CORRELATION OF THE ANALYTICAL AND EXPERIMENTAL RESPONSES OF A 1/5-SCALE MODEL SEVEN-STORY R/C FRAME-WALL STRUCTURE

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SUMMARY

Analysis carried out before a reinforced concrete frame-wall structure was tested on the Earthquake Simulator at the University of California at Berkeley did not correlate well with the experimental response, particularly with respect to the structure's total lateral load capacity. It has been found from studying the experimental response that three-dimensional behavior, which was neglected in the analytical studies, has a profound effect on overall strength. Also, it was determined that the slab reinforcement contributed more to lateral strength than originally computed. In this paper, these effects are quantified, and techniques are suggested for improving the analytical response.

INTRODUCTION

As part of the comprehensive U.S.-Japan Cooperative Research Program on the Use of Large Scale Test Facilities (Ref. 1), a 1/5-scale model of a 7-story R/C frame-wall structure has recently been tested on the Earthquake Simulator at the University of California at Berkeley. This structure, shown in Fig. 1, consists of three parallel frames in the principal direction: two exterior moment-resisting frames (Frames A and C), and one interior frame-wall system (Frame B). A review of the design of the prototype of this structure is given in Ref. 2, and a summary of the overall test program is reported in Ref. 3.

Prior to the selection of the scale for the model to be tested, extensive analytical computations were performed (Ref. 2) using currently existing computer programs (Refs. 4-6). The primary objectives of these computations were to evaluate the strength and deformation capacity of the structure, and to use these results to select the largest scale of the model that could be tested on the Earthquake Simulator at Berkeley.

Subsequent to the testing of the model on the shaking table, it was found that the structure resisted lateral inertial forces far in excess of that predicted before the tests. There are two main reasons for the low computed strength: first, the effect of slab reinforcement on the strength of the girders was significantly underestimated, and second, certain aspects of the three-dimensional behavior of the structure, which were ignored in the analytical model, were found to contribute greatly to the lateral load capacity of the experimental specimen.

EXPECTED RESPONSE OF THE STRUCTURE

In the earliest stage of the project, after agreement had been reached about the design of the prototype, it was clear that due to the large relative stiffness and strength of the centrally loaded shear wall, this wall would dominate the response of the structure in the principal direction. Analytical studies carried out at Berkeley on the prototype structure (before the tests were run) confirmed the dominance of the wall in both the elastic and inelastic ranges of response. This dominance is particularly apparent if the response is controlled by flexural behavior. Once the shear wall hinges at its base, subsequent displacements are essentially rigid body, with rotation occurring about the base of the wall.

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ACTUAL RESPONSE OF STRUCTURE

The response of the test structure was clearly dominated by the shear wall but not exactly in the manner that was expected. During moderate loadings, it was noticed that a large flexural crack had formed at the base of the wall, with this crack extending the full width of the wall panel. Subsequent to the formation of this crack, the wall rotated not about its centroidal axis, but rather about the compression-side boundary element. This behavior, termed "rocking," caused three-dimensional interaction between the exterior and interior frames, with the transfer mechanism being the transverse girders spanning between the frames, i.e. framing perpendicular to the plane of the wall.

The lateral load capacity of the test specimen was much greater than that computed prior to testing. In order to obtain an understanding of this overstrength, an extensive series of limit analyses have been carried out (Ref. 7). From the results of this limit analysis, the individual components of structural strength may be identified and measured.

Modeling of Basic Mechanism

The limit analyses were carried out under monotonically increasing lateral forces corresponding to two patterns: inverted (upper) triangular, and rectangular (uniform). These loads should bound the response of the actual structure, whose inertial force profiles remained within these limits during the peak response. Three different mechanisms are considered. First, it is assumed that the shear wall rotates about its centroidal axis, and that only the girders in Frames A, B, and C contributed to the strength. In other words, the transverse girders are assumed to have an insignificant effect on the response when the inertial forces are parallel to the principal frames. Note that this is a very common modeling assumption, is implicitly assumed in computer programs such as TABS and DRAIN-TABS (Refs. 8 & 9), and was used in the pre-test strength predictions.

For the second analysis, it is recognized that under large lateral deformations, the extreme fibers of the shear wall will displace vertically, up for the tension side and down for the compression side. However, since the shear wall is assumed to rotate about its centroidal axis, the vertical deformations at the extreme fibers of the shear wall are equal and opposite. These vertical deformations must be resisted by the transverse beams, which may subsequently yield, thus increasing the internal work performed by the structure.

In the third case, it is assumed that the shear wall rotates not about its centroidal axis, but about the centerline of the compression-side boundary element. This is the mechanism that was observed during the shaking table tests. When the shear wall rotates about the boundary element, several things happen. First, the vertical deformations at the extreme fibers of the shear wall are not equal and opposite. Indeed, the tension side upward displacement far exceeds the compression side downward displacement. Therefore, the transverse and longitudinal beams attached to the tension side element yield, while those attached to the compression side do not. Furthermore, since the centerline of the shear wall displaces upwards, the transverse beams attached to the center of the wall may also yield. Since the beams attached to the tension side yield and those on the compression side do not, there is a net increase in axial compressive force in the wall, caused by unbalanced shears being transferred into the wall from the beams. The increase in wall axial force causes an increase in its flexural capacity as well. The net effect of the wall rotating about its compression-side boundary element, then, is to increase significantly the internal work being performed by the structure. Note that the above description is for the behavior immediately after rocking commences (under monotonically increasing load). When load and deformations reverse, the transverse and longitudinal beams may also restrain the tendency of the wall to grow in length.

Another effect of this wall rocking is to cause a portion of the gravity forces in the structure to be lifted vertically, thus decreasing the external work being performed. This lifting phenomenon occurs only when the mechanism includes shear wall rocking about the compression side boundary element.

The results from the three limit analyses are given in Table 1. The three cases—1, 2, and 3—represent the idealizations described above. As can be observed from the table, the computed lateral strength F of the structure increases by 31% when the full three-dimensional behavior, including wall rocking, and yielding of transverse beams, is considered.

TABLE 1 RESULTS OF LIMIT ANALYSIS

LOAD / CASE	INTERNAL WORK (k-in.)			EXTERNAL WORK (k-in.)			LATERAL STRENGTH F (k)		
	1	2	3	1	2	3	1	2	3
TRIANGULAR	4694	5118	5658	122F	122F	(122F-506)	38.5	41.9	50.5
UNIFORM	4694	5118	5658	101F	101F	(101F-506)	46.5	50.7	61.0

UNITS: Inches and Kips: 1 inch = 25.4mm, 1 kip = 4.45 kilonewton

Components of Resistance for Case 3

It is interesting to break down the contributions of each element in the structure to the total internal work shown for Case 3. This information is summarized in Table 2. As can be observed from the table, the majority of the work is being done by the shear wall and by the girders. Therefore, the global response of this structure is extremely sensitive to the strengths of these components.

Any uncertainties that exist in the determination of the strengths of the girders or shear wall will lead to uncertainties in the response of the structure. For the shear wall, assuming that the material properties, material distribution, and axial load are known, the flexural strength may be computed with little error. When the wall is loaded axially in compression, but well below the balance point, however, and when the reinforcement ratio is small (as in the case being considered), uncertainties in the axial load and material properties, particularly the reinforcement, may have a large effect on the computed flexural strength of the wall, as can be seen from the interaction diagram of Fig. 2. Any variation of the strength of the wall has a significant influence on the strength of the structure, as shown in Table 2. The increase in axial load that occurs in the shear wall is due to unbalanced shears being transferred in from the girders, and the maximum increase in wall axial load is a direct function of the flexural strengths of the girders (assuming that they do not fail in shear). Due to tremendous uncertainties in establishing the strengths of the girders (as explained below), significant uncertainties exist in establishing the strength of the shear wall.

TABLE 2 CONTRIBUTIONS TO INTERNAL WORK FOR CASE 3 OF TABLE 1

COMPONENT	INTERNAL WORK (k-in.)	PERCENTAGE OF TOTAL
PRINCIPAL GIRDERS (POS. MOMENT)	799.4	14.1
PRINCIPAL GIRDERS (NEG. MOMENT)	2205.6	39.0
TRANSVERSE GIRDERS	648.0	11.5
COLUMNS	255.2	4.5
SHEAR WALL	1750.0	30.9
TOTAL INTERNAL WORK	5658.2	100.0

UNITS: Inches and Kips: 1 inch = 25.4mm, 1 kip = 4.45 kilonewton

MODELING OF GIRDER BEHAVIOR FOR NONLINEAR ANALYSIS

Since the girders are cast monolithically with the floor slab, and since the floor slab contains reinforcement running parallel to the main reinforcement in the girders, it can be expected that this slab will have a considerable influence on the flexural stiffness and strength of the girder. In order to quantify this, a series of moment-curvature analyses were carried out for each of the girders of the structure (Ref. 7).

Results of Moment-Curvature Analysis

In Fig. 3, the moment-curvature relations for Girder G3 are shown for both positive and negative moment at the face of the support. For each case, three effective flange widths were assumed, corresponding to either the full, 1/2, or 1/4 center-to-center slab dimensions. The basic assumption made in the moment-curvature analysis was that cross sections that are plane before loading, remain plane after loading. While this assumption may be fairly accurate for rectangular sections (with relatively large span to depth ratios, and relatively low shear stress), it may not apply when the sections have very wide flanges since the strains in the slab steel may attenuate sharply for bars located at some distance from the beam web. In order to obtain a better idea of the strength and deformation capacity of the girders as subjected to negative moment, a second series of moment curvature relationships was obtained, with assumptions being made with regard to how the strains attenuate in the slab steel. The results of this analysis, shown in Fig. 4, indicate that the sections may not be as strong as assumed using fully effective flange and plane sections remaining plane, and show also that the ductility (defined as ultimate curvature divided by yield curvature) of the section is significantly greater than the plane sections remain plane ductility. Similar observations have been verified by results from subassemblage tests including Girder G3 (Ref. 10).

Comparison of Actual Strength with Strengths Based on Limit Analysis

The lateral strengths of the structure as given in Table 1 are based on reasonable assumptions with regard to the girders' flexural capacity (including slab steel with attenuating strains). Thus, the computed capacity of 50.5 kips (224.7 kilo-N) for triangular loading, and 61.0 kips (271.4 kilo-N) for uniform loading should provide bounds on the actual strength of the structure as measured during the shaking table tests. Indeed, the largest lateral force resisted by the structure, obtained during the response to a 40 percent g Taft ground acceleration, was measured as 53.9 kips (239.9 kilo-N), which is between the bounds, but closer to the analytical strength determined for triangular loading. This is consistent with the fact that the inertial force profile as recorded at peak response for the Taft test was not quite triangular, but was rather "upper-parabolic." Envelopes of maximum force and displacement as measured during the shaking table tests are compared to the similar response attained in the limit analysis of the Case 3 strengths, as given in Fig. 5.

Analysis carried out prior to the testing of the structure did not include the three-dimensional behavior, nor did it include the full effect of the slab. For this preliminary analysis, the slab was assumed to be effective over only 1/4 of the width of the bay. It is not surprising, then, that the predicted strength of the model was only 53% of that measured during shaking table tests.

NEEDS FOR IMPROVED MODELING TOOLS

Although the three-dimensional aspects of response are easy to include in a limit analysis, it may be quite difficult to include all of the effects in a general purpose static or dynamic nonlinear computer analysis, mainly because these programs do not incorporate all of the modeling tools required to capture the response. Using the currently existing computer programs, and by utilizing proper modeling techniques, some aspects of the three-dimensional response (such as the transverse beam behavior) may be included. Other aspects, such as wall rocking, are unfortunately not so easy to model.

Modeling of Three-Dimensional Behavior and Wall Rocking

In order to capture the three-dimensional aspects of structural response, it is not necessary to model the structure in three dimensions. It has been shown that by using fictitious beams (Ref. 2), or by using extensional springs (Ref. 11), the effects of transverse beams on the stiffness and strength of a structure may be approximated (even when the structure itself is modeled only in two dimensions). When using fictitious beams or springs, it is essential that the non-linear aspects of the response of the transverse beams be included, i.e. these beams must be allowed to yield during the loading.

The modeling of the "rocking" phenomenon of the shear wall is not easy, and requires extensive alterations to the currently existing computer programs. This rocking behavior (or neutral axis migration) actually consists of two components: inelastic deformation within the plastic hinge shown in Fig. 6-A, and fixed-end rotations at the wall-foundation interface, Fig. 6-B. (For the tests of the 1/5-scale model, it was found that due to the presence of a large flexural crack at the base of the shear wall, the fixed-end deformations contributed greatly to the rocking, while the effect of plastic deformation within the hinge contributed very little.) These two effects may be modeled by using an assemblage of beam truss, spring and rigid link elements, as in Ref. 11. Unfortunately, such models, although very effective, may not be created directly, since most non-linear computer codes do not have all of the required components in their element libraries. Therefore, extensive coding effort may be required to incorporate such behavior into the currently existing programs.

Modeling of Girder Sections for Strength and Stiffness

As noted earlier, assumptions regarding the effective width of the slab, or of distributions of strain across the slab, may have a pronounced effect on the strength, stiffness, and ductility of the girders. Uncertainties will remain in the estimation of these quantities until an effort is made to perform coordinated experimental-analytical research on the problem. In the meantime, it would be beneficial to alter the currently existing moment-curvature analysis computer programs such that arbitrary distributions of strain across the slab may be assumed, together with parameters which may affect the shape of the distribution as curvatures increase. Although such interim solutions cannot be expected to yield precise predictions on the behavior of the girders, they may be used to determine how the strength and ductility may be affected by changes in slab width and strain distribution.

SUMMARY AND CONCLUSIONS

It has been shown that by ignoring the three-dimensional aspect of the response of the test structure, the strength under lateral loadings may be underestimated. Furthermore, the influence of slab reinforcement on the strength of the girders and (indirectly) on the maximum moment carried by the shear wall, and thus on the strength of the structure itself, is very critical. Before such behavior may be included in computer analysis of structures, the currently existing programs need to be modified to allow for more comprehensive modeling of the local and global response. In addition, it is essential that coordinated analytical-experimental research be carried out such that the influence of slab steel on the stiffness, strength, and energy dissipation capacity of a structure may be properly evaluated.

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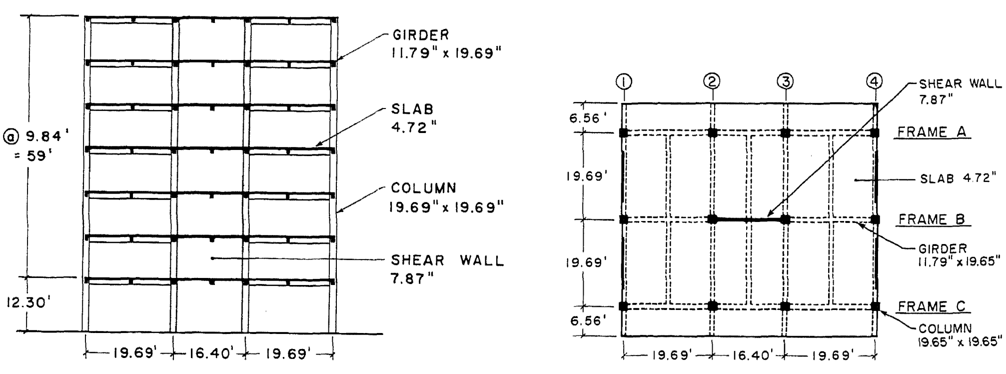

FIG. 1 PLAN & ELEVATION OF PROTOTYPE STRUCTURE

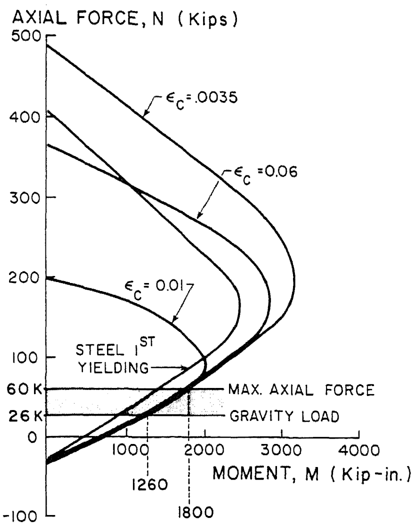

FIG. 2 AXIAL-FLEXURAL FORCE INTERACTION DIAGRAM FOR SHEAR WALL

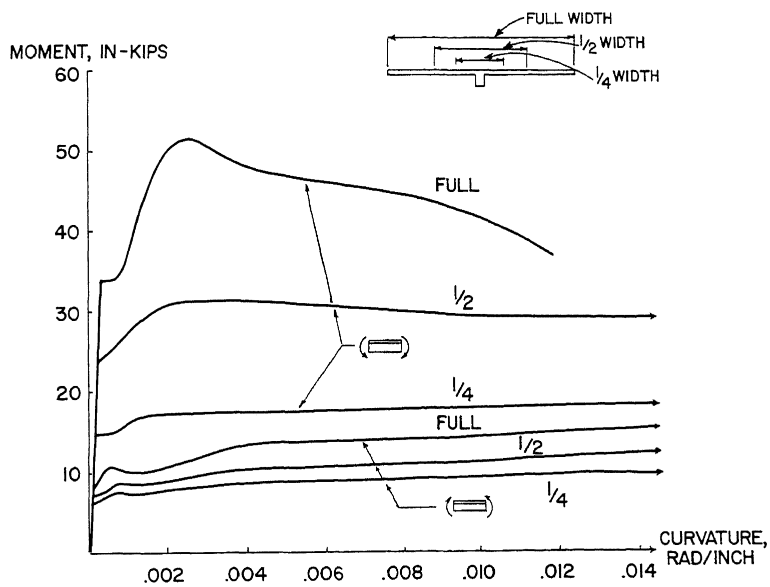

FIG. 3 MOMENT-CURVATURE DIAGRAM FOR GIRDER G3
ASSUMING PLANE SECTIONS REMAIN PLANE

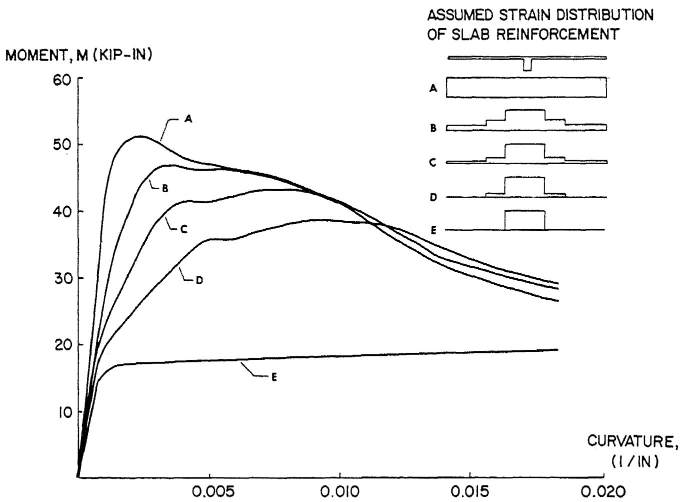

FIG. 4 MOMENT-CURVATURE RELATIONSHIPS FOR GIRDER G3 ASSUMING PLANE SECTIONS DO NOT REMAIN PLANE

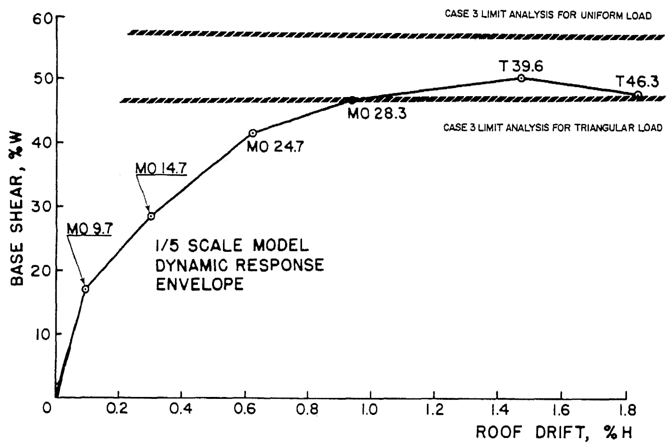

FIG. 5 STRENGTH FROM 3-DIMENSIONAL LIMIT ANALYSIS COMPARED TO EXPERIMENTAL STRENGTHS

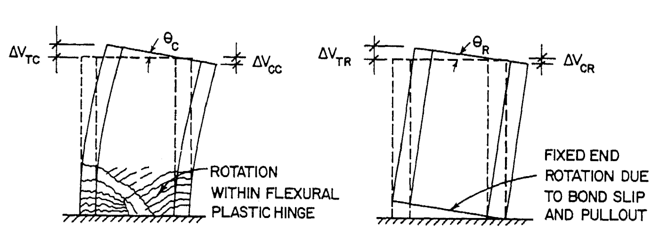

FIG. 6 COMPONENTS OF SHEAR WALL ROCKING BEHAVIOR