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Scale Model Testing of One-Way Reinforced Concrete Pier Hinges Subjected to Combined Axial Force, Shear and Flexure

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Abstract

This report presents the results of the second phase of an ongoing study at the University of Nevada, Reno. This phase involved laboratory and analytical evaluation of one-way reinforced concrete pier hinges subjected to a combination of uniaxial moment transfer, shear, and axial compression. Four one-sixth scale hinge models were built and tested in the strong direction. There were two primary variables in the testing sequence: shear-span to depth ratio (aspect ratio), and monotonic versus cyclic loading.

Analysis of the hinged specimens involved determining flexural and shear strengths, concentrated hinge rotations, and displacement of the column elements. A comparison between the measured and the calculated yield and failure loads is presented for each specimen. Various shear capacity equations and their accuracies are also examined relative to the measured data.

Hinge rotation and column deflection consisted of two components: reinforcement bond slippage and plastic deformation. Flexural displacements were determined from the curvature distribution along the column and included elastic deformation of the column and plastic deformation of the hinge. Empirical formulas used to estimate the rotations and displacements are discussed.

Results of the testing indicate that cyclic loading reduces the stiffness of the connection substantially and reduces the energy absorbing capabilities of the hinge.
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Notations

\( A_s \)  
Area of tensile steel.

\( a \)  
Depth of rectangular stress distribution from compression face.

\( a_y \)  
Stiffness reduction at yielding.

\( b \)  
Width of compression face of member.

\( d \)  
Distance from the extreme compression fiber to the centroid of tension reinforcement. Also called effective depth of the section.

\( d'' \)  
Distance from the centroid of compression steel to centroid of tension reinforcement.

\( e \)  
Steel elongation due to bar slippage.

\( E_c \)  
Modulus of elasticity of concrete.

\( E_s \)  
Modulus of elasticity of steel.

\( f'_c \)  
Compressive strength of concrete.

\( f_t \)  
Steel tensile stress.

\( f_y \)  
Yield strength of steel.

\( h \)  
Overall section depth.

\( I_s \)  
Gross section moment of inertia, \( \text{in}^4 \).

\( k_1, k_2, k_3 \)  
Factors for Baker's equation (Ref. 18).

\( k_d \)  
Distance from extreme compressive fiber to the neutral axis.

\( L \)  
Span length measured from the center of the support to the hinge face.

\( l_u \)  
Reinforcing steel development length.

\( l_p \)  
Equivalent plastic hinge length.

\( M_y \)  
Yield moment.

\( P_a \)  
Axial strength of member under pure compression.
\( P \)  
Axial compressive force applied to member.

\( z \)  
Distance from critical section to point of contraflexure.

\( \delta \)  
Displacement due to steel reinforcement bond slip.

\( \varepsilon \)  
Concrete strain at the extreme compression fiber.

\( \phi \)  
Curvature at yielding.

\( \theta \)  
Rotation due to bond slip.

\( \theta_y \)  
Rotation at yielding.
Chapter 1

Introduction

1.1 Background

Reinforced concrete hinges have been extensively used in structures for the past 70 years. In many reinforced concrete highway bridges, hinge details are used to connect foundations to columns and columns to decks. Column hinges fall into two categories, one-way and two-way hinges, as illustrated in Figure 1-1. A one-way hinge will prevent moment transfer in the weak direction, i.e., intended direction of rotation, while resisting bending moments in the strong direction. To induce hinge action in one-way pins, the column dimension is decreased in the direction of rotation and the reinforcing steel is aligned in a single row. In a two-way hinge connection, the column section is reduced in both directions and the reinforcing steel is grouped at the center. Two-way hinges are usually used in circular columns; one-way hinges are used in both rectangular and circular columns.

In the actual construction of a bridge, concrete is placed separately for the foundation and the column, resulting in a construction joint between the column and the footing. A shear key is incorporated at the joint to transfer horizontal forces from the column to the foundation. To allow the column to rotate with respect to the footing, the hinge "throat" typically has a depth of 1 to 4 inches.

One-way hinges in bridge columns may be subjected to a combination of loads, such as axial compressive force due to the dead load of the bridge superstructure, shear forces in the hinge in the two principal directions, and bending moments in the strong direction. The latter two loads may be due to lateral loading caused by either ground motion or wind.

One-way hinge connections are designed to carry axial compressive forces according to Sec. 10.3.5 of the American Concrete Institute (ACI) building code, commonly referred to as ACI-318. Sec. 11.7, which describes the shear friction method (SFM), is used to design for shear.

Many modern highway bridges in areas of high seismic risk are supported by bents consisting of one or more columns. Seismic design of highway bridge columns is usually based on building codes for columns in frame buildings. However, the basis for designing bridge columns using building codes may not be valid, due to several important differences which exist between bridge and building columns:

1. Building columns usually have smaller cross-sections than bridge columns.
2. Because of their smaller dimensions and more complex beam to column joint
details, the use of reinforcing steel greater than No. 11 bars is not a common
practice in building columns; however, No. 14 and No. 18 bars are frequently
used in bridge columns. The differences in bonding characteristics between the
smaller and the larger bars may also contribute to performance differences.

3. Building columns typically carry higher axial stresses than bridge columns.

4. The general design approach for building frames is based on developing plastic
hinges in beams and not in columns. In contrast, development of plastic hinges
in bridge columns is necessary for energy dissipation under lateral loads.

5. The reinforcement ratio in bridge columns is smaller than in building columns.
Bridge columns typically have a reinforcement ratio of less than 2 percent.

Because of the devastating effects on highway bridges of the 1971 San Fernando
earthquake, seismic design procedures for bridges in the United States have changed
significantly. Damage to highway bridges from the earthquake included five collapsed
bridges and 42 which suffered major damage. The primary causes of pier damage were
identified as:

1. Insufficient ductility of bridge columns to absorb the inelastic displacements
   experienced.

2. Shear dominated failures in shorter columns.

3. Anchorage failures of longitudinal reinforcement in the plastic hinge locations
   formed at the base of columns.

1.2 Previous Work

Very little research has been done on bridge column-to-foundation one-way hinge
connections subject to a combination of axial, shear, and flexural loadings in the
moment-resisting direction. However, there has been extensive research and testing to
determine the bearing, shear, and flexural capacity under monotonic and cyclic loading
induced in the weak direction.

In 1965, G. D. Base conducted research on four prototype reinforced concrete
hinges with the loading applied in the rotation direction (about the weak axis). He tested
three different types of hinges: a Freyssinet hinge, which has very little reinforcement
through the hinge section; a Messanger hinge; and a saddle bearing hinge. Base induced
a series of different loadings on the reinforced concrete hinges: axial load only, combined
axial and shear loadings, and axial loading with cyclic flexural loadings.
The results of these tests indicate that the specimens were able to carry design loads with substantial factors of safety and allow for rotations greatly in excess of the design requirements. Concrete compressive stresses in the hinge throats reached values several times the compressive strength of the concrete without causing crushing. Base also discovered that the reinforcing steel through the hinge throat appeared to be unnecessary. The static shear resistance of the hinge section appeared sufficiently adequate, and only the consideration of impact shear would create a need for diagonal reinforcement to assist in shear resistance.

A preliminary study was conducted on one-eighth scale reinforced concrete bridge piers at the University of Nevada in 1988; four model bridge piers were built and tested. Three of the specimens with shear-span to depth ratio (aspect ratio), $l/h$, varying from 1 to 3 were loaded monotonically to failure. The fourth specimen had an aspect ratio of 3 and was tested cyclically. The scope of the research was to subject a typical one-way bridge pier hinge to lateral loads and determine if the shear friction theory was indeed valid for this type of application. The results of these tests indicated that the shear friction method can overestimate the shear capacity of a typical one-way hinge by as much as 100 percent.

The pilot study showed that the mechanism for shear resistance in a one-way hinge is different than what the shear friction theory has indicated. The shear friction method assumes that aggregate interlock takes place over the entire length of the crack in the hinge region. According to this method, when an initially cracked reinforced concrete specimen is loaded monotonically in shear, slippage will occur along the crack interface. As the two concrete segments on opposite sides of the crack slide relative to each other, tension is introduced into the reinforcement bars, as shown in Figure 1-2. To maintain equilibrium, the reaction from this tensile force is a net compressive force normal to the crack. This net compressive force is multiplied by a friction factor; the product is the shear resistance of the section.

The sliding mechanism along the crack face subjects the reinforcing steel to a shearing action, commonly referred to as “dowel action.” Dowel action can be developed by three mechanisms: flexure of the reinforcement, shear stress across the steel bars, and kinking of the reinforcing steel.

Initially-cracked reinforced concrete members behave differently under cyclic loading. After one cycle, the specimen builds up residual tensile strain in the reinforcing steel which prevents the crack along the shear plane from closing immediately after the load has reversed. This results in shear transfer by dowel action as well as aggregate interlock. After the initial cycle is completed, the shear stiffness of the specimen is much lower than the stiffness during the previous loading cycle. The specimen experiences slip equal to the previous maximum slip until the contact sections again come into bearing. The resistance of the contact areas to deformation results in an increase in shear resistance. Further cyclic motion results in similar behavior until the two surfaces in contact are worn smooth, thus reducing the portion of shear resistance normally provided by aggregate interlock.
The mechanical behavior of a one-way hinge, shown in Figure 1-3, is considerably different than that discussed in the shear friction method. Usually, a large flexural crack forms in the concrete, limiting the contact area for aggregate interlock to the compression zone of the section. The net compressive force is the summation of the reinforcing steel force and the concrete force crossing this compression region. Thus, the resultant compressive force may be very different from that obtained when all bars are acting in tension as indicated by the shear friction method.

Researchers at Washington State University in Pullman are developing a modified hinge detail to reduce the size of the foundation for economic reasons. Their objective is to build forty, one-twentieth scale and six one-sixth scale specimens and subject them to cyclic loadings that will produce lateral deflections of up to 14 times the yield displacement. Preliminary findings of the small-scale study indicate that the modified hinge detail appears to be better than the unmodified hinge detail in absorbing the energy induced by lateral forces. This improvement is a result of the confinement provided around the hinge throat by the addition of an outer segment of the architectural column. At the time of this writing, work is being conducted on the one-sixth scale models to substantiate the findings of the small scale specimen tests.

1.3 Object and Scope

A continuation of the research which started at the University of Nevada in 1985, the present study involves testing hinge details for bridge column-to-foundation connections subjected to lateral loading in the strong direction. The objective of this part of the study was to determine the effects of pier aspect ratio and cyclic loads on the lateral response of one-way hinges in the presence of a constant axial load.

The major differences between the current study (Standard Detail specimens) and the pilot study (Concrete Hinge specimens) are the following: the scale of SD specimens is one-sixth compared to one-eighth scale for the CH series; deformed No. 3 reinforcing bars were used in the SD specimens, whereas plain No. 2 bars were used in the original study; and a constant axial load was applied to the SD specimens, whereas there was no axial load for the CH series.

In the current study, two test variables were considered: monotonic versus cyclic loading and shear-span to depth ratio (aspect ratio). The shear-span to depth ratio is defined as the distance from the point of zero moment to the point of maximum moment, \( l \), divided by the total depth of the section in the strong direction, \( h \). The first two specimens, with aspect ratios of 1 and 2, were loaded monotonically to failure. The last two test specimens, also with aspect ratios of 1 and 2, were subjected to cyclic lateral deformations with increasing amplitude levels until failure. The shear-span to depth ratio was varied to determine its effects on shear and flexural capacity of the hinge and to determine a limiting shear span to induce a shear failure. By decreasing the shear span of the column, the shearing effect in the hinge region will become more dominant.
Previous research has shown that the strength of columns having a shear-span to depth ratio less than 2.5 is controlled by shear related failures.\textsuperscript{12,23,27}

During testing, specimens subjected to cyclic lateral loading experienced ultimate displacements of up to four times the yield displacement. It should be noted, however, that very slow static lateral displacements were applied to each test specimen. This was done to monitor the cracking pattern and the overall response of each specimen. The cyclic testing was performed to obtain a general insight into the effects that load reversals have on shear and flexural strengths and to give an indication of energy dissipation in the hinge section.

The long-term goal of this project is to develop a method for estimating the shear behavior of common reinforced concrete hinge details subjected to lateral loads, and to develop details that will improve the energy dissipation capacity of one-way hinges.
Chapter 2
Experimental Study of Hinged Specimens

2.1 Introduction

Four one-sixth scale model foundation-to-column hinge connections were tested. These models were designed to represent piers 2 and 3 of the Rose Creek Interchange (I-862) located on Interstate-80 in Winnemucca, Nevada, shown in Figure 2-1. The test specimens, shown in Figure 2-2, consist of two elements: a lower portion, representing the foundation, and an upper portion, representing the column. All specimens were tested in the upright position with an axial load applied at the top of the column to simulate the dead load of the bridge deck. Specimens were laterally loaded (combined shear and flexure) in the strong direction.

This chapter describes the test specimens, equipment, and procedure used in the testing program.

2.2 Test Specimens

Four standard detail (SD) specimens were tested: SD1M, SD2M, SD1C and SD2C. The first two specimens were loaded monotonically; the last two specimens were loaded cyclically. The first and third specimens had a shear span to depth ratio (aspect ratio), l/h, of 1; the ratio for the second and fourth specimens was 2.

All specimens had a 14.5-inch by 18-inch by 24-inch footing section. The column section measured 6.5 inches by 16 inches by 22 inches for specimens SD1M and SD1C and 6.5 inches by 16 inches by 38 inches for SD2M and SD2C. Six No. 3 deformed Grade 60 reinforcing bars were used to connect the footing to the column. Concrete cover was 1 inch for the outer two dowels. Inner dowels were spaced at approximately 2.75 inches, on center.

The six reinforcing dowels had 90 degree, 6-inch standard hooks on either end with a straight segment of 8 inches in the footing section. Test specimens SD1M and SD2M had No. 3 Grade 60 deformed bars for U-stirrups and no horizontal ties located in the column section. In specimens SD1C and SD2C, both U-stirrups and horizontal ties were used, to enhance concrete confinement in the column section. Horizontal ties were plain No. 2 bars, spaced at 6.5 inches, on center, with 1 inch of concrete cover both top and bottom. The connection between the footing and the column was formed with a 2.5-inch by 16-inch by ½-inch keyway. The hinge throat was fabricated by placing a ½-inch thick piece of styrofoam in the base section prior to concrete placement. Two ¾-inch thick styrofoam inserts were placed on either side of the keyway to ensure no bonding.
occurred between the outer portions of the column and the footing. After the footing section was poured, the throat area was scraped to achieve a roughness amplitude of approximately 1/8-inch before the upper section was poured. Figure 2-3 is a photograph of the base segment after curing. The reinforcing bars, the upper strain gage location, the keyway, and the styrofoam used to keep the column and the footing from adhering to each other can be seen. The cross-sectional area of concrete at the keyway was 40 square inches. The reinforcement ratio at the hinge throat was 1.65 percent; the steel ratio in piers 2 and 3 of the Rose Creek Interchange is 1.30 percent.

2.3 Materials and Fabrication

Fine and coarse aggregates for the concrete were obtained from a local pit in the Reno area. The coarse aggregate was sieved to remove material larger than 1/4-inch. The coarse aggregate failed to meet the requirements of American Society for Testing and Materials (ASTM) Specification C33 on the No. 8 sieve; the fine aggregate did meet all requirements for ASTM C33. Further information concerning the aggregates is contained in Appendix A. The concrete mixture used type I-II low-alkali portland cement; proportions for the four specimens are listed in Appendix B.

The concrete was batched in a 4-cubic foot revolving drum mixer with a mixing time of approximately 10 minutes. Three 6-inch diameter by 12-inch high cylinders were cast during both footing and column pours. Compressive strength testing was performed after 7 days, after 28 days, and on the day the corresponding specimens were tested. Appendix C lists the results of the compressive strength testing.

Reinforcing dowels for the four test specimens consisted of No. 3 Grade 60 deformed bars. The yield stress was 58,000 psi for the bars used in SDIM and SD2M and 57,000 psi for the bars in SD1C and SD2C. Detailed information about the steel properties is contained in Appendix D. The horizontal ties used in specimens SD1C and SD2C were plain, i.e., non-deformed, No. 2 Grade 40 bars.

Forms were constructed using 2×4 lumber and 1/8-inch plywood. To allow easy removal of the forms after concrete placement, screws were used in the side panels. The forms were cleaned and coated with polyurethane to reduce water absorption and ease form stripping.

The two portions of the model bridge pier were poured separately to simulate field construction. The footing section was first poured and moist-cured for 24 hours. The styrofoam used to form the keyway was then removed and a chisel used to roughen the concrete until the keyway had an amplitude of approximately 1/8-inch, to ensure good bonding between the foundation and the column. After an additional 48 hours of moist-curing, the column was cast onto the footing. The entire specimen was then moist-cured for an additional four days. After the moist-curing period, the forms were removed and the specimens were allowed to cure at room temperature until testing.
2.4 Instrumentation

A Hewlett-Packard 9000 Series microcomputer interfaced with a 3054 data acquisition system was used to record electrical strain gage and linear variable differential transformer (LVDT) measurements for the test specimens. A computer software program named PIERHINGE, listed in Appendix E, was developed to convert, collect, and store the test data (strains, displacements, and loads). A flow chart for the computer program is shown in Figure 2-4. The user’s manual for PIERHINGE is presented in Appendix F.

An MTS structural testing system was used to laterally load the test specimens. The MTS system has a 55,000-pound load cell and an actuator arm with a stroke of ±3 inches. The hydraulic arm is displacement controlled from a 458.20 microconsole. An axial load of 26,000 pounds was applied by means of a 300,000-pound Riehle machine. The test specimens were thus loaded with a combination of flexure, shear, and axial force.

Instrumentation for the different specimens varied slightly: specimens SD1M and SD2M had 24 electrical resistance strain gages while SD1C and SD2C had 18 strain gages. The strain gages were mounted on the reinforcing bars, within each specimen. Twelve gages were located in the hinge region; the remaining gages were located above the hinge throat, at 3-inch intervals (See Figures 2-5 and 2-6). The bars in the columns of specimens SD1M and SD2M had one gage on both front and back face to compensate for out-of-plane bending. Fewer gages were used in SD1C and SD2C because out-of-plane bending was found to be negligible. The deformed bars were ground smooth and thoroughly cleaned at the strain gage locations. The strain gages were bonded to the reinforcing steel using an epoxy adhesive.

Specimens SD1M and SD2M used two 2-inch LVDTs to measure the rotation of the column section relative to the foundation and one 1-inch LVDT to measure the horizontal slippage of the column with respect to the base. In addition to the LVDTs, two 1-inch dial gages were used to observe torsional effects from the lateral loading. Figure 2-7 shows this instrumentation.

To achieve better sensitivity for specimens SD1C and SD2C, two 1-inch LVDTs were used to measure the rotation of the column and one ½-inch LVDT was used to measure horizontal slip of the column relative to the footing. In addition to the dial gages described above, two 1-inch dial gages were used to monitor foundation movement relative to the column, to check whether the footing was rocking under load, and contributing to the lateral displacement of the column. The additional dial gages were placed as shown in Figure 2-8.

The test specimens were connected to the load frame by eight 1-¼ inch diameter, 125,000 psi (tensile strength) threaded rods. To create a passageway for the threaded rods in base section of the test specimen, eight 24-inch long segments of 1-¼ inch
Schedule 40 PVC pipe were included in the foundation formwork. A 1-inch thick steel bearing plate was used at the end of the test specimens to distribute the load across the face of the foundation. Figure 2-9 illustrates the test frame setup.

A steel collar, designed to carry a load of 55,000 pounds, was built to connect the hydraulic ram to the column section of the specimen. The collar was built from \( \frac{1}{4} \)-inch thick by 5-inch wide steel plates, with four 1-inch A325 bolts on each side. Two 1.875-inch diameter pins were attached to allow for rotation as the ram pushed and pulled the specimen. Two sections of C6×13 channel were used to connect the collar to the hydraulic ram. Figure 2-10 illustrates the details of the collar.

2.5 Test Procedure

The test procedure was essentially the same for all four specimens; however, the loading was monotonic for SD1M and SD2M and cyclic for SD1C and SD2C. The specimens were laterally displaced a predetermined amount, the displacement halted, and the computer triggered to record electrical strain gage measurements, center span deflection, lateral load, and LVDT displacements. A pen plotter was used to simultaneously plot lateral load versus center span deflection. Dial gage readings were also taken and recorded on data sheets. The applied vertical load was 26,000 ± 1000 pounds, representing the same stress in the hinge throat as the Rose Creek Interchange. An outline for the loading procedure and failure modes for each specimen follows.

The first step in the testing procedure was to apply the axial load of 26,000 pounds in increments of approximately 5000 pounds. Once the total axial load was applied, the threaded rods were tightened to prevent movement during loading. The hydraulic ram was then bolted to the column collar and the actual test was ready to proceed.
Chapter 3

Results of Hinged-Specimen Testing

3.1 Introduction

The results presented in this chapter describe the experimental data recorded during each of the model tests. The following are discussed:

1. Lateral load versus deflection.
2. Lateral load versus strain.
3. Lateral load versus rotation.
4. Lateral load versus slip.
5. Axial load versus deformation.
6. Column twist and base rotation contributing to total lateral deflection.

These data give an indication of the stiffness characteristics and strength decay for each test specimen.

Before testing, each specimen was inspected for unusual or dominant cracking patterns. There were no major visible cracks in any of the test specimens; however, there were some minor shrinkage cracks.

Specimens SD1M and SD2M were monotonically loaded to failure, where failure is defined as the point where the lateral load has decreased to 85 percent of the maximum load. Points at which individual bars yielded are indicated on the measured response curves for specimens SD1M and SD2M. The number indicates which layer of steel yielded in tension during loading.

The cyclically-loaded specimens, SD1C and SD2C, were subjected to several cycles of increasing lateral displacement amplitudes and ductility levels. No particular earthquake response history was simulated during the testing phase. Figure 3-1 shows the numbering system used for the reinforcing steel located within each specimen: bar 1 being on the front face (right side of the diagram) of the column; bar 6 on the back face of the column (far left side).
3.2 Specimen SD1M

3.2.1 Load–Deflection Response

Figure 3-2 shows the load–deflection curve for specimen SD1M, which had an ultimate lateral load of 25,300 pounds. The specimen cracked at a load of 3600 pounds as indicated by the change in the slope of the load–deflection diagram. Yielding of bar 1 in tension occurred at a load of 18,240 pounds, corresponding to a lateral displacement of 0.20 inches. Failure of the specimen occurred at 21,500 pounds and a lateral displacement of 1.54 inches; this displacement corresponds to an apparent displacement ductility factor of 7.70. Displacement ductility is defined as the maximum lateral deflection divided by the yield deflection.

Figure 3-3 shows the actual cracking pattern of the specimen near the failure point. The photograph was taken at a load of 25,300 pounds and a lateral deflection of 1.29 inches. The photograph shows that there was major cracking on the compression side of the hinge just above the interface between the column and the foundation. The cracks propagate up the column face, indicating severe stresses in the upper part of the column. The severity of cracking is partly due to the absence of horizontal ties in the column region which would have provided some confinement.

3.2.2 Load–Strain Response

Figures 3-4 through 3-15 illustrate the relationships between lateral load and strain distribution in the reinforcing steel.

After the initial axial loading of the model with the 26,000-pound axial load, all of the bars in the hinge throat were in compression. As the lateral load was applied, the strain distribution changed from compression to tension in bars 1 through 5. Because of its location, bar 6 remained in compression.

To explain the load–strain curves, the curves for bars 4, 5, and 6 can be examined. Figure 3-10 shows the relationship for bar 4, and is composed of three distinctly different segments. The first segment, from point A to B, indicates that the lateral load creates negligible strain in the bar; the strain is mainly a result of the applied axial load. Between points B and C, the lateral load becomes sufficiently large to overcome the compressive stress from the axial load, causing tensile strains to develop in the bar. The last segment, from point C to point D, shows large strains due to the rotation of the column section and separation of the column from the footing; there is no concrete contact between the foundation and the column except in the compression zone at the far left end of the specimen. The reinforcing bar eventually yields, and large strains develop with little increase in section load capacity.
Figure 3-12, representing bar 5, also has three distinct regions. The first region, from point A to point B, shows the response from the initial axial loading. As the lateral load is increased, compressive strain increases to a maximum at point B, where the applied lateral load is approximately 11,000 pounds. The second region, from point B to C, shows that the lateral load is large enough to cause tension and a decrease in the compressive strain in the bar. At a lateral load of approximately 20,000 pounds, the bar undergoes a stress reversal to tension. The final region, from point C to D, shows that the bar is undergoing large strains with little increase in lateral load. The bar eventually yields in tension. This is a result of the large relative rotations present, which limit the contact area between the column and the foundation.

Figures 3-13 through 3-15 show that bar 6 remains in compression. This bar is located on the left end of the column, where the large rotations cause the concrete column and footing to bear against each other.

The extent of yielding penetration can be observed in Figure 3-4, for tension, and in Figure 3-13, for compression. The bars yielded at a distance up to 6 inches from the hinge throat, as the lower sections underwent strain-hardening.

3.2.3 Load-Rotation Response

Figure 3-16 shows the load-rotation curve for specimen SD1M. Up to yielding of bars 1 through 4, the specimen exhibited small rotations, but after yielding, large rotations developed. The initial yield rotation was 0.0044 radian. The rotation near the end of the test, when the load dropped to 85 percent of the peak load, was 0.083 radian. This corresponds to a rotation ductility of 18.9, which is quite high for a specimen with an aspect ratio of 1.

3.2.4 Load-Horizontal Slip Response

The load-horizontal slip curve, Figure 3-17, shows that up to yielding very little slip has occurred between the column and the footing. After bars 1 through 4 yielded in tension, and the column had separated from the footing over most of the hinge area, large horizontal slips were recorded. The initial yield slip was 0.014 inch; the slip at failure was 0.152 inch. The slip ductility factor was 10.9.

3.2.5 Axial Load-Deformation Response

Figure 3-18 shows the relationship between axial force and axial deformation of the column section. The LVDTs that measured axial deformation were located approximately 6 inches from the top of the footing. Shortening was measured over a distance of 6.5 inches, including the ½-inch hinge throat depth. The discontinuity at 10,000 pounds applied lateral load is most likely due to the LVDT signal noise level.
3.2.6 Load–Column Twist Response

Two dial gages were used to monitor out-of-plane twisting caused by the hydraulic ram. As shown in Figure 3-19, the column experienced very little out-of-plane rotation relative to the footing. A column twist of 0.0005 radian was measured at the initial yield load, corresponding to a displacement component of 0.0016 inch at the corner of the column in the loading direction. This is negligible when compared to the yield displacement of 0.24 inch.

3.3 Specimen SD2M

3.3.1 Load–Deflection Response

The load–deflection curve for SD2M, Figure 3-20, shows that bar 1 yielded in tension at a lateral load of 9120 pounds and a corresponding deflection of 0.19 inch. The peak load was 13,100 pounds at a displacement of 1.0 inch. Failure occurred at a lateral load of 11,110 pounds with a corresponding displacement of 1.38 inches. This relates to a ductility factor of 7.26, which is close to the ductility factor of 7.70 observed for SD1M. Specimen SD2M initially cracked at 4500 pounds.

Figure 3-21 is a sketch of the cracking pattern for test specimen SD2M. Because the photographs taken during this test were of poor quality, a video tape was reviewed and a hand drawing was made to show the cracking pattern at the failure point. Major cracks are present on the compression face of the column. Vertical crack propagation is apparent on the compressive side of the column. The tests of other tied specimens revealed that the cracking of SD2M was a result of the lack of horizontal ties in the pier section. The cracking pattern for SD2M is similar to what was observed for SD1M.

3.3.2 Load–Strain Response

Figures 3-22 through 3-33 present the data collected for the load–strain response of specimen SD2M. The data can be grouped into three groups: the first for bars 1 through 4 (Figures 3-22 through 3-28); the second for bar 5 (Figures 3-29 through 3-30); and the last for bar 6 (Figures 3-31 through 3-33).

Figures 3-22 through 3-28 show three distinct line segments on the load–strain diagram; Figure 3-22 will be used to explain the load–strain response curve. Initially, as the axial load is applied the bar goes into compression (point A to point B). As the lateral load is applied, from point B to point C, the strain reverses from compression to tension. The last segment, from C to D, is a result of the large flexural crack which developed between the footing and the column. As the flexural crack propagates, large rotations are developed between the two concrete segments with minimal contact only in the
compression region. Beyond this point, the reinforcing steel has yielded, and larger tensile strains are recorded with every increase in lateral load.

Figures 3-29 and 3-30 represent the load–strain response for bar 5. Again there are three distinct line segments to the curve. The first segment of Figure 3-30, from point A to point B, shows the application of the axial load. At point B (about 6000 pounds lateral load), the maximum compressive strain has been reached. A slow decrease in compressive strain is then seen, due to the increasing lateral load. The final segment, point C to point D, shows that the strain has reversed from compression to tension. The only concrete section of the column still in contact with the footing is the far left edge which contains bar 6.

Figures 3-31 through 3-33 show the data for bar 6, which remains in compression throughout the test. As the pier rotates counter-clockwise, the rear of the column is forced downward into the footing, resulting in increasing compressive strains.

3.3.3 Load–Rotation Response

The load–rotation curve for SD2M, Figure 3-34, shows that prior to yielding in tension, small rotations were experienced in the section. The initial yield rotation was 0.0022 radian. After yielding of bar 4, large rotations occurred. At the point of failure, a rotation of 0.039 radian was recorded, corresponding to a rotation ductility of 17.7, almost the same as that obtained for specimen SD1M.

3.3.4 Load–Horizontal Slip Response

Figure 3-35 shows the load–horizontal slip curve for specimen SD2M. Up to a lateral load of 5000 pounds, practically no slip has occurred between the column and the footing. Between 5000 and 10,000 pounds, slippage is starting to occur, but no major movement has developed. An initial yield slip of 0.0026 inch was measured. The slip was 0.04 inch at failure, which relates to a slip ductility of 15.4. This ratio is 1.5 times the value of 10.9 observed for SD1M, indicating that shear was more dominant in SD1M.

3.3.5 Axial Load–Deformation Response

Figure 3-36 shows the relationship between the axial force and the deformation in the lower 6.5 inches of the column. No significant deformation was noted. The relationship is nearly linear; the slight discontinuity is probably due to the noise levels of the LVDTs.
3.3.6 Load–Column Twist Response

Figure 3-37 shows the load–column twist data for specimen SD2M. The column experienced very little twist: even at the maximum load of 13,100 pounds, the measured angle of twist was only 0.0035 radian. The displacement of 0.011 inch, which results at the level and direction of horizontal load, is insignificant when compared to the 1.0 inch of deflection due to the lateral load.

3.4 Specimen SD1C

3.4.1 Load–Deflection Response

Figure 3-38 presents the lateral loading history for specimen SD1C. The specimen was subjected to nine displacement cycles at four different amplitude levels. Figure 3-39 shows the load–deflection response for the specimen. Initially, two cycles of ±0.1-inch displacement were completed to capture the cracking point of the specimen. The specimen was then subjected to two cycles of ±0.25-inch displacement. This displacement was the initial yield point of specimen SD1C. Three cycles were applied with a deflection of ±0.50 inch, for a displacement ductility factor of 2, to monitor the effects of cyclic displacements, with a moderate degree of nonlinearity, on strength degradation of the specimen. Figure 3-39 illustrates the pinching of the hysteresis loops, indicating a reduction in the energy absorption capacity of the hinge. Finally, a displacement corresponding to a ductility level of 4.0 was applied (±1 inch lateral displacement) for two cycles; significant pinching was noted in the hysteresis loops. The sudden drop in lateral load during the last cycle was caused by the tensile failure of bar 1.

The positive load side of Figure 3-39 shows that the largest load achieved was 26,800 pounds during the fifth cycle. On the final cycle, a maximum load of 20,400 pounds was reached. The strength degradation caused by the cyclic loading was 25 percent at failure.

Figure 3-40 shows the cracking pattern during the ninth cycle at a lateral load of 17,040 pounds and a corresponding displacement of 0.89 inch. In the front of the photograph, the column has separated from the footing. At the rear of the photograph, the right corner of the column has a major crack running diagonally, starting approximately 3 inches up the side of the column and propagating to the bottom center of the column. This crack formed after the expansion joint material had been compressed and the concrete outside the hinge throat had come in contact with the footing. Horizontal ties were used in the column section, to provide confinement and enhance ductility, which significantly reduced the amount of column cracking. Figure 3-41 shows a close-up of the hinge section with bar 1 failing in tension.
3.4.2 Load–Strain Response

In specimens SD1C and SD2C, 18 strain gages were used to record strain values. Two strain gages were used on each bar, front and back, through the hinge throat region; however, only one gage was used above the hinge region to monitor the strains in the bars. Because there was negligible out-of-plane bending of the bars in specimens SDIM and SD2M, redundant gages were eliminated.

Figures 3-42 through 3-53 present the lateral load–strain data collected during the cyclic loading of specimen SD1C. In bars 1 and 6 (Figures 3-42 and 3-47, respectively), the strains change from compression to tension, as expected for the outer bars during cyclic testing. However, failure of the strain gages during testing resulted in limited strain data collection.

Figures 3-43 through 3-46 show the strain distribution for the gages located in the hinge throat on bars 2 through 5. After the lateral load was sufficiently large to overcome the effect of the axial force, the strains remained tensile. As the loading was cycled back and forth, the concrete in the hinge throat failed, and the bars were pushed and pulled in tension.

Figures 4-48 through 3-53 show the load–strain response for the gages located above the hinge throat for bars 1, 2, 5, and 6. The cyclic nature of the strain distribution is a result of lateral loading: one end of the column is subjected to tension while the other is in compression. As the load is applied toward the left, bars 1 and 2 (Figures 3-48 and 3-49) go into tension while bars 5 and 6 (Figures 3-50 and 3-51) experience compression. Just the opposite occurs when the load is reversed. The maximum strains show that yielding extended well into the column.

3.4.3 Load–Rotation Response

Figure 3-54 shows the load–rotation response of specimen SD1C. Initially, two cycles approximately equal to the cracking point of the section were completed; very small rotations of about 0.00077 radian were experienced. The initial yield displacement was applied and the corresponding rotation of 0.0079 radian was measured. Next, three cycles at a displacement ductility level of 2 (±0.50 inches), with a corresponding rotation of 0.021 radian, were completed. Finally, the last two cycles were performed at a displacement ductility level of 4 (±1.0 inch). The ultimate rotation was 0.051 radian, which correlates to a rotation ductility factor of 6.46. This is approximately 36 percent of that obtained for specimen SD1M.

A comparison of Figures 3-54 and 3-39 shows that the general shape of hysteresis curves for rotation is similar to that of displacement. This is because a major portion of horizontal displacement of the column is due to rotation at the base. However, the pinching effects in Figure 3-54 are less severe because the column horizontal slippage is not reflected in the rotations.
At large-amplitude cycles, peak rotations are larger in the positive direction than they are in the negative direction. This is because the effective axial load in the negative region was larger, thus reducing the curvature and the resulting rotation.

3.4.4 Load–Horizontal Slip Response

The load–horizontal slip curve, Figure 3-55, shows that at the cracking point (±0.08 inches), the slip between the column and the footing was negligible. At the initial yield point, a slip of 0.03 inch was recorded, twice that of specimen SD1M. On the seventh cycle, with a displacement ductility of 2, a horizontal slip of 0.13 inch was noted, corresponding to a slip ductility of 4.3. Finally, on the ninth cycle, a maximum slip displacement of 0.25 inch was achieved, which corresponds to an ultimate horizontal slip ductility factor of 8.3.

The peak slip in the positive direction was lower than the negative slip. The trend is opposite to that observed in the rotation response. Considering that the displacements in the positive and negative directions are forced to be the same, and the horizontal displacement is dominated by a component due to the rotation at the bottom of the column and a component due to the horizontal slip, the trend is logical because rotation and slip have to compensate for each other.

3.4.5 Axial Load–Deformation Response

Figure 3-56 shows the relationship between the axial force and the deformation of the lower portion of the column. The response was elastic, as anticipated. The discontinuity between line segments at an 11,000-pound lateral load is a result of the noise level of the external LVDTs.

3.4.6 Load–Column Twist Response

Figure 3-57 shows the lateral load–column twist for specimen SD1C. Only the extreme data points were plotted. The maximum column twist angle recorded was 0.0033 radian, corresponding to a corner displacement of 0.011 inch. This is insignificant compared to the total lateral deflection of 1.0 inch.

3.4.7 Load–Base Rocking Response

Figure 3-58 shows the reduced data acquired from dial gages placed on the foundation of the specimen to monitor rocking of the base. This rotation was converted to a deflection to compare with the displacement measured during testing. Only the extreme data points were used to calculate foundation rocking. The maximum base movement contributing to the total deflection of the column was 0.016 inch; when compared to the total lateral displacement of 1.0 inch, this value is negligible.
3.5 Specimen SD2C

3.5.1 Load-Deflection Response

Figure 3-59 shows the load history response for specimen SD2C. The specimen underwent a total of ten and one-quarter cycles with five different amplitude levels.

Two small cycles of ±0.1 inches were performed to capture the cracking point of the specimen. Next, two cycles at the initial yield displacement (±0.25 inches) were conducted. Three cycles were then completed at a displacement ductility level of 2 (±0.50 inch) to determine strength degradation during moderate ground motions. A ductility level of 4 (±1.0 inch) was applied for three cycles to monitor the effects of strong earthquakes. Finally, the specimen was subjected to one-quarter cycle at a displacement of 1.33 inches (ductility factor of 5.32) to cause failure, since the column still exhibited considerable strength in the joint region.

The load-deflection response curve for SD2C is shown in Figure 3-60. A maximum load of 14,400 pounds, with a displacement of 0.93 inch, was achieved on the eighth cycle. It should be noted that there was no appreciable strength degradation in the connection, indicated by the almost perfect overlapping of the cycles. On the last cycle, the lateral load was 13,960 pounds, only a 2.9 percent drop in lateral load. Even at a ductility level of 2 (±0.50 inch lateral displacement) some slight pinching of the hysteresis loops is present. At a ductility level of 4, significant pinching is apparent, indicating a loss of the energy-absorbing capability of the connection.

Figure 3-61 shows the cracking pattern at the two-hundredth loading increment: the lateral load was 13,500 pounds and the horizontal displacement was 1.03 inches. Very little column cracking can be seen, due to the presence of horizontal ties in the column which provided confinement for the concrete.

3.5.2 Load-Strain Response

Figures 3-62 through 3-73 show the relationship between lateral load and strain distribution in specimen SD2C. As in SD1C, gages 1 through 12 were located in the hinge throat and gages 13 through 18 above the joint interface. In bars 1 through 5 (Figures 3-62 through 3-66), the strain remained in the tension zone during most of the testing. During small amplitude cycles (approximately equal to the cracking point of the specimen), however, the strain distribution did cycle from compression to tension.

Figure 3-62 shows that bar 1, unlike other tests, remained in tension during most of the cyclic loadings. For this result to be correct, the neutral axis for negative loading would have to be to the right of bar 1. The compression area would then be unrealistically small. Furthermore, strains on bar 1 at other locations (Figures 3-68 and
3-72) show significant compression in this bar. It is, hence, concluded that the data in Figure 3-62 are erroneous.

In Figures 3-67 through 3-73 the load-strain curves are quite different: the strains cycle from positive to negative as expected for a cyclic test. During positive loading (pushing), bars 4 through 6 experience compression and bars 1 through 3 are in tension. During negative loading (pulling), just the opposite occurs.

3.5.3 Load-Rotation Response

The load-rotation curve (Figure 3-74) shows that at displacement amplitudes equal to that of the cracking point, a rotation of 0.00124 radian was recorded. At the initial yield point (±0.25 inches), the rotation was 0.00267 radian, about twice that obtained at the cracking point. At a displacement ductility of 2 (±0.5 inches) during the seventh cycle, a rotation of 0.0123 radian was measured, corresponding to a rotation ductility of 4.61. On the tenth cycle (1.0 inch deflection), the specimen had a rotation of 0.028 radian; this relates to a rotation ductility of 10.5, almost twice that obtained for specimen SD1C. Finally, on the eleventh cycle, the rotation was 0.0384 radian, corresponding to a rotation ductility of 14.4. The lateral displacement was 1.33 inches for this cycle.

The hysteresis loops for rotation are not as narrow as those in the displacement response (Figure 3-60) because horizontal slip deformations are not included in rotations. Similar to what was observed for SD1C (Figure 3-54), the peak rotation in the positive area was larger than that in the negative direction due to differences in the axial loads in the two different directions.

3.5.4 Load-Horizontal Slip Response

Figure 3-75 shows the load-horizontal slip response for specimen SD2C. At the initial yield point, a slip of 0.0061 inch was measured, which is insignificant compared to the overall slip monitored during testing. At a displacement ductility level of 2 (±0.5 inches), the horizontal slip was 0.025 inch; this is negligible compared to the slip monitored for specimen SD1C. On the tenth cycle (ductility level of 4) a maximum slip of 0.123 inch was reached. The ultimate slip ductility was 20.2, about 25 percent higher than that obtained for SD1C. The explanation for the lack of symmetry is similar to that given for SD1C.

3.5.5 Axial Load-Deformation Response

Figure 3-76 shows the relationship between axial load and axial shorting of the lower portion of the column. The several minor discontinuities are the result of electronic noise in the LVDTs. The general shape of the diagram is nearly linear, as expected.
3.5.6 Load–Column Twist Response

Dial gages were attached to the column to monitor twisting during testing. However, soon after testing began it was apparent that the column was experiencing insignificant movement; therefore, dial gage readings were discontinued.

3.5.7 Load–Base Rocking Response

Figure 3-77 shows the rotation that was measured with dial gages located on the foundation during testing. The distance from the lateral loading point to the gage was multiplied by the rotation angle to determine the amount of lateral deflection due to rocking of the foundation. Only the envelope was plotted. A maximum lateral displacement value of 0.015 inch was obtained, which is insignificant compared with the deflectiondeflections which were applied to the column.
Chapter 4

Analysis of Specimens and Comparison to Observed Test Results

4.1 Introduction

This chapter describes the analysis of the test specimens and compares the analytical results to the behavior observed in laboratory testing. Flexural strengths, shear strengths, horizontal deflections, and column rotations relative to the foundations are discussed. Comments concerning the actual behavior versus the calculated behavior are also presented.

Flexural analyses of the hinges rotating about their strong axis were performed using a moment-curvature program called IAIUNR. This program calculates the moments required to cause yielding in different layers of steel, along with the corresponding column curvatures using standard techniques. The ultimate moment and curvature were determined at the crushing point of concrete, assumed at an ultimate strain of 0.004 on the extreme compression fiber of the concrete.

Several formulas from the American Concrete Institute (ACI) Building Code, commonly called ACI-318, were used to calculate the shear capacity of the hinged specimens. The formulas used include the shear-axial compression formulas (ACI-318 Eqs. 11-4, 11-7, and 11-8) and the shear-friction formula (ACI-318 Eq. 11-26). In addition, the empirical dowel action formulas discussed in Ref. 18 were also employed.

Deflection calculations were performed using the moment-area method, and included plastic deformation of the hinge throat and elastic deformation of the column. Deformations due to bond slip were calculated and added to the elastic deformation to obtain the total deflection. The total hinge rotation was determined by summing the elastic and inelastic hinge rotations with the rotation caused by bond slippage.

4.2 Flexural Analysis

Results of the flexural analysis performed on each of the four test specimens are presented in Tables 4-1 through 4-4. The load required to cause yielding of the various steel layers and the load at the point where the concrete reaches a compressive strain of 0.004 were calculated and are compared to the measured loads. It can be seen that the calculated and measured results were within 15 percent of each other.

The computer program IAIUNR was also used to calculate moment interaction diagrams and moment-curvature diagrams for the four test specimens. The program calculates the axial load-moment data points, so that an interaction diagram can be plotted, and calculates the moments required to initiate yielding in different layers of
steel, to produce a moment-curvature response. The input data consists of concrete and steel properties, section geometry, and applied axial load.

IAIUNR's concrete constitutive relationship is based on the Hognestad model. Figure 4-1 shows the Hognestad model for concrete; the idealized stress-strain curve consists of a parabolic and a linear segment. For the reinforcing steel, a tri-linear stress-strain relationship with an elastic branch, yield plateau, and a strain hardening branch is used. The program assumes that plane sections remain plane before and after bending, and that the stress-strain response for the concrete and the steel are known. IAIUNR computes bending moments, axial loads, and corresponding curvatures based on material properties, strain compatibility and force equilibrium.

Figures 4-2 and 4-3 show the interaction diagrams for test specimens SOIM and SD2M, respectively. The 26,000 pound axial load applied to each specimen is indicated on the figures. Measured concrete and steel properties were used in the analysis. From the interaction diagram for SD1M, Figure 4-2, the moment corresponding to the 26,000 pound load was 367,000 pound-inches. The maximum moment (based on the product of the measured load and the distance to the base) achieved during testing was 405,000 pound-inches. Figure 4-3 shows a theoretical moment for specimen SD2M of 375,000 pound-inches, compared to the actual moment of 419,000 pound-inches. In both cases, the measured moment was approximately 10 percent greater than the calculated moment, indicating that the specimens were stronger than the computer model indicated.

The interaction diagrams for specimens SOIC and SD2C are shown in Figures 4-4 and 4-5, respectively. At an axial load of 26,000 pounds, specimen SOIC has a corresponding nominal moment of 366,000 pound-inches. The measured ultimate moment for specimen SD2C was 461,000 pound-inches, approximately 30 percent greater than the 364,000 pound-inch calculated moment shown in Figure 4-5. Part of the difference is due to the fact that the peak loads were reached when the column was moving towards the actuator and the column was subjected to an effective axial load which exceeded 26,000 pounds.

Figures 4-6 through 4-9 show the moment-curvature diagrams for the four test specimens. All of these curves are similar in appearance, so Figure 4-6 will be used to describe the data. There are five distinct break points, labeled A through E. Point A is the concrete cracking point, which occurred at a moment of 124,000 pound-inches. The next three break points (B, C, and D) are the tensile yielding points for bars 1, 2, and 3, respectively. The ultimate point, E, corresponds to the crushing of concrete at a strain of 0.004 inch/inch.

In the testing of specimens SOIM and SD2M, bars 1 through 5 yielded in tension before the failure points of the specimens were reached. In order to produce tensile yielding in bars 1 through 4 in the computer model of IAIUNR, the ultimate strain of concrete was redefined from 0.004 to 0.10. Bar 5 would yield in tension only when the compressive strength of the concrete was raised to 10,000 psi. These results indicate that the hinge section geometry creates an "apparent" increase in the ultimate crushing strain.
and the compressive strength of the concrete. This is a result of concrete confinement in the hinge section.

4.3 Shear Analysis

Results from the shear strength calculations are presented in Table 4-5. The various formulas used to calculate the shear strength of the hinge section produce vastly different results.

The maximum shear applied to specimens SD1M and SD2M was 25,300 and 13,000 pounds, respectively, and 30,500 and 14,400 pounds for the cyclically-loaded specimens, SD1C and SD2C. Specimens SD1M and SD2M had lower shear capacity values because no horizontal ties were used in the column section.

The ACI-318 equations for shear-axial compression (Eqs. 11-4, 11-7, and 11-8), considerably underestimate the shear capacity of the hinge section. These low calculated values were expected, since a shear-axial compression failure was not anticipated nor present in any of the specimens. These formulas were included for completeness.

As discussed in Chapter 1, the shear-friction method (SFM) is currently used by designers to determine the ultimate shear capacity of the hinge section. For the shear-friction method, shear strengths were calculated for one bar contributing, two bars contributing, and all six bars contributing. The ACI-318 Commentary Sec. 11.7.7 states that it has "been demonstrated experimentally that if a resultant compressive force acts across a shear plane, the shear-transfer strength is a function of the sum of the resultant compressive force and the force $A_f f_c$ in the shear-friction reinforcement." Therefore, the force due to the axial load and the weight of the column section was added to the results obtained from ACI-318 Eq. 11-26. These results are presented in Table 4-5, and are labeled SFr1, SFr2, and SFr6; the numerals indicate the number of bars contributing to the axial force. A friction coefficient of one was used.

All four test specimens failed in flexure, not in a shear slip mode. However, the shear capacities were reached at failure in specimens SD1M and SD1C, since significant horizontal slippage occurred in the column section relative to the footing. The same argument cannot be made for specimens SD2M and SD2C, since relatively little slippage was noted.

Shear forces can also be developed by dowel action across the shear plane, once significant slippage has occurred. Figure 4-10 illustrates the three mechanisms for development of shear strength due to dowel action: flexure of the reinforcing steel, shear across the dowels, and "kinking" of the dowels. Kinking was not considered because kinking angles were not measured during testing. From Table 4-5, results indicate that dowel action through flexure considerably underestimated shear capacity and was most likely not the observed mode in which shear strength was developed. Dowel action from
shear was a more probable mechanism; however, this also underestimated the actual shear forces developed in the test specimens.

4.4 Rotation Analysis

Calculated rotation consisted of rotation due to elastic and plastic deformation of the hinge section plus rotation due to bar slippage. The calculated load-rotation response curves are presented in Figures 4-11 through 4-14. The calculated points are superimposed on the measured response curves.

Two empirical formulas were used in determining the rotations for the test specimens.\textsuperscript{11,23} Both Eqs. (4-2) and (4-3) attempt to compensate for the existence of so-called "shear effects," or effects of "plastic spread" which develop throughout the length of a concrete member during bending.\textsuperscript{11,24} Plastic spread begins occurring in a flexural member immediately after the concrete cracks, increasing the curvature and displacement of the member.

The first method to calculate the rotation of the column with respect to the base, caused by plastic deformation of the hinge throat, involved determining the yield curvature over the plastic hinge section. The rotation at yielding, $\theta_y$, of each bar layer is:

$$\theta_y = \phi_y l_y,$$

(4-1)

where $\phi_y$ is the curvature at yielding of each particular bar layer, calculated by the computer program $\text{LAIUNR}$, and $l_y$ is the equivalent length of the plastic hinge. The plastic hinge length can be determined using Baker's equation:\textsuperscript{11}

$$l_p = k_1 k_2 k_3 d \left( \frac{z}{d} \right)^{0.25},$$

(4-2)

where $k_1$ is equal to 0.7 for mild steel and 0.9 for cold-rolled reinforcing steel; $k_2$ is equal to $1+0.5(P_a/P_s)$, where $P_a$ is the axial compressive force applied to the member and $P_s$ is the member's axial compressive strength; $k_3$ is a factor which ranges from 0.9 for a concrete compressive strength, $f'_c$, of 1700 psi to 0.6 for an $f'_c$ of 5100 psi; $z$ is the distance from the critical section to the point of contraflexure; and $d$ is the effective depth of the section.

The additional curvature area developed from the plastic hinge length is added to the existing curvature diagram, shown in Figure 4-15, improving the accuracy of the calculated rotations.
The second method used was a formula developed by S. Sugano. This empirically-derived method determines a new yield curvature value for the member, based on certain geometric and material properties. The equation also considers the effects of diagonal tension on the member's yield displacement. With this new curvature, the corresponding yield rotations are computed using the moment-area method of Eq. (4-1). Sugano's formula for yield curvature, $\phi_y$, is:

$$\phi_y = \frac{M_y}{a_y E_c f_s}.$$  \hspace{1cm} (4-3)

where

$$M_y = d'' A_w f_s,$$

$$a_y = \left[ 0.043 + 1.64 \left( \frac{E_s}{E_c} \right) \left( \frac{A_w}{b h} \right) + 0.043 \left( \frac{d''}{h} \right) \left( \frac{d''}{h} \right)^2 \right].$$

In these equations, $M_y$ is the yield moment, $a_y$ is the stiffness reduction factor at yielding, $a$ is the depth of the rectangular compressive stress block, $A_w$ is the area of tensile steel, $b$ is the cross-sectional width, $d$ is the effective depth of the section, $d''$ is the distance between tension and compression steel, $E_s$ is the concrete modulus of elasticity, $E_c$ is the steel modulus of elasticity, $f_s$ is the yield strength of the tensile steel, and $h$ is the total depth of the section.

The bond-slip rotation was calculated with respect to the location of the neutral axis using the following relationship:

$$\theta_s = \frac{e}{d - kd'},$$  \hspace{1cm} (4-4)

where $d$ is the distance from the extreme compressive fiber to the centroid of the rebar and $e$ is the steel elongation due to bar slippage. $kd$ is the distance from the extreme compressive fiber to the neutral axis, and is calculated as:

$$kd = \frac{e_c}{\phi_y}.$$

The strain in the concrete at the extreme compressive fiber, $e_c$, is computed from strain compatibility for each layer of steel that is yielding. An ultimate concrete strain of 0.004 was used as the crushing point of concrete. Bar elongation due to bond slip, $e$, is computed using:
\[ e = \frac{f_y l_d}{2E_s} \]

where \( f_y \) is the steel yield stress, \( l_d \) is the development length of the reinforcement from ACI-318 Sec. 12.3.2, and \( E_s \) is the steel's modulus of elasticity.

Bar slippage in an ordinary reinforced concrete connection exists primarily in the column anchorage system, as shown in Figure 4-16.16 Bar slippage in a typical reinforced concrete beam section is very small due to the flexibility and cracking distribution of the element. However, in hinged specimens, there are two rigid concrete blocks that are rotating relative to each other with no crack development in either section; thus, bar slippage can take place in both portions, as shown in Figure 4-17. Bond-slip due to rotation can, therefore, be twice that of what would be anticipated in a normal reinforced concrete member.

4.4.1 Specimen SD1M

Figure 4-11 shows that the calculated load-rotation values for both Baker's and Sugano's methods overestimate the rotations until yielding of bar 1. At yielding of bars 2 and 3 the calculated values underestimate the actual rotation, most likely due to a reduction in stiffness, for which the calculation methods do not account. The ultimate rotations determined from Baker's and Sugano's formulas greatly underestimate the actual rotations. However, Sugano's formula approximates the actual rotations more closely than does Baker's.

4.4.2 Specimen SD2M

Figure 4-12 shows the load-rotation diagram for specimen SD2M. During small displacements, the measured curve shows a much higher hinge stiffness than the calculated curves. This stiffness retention is likely due to the aspect ratio of 2, in which the shearing effect on the hinge was not yet apparent. At yielding of bars 2 and 3, both Baker's and Sugano's formulas give good approximations of the measured response curve. In both of the monotonically loaded test specimens (SD1M and SD2M) the calculated ultimate rotations greatly underestimate the actual failure rotation.

4.4.3 Specimen SD1C

Figure 4-13 shows only the positive portion of the load-response curve envelope of Figure 3-54. In general, the load-response curves for Baker's and Sugano's methods fit the trend of the measured curve well. For the yielding of bars 1 and 3, the calculated results are very close to the cyclic response envelope; however, at yielding of bar 2 the calculated values do not as closely match the envelope. Both calculation methods underestimate the stiffness of the hinge envelope.
4.4.4 Specimen SD2C

Figure 4-14 shows the relationship between the calculated and measured load–rotation curves. As with specimen SD1C, the measured response curve is only the positive envelope of the load–rotation curve of Figure 3-75. The two methods overestimate the column rotation at yielding of bars 1 and 2. However, even though at yielding of bar 3 the methods again overestimate the rotation, the calculated values and the measured response are fairly close.

4.4.5 General Comments

All of the theoretical load–rotation response diagrams initially overestimate the column rotation, but overall correlation between the curves is good. In the early stages of loading there is only concentrated hinge rotation, with very little plastic rotation contributing to the overall column rotation. As loading proceeds, plastic rotation and bond-slipage effects are induced and very large rotations are experienced. This accounts for the calculated ultimate rotations greatly underestimating the actual failure rotations. In the theoretical calculations, crushing of concrete was assumed to occur at a strain of 0.004; however, all four test specimens exhibited a much higher ultimate strain. This appears to be the major reason for the differences between the measured and the calculated curves at ultimate rotation. It should also be noted that neither of the calculation methods were based on hinged specimens; they were used in this study to explore their applicability.

4.5 Deflection Analysis

The deflection components that contribute to column displacement are: deflection due to plastic deformation of the hinge throat, elastic deformation of the column, and deflection due to bond slip of reinforcing bars crossing the hinge throat. Calculated load–deflection curves are presented in Figures 4-18 through 4-21, and are superimposed on the measured load–deflection curves to examine the applicability of the analytical procedure.

To determine the yield displacements for each bar, Baker’s equation, Eq. (4-2), was used to determine the plastic hinge length, \( l_p \). The results from the computer program \( I41UNR \) were then used, in conjunction with \( l_p \), to calculate the yield displacements for bars 1 through 3 and the ultimate deflection at concrete crushing.

The model used in the moment-area calculations was a cantilever beam representing the column, with a fixed end as the foundation. The cantilever section consisted of a 6.5-inch by 16-inch element connected to the fixed base by a 2.5-inch by 16-inch by \( \frac{1}{2} \)-inch hinge section, as shown in Figure 4-22. The curvature varied linearly along the length of the element, from zero at the unsupported end to \( M/EI \) at the fixed
end. The moment of inertia, $I$, was based on the uncracked section and the modulus of elasticity, $E_c$, was calculated from:

$$E_c = 57,000 \sqrt{f_c}.$$  

The curvature over the plastic hinge length was calculated using the computer program IAUNR. This curvature, combined with Baker's equation, produced the deflection caused by inelastic deformation. The idealized curvature distribution for the test specimen is shown in Figure 4-15. Deflections were calculated at yielding of the different layers of steel, with the value of $M_y$ determined from IAUNR.

Deflection due to bond slip was calculated from the following equation:

$$\delta_s = \theta_s L,$$

where $\theta_s$ is the rotation due to bond slip determined from Eq. (4-4), and $L$ is the length measured from the support to the intersection of the hinge throat.

4.5.1 Specimen SD1M

Figure 4-18 shows the relationship between the measured and the calculated load-deflection diagrams. The calculated displacements at the different steel yielding locations underestimate the actual deformations, but Sugano's formula appears to approximate the curvature and the displacement response better than Baker's. The disparity between the calculated data and the observed results is partly a result of the horizontal slippage experienced during testing. (See Figure 3-17.) The horizontal slip causes a reduction in column stiffness, as indicated by the change in slope of the load-deflection diagram.

4.5.2 Specimen SD2M

Figure 4-19 shows the relationship between the calculated response and the measured deflection for this specimen. The calculated curves approximate the actual displacements fairly well up to the yielding of bars 1 through 3. At the concrete crushing point the calculated curves are much lower than the measured point, most likely because no horizontal ties were used in the column. The correlation between calculated and measured curves is much better for SD2M than for SD1M, probably due to SD2M's aspect ratio of 2.

4.5.3 Specimen SD1C

The load-deflection curve, Figure 4-20, shows the measured and calculated response for specimen SD1C. Only the cyclic deflection envelope has been plotted, not
the hysteresis loops. The calculated curve underestimates the displacement envelope, a result of the stiffness reduction caused by the large horizontal slippage and subsequent large lateral deformation. SD1C did have horizontal ties located in the pier section, which reduced the amount of stiffness loss in the specimen; this is apparent when Figure 4-18 and 4-20 are compared.

4.5.4 Specimen SD2C

Figure 4-21 shows the load–deflection envelope for specimen SD2C. The calculated deformation response using Baker’s method overestimates the deflection slightly; just the opposite from the first three test specimens.

4.5.5 General Comments

The Baker and Sugano procedures used to calculate the deformations underestimate the actual deflections. Agreement between the calculated and measured deflections is generally acceptable up to the yield point for bar 3, and is actually better than indicated by the load–deflection diagrams: a 0.10-inch gap between the hydraulic ram collar and the specimen resulted in measured displacements slightly larger than actual. Beyond this point, the specimens were able to sustain large strains which were well above the assumed ultimate concrete strain of 0.004; this is attributed to concrete confinement in the hinge throat.
Chapter 5
Comparison with Previous Testing

5.1 Introduction

The study presented in this report is part of a continuing study at the University of Nevada, Reno. The pilot study for this project, conducted in 1988, also considered four specimens. The variables in that study were the same as in the present study; however, there are several differences between the two projects. This chapter highlights those differences, and discusses how the new results relate to the conclusions of the first study.

5.2 Comparison of Test Specimens

The research performed in 1988 was a pilot study of reinforced concrete hinges subject to shear and flexure in the strong direction. Four one-eighth scale specimens were fabricated and tested in the laboratory. The reinforcing steel used in each specimen was Grade 40 plain No. 2 bars. Shear keyways measured 2 inches by 12 inches and had a roughness amplitude of approximately \( \frac{1}{4} \) inch. Two test variables were considered in the preliminary study: cyclic versus monotonic loading, and shear-span to depth ratio. Only one specimen was tested cyclically, CH4; this specimen had an aspect ratio of 3. The remaining specimens, CH1, CH2 and CH3, were loaded monotonically. CH1, CH2 and CH3 had shear-span to depth ratios of 3, 2, and 1, respectively, to simulate the Rose Creek Interchange, which has effective shear-span to depth ratios ranging from 1.2, for double-curvature column deformation, to 3, for single curvature deformation.

The objective of the current study was to construct and test four one-sixth scale model bridge pier specimens. All specimens were subjected to the simultaneous effects of axial load, shear, and flexure. The reinforcing steel used in each specimen was Grade 60 deformed No. 3 bars. The shear keys incorporated in the models measured 2.5 inches by 16 inches and had a roughness amplitude of \( \frac{1}{2} \) inch.

The same test variables were considered as in the previous study: cyclic versus monotonic loading, and shear-span to depth ratio. Only two aspect ratio were considered in this study, 1 and 2, because aspect ratios of 2 and 3 gave nearly the same results in the pilot study. These two ratios still provided the ability to monitor the behavior of a short, stiff column and a taller, more flexible column. For each shear-span to depth ratio, one specimen was subjected to monotonic loading and another subjected to cyclic loading.
Specifically, the differences between the previous study and the current research are the following:

1. One-sixth scale models were used in the present SD (Standard Detail) specimen series as opposed to one-eighth scale used in the original CH (Concrete Hinge) specimens. In the SD series, specimen size was increased to better represent an actual bridge pier. This increase in size, however, is not expected to significantly alter the behavior of the column.

2. Deformed (ribbed) reinforcing bars were used in the SD specimens, whereas plain bars were used in the original CH series. Deformed bars were used to enhance the bond between concrete and steel, and minimize crack widths at the hinge throat. Plain bars were used in the original specimens because deformed No. 2 bars were not available.

3. The original CH specimens consisted of two "column" elements joined together with a large "footing" block. The specimens were treated as beams for testing, with the columns simply-supported and the lateral load applied vertically to the footing section. The SD specimens were tested in an upright position with the lateral load applied horizontally to the column.

4. An axial load was applied to each of the SD specimens to simulate the effects of dead load from the bridge deck. The CH specimens had no applied axial load. An axial load will tend to enhance the shear resistance of a column, and creates a more realistic model.

5.3 Failure Modes

5.3.1 Original Concrete Hinge Series

Specimen CH1 had a shear-span to depth ratio of 3 and was loaded monotonically. An ultimate lateral load of 4200 pounds was obtained. Failure was caused by flexure and not by shear slip.

Specimen CH2, which had an aspect ratio of 2, was also loaded monotonically until failure. CH2 withstood a peak lateral load of 6000 pounds. CH2 also failed in flexure, as expected for a large shear-span to depth ratio. Neither CH1 nor CH2 showed any sign of cracking in the column, outside of the hinge area.

The last specimen loaded monotonically was CH3, which had an aspect ratio of 1. The ultimate load was 12,500 pounds, approximately twice that of CH2. CH3 exhibited a rapid strength deterioration after the peak load was reached, indicating a shear-slip failure immediately after the steel reinforcement yielded.
Specimen CH4 was loaded cyclically. This specimen had an aspect ratio of 3, as did CH1. The ultimate load for CH4 was 3605 pounds, after being subjected to five cycles of increasing lateral displacement. The maximum displacement ductility level achieved was 5. Failure was dominated by flexure.

5.3.2 Standard Detail Series

Test specimen SD1M was loaded monotonically until failure. The ultimate load was 25,300 pounds. Failure was caused by shear, as indicated by the rapid decrease in lateral load resistance. This was preceded by yielding of five of the bars in tension.

SD2M had an aspect ratio of 2, and was also loaded monotonically. The peak load achieved during testing was 13,100 pounds. A gradual decline in lateral load was observed, indicating a flexural failure.

Test specimens SD1C and SD2C were both loaded cyclically. SD1C withstood nine complete cycles, with a maximum ductility factor of 4.0. The ultimate load was 28,200 pounds, achieved on the fifth cycle. At the higher amplitude levels there was considerable pinching of the hysteresis loops, indicating rapid stiffness deterioration. The failure of specimen SD1C was caused by excessive flexural deformation.

Specimen SD2C was subjected to ten and one-quarter cycles of increasing amplitude displacements. SD2C achieved a maximum ductility level of 5.32 on the last quarter-cycle of testing. The peak load occurred on the eighth cycle and was 14,400 pounds. SD2C also failed in flexure. At higher levels of displacement amplitudes, pinching was present in the hysteresis loops, indicating a reduction in the energy absorbing capability of the hinge.

5.4 Scale Effect

The scale difference between the previous study (CH group) and this study created minimal performance differences. With the increase in size, SD specimens were able to withstand larger lateral loads before failure. Comparison of the test results, however, indicates that the scale had an insignificant influence on the performance of the specimens. The size contributed only to the ability to sustain larger loads.

5.5 Effects of the use of Deformed Bars

In the original CH series the steel reinforcement consisted of six No. 2 Grade 40 plain bars placed in a single row in the direction of bending. In the SD group, the steel crossing the hinge throat consisted of a single row of six No. 3 Grade 60 deformed bars.
The use of deformed bars in the SD specimens enhanced the bond between concrete and steel. Additionally, deformed bars helped reduce the crack widths across the hinge throat.
Chapter 6
Summary and Conclusions

6.1 Summary

This report presents the results of a study conducted on one-way reinforced concrete pier hinges subjected to a combination of axial load and uniaxial moment and shear in the strong direction. This type of hinge is typically used as the connecting link between the foundation and the columns of highway bridges. This “pinned” connection detail is typically known as a Freyssinet-type reinforced concrete hinge, and allows rotation in the weak direction while providing resistance to bending in the strong direction. The scope of this study was to construct and test four one-sixth scale hinge models. There were two primary variables in the testing sequence: shear-span to depth ratio (aspect ratio), and monotonic versus cyclic loading.

The first two specimens, with shear-span to depth ratios of 1 and 2, respectively, were loaded monotonically to failure. The last two specimens, also with aspect ratios of 1 and 2, were subjected to cyclic loading until failure. The purpose of the cyclic loading was to determine the effects on shear stiffness of the hinge. The use of various aspect ratios was to determine a limiting shear-span to depth ratio that would produce a shear failure. By reducing the shear span, the effect of shear failure was significantly increased.

Analysis of the hinged specimens involved determining flexural and shear strengths, concentrated hinge rotations, and displacement of the column elements. Flexural analysis was performed using a moment-curvature program called IAIUNR. A comparison between the measured and the calculated yield and failure loads were presented for each specimen. The various shear equations that were used to determine shear capacities, and their relative accuracies, were presented. Shear strengths of the hinges due to dowel action, in both flexure and shear, were also examined.

Concentrated hinge rotation and column deflection consisted of two components: reinforcement bond slippage and column flexure. Flexural displacements were determined from the curvature distribution along the column and include elastic deformation of the column and plastic deformation of the hinge. Empirical formulas used to estimate the rotational and deformation effects were also discussed.

6.2 Observations

During testing and analysis, the following observations were made:
1. Cyclic loading of specimen SD1C and SD2C showed that slow load reversals reduce the stiffness of the hinge region, until closure of the crack on the compression side of the column is achieved.

2. The absence of horizontal ties in the column section of SD1M and SD2M reduced the shear and flexural strength of the specimens after some of the bars yielded.

3. Shear resistance is controlled mainly by friction forces within the compression zone of the hinge throat, and not along the entire length of the hinge as the shear-friction method assumes.

4. Significant flexural deformations occurred in all four specimens during testing. Even in specimens SD1M and SD1C, which had aspect ratios of 1, appreciable ductility was apparent.

5. The maximum applied shear was developed in specimen SD1C, which had a shear-span to depth ratio of 1 and was cyclically loaded until failure. This, however, did not determine the limiting shear-span to depth ratio of the specimens tested, since the maximum shear was obtained after the specimen had yielded in flexure. The shear span must be further reduced to obtain shear slip failure.

6. The moment-curvature program IAIUNR produced a good correlation between the theoretical flexural capacities of the specimens and the actual measured values.

7. In all four specimens, the engagement of the shear key provided extra shear strength and ductility at large displacements.

8. The presence of a constant axial load applied to the specimens reduced the crack width and increased the apparent shear strength.

9. The use of deformed bars reduced bond slip and reduced crack width at the hinge throat.

10. A gap of approximately 0.10 inch between the hydraulic ram collar and the test specimen led to larger displacements in the load-deflection curves than actually existed. The correlation between the actual and calculated yield displacements improves when this gap is taken into account.

6.3 Conclusions

The shear-friction method did not produce reasonable estimates of shear capacity in the hinged sections. From the test results, primary shear resistance was developed only in the compression zone of the hinge interface. Further research should be conducted to develop an accurate method for determining the shear capacity of hinges.
The effects of dowel action (primarily developed from flexure and/or shear) appeared only after noticeable deformations had occurred. The primary resistance to shear in the hinge for small deflections was the friction force created by aggregate interlock. As the deflections became significantly greater, crack propagation became increasingly larger, and primary shear resistance was generated from reinforcing steel dowel action.

As the shear-span to depth ratio (aspect ratio) was decreased, the energy dissipation capacity of the hinge also decreased. The ductility levels for specimens SD1C and SD2C were 4.0 and 5.3, respectively. Both specimens exhibited considerable hysteresis pinching. Hinge detail modifications to increase both the ductility and the energy absorbing capability of the connection during cyclic loading need to be explored.
References


Table 4-1. Measured and Computed Yield Loads for Specimen SD1M.

<table>
<thead>
<tr>
<th>Layer of Steel</th>
<th>Measured Load (lbs.)</th>
<th>Calculated Load (lbs.)</th>
<th>Measured/Calculated</th>
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Table 4-2. Measured and Computed Yield Loads for Specimen SD2M.

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Table 4-3. Measured and Computed Yield Loads for Specimen SD1C.

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Table 4-4. Measured and Computed Yield Loads for Specimen SD2C.

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Table 4-5. Shear Strength Analysis based on Different Methods.

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<th>Method</th>
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</tr>
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<td>SFr6 Shear Friction Method, 6 bars</td>
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<td>Maximum Applied Shear, Measured in Laboratory</td>
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Figure 3-6. Load-Strain Diagram for SD1M, Bar 1 (Avg. of Gages 5 and 6).
Figure 3-7. Load–Strain Diagram for SD1M, Bar 2 (Avg. of Gages 7 and 8).

Figure 3-8. Load–Strain Diagram for SD1M, Bar 2 (Avg. of Gages 9 and 10).
Figure 3-9. Load–Strain Diagram for SD1M, Bar 3 (Avg. of Gages 11 and 12).

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Figure 3-12. Load–Strain Diagram for SD1M, Bar 5 (Avg. of Gages 17 and 18).
Figure 3-13. Load–Strain Diagram for SD1M, Bar 6 (Avg. of Gages 19 and 20).

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Figure 3-16. Load–Rotation Diagram for Specimen SDIM.
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Figure 3-19. Load–Column Twist Diagram for Specimen SD1M.

Figure 3-20. Load–Deflection Diagram for Specimen SD2M.
**Figure 3-21.** Cracking Pattern of Specimen SD2M.

**Figure 3-22.** Load-Strain Diagram for SD2M, Bar 1 (Avg. of Gages 1 and 2).
Figure 3-23. Load-Strain Diagram for SD2M, Bar 1 (Avg. of Gages 3 and 4).

Figure 3-24. Load-Strain Diagram for SD2M, Bar 1 (Avg. of Gages 5 and 6).
Figure 3-25. Load–Strain Diagram for SD2M, Bar 2 (Avg. of Gages 7 and 8).

Figure 3-26. Load–Strain Diagram for SD2M, Bar 2 (Avg. of Gages 9 and 10).
Figure 3-27. Load–Strain Diagram for SD2M, Bar 3 (Avg. of Gages 11 and 12).

Figure 3-28. Load–Strain Diagram for SD2M, Bar 4 (Avg. of Gages 13 and 14).
Figure 3-29. Load-Strain Diagram for SD2M, Bar 5 (Avg. of Gages 15 and 16).

Figure 3-30. Load-Strain Diagram for SD2M, Bar 5 (Avg. of Gages 17 and 18).
Figure 3-31. Load–Strain Diagram for SD2M, Bar 6 (Avg. of Gages 19 and 20).

Figure 3-32. Load–Strain Diagram for SD2M, Bar 6 (Avg. of Gages 21 and 22).
Figure 3-33. Load–Strain Diagram for SD2M, Bar 6 (Avg. of Gages 23 and 24).

Figure 3-34. Load–Rotation Diagram for Specimen SD2M.
Figure 3-35. Load–Horizontal Slip Diagram for Specimen SD2M.

Figure 3-36. Axial Load–Deformation Diagram for Specimen SD2M.
Figure 3-37. Load-Column Twist Diagram for Specimen SD2M.

Figure 3-38. Cyclic Loading for Test Specimen SD1C.
Figure 3-39. Load-Deflection Diagram for Specimen SD1C.

Figure 3-40. Cracking Pattern of Specimen SD1C.
Figure 3-41. Failure of SD1C Bar Number 1 in Tension.

Figure 3-42. Load–Strain Diagram for SD1C, Bar 1 (Avg. of Gages 1 and 2).
Figure 3-43. Load–Strain Diagram for SD1C, Bar 2 (Avg. of Gages 3 and 4).

Figure 3-44. Load–Strain Diagram for SD1C, Bar 3 (Avg. of Gages 5 and 6).
Figure 3-45. Load-Strain Diagram for SDIC, Bar 4 (Avg. of Gages 7 and 8).

Figure 3-46. Load-Strain Diagram for SDIC, Bar 5 (Avg. of Gages 9 and 10).
Figure 3-47. Load–Strain Diagram for SD1C, Bar 6 (Avg. of Gages 11 and 12).

Figure 3-48. Load–Strain Diagram for SD1C, Bar 1 (Gage 13).
Figure 3-49. Load–Strain Diagram for SD1C, Bar 2 (Gage 14).

Figure 3-50. Load–Strain Diagram for SD1C, Bar 3 (Gage 15).
**Figure 3-51.** Load–Strain Diagram for SD1C, Bar 4 (Gage 16).

**Figure 3-52.** Load–Strain Diagram for SD1C, Bar 5 (Gage 17).
Figure 3-53. Load–Strain Diagram for SD1C, Bar 6 (Gage 18).

Figure 3-54. Load–Rotation Diagram for Specimen SD1C.
Figure 3-55. Load-Horizontal Slip Diagram for Specimen SD1C.

Figure 3-56. Axial Load-Deformation Diagram for Specimen SD1C.
Figure 3-57. Load-Column Twist Diagram for Specimen SD1C.

Figure 3-58. Load-Base Rotation Contributing to Total Deflection for SD1C.
Figure 3-59. Cyclic Loading for Test Specimen SD2C.

Figure 3-60. Load-Deflection Diagram for Specimen SD2C.
Figure 3-61. Cracking Pattern of Specimen SD2C.

Figure 3-62. Load-Strain Diagram for SD2C, Bar 1 (Avg. of Gages 1 and 2).
Figure 3-63. Load–Strain Diagram for SD2C, Bar 2 (Avg. of Gages 3 and 4).

Figure 3-64. Load–Strain Diagram for SD2C, Bar 3 (Avg. of Gages 5 and 6).
Figure 3-65. Load–Strain Diagram for SD2C, Bar 4 (Avg. of Gages 7 and 8).

Figure 3-66. Load–Strain Diagram for SD2C, Bar 5 (Avg. of Gages 9 and 10).
Figure 3-67. Load–Strain Diagram for SD2C, Bar 6 (Avg. of Gages 11 and 12).

Figure 3-68. Load–Strain Diagram for SD2C, Bar 1 (Gage 13).
Figure 3-69. Load-Strain Diagram for SD2C, Bar 2 (Gage 14).

Figure 3-70. Load-Strain Diagram for SD2C, Bar 3 (Gage 15).
Figure 3-71. Load–Strain Diagram for SD2C, Bar 4 (Gage 16).

Figure 3-72. Load–Strain Diagram for SD2C, Bar 5 (Gage 17).
Figure 3-73. Load–Strain Diagram for SD2C, Bar 6 (Gage 18).

Figure 3-74. Load–Rotation Diagram for Specimen SD2C.
Figure 3-75. Load-Horizontal Slip Diagram for Specimen SD2C.

Figure 3-76. Axial Load-Deformation Diagram for Specimen SD2C.
Figure 3-77. Load–Base Rotation Contributing to Total Deflection for SD2C.

Figure 4-1. Idealized Stress-Strain Curve for Concrete.
Figure 4-2. Axial Load-Moment Interaction Diagram for Specimen SD1M.

Figure 4-3. Axial Load-Moment Interaction Diagram for Specimen SD2M.
Figure 4-4. Axial Load-Moment Interaction Diagram for Specimen SD1C.

Figure 4-5. Axial Load-Moment Interaction Diagram for Specimen SD2C.
Figure 4-6. Moment-Curvature Diagram for Specimen SD1M.

Figure 4-7. Moment-Curvature Diagram for Specimen SD2M.
Figure 4-8. Moment-Curvature Diagram for Specimen SD1C.

Figure 4-9. Moment-Curvature Diagram for Specimen SD2C.
Figure 4-10. Mechanisms for Dowel Action.
Figure 4-11. Measured versus Calculated Load–Rotation Diagrams for SD1M.

Figure 4-12. Measured versus Calculated Load–Rotation Diagrams for SD2M.
Figure 4-13. Measured versus Calculated Load-Rotation Diagrams for SD1C.

Figure 4-14. Measured versus Calculated Load-Rotation Diagrams for SD2C.
Figure 4-15. Idealized Curvature Distribution Along Cantilevered Element.

Figure 4-16. Bar Slip Mechanism in a Typical Reinforced Concrete Beam.
Figure 4-17. Assumed Bar Slip Mechanism in Hinged Specimens.

Figure 4-18. Measured versus Calculated Load-Deflection Diagrams for SD1M.
Figure 4-19. Measured versus Calculated Load-Deflection Diagrams for SD2M.

Figure 4-20. Measured versus Calculated Load-Deflection Diagrams for SD1C.
Figure 4-21. Measured versus Calculated Load-Deflection Diagrams for SD2C.

Figure 4-22. Cantilevered End Element.
Appendix A

Aggregate Properties

Coarse Aggregate:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum Size</td>
<td>½ inch</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.52</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>0.78%</td>
</tr>
<tr>
<td>Absorption Capacity</td>
<td>2.52%</td>
</tr>
<tr>
<td>Dry Rodded Weight</td>
<td>110 lb/ft³</td>
</tr>
</tbody>
</table>

Fine Aggregate:

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specific Gravity</td>
<td>2.52</td>
</tr>
<tr>
<td>Moisture Content</td>
<td>7.95%</td>
</tr>
<tr>
<td>Absorption Capacity</td>
<td>3.08%</td>
</tr>
<tr>
<td>Fineness Modulus</td>
<td>3.16</td>
</tr>
</tbody>
</table>
Figure A-1. Grading Chart for Coarse Aggregate.

Figure A-2. Grading Chart for Fine Aggregate.
Appendix B

Concrete Mixture Design

Batch weights to make one cubic yard of concrete:

<table>
<thead>
<tr>
<th>Component</th>
<th>Weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water/Cement Ratio</td>
<td>0.40</td>
</tr>
<tr>
<td>Cement</td>
<td>738 lb/yd³</td>
</tr>
<tr>
<td>Water</td>
<td>295 lb/yd³</td>
</tr>
<tr>
<td>Coarse Aggregate</td>
<td>1377 lb/yd³</td>
</tr>
<tr>
<td>Fine Aggregate</td>
<td>1195 lb/yd³</td>
</tr>
<tr>
<td>Air Entraining Admixture</td>
<td>1 fl. oz. per 100 lbs. of cement</td>
</tr>
</tbody>
</table>

The concrete mixture was proportioned using the PCA Absolute Volume Method.
### Appendix C

**Concrete Properties**

<table>
<thead>
<tr>
<th></th>
<th>Compressive Strength, $f'_c$</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7-Days</td>
<td>28-Days</td>
<td>Day of Test</td>
<td>Slump</td>
</tr>
<tr>
<td><strong>SD1M</strong></td>
<td>Column</td>
<td>3750 psi</td>
<td>4070 psi</td>
<td>4690 psi</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>3540 psi</td>
<td>3710 psi</td>
<td>4600 psi</td>
</tr>
<tr>
<td><strong>SD2M</strong></td>
<td>Column</td>
<td>4050 psi</td>
<td>4690 psi</td>
<td>5140 psi</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>4620 psi</td>
<td>4630 psi</td>
<td>4900 psi</td>
</tr>
<tr>
<td><strong>SD1C</strong></td>
<td>Column</td>
<td>3550 psi</td>
<td>4330 psi</td>
<td>4440 psi</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>4100 psi</td>
<td>4610 psi</td>
<td>4950 psi</td>
</tr>
<tr>
<td><strong>SD2C</strong></td>
<td>Column</td>
<td>2830 psi</td>
<td>4070 psi</td>
<td>4420 psi</td>
</tr>
<tr>
<td></td>
<td>Footing</td>
<td>3910 psi</td>
<td>4560 psi</td>
<td>4790 psi</td>
</tr>
</tbody>
</table>
Appendix D

Reinforcing Steel Properties

Bars: #3 Grade 60, Deformed
Diameter: 0.375 in.
Area: 0.11 in\(^2\)
Modulus of Elasticity: 29,000 ksi

<table>
<thead>
<tr>
<th>SDIM &amp; SD2M</th>
<th>SD1C &amp; SD2C</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield Strength:</td>
<td>58 ksi</td>
</tr>
<tr>
<td>Yield Strain:</td>
<td>0.002</td>
</tr>
<tr>
<td>Ultimate Strength:</td>
<td>83 ksi</td>
</tr>
<tr>
<td>Ultimate Strain:</td>
<td>0.232</td>
</tr>
</tbody>
</table>
Figure D-1. Reinforcing Steel Stress-Strain Diagram for SD1M and SD2M.

Figure D-2. Reinforcing Steel Stress-Strain Diagram for SD1C and SD2C.
Appendix E
Listing of PIERHINGE Computer Program

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Computer Program "PIERHINGE"
Version 1.0
By David L. Straw
Civil Engineering Department
University of Nevada, Reno
March 1988

This computer program was written by David L. Straw by modifying
computer program "AQUIREDAT" written by James L. Orii [1].
The program was written to acquire data from testing of one-sixth
scale model bridge pier-to-foundation connections (one-way hinges)
used in the Strong Earthquake during the spring of 1989. The computer
program is written in Hewlett-Packard Basic Version 4.0.

This program requires that the following HP equipment be turned on:
347A System Voltmeter, 3456A Digital Voltmeter, 3407A Data
Acquisition System/Control Unit, Hp Thinkjet Printer, and HP 7470A
External Plotter.

Data is acquired using electronic resistance strain gages,
external LVDT's, and the MTS Structural Testing Machine (load
cells and ram arm).
The strain gages are located on reinforcing bars within the concrete
test specimens. 3-external LVDT's are used to measure the following:
(1) deflection of the pier relative to the foundation as the
specimen is being loaded (to monitor the rotation of the column) and
(2) slip of the column relative to foundation.
The MTS was used to load the R/C test specimen and the following
was recorded: (1) load from the load cell and (2) center span
displacement of the hydraulic ram.
The HP 7470A external plotter plotted lateral load vs. center
span deflection of the column during testing.

---

DIN Volt(300), V(300), Chan(300), Gf(300), C1vld(300), Gif(300), Disp(300)
DIN Loadp(300), Defi(300), Sp(300), F(I300), On(300), Disp(300)
DIN Title(300), Tit(300)
DIN U_ratio(300), S_ratio(300)

L=1 COUNTER FOR TESTING
D=1 COUNTER FOR FILE1
D=2 COUNTER FOR FILE2

Load=0

PRINTER IS 1
Gage="gage"

TWO DATA FILES ARE CREATED FOR STORING TEST RESULT DATA.
FILE 1 IS THE STRAIN REFERENCE DATA AND FILE 2 IS THE STRAIN GAGE
AND FINALLY THE LATERAL LOAD FORCE.
OUTPUT KBD USING "#1",CHR$(255)$"K"
INPUT "INPUT FILE NAME FOR REFERENCE DATA", File1$
PRINT "INPUT FILE NAME FOR OUTPUT DATA FILE", File2$ 
PRINT "ENTER THE TITLE OF THIS EXPERIMENT", Title$ 
PRINT "IS THIS THE READING TO BE USED AS A REFERENCE POINT FOR FUTURE MEASUREMENTS (Y/N)?" 
INPUT Rp$ 
INPUT THE NUMBER OF STRAIN GAGES USED IN EACH SPECIMEN 
INPUT THE NUMBER OF EXTERNAL LVDT'S USED 
IF Rp$="Y" THEN GOTO 880 
I SETTING THE TIME AND DATE 
I ------------------------------- 
OUTPUT KBD USING "#,K",CHR$(255),"K" 
INPUT "DO YOU WISH TO SET THE TIME AND DATE (Y/N)?",Set$ 
IF Set$="N" THEN GOTO 720 
INPUT "ENTER THE TIME OF DAY (HR:MIN:SEC)",Td$ 
INPUT "ENTER THE DATE (DAY MONTH YEAR)",Dd$ 
SET TIMEDATE DATE(Dd$) 
SET TIME TIME(Td$) 
PRINT "TIME=",TIME$(TIMEDATE) 
PRINT "DATE=",DATE$(TIMEDATE) 
OUTPUT KBD USING "#,K",CHR$(255),"K" 
CALL Plotter_1(Title$,X$,Y$,Minx,Maxx,Miny,Maxy) | INPUT GRAPH DATA FOR EXTERNAL HP PLOTTER 
CALL Printerin(Lowx,Higx,Lowy,Higy,X ticl,X tic2,Y ticl,Y tic2,Spacex,Spacey) 
I INPUT GRAPH DATA FOR SCREEN PLOT 
1 | RETRIEVING OLD REFERENCE VALUES 
| ------------------------------- 
ASSIGN @Path1 TO File1$ 
FOR I=1 TO Mg 
ENTER @Path1, D1;U_ratio(I) 
D1=Di+1 
NEXT I 
ASSIGN @Path1 TO * 
CREATE BDAT File2$,6900,8 
ASSIGN @Path2 TO File2$ 
COM /hp3054/ Scn, Dm, Sw, Prt, Error, Err$[8],Err$t[15] 
| ------------------------------- 
SUBROUTINE TO ASSIGN CHANNELS TO THE STRAIN GAGES 
THE STRAIN GAGE READINGS WILL BE ASSIGNED AUTOMATICALLY TO CHANNELS BY THE COMPUTER PROGRAM. 
| ------------------------------- 
C1=20 
C2=40 
D4=60 
FOR I=1 TO 8 
Chan(I)=C1 
6f(I)=2.045 
C1=C1+1 
NEXT I
FOR K=9 TO 16
CHAN(K)=C3
G(K)=2.845
C3=C3+1
NEXT K
FOR J=17 TO 18
CHAN(J)=C4
G(J)=2.845
C4=C4+1
NEXT J
SUBROUTINE TO ASSIGN CHANNELS TO THE LVDT's.
CHANNELS 2-4 WILL AUTOMATICALLY BE ASSIGNED TO THE EXTERNAL LVDT's.
**********************************************************************
C2=2
FOR J=1 TO NUN
Cf(1)=10.091
Cf(2)=10.019
Cf(3)=26.773
Clbd(J)=C2
C2=C2+1
NEXT J
SUBROUTINE TO ASSIGN CHANNELS TO THE HTS LOAD CELL AND RAM ARM
TO MONITOR CENTER SPAN DEFLECTION.
**********************************************************************
Clrd=1
HTS LOAD CELL IS CHANNEL No.1
Cld=5.0
(+-) 10 VOLTS = (+-) 50 KIPS CALIBRATION FACTOR
Cd=0
HTS LVDT IS CHANNEL No.0
Cd=5.3
(+-) 10 VOLTS = (+-) 3 INCHES CALIBRATION FACTOR
**********************************************************************
OUTPUT KBD USING "&K\CHR(255)K" CLEAR CRT
CALL Init(3456)
IF R="N" THEN GOTO 1580
Get Bridge voltage, Excitation voltage, and Ratio for reference gages
FOR K=1 TO Ng
CALL Bmst(Chn(K),Bridge,Excitation,Ratio) BRIDGE MEAS.
U_ratio(K)=Ratio
V(K)=Excitation
PRINTER IS 701
PRINT "REFERENCE RATIO: ";K;U_ratio(K)
PRINT "REFERENCE EXCITATION: ";V(K)
NEXT K
WAIT 1
OUTPUT KBD USING "&K\CHR(255)K" CLEAR CRT
**********************************************************************
NOW STORING REFERENCE DATA
**********************************************************************
CREA T F ile1\50.8
ASSIGN @Pathl TO File1\8
CALL Detsor(@Path1,Ol,Ng,U_ratio(*))
SUBROUTINE TO DETERMINE LATERAL LOAD AND CENTER SPAN DEFLECTION FROM MTS

FOR I=1 TO Ng
CALL BrReatChen(I),Brige,Excllation.Ratio)  // Bridge Measurement
SF_ratio(I)=Ratio              // Ratio of Bridge to Excllation Voltage
P(I)=FSstrain(I),S_ratio(I),U_ratio(I))  // Determines strain
IF Ln=1 THEN GOTO 1680
Sp(Ln,J)=P(I)-Sp(1,J)
GOTO 1640
GOTO 1680
NEXT J

PRINTING STRAIN GAGE VALUES

FOR I=1 TO Ng
IF Ln>1 THEN 60 TO 1790
Loadp(Ln)=Loadp(Ln)-Loadp(I)
Load=Loadp(Ln)
Displ=FN0cv(Cdist)
IF Ln=1 THEN GOTO 1630
Displ=Displ-Defl(Ln)
Displ=Defl(Ln)-Displ*CFdist
Plotval=Defl(Ln)
FOR K=1 TO Num
SUBROUTINE TO PRINT RESULTS ON HP THINK JET PRINTER
CALL Dataprint(Ng,P(*),Chen(*),Load,Time(*),Data(*),Title,Ln,Plotval)
FOR I=1 TO Ng
SUBROUTINE TO DETERMINE LVDT DISPLACEMENTS
CALL Lvdt(I,Clvdth(I),Cf(I),Volt(I),Displ(I))
NEXT I
NEXT K
I  ******************************************************
I SUBROUTINE TO STORE TEST DATA ON COMPUTER HARD DRIVE
I ******************************************************
2870 CALL Datasets(Path2, D2, Ln, Load, Plotval, Ng, Num, P(*), Dm(*))
2880 PRINT "*********************************************************************************/
2890 PRINT
2900 IF AagS="N" THEN GOTO 2140
2910 CALL SUBROUTINE "Plotrplot": TO PLOT LOAD VS. CENTER SPAN DEFORMATION
2920 ON EXTERNAL PLOTTER.
2930 CALL Plotrplot(Plotval, Load, Maxx, Mnya, Maxy, Def11, Load1)
2940 PRINT
2950 PRINT "DO YOU WANT TO SEE ANOTHER GRAPH ON THE SCREEN (Y/N)?"
2960 INPUT AagS
2970 IF AagS="N" THEN GOTO 2140
2980 CALL SUBROUTINE "Printplot": TO PLOT LOAD VS. STRAIN ON CRT SCREEN.
2990 CALL Printplot(Lowx, Hlx, Lowy, Hly, Xtic1, Ytic1, Ytic2, SpecX, Specy, Ln, Spc(*), Loadp(*))
3000 PRINT "DO YOU WANT TO SEE A GRAPH ON THE SCREEN (Y/N)?"
3010 INPUT RespS
3020 IF RespS="Y" THEN 2200
3030 PRINT
3040 Ln=Ln+1
3050 PRINT
3060 PRINT "*********************************************************************************/
3070 PRINT
3080 IF FgS="N" THEN 2450
3090 PRINT
3100 OUTPUT KBD USING "\$K":CHR$(255)&"K"
3110 PRINT
3120 PRINT "LOAD THE STRUCTURE AND PRESS (CONTINUE)"
3130 PAUSE
3140 GOTO 1560
3150 PRINT
3160 OUTPUT KBD USING "\$K":CHR$(255)&"K"
3170 PRINT "<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<"
3180 PRINT " WARNING: THIS WILL TERMINATE PROGRAM OPERATION. <" 
3190 PRINT " IS THIS WHAT YOU REALLY WANT TO DO (Y/N)?" <" 
3200 PRINT " <<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<<"
3210 INPUT AagS
3220 IF AagS="N" THEN 2300
3230 PRINT
3240 PRINT
2550 PRINT "END OF TESTING !!!!" 
2560 END 
2570 | SUBPROGRAM Datascreen TO STORE REFERENCE VALUES TO FILE #1. 
2580 | ------------------------------------------------------------- 
2590 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2600 FOR K=1 TO Ng 
2610 | OUTPUT Path1,Di,U_ratio(K) TO STORE REFERENCE VALUES 
2620 | D1=D1+1 
2630 | NEXT K 
2640 | SUBEND 
2650 | SUBPROGRAM Datascreen TO STORE REFERENCE VALUES TO FILE 
2660 | ------------------------------------------------------------- 
2670 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2680 PRINT "SUBPR06RNI Datascreen TO STORE REFERENCE VALUES TO FILE II. 
2690 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2700 PRINT "SUBPR06RNI Datascreen TO STORE REFERENCE VALUES TO FILE II. 
2710 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2720 FOR K=1 TO Ng 
2730 | STORE VALUES TO REFERENCE VALUES 
2740 | NEXT K 
2750 | SUBEND 
2760 | SUBPROGRAM Datascreen TO STORE REFERENCE VALUES TO FILE 
2770 | ------------------------------------------------------------- 
2780 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2790 PRINT "SUBPR06RNI Datascreen TO STORE REFERENCE VALUES TO FILE II. 
2800 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2810 FOR K=1 TO Ng 
2820 | STORE VALUES TO REFERENCE VALUES 
2830 | NEXT K 
2840 | SUBEND 
2850 | SUBPROGRAM Datascreen TO STORE REFERENCE VALUES TO FILE 
2860 | ------------------------------------------------------------- 
2870 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2880 PRINT "SUBPR06RNI Datascreen TO STORE REFERENCE VALUES TO FILE II. 
2890 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2900 FOR K=1 TO Ng 
2910 | STORE VALUES TO REFERENCE VALUES 
2920 | NEXT K 
2930 | SUBEND 
2940 | SUBPROGRAM Datascreen TO STORE REFERENCE VALUES TO FILE 
2950 | ------------------------------------------------------------- 
2960 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2970 PRINT "SUBPR06RNI Datascreen TO STORE REFERENCE VALUES TO FILE II. 
2980 | SUB Datascreen(Path1,Di,Ng,U_ratio(K)) 
2990 FOR K=1 TO Ng 
3000 | STORE VALUES TO REFERENCE VALUES 
3010 | NEXT K 
3020 | SUBEND 
3030 | SUBROUTINE F1R INITIALIZING THE VOLTMETERS 
3040 | ------------------------------------------------------------- 
3050 | SUBROUTINE F1R INITIALIZING THE VOLTMETERS 
3060 | ------------------------------------------------------------- 
3070 | SUBROUTINE F1R INITIALIZING THE VOLTMETERS 
3080 | ------------------------------------------------------------- 
3090 | SUBROUTINE F1R INITIALIZING THE VOLTMETERS 
3100 | -------------------------------------------------------------
3060 Prthere=e  I PRINTER FLAG
3070 Error=0  I ERROR CODE 0'S
3080 Error=x"00000000000000000000000000000000" I ERROR MASK
3090 Error=+”  I ERROR SUB NAME
3100 I INITIALIZE DEVICE ADDRESSES
3110 Scn=709  I 3497A SCANNER ADDRESS
3120 Dvm=722  I 3456A DIGITAL VOLTMEETER ADDRESS
3130 Svn=724  I 3437A SYSTEM VOLTMEETER ADDRESS
3140 Prt=761  I SYSTEM PRINTER ADDRESS
3150 Bus=Scn DIV 100  I HP19 SELECT CODE
3160 Initdvm=Dvm  I TEMP. STORAGE FOR HP3456 ADDRESS
3170 I VERIFY THAT INTERFACE IS HP-IB
3180 Message="THE NUMBER "$VAL$(Bus%) IS NOT A VALID SELECT CODE"
3190 IF Bus<7 OR Bus>31 THEN Terminate
3200 Message="NO INTERFACE FOUND AT SELECT CODE "$VAL$(Bus)
3210 ON ERROR 6010 Terminate
3220 STATUS Bus,88status
3230 Message="THE INTERFACE AT SELECT CODE "$VAL$(Bus%) IS NOT HP-IB"
3240 IF BitStatus<1 THEN Terminate
3250 ON TIMEOUT Bus,16010 Time
3260 CONTROL Bus,011  I CLEAR THE INTERFACE
3270 SEND Bus,111  I UNLISTEN THE BUS
3280 OFF TIMEOUT
3290 OFF ERROR
3300 CLEAR Bus  I SENDS DEVICE CLEAR (DCL)
3310 IF (Dvmeter<>3456) AND (Dvmeter<>3497) THEN Abort
3320 IF Dvmeter<>3497 THEN Dvm=Scn
3330 IF CHECK FOR EQUIPMENT ON BUS AT ALL ADDRESSES AND PRINT DEVICE NAMES
3340 OUTPUT 1 USING "0,/10X,K,00";" EQUIPMENT PRESENT ON BUS 8";Bus
3350 OUTPUT 1 USING "10X,";"-----------------------------";,"/
3360 FOR Address=Bus+100 TO Bus+100+30
3370 OUTPUT Address USING "8" I ADDRESS DEVICE TO LISTEN
3380 STATUS Bus,7;Bitstatus  I SEE IF IT LISTENED
3390 IF (NOT Bit(BitStatus,13)) THEN NXT  I BIT 13 TRUE IF DEVICE PRESENT
3400 Message=""  I Device Unknown"
3410 IF Address=Scn THEN Message="3497A Mainframe"
3420 IF Address=Initdvm THEN Message="3456A Digital Voltmeter"
3430 IF Address=Svn THEN Message="3437A System Voltmeter"
3440 IF (Dvm<>Scn) AND (Address<>Scn) THEN Message=Message&", DVM"
3450 IF Address=Prt THEN
3460 Message=""  I System Printer"
3470 Prthere=1
3480 END IF
3490 OUTPUT 1 USING Fmt,Messages,Address MOD 100
3500 NXT Address
3510 STATUS Bus,3;Bitstatus
3520 OUTPUT 1 USING Fmt,", System Computer",BINAND(Bitstatus,31)
3530 Fmt:IMAGE 6X,24A," at address ",22
3540 IF NOT Prthere THEN Prt=1
3550 SUBEXIT
3560 Time:CONTROL Bus,011  I RESET INTERFACE
Messages="NO DEVICES RESPOND OVER HP-I8 SELECT CODE "$VAL(Bus)"

3580 Terminate:OUTPUT 1 USING Fmt2:"CAUTION!","PROGRAM TERMINATED BECAUSE:"
3590 Fmt2:IMAGE $,5/.,K,2/.,K,/
3600 OUTPUT 1 USING "$X,K",******Message$
3610 STOP
3620 Abort:Error=2
3630  Error="Init"
3640 CALL Warn
3650 SUBEND
3660 |
3670  |SUBPROGRAM FOR RECORDING BRIDGE MEASUREMENT
3680 |
3690 SUB Brwme(Channel,Bridge,Excitation,Ratio) | 200 SERIES 02/02/83
3700 COM /Ha3054/ Scn,Dvm,Smv,Pr1,Error,Err(6),Error(15)
3710 Exc_chn=Channel
3720 Bridge=5.E+18
3730 Excitation=9.E+19
3740 Ratio=9.E+19
3750 OUTPUT Dvm USING "$"
3760 STATUS Dvm DIV 100,7(Here1
3770 Here=8192*(Scn<>0)
3780 IF Dvm<>Scn THEN Chk
3790 OUTPUT Scn USING "$".
3800 STATUS Dvm DIV 100,7(Here2
3810 Chk:Error=2*((Channel<6) OR (Channel>999) OR (Channel MOD 20)=10)
3820 Error=Error+16*((NOT BIT(Here2,13)) OR (NOT BIT(Here1,13)) AND (Dvm<>Scn) ),
3830 Error=Error+32*((NOT BIT(Here1,13)) AND (Dvm<>Scn))
3840 IF Error THEN Abort
3850 Read:OUTPUT Scn,"AC",INT(Exc_chn);"ST0" (CLOSE CHAN. & WAIT UNTIL EXECUTED
3860 IF Dvm<>Scn THEN OUTPUT Dvm;"VR1,VR5V11VA1VF1V0DSV0USV0UT3";13497 SETUP & TRG
3870 IF Dvm<>Scn THEN OUTPUT Dvm;"HR2RIST1G66T6T3";13456 SETUP & TRG
3880 ENTER Dvm=Excitation
3890 IF Channel<>Exc_chn THEN Done
3900 Bridge=Excitation
3910 Exc_chn=(Channel DIV 10)*10+10 (COMPUTE EXCITATION CHANNEL NUMBER
3920 60TO Read
3930 Done:IF ABLS(Bridge)<.119 THEN Error=128 I BRIDGE VOLT. NOT READ ON IV RANGE
3940 IF (ABS(Excitation)<.01) OR (ABS(Excitation)>5.4) THEN Error=Error+2561E XCIT. VOLT. TEST
3950 IF Error THEN Abort
3960 Ratio=Bridge/Excitation
3970 SUBEXIT
3980 Abort:Error="Brwme"
3990 CALL Warn
4000 SUBEND
4010 |
4020 |USER DEFINED FUNCTION FOR COMPUTING VALUES OF STRAIN.
4030 |
4040 DEF FNStrain(S,ratio,LR_ratio) | 200 SERIES
4050 COM /Ha3054/ Scn,Dvm,Smv,Pr1,Error,Err(6),Error(15)
4060 LR=S_ratio-UL_ratio
4070 Error=Error+128*(((LR<-.25) OR (LR>1) OR (S_ratio=9.E+19))
4080 |
122
4080 IF Error THEN Abort
4089 Strain=-(4*Diff)/(Ef*(1+2*Diff))  ! 1/4 BRIDGE EQUATION
4100 RETURN SGH(Strain)+INT(ABS(Strain)*1.E+6)+.5)  ! RETURN MICROSTRAIN
4110 AbortError="Strain"
4120 CALL Warn
4130 RETURN S.E+19
4140 END
4150 ! ******************************************************************************************
4160 ! SUBPROGRAM FOR DETECTING ERRORS
4170 ! ******************************************************************************************
4180 SUB Warn
4190 CON /Hp3054/ Scn,Dmv,Smv,Prt,Error,Err@[6],Err@[15]
4200 CON /Hp3054.warn/ Da@[14],Ern@[0:14]@[15],[Aster@[0:88]]
4210 INTEGER Max_warns,Flag,Index,Marrs,Status,Line_len,Gap
4220 Max_warns=2  ! MAXIMUM NUMBER OF WARNINGS ON EACH ERROR ( <=9 )
4230 OFF INTR  ! PREVENTS WARN FROM BEING EXITED EARLY
4240 Flag=0
4250 Marrs=0
4260 Line_len=80
4270 IF LEN(Err@[1])>15 THEN Err@="0000000000000000"  ! FILL Err@ IF NECESSARY
4280 FOR Index=0 TO 14.  ! ANY ERRORS THAT NEED WARNINGS?
4290 IF BIT(Error[Index]) AND (VAL(Err@[1],Index+1)<Max_warns) THEN
4300 Flag=Flag+2"Index
4310 NEXT Index
4320 IF Flag=0 THEN Exit  ! NO...RETURN
4330 READ Err@[1],Aster@  ! YES...DO SETUP
4340 IF Prt=1 THEN Line
4350 ON ERROR GOTO Noprt
4360 OUTPUT Prt USING "$"
4370 STATUS Prt DIV 100,7,Status
4380 IF BIT(STATUS@[13]) THEN Nocheck! PRINTER PRESENT
4390 Noprt=Prt=1.
4400 Line:=STATUS Prt,BLine_len
4410 Nocheck:=ERROR GOTO Nodate
4420 ON TIMEROUT Scn DIV 100,1 GOTO Nodate
4430 OUTPUT Scn USING "$"
4440 STATUS Scn DIV 100,7,Status
4450 IF NOT BIT(Status@[13]) THEN Nodate
4460 OUTPUT Scn:"TD"
4470 ENTER Scn:Da@
4480 IF Da="01:01:00:00:00:00" THEN Nodate! Sorry if it's Dec.21,23:59:59+1 sec
4490 Gap=INT(Line_len/30)/2
4500 OUTPUT Prt USING Fmt:Aster@,[Gap]:Da@[1,2],Da@[4,5],Da@[7,14],Aster@,[1, Gap]
4520 GOTO Warnout
4530 Nodate:=OFF TIMEROUT
4540 OFF ERROR
4550 OUTPUT Prt[Aster@[1],Line_len]
4560 Warnout:=OFF ERROR
4570 OFF TIMEROUT
4580 IF PRINT="WARNING: " SUBPROGRAM "$Err#" WAS NOT EXECUTED"
FOR Index=0 TO 14  
! IDENTIFY THE INDIVIDUAL ERRORS

IF (NOT BIT(Flag,Index)) THEN Next_index

Errcnt[Index+1,Index+1]=max(Wall(Errcnt[Index+1,Index+1]))+1)
INCREMENTS

Next_index=INDEX

OUTPUT Prt USING "S(K)"*" ERROR CODE ",Index," T",Errs(Index)

IF Errcnt[Index+1,Index+1]=max_warns THEN

OUTPUT Prt; "This is the last warning for this error." END IF

NEXT_INDEX=INDEX

OUTPUT Prt USING "$K$",Index,";" T",Errs(Index)

IF Errcnt[Index+1,Index+1]=max_warns THEN

OUTPUT Prt; "See the ",Errs," DOCUMENTATION FOR AN"

IF Errs<>1 THEN OUTPUT Prt; "EXPLANATION OF THESE ERRORS.

IF Errs<>1 THEN OUTPUT Prt; "EXPLANATION OF THIS ERROR.

OUTPUT Prt;[Asterisk][Line_len]

DATA "First pass parameter out of range." DATA "Second pass parameter out of range." DATA "Third pass parameter out of range." DATA "2347A does not respond to bus commands." DATA "2345A does not respond to bus commands." DATA "Measurement out of range." DATA "Reference temp. or excitation voltage out of range." DATA "The axis endpoints equal each other." DATA "Scaling value is <0 on a log axis." DATA "Datapoint <= 0 on a log axis." DATA "Level crossing not found." DATA "User definable error." DATA "User definable error." DATA "User definable error." DATA 

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

***** USER DEFINED FUNCTION FOR RECORDING A DC VOLTAGE

DEF FNDCv(Channel)

CON /Hp3054/ Scn,Dvm,Prv,Error,Errs[6],Errcnt[15]

OUTPUT Dvm USING "S"

STATUS Dvm DIV 100,7,Here1

Here2=8192*(Scn<>0)

MAKE A DEVICE NOT THERE CLEAR BIT 13

IF (Dvm<>Scn) OR (Channel<>-1) THEN Chk

IF (Dvm<>Scn) OR (Channel<>-1) THEN Chk

OUTPUT Scn USING "S"

STATUS Scn DIV 100,7,Here2

Chk:Error=2*(((Channel<>0) OR (Channel>999)) AND (Channel<>-1))

Error=Error+16*(((NOT BIT(Here2,13)) OR (NOT BIT(Here1,13)) AND (Dvm<>Scn))

Error=Error+32*(((NOT BIT(Here1,13)) AND (Dvm<>Scn))

IF Error THEN Abort

IF Channel<>-1 THEN OUTPUT Scn:"AC"*INT(Channel):"ST0"*1 CLOSE CHANNEL & WAIT UNTIL EXECUTED

IF Dvm<>Scn THEN OUTPUT Dvm:"VRSUN1DAVF1VD5VC8V50U70VT3"13457 SETUP & TR

IF Dvm<>Scn THEN OUTPUT Dvm:"HIST1G5TGT3"13456 SETUP & TRG

ENTER Dvm

RETURN Reading

Abort:Errs="Dcv"
CALL Warn
RETURN 9.E+19
1300 END
1301 SUB Lvdt(IJ,Channel,Cal,Mes,Dist)
1700 COM /HpS34/Son,Dvm,Pr,Eorea,Err@[61,Errcnt@15]
1800 Mes=FD0cv(Channcl)
1900 Dist=Mes/Cal
2000 PRINT "LVDT VOLTAGE FOR LVDT #";i;Mes
2100 PRINT "DISPLACEMENT FOR LVDT #";i;Dist
2200 PRINT
2300 SUBEND
2400 SUB Plotter_l(Titles,Xs,Ys,Minx,Maxx,Miny,Maxy) ; Routine To Plot Graph On Plotter.
2500 PRINT
2600 PRINT "<><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><><<
5610 MOV E 0,Y gdu max/2
5620 LABEL "LOAD (KIPS)"
5630 LOIR 8
5640 MOV E X gdu max/2..07+Y gdu_max
5650 LABEL "DEFLECTION (IN.)"
5660 VIEPOMPT .1*K gdu_max..90*X gdu max..15+Y_gdu max..9+Y_gdu max
5670 WINHON Mnx ,Maxx,Mny,Maxy
5680 AXE& Speae,Speae,Minx,Miny,Txmin,Tym1n,3
5690 AXE& Speae,Speae,Maxx,Maxy,Txmin,Tym1n,3
5700 1 NUMBERING X AND Y AXES
5710 CLIP OFF I CAN NOW LABEL OR PLOT OUTSIDE VIEWPORT LIMITS
5720 CSIZE 2.6,.5
5730 LOAD 6
5740 FOR I=Mnx TO Maxx+Spaex STEP Speae+Txmin
5750 MOV E I,Mny
5760 LABEL USING "K";I
5770 NEXT I
5780 LOAD 8
5790 FOR I=Mny TO Maxy STEP Speae+Tym1n
5800 MOV E Mn1x,I
5810 LABEL USING "K",X";I
5820 NEXT I
5830 MOV E 0,Mny
5840 MOV E 0,Maxy
5850 MOV E Mn1x,0
5860 MOV E Maxx,0
5870 DRAW Maxx,0
5880 CLIP ON I NOW CAN ONLY LABEL OR PLOT WITHIN VIEWPORT LIMITS
5890 SUBEND
5900 ******************************************************
5910 ******************************************************
5920 SUB Prinplot(Lowx,Higx,Lowy,Higy,Xtich,Yticl,Ytich,Spaeex,Spaeey,Ln

5930 OUTPUT KBD USING "$K$";CHR$(255)&"K"
5940 PRINT
5950 PRINT "DO YOU WANT TO CHANGE PLOTTING LIMITS FOR CRT GRAPH (Y/N)?"
5960 INPUT Ans
5970 IF Ans="Y" THEN 6010 5990
5980 6010 6050
5990 INPUT "INPUT PLOT AXES LIMITS (Lowx,Higx,Lowy,Higy)";Lowx,Higx,Lowy,Highy
6000 PRINT
6010 PRINT
6020 INPUT "INPUT NUMBER OF MAJOR AND MINOR TICK MARKS (Tmax,Txmin,Tmax,Tym1

6030 Spaeex=(Higx-Lowx)/(Xticl+Xtich)
6040 Spaeey=(Higy-Lowy)/(Yticl+Ytich)
6050 ALLOCATE X(400),Y(400)
6060 ON ERRO1 6080 SUB Recov
6070 OUTPUT KBD USING "$K$";CHR$(255)&"K"
6080 PRINT  "**********************************************************"
6090 PRINT   " PLOT OF LOAD VS. STRAIN    "
6100 PRINT  " **********************************************************"
6110 PRINT " NOTE: TO PROCEED AFTER GRAPH    "
6120 NEXT I
6120 PRINT "PRESS <CONTINUE> I E"
6130 PRINT "-------------------------------------"
6140 PRINT
6150 INPUT "INPUT NUMBER OF STRAIN GAGE TO BE PLOTTED", Xnum
6160 FOR I=1 TO Ln
6170 X(I)=Sp(I,Xnum)
6180 Y(I)=Loadp(I)
6190 NEXT I
6200 X=x"gage" (continues)
I CLEAR GRAPHICS SCREEN

1 PLOT LOAD 'JERSUS IVDT DISPLACEMENT

"UNEXPECTED ERROR. PLOTTING SEQUENCE WILL BE REPEATED."

"NEED TO CHANGE PLOTTING LIMITS"

FOR I=Lowy TO Highy STEP Spacey+yticl

FOR I=Lowx TO Highx

DRAW @Lowy

MOVE @Highy

DRAW @Highx

MOVE @Highx

FOR I=1 TO Ln

PLOT X(I),Y(I)

NEXT I

NEXT I

DEALLOCATE X(*),Y(*)

OUTPUT KBD USING "$K",CHR$(255)&"K" CLEAR CRT

PAUSE

GOTO 6840

Recov:

PRINT "UNEXPECTED ERROR. PLOTTING SEQUENCE WILL BE REPEATED."

PRINT "NEED TO CHANGE PLOTTING LIMITS"

PRINT PAUSE

GOTO 6840

SUB PROGRAM PLOTPLTS FOR PLOTTING DATA ON PLOTTER AS TEST PROCEEDS

SUBEND

SUB PLOTPLTS(Pplotval,Load,Minx,Maxx,Miny,Maxy,Defl,Load)

PLOTTER IS 705."HPGL"

OUTPUT 705:"SP2"

X_ddu_max=100+MAX(1,RATIO)

Y_ddu_max=100+MAX(1,1/RATIO)

WINDOW Minx,Maxx,Miny,Maxy

PLOT Defl,Load!

PLOT Plotval,Load 1 PLOT LOAD VERSUS LVDT DISPLACEMENT

PENUP

Defl=Plotval

Load=Load

SUBEND

SUBROUTINE Printerin FOR INPUTING GRAPH DATA FOR SCREEN GRAPH

SUBEND

SUB Printerin(Lowx,Highx,Lowy,Highy,xticl,xtich,yticl,yticl,Spacex,Spacey)

OUTPUT KBD USING "$K" CLEAR(255)&"K"

PRINT "" ENTER PLOT PARAMETERS FOR GRAPH ON CRT SCREEN ""

PRINT "" (AXIS AND MAJOR/MINOR TICK MARKS) ""

PRINT "" (SPACE FOR MAJOR/MINOR TITLES) ""

PRINT ""

INPUT ""INPUT PLRT AXES LIMITS (Lowx,Highx,Lowy,Highy)"

INPUT ""INPUT NUMBER OF MAJOR AND MINOR TICK MARKS (Tmaj,Tmin,Tmaj,Tmin)

Spacex=(Highx-Lowx)/xticl*xtich

128
7140  Spacey=(High-Low)/(Ytie1*Ytiech)
7150  SUBEND
7160  ! ****************************************
7170  ! SUBPROGRAM DataTypes STORES DATA IN FILEZ FOR TEST RESULTS
7180  ! ****************************************
7190  SUB DataTypes(Path2,D2,Ln,Load,Plotval,Ng,Num,P(+),On(+))
7200  OUTPUT @Path2,D2,Ln       ! STORE LOAD NUMBERS
7210       D2=D2+1
7220  OUTPUT @Path2,D2,Load      ! STORE LOADS
7230       D2=D2+1
7240  OUTPUT @Path2,D2,Plotval   ! STORE CENTER SPAN DEFLECTIONS
7250       D2=D2+1
7260  FOR K=1 TO Ng
7270  OUTPUT @Path2,D2,P(K)      ! STORE STRAIN VALUES
7280       D2=D2+1
7290  NEXT K
7300  FOR K=1 TO Num
7310  OUTPUT @Path2,D2,On(K)     ! STORE LVDT DISPLACEMENTS
7320       D2=D2+1
7330  NEXT K
7340  SUBEND

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Appendix F

*PIERHINGE* User’s Guide

*PIERHINGE* is a general program for collecting, storing, and processing test data using a Hewlett-Packard series 9000 microcomputer and a series 3000 data acquisition unit. The program collects data from strain gages and linear variable differential transformers (LVDT’s) attached to the test specimen and from a load cell and LVDT to measure axial load and lateral deflection. The collected data is stored on the computer’s hard drive for later processing. Data collected from the load cell and the LVDT mounted in the hydraulic ram arm are plotted during testing. A screen plot of the applied lateral load versus strain gage readings is also available.

Before testing can commence, certain parameters must be entered: file names to tell the program where to store the data, the date and time of the test, and measurements for plotter scaling. Other information required by the program is coded into the software. This information includes:

1. The total number of strain gages and LVDT’s used to record test data.
2. Gage factors and calibration factors for the strain gages, the LVDT’s and the load cell.
3. Channel assignments for the input devices.

During testing, data collection involves triggering the system and recording strain, displacement, and load values. These values are converted from voltage measurements by an analog-to-digital (A/D) converter. The test data is stored on the hard drive and sent to the printer to provide a log of strains, displacements, and loads. A plot of the axial load versus lateral displacement hysteresis curve is made during the test to provide “visual feedback.” After testing is complete, the user can view plots of the test data on the computer’s monitor. The program was designed to allow the operator to trigger one data point at a time, and is not set up for continuous real-time data acquisition. *PIERHINGE* was written in Hewlett-Packard Basic. Comments, denoted by a leading “!”, are provided throughout the program to document the logic flow.

The following is a line-by-line explanation of the program:

10-330

Comments on initial equipment set-up and basic testing operation.

340-570

Variable initialization. The program prompts the user to input the names of the strain gage reference data file, the output file name for the test data, and the title of the experiment.
Input the number of stain gages and LVDT's.

Set the date and time of the test.

The subprogram "Plotter_i" is called to ask the user to enter information about the load versus lateral displacement plot. "Plotter_i" sets up the graphics for the hard copy plot on the HP 7470A plotter. The user is asked to enter the x- and y-axis plot limits, and the number of major and minor tick marks. Plot axis limits refer to the minimum and maximum load and displacement limits. After inputting the required data the plot axes will be drawn and the graph will be labeled.

The user is now asked for plotting information from subprogram "Printerin," which sets up the on-screen plotting of load versus strain. Plot axis limits and tick marks are inputed from the computer. Viewport limits are set to maximum.

Retrieve the reference strain gage values taken prior to the start of the test. Reference values are obtained for each strain gage. These reference values are subtracted from test readings to obtain true values.

Memory is allocated for a maximum of 300 data points. Space is provided for 18 strain gages, 3 LVDT's, 1 load cell, and 1 LVDT located on the MTS hydraulic ram arm.

The program automatically assigns channels to the strain gages, LVDT's, load cell, and the LVDT located on the MTS ram. Also, within these lines are the gage and calibration factors for each device.

The HP-IB interface bus is initialized for proper communications. The following message will appear on the screen:
Equipment Present on Bus #7

<table>
<thead>
<tr>
<th>Equipment</th>
<th>Address</th>
</tr>
</thead>
<tbody>
<tr>
<td>3497A Mainframe</td>
<td>09</td>
</tr>
<tr>
<td>3456A Digital Voltmeter</td>
<td>22</td>
</tr>
<tr>
<td>3437A System Voltmeter</td>
<td>24</td>
</tr>
<tr>
<td>System Computer</td>
<td>21</td>
</tr>
<tr>
<td>System Printer</td>
<td>01</td>
</tr>
</tbody>
</table>

1380-1460

Read unstrained bridge imbalance-to-excitation voltage ratio and excitation voltage for each strain gage, and print the results. Lines 1340 and 1380-1460 are executed initially to obtain the reference strain gage data and each time a data point is triggered during the test.

1500-1520

Store the reference strain gage data for later use.

1560-1710

Calculate actual strain gage values with respect to the reference readings and print the values.

1750-1840

Compute lateral force and horizontal displacement from the load cell and LVDT located on the MTS ram. These values are used for the load versus displacement plot.

1860-1890

Print the test data.

1930-2030

Call subroutine “Lvdt,” which calculates the external LVDT’s displacements and then prints the results.

2050-2070

Call subroutine “Datasave” to save the test data on the hard drive.

2570-2650

Subroutine “Datastor;” saves the strain gage reference data.

2660-2940
Subroutine “Dataprint;” prints the following test data: test specimen name; time and date; lateral load; center span deflection; load number; strain gage number, channel, and the strain for each gage; and the LVDT number displacement.

3010-3650

Subroutine “Init.” All of the system voltmeters are initialized and the interface bus is cleared for transfer of data from the controller to the computer. This portion of the program initializes the HP-IB, assigns device names to select codes, and lists the instruments connected to the HP-IB with their address.

3690-4000

Subroutine “Brmeas.” Takes a strain gage bridge measurement in conjunction with the user defined function DEF FNStrain to perform a complete strain gage measurement. This subroutine is called before stress is applied to the gage; after stress is applied to the gage, the subroutine is again called for a second set of measurements. This information is passed to DEF FNStrain to calculate the strain.

4040-4140

User defined function DEF FNStrain.

4180-4880

Subroutine “Warn.” This subroutine detects errors, such as when a strain gage fails during testing. An error message will be printed (the message will only be printed once, when the gage fails), and subsequent readings from the failed gage will be printed with a value of 9.E+19 printed. This error, however, will not halt program execution.

4920-5120

User defined function DEF FNDCv(Channel). A user defined function for recording DC voltages.

5160-5230

Subroutine “Lvd.” In line 5180 a voltage measurement for the LVDT is taken by the user defined function DEF FNDCv. The voltage is converted to displacement by dividing the voltage by the appropriate calibration factor.

5280-5880

Subroutine “Plotter_i.”

5920-6860

Subroutine “Printplot.”
6890-7000
Subroutine "Plotrplot."

7040-7150
Subroutine "Printerin."

7190-7340
Subroutine "Datasave."
### Appendix G

**List of CCEER Publications**

<table>
<thead>
<tr>
<th>Report No.</th>
<th>Publication</th>
</tr>
</thead>
</table>


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CCEER-90-1  Saiidi, M., E. Maragakis, G. Ghusn, Jr., Y. Jiang, and D. Schwartz. 
"Survey and evaluation of Nevada's transportation infrastructure, task 
Reno: University of Nevada, Department of Civil Engineering. 
October 1990.

CCEER-90-2  Abdel-Ghaffar, S., E. Maragakis, and M. Saiidi. "Analysis of the 
response of reinforced concrete structures during the Whittier 
of Nevada, Department of Civil Engineering. October 1990.

CCEER-91-1  Saiidi, M., E. Hwang, E. Maragakis, and B. Douglas. "Dynamic 
testing and analysis of the Flamingo Road Interchange." Report 
number CCEER-91-1. Reno: University of Nevada, Department of 

CCEER-91-2  Norris, G., R. Siddharthan, Z. Zafir, S. Abdel-Ghaffar, and P. 
Gowda. "Soil-foundation-structure behavior at the Oakland Outer 
Harbor Wharf." Report number CCEER-91-2. Reno: University of 

CCEER-91-3  Norris, G. M. "Seismic lateral and rotational pile foundation stiffness 
at Cypress." Report number CCEER-91-3. Reno: University of 
Nevada, Department of Civil Engineering. August 1991.

highway bridge decks in Nevada, with emphasis on polyester-styrene 
polymer concrete." Report number CCEER-91-4. Reno: University of 
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