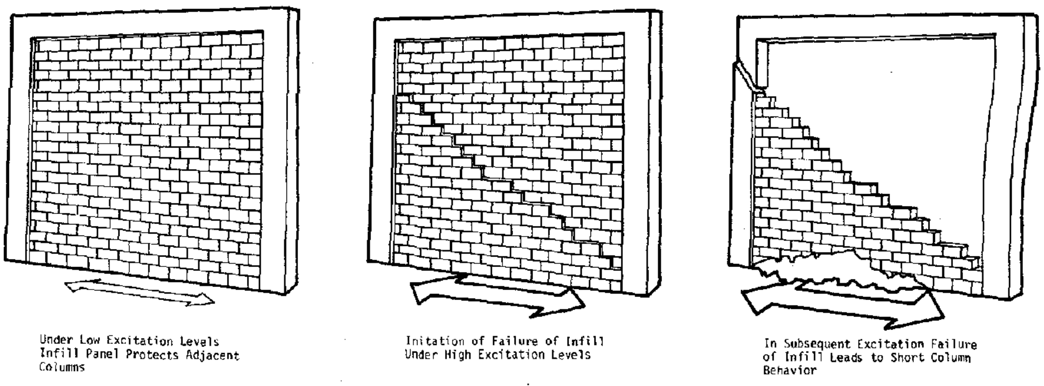


FIG. B5.34 TRANSFORMATION FROM FULL HEIGHT TO PARTIAL HEIGHT INFILL AND ASSOCIATED COLUMN FAILURE

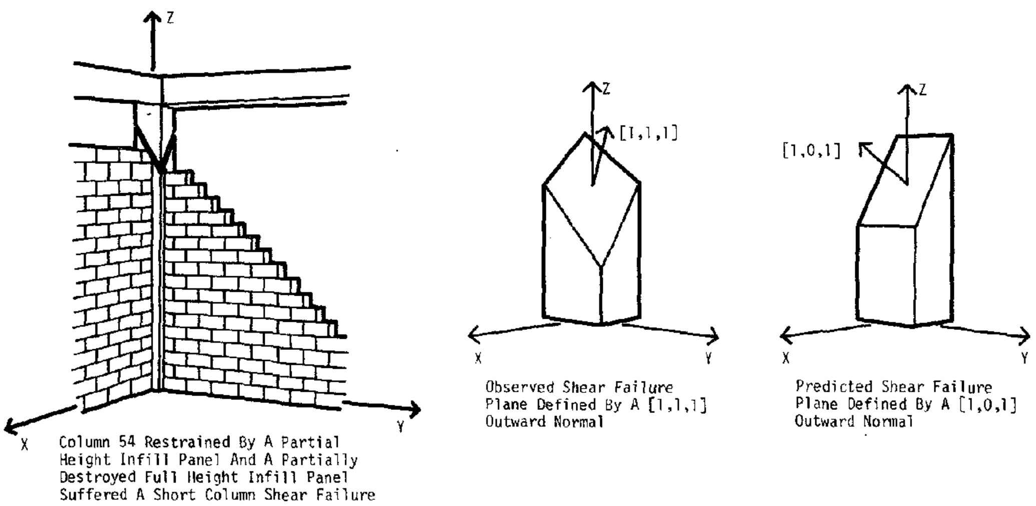


FIG. B5.35 DETAIL OF THE DAMAGE TO COLUMN 54

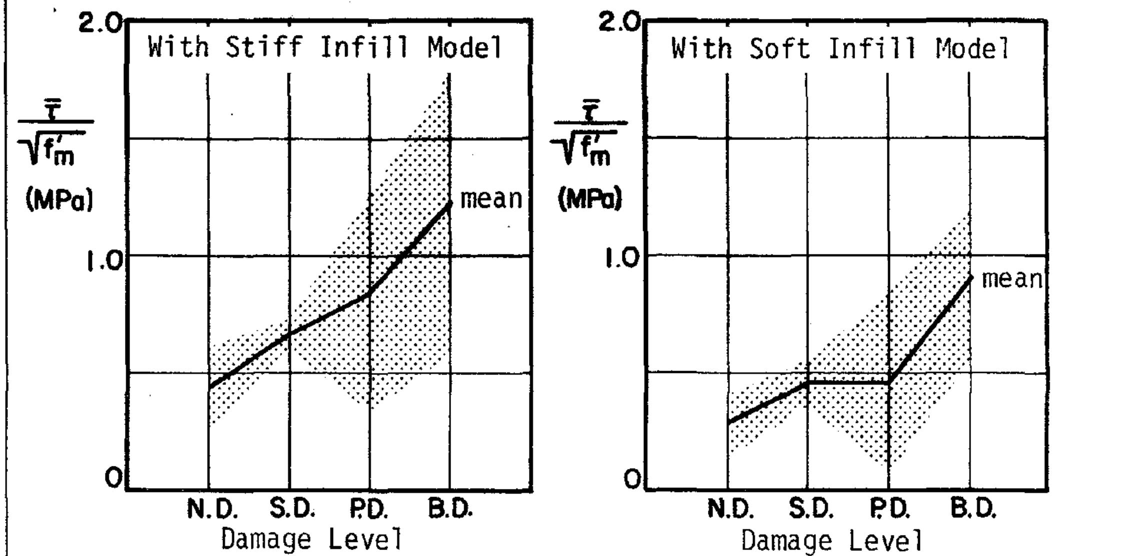


FIG. B5.36 CORRELATION OF NOMINAL INFILL SHEAR STRESS, $\bar{\tau}$, AND OBSERVED DAMAGE LEVEL
N.D-No Damage, S.D.-Slight Damage, P.D.-Partial Damage, B.D.-Bad Damage

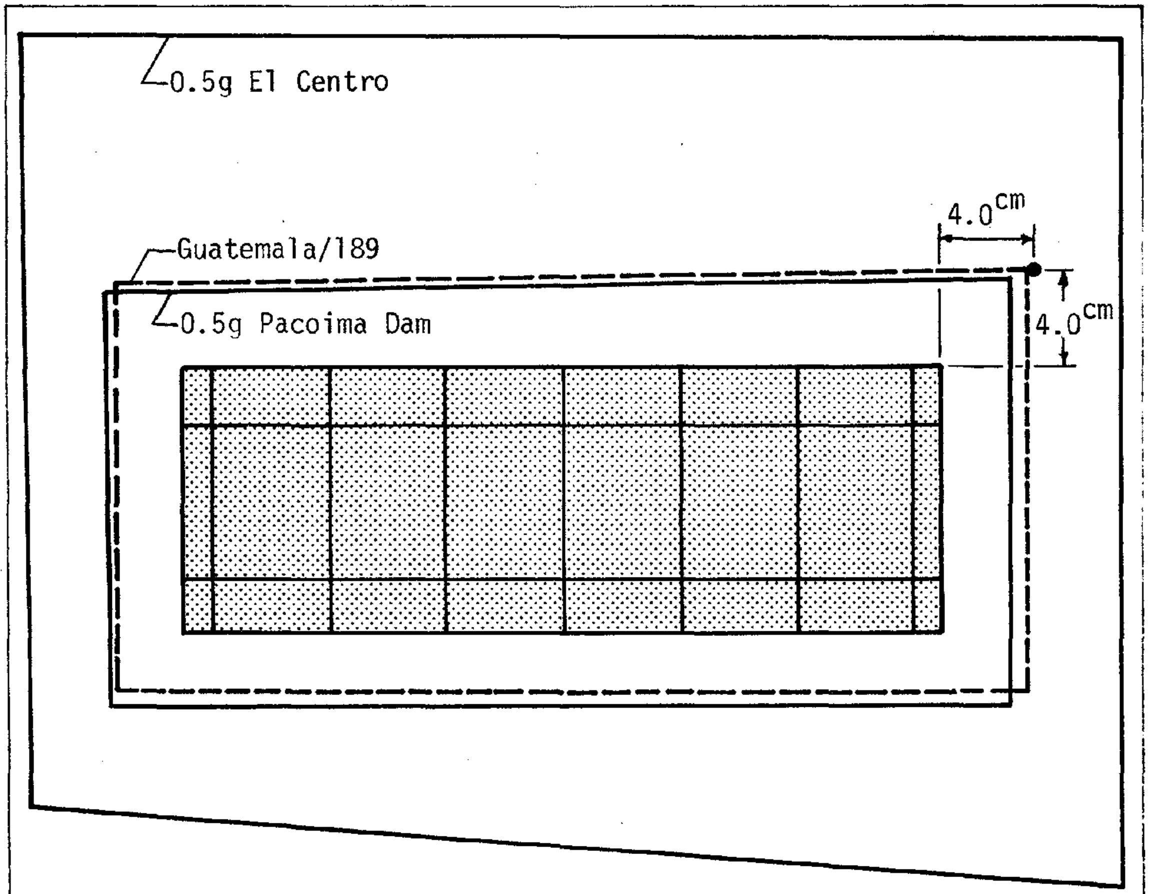


FIG. B5.37 ESCUELA DE NIÑERAS ROOF DISPLACEMENT ENVELOPES
FRAME + SLABS MODEL

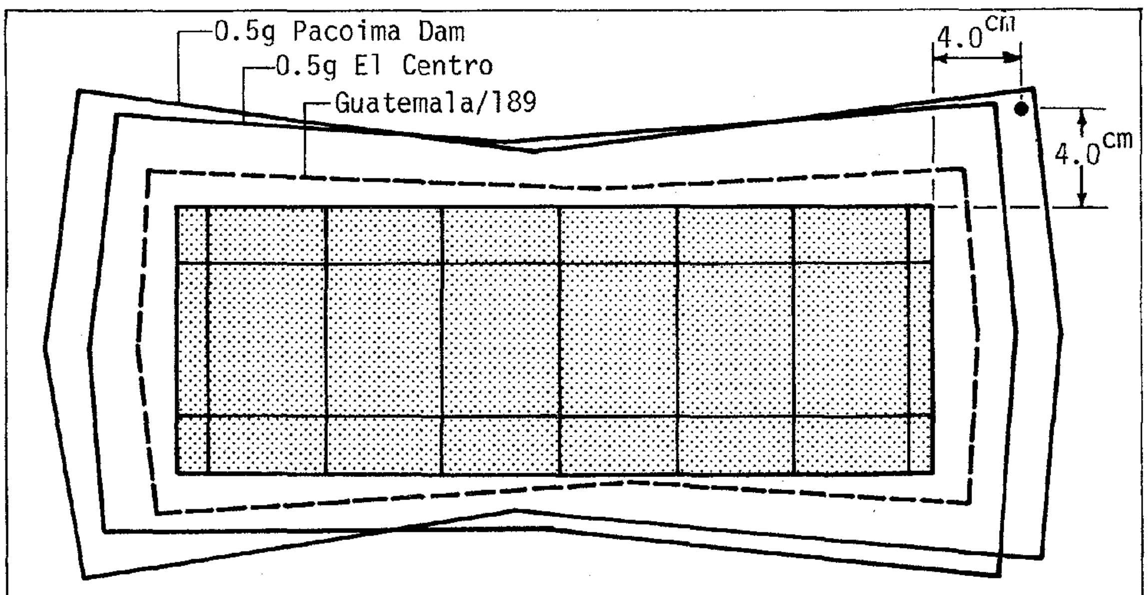


FIG. B5.38 ESCUELA DE NIÑERAS ROOF DISPLACEMENT ENVELOPES
FRAME + SLABS + SOFT INFILL MODEL

APPENDIX

Implementation and Use of the Infill Element Code for SAP IV

The implementation and use of an infill element code is outlined in this section. The code is written to be compatible with SAP IV*, a well known and popular structural analysis program for the dynamic and static analysis of linear structural systems, and is simply inserted into it.

The code is based upon the approach to modeling infill panels presented in Part A of this report; the constraint approach (section 3. Part A) and its approximate extension (section 4. Part A). The code reads in material, geometric and location data and accepts only rectangular elements with sides parallel to the global coordinate axes. Nondimensional stiffness terms are then computed, based upon the element aspect ratio, using the polynomial approximations discussed in section 4, Part A. The nondimensional infill stiffness is then dimensionalized (the inverse of equation 4.2, section 4.) using the infill geometric and material characteristics, transformed to global coordinates and added to the global system stiffness. Four types of infill may be considered;

1. complete stiff infill - infill that completely infills the surrounding frame and is constrained by this frame to deform along its edge in both transverse and longitudinal senses,
2. complete soft infill - infill that completely infills the surrounding frame and is constrained by this frame to deform along its edge in only the transverse sense,
3. partial stiff infill - infill that partially infills the surrounding frame and is constrained by this frame to deform along its edge in both transverse and longitudinal senses,

and,

* Bathe, K.J., Wilson, E.L., & Peterson, F.E., "SAP IV: A Structural Analysis Program for Static and Dynamic Response of Linear Systems", EERC Report No. EERC 73-11, June 73/ April 74

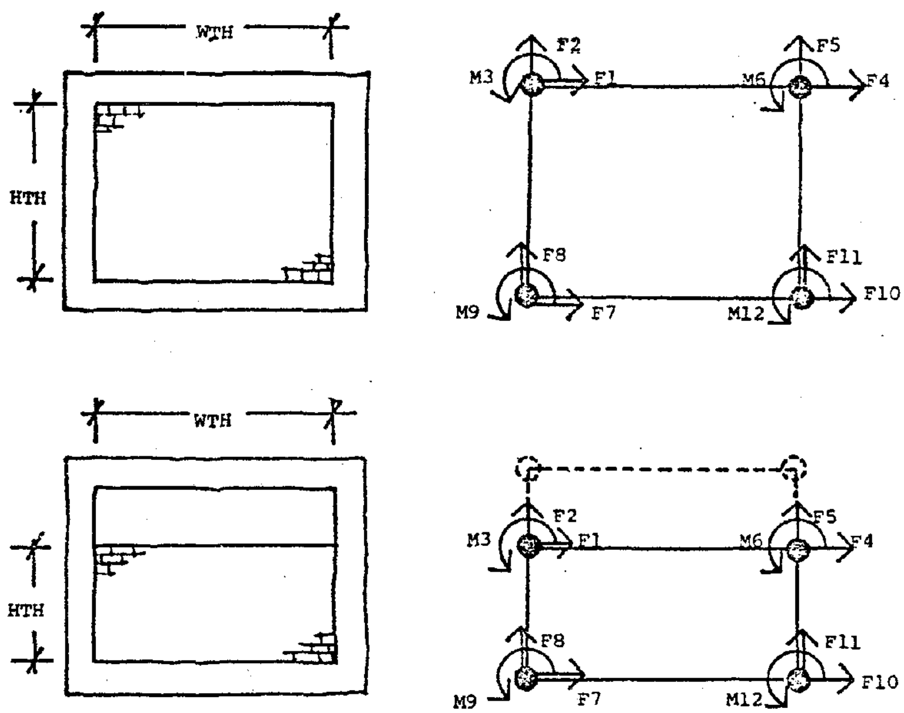
4. partial soft infill - infill that partially infills the surrounding frame and is constrained by this frame to deform along its edge in only the transverse sense.

The implementation of the code is straightforward. The code consists of one subroutine, ELMT9, that simply replaces the do-nothing subroutine of SAP IV of the same name, and an overlay, OVL9, that simply replaces the do-nothing overlay of the same name. This overlay consists of an overlay directive, a program (ELT9), three subroutines (INFILL, INFILK & NONDIM) and a block data subroutine (IFDATA). The subroutine NONDIM forms the non-dimensional infill stiffness using the polynomial approximations whose coefficients are stored in IFDATA. The subroutine INFILK dimensionalizes the nondimensional infill stiffness (using transformation code flags from subroutine INFILL), improves the rigid body mode representation of this stiffness (see pg. A6), and forms the local force-to-global displacement transformation matrix. The subroutine INFILL reads in the pertinent input data, checks admissibility of elements and computes the transformation code flags. (In as much as admissible elements must be rectangular with sides parallel to the global axes the transformation from local to global degrees of freedom involves only changes of sign of individual stiffness terms.)

Once these subroutines are inserted into SAP IV the use of the infill element is straightforward and completely analogus to the use of other elements of the SAP IV element library. A user's manual for the infill element follows that may be inserted into the "IV Element Data" section of the SAP IV user's manual. The infill element source code is also included.

TYPE 9 INFILL ELEMENTS

Infill elements are identified by the number 9. Forces and moments corresponding to the local degrees of freedom, shown below, are calculated for each infill panel. Only the stiffness contribution of the infill panel is computed, associated gravity loads, inertial mass and other panel loads are not considered.



The infill elements were developed to be used in conjuction with surrounding frames composed of conventional beam elements, as indicated above. As such, the member force variation in each surrounding framing member will be estimated by straight line approximations to the actual force variation. Each straight line member force variation approximation will be a (minimum elastic strain energy) mean fit to the exact solution and as such may significantly underestimate extreme values of member forces (especially at member ends).

The infill elements are described by the following sequence of cards;

A. **Control Card** (3I5)

notes	columns	variable	entry
	1-5	NPAR(1)	The number 9
	6-10	NELEM	The total number of infill elements
	11-15	NPROP	The number of element property sets

B. **Element Property Cards** (I5,2F10.0)

notes	columns	variables	entry
	1-5	N	Element property number
	6-15	E(N)	Infill modulus
	16-25	T(N)	Infill thickness

C. **Element Data Cards** (6I5,2F10.0,2I5)

notes	columns	variable	entry
	1-5	INEL	Element number
(1)	6-10	INI	Node number I
(1)	11-15	INJ	Node number J
(1)	16-20	INK	Node number K
(1)	21-25	INL	Node number L
	26-30	IPROP	Element property number
(2)	31-40	WTH	Element width; dimension of infill panel along side IJ
(2)	41-50	HTH	Element height; dimension of infill panel along side KI
(3)	51-55	INC	Generation increment
(4)	56-60	ITYPE	Infill type code 1.EQ. Complete infill with both cubic-transverse and linear-longitudinal edge constraint (ie. "complete stiff infill") 2.EQ. Complete infill with only cubic-transverse edge constraint (ie. "complete soft infill") 3.EQ. Partial infill with both cubic-transverse and linear-longitudinal edge constraint(ie. "partial stiff infill") 4.EQ. Partial infill with only cubic-transverse edge constraint (ie. "partial soft infill")

NOTES/

(1) Only rectangular infill panels located in principal global planes with sides parallel to any two principal axes are admissible. This limitation is enforced to avoid computationally expensive transformations.

The local degrees of freedom and local node numbering are indicated above. Note the nodes IJKL are ordered neither clockwise nor counterclockwise. Side IJ must correspond to the free side of a partial panel.

(2) The user may specify the element width, (height), less than or equal to the nodal distance IJ, (KI). In this way one may attempt to gain some correction for the lack of consideration of beam, column and infill true dimensions. The local infill stiffness will be computed based upon the given width, (height), and will be added directly to the centerline frame stiffness. There is no attempt to model the implied rigid joint frame as the transformations necessary to do so were considered to be computationally prohibitive.

(3) If a series of infill elements occurs in which each element number NE_i is one greater than the previous number $NE_{i-1}+1$ ie;

$$NE_i=NE_{i-1}+1$$

and

i. The element in the series have the same element properties, (ie. modulus and thickness)

ii. The elements in the series have the same element dimensions, (ie. width and height)

iii. The elements in the series are of the same type, ITYPE

then, only the element card for the first element in the series need be given as input provided the element nodal point numbers increase by the same increment, INC as;

$$INI_i=INI_{i-1}+INC$$

$$INJ_i=INJ_{i-1}+INC$$

$$INK_i=INK_{i-1}+INC$$

$$INL_i=INL_{i-1}+INC$$

The value of INC, if left blank, is taken to be one. The element data for the last infill element must always be given.

(4) A complete infill panel is defined as one that completely fills the frame while a partial infill panel fills only part of the frame with a gap on one side. It is assumed that the gap of the partial panel is sufficiently large to avoid contact-release nonlinearities during the system deformation.

The infill elements are based upon an approximate assumption that the frame constrains the form of the deformation of the infill panel. Two types of constraints may be utilized here, both assuring conformation of the infill panel and frame. In the first constraint, (ITYPE.EQ. 1 or 3), the boundary of the panel is constrained to deform transversely to the flexural beam shape function, the cubic hermitian shape function, and longitudinally to the truss shape function, the linear shape function. The second constraint, (ITYPE.EQ. 2 or 4), utilizes only the transverse cubic hermitian constraint. Constraints 1 or 3 may, then, be thought to approximate the behavior of stiff panels monolithic with the frame while constraints 2 or 4 will result in a softer panel that may better approximate typical masonry panel behavior.

A Warning

The infill elements are based upon a kinematic assumption that the surrounding frame constrains the form of the deformation of the infill panel. To avoid the computationally prohibitive task of forming infill element stiffnesses by constraining a suitable mesh of plane stress elements using appropriate congruent transformations a nondimensional parameter study was undertaken to relate individual nondimensional infill stiffness terms to the aspect ratio of the panel by polynomial approximation. It is by these polynomial approximations that infill stiffness are actually computed in this element.

Due to the polynomial approximation one may expect individual infill stiffness terms to be accurate to only about three significant figures. As a result the generated infill stiffness, which in principal should be positive semi-definite, will in many cases be "numerically" indefinite. This is true for other elements as well but here the degree of indefiniteness is relatively more significant.

As the infill stiffness contribution to a system is often very large, it is possible that the addition of the significantly indefinite infill stiffness to a very soft portion of the structure, (ie a portion of the structure with small stiffness in some sense or norm), will result in a structural system stiffness that is also indefinite. This will inhibit further analysis as SAP4 does not admit indefinite systems.

In typical applications this problem should not arise. This problem has been detected, however, when infill elements were used to add stiffness to frames composed of only three, very light, beam elements rather than the usual four. Although the theoretical basis of the elements will allow such a use of the elements the "numerical" indefiniteness of the approximated infill stiffness was able to swamp the positive definiteness of the surrounding partial frame resulting in an indefinite structural system, evidenced by negative eigenvalues.

A scheme was therefore devised to improve the rigid body mode representation offered by the polynomial approximation to the constrained infill element stiffness based upon shifting eigenvalues.

Given the approximate infill stiffness, $\tilde{\mathbf{K}}$, having nonzero eigen values;

$$\tilde{\Lambda} = \begin{bmatrix} \tilde{\lambda}_1 & 0 & 0 \\ 0 & \tilde{\lambda}_2 & 0 \\ 0 & 0 & \tilde{\lambda}_3 \end{bmatrix}$$

corresponding to the rigid body modes, Φ, which are known apriori as;

$$\Phi = \begin{bmatrix} .5 & 0 & -HTH/A \\ 0 & .5 & -WTH/A \\ 0 & 0 & 2/A \\ .5 & 0 & -HTH/A \\ 0 & .5 & WTH/A \\ 0 & 0 & 2/A \\ .5 & 0 & HTH/A \\ 0 & .5 & -WTH/A \\ 0 & 0 & 2/A \\ .5 & 0 & HTH/A \\ 0 & .5 & WTH/A \\ 0 & 0 & 2/A \end{bmatrix}$$

where HTH, the height of the panel, and WTH, the width of the panel are defined in the figure page A3 and;

$$A = \sqrt{4HTH^2 + 4\,WTH^2 + 16}$$

We seek to find an improved stiffness matrix, $\mathbf{K}$, having having zero eigenvalues;

$$\Lambda = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

corresponding to the rigid body modes, Φ.

Furthermore we require this improved stiffness matrix, $\mathbf{K}$, to have all other eigen-pairs corresponding to the elastic stiffness of the infill to be unchanged from those of $\tilde{\mathbf{K}}$. Then;

$$\mathbf{K}=\tilde{\mathbf{K}} - \Phi\Phi^T\tilde{\mathbf{K}}\Phi\Phi^T$$

as;

$$\Phi^T\mathbf{K}\Phi = \Phi^T\tilde{\mathbf{K}}\Phi - \Phi^T\Phi\Phi^T\tilde{\mathbf{K}}\Phi\Phi^T\Phi$$

or;

$$\Lambda = \tilde{\Lambda} - \tilde{\Lambda} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{bmatrix}$$

as desired, recognizing;

1.

$$\Phi^T\Phi = \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

2.

$$\Phi^T\mathbf{K}\Phi = \Lambda$$

and,

3.

$$\Phi^T\tilde{\mathbf{K}}\Phi = \tilde{\Lambda}$$

Unfortunately $\tilde{\mathbf{K}}$ not only misrepresents the eigenvalues of the rigid body modes but also does not accurately capture the eigenvectors corresponding to these modes. Consequently this scheme of improving the rigid body representation does not solve the problem exactly but does appear to improve the representation. Therefore this warning should still be kept in mind.

INFILL ELEMENT SUBROUTINES FOR SAP IV

```
*************************************************************************
      SUBROUTINE ELMT9
C
      COMMON/ELPAR/NPAR(14),NUMNP,MBAND,NELTYP,N1,N2,N3,N4,N5,MTOT,NEQ
      COMMON/JUNK/LT,LH,L,SIG(20),N6,N7,N8,N9,N10
      COMMON/EXTRA/MODEX,NT8,N10SV,NT10
      COMMON A(1)
C
      IF (NPAR(1).EQ.0) GO TO 10
      N6=N5+NPAR(3)*NUMNP
      N7=N6+NPAR(3)
      IF (N7.GT.MTOT) CALL ERROR (N7-MTOT)
C
      CALL OVERLAY (4HSSAP,9,0,6HRECALL)
C
      RETURN
10    WRITE (6,30)
      NUME=NPAR(2)
      DO 20 MM=1,NUME
      CALL STRSC (A(N1),A(N3),NEQ,0)
      WRITE (6,40)
      DO 20 L=LT,LH
      CALL STRSC (A(N1),A(N3),NEQ,1)
      WRITE (6,50) MM,L,(SIG(I),I=1,12)
C     LOCAL FORCE PORTHOLE
      IF (N10SV.EQ.1) WRITE (NT10) MM,L,(SIG(I),I=1,12)
20    CONTINUE
      RETURN
C
30    FORMAT(/31H .....INFILL FORCES AND MOMENTS//
     1* ELEM LOAD*,T16,*F1*,T26,*F2*,T36,*M3*,T46,*F4*,T56,*F5*,T66,*M6*
     2,T76,*F7*,T86,*F8*,T96,*M9*,T106,*F10*,T116,*F11*,T126,*M12*/
     3*  NO.  NO.*)
40    FORMAT(/)
50    FORMAT(2I5,12(1X,E9.3)/)
      END
*************************************************************************

      OVERLAY(SSAP,9,0)

*************************************************************************

      PROGRAM ELT9
C
C     INFILL ELEMENT OVERLAY
C
      COMMON A(1)
      COMMON/ELPAR/NPAR(14),NUMNP,MBAND,K1,N1,N2,N3,N4,N5,K2,K3
      COMMON/JUNK/DUM(23),N6,N7,N8,N9,N10
C
      N5A=N5+NUMNP
      CALL INFILL (NPAR(2),NPAR(3),A(N1),A(N2),A(N3),A(N4),A(N5A),A(N6),
     1NUMNP,MBAND)
      RETURN
      END
*************************************************************************

      SUBROUTINE INFILL (NELEM,NPROP,ID,X,Y,Z,E,T,NUMNP,MBAND)
C
C     INFILL ELEMENT
C
      COMMON/EXTRA/MODEX,NT8
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24),
     1ZERO(12,4)
      DIMENSION X(1),Y(1),Z(1),ID(NUMNP,1),E(1),T(1)
      DIMENSION DL(3,3),IX(3)
C
C     INITIALIZATION
C
      WRITE (6,220) NELEM,NPROP
      N=0
      DO 10 I=1,24
      LM(I)=0
10    XM(I)=0.0
      DO 20 I=1,12
      DO 20 J=1,12
      ASA(I,J)=0.0
20    SA(I,J)=0.0
      DO 30 I=1,48
30    ZERO(I)=0.0
C
C     READ AND PRINT ELEMENT PROPERTY DATA
C
      WRITE (6,230)
      DO 40 I=1,NPROP
      READ (5,240) N,E(N),T(N)
40    WRITE (6,250) N,E(N),T(N)
C     DATA PORTHOLE SAVE
      IF (MODEX.EQ.1) WRITE (NT8) (E(N),T(N),N=1,NPROP)
C
C     READ AND PRINT ELEMENT DATA. GENERATE MISSING INPUT.
C
      WRITE (6,260)
      L=0
50    KKK=0
      READ (5,270) INEL,INI,INJ,INK,INL,IPROP,WTH,HTH,INC,ITYPE
      IF (INEL.NE.1) GO TO 60
      NI=INI
      NJ=INJ
```

```
      NK=INK
      NL=INL
60    IF (INC.EQ.0) INC=1
70    L=L+1
      KKK=KKK+1
      ML=INEL-L
      IF (ML) 80,90,100
80    WRITE (6,280) INEL
      CALL EXIT
90    NEL=INEL
      NI=INI
      NJ=INJ
      NK=INK
      NL=INL
      MPROP=IPROP
      GO TO 110
100   NEL=INEL-ML
      NI=IN+KKK*INCR
      NJ=JN+KKK*INCR
      NK=KN+KKK*INCR
      NL=LN+KKK*INCR
110   CONTINUE
      WRITE (6,290) NEL,NI,NJ,NK,NL,MPROP,WTH,HTH,ITYPE
C     DATA PORTHOLE SAVE
      IF (MODEX.EQ.1) WRITE (NT8) NEL,NI,NJ,NK,NL,MPROP,WTH,HTH,ITYPE
C
C     LOCAL TO GLOBAL TRANSFORMATION
C
C        1. AXES VECTORS
C
      DL(1,1)=X(NL)-X(NK)
      DL(1,2)=Y(NL)-Y(NK)
      DL(1,3)=Z(NL)-Z(NK)
      DL(2,1)=X(NI)-X(NK)
      DL(2,2)=Y(NI)-Y(NK)
      DL(2,3)=Z(NI)-Z(NK)
      DL(3,1)=DL(1,2)*DL(2,3)-DL(1,3)*DL(2,2)
      DL(3,2)=DL(1,1)*DL(2,3)-DL(1,3)*DL(2,1)
      DL(3,3)=DL(1,1)*DL(2,2)-DL(1,2)*DL(2,1)
      XX=0.0
      YY=0.0
      ZZ=0.0
      DO 120 I=1,3
      XX=XX+DL(1,I)*DL(1,I)
      YY=YY+DL(2,I)*DL(2,I)
120   ZZ=ZZ+DL(3,I)*DL(3,I)
C
C        2. TRNSFRMTN CODE-IXX,IYY,IZZ AND ELEMENT ADMISSIBILITY CHECK
C
C               IXX.EQ. GLOBAL DIRECTION OF LOCAL XX AXIS
C               IYY.EQ. GLOBAL DIRECTION OF LOCAL YY AXIS
C               IZZ.EQ. GLOBAL DIRECTION OF LOCAL ZZ AXIS
C
      IXX=0
      IYY=0
      IZZ=0
C
C        COMPUTE LOCAL AXIS DIRECTION COSINES (ADMIT ONLY LIJ=1.0)
C
      XX=SQRT(XX)
      YY=SQRT(YY)
      ZZ=SQRT(ZZ)
      DO 130 I=1,3
      IDL1=INT(DL(1,I)/XX)
      IDL2=INT(DL(2,I)/YY)
      IDL3=INT(DL(3,I)/ZZ)
      IDL=IABS(IDL1)+IABS(IDL2)+IABS(IDL3)
      IF (IDL.NE.1) GO TO 140
      IF ((IABS(IDL1)).EQ.1) IXX=ISIGN(I,IDL1)
      IF ((IABS(IDL2)).EQ.1) IYY=ISIGN(I,IDL2)
130   IF ((IABS(IDL3)).EQ.1) IZZ=ISIGN(I,IDL3)
      IF ((IXX.EQ.0).OR.(IYY.EQ.0).OR.(IZZ.EQ.0)) GO TO 140
      GO TO 150
140   WRITE (6,300) NEL
      CALL EXIT
150   CONTINUE
C
C     CHECK IF NEW STIFFNESS IS NEEDED
C
      IF (NEL.EQ.1) GO TO 160
      IF ((IPROP.NE.IPOLD).OR.(WTH.NE.WTHOLD).OR.(HTH.NE.HTHOLD).OR.(ITY
     1PE.NE.ITOLD).OR.(IXX.NE.IXXOLD).OR.(IYY.NE.IYYOLD).OR.(IZZ.NE.IZZO
     2LD)) GO TO 160
      GO TO 170
160   IPOLD=IPROP
      WTHOLD=WTH
      HTHOLD=HTH
      ITOLD=ITYPE
      IXXOLD=IXX
      IYYOLD=IYY
      IZZOLD=IZZ
C
C     FORM NEW GLOBAL STIFFNESS,ASA, AND(LOCAL-FORCE)-(GLOBAL-DISPL.),SA
C
      CALL INFILK (E,T,WTH,HTH,IPROP,ITYPE,IXX,IYY,IZZ)
C
C     FORM ELEMENT LOCATION MATRIX,LM
C
170   CONTINUE
      IX(1)=IABS(IXX)
      IX(2)=IABS(IYY)
      IX(3)=IABS(IZZ)+3
      DO 180 M=1,3
      IXA=IX(M)
      LM(M)=ID(NI,IXA)
      LM(M+3)=ID(NJ,IXA)
      LM(M+6)=ID(NK,IXA)
180   LM(M+9)=ID(NL,IXA)
C
C     WRITE ELEMENT INFORMATION ON TAPE
C
      NS=12
```

```
      ND=12
      NDM=24
      CALL CALBAN (MBAND,NDIF,LM,XM,ASA,RF,ND,NDM,NS)
      IF (MODEX.EQ.1) GO TO 190
      WRITE (1) ND,NS,(LM(I),I=1,ND),((SA(I,J),I=1,NS),J=1,ND),((ZERO(I,
     1J),I=1,NS),J=1,4)
C
C     CHECK FOR LAST ELEMENT
C
190   IF (NELEM-NEL) 80,210,200
200   CONTINUE
      IF (ML.GT.0) GO TO 70
      IN=INI
      JN=INJ
      KN=INK
      LN=INL
      INCR=INC
      GO TO 50
210   RETURN
C
220   FORMAT(1H1/* I N F I L L   E L E M E N T S*///
     141H             NUMBER OF INFILL ELEMENTS     =,I5/
     241H             NUMBER OF INFILL PROPERTY SETS=,I5)
230   FORMAT(///26H INFILL ELEMENT PROPERTIES,//
     1T2,*TYPE*,T14,*MODULUS*,T27,*THICKNESS*)
240   FORMAT(I5,2F10.0)
250   FORMAT(I5,2(5X,G10.3))
260   FORMAT(///20H INFILL ELEMENT DATA,//
     1T5,*ELEMENT*,T21,*NODE*,T36,*NODE*,T51,*NODE*,T66,*NODE*,
     2T81,*PROP.*,T95,*WIDTH*,T109,*HEIGHT*,T120,*ITYPE*/
     3T23,*I*,T38,*J*,T53,*K*,T68,*L*,T82,*SET*)
270   FORMAT(6I5,2F10.0,2I5)
280   FORMAT(36H0ELEMENT CARD ERROR, ELEMENT NUMBER=,I6)
290   FORMAT(3X,I5,5(10X,I5),5X,2(5X,G10.3),I5)
300   FORMAT(//*.....ADMISSIBLE INFILL ELEMENTS MUST HAVE LOCAL AXIS PAR
     1ALLEL TO ANY PAIR OF GLOBAL AXIS.*/,5X,*ELEMENT*,I4,* IS INADMISSI
     2BLE. EXECUTION TERMINATED.*)
      END

*************************************************************************

      SUBROUTINE INFILK (E,TH,WTH,HTH,IPROP,ITYPE,IXX,IYY,IZZ)
C
C     FORMS GLOBAL STIFFNESS,ASA, AND(LOCAL-FORCE)-(GLOBAL-DISPL.),SA
C
      COMMON/EM/LM(24),ND,NS,ASA(24,24),RF(24,4),XM(24),SA(12,24)
      DIMENSION E(1),TH(1),IT(12,12),T(50),SHIFT(3,3)
C
C     FORM NONDIMENSIONAL STIFFNESS TERMS ,T, AND LOCATION ARRAY ,IT.
C         ITYPE.EQ.1 COMPLETE INFILL PANEL WITH BOTH TRANS. AND LONG.
C                    EDGE CONSTRAINT.
C         ITYPE.EQ.2 COMPLETE INFILL PANEL WITH ONLY TRANS.EDGE
C                    CONSTRAINT.
C         ITYPE.EQ.3 PARTIAL INFILL PANEL WITH BOTH TRANS. AND LONG.
C                    EDGE CONSTRAINT.
C         ITYPE.EQ.4 PARTIAL INFILL PANEL WITH ONLY TRANS. EDGE
C                    CONSTRAINT.
C
      CALL NONDIM (WTH,HTH,IT,T,ITYPE)
C
C     FORM LOCAL STIFFNESS
C        TT,TR,RR ARE FACTORS TO UN-NONDIMENSIOALIZE STIFFNESS TERMS
C        TR=FACTOR FOR TRANSLATION TO ROTATION TERMS. TT,RR SIMILARLY.
C
      TT=E(IPROP)*TH(IPROP)
      TR=TT*SQRT(WTH*WTH+HTH*HTH)
      RR=TT*WTH*HTH
      DO 10 I=1,12
      DO 10 J=1,12
      NT=IT(I,J)
      XTS=ISIGN(1,NT)
      NT=IABS(NT)
      IFLAG=0
      JFLAG=0
      IF (MOD(I,3).EQ.0) IFLAG=1
      IF (MOD(J,3).EQ.0) JFLAG=1
      IJ=IFLAG+JFLAG
      IF (IJ.EQ.0) SA(I,J)=XTS*T(NT)*TT
      IF (IJ.EQ.1) SA(I,J)=XTS*T(NT)*TR
      IF (IJ.EQ.2) SA(I,J)=XTS*T(NT)*RR
10    ASA(I,J)=SA(I,J)
C
C     IMPROVE RIGID BODY MODES BY SHIFTING
C
C     FORM RIGID BODY MODE VECTOR, PHI, WRITE OVER SA
      A=SQRT(4.*WTH*WTH+4.*HTH*HTH+16.)
      DO 20 I=1,10,3
      SA(I,1)=0.5
      SA(I,2)=0.0
20    SA(I,3)=HTH/A
      DO 30 I=2,11,3
      SA(I,1)=0.0
      SA(I,2)=0.5
30    SA(I,3)=WTH/A
      DO 40 I=3,12,3
      SA(I,3)=2.0/A
      DO 40 J=1,2
40    SA(I,J)=0.0
      SA(1,3)=-SA(1,3)
      SA(2,3)=-SA(2,3)
      SA(4,3)=-SA(4,3)
      SA(8,3)=-SA(8,3)
C
C     FORM EIGENVALUE SHIFT MATRIX - SHIFT(3,3)
C
      DO 80 J=1,3
      DO 60 M=1,12
      SUM=0.0
      DO 50 K=1,12
50    SUM=SUM+ASA(M,K)*SA(K,J)
60    T(M)=SUM
      DO 80 I=1,3
```

```
      SUM=0.0
      DO 70 K=1,12
70    SUM=SUM+SA(K,I)*T(K)
80    SHIFT(I,J)=SUM
C
C     REPLACE STIFFNESS MATRIX (ASA) WITH (ASA - PHI*SHIFT*PHI TRANS)
C
      DO 120 J=1,12
      DO 100 M=1,3
      SUM=0.0
      DO 90 K=1,3
90    SUM=SUM+SHIFT(M,K)*SA(J,K)
100   T(M)=SUM
      DO 120 I=1,12
      SUM=0.0
      DO 110 K=1,3
110   SUM=SUM+SA(I,K)*T(K)
120   ASA(I,J)=ASA(I,J)-SUM
      DO 130 I=1,12
      DO 130 J=1,12
130   SA(I,J)=ASA(I,J)
C
C     FORM GLOBAL STIFFNESS,ASA AND (LOCAL-FORCE)-(GLOBAL-DISPL.),SA
C
      DO 200 L=1,3
      GO TO (140,150,160), L
140   IF (IXX.GT.0) GO TO 200
      KK=1
      GO TO 170
150   IF (IYY.GT.0) GO TO 200
      KK=2
      GO TO 170
160   IF (IZZ.GT.0) GO TO 200
      KK=3
170   DO 190 K=1,4
      KKK=KK+(K-1)*3
      DO 180 I=1,12
      SA(I,KKK)=-SA(I,KKK)
180   ASA(I,KKK)=-ASA(I,KKK)
      DO 190 J=1,12
190   ASA(KKK,J)=-ASA(KKK,J)
200   CONTINUE
      RETURN
      END
```

```
*****************************************************************************
```

```
      SUBROUTINE NONDIM (WTH,HTH,IT,T,ITYPE)
      COMMON/NOND/C1(4,22),C2(4,22),C3(4,42),C4(4,23),ITV(4,78)
      DIMENSION IT(12,12),T(50)
C
C     COMPUTE INDEPENDANT STIFFNESS TERMS
C
      ASPECT=WTH/HTH
      GO TO (10,30,50,70), ITYPE
10    DO 20 I=1,22
20    T(I)=C1(1,I)+C1(2,I)*ASPECT+C1(3,I)*ASPECT**2+C1(4,I)*ASPECT**3
      GO TO 90
30    DO 40 I=1,22
40    T(I)=C2(1,I)+C2(2,I)*ASPECT+C2(3,I)*ASPECT**2+C2(4,I)*ASPECT**3
      GO TO 90
50    DO 60 I=1,42
60    T(I)=C3(1,I)+C3(2,I)*ASPECT+C3(3,I)*ASPECT**2+C3(4,I)*ASPECT**3
      GO TO 90
70    DO 80 I=1,23
80    T(I)=C4(1,I)+C4(2,I)*ASPECT+C4(3,I)*ASPECT**2+C4(4,I)*ASPECT**3
C
C     FORM LOCATION ARRAY,IT
C
90    DO 100 I=1,12
      DO 100 J=I,12
      IJ=(I*(25-I))/2-12+J
100   IT(I,J)=ITV(ITYPE,IJ)
      DO 110 I=2,12
      I1=I-1
      DO 110 J=1,I1
110   IT(I,J)=IT(J,I)
      RETURN
      END
```

```
*****************************************************************************
```

```
      BLOCK DATA IFDATA
C
C     DATA USED BY SUBROUTINE NONDIM
C
      COMMON/NOND/C1(4,22),C2(4,22),C3(4,42),C4(4,23),ITV(4,78)
      DATA(C1(I),I=1, 80)/ .7265E+00,-.3498E+00, .1390E+00,-.1373E-01,
     1-.1488E+00,-.2691E-03, .3177E-03,-.5013E-04,
     2 .8107E-01,-.5702E-01, .1697E-01,-.1718E-02,
     3-.7130E+00, .5679E+00,-.1385E+00, .1365E-01,
     4 .7262E-01,-.5619E-03, .4132E-04, .3585E-05,
     5-.7422E-01, .5316E-01,-.1594E-01, .1617E-02,
     6 .3202E+00,-.3662E+00, .6147E-01,-.6004E-02,
     7-.5925E-01, .6881E-01,-.2034E-01, .2043E-02,
     8-.3332E+00, .1472E+00,-.6162E-01, .6038E-02,
     9 .6609E-01,-.7267E-01, .2138E-01,-.2145E-02,
     $ .3003E+00, .1434E+00, .6471E-01,-.6443E-02,
     $-.2222E-02, .3828E-01, .3652E-02,-.4065E-03,
     $-.3049E+00, .3753E+00,-.6772E-01, .6823E-02,
     $ .4402E-01,-.5872E-01, .6549E-02,-.5929E-03,
     $ .1625E+00,-.4514E+00, .2084E-01,-.1959E-02,
     $ .3047E-02,-.3363E-01,-.5392E-02, .5911E-03,
     $-.1619E+00,-.6105E-01,-.2040E-01, .1896E-02,
     $-.4343E-01, .6373E-01,-.8445E-02, .7958E-03,
     $ .3844E-01,-.2163E-01, .1459E-01,-.6170E-03,
     $-.9277E-02, .8225E-02,-.9903E-02, .3347E-03/
      DATA(C1(I),I=81,88)/ -.2526E-01, .3084E-01,-.1704E-01, .8419E-03,
     1 .4687E-02,-.1488E-01, .1236E-01,-.6149E-03/
      DATA(C2(I),I=1, 80)/ .7095E+00,-.3626E+00, .1063E+00,-.1067E-01,
     1-.1261E+00, .2550E-01,-.8947E-04,-.4382E-03,
     2 .7244E-01,-.5291E-01, .1646E-01,-.1750E-02,
```

```
3-.7324E+00, .5770E+00,-.1744E+00, .1758E-01,
4 .4949E-01,-.2563E-01, .1280E-02, .4344E-03,
5-.7686E-01, .5977E-01,-.1707E-01, .1717E-02,
6 .3372E+00,-.3533E+00, .9423E-01,-.9071E-02,
7-.6786E-01, .7290E-01,-.2084E-01, .2011E-02,
8-.3134E+00, .1376E+00,-.2553E-01, .1681E-02,
9 .6348E-01,-.6608E-01, .2024E-01,-.2045E-02,
$ .1997E+00, .1986E+00, .4905E-01,-.4891E-02,
$-.1115E-01, .4307E-01, .2667E-02,-.3388E-03,
$-.1981E+00, .3103E+00,-.4786E-01, .4737E-02,
$ .3528E-01,-.5423E-01, .5703E-02,-.5437E-03,
$ .1272E+00,-.4636E+00, .2709E-01,-.2570E-02,
$ .1697E-01,-.4859E-01,-.9909E-03, .1710E-03,
$-.1270E+00,-.4832E-01,-.2691E-01, .2551E-02,
$-.2917E-01, .4823E-01,-.3799E-02, .3444E-03,
$ .2621E-01,-.1760E-01, .1398E-01,-.6128E-03,
$-.9971E-02, .6304E-02,-.8614E-02, .1809E-03/
 DATA(C2(I),I=81,88)/-.2694E-01, .3164E-01,-.1772E-01, .9203E-03,
1 .1424E-01,-.1959E-01, .1336E-01,-.6773E-03/
 DATA(C3(I),I=1,80 )/ .5890E+00,-.1341E+00, .2728E-01,-.1671E-02,
1 .1289E+00, .1164E-01,-.2593E-02, .1514E-03,
2 .6575E-01,-.3092E-01, .5458E-02,-.3132E-03,
3 .5275E+00,-.3149E+00, .5881E-01,-.3417E-02,
4 .7571E-01,-.4725E-01, .9265E-02,-.5703E-03,
5 .5564E-01,-.3550E-01, .6949E-02,-.4175E-03,
6-.2332E+00, .2510E+00,-.3995E-01, .2154E-02,
7-.1210E+00, .7657E-01,-.8935E-02, .3639E-03,
8-.4158E-01, .3886E-01,-.7012E-02, .3937E-03,
9 .2947E+00,-.7019E-01, .8430E-02,-.4073E-03,
$ .1742E+00,-.1768E-01,-.2923E-02, .3578E-03,
$-.4343E-01, .4388E-01,-.9255E-02, .5668E-03,
$ .3105E+00, .2831E-02, .7824E-02,-.3413E-03,
$-.4681E-02, .2804E-02,-.5067E-03, .2897E-04,
$-.3598E-01, .2450E-01,-.4583E-02, .2822E-03,
$-.1080E-02, .1026E-02,-.2138E-03, .1407E-04,
$ .5939E-01, .4178E-01,-.7915E-02, .4538E-03,
$ .2692E-01, .1626E+00,-.2175E-01, .1382E-02,
$ .2490E-01, .9673E-02,-.3158E-02, .2220E-03,
$ .1452E+00,-.7738E-01, .1459E-01,-.8726E-03/
 DATA(C3(I),I=81,160)/.2476E+00,-.1353E+00, .2499E-01,-.1441E-02,
1 .1628E-01,-.2870E-04,-.9599E-03, .9256E-04,
2 .2533E-01,-.1270E-01, .2304E-02,-.1338E-03,
3 .1070E-01,-.6969E-02, .1394E-02,-.8436E-04,
4-.3372E-01, .2623E-01,-.5226E-02, .3138E-03,
5 .2016E-01,-.8794E-02, .1323E-02,-.6664E-04,
6-.1593E-01, .9344E-02,-.1674E-02, .9562E-04,
7 .4383E-01,-.2166E-01, .3735E-02,-.2095E-03,
8 .1656E-01,-.7016E-02, .1030E-02,-.5174E-04,
9-.6503E-02, .5665E-02,-.1143E-02, .6760E-04,
$ .5723E+00,-.1346E+00, .2810E-01,-.1741E-02,
$-.1905E+00, .1067E+00,-.1426E-01, .6662E-03,
$ .4776E-01,-.1391E-01, .3009E-02,-.1999E-03,
$-.5108E+00, .3154E+00,-.5963E-01, .3487E-02,
$ .1047E+00, .1245E-01,-.8244E-02, .6601E-03,
$ .4592E-01,-.8898E-02, .7659E-03,-.2692E-04,
$ .4012E+00,-.3078E-01, .1177E-01,-.4947E-03,
$ .3604E-01, .7791E-02,-.3303E-02, .2430E-03,
$ .1267E+00,-.5811E-01, .8534E-02,-.4356E-03,
$ .5141E-02, .1853E-02,-.8556E-03, .7162E-04/
 DATA(C3(I),I=161,168)/ .2363E-01, .8083E-02,-.1791E-02, .1005E-03,
1 .6416E-02, .7200E-02,-.1956E-02, .1308E-03/
 DATA(C4(I),I=1, 80)/ .5554E+00,-.1851E+00, .3431E-01,-.1991E-02,
10.          ,0.          ,0.          ,0.          ,
2 .6532E-01,-.3416E-01, .6183E-02,-.3574E-03,
3 .5507E+00,-.2943E+00, .5510E-01,-.3261E-02,
4 .5580E-01,-.3281E-01, .6231E-02,-.3692E-03,
5-.2459E+00, .1929E+00,-.3194E-01, .1786E-02,
6 .1030E+00,-.1750E-01,-.7635E-03, .1792E-03,
7-.5396E-01, .4046E-01,-.7084E-02, .3889E-03,
8 .2507E+00,-.8366E-01, .1115E-01,-.5161E-03,
9-.4243E-01, .3311E-01,-.6727E-02, .4065E-03,
$ .2233E-01,-.1159E-01, .2123E-02,-.1237E-03,
$ .1181E-01,-.7232E-02, .1412E-02,-.8440E-04,
$-.3474E-01, .2335E-01,-.4568E-02, .2736E-03,
$ .9390E-02,-.5080E-02, .9010E-03,-.5095E-04,
$-.1504E-01, .9577E-02,-.1840E-02, .1090E-03,
$ .4426E-01,-.2470E-01, .4519E-02,-.2618E-03,
$-.8961E-02, .5937E-02,-.1171E-02, .6994E-04,
$ .4610E-01,-.1705E-01, .3666E-02,-.2420E-03,
$ .5762E-01,-.2440E-01, .4024E-02,-.2245E-03,
$ .2975E+00,-.1563E+00, .2699E-01,-.1490E-02/
 DATA(C4(I),I=81,92)/ .1423E-01,-.8234E-02, .1506E-02,-.8659E-04,
1 .1325E-01, .2800E-02,-.2222E-03,-.7326E-05,
2 .4677E-02, .4576E-02,-.1233E-02, .7950E-04/
 DATA(ITV(I),I=1,312)/1,1,1,1,2,2,-2,2,3,3,3,3,4,4,-4,-4,5,
15,5,2,6,6,-6,-5,7,7,-7,-6,-5,-5,8,-7,8,8,9,8,9,9,-10,-9,-2
2,-2,11,7,10,10,-12,-10,11,11,13,2,12,12,-14,2,-5,-5,-5,2,1
33,13,15,2,14,14,-16,2,5,5,17,2,15,15,-18,2,16,16,-19,2,-2,
4-2,20,2,17,17,-21,2,18,18,22,2,19,19,23,11,6,6,-6,-5,-14,
5-14,16,2,20,20,-24,-12,-8,-8,-25,-13,16,16,-26,-14,21,21,2
67,15,-10,-10,-28,-16,-18,-18,29,14,22,22,-30,-17,1,1,1,1,-
72,-2,2,2,3,3,3,3,9,9,-10,-9,2,2,-11,-7,10,10,-12,-10,7,7,-
87,-6,5,5,-8,7,8,8,9,8,11,11,13,2,-12,-12,14,2,2,2,-20,2,17
9,17,-21,2,-18,-18,-22,2,-5,-5,-17,2,15,15,-18,2,-16,-16,19
$,2,19,19,23,11,-10,-10,-28,-16,18,18,-29,-14,22,22,-30,-17
$,-8,-8,-25,-13,-16,-16,26,14,21,21,27,15,1,1,31,1,-2,-2,-3
$2,7,-3,-3,-33,-18,4,4,34,-4,-5,-5,-35,-7,-6,-6,36,19,11,11
$,37,20,12,12,38,21,5,5,35,7,13,13,-39,-20,14,14,-40,21,19,
$19,41,22,-6,-6,36,19,-14,-14,40,-21,20,20,-42,-23,1,1,31,1
$,-2,2,32,-7,-3,-3,-33,-18,11,11,37,20,-12,-12,-38,-21,19,19
$,41,22/
 END
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