

1966

Earthquake response of steel frames (Slide lecture prepared by Committee of Structural Steel Producers and Committee of Steel Plate Producers) AISI, Feb. 1966.

American Iron & Steel

Follow this and additional works at: <http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports>

---

#### Recommended Citation

American Iron & Steel, "Earthquake response of steel frames (Slide lecture prepared by Committee of Structural Steel Producers and Committee of Steel Plate Producers) AISI, Feb. 1966." (1966). *Fritz Laboratory Reports*. Paper 290.  
<http://preserve.lehigh.edu/engr-civil-environmental-fritz-lab-reports/290>

This Technical Report is brought to you for free and open access by the Civil and Environmental Engineering at Lehigh Preserve. It has been accepted for inclusion in Fritz Laboratory Reports by an authorized administrator of Lehigh Preserve. For more information, please contact [preserve@lehigh.edu](mailto:preserve@lehigh.edu).

396

570.7

332.4



INSTITUT FÜR  
UNIVERSITÄT  
FÜR ANGEWANDTE  
WISSENSCHAFTEN  
FÜR INGENIEURWISSENSCHAFTEN  
UND TECHNOLOGIE

EARTHQUAKE RESPONSE OF STEEL FRAMES

Committee of Structural  
Steel Producers

Committee of Steel Plate  
Producers

FRITZ ENGINEERING  
LABORATORY LIBRARY

F.L. Report No. 332.4

WCD

~~WCD~~

## EARTHQUAKE RESPONSE OF STEEL FRAMES

Committee of Structural Steel Producers

Committee of Steel Plate Producers

American Iron and Steel Institute  
150 East 42nd Street  
New York, New York 10017

## SYNOPSIS

This slide lecture describes the major points of three investigations currently sponsored by the Committee of Structural Steel Producers and the Committee of Steel Plate Producers, American Iron and Steel Institute, in the area of the design of steel frames against earthquake forces. The investigations include Professor E. P. Popov's study, "Steel Beam To Column Connections Subjected To Cyclically Reversed Loading," conducted at the University of California; Professor Glen Berg's study of the "Earthquake Response of Inelastic Systems" at the University of Michigan, and the study of "Dynamic Properties of a Steel Frame Building" conducted by Professors J. E. Bouwkamp and R. W. Clough at the University of California.

The slide lecture describes the goals, the interrelationships, and the principal findings of their investigations: the characteristics of the hysteresis loop for beam-to-column connections subjected to load reversals; the analysis of a one-story steel frame building subjected to earthquake motions; and the effect of nonstructural elements on damping properties of the Medical Center Building in San Francisco.

SLIDE #1 (Black)

All of you are well aware of the tremendous property damage and loss in lives that were caused during this decade by the major earthquakes in Morocco, Chile, Iran, Yugoslavia, Libya, and the United States. These reminded us again that one of the worst catastrophes of an earthquake is the collapse and damage of buildings. Thus, one of the important defenses against earthquakes is to build structures that can withstand strong ground motions.

The particular ability of steel structures to resist seismic forces was first recognized in 1907 by the committee of civil engineers that studied the building damage after the San Francisco earthquake. Since that time, and particularly during the past three decades, engineers, scientists and building code officials have been making a concentrated effort to learn more about the structural response of various types of buildings. These studies demonstrated repeatedly the excellent behavior of structural steel frames.

A qualitative explanation of the performance of steel-framed buildings is readily available: because of steel's inherent ductility and strength, and the presence of reliable connections between individual elements, steel structures are tough. However a quantitative explanation is a different matter.

To provide the quantitative explanation needed by code-writing bodies, research is now in progress at the University of California and the University of Michigan

SLIDE #2

under the sponsorship of American Iron and Steel Institute's Committees of Structural Steel and Steel Plate Producers.

SLIDE #3

In an effort to keep you abreast of the latest developments, this first progress report is being presented on

SLIDE #4

the Earthquake Response of Steel Frames. The studies in progress are concerned with three aspects of the problem:

SLIDE #5

- . Behavior of Connections
- . Analysis of frame response and
- . Nonstructural damping

\* \* \* \* \*

Before discussing the three studies now in progress, let us first review

SLIDE #6

the familiar stress-strain relationship for low-carbon steel.

SLIDE #7

If a steel specimen is stressed in direct tension, it elongates in proportion to the stress as long as the stress remains below the yield point. The slope of the stress-strain curve is equal to the modulus of elasticity,  $E$ . If the load is released, the specimen returns to its original length. In other words there is no permanent deformation.

SLIDE #8

For low-carbon structural steel, the strain at the yield point is approximately 0.12%.

SLIDE #9

At the yield point, the material exhibits an appreciable increase in strain with no increase in stress. If the load is released at the strain level indicated by point B,

SLIDE #10

the stress-strain curve returns to the strain axis in a line parallel to that for initial loading. The specimen does not return to its original length - it has a permanent elongation.

SLIDE #11

The area under the curve OABCO represents the energy absorbed by the steel. This property is of basic significance in earthquake

resistance of structures.

If the steel specimen is stressed again, the stress-strain curve returns to

SLIDE #12

point B, and then exhibits yielding as soon as the yield point stress is reapplied. This yielding continues until the point of strain hardening is reached.

It should be noted that the steel deforms plastically about 10 times the strain at the yield point and then commences to strain harden. The total strain at this point is approximately 1.3%.

If the loading continues beyond the point of strain hardening,

SLIDE #13

the stress increases until fracture. Total elongation at fracture is of the order of 20 to 30%; that is more than 10 times the strain at the point of strain hardening, or more than 100 times the yield point strain.

\* \* \* \* \*

Thus far we have examined only tensile loading.

SLIDE #14

For compression, the stress-strain diagram is essentially the same as for tension except for one major difference:

SLIDE #15

buckling may lead to failure at a lower strain.

\* \* \* \* \*

In an earthquake, a structure is subjected to

SLIDE #16

vibrations; thus the one-directional or monotonic loading discussed so far is a very special case; rather, the specimen or structure usually goes through a number of reversals of the load.

How does a specimen respond to a load reversal?

Let us follow the stress-strain relationship of an axially loaded specimen subjected to repeated

SLIDE #17

tension and compression. The corresponding deformations are designated as elongation and shortening.

This type of testing was made on several types of steel and the results are readily available in technical literature.

SLIDE #18

At the start of the test, until the load is reversed at point B, the curve is the same as the monotonic stress-strain curve for a tensile

loading. Note the departure of the actual curve from the idealized curve in the vicinity of the yield point. This rounding-off of the curve is caused by imperfections in the specimen and in the testing techniques.

Upon release of the load,

SLIDE #19

the curve follows a straight line downward. When the compression is applied, the curve at some point starts deviating from the straight line. Note that the corresponding stress ordinate is smaller than the yield stress,  $f_y$ . This phenomenon was observed in the late 1800's by Professor Bauschinger, and is generally referred to as the Bauschinger effect.

Let us now continue

SLIDE #20

the downward loading until the negative deformation or shortening is equal to the previously reached elongation and then reapply the upward loading. The shape of the stress-strain curve on reapplication of the upward loading is the same as that observed during the downward loading.

The resulting closed curve is called the hysteresis loop and the enclosed area represents

SLIDE #21

the energy absorbed per unit volume of the material during one cycle of loading.

A study by Morrow, published in 1965 in ASTM Special Technical Publication No. 378, showed that the life of the specimen is related to this energy.

Morrow reported on low cycle fatigue tests of specimens subjected to repeated reversals of direct stress. Keeping the elongations and shortenings the same for any one specimen, he found that the log of

SLIDE #22

the total energy to failure is proportional to the log of the number of cycles to failure. Furthermore, he showed that monotonic loading, i. e., the ordinary test with direct stress, requires the minimum total energy to failure. According to our definition, the monotonic loading is equal to half-a-cycle.

In a frame subjected to earthquake loading, yielding is likely to occur principally at points of high stress near the beam-to-column connections.

SLIDE #23

The behavior of steel connections under load reversals causing plastic strains is being studied by Professor Popov at the University of California. Thus Professor Popov is essentially evaluating the

properties of the hysteresis loops of several common beam-to-column connections.

SLIDE #24

The first tested was a beam section welded directly to the column flange. The flanges and the web of the beam were welded to the column flange with a full penetration butt weld. An erection angle was tack-welded to the column flange and bolted to the beam web. This angle also served as a back-up bar for the full penetration weld between the beam web and the column flange.

SLIDE #25

The second was a beam section attached to the column with flange plates butt welded to the column and fillet welded to the beam flanges. The beam web was welded to the flange of the column, and the erection angle was attached as in the previous case.

SLIDE #26

The third also had flange plates butt welded to the column but the plates were bolted to the beam flange. The shear transfer was provided for by bolting the beam web to a standard clip angle. All bolts were of the high strength type.

SLIDE #27

The specimens were tested as cantilevers loaded at the free end. Starting from the unloaded position, a load was applied downward at the end of the beam until a desired value of strain was reached in the flanges near the connection.

At this point the downward load was gradually removed and

SLIDE #28

an upward load was applied until the same strain was reached in the opposite direction.

Finally, the load was gradually removed. This overall movement from zero load through the negative maximum to the positive maximum and back to zero load is defined as one cycle of loading.

The connections were subjected to the following sequence of nominal cyclic loading.

SLIDE #29

Three cycles of working stress.

SLIDE #30

Five cycles of 1/2% strain loading - that is the maximum strain in the flanges of the beam fluctuated from +1/2% to -1/2%. This means that the maximum strain was equal to about four times the yield point strain.

SLIDE #31

Five cycles of 1% strain loading came next, and

SLIDE #32

five cycles of 1 1/2% strain loading followed.

SLIDE #33

Finally, if failure did not occur earlier, cycling at  $\pm 2\%$  strain, or 16 times the yield point strain, was continued until the specimen fractured.

SLIDE #34

Here we see a schematic representation of the specimen in the test frame. The maximum positive and negative beam deflections are shown in phantom.

SLIDE #35

Lateral buckling of the cantilever was inhibited by vertical guide channels providing a lateral resistance similar to that provided in building floors.

\* \* \* \* \*

The results are illustrated with the data obtained from tests of the beam connected to the column with the aid of welded flange plates.

SLIDE #36

The moment-deformation relationship for the beam when loaded to the working stress was a straight line. The deformation plotted in this series of slides is the curvature near the connection at the location of the controlling strain gauges.

SLIDE #37

When the strain was increased to 1/2%, so that plastic deformation had taken place, the five successive cycles resulted in the average deformation curve shown.

SLIDE #38

The average loop at 1% strain had similar characteristics to the one for 1/2% controlling strain, but the specimen carried a slightly increased load. Note also the tremendous increase in the absorbed energy.

The five loops obtained from the five cycles of 1% strain

SLIDE #39

differed slightly because of inaccuracies in monitoring the control strains. The shapes of the individual loops show the remarkable stability of structural steel even when strained plastically.

Structural steel does not change its physical character under reversed cycles of loading such as those caused by earthquakes. This unique characteristic is made evident by the stable shape of the

load-deformation curve. The initial slope for all five loops is the same function of the initial modulus of elasticity,  $E$ . Experimental data of hysteresis loops for load vs. maximum deflection exhibit these same general characteristics, except that the initial slope is flatter. For all cases the hysteresis loops for steel showed remarkable stability. The total energy absorbed per unit length of beam is the summation of the areas enclosed by individual loops.

As the values of strain were increased further,

#### SLIDE #40

new loops were formed. Even at these higher strain levels, the initial slope of the hysteresis loops was the same function of  $E$ .

For the three test specimens investigated at the time this report was written, the number of cycles to failure varied from 9 1/2 to 28. Each cycle represented two excursions into the inelastic range; that is, the test connections failed after 19 to 56 excursions into the inelastic range.

It must be remembered that the scheme of loading was such that only five cycles were applied at any particular strain level. After five cycles, strain was increased by  $\pm 1/2\%$ . A substantially greater number of load cycles is required to cause failure at lower strains.

Some indications of this can be seen from earlier research reported by Professors Bertero and Popov in the April 1964 issue of the Journal of the Structural Division, ASCE.

SLIDE #41

Four-inch deep rolled I-beams were clamped at one end and loaded with a fluctuating concentrated load at the free end. The maximum strain at the clamped end was kept the same throughout the test of any one specimen. The load fluctuations were continued until the specimen fractured.

SLIDE #42

The number of cycles to complete failure of the beam was plotted as a function of the controlling strain. When the maximum controlling strain was set at 1%, the fracture of the beam occurred after 650 cycles. As the controlling strain was increased, the fatigue life of the beam rapidly decreased. But even under the controlling strain of 2.5%, equal to about 20 times the strain at first yielding, it took 16 complete cycles, or 32 excursions into the inelastic range, to fracture the specimen.

A similar relationship, although possibly with a somewhat smaller number of cycles to failure, is probably applicable to the connections investigated by Professor Popov.

SLIDE #43

It was interesting to note during the tests of connections that buckling of the beam flanges did not represent failure. As the

curvature was reversed after buckling and the stress in the buckled flange was changed from compression to tension, the buckle gradually disappeared. This behavior will be illustrated in the film of one test that will follow this slide presentation.

First buckling was usually observed at about 1 1/4% strain while failure occurred at larger strains.

SLIDE #44

The location of failure depended on the type of connection. In the bolted connections it occurred through the bolt holes.

SLIDE #45

In the welded connection where the beam was attached to the column with flange plates, failure originated at the end of the fillet weld which joined the beam flange to the moment connection plate.

SLIDE #46

In the welded connection where the beam was welded to the column with a full penetration butt weld, failure originated in the beam flange in the vicinity of the weld.

\* \* \* \* \*

SLIDE #47

While Professor Popov is continuing his work with connections, Professor Berg at the University of Michigan conducts analytical

studies of the earthquake response of single-story and multi-story steel frames. His principal tool is dynamic analysis.

A few years ago all dynamic analyses accounted only for elastic behavior.

Recently, the effect of plastic deformations was introduced in the analytical studies of earthquake response of structures on the basis of the so-called

SLIDE #48

elastoplastic load-deformation relationship.

As we have already learned, the load-deformation curve for steel has an

SLIDE #49

elastic branch followed by a transition curve that leads to the plastic range. Upon reversed deformation, the Bauschinger effect makes the transition more gradual, resulting in the familiar hysteresis loop.

This behavior can be represented quite closely by the Ramberg-Osgood

SLIDE #50

exponential function.

Professor Popov's hysteresis loops for 1% controlling strain are shown with the Ramberg-Osgood curve superimposed. Notice the

closeness of fit - this closeness of fit is typical.

SLIDE #51

Expressed nondimensionally, the Ramberg-Osgood exponential function employs three parameters: the yield load  $Q_y$ , the yield displacement  $x_y$ , and an exponent,  $r$ . The exponent  $r$  governs the sharpness of the break-away from the elastic branch.

Fitting the curve to experimental data requires rather voluminous calculations. Of course, an electronic

SLIDE #52

computer coupled with an automatic plotter solves the problem of

SLIDE #53

curve-fitting in a hurry.

SLIDE #54

Professor Berg completed analytical studies of structures having just one degree of freedom. These represent one-story frames. Research is continuing on multi-story frames.

A structure with one degree of freedom has only one mode of vibration. It can be represented as

SLIDE #55

a mass supported on a column.

The load-deformation relationship for the system used in the analytical studies was represented by the Ramberg-Osgood equation.

In the example that follows, this system was subjected to the

SLIDE #56

ground accelerations recorded in the N-S direction at El Centro during the May 18, 1940 earthquake.

SLIDE #57

The total displacement of the mass, shown in black, was determined as a function of time for the first 20 seconds of the earthquake. The ratio of the total displacement  $x$  to the yield displacement  $x_y$  was plotted against time.

The response curve, shown in black, can be readily compared with the ground acceleration curve, shown in red. It is apparent that the structure tended to vibrate at its own natural frequency rather than at the frequency of the ground motion.

SLIDE #58

Upon close examination of the displacement response curve, it is found that the structure exceeded the yield displacement 14 times. These excursions into the inelastic range are shown in red.

A plot similar to that shown for  $x/x_y$  can be made also for the lateral force  $Q$

SLIDE #59

generated by the acceleration of the mass. The initial yield load  $Q_y$  was used for this nondimensional representation of the response. The ratio  $Q/Q_y$  is shown for the first five seconds of the quake.

With the information on displacements  $x$  and on lateral force  $Q$ , the load-displacement history of the structure may be presented in the form of a series of hysteresis loops.

During the first two and a half cycles, the deformations were elastic. On the subsequent sway the structure reached the yield strain at the

SLIDE #60

ratio of  $x/x_y$  equal to one.

SLIDE #61

Beyond this ratio yielding took place.

SLIDE #62

The direction of load was then reversed and the compressive yield strain was reached when  $x/x_y$  was equal to two.

SLIDE #63

Beyond this ratio, yielding again took place.

SLIDE #64

When the direction of load was reversed, the curve moved upward and the first cycle of loading was completed when the load was equal to zero.

SLIDE #65

As the earthquake continued, the structure swayed back and forth, going through the second cycle of loading shown in blue, the third cycle shown in yellow and so on.

The heavy portions of the curves represent strains in excess of the yield strain.

The total energy absorbed is the summation of the areas

SLIDE #66

enclosed by the individual loops. Thus, by using this technique one can evaluate the total energy absorbed by the frame during an earthquake. Obviously, as long as this total energy is less than the energy absorbing capacity of the frame, the frame can ride out the earthquake safely.

Not all of the energy imparted to a building by an earthquake is absorbed by the structural framing. Other elements, such as the cladding and partitions, contribute their share of damping.

SLIDE #67

The magnitude of this "nonstructural" damping is the primary concern of an investigation conducted by Professors Bouwkamp and Clough at the University of California at Berkeley. The study includes field testing with vibration generators of a multi-story steel-framed building and a thorough analysis of the results.

SLIDE #68

The main building studied was the 15-story east building of the new Medical Center of the University of California in San Francisco. The building is 195 feet high. This particular building was selected since it offered the advantage of almost complete bi-axial symmetry. Furthermore, the absence of shear walls was considered advantageous from the standpoint of the analysis.

SLIDE #69

In plan view, the dimensions of the main east building are 107 feet by 107 feet. The east building has a moment-resistant steel frame, with the floor slabs designed compositely. Adjacent to the main building

is a mechanical service tower structurally independent of the main building. The tower, approximately 20 by 36 feet in plan, is framed with vertical steel trusses

SLIDE #70

encased in concrete.

To evaluate the contribution of the various building components to the overall dynamic response, the plans called for tests at several different stages of construction.

The first series of tests on the 15-story east building were carried out

SLIDE #71

when only the bare steel frame and the reinforced concrete floor slabs were in place.

The second series of tests on the 15-story east building were completed shortly

SLIDE #72

before the writing of this report after the exterior window walls were installed and the frame completely fireproofed. Interior partitions were in place only on the first two floors.

SLIDE #73

The vibration generators, or "shaking machines" used in testing the 15-story building were developed at the Earthquake Research Institute for the California State Division of Architecture. Each machine consists of an electric motor and a drive mechanism that rotates two pie-shaped baskets, or rotors, producing a centrifugal force as a result of the rotation. The speed of rotation can be varied over the practical range of building frequencies.

SLIDE #74

The vibration generators were mounted on the top floor or the roof of the building. By adjusting the orientation of the rotors, the vibration was applied in the north-south or the east-west direction. Five modes of vibration of the building were determined in both directions.

SLIDE #75

Lateral accelerations of the structure were measured by five accelerometers, mounted at several different locations, and recorded continuously during the shaking.

The tests showed a significant increase in damping and a decrease in the natural period of the structure as a result of enclosing the frame with exterior window walls.

SLIDE #76

The damping in the first mode, averaged for the two directions, was found to be 1.15% of the critical for the bare frame but roughly 5% for the frame enclosed in the window walls. The relative increase in damping was even greater for the second, third, fourth and fifth modes of vibration.

SLIDE #77

The period of the first mode decreased from 1.19 seconds for the bare frame to 0.95 second for the frame enclosed in the window walls. Similar decreases were found for the higher modes of vibration.

SLIDE #78

In addition to testing, the bare frame was the subject of theoretical analyses aimed at determining an analytical model that would give natural periods in close agreement with the test values.

The results were most

SLIDE #79

encouraging. The agreement between the test and theory was excellent particularly for the second and third modes of vibration for which the test data were most accurate.

The second phase of the analytical studies concerns the critical damping

SLIDE #80

ratio for each of the modes. In this phase, further computer calculations are necessary for a closer correlation between the experiment and the theory.

Once an accurate analytical determination of the modes of vibration and damping becomes possible, the theoretical studies of the response of a building to an earthquake, such as those conducted by Professor Berg, can be refined further and brought closer to reality.

SLIDE #81

Gentlemen, the steel industry is continuing and expanding its research program in the field of earthquake design. We intend to keep you informed of their progress. The work discussed in this presentation was reported by the investigators in more detail at the 1965 annual meeting of the Structural Engineers' Association of California. A supply of their papers is in the back of the room; you can pick up your copies after the completion of this program. But before we adjourn, I will show you a five minute film of one of

Professor Popov's tests of beam-to-column connections. I am sure you will find it interesting to witness the test.

\* \* \* \* \*

(Film)

\* \* \* \* \*

(Concluding remarks, call for questions)