Mathematical Modeling of a Steel Frame Structures

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SYNOPSIS

By comparing the computed dynamic response with the performance of a three story steel frame structure observed during shaking table tests, three different mathematical models of the structure are evaluated. The testing was done in two phases: first with under-designed joint panel zones, and second after reinforcing the panel zones. The study demonstrates that a rationally formulated mathematical model can predict adequately both the linear and nonlinear seismic response of steel frames.

Introduction

Many analytical procedures for calculating the nonlinear dynamic response of structures have been developed in recent years. The accuracy of the results obtained with such programs depends, however, on the validity of the mathematical model chosen to represent the structure. Mathematical models often are derived from experimental studies of typical structural components and subassemblages; but their adequacy in depicting dynamic behavior can only be assessed by correlation with the results of tests on complete structures subjected to simulated earthquake motions. The Earthquake Simulator Facility at Berkeley was designed to provide this correlation capability, and results from tests of a three story steel building frame will be considered in this paper. The testing was done in two phases: during Phase I, the joint panel zones were deliberately underdesigned so that they would yield; during Phase II, 3/8 inch doubler plates were welded to each side of the panel zones so that yielding would be forced into the girder or column sections. Preliminary reports on these tests have appeared (1,2,3); subsequently two comprehensive reports were published on the experimental and analytical work, respectively (4,5). In this paper, three mathematical models used for data correlation are described and their relative efficiency is demonstrated.

Test Structure and Instrumentation

The test structure consists of two identical three story moment resistant steel frames, 17'-4" high by 12'-0" span (see Fig. 1), interconnected by rigid floor diaphragms. Columns and girders are W5x16 and W6x12; standard welded connections were provided for Phase I tests, and the doubler plates added for Phase II, as mentioned above. Concrete blocks weighing 8000 lbs were supported at each floor level in a way that did not alter the member stiffnesses. The column base plates were bolted to the shaking table through heavy steel footings to provide fixed end conditions. The dynamic performance of the frame was monitored by 67 instrumentation channels during Phase I, and by 85 channels during Phase II. Each channel was scanned 50 times per second and the signal recorded in digital form on a magnetic disc(6). Instrumentation included accelerometers and displacement gages at the floor levels, as well as strain and curvature transducers at selected column and girder locations; during Phase I, panel zone strains were measured by strain gages and displacement gages (see Figs. 2 and 3).

Test Results

During Phase I with the structure bolted to the laboratory floor, the first mode, free-vibration frequency and damping ratio were found to be 2.24 Hz

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and 0.1% respectively; during Phase II with the structure mounted on the shaking table the corresponding quantities were 2.40 Hz and 0.5%. Four shaking table tests will be discussed here: Phase I tests EC100-I and EC400-I; Phase II tests EC400-II and EC900-II. All used the 1940 El Centro N-S record as the horizontal input signal. Table accelerograms for these tests are shown in Fig. 4; peak accelerations for the four cases were 0.16g, 0.53g, 0.25g and 0.57g respectively. The response was linear elastic during EC100-I and EC400-II; nonlinear strains were produced during the other two tests, as may be seen in Figs. 5 and 6 where the yield strain level (45.9 ksi) is indicated by the horizontal lines. The panel zone distortion ductility factor was found to be about 5 in test EC400-I. Maximum end rotation ductility factors during test EC900-II were 1.9 and 1.2 for columns and girders, respectively; permanent set was seen clearly at the girder ends, but only minimally at the column ends.

Mathematical Models and Data Correlation

For the purpose of calculating dynamic response results to compare with the test data, computer program DRAIN-2D(7) was used; both linear and nonlinear analyses were made with the same program, merely by setting the yield limit appropriately. The first mathematical model (designated Model A) included shear, axial and flexural deformations of columns and girders -- considering their clear span lengths; panel zones were treated as additional shear deformation elements. Masses were assumed lumped at story levels on the column lines, and mass-proportional damping was prescribed giving 0.5% critical in the first mode. The response to EC400-II was computed with this model; the third floor displacement result is shown in Fig. 8. The poor correlation with the observed data is due mainly to the first mode analytical frequency (2.44 Hz) being much higher than the apparent frequency observed during the test (2.25 Hz), even though it agrees well with the free vibration result (2.40 Hz). To demonstrate the source of the poor correlation, Model B was developed from Model A merely by using the center-to-center column lengths rather than clear span (Fig. 7B). The fundamental frequency provided by Model B is 2.24 Hz, and the response given by it is seen in Fig. 9 to correlate well with the observed result.

The third model, Model C, was developed from Model A by including the rotational flexibility of the shaking table (provided by two vertical springs). The spring stiffnesses were adjusted to provide the observed response frequencies (see Table I), otherwise the model was identical to Model A. Correlations of Model C results with observed data are shown in Figs. 10-13. Linear response to EC400-II, for both third story displacement and first floor girder moment are seen in Fig. 10 to correlate perfectly with the test data. The corresponding Phase I linear test (EC100-I) displacement correlation is shown in Fig. 11; to provide this good agreement the first mode damping was taken to be 1.5%. The same damping was used in the analysis of EC400-I, the Phase I nonlinear response, with results shown in Fig. 12. The panel zone yield moment of 170 k-in and 17% post-yield stiffness adopted for this analysis were taken from the experimental results (Fig. 5); dead load and residual stresses were neglected. The final correlation, for the Phase II nonlinear test (EC900-II), is shown in Fig. 13. In this analysis, the post-yield stiffness was set at 8% and 4% for the girders and columns, respectively, reflecting the rather significant yielding which took place. The correlation is generally very good. Note that the plot of girder end rotations compares the observed rotations at the two ends of the same member (not analysis vs. experiment) and shows the significant bias caused by dead load moments.

Conclusions

Based on these correlation studies, the following conclusions may be drawn on the mathematical modeling of steel frame structures.

1. The basic model may be derived from the mass and stiffness properties of the frame components, using clear span dimensions and treating the panel zone as an additional element.
2. It is essential to include the rotational flexibility of the shaking table in modeling the complete dynamic system.
3. Post yield behavior of the members must be specified reliably in both yield level and bilinear stiffness; predicting these from coupon test data may not be easy, due in part to the influence of dead load and residual strains.
4. Further tests will be needed to demonstrate model effectiveness for large nonlinear responses (ductility factors greater than two).

Acknowledgement

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TABLE 1 MODEL PARAMETERS

Test Run	1st Mode frequency (Hz)	Vertical Spring Stiffness (kip/in)	1st Mode Damping (%)
EC400-II	2.312	212	0.5
EC100-I	2.155	212	1.5
EC400-I	2.155	212	1.5
EC900-II	2.279	193	0.5

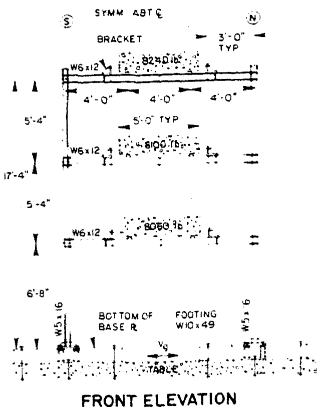

Fig. 1 The Test Structure

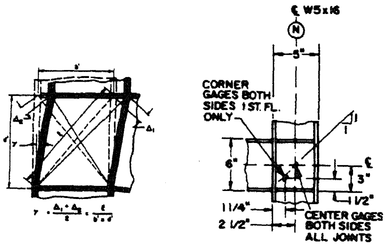

Fig. 2 Deformation Measurements (Phase I)

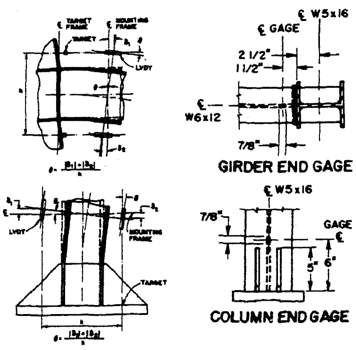

Fig. 3 Deformation Measurement (Phase II)

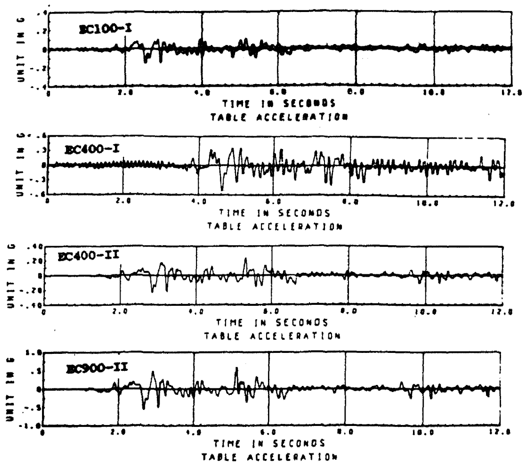

Fig. 4 Measured Table Accelerations

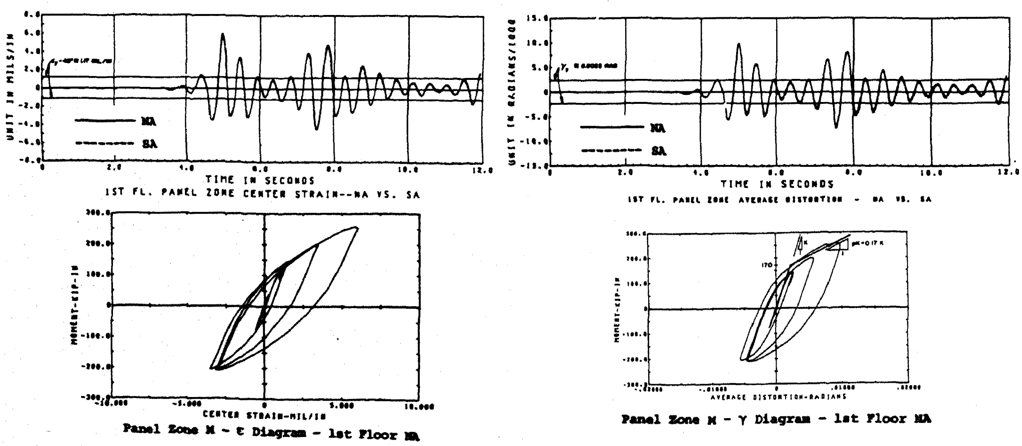

Fig. 5 Measured Nonlinear Structural Responses - EC400-I