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Technical Report No. 2

PROCEDURES AND SPECIFICATIONS FOR EXPERIMENTAL DETERMINATION OF LOAD-DEFLECTION CHARACTERISTICS OF FULL-SCALE BUILDINGS

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Approved by
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ABSTRACT

Procedures are outlined for conducting static and dynamic tests on full-scale structures. Buildings which were considered for testing were of steel or reinforced concrete construction, and were either single-story industrial structures or in the range of from 3 to 5 stories. The report includes specifications covering the selection, inspection, and modification of test structures; discussion of vibration, shock, pulldown, and story shear tests; and description of methods of loading, instrumentation, and recording. Estimated costs for the various major items of required test equipment for the static and dynamic tests are tabulated in an appendix.

This report has been reviewed and approved.

FOR THE COMMANDER:

IRVING L. BRANCH
Colonel, USAF
Deputy Commander

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LIST OF ILLUSTRATIONS

OBJECTIVES

The objectives of this program are stated in Call No. 2:

"I. General

. . .

B. Purpose

The purpose of this Call is to set up procedures and specifications for determining the collapse loads and dynamic properties of full scale buildings.

II. Detailed Requirements

A. Specifications

The specifications will be divided into two major parts and will be set up for a maximum and minimum desirable program. The first part of the specifications will establish the requirement for inspection and modifications of the buildings. Provision for repair, removal or reinforcement of parts of the building will also be included.

The second portion of the specifications will pertain to the test procedure, including test descriptions, instrumentation, installation of loading and recording equipment, a system for recording and analyzing data and cost estimates when possible. In connection with this portion of the specifications, a literature search will be made in the field of full scale structural tests, including correlation with earthquake experience, and a summary report prepared.

B. Scope of Work

Structures to be considered in writing the specifications will be:

1. A three to five story steel frame office building.
2. A three to five story reinforced concrete frame office building.
3. A one story steel frame industrial building with trussed roof.
4. A one story reinforced concrete rigid frame industrial building.
5. A one story welded steel rigid frame industrial building.

Tests to be considered for inclusion in the specifications will include the following:

1. Vibration tests, to determine the natural frequencies of vibration in the important normal modes, and to study the damping in each normal mode in order to determine the extent to which the assumption of viscous damping is a satisfactory approximation.

2. Shock tests to determine the actual dynamic response to transient loads, if feasible means of applying sufficiently large impact or impulsive loads can be devised.

3. Story shear tests, to determine the load-deflection characteristics in each story in the elastic and early plastic ranges.

4. Pulldown tests, to determine the actual collapse loads and the load-deflection characteristics over the complete range of deformation. No actual testing of any full scale structure is to be performed as part of this call."

INTRODUCTION

In recent years a number of advances have been made toward the development of procedures suitable for computing the behavior of structures under dynamic as well as static loading. Many of these advances required that certain idealizing assumptions be made (for example, the assumption that Tee-connected beam-to-column connections are perfectly rigid). Some of these assumptions can be justified on the basis of data obtained from static and dynamic laboratory tests on structural members and structural frames; however, the validity of extending such assumptions to an actual structure has not been thoroughly checked. It would be desirable to compare theoretical computations with data obtained from testing complete full-scale structures under both dynamic and static loading, if structures could be obtained for this purpose.

During the course of new highway or new building construction, it is sometimes necessary to destroy existing buildings. With cooperation of proper authorities these buildings may become available for the purpose of full-scale static and dynamic testing. In the event that structures and funds do become available, it is desirable that a general guide in the form of specifications and test procedures be available to expedite setting up and carrying out the tests.

Obviously, not every type of building can be considered for testing. The material contained herein is pointed toward the testing of structures of the following types:

1. Office-type tier buildings of reinforced concrete or steel construction between 3 and 5 stories high.

2. Industrial buildings of the rigid frame type, of either reinforced concrete or steel construction.

3. Industrial buildings with trussed roofs.

I. TEST STRUCTURES

STRUCTURES IN THE 3- TO 5-STORY RANGE

Within the proposed scope, the most difficult buildings to be tested will be those in the 3- to 5-story range. A considerable amount of heavy testing equipment will be required for these structures. Skilled and unskilled labor and heavy construction equipment will be needed to prepare for the tests. Instrumentation will be much more extensive than that required for testing the one-story structures. In the static tests, load and read cycles will be time-consuming.

The buildings in this class vary considerably both in plan and construction. They range from the doubly symmetric rectangular office building to the complicated F-shaped school building. To outline test procedures and specifications for all possible building forms that might become available for testing is out of the question.

The present state of analytical knowledge of the dynamic and static behavior of multistory structures is largely limited to the case of buildings which are rectangular in plan and have at least one plane of structural symmetry. In the experimental field, a considerable number of vibration tests have been made on full-scale structures of all types,[1-4] but most of these tests have been limited to the determination of natural periods of vibration. A literature survey failed to turn up any recorded attempt at conducting story shear or pulldown tests in which an entire multistory structure was involved.*

The lack of previous experimental background in this type of test, together with an incomplete knowledge of methods of analysis for complete structures, makes it desirable to restrict the test program to simple cases so that experimental and analytical results can be compared. This means that at least the initial tests should be restricted to buildings which are rectangular both in plan and elevation, and structurally symmetric or nearly so. Most buildings in the 3- to 5-story range fall in this class. L-shaped buildings might in some cases be modified by removing part of the structure to leave a rectangular, symmetric test structure.

Tests of very irregular structures ("flat iron" buildings, etc.) would be of questionable general value, and should be undertaken only if special circumstances indicate the desirability of such tests. Irregular structures are, of course, much fewer in number than rectangular structures. Variations in plan tend to be rather great, since the plan of an irregular structure is usually dictated by the shape of the lot on which it is built. Since the tests will involve deformations far beyond the elastic range, the load-deflection curves will be sufficiently nonlinear to make it impossible to draw any significant general conclusions from only a few tests.

SINGLE-STORY STRUCTURES

Industrial structures, like tier buildings, are constructed in a variety of sizes and shapes. Fortunately, most of these structures are rectangular in

*Reference 5 contains a summary report of lateral-load tests made on a wing of a 3-story dental hospital in Johnnesburg, South Africa.

plan. If not rectangular, they generally have expansion joints located and constructed in such a way that the building can easily be divided into simple substructures which are rectangular in plan and nearly symmetric as to structure. Practically all industrial structures with trussed roofs will be suitable for testing in an "as is" condition except in the case of fairly old structures. These older structures may be in need of repair due to corrosion, overloading of members, and/or hard usage (e.g., structural members may be damaged from being struck with heavy materials being carelessly handled). Rigid frame structures are apt to be of fairly recent construction and should also be suitable for testing without modification. Three types of rigid frame industrial buildings are considered for testing. They are the single-bay gabled bent, the single-bay shed-type structures, and the portal-type structures 1 or 2 bays wide.

SELECTION, INSPECTION, AND MODIFICATION OF TEST STRUCTURES

It is clear that certain types of structures cannot be considered for testing. It is also clear that structures which are scheduled for testing may be in need of repair before testing can be started. Certain modifications of the structures may also be required to maintain the safety of test personnel and equipment. The decision as to whether any one structure should be tested depends on the design of that particular structure and the magnitude of repairs and/or modifications which are necessary, and should be made only after the structure has been thoroughly inspected.

Appendix A consists of a set of specifications written to guide the testing parties in the selection, inspection, and modification of test structures. The specifications are written in an attempt to insure testing of only the simplest structures, in a condition as near as possible to the condition existing when in service. Because the scope of the structures included in this program is so large, and because there are so many different types of modifications which can be required, it is neither expected nor desired that the specifications be all-inclusive. Each job will require individual specifications.

PRELIMINARY MEASUREMENTS, CALCULATIONS, AND MATERIALS TESTS

The post-test analysis of data and correlation of experimental results with theoretical calculations requires a knowledge of the important dimensions of the structure tested and the physical properties of the structural materials. Nominal dimensions, of course, can be obtained from plans of the structure; however, it is not uncommon for changes to be made in the plans during the construction process. It is therefore necessary that the dimensions of the structure and of its component parts be checked and significant changes recorded on the building plans, which are to be incorporated with test data. This checking operation can become quite time-consuming for some structures. It is therefore limited to checking only the dimensions and spacing of structural members which will be under primary stress during the tests.

The procedures outlined for execution of the static load tests require that a preliminary analysis of each structure be made to determine the elastic yield loads for each case of loading. The time required to carry out the calculations, and the degree to which the calculated loads differ from the actual yield loads, depend on the complexity of the structure and the refinements introduced. Since time may be an important factor, simplifying assumptions and approximate methods may be used to expedite the calculations. In carrying out the tests, loads are applied in increments of approximately 10% of the calculated yield load until actual yield takes place. Loads are applied in this way to insure well-defined elastic load-deflection curves. Therefore the approximations and assumptions used in the analyses should be such that the calculated yield loads will be less than the actual yield loads.

Material constants used in theoretical calculations are to be obtained from physical tests made on samples taken from the structure. Samples for the physical tests are to be taken after all tests have been completed. Specimens are to be taken from undamaged portions of the structure and are to include the following where applicable:

1. Coupons from flanges and webs of steel columns and beams.
2. Concrete specimens from floor slabs, columns, and beams.
3. Specimens of reinforcing steel are to be taken from the floor slabs, columns, and beams. At this time both the size and spacing of reinforcing bars are to be compared with construction plans and significant changes recorded on the plans.
4. Samples from masonry walls. These specimens should be a minimum of four masonry units long by four courses high.

All specimens are to be marked to identify the location or member from which they were taken.

II. DYNAMIC TESTS

Two types of dynamic tests were to be considered for inclusion in the test program: (a) vibration tests to determine the natural frequencies and damping characteristics of the structures, and (b) shock tests to determine the actual dynamic response of the structures to transient loading, if feasible means of applying sufficiently large impact loads could be devised.

VIBRATION TESTS

The objective of the vibration test is to determine the natural frequencies of vibration in the important normal modes, and to study the damping in each normal mode, to determine the nature of the damping and the extent to which the assumption of viscous damping is a satisfactory approximation.

A considerable number of vibration tests have been made on structures of all types.[1-4] Summaries of many of these tests may be found in Ref. 1. In most of these tests the objectives were to determine natural frequencies of vibration and equivalent damping coefficients, without investigating the extent to which the assumption of viscous damping is valid. In these tests it has been customary to excite the structures by means of a single vibrator located on the roof of the building. With proper instrumentation, excitation of this type permits the determination of the natural frequencies and yields sufficient data to permit the plotting of resonance curves, from which equivalent damping coefficients can be calculated. This procedure neither verifies nor disproves the validity of the assumption of viscous damping.

To determine the extent to which the assumption of viscous damping is valid, it is necessary to obtain vibration-amplitude decay curves. The precedure is to excite the test structure in one normal mode, then cut off the exciting forces and observe the decay curves at dynamic pickup stations. If damping is small and truly viscous, the decay record should show:

1. That the decay curves at all stations are exponentially decreasing.
2. That there are no phase shifts between the various output stations.

It becomes increasingly difficult by means of a single shaker to produce normal mode shapes as the order of the mode increases. This is graphically illustrated in Fig. 9 of Ref. 6. As shown in Ref. 6, pure normal mode shapes can be produced at all natural frequencies by using several shakers operating simultaneously.

The possibility of using a multiple shaker system was considered for use in the vibration tests on tier buildings. This idea was discussed at length with Mr. R. T. McGoldrick and other members of the vibrations division at the David W. Taylor Model Basin. During the course of the discussion it was pointed out that the coupling of only two vibrators would be very difficult and expensive, and that the use of a multiple shaker system for buildings would be highly impractical. As a result of these discussions, it was decided that the vibration tests should be conducted as in the past with a single shaker located on the roof of the structure. Testing in this way will permit determination of the natural frequencies of vibration. Also a crude study of the assumption of viscous damping can be made by determining resonance curves for two (or more, if possible) different amplitudes of maximum shaker force input over the frequency range. If the damping is truly viscous, then the equivalent damping coefficients obtained from the various resonance curves should be the same, since response is proportional to the input force in the case of viscous damping. It should be noted that, even if the equivalent damping coefficients corresponding to two different resonance amplitudes do agree, the assumption of viscous damping is not necessarily verified. On the other hand, a disagreement in the equivalent damping coefficients will assure us that damping is not constant, and thus disprove the assumption of pure viscous damping.

VIBRATION-TEST DESCRIPTION

A minimum of four vibration tests is to be run on each structure, two before and two after the "pulldown" test has been completed.* Excitation for the tier buildings is accomplished by means of a single mechanical, counter-rotating, eccentric weight-type vibrator mounted on the roof of the structure. The vibrator is oriented so that vibration takes place in a direction parallel to the strong direction of the building columns. Dynamic displacements are measured at two locations at each floor level and the roof of the structure so that both translational and torsional vibrations can be identified. Output from the various dynamic pickups is simultaneously recorded on a multichannel oscillograph. The driving force produced by the shaker is sinusoidal and of constant amplitude for any one frequency. A permanent record, from which the input frequency of the driving force, and the driving force, can be calculated, is obtained by keying the shaker into the oscillograph so that a timing mark, corresponding to each revolution of the input force vector, is recorded.

The one-story, single-degree-of-freedom structures are set into free vibration by release from an initially applied displacement or by application of an initial velocity. As in the case of tier buildings, dynamic displacements are measured at two locations. However, pickup output is recorded on a two-channel direct-inking Brush-type recorder.

VIBRATOR

Vibration tests conducted in California[1] indicate that the range of the fundamental period of vibration, for structures in the 3- to 5-story range, is from 0.14 sec to 0.95 sec, with an average period of about 0.40 sec. The shortest period of vibration other than the fundamental, 0.03 sec, was for a 5-story reinforced concrete building having brick walls and tile partitions.** Usually, however, the lowest periods other than the fundamental were found to be about 0.10 sec. Based on the average of the periods, with a 20% extension at the extremes, it was concluded that the vibrator should have a working frequency range of from 2 to 13 cps.

Since the buildings that will become available for testing will be scheduled for razing, there should be no objections to heavy vibration testing. Depending on the size and construction of the test structures, shaker force amplitudes of a thousand pounds or more will be required over the full operating frequency range to produce large amplitudes of vibration.

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*Actual collapse of a test structure is not produced in the pulldown test.

**In these tests all periods other than the fundamental are classified as others, and are not designated as second, third, etc.

A heavy vibrator is required to satisfy the force and frequency requirements. One such machine is located at the David W. Taylor Model Basin and possibly could be made available on loan for the vibration tests.* This machine does have the disadvantage of being large and heavy. The machine proper is 53 in. long, 49 in. wide, and 25 in. high, and weighs 5600 lb. The combined weight of the machine and controls, when boxed for shipping, is 6500 lb.

The United States Coast and Geodetic Survey has shakers which have been used in building vibration tests and which can possibly be made available on loan. These machines are compact and can easily be set up by two men. These shakers have the disadvantage of delivering a very small driving force at low operating frequencies, as can be seen from the curve in Fig. 1. If a machine with the characteristics shown in Fig. 1 is used in the tests, the dynamic pickup output signal may be very small and necessitate pre-amplification before recording.

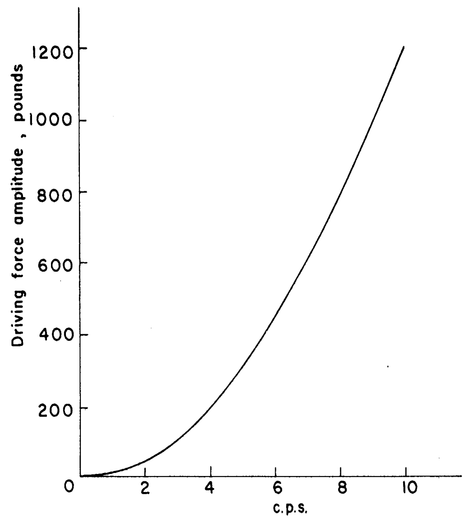


Fig. 1. Vibrator force output (typical for USCGS shakers).

The shaker is installed at the approximate mass center of the roof of the test structure. Alignment of the shaker is to be such that the horizontal driving force will have its line of action parallel to the strong direction of the building columns. If a large shaker is used, such as the TMB medium 5000-lb vibrator, a special foundation base securely attached to the roof slab is necessary to prevent sliding of the machine. The light-weight-type vibrator may be bolted directly to the roof slab. As a safety precaution, the shaker is removed after the first series of vibration tests has been run, and is re-installed after the static tests have been completed.

DYNAMIC PICKUPS AND RECORDING

Dynamic pickups for the vibration tests consist of linear accelerometers, the output of which is integrated twice to give a signal proportional to displacement. To cover the full range of frequencies expected in the tests, it is necessary that the pickups have a linear response range from 0 to 30 cps. Pickups should have either magnetic damping or adjustable fluid damping so that the linear response range can be maintained over a wide variation in temperature (30 to 100°F). A number of accelerometers satisfying these requirements are commercially available. The relative merits and demerits of the various instru-

*This machine is the TMB medium 5000-lb vibration generator. It is capable of delivering an exciting force of 2000 lb at an operating frequency as low as 2 cps. Reference 7 contains a complete description of this machine.

ments are not discussed here. Reference 8, however, contains a summary of the characteristic properties of a great number of dynamic pickups.

Two pickups are installed at each floor level and the roof in tier buildings and are placed symmetrically with respect to the vibrator. In the vibration tests on industrial buildings, accelerometers are attached to the building columns of the two exterior bents. They are installed at the same elevation and as high as possible on the columns so that a maximum pickup output is obtained.

The twice integrated output from the accelerometers is recorded on a multichannel oscillograph in the case of tier buildings, and on a two-channel direct-inking recorder in the case of industrial structures. The oscillograph is to have a minimum of 11 active recording channels to record a possible maximum of 10 deflections, and a timing mark corresponding to each revolution of the shaker. No special type of accelerometers, recorders, or associated accessory equipment is specified since this equipment can very likely be made available on a loan basis. If the equipment must be purchased, the testing parties may have a preference for equipment of a certain make which has given good results in similar tests.

VIBRATION-TEST PROCEDURE

Various procedures may be used in carrying out the vibration tests. The particular method to be used depends on the type of recording equipment employed, the degree of control available on the speed regulation of the vibration generator, and the spread between the natural frequencies of vibration of the structure. The recording equipment and range of expected natural frequencies (2 to 13 cps) are such that all necessary test data can be obtained by recording the vibrations resulting from a single-speed run with the vibration generator. The single-speed-run method, however, requires that the vibration generator be supplied with a speed-control system which will allow for a uniform rate of change of speed, slow enough to permit development of steady-state vibration of the structure. The procedures to be used in these tests are based on the assumption that the vibration generator does not have a speed-control system suitable for a single-speed-run vibration test. This assumption is made since the shaker will be obtained from a source as yet unknown.

The two series of vibration tests are carried out in three phases. In the first phase the approximate natural frequencies of vibration are established from a speed run. The eccentric weights on the shaker are adjusted so that a maximum force amplitude will be obtained at the maximum expected frequency of 13 cps. The vibration generator is then run up to this speed and allowed to coast to a stop. During the coasting period, the roof deflections, as given by the twice integrated output from one of the accelerometers, and the vibration generator speed, are recorded on a two-channel direct-inking-type oscillograph. A plot of the peak deflections against the corresponding shaker speed will give a resonance curve from which the approximate natural frequencies can be determined.*

The second phase of the test consists of obtaining data necessary to plot accurate resonance curves for the structure. One set of points is obtained by putting the structure into forced steady-state vibration at some fixed frequency, and recording this frequency and the output of all dynamic pickups. The sets of points necessary to plot all resonance curves are obtained by repetition of the above process at frequencies of forced vibration both below and above the natural frequencies as determined in phase one. To insure the establishment of a sufficient number of sets of points, the above process should be started at a frequency 1 cps below, increased in steps of 0.10 cps, and finished at 1 cps above each of the natural frequencies as determined in phase one. In this second phase the eccentric weights on the shaker are adjusted so that the machine will produce the maximum possible force amplitude compatible with safe operation of the machine in each of the frequency ranges at which it is run.

The objective of the third phase is to obtain resonance curves corresponding to amplitudes of vibration approximately one-half as large as those obtained in phase two, so that a study can be made of the variation of damping with the amplitude of vibration. The procedures are identical with those used in phase two except for reduction to amplitude one-half as large.

SHOCK TESTS

The objective of the shock test is to determine the actual dynamic response of complete full-scale structures to transient loads. Shock tests were to be included only if feasible means of applying sufficiently large impact loads could be devised. Three possible methods of dynamic load application were considered. Each method could be used successfully; however, each has inherent disadvantages which make it impractical for use in the test program under consideration.

The methods considered and the reasons for their impracticability are:

1. Loading by releasing the energy stored in a device such as the 60-kip pneumatic loading unit in use at the University of Illinois. To develop a total impulse of sufficient magnitude for all structures included in the test program, it would be necessary to have about 10 such units on hand. Whether such devices should be used for impulsive loading of structures depends on the overall cost of manufacturing, testing, maintaining, and installing not only the units but also all accessory equipment. No cost estimates were obtained for setting up such a loading system. However, it is believed that the cost would be extremely high and unjustifiable for this test program.

*Only approximate values for the natural frequencies will be obtained from this curve since it will deviate from the actual resonance curve by an amount which depends on the rate at which the vibration generator slows down.

2. Impact loading as the result of a heavy cylindrical weight or weights rolling down an inclined plane and colliding with the test structure. This method is not objectionable from the point of view of cost, since the plane could be of wood construction and the weights borrowed from a steel mill or manufactured if necessary. This type of loading is hazardous since there is no way to control the motion of the weights after they have been released. Also lack of coordination in contact time may become a serious problem.

3. Dynamic loading as the result of either an air or underground detonation of an explosive charge. The use of explosives is qualitatively ideal from the point of view of shock loading; however, the method has obvious drawbacks which restrict its use to isolated structures. Control and instrumentation would also be a problem.

Although the feasibility studies of shock-test procedures were not carried out in detail, it seemed apparent that much more certain and valuable results could be obtained by a combination of controlled vibration and static load tests. Thus the proposed program does not omit a study of dynamic behavior but omits shock loading as too costly and uncertain to be included.

III. STATIC TESTS

Two types of static tests are to be performed on each test structure, a "pulldown" test and 2-story shear tests. In all these tests, loads are applied horizontally and in the weak direction of the building resistance. Loading is confined to the elastic and early inelastic ranges of deformation in the story shear tests. In the pulldown tests, loads are applied with increasing intensity until the maximum resistance of the structure has been developed. Actual collapse will be avoided.

PULLDOWN TEST

The objectives of the pulldown tests are to determine the actual collapse loads and load-deflection characteristics of the structures over the full or nearly full range of deformation. Collapse loads for a structure are taken to be the loads at which the maximum resistance of the structure is developed. This test is to be conducted under a proportional loading scheme in which several test loads are applied simultaneously. For this type of loading each individual applied load remains a fixed proportion of the total applied load throughout the test.

The test is carried out in a stepwise manner consisting of a number of load-read cycles. Successive load-read cycles are executed until the resistance of the structure reaches a maximum and subsequently decreases to about 20% of the maximum. Loads are applied by means of hydraulic jacks operated from a remotely placed power supply and control system.

During the read cycle, deflections are measured, checked, and recorded. Individual loads and the total loads are also read, checked, and recorded. Since this is a static-loading test and is conducted in steps, data are recorded manually by observers positioned at the various instrument stations. Where possible, the condition of structural members and walls is inspected. Significant changes in their condition, due to the increase of load during the load cycle, are photographically recorded along with the corresponding total load. In some cases, for example, at a location where the formation of a plastic hinge is a certainty, cameras may be used to record automatically conditions at the end of each load cycle. Slaked lime whitewash is used to indicate yielding and the spread of inelastic deformation in bare steel members.

Collapse of a test structure must be prevented to protect personnel and equipment. Collapse of a test structure can come about in two ways. First, it can occur by deforming the structure to the point where effect of dead load is greater than the resistance of the structure. Secondly, collapse can occur as the result of the failure of key structural members and/or connections.

The pulldown tests are to be stopped when, as a result of continued loading, the resistance of the structure has decreased to about 20% of the maximum loads. With this restriction on loading, dead-load collapse can probably be prevented. The 20% figure is arbitrary and may be raised or lowered as the structure and safety requirements warrant.

Collapse due to the failûre of key structural members and/or connections is possible at almost any stage of loading and can occur with little or no forewarning. It is obvious that the loads or deflections at which a structure will collapse in this way cannot be predicted from theoretical considerations. Therefore collapse must be prevented by means of a safety device. The device used in these tests consists of a system of holdback cables which are adjusted to limit the amount the test structure can deflect at any stage of testing. The holdback cable system is included as part of the loading apparatus.

TEST LOADS

The test loads are applied at several locations on each structure. The number of loads and the positions at which they are to be applied will vary according to the type and size of the structure. All applied loads have mutually parallel horizontal lines of action.

In testing tier buildings, the loads are applied parallel to the lines of intersection of the planes of the structural bents with the various floor levels and the roof of the structure. The test load at each location is distributed to all columns in its line of action to minimize possible local crushing of concrete floors, tearing of connections, and local crushing of building columns. The relative magnitude of the various applied loads is such that the sum of the loads at each floor level is the same and the sum of the loads at the roof level is one-half of that at the floor levels. The loads at any one level are balanced about the vertical center plane of the structure. Insofar as possible, loads at any one level are arranged to transfer directly to the bents and to avoid using the floor slabs as load distributing members. Figure 2a shows how the test loads would be arranged for a structure 3 bays wide and 3 stories high and having solid shear walls. Figure 2b shows the load arrangement for the same structure with no shear walls.

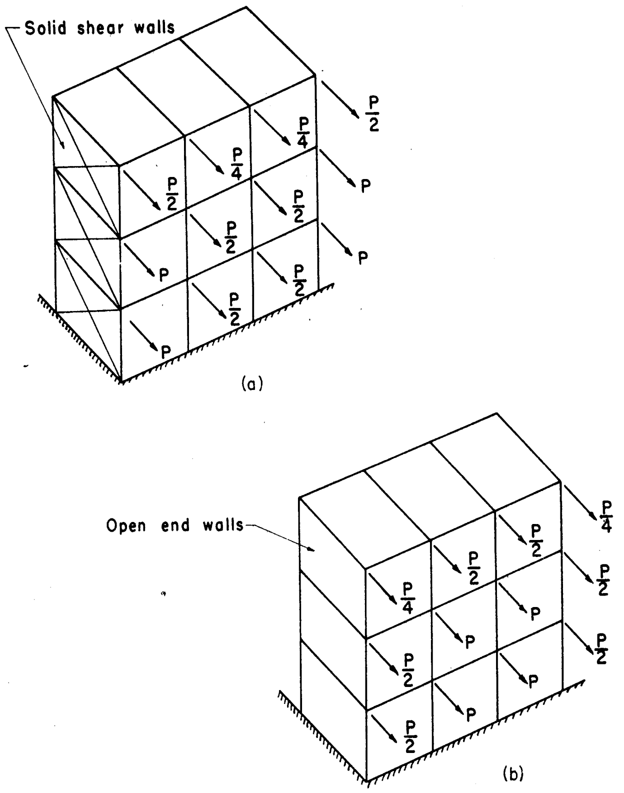


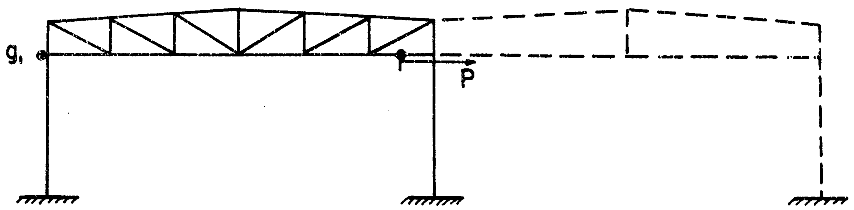
Fig. 2. Tier-building loads.

Equal total test loads are applied to each bent of the single-story industrial-type structure. The number and distribution of the loads to a bent vary according to the type of construction.

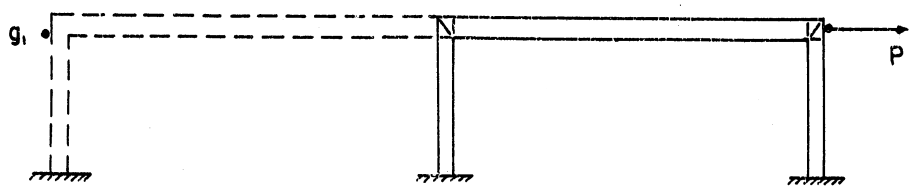
For industrial buildings with trussed roofs, one load is applied to each bent. The load is applied along a line passing through the bottom chord of the truss. The test load is distributed to both trusses for bents 2 bays wide.

Loading for portal-type rigid frame buildings 1 and 2 bays wide is the same as for the trussed roof-type structure, except that the line of action of the applied load passes through the mid depth line of the horizontal roof beam. Full line drawings in Figs. 3a and 3b schematically show the loading situation on one bent of a single-bay portal-type rigid frame and a trussed roof-type frame. The position of the second bay, when the frame is 2 bays wide, relative to the point of load application, is shown with dashed lines in Figs. 3a and 3b.

Several possible types of loading were considered for testing the rigid frame gabled and shed-type structures. Included were a single horizontal load



(a)



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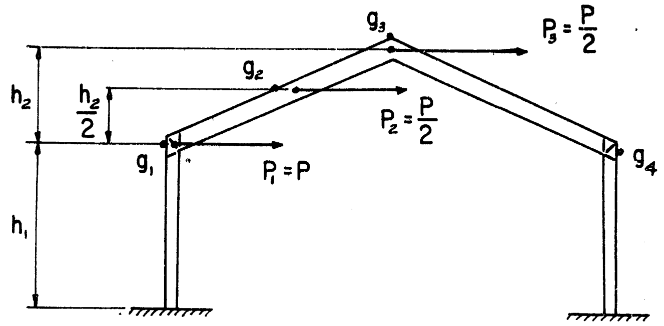
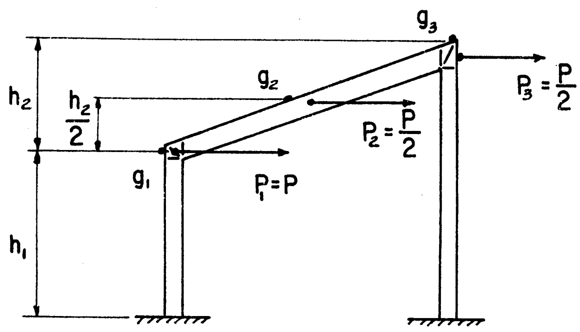


(c)



(d)

Fig. 3. Industrial building loads (deflection measuring stations indicated by letter g).

at either end or mid span of the sloping roof member and various combinations of these loads. It was decided that, to produce a collapse mode which could possibly occur under blast loading, the test loads should be applied as shown in Figs. 3c and 3d. The loads P_1, P_2, and P_3 should probably be varied for structures of different proportions. They are, however, fixed at $P_1 = P$, $P_2 = P_3 = P/2$, since this will utilize proposed test equipment and eliminate the necessity of providing equipment which may be used only sparingly or not at all.

LOADING APPARATUS

Two rather severe requirements are imposed on the loading system. The first requirement is that the system must be such that load application can be controlled from remote positions to maintain the safety of test personnel and equipment.

Secondly, the load system must transfer the loads to the test structure by means of tension members only. This requirement arises as a result of the fact that deflections encountered in the tests will be very large, on the order of from 3 to 6 ft. At deflection of this magnitude, the elevations of the various floor levels will drop from 6 in. to 1 ft below their original positions. Horizontal motion perpendicular to the direction of testing and some twisting also may occur. Under these conditions, extreme difficulty would be encountered in maintaining the alignment of compression members used to transfer loads to the structure. In addition, bending and possibly buckling of the extended portion of the jacking devices may occur, unless jacks of extremely heavy design are used.

In initial studies the possibility of loading a structure internally, with either double or single tension diagonals as shown in Fig. 4, was considered. Both of these systems have the advantage of simplicity and are easy to set up. Loading with double diagonals has the additional advantage of giving the test personnel virtually complete control of the structure at all times so that a test structure can be deformed into any desired configuration.

However, loading with tension diagonals has certain disadvantages which cannot be ignored. The foremost disadvantage is that compressive forces are set up in the exterior columns of the structure, on the side away from which it is deformed. The magnitude of these forces depends on the tension in the diagonal and the angle which the diagonal makes with a horizontal line.* In a wide, low, single-story industrial type of structure, these forces are probably unimportant as far as column buckling, reduction in stiffness of the structures, and ultimate strength of the structure are concerned. However, the same cannot be said for a tier building.

When a tier building is deformed with tension diagonals, the compressive force in the exterior column in any story is the vertical component of the

*Column direct stress due to beam flexure is considered a negligible secondary stress.

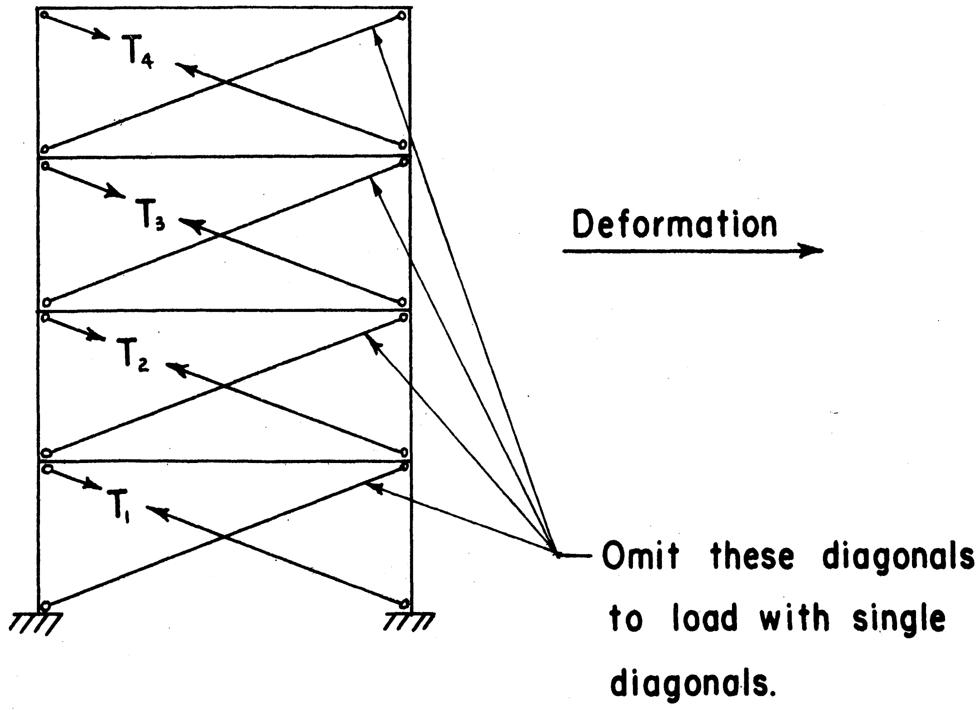


Fig. 4. Loading by internal jacking.

diagonal force in that story plus the direct force in the column above. In the lower stories these forces can become large enough to reduce the flexural stiffness of the columns, to cause premature yielding, and to reduce the plastic hinge moment of these members. Buckling of individual columns and stability of the structure becomes a problem. A pretest analysis would be required to determine what influence these factors might have on the overall behavior of a possible test structure. If the analysis indicates a considerable reduction in the collapse strength, then it would be necessary to eliminate the structure from the test program. In the analysis of the experimentally determined load-deflection characteristics and collapse loads of a structure, it would be difficult, if not impossible, to separate the effects of the horizontal and vertical components of the applied loads. In view of these undesirable characteristics, it is felt that loading with diagonals should not be used in the static tests.

The possibility of jacking against another structure was also considered but was discarded because:

1. A nearby building, suitable for use as a loading frame, may not be available for that purpose.
2. A building close enough to be used as a loading frame may not be oriented properly or may not be strong enough to develop the required forces.

The loading system, as finally adopted, is schematically shown in Fig. 5 as it would be set up for a tier building. The system consists of a number of

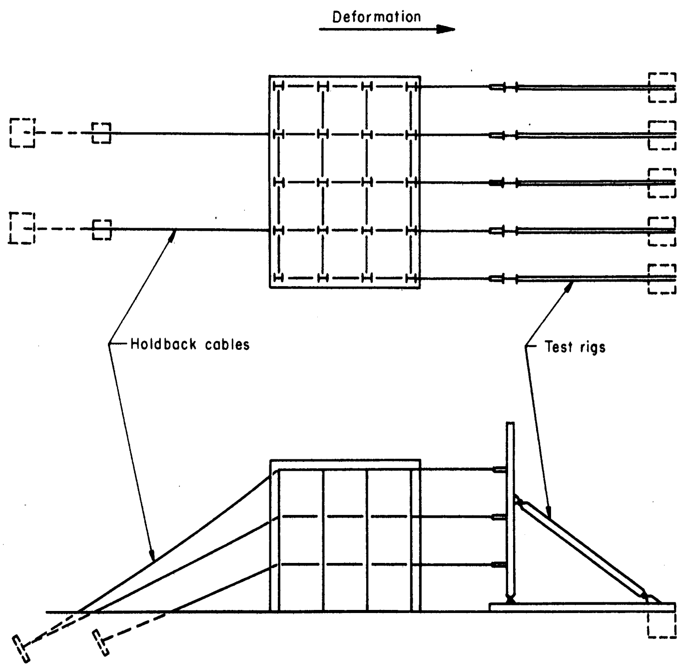


Fig. 5. Load system.

independent test rigs set up to transmit horizontal loads parallel to the planes of the structural bents. Load transmittal is accomplished by means of hydraulic tension jacks and wire-rope cables. The jacks are operated from a remotely placed power supply and control system. The vertical reaction necessary to prevent overturning of a rig is provided by earth ballast. The horizontal reaction necessary to prevent sliding is developed by earth pressure against the side of the earth-ballast pit. Bracing between test rigs, to maintain them in a vertical position and to prevent lateral buckling, is made up of horizontal wire-rope cables at the top and mid height of the vertical mast. Overall stability is maintained by guying the end masts at the top and mid height. Wire rope is used for bracing to provide flexibility in the spacing of the test rigs (bent spacing from one structure to the next may vary from 12 to 26 ft).

The movement of the test structure, during any stage of loading, is restricted by means of wire-rope holdback cables. Each cable is adjusted by means of a chain-fall connected in series with the cable and deadman. One or more cables can be connected to each deadman as shown in Fig. 5.

The holdback system is provided to prevent collapse of the test structure, when collapse is due to the disintegration or failure of key structural members and/or connections. In this case it is very difficult to calculate the forces for which the holdback system should be designed, since their magnitude will depend on the type of failures leading to collapse (e.g., columns buckling, connections tearing, etc.) as well as the dimensions of the structure and its configuration at the onset of collapse. In addition, impact on the cables would have to be considered since this type of collapse may occur suddenly. No method has been developed to evaluate these forces; however, it seems that an adequate system can be obtained as follows.

1. Determine the collapse mechanism and plastic hinge locations for the structure under the loading to be applied in the test.

2. Replace all plastic hinges by momentless hinges and compute the total lateral force required to prevent dead load collapse when the deflection of the roof of the structure (in moving in this mechanism) is equal to the maximum deflection which the loading apparatus can produce. This is readily accomplished by assuming that the total force is distributed to each floor level in proportion to the weight of the floor and applying the principle of virtual work.

3. Design the cables at a floor level to develop twice its proportion of the total required force computed in step 2.

Figure 6 shows the layout of a test rig. The main members of the test rigs were designed for the forces indicated at the various floor levels of the 4-story bent. These loads are approximately twice the maximum resistances the bent would develop, as calculated by a multidegree-of-freedom blast-load response analysis. The main parts of the test rig are the vertical mast, tieback, base beam, and the members used to prevent sliding or overturning of the rig.

The vertical mast consists of two cover-plated wide flange sections which are pin-connected to the base beam and tieback. The mast flanges nearest the tieback have holes at 4-in. intervals beginning at 6 ft above the pin connection and ending 2 ft from the top, permitting jack-height adjustment to accommodate structures of different story heights. Bolt-connected diaphragms are provided between the wide flange sections to prevent flange buckling and to maintain the shape of the mast.

The tieback consists of two wide flange sections laced together and is pin-connected to the base beam and vertical mast. During erection the tieback serves as a stabilizing member for the vertical mast.

The horizontal base beam is made up of two wide flange sections resting on the ground, except directly under the mast where it is supported on timber mats. The base beam ties the vertical mast and tieback into a complete unit, and supports the earth fill ballast.

The earth fill ballast rests on a timber platform which is supported by two wide flange sections. The platform is suspended from the base beam by means of wire rope.

A vertical timber platform backed up by a pair of frames provides the means for developing the horizontal reaction necessary to prevent sliding of the test rig. The frames are made up of wide flange sections and are pin-connected to the base beam.

Figure 7 illustrates how the hydraulic jacks are mounted to the test rigs. The eye and U bolt are used to maintain axial loading in the jacks. The short 14WF sections are welded to the bearing plate and bolted to the mast. Shoulders

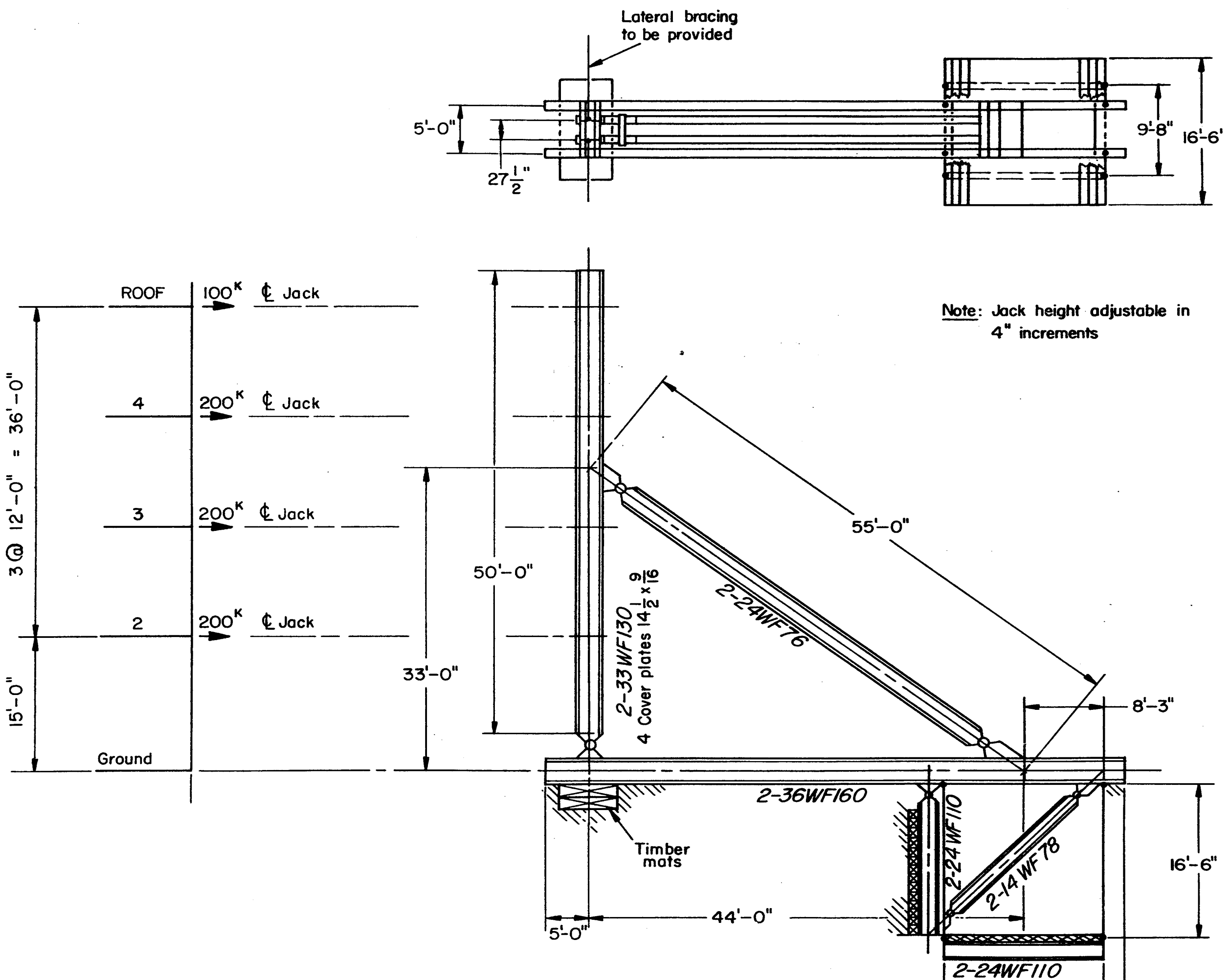


Fig. 6. Test-rig design.

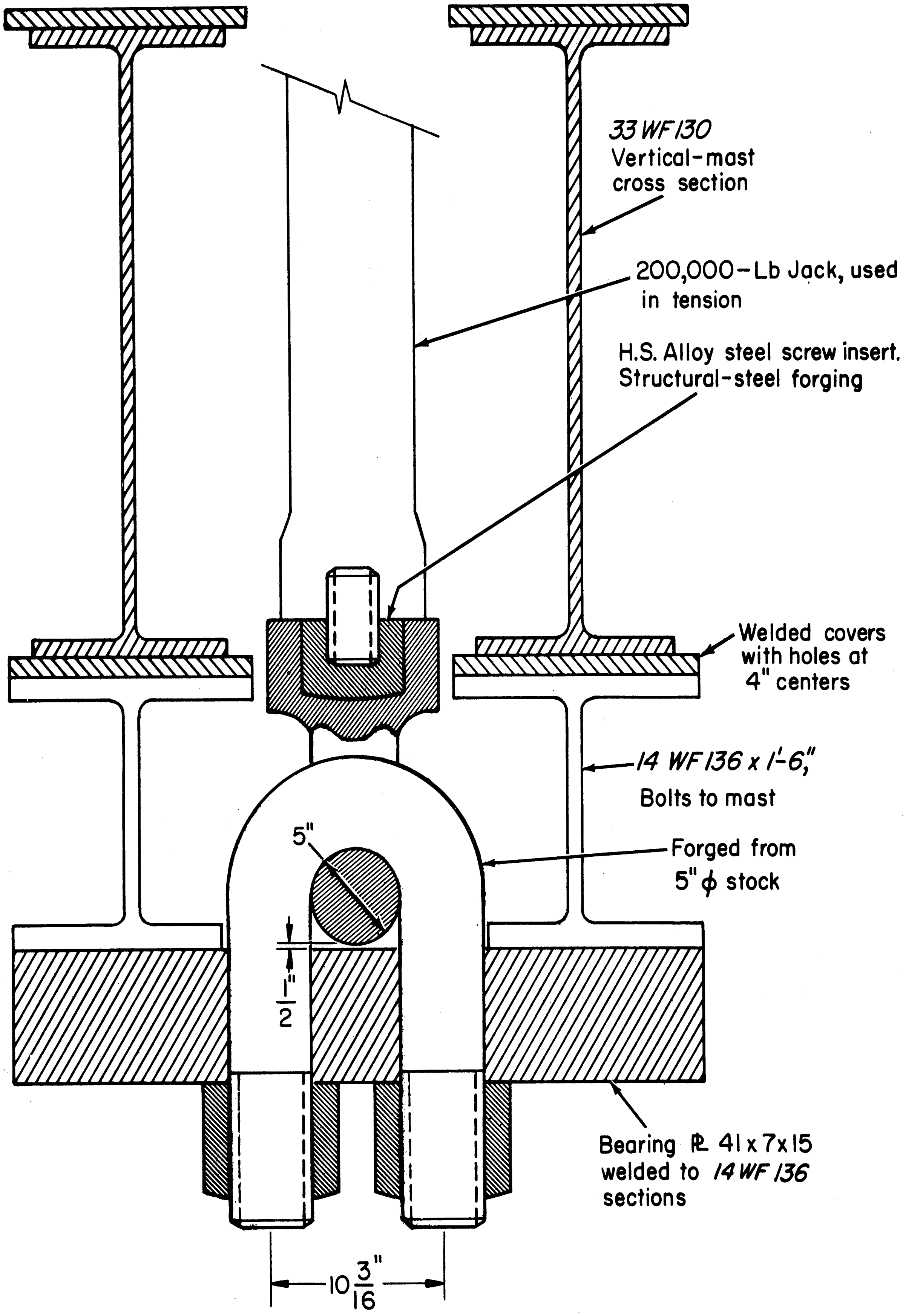


Fig. 7. Hydraulic jack mounting.

are provided on the U bolt to maintain clearance for the eye and alignment of the U bolt.

Load cells are connected in series with the hydraulic jacks as shown in Fig. 8. The ball and socket-type linkage is used to maintain axial load in the load cells during tests. These couplings and the U and eye connection provide the freedom of motion required during installation to allow for misalignment of the vertical mast.

Connection of the pulling cables to a test structure can be accomplished in a number of ways. The particular method to be used will depend on the type of structure being tested and construction details. For portal-type rigid frame structures, a pulling eye welded to a column is adequate. In testing tier buildings, a setup as shown in Fig. 9 can be used.

HYDRAULIC LOADING SYSTEM

No specific design of a hydraulic load and control system has been made since this is considerably beyond the scope of this report. Requirements have, however, been set up for the design of the hydraulic jacks and a power and control system. In setting up the requirements, primary consideration was given to obtaining a simple and flexible system. The specifications and reasons for some of them are given below.

1. Jacks.—

(a) Twenty-eight hydraulic jacks will be required in all. Maximum load in each jack is to be obtained on the basis of 10,000 psi of fluid pressure. Each jack is to have a stroke of 6 ft-0 in.* Jacks are to be used for horizontal tension loading only. The number and maximum force capacities of the various jacks are to be as follows:

12 jacks	at	200 kips	(effective area = 20.00 sq in.)
11 jacks	at	100 kips	(effective area = 10.00 sq in.)
2 jacks	at	50 kips	(effective area = 5.00 sq in.)

The worst case of loading with respect to the number of jacks required would occur in testing a 5-story structure 4 bays long. In this case loads would be applied simultaneously at 25 different locations. Twenty-five jacks of the maximum capacity stated will take care of this loading situation and will certainly be sufficient for all other loading conditions.

(b) The frictional force in each jack to be not greater than 2% of the maximum design force capacity. Variation of friction between jacks of identical maximum capacity is not to exceed 1%.

*A manufacturer, already experienced in heavy-jack production, has stated his ability and willingness to manufacture tension jacks with a 6-ft stroke and a 200,000-lb force capacity.

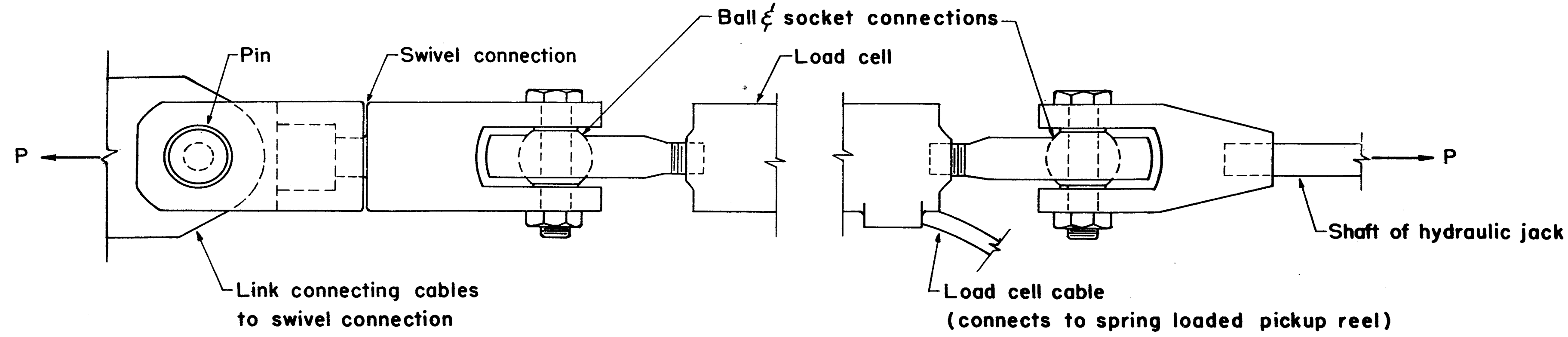


Fig. 8. Load-cell installation.

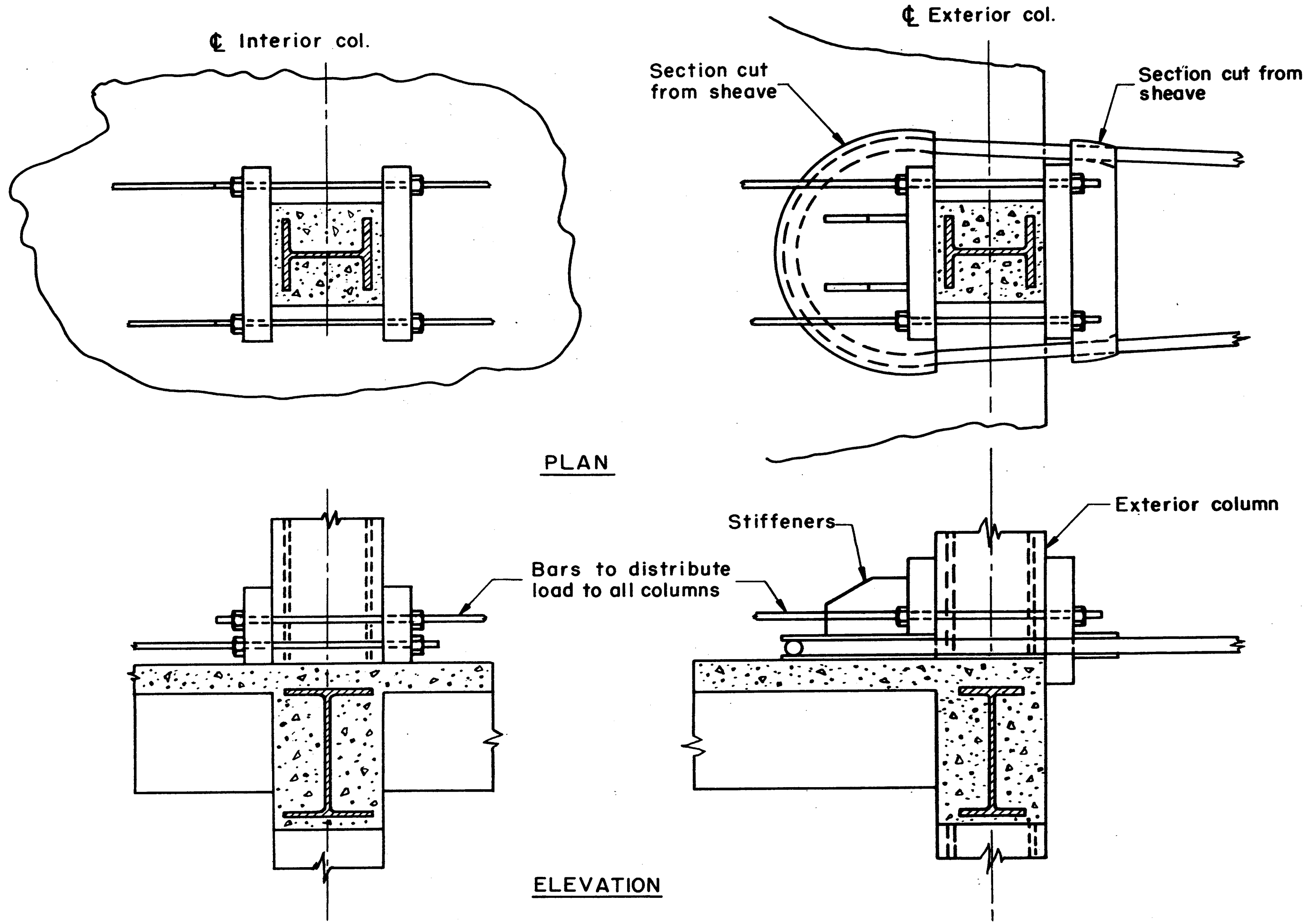


Fig. 9. Cable installation.

(c) Jacks to be designed to allow for the following conditions:

(1) Jack simply supported in a horizontal position with ram fully extended and no pressure in the hydraulic system.
(2) Jack supported at ends of cylinder with ram fully extended.

These requirements are included to decrease tendencies for the jacks to bind in initial stages of the test.

(d) Jack design is to be such that identical fittings are required to connect hydraulic lines and to mount the jacks.

(e) Each jack to be provided with plugs and caps, as required, to prevent damage to exposed threaded parts.

2. Power Unit and Oil Reservoir.—A single power unit is to be provided. This unit is to be capable of delivering 2 gpm at a maximum operating pressure of 10,000 psi. The oil reservoir is to have a capacity of 135 gal and is to be supplied with an easily cleaned or disposable oil filter. Reservoir capacity is 120% of the volume of oil required to operate the entire system of jacks over a stroke length of 6 ft-0 in.

3. Control System.—The control system is to do the following:

(a) Be such that from 2 to 25 of the hydraulic jacks, in any combination, can be loaded or unloaded simultaneously.

(b) Allow for increasing fluid pressure from 0 to 10,000 psi in a stepwise manner.

(c) Automatically control flow, under increasing or decreasing pressure, so that the following rates of flow are not exceeded:

1. For each 200-kip jack, flow not to exceed 20 in.^{3}/min
2. For each 100-kip jack, flow not to exceed 10 in.^{3}/min
3. For each 50-kip jack, flow not to exceed 5 in.^{3}/min

(d) Provide for flow control from 0 to full flow (0 to 2 gpm) by means of a hand wheel.

These flow-control requirements are included as a safety measure. The above flow rates permit a maximum possible displacement rate of 1 in./min.

(e) Contain an emergency shutdown feature which will permit partial draining of all jacks to reservoir by releasing stored up pressure.

(f) Contain a large, easy-to-read accurate pressure gage.

(g) Contain a provision to drain all jacks to the reservoir by means of a power unit.

The control system should be as compact as possible.

4. Fluid Supply Lines.—Flexible high-pressure (10,000 psi) fluid supply lines are to be provided for simultaneous operations of 25 jacks as follows:

5 jacks at 150 ft from power and control units
5 jacks at 125 ft from power and control units
10 jacks at 100 ft from power and control units
5 jacks at 75 ft from power and control units

5. General Design Features of the Entire System.—The system as a whole, i.e., jacks and power and control units are to be designed to operate under extremely dirty and dusty conditions. Required auxiliary sleeves, grease, etc., are to be specified as required. Since the system is to be used in field tests, it should be of rugged construction to minimize possible damage during transport and erection. Suitable shock mountings are to be provided for delicate parts while in transit.

INSTRUMENTATION AND RECORDING

Instrumentation for the pulldown and story shear tests is restricted to providing for deflection and load measurement only. Consideration was given to the possibility of providing strain-gage instrumentation. Generally speaking, the interpretation of such data in a test of this kind would be complex and of uncertain value. Strain gaging is not recommended but may be added at the option of the test engineer if special circumstances warrant it.

It is planned that all individual loads and deflections will be read and recorded manually. Manual recording was chosen because these tests are conducted under static conditions and ample time should be available to read, check, and accurately record all measurements. Automatic electrical means of measuring and recording the loads and deflections was considered and dropped, since particular advantages would be realized only if the test were carried to complete collapse. It is not considered practical to carry the test to complete collapse because few if any additional meaningful data would be obtained and the cost of protecting equipment and personnel would be greatly increased.

Tier-building deflections are measured at all floor levels and the roof. One deflection-measuring station is provided at the centerline of each bay running perpendicular to the direction of testing, at all floor levels. Deflections are measured at the spandrel beams on the side of the structure furthest from the loading apparatus, at approximately half the thickness of the floor slab below the top of the floor.

Deflections of all bents are measured for each type of industrial structure. One gaging station is provided for all but the rigid frame gabled bent and shed-type structures. For structures with trussed roofs, the deflection of a bent is measured at the outside flange of the exterior column furthest from the loading apparatus, at the approximate elevation of the line of intersection of the column with the top chord. For rigid frame portal frame structures, the deflection of a bent is measured at the outside flange of the exterior column furthest from the loading apparatus, at the elevation of the mid depth line of the roof beam. The points at which deflections are to be measured for the gabled bent and shed-type structures are shown in Figs. 3c and 3d.

DEFLECTION GAGE

The range of the device to be used to measure deflections was fixed at 6 ft-0 in., which is the length of stroke of the hydraulic jacks. Since deflections will be large, even in the elastic range, it was decided that a minimum instrument reading of 0.01 in. would provide sufficient sensitivity. In many cases a sensitivity coarser than 0.01 in. would be admissible; however, a sensitivity of 0.01 in. may often be not only desirable but necessary. As a case in point, the reader is referred to Ref. 5, p. 307, under the section heading "Horizontal Loading Tests on Three-Story Frameworks." In addition to the range sensitivity, and the accuracy requirements, it is desirable that the instrument be direct-reading. Figure 10 schematically shows the setup for a simple displacement gage meeting these requirements. This gage is essentially a leveling rod used in a horizontal position. The gage is maintained in a horizontal position by means of an adjustable leveling bar. Its position in the vertical plane can easily be checked with the transit used to set the target. Alignment of the rod in the horizontal plane is easily maintained with a plumb bob suspended from a bar clamped to the scaffolding as shown in Fig. 10.

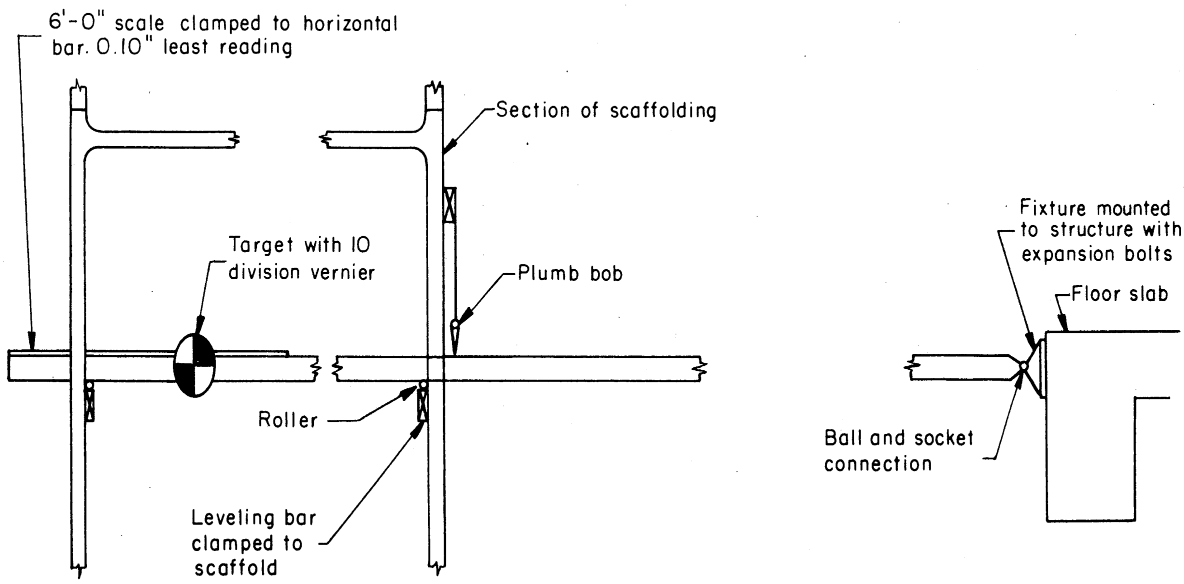


Fig. 10. Deflection-gage setup.

Alignment of the deflection gage in the horizontal plane with only a plumb bob should give deflection measurements that are in error, as a result of alignment only, by no more than the minimum reading of the gage. Table I contains a listing of errors due to alignment only, when the gage is set up as shown in Fig. 10, and assuming that motion of the structure perpendicular to the direction of testing can be ignored.

TABLE I

ALIGNMENT ERRORS

E, in.	e, in.
± 0.75	+ 0.0106
± 0.50	+ 0.0058
± 0.25	+ 0.0014

E = error in alignment in horizontal plane.
e = error in deflection reading when plumb bob is 3 ft-0 in. from center of ball joint and target 4 ft-0 in. from center of ball joint.

The target of the gage is set from one of two transit stations. Two transit stations are set up on a base line running parallel to the long direction of the structure, approximately 4 ft from the side of the structure furthest from the loading apparatus, and approximately 40 ft beyond the two exterior bents running parallel to the direction of testing. Each transit is supported on a circular wiremesh reinforced concrete slab resting on the ground. The slabs are 4 in. thick and 3 ft-6 in. in diameter. Inserts are provided in the slabs to receive the tripod legs. Each slab also contains an insert marking the center of the slab. As a precautionary measure, the position of each transit is referenced with respect to 4 points (2 for each transit) marked on nearby permanent structures. In addition, each transit is to be protected by means of a temporary portable shelter.

CREEP DETECTION

In the inelastic range of deformation, a certain amount of creep can be expected before the structure settles in a stable configuration. The rate at which creep occurs will be relatively high immediately after an increment of load is applied. Deflections should be measured only after the rate of creep has decreased to a negligible amount. Creep in the static test is detected by means of a linear variable differential transformer and recorded on a direct-writing type of recorder. Instrumentation for creep detection is provided at two stations at the roof level of the structure. The setup is schematically shown in

Fig. 11. The device is set up to detect only the creep which occurs after each increment of load is applied. Thus the clamping screw C must be loose during load application.

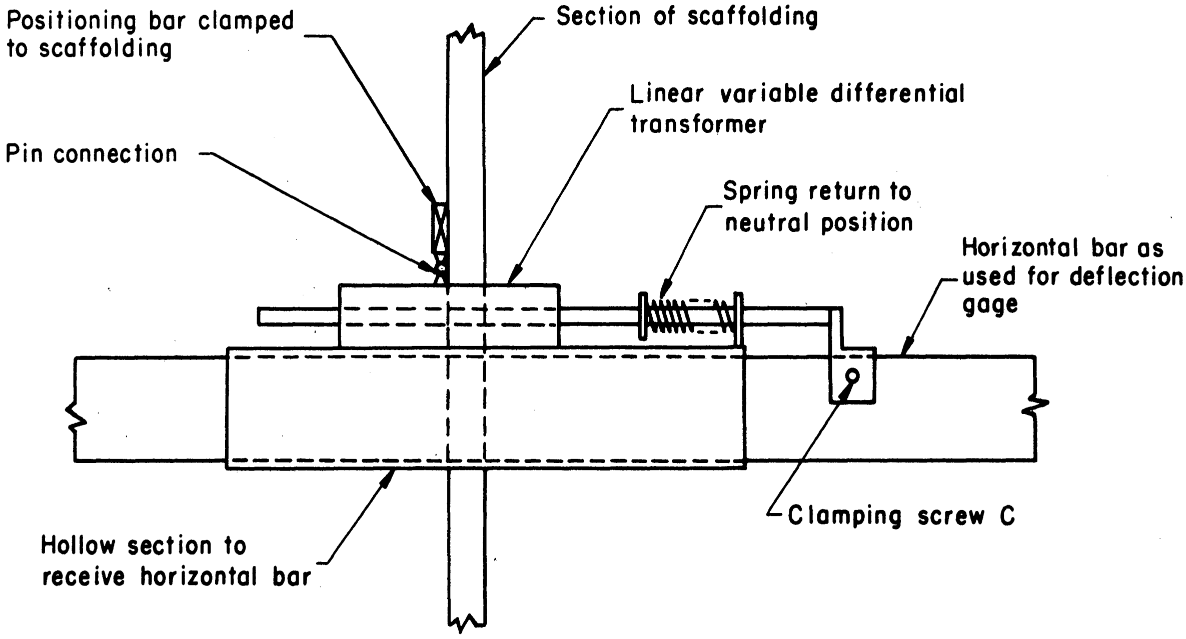


Fig. 11. Creep-detection pickup.

LOAD MEASUREMENT

Measurement of individual loads is accomplished by means of load cells connected in series with the hydraulic jacks. Output from the cells is fed into a null-balance indicator system calibrated in kips of force. The load cells to be used are special but similar to the Baldwin-Lima-Hamilton type U-1 load cells, suitable for tension and compression loading. Twelve load cells of 200,000-lb capacity, eleven cells of 100,000-lb capacity, and two cells of 50,000-lb capacity are required. Load cells are installed during the process of setting up the test rigs and hydraulic jacking system.

The indicator system is to be built in as part of the control system for the hydraulic jacks. The indicator system contains:

1. A 3-stage dial to indicate individual loads in ranges from 0 to 50, 50 to 100, and 100 to 200 kips of force;
2. A provision for picking up the output from 25 load cells;
3. A selector switch and lights to indicate which particular load is being measured; and
4. A circuit to sum the output of all active load cells, and a 4-stage dial gage to indicate the total force. This gage is to be active during load and read cycles. Dial stages are to vary in steps of 1000 kips so that a total maximum load of 4000 kips can be recorded.

TEST PERSONNEL

To run the pulldown tests efficiently, it will be necessary to assign specific jobs to the various members of the testing party. Assignments are made by the chief testing engineer who will determine and signal when loading will start and stop, and when recording is to begin (a public address system or a buzzer signal system is used for this purpose). The total number of men required during a test will depend on the quantity of data to be recorded and the possibility of having one man do two or more jobs. The testing of a small industrial structure can possibly be handled by as few as four men. On the other hand, testing a tier building may require a dozen or more men. Below is a title listing and the work which is to be done by the test personnel.

Chief Testing Engineer.—Directs and coordinates testing and operates loading equipment. In a large test, loading equipment may be operated by an assistant to the chief testing engineer.

Recorders.—Read and record deflections. One recorder may be able to handle as many as 4 deflection gages depending on their location. Recorders also act as observers at these stations.

Transit Men.—Align targets on deflection gages and may possibly be able to serve as observers.

Observers.—Observe and photograph any significant changes in the condition of walls, connections, etc. Personnel with the exclusive task of observation would be required only in testing the largest of tier buildings.

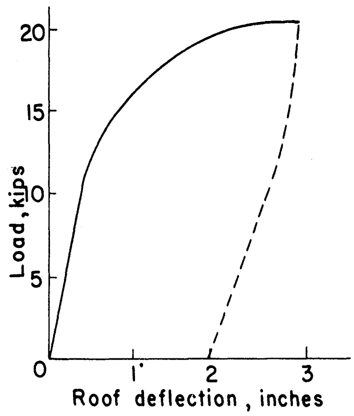
Computer.—A continuous plot of the total load versus average roof deflection is used as an aid to determine the increment in load in going from one load step to the next. Computation and plotting of this curve is done by the computer who also reads and records all loads. In a small test this function can be handled by the chief testing engineer.

PULLDOWN TEST PROCEDURES

The pulldown test is conducted in three phases: (1) initializing, (2) loading, and (3) unloading.

Initializing.—The purpose of initializing is to take up the slack in the loading system and to zero in instrumentation. This is accomplished by applying an initial load of 1/40 of the predicted yield load. The output signal of each load cell is attenuated to indicate a load reading of zero kips. Deflection readings are taken and where necessary the gages are reset to give zero readings.

Loading.—The loading phase is carried out in a stepwise manner consisting of a number of load-read cycles. Successive load-read cycles are executed until the collapse loads of the structure have been established. Each increment of load may be approximately 10% of the estimated yield load of the structure. After yielding commences, it may be preferable to change from load increments to deflection increments, especially in the range where the rate of load increase with deflection is diminishing toward zero. During this phase of the test, a continuous plot of the total load versus average roof deflection is kept. The purpose in plotting this curve is two-fold. First, the curve will give a good indication of the approximate deflection at which the maximum loads will occur. Secondly, by using the points already established, the curve can be projected to determine an increment of load which will produce a sufficiently small increment in deflection, which in turn will insure a well-defined load-deflection curve. The usefulness of such a curve can be illustrated by referring to Fig. 12. Figure 12 is a reproduction of an actual load-deflection curve obtained in testing one bent of a 3-story reinforced concrete building.[5] From this curve, it can be seen that each point along the curved portion, up to the maximum load, could have been fairly well predicted by simple forward projection with a French curve.



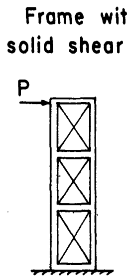


Fig. 12. Load-deflection curve (3-story reinforced concrete bent).

The load cycle begins after all deflections and loads have been measured and recorded in a previous read cycle. The steps followed in a load cycle are:

1. Announcement that a load cycle is about to begin. This announcement is given by the chief testing engineer and is a signal to other test personnel to prepare for reading and recording a new set of deflections.

2. When all preparations have been made, loading is initiated by hydraulic control.

3. After the load increment has been applied the creep-detecting devices are clamped to the leveling bars and their output recorded as a function of time. During the period of time required for the structure to settle in a stable deflection pattern, observers note significant changes in the condition of structural members and report any changes which may require modification or immediate stopping of the test.

4. When the rate of creep falls below 0.005 in./min,* the read announcement is given. The read cycle begins with an announcement of the total load followed by an instruction to begin reading. At this time the individual loads and total load are read and recorded. Deflection readings are taken and recorded. Roof deflections are measured first and reported to the computer, who plots their average against the total load. After the final deflection has been recorded, the creep detecting instruments are released and a new load cycle started.

*May be varied depending on rigidity of particular structure.

Unloading.--After the collapse loads have been established, the unloading phase begins. The procedures used are the same as in the loading phase, i.e., unloading is carried out in steps, and an unloading load-deflection curve obtained. Unloading increments may be taken as large as 20% of the total applied load at the time unloading begins.

STORY SHEAR TEST

The objective of the story shear test is to determine experimentally the load-deflection characteristics in each story of the test structure, up to the early range of plastic deformation. These tests provide an experimental determination of the lateral stiffness matrix for the test structure.

The stiffness matrix can be found by either a direct or an indirect process. The direct method for experimental determination of the stiffness matrix corresponds to that used in the analytical procedure. In the analytical procedure the structure is first given a "unit" lateral deflection at one floor level, the deflections at all other floor levels being held to zero deflection. Then the forces necessary to maintain the structure in this configuration are calculated, thereby determining one column of the stiffness matrix.[9] The process is repeated until the unit deflection has been applied at all floor levels, and the complete stiffness matrix calculated.

In the experimental method the "unit" deflection is taken to be the maximum deflection which can be induced at a floor level, within the elastic range of deformation, while the deflections at all other floor levels are held to zero deflection. The unit deflection at a floor level is induced in increments and the corresponding forces at all floor levels are recorded. If the unit deflection is applied at the jth floor level, then the stiffness coefficient at the ith floor, for this case, is the slope of the elastic portion of the load-deflection curve obtained by plotting the force at the ith floor against the deflection at the jth floor.

The indirect method is substantially the inverse of the direct method in that the flexibility matrix is determined and the stiffness matrix calculated by matrix inversion. In the analytical method for determining the flexibility matrix, a unit load is applied at one floor level with no load at all other floors. The deflections of all floor levels are calculated, thereby determining one column of the flexibility matrix. The process is repeated for all floor levels until the complete flexibility matrix is determined. In the experimental method the unit load for any floor level is the maximum force which can be applied at that floor level without exceeding the elastic range of deformation. The unit load is applied in increments and the deflection of each floor level is recorded. If the unit force is applied at the jth floor, then the flexibility coefficient at the ith floor is the slope of the elastic portion of the deflection-load curve obtained from the plot of the deflection at the jth floor against the force at the ith floor.

The story shear tests are to be performed so that the flexibility matrix rather than the stiffness matrix is obtained. This method is chosen for its three main advantages. First, less work is involved in determination of the flexibility matrix since control only over the applied loads is necessary. Secondly, this procedure requires the same instrumentation and loading apparatus, in the same setup, as used in the pulldown test. Finally, large deflections can be produced in the elastic range by this method so that a small error in deflection measurement is not as critical as it might be in the experimental determination of the stiffness matrix.

Two story shear tests are to be run for each structure, one before and one after the pulldown test has been completed. Data from the second test are used to compute the change in stiffness as a result of inelastic deformation produced in the pulldown test.

The test loads for the story shear tests, for any one structure, are set up exactly as for the pulldown test for the same structure. For example, in the story shear test of a tier building the jacks are arranged so that if loading took place at all jacks at the same time, then the total load at each floor level would be P and at the roof level P/2. The same holds true for the gabled bent and shed-type rigid frame structures. For these two types of structures, if all loads were applied at once, the load at the eave would be P, and at the peak and mid span of the sloping roof member, P/2. The purpose of setting up for loading at all locations is to avoid the necessity of interrupting the story shear test to remove or install loading equipment. Also when the first story shear test is complete, the loading system for conducting the pulldown tests is automatically set up.

STORY-SHEAR-TEST PROCEDURE

In the experimental work the data necessary to calculate the various columns of the flexibility matrix are to be obtained by working from the top of the structure down. For a tier building the first set of data is obtained by loading at the roof only. This is accomplished by blocking out all jacks except those at the roof level. The process of loading is much the same as in the pulldown tests in that it consists of a number of load-read cycles. Loads may be increased, in successive load cycles by no more than 10% of the estimated yield load of the structure. However, the total load is increased from one cycle to the next until a plot of the applied load versus roof deflection shows a definite break away from elastic behavior. Deflections are measured at all floor levels for each increment of load. Load and read cycles are to be carried out in exactly the same manner as in the case of the pulldown test.

The second and subsequent columns of the flexibility matrix are obtained by loading at the first, second, etc., floor levels below the roof level.

For industrial structures of the trussed roof or portal rigid frame type (or any single-degree-of-freedom test structure) the flexibility and stiffness constants are automatically obtained from the pulldown-test data.

IV. TEST SEQUENCE

The sequence in which the various tests are to be performed is contained in Table II. This sequence was set up primarily to avoid the necessity of setting up the static test equipment more than one time; however, this sequence of testing also provides all the data necessary to study the effect of large inelastic deformation on the static and dynamic properties of the structures.

TABLE II

TEST SEQUENCE

Test No.	Test Type	Identifying* Symbol	Comments
1	Vibration	V1.1 V1.2	Test is to be completed before static loading equipment is connected to structure.
2	Story shear	S1	Shaker removed from structure before test begins.
3	Pulldown	P1	
4	Story shear	S2	
5	Vibration	V2.1 V2.2	Test begins after static load equipment has been disconnected.

*These symbols are to be used to identify test data.

V. POST-TEST ANALYSIS OF DATA

The post-test analysis is accomplished in two phases. The first phase consists of reducing the test data to a numerical form, plotting curves, and making drawings to show the areas in which severe inelastic deformation or fracture of structural members took place. Curves to be plotted are:

1. Resonance curves from the vibration test data.

2. Total applied load versus average floor deflection curves, for all floor levels and the roof of the structure tested, for the pulldown test, and for each case of loading in the story shear tests.

This work is to be carried out by members of the testing parties and should be accomplished as soon as possible after each test has been completed. Thus any peculiarities or errors in test data can be accounted for while knowledge of the test is still fresh in mind.

Phase two consists of analyzing the structure and correlating theoretical calculations with test results. Since each structure is a special case, it is not realistic to specify methods to be used or assumptions to be made. Detailed analyses, however, should be carried out under a number of assumptions compatible with test results and the dimensions of the primary structural members; for example, buildings in the 3- to 5-story range can be treated as shear buildings as well as buildings with flexible girders, if only minor or no inelastic deformation was produced in the girders. The effect of dead load and concrete encasement of structural members and connections should be taken into account. Material constants for use in the analyses should be taken from the results of the physical tests.

VI. MAXIMUM AND MINIMUM TEST PROGRAMS

The maximum and minimum test programs are set up with respect to the number of structures to be tested rather than with respect to the quantity of information to be obtained from any one test structure. This was done primarily because the tests are to be carried out on full-scale structures which had actually been in service and were not specifically designed as test specimens. These structures, even with a maximum amount of modification and repair where necessary, will be far from laboratory-type specimens. As a result they are not suitable for obtaining highly detailed data in the form of stress, strain, joint rotation, etc.

Instrumentation, loading apparatus, test procedures, etc., are the same in both the maximum and minimum test programs. Modifications necessary to prepare the structures are also the same in both programs.

In the maximum test program no limit is set on the number of structures to be tested. The objective of this program is to test all structures which can feasibly be put in condition for testing. Test results would be continually accumulated and classified according to the type of structure tested. The results would be used to form part of a statistical basis for predicting the static and dynamic characteristics of a structure from its significant dimensions. The maximum test program would automatically include the minimum desirable test program.

In the minimum desirable test program only one or two buildings of each type out of each class of possible test structures would be tested. The objective of this program is to obtain data which can be compared with analytical calculations based on various assumptions of the structural behavior of the main frame and contiguous elements. The purpose of the comparison, of course, is to investigate the validity of the assumptions. Toward this end possible test structures would be carefully inspected and only the most ideal selected for testing. Each test structure in this program should be such that, with a minimum amount of modification, it can be represented reasonably well by a mathematical model which can be solved after evaluation of the material and frame constants that relate actual to analytical behavior.

APPENDICES

APPENDIX A

SPECIFICATIONS COVERING THE SELECTION, INSPECTION, AND MODIFICATION OF TEST STRUCTURES

Introduction

To our knowledge, no large-scale testing of actual structures of the type under consideration has been made. As a result, there is a lack of knowledge relating to the difficulties which can be encountered in carrying out the tests. The success or failure of these tests will, to some extent, depend on the complexity of the structure. Generally it may be expected that the more simple the structure, the better the chances are for successful completion of the tests. Post-test analysis of data and correlation and interpretation of differences with theoretical calculations will be a less difficult task if testing is confined to structures of simple design. The specifications which follow are related to the selection and modification of test structures. They are written in an attempt to insure the testing of only the simplest structures, in a condition as near as possible to their condition when in service.

Part I: Definitions

SECTION 1. TEST STRUCTURE—DIMENSIONS AND NOMENCLATURE

The general dimensions of test structures and the terms as used in the specifications are given in Fig. A-1.

SECTION 2. ACCEPTABLE TEST STRUCTURE

Any industrial structure or tier building substantially meeting the requirements of Sections 3 and 4 of Part I, or which can be feasibly modified to meet substantially the requirements of Sections 3 and 4 of Part I, is herein defined as an acceptable test structure.

SECTION 3. INDUSTRIAL STRUCTURES

(a) Industrial Structures.—Industrial structures shall be of riveted or welded steel construction, of the trussed roof or rigid frame type, or of

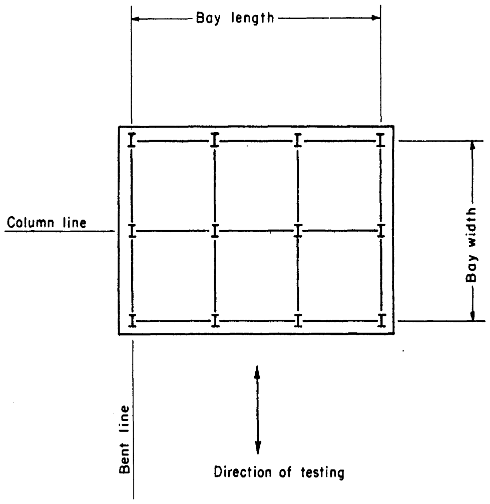


Fig. A-1. Test-structure dimensions.

reinforced concrete construction. They shall be at most 2 bays wide and from 1 to 4 bays long. In addition, they shall be structurally symmetric about a central vertical plane parallel to the direction of testing.

(b) Trussed Roofs.—Riveted or welded trussed roofs, of all types, with a straight horizontal bottom chord, either continuous or simply supported, are permitted.

(c) Rigid Frame Structures.—Rigid frame industrial structures shall be of the portal, shed, or symmetrical gable type only. Shed or gabled bent structures more than 1 bay wide are not permitted.

SECTION 4. TIER BUILDINGS

(a) Tier Buildings.—Tier buildings shall be of welded or riveted steel, or reinforced concrete construction. They shall be from 3 to 5 stories high, at most 4 bays long and from 1 to 3 bays wide. They shall have straight and mutually perpendicular column and bent lines and shall be symmetrical with respect to a central vertical plane, parallel to the direction of testing. All columns shall run in a continuous straight line from ground to roof level.

(b) Concrete Construction.—Only reinforced beam and girder type concrete construction is permitted.

SECTION 5. STRUCTURAL AND NONSTRUCTURAL ELEMENTS

(a) Structural and Nonstructural Elements.—Any structural member, beams, columns, girders, walls, etc., which are expected to contribute significant resistance to lateral deformation of a structure, or the absence of which will cause hazardous conditions to exist at any time during testing, are herein defined as structural elements. All other members and parts of a structure are defined as nonstructural elements.

Part II: Inspection and Modification of Test Structures

SECTION 1. GENERAL

(a) Scope of Work.—The testing contractor shall make a preliminary visual inspection of each possible test structure as soon as possible after he has knowledge that the structure will be available for testing. He shall determine if the structure is an acceptable test structure according to the provisions of Sections 3 and 4 of Part I. He shall obtain plans of the structure, and from the inspection note thereon what modifications are necessary to bring the structure into substantial accordance with the provisions of Sections 3 and 4 of Part I, and to insure the safety of personnel and equipment during all phases of the testing program. He shall make such plans as are required to modify the structure, and inspect the work of modification.

SECTION 2. INSPECTION

(a) Structural Bents.—Each bent of a possible test structure shall be inspected to ascertain its soundness and worth as part of a test structure. The inspection shall include all trusses, beams, columns, connections, bracing, and other structural members.

(b) Members Connecting Structural Bents.—All members, such as struts, cross bracing, purlins, girts and their connections, tying the bents together so as to cause the bents to interact, shall be examined to determine what modifications or repairs, if any, are necessary. Crane girders shall also be included in this group.

(c) Connections Encased in Concrete.—Riveted and/or welded steel beam-to-column connections encased in uncracked or mildly cracked concrete shall be assumed to be sound. Where large cracks are present, the testing contractor shall remove the concrete and inspect the connection.

(d) Modifications by Previous Owners.—Structural modifications in the form of repairs, removals, and/or additions, which had been made by former owners of a test structure, shall be inspected to determine if any further modifications are required.

(e) Structural Walls.—Exterior reinforced concrete and masonry walls which will act as shear walls during testing shall be examined for large cracks. Solid masonry partitions shall be considered as shear walls.

(f) Nonstructural Elements.—Nonstructural elements, such as suspended ceilings and interior masonry work running perpendicular to the direction of testing, which may constitute a hazard during tests, shall be examined to determine their disposition.

(g) Miscellaneous.—A survey shall be made to determine the disposition of all machinery, plumbing, and electrical work not removed by former building occupants.

SECTION 3. MODIFICATIONS

(a) Plans, Drawings, and Inspection.—Having determined what repairs, removals, additions, or reinforcements are necessary to set the structure up for testing, the testing contractor shall proceed to make all necessary plans, drawings, and work schedules to execute the work. The plans and drawings are to include such additional modifications as may be necessary to set up instrumentation and loading apparatus. The testing contractor shall inspect all work of modification. He shall also control, inspect, and be responsible for all safety measures taken in the work of modification.

All repairs and modifications shall be designed according to the following specifications listed in order of preferred use:

1. Codes and Specifications under which the structure was designed, such as a City Building Code.
2. The AISC and/or ACI codes in effect at the time the structure was originally designed.
3. Current AISC and/or ACI code.

(b) Industrial Structures.—Industrial structures more than 4 bays long may be cut into two or more substructures, one of which will serve as the test structure. Modifications of this magnitude shall be made at expansion joints, where possible. Care shall be taken to insure complete severance. Sufficient clearance to prevent any interference as a result of twisting during the tests shall be provided between the substructure to be tested and all other parts of the parent structure.

(c) Defective Structural Members—Steel Construction.—Defective structural members shall be repaired, reinforced, or replaced according to the following schedule:

(1) Columns.—In tier buildings columns may be repaired or reinforced only. Columns may be replaced in industrial buildings with trussed roofs only when feasible and absolutely necessary.

(2) Beams and Girders.—Beams and girders expected to be under primary stress during testing shall be repaired or reinforced only. Defective beams and girders expected to be under secondary stress shall be supported or braced in a suitable manner.

(3) Connections.—Beam-to-column and beam-to-girder connections may be repaired, reinforced, or replaced, whichever is most economical. High-strength steel bolts shall be used in repairing riveted connections. Welded connections shall be rewelded as required. Concrete fireproofing shall be replaced at any repaired connection.

(4) Bracing Between Structural Bents.—Defective bracing between bents shall be repaired, reinforced, or replaced as required. Bracing shall be added as required.

(d) Defective Reinforced Concrete.—Defective reinforced concrete members shall be reinforced or braced only.

(e) Defective Shear Walls.—Solid reinforced concrete or masonry may be removed or repaired, whichever procedure is estimated to be the least expensive.

(f) Exterior Walls Running Perpendicular to the Direction of Testing.—All exterior masonry walls running perpendicular to the direction of testing, except walls at the ground-floor level, shall be removed. All reinforced concrete walls, and masonry walls at the ground-floor level and not more than 4 ft high, shall be left in place.

(g) Access Holes.—Access holes, necessary to overcome difficulties in setting up test apparatus, are permitted in steel members only. Reinforcement shall be provided around all access holes and shall be designed to develop the full plastic moment and shear stress, or direct stress, as the case of the member may be. Access holes are permitted in either reinforced concrete or masonry shear walls.

(h) Points of Load Application.—Stiffeners and/or load-distributing devices shall be used at all points of load application to prevent local buckling or crushing of rolled or built-up structural steel members. Load-distributing devices shall be used to prevent crushing of concrete at all points of load application in reinforced concrete structures.

(j) Miscellaneous and Nonstructural Elements.—All miscellaneous and nonstructural elements or objects, the presence of which may constitute a hazard during any phase of testing, shall be braced or removed from the structure. All glass work shall be removed from the test structure.

APPENDIX B

COST ESTIMATES

Complete cost estimates for the test program could not be obtained. Those estimates which were obtained pertain to the major items of required test equipment only. Table B-I contains a listing of the required static test equipment and the costs. Table B-II contains a similar listing for the dynamic tests. Generally, the cost estimates are based on prices found in the manufacturers' catalogues. In some cases (e.g., the cost of the test rigs), the costs were estimated by assuming appropriate unit prices.

One major item of test equipment which is not included is the shaker for the vibration tests. It has been assumed that a shaker can be obtained on loan from the David Taylor Model Basin or the USCGS. The David Taylor Model Basin is mentioned as a source for obtaining a shaker since it possesses a number of these machines, and it has been indicated that one could be obtained on loan. The USCGS would be interested in any vibration-test results (for application in earthquake studies), and it seems that they would be willing to make a shaker available for these tests.

TABLE B-I

ESTIMATED COSTS FOR STATIC TEST EQUIPMENT

Quantity	Description	Unit Cost	Source	Total Cost
5	Test rigs*	$59,200	(assumed)	$296,000
2	50-kip hydraulic jacks	2,435	Messinger Bearings	4,870
11	100-kip hydraulic jacks	3,546	Messinger Bearings	39,006
12	200-kip hydraulic jacks	5,443	Messinger Bearings	65,316
1	Power and control unit			Unavailable
2	50-kip load cells	770	Baldwin-Lima-Hamilton	1,540
11	100-kip load cells	1,200	Baldwin-Lima-Hamilton	13,200
12	200-kip load cells	2,000	Baldwin-Lima-Hamilton	24,000
50	Ball and socket fittings	50	(assumed)	2,500
1	Indicator system	1,096	Minneapolis-Honeywell	1,096
2250 feet	Cable	0.35	Baldwin-Lima-Hamilton	788
2	Creep-detection pickups	200	(assumed)	400
25	Deflection gages	50	(assumed)	1,250
2	Transits	800	Gurley	1,600

*Each test rig weighs about 37 tons. A fabrication cost of $0.80 per pound has been assumed so that the price of one test rig is about $59,200.

TABLE B-II

ESTIMATED COSTS FOR DYNAMIC TEST EQUIPMENT

Quantity	Description	Unit Cost	Source	Total Cost
10	Velocity pickups	$ 170	Consolidated Electrodynamics Corp.	$1700
1	Tachometer	100	Hewlett-Packard	100
1	Direct writing oscillograph	1750	Consolidated Electrodynamics Corp.	1750
1	14-channel oscillograph	3375	Consolidated Electrodynamics Corp.	3375
11	Galvanometers	135	Consolidated Electrodynamics Corp.	1485
2	Dynamic reference galvanometers	120	Consolidated Electrodynamics Corp.	240
1	Dummy galvanometer	15	Consolidated Electrodynamics Corp.	15
1	Integrating amplifier system (10 channels)	5160	Consolidated Electrodynamics Corp.	5160
5	Cables, 25 feet long	17	Consolidated Electrodynamics Corp.	85
5	Cables, 50 feet long	20	Consolidated Electrodynamics Corp.	100

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