PROGRESSIVE COLLAPSE RESISTANCE OF STEEL BUILDING FLOORS

FINAL REPORT

By

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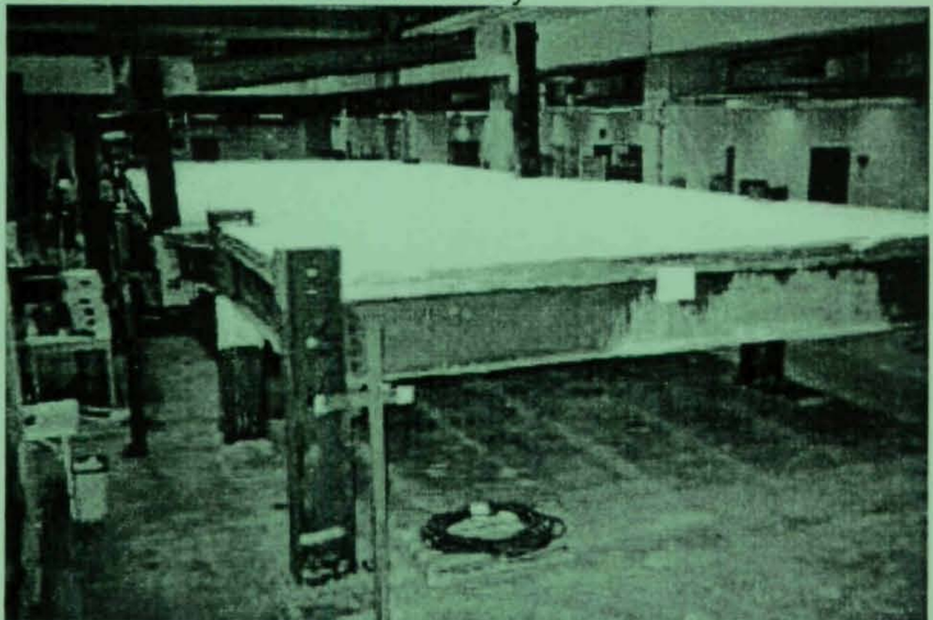

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Report to the Sponsors: American Institute of Steel Construction and General Services Administration
September 11, 2001

In memory of those who perished on September 11, 2001 in terrorist attacks and dedicated to the firefighters, police and rescue workers who gave their lives to save others.

PROGRESSIVE COLLAPSE RESISTANCE OF STEEL BUILDING FLOORS

FINAL REPORT

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The research reported herein was done in the Department of Civil and Environmental Engineering of University of California, Berkeley and was sponsored by the American Institute of Steel Construction and the General Services Administration of the U.S. Federal Government. The opinions expressed in this report are those of the authors and do not necessarily reflect the views of the University of California, Berkeley or the Sponsors.

This report presents the background, test set-up, specimens, test procedures and the test results of "column-drop" tests of a one story steel structure. The specimen was a 60ft by 20ft one story typical steel structure with steel deck and concrete slab floor and wide flange beams and columns. The connections were either standard shear tab or bolted seat angle under bottom flange and a bolted single angle on one side of the web. The main objectives of these studies were to explore the strength of a typical steel structure and floor system to resist progressive collapse in the event of removal of a column by a blast and to establish the failure modes. An added objective was using the test results to provide the sponsors with design-oriented information on the potential of existing typical steel structures to resist progressive collapse and the possible research needs in this field.

The tests indicated that after removal of the middle perimeter column the catenary action of the steel deck and girders was able to redistribute the load of removed column to other columns and the design dead load and live load of the floor could be resisted without collapse of the floor. Damage to the system was primarily in the form of cracking of floor slab, tension yielding of the steel corrugated deck in the vicinity of collapsed column, bolt failure in the seat connections of the collapsed column and yielding of web of girders acting in a catenary configuration. Although the system was able to resist the collapse after removal of one column, it was suggested that future research can include a retrofit scheme where cables can be added to the side of beams to assist in developing larger catenary action with greater factor of safety to prevent progressive collapse.

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ABSTRACT

This report presents the background, test set-up, specimens, test procedures and the test results of "column-drop" tests of a one story steel structure. The specimen was a 60ft by 20ft one story typical steel structure with steel deck and concrete slab floor and wide flange beams and columns. The connections were either conventional shear tab or bolted seat angle under bottom flange and a bolted single angle on one side of web.

The main objectives of these studies were to explore the strength of a typical steel structure and floor system to resist progressive collapse in the event of removal of a column by a blast and to establish failure modes. An added objective was using the test results to provide the sponsors with design-oriented information on what is the potential of existing typical steel structures to resist progressive collapse and what are the possible research needs in this particular field.

The tests indicated that after removal of the middle perimeter column, the design dead load and live load of the floor could be resisted by catenary action of floor. As a result, the floor did not collapse. Damage to the system was primarily in the form of cracking of floor slab, tension yielding of the steel corrugated deck in the vicinity of collapsed column, bolt failure in the seat connections of the collapsed column and yielding of web of girders acting in a catenary configuration. Although the system was able to resist the collapse after removal of one column, it was suggested that future research can include a retrofit scheme where cables can be added to the side of beams to assist in developing larger catenary action with greater factor of safety.

ACKNOWLEDGMENTS

The study reported here was primarily supported by the American Institute of Steel Construction and the General Services Administration (GSA) of the U.S. Federal Government. The authors would like to express their appreciation for the support as well as technical advice and input received from the AISC engineers. Particularly, the support and technical input from Dr. Nestor Iwankiw, Tom Schlafly and Dr. Sergio Zoruba of the AISC were very useful and essential to the success of this project and are sincerely appreciated. The input from Mr. Willie Hirano of the General Services Administration and their support for fabrication of the test specimen is acknowledged and appreciated. Mr. Brian Dickson of Skilling Ward Magnusson Barkshire, a consultant to GSA, also provided invaluable technical contributions in design and fabrication of test specimen as well as testing procedures. The tests reported herein were conducted as a continuation of earlier tests conducted on the specimen. The earlier tests were conducted to investigate the effects of cables placed in the floors to prevent progressive collapse. The earlier project, sponsored by the General Services Administration of the U.S. federal government and by the Skilling Ward Magnusson Barkshire, is summarized in a report by Astaneh-Asl et al. (2001).

The tests reported here were conducted in the Department of Civil and Environmental Engineering of University of California at Berkeley. The testes were primarily conducted by the second author, Brant Jones, as a CE-299 Independent Research Project and as part of his Master of Science studies supported by a Professional Development Program, a privilege for which he is grateful to the Defense Nuclear Facilities Safety Board.

The contributions of the Department staff particularly William Mac Cracken, Dr. Lev Stepanov, Chris Moy, Frank Latoya, Richard Parsons, Douglas Zulaica and Mark Troxler were very valuable in fabrication and installation of instrumentation, erection of steel frame, conducting the tests as well as in data acquisition and processing. Without their dedication and professional work, this project would not have been possible. Roger Jung, Erik Madsen and Qiuhong Zhao , the graduate student research assistants participated in various aspects of the project. Wendi Li and Sunil Tellis , undergraduate student research assistants, were also involved in preparing the specimen, conducting material tests and other aspects of this large scale testing. The contributions of these students were very valuable, acknowledged and sincerely appreciated.

Dr. David McCallen and Charles Noble of the Lawrence Livermore Laboratory assisted the project in conducting inelastic analyses of the test set-up and specimen.

Herrick Corporation of Stockton California fabricated the test specimen with diligence, on time and on budget and according to drawings and specifications. The efforts of Jamie Winens, the project engineer with Herrick Corporation are sincerely appreciated.

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CHAPTER ONE

Introduction

1.1. Introduction

In recent years, a number of tragic terrorist attacks, particularly, in the U.S. soil, have resulted in a number of initiatives to study the resistance of structures to blasts. In addition, a number of research projects have been undertaken or are underway to develop mechanisms and systems to reduce the hazard of such attacks. The main aim of these efforts is to protect the safety of the occupants of the building, the rescue workers and those who are around the building who can be killed or injured by collapsing structure and the falling debris.

One of the main areas of research and development in this field is the progressive collapse prevention. From structural engineering and construction point of view, of course one can design a building that can withstand a terrorist bomb attack with minimal or no damage. This has been done for years and continues to be done for militarily sensitive and other critical buildings that are necessary to be functional and occupied even after a bomb attack on them. Of course, designing such a highly protected building requires a significant amount of funding as well as resources. In addition, to achieve the objective of the minimal damage, the designers may end up sacrificing the exterior aesthetics and in some cases the internal functionality of the building. Although in case of military installations, the high cost and bunker-like appearance of a building can be justified, however, for civilian buildings, such high costs cannot be afforded and the loss of aesthetics may not always be acceptable. This was because of the assumption that civilian buildings had a very low probability to be a target of terrorist attack.

In order to make a civilian building structure blast-resistant, first, one has to define the performance criteria for blast resistance. The performance criteria should be specific enough to provide structural engineers with clear parameters such that they can design structures to achieve an expected performance. The following section discusses Performance Criteria in detail.

1.2. Blast-Resistance Performance Criteria

The blast performance criteria should be established in a way that the society as a whole is satisfied with the target performance level and can afford the associated cost. Currently there are no formal blast performance criteria for civilian buildings. It seems that a criteria that may be acceptable as well as affordable by the society at this time and with present probability of terrorist attacks, which still is relatively low, to ensure that the massive loss of life and severe injuries are avoided and partial or full collapse is prevented. Following this proposed performance criteria, damage to a building in a terrorist attack is accepted as long as such damage does not result in death, severe injuries or collapse.

There are several reasons for death and injuries in a blast attack. Death and injury can occur because of direct impact from explosions, heat, fire, inhaling smoke and hazardous gases emanating from the explosion or building contents, flying debris or building contents, falling debris or building contents, shrapnel, air impact or other related causes such as heart attacks.

Immediate collapse of a structure and the resulting pancaking of floors has been the main cause of deaths and injuries in past blast attacks on buildings. It is clear that by preventing immediate collapse, especially by preventing catastrophic collapse of floors, one can save many lives and be able to evacuate the occupants to safety. Therefore, making civilian buildings blast-resistant can be translated to making sure that partial and full collapse of the structure of these buildings are prevented or delayed to be able to evacuate the occupants in time and save their lives. This occurred in the aftermath of tragic events of September 11, 2001 when the towers of World Trade Center, attacked by jetliners, were able to withstand the impact for more than an hour and save the lives of estimated 20,000 people who evacuated during that period of time between the attacks and the collapse of the towers (Astaneh-Asl, 2001).

Some structures have highly indeterminate structural systems with more than one load paths to resist the applied loads. In these highly indeterminate systems with many redundant members, it is possible to eliminate a number of structural elements without inducing collapse to the structure or floors. Obviously, if such systems are identified, their use will result in a significant savings in preventing collapse of building structures during or after an attack by explosives. Such characteristics denoted as " progressive collapse" resistance was the main theme of this research project.

1.3. Objectives

The tests reported herein were conducted to collect data on progressive collapse resistance of typical steel structure with steel deck concrete slab floor supported on steel framing with simple connections.

1.4. Scope and Content of the Report

The focus of this report is to investigate the ability of conventionally designed steel building structures to resist progressive collapse. The discussion will be limited to observed behavior and test data resulting from experimental research of a full-scale floor system, typical to commercial office buildings. Conclusions and recommendations will focus on overall failure mechanisms. Some recommendations are made for future research needs.

CHAPTER TWO

Experimental Research

2.1. Introduction

Early in 2001, a series of tests were conducted at the University of California at Berkeley on a full-scale one-story steel structure to study a cable-based mechanism that could prevent the progressive collapse of steel structures. The project, funded by the General Services Administration, was successfully completed and the results were reported in Reference (Astaneh-Asl et al, 2001).

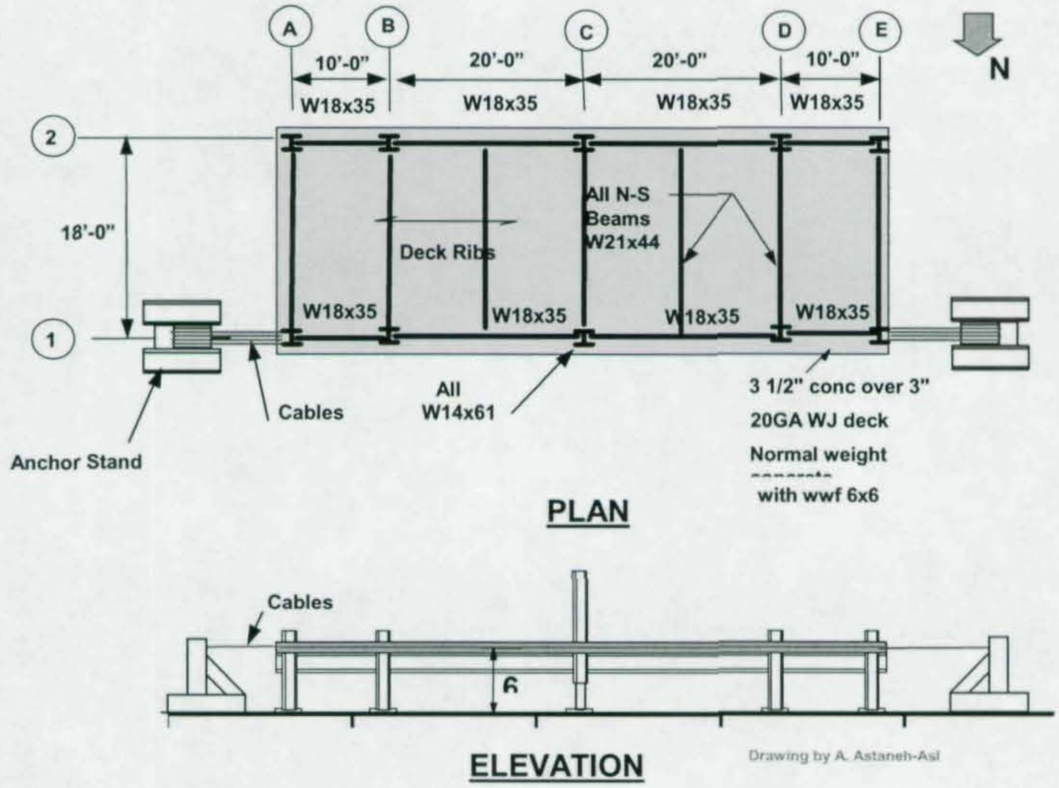


Figure 2.1. Test Specimen

The tests indicated that by placing steel cables inside the floor slab along the column lines, one can prevent collapse of the floor if a supporting column is removed by blast forces during a terrorist attack. The specimen used in these tests was a 60ft by 20 ft floor supported by a steel structure with ten columns. The one-story structure, shown in Figure 2.1, was a realistic representative of a typical steel structure.

After completion of the above-mentioned GSA tests, which were conducted on one side of the specimen (the North side), the opposite other side of the specimen was almost undamaged with minor cracking in the concrete slab. It was decided to take advantage of the availability of the pre-existing specimen and to conduct similar tests on the south side of the specimen, where there was no cable (see Figure 2.1.) This report is focused on these tests. The reader is referred to Reference (Astaneh-Asl, Madsen and Jung, 2001) for information on the tests conducted on the North side of specimen where steel cables placed were within the floor slab.

2.2. Objectives of the Research and Testing

The main objectives of the research and testing reported herein were:

a. To investigate the potential of typical steel floors to develop catenary action in the event of removal of a supporting column.
b. To establish the potential of typical steel structures to resist progressive collapse by redistributing column loads through Catenary action of floors.
c. To develop a better understanding of how connections of steel structures perform when subjected to large deformations and large Catenary forces.
d. To develop recommendations for future research as well as tentative suggestions for blast-resistant design of typical steel structures.

2.3. Design of Specimen

The structural design of the test specimen was done by SWMB engineers. The Principal Investigator at the University of California, Berkeley (UCB) was involved in the design process, ensuring that the specimen was designed within the test limitations and would fit properly the test set-up and the laboratory reaction floor. The specimen consisted of steel frames with simple shear connections and a concrete slab cast on a steel deck. The structure was designed to satisfy the applicable code requirements for a typical structure in Seattle, Washington.

The specimen, shown in Figure 2.1, had four bays in the longitudinal direction totaling 60 feet and one bay in the transverse direction with a width of 18 feet. The exterior bays were both 10 feet long while the interior bays were 20 feet. The concrete slab was 6 feet above the laboratory floor level. Design drawings as well as shop drawings of the specimen are provided in Appendix A.

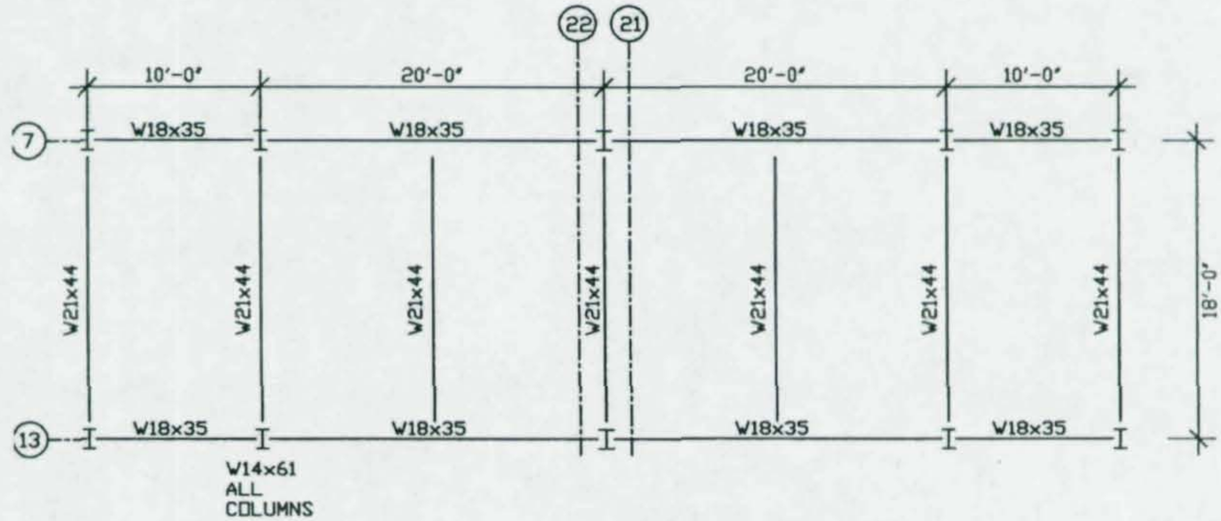


Figure 2.2 Framing Plan of Test Specimen

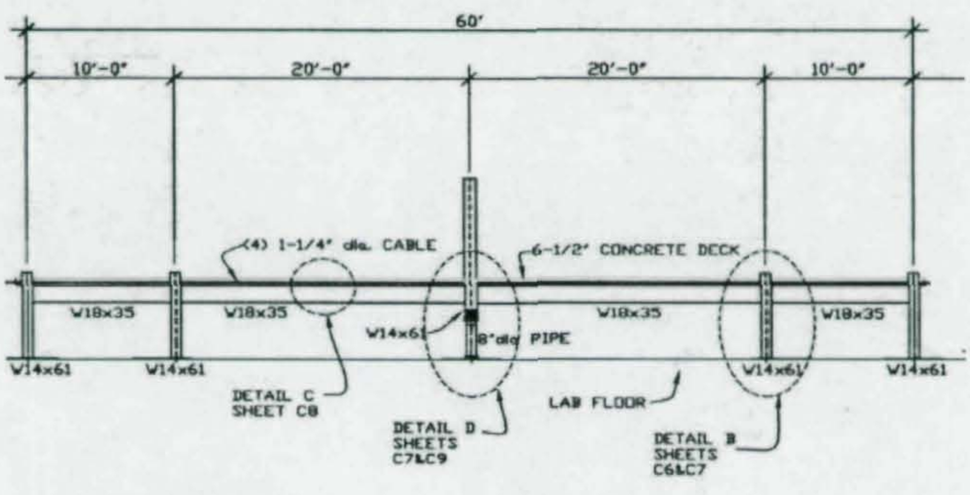


Figure 2.2 Framing Plan of Test Specimen

The design produced W14x61 steel columns, W18x35 beams in the longitudinal direction, and W21x44 beams in the transverse direction at 10 feet intervals. Figures 2.2 and 2.3 show framing plan and elevation view of the test specimen.

Figure 2.4 shows a cross section of test specimen. The right side frame was tested in this program.

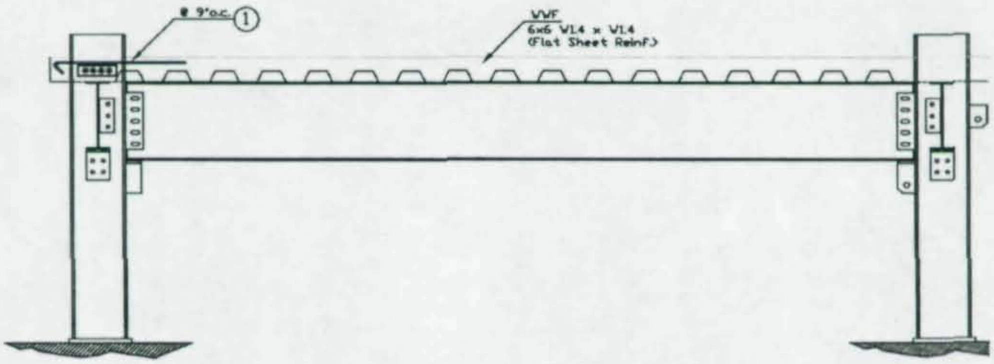


Figure 2.3. Cross Section of Test Specimen

The steel deck was a typical 20-gage deck connected to the framing with "puddle" welds and with shear studs. Figure 2.4 shows a general view and close up of the deck.

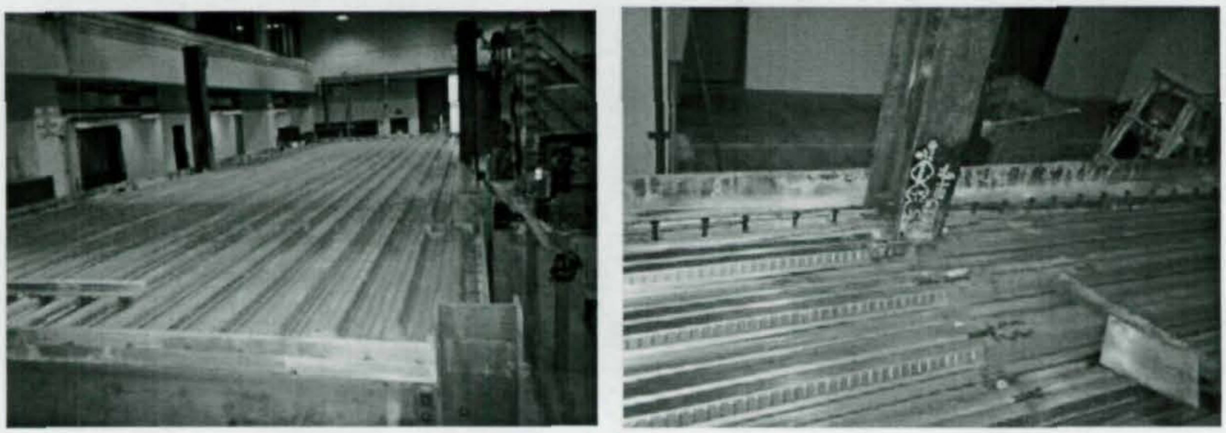


Figure 2.4. A view of Steel Deck and Close-up of Deck and Shear Studs

Figure 2.5. shows plan view of the shear studs. Placed on top flange of beams and girders.

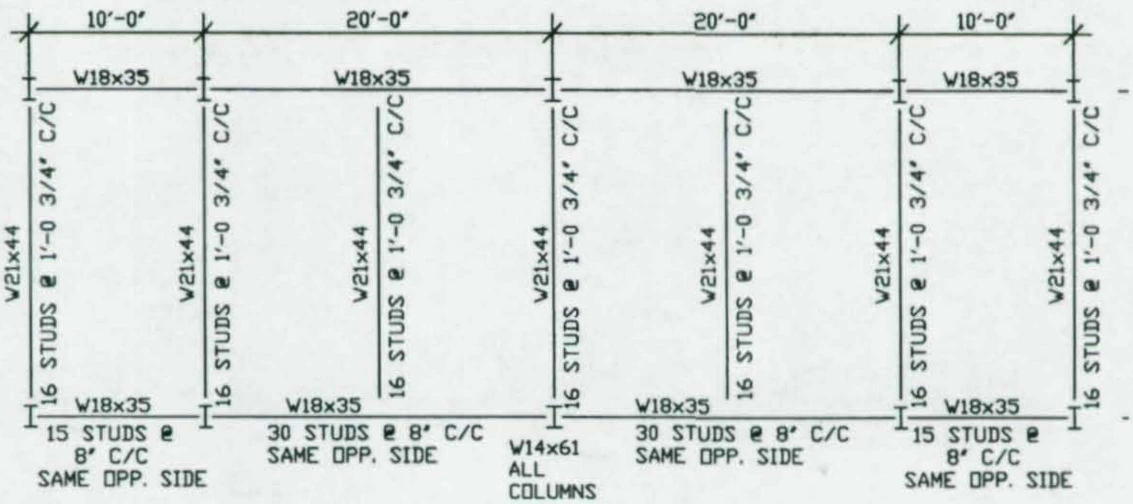
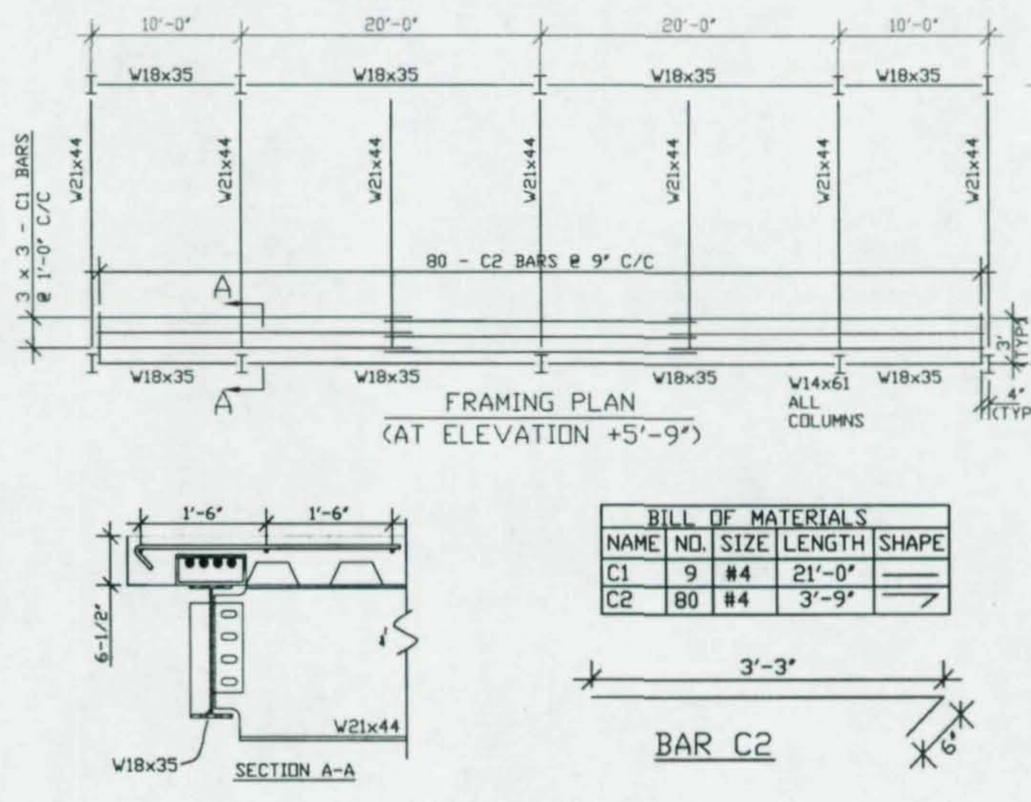


Figure 2.5. Plan of Shear Studs



BILL OF MATERIALS				
NAME	NO.	SIZE	LENGTH	SHAPE
C1	9	#4	21'-0"	——
C2	80	#4	3'-9"	——/

Figure 2.6. Plan of Reinforcement and Details

The floor slab in the test specimen was a 6.5-inch maximum thickness slab. The North side which had cables in it had reinforcement but the South side, the subject of tests reported here did not have any additional rebars other than the standard wire mesh as shown in Figure 2.1. Figure 2.6. shows plan of reinforcement and details. Figure 2.7 shows a plan views of slab around the columns.

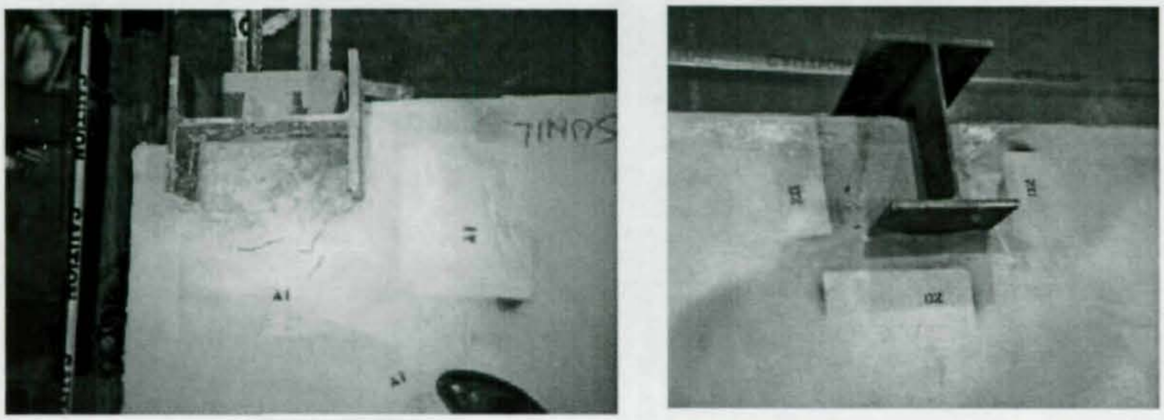


Figure 2.7 Close-up View of Slab in the Column Areas

Figure 2.8 shows views of beam-to-column and beam-to-beam connections. The connections in the specimens were all either traditional shear tabs or bolted seat angles plus a bolted single angle on the web. Figure 2.8 shows details of the connections.

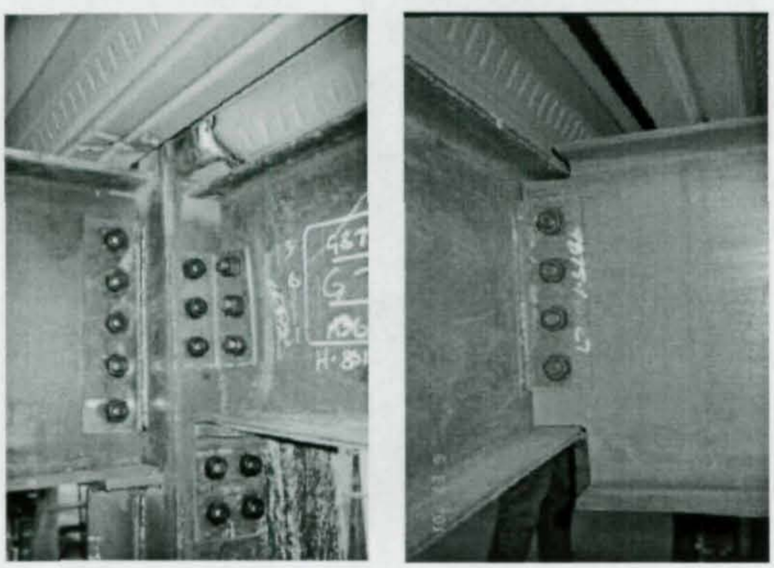


Figure 2.8. Beam-to-Column and Beam-to-Beam Connections

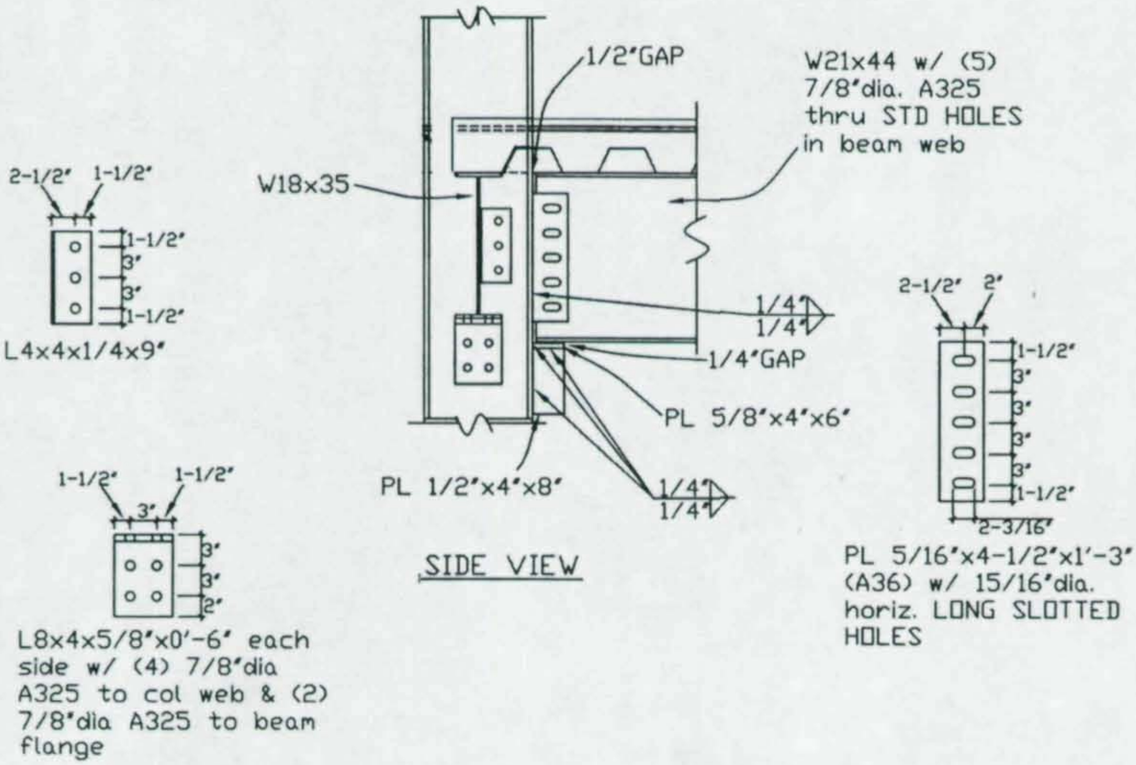


Figure 2.9. Details of Beam-to-Column and Beam-to Beam Connections

2.3. Construction of Specimen

The steel members and connections of the test specimen were fabricated by Herrick Corporation and delivered to Davis Hall laboratory at the UC-Berkeley. The steel framing was erected inside the lab by UC-Berkeley machinists and welders. After completing the deck and installation of the shear studs, wire mesh and rebars were added and the concrete slab was cast. The construction process was similar to constructing a typical steel structure. Figure 2.10 shows various stages of construction.

2.4. Material of Test Structure

The beams, columns, angles, and shear tabs were specified as ASTM A36. The concrete in the floor slab was specified as normal weight concrete with $f'_c = 4000$ psi. The slump of the concrete was measured to be 4-½ inches as shown in Figure 2.11. Concrete tests confirmed a compressive strength greater than 4000 psi after 21 days. The concrete tests were performed at the UC-Berkeley material lab. The results of tests are shown in Figure 2.12.

Figure 2.10. Construction of Test Structure

Figure 2.11. Slump Test and Cylinder Tests on Concrete Used in Slab

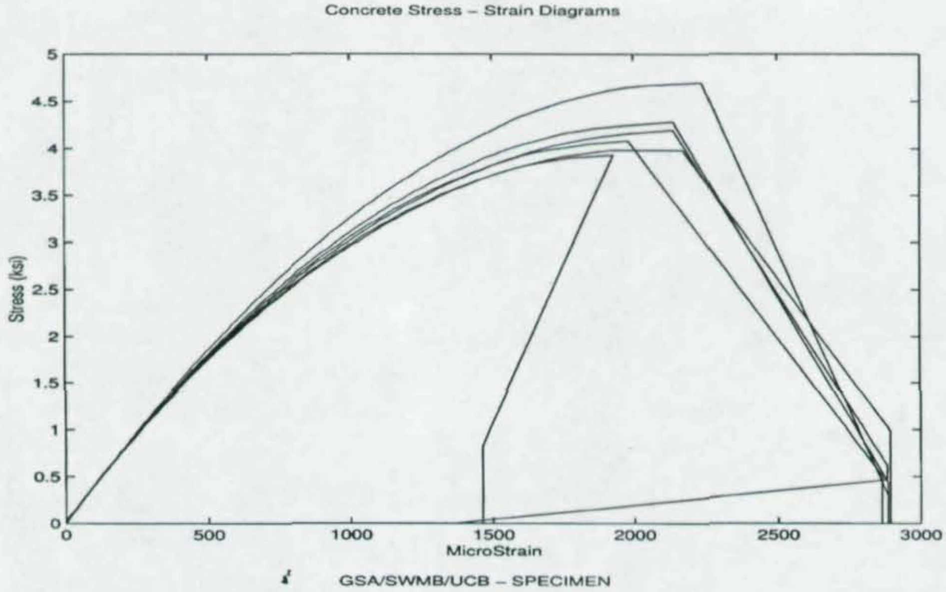


Figure 2.12. Results of Concrete Compression Tests

All bolts were A325, 7/8-inch diameter. The lengths of the bolts were governed by the widths of the connecting material, with either 2-¼ inch or 3-inch bolts providing the required length. The welds were shop welds and were performed by Herrick Corporation.

Verco Structural Steel Decking provided the Type W3 Formlok, Gauge 20 steel deck. The ¾ inch diameter, 4 ½ inch long shear studs used in the test structure were supplied and installed by Nelson Studs. They were placed at 8 inch centers along the longitudinal beams and 1'-0 ¾" centers along the transverse beams as shown in Figure 2.5. W1.4xW1.4 flat sheet reinforcement with 6 inch by 6-inch openings was utilized within the concrete floor slab.

2.5. Instrumentation of Specimen

Displacement transducers and strain gages were used at various locations on the specimen. Strain gauges were installed on the structure to monitor the strain at locations of interest.

2.5.1. Displacement Transducers

Nineteen displacement transducers were positioned on and around the test structure. Specific locations of the transducers are included in Figures 2.13, 2.14, and 2.15. Transducers located vertically around the specimen measured the drop of the floor at those locations while horizontal transducers measured the translation. Pairs of displacement transducers located on the beams connecting into the test column were used to measure the rotational demands of the connections. Whenever possible, the transducers were aligned parallel to the expected direction of displacement.

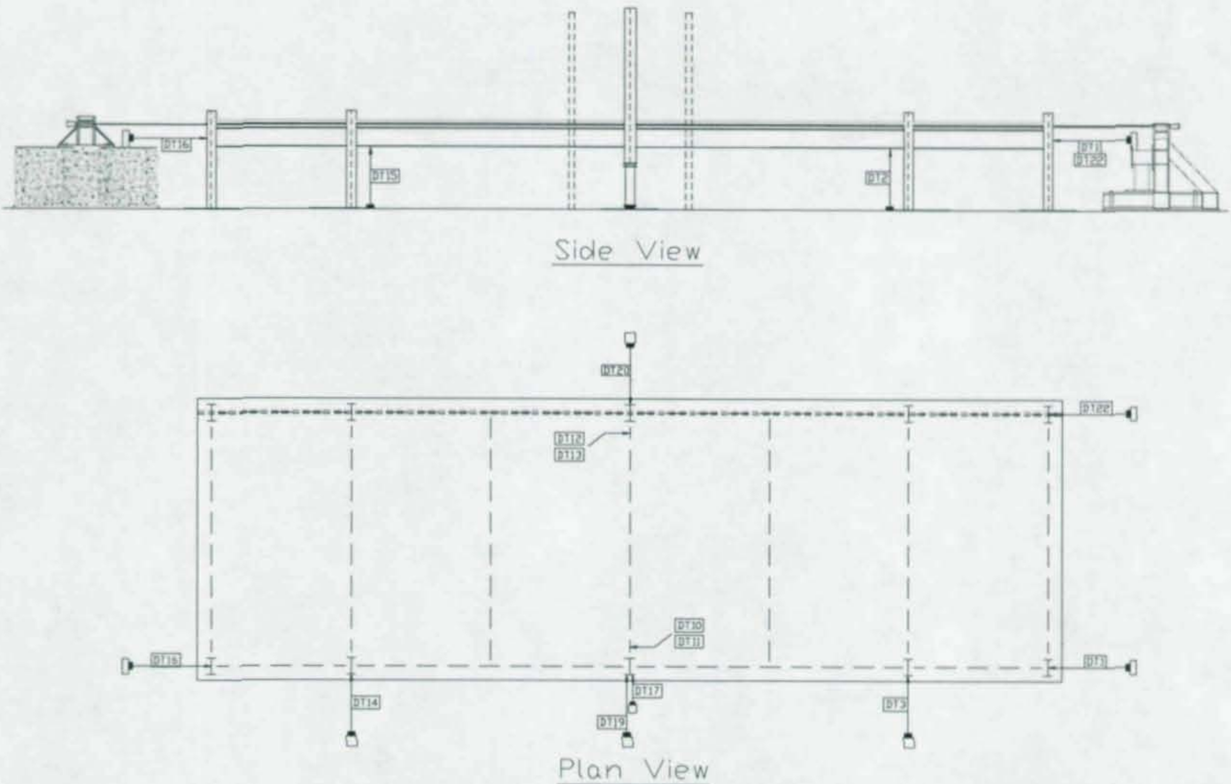


Figure 2.13. Plan view of Displacement Transducers.

Six transducers were positioned horizontally around the outside of the structure to monitor the in-plane translation of the specimen, Figure 2.13. The transducers were secured to rigid supports independent of the test specimen. Connecting wires were secured to the structure and tied to the transducer. The horizontally placed transducers were labeled Displacement Transducer (DT) 1, 3, 14, 16, 20, and 22.

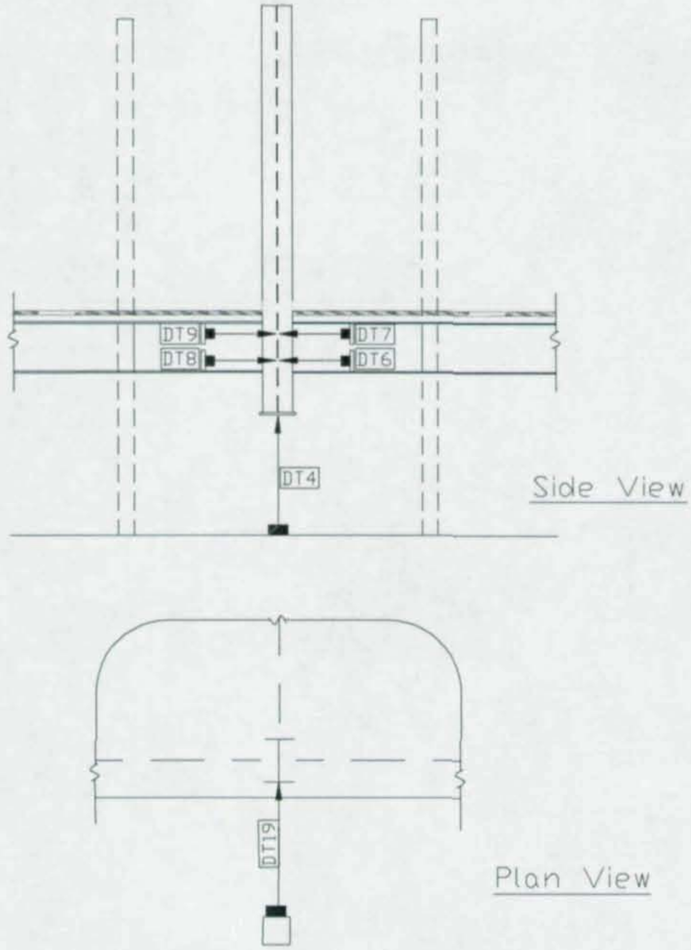


Figure 2.14. Detail of Displacement Transducers at Test Column.

A pair of transducers was placed on the ends of each of the three beams framing into the test column. An additional pair was placed at the opposite end of the transverse beam. The transducers were aligned parallel to the webs of the beams, two inches from the outermost surface of the closest flange. A threaded rod was welded to the web of the beam onto which each transducer could be attached. A connecting wire connected the transducer to the test column. The eight rotational transducers were labeled DT 6 through 13.

Three displacement transducers were positioned along the test column line to measure the vertical movement of the frame. One transducer was positioned at the test column while two other transducers were placed near the adjacent columns. The three transducers were labeled DT 2, 4, and 15.

Two additional displacement transducers were attached to the test column as can be seen in Figure 2.15. The transducers were labeled DT 17 and 19. The test column was not only expected to displace vertically, but also to rotate as it displaced. For this reason, it was not possible to orient the transducers in a way to reduce the measurement to one dimension. Initial connecting wire lengths were recorded and are included in the figure.

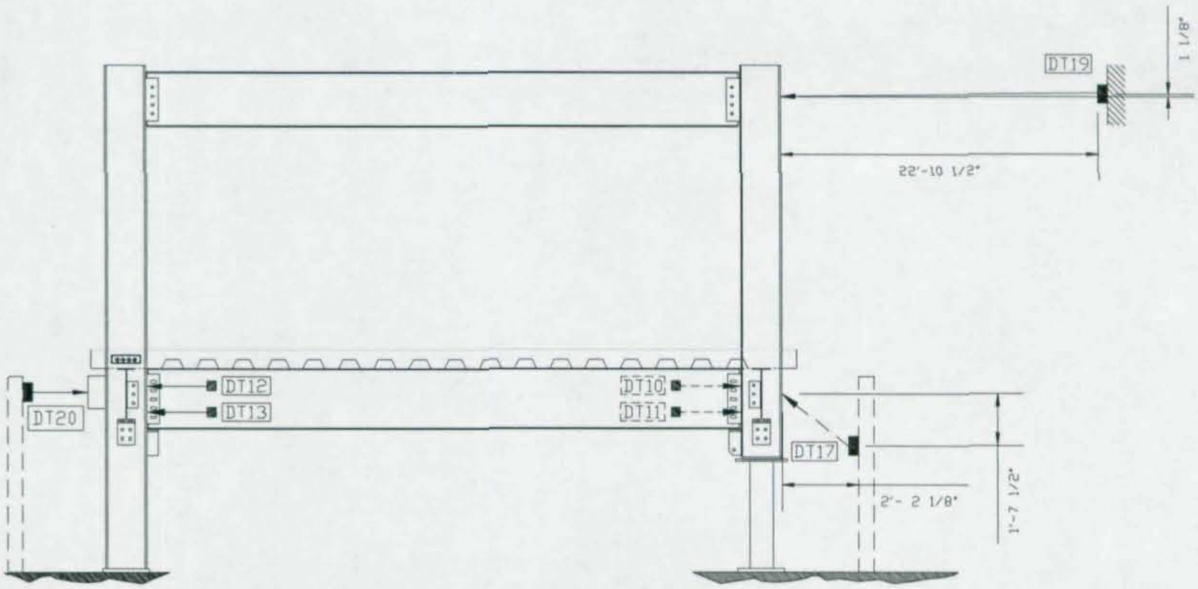


Figure 2.15. Side View of Displacement Transducers at Test Column.

Displacement transducers with labels numbered 5, 18, and 21 were not utilized in the testing. The transducers were labeled in a fashion so that there would be a one-to-one correspondence to the transducers used in the tests conducted on the north side of the specimen where steel cables were placed within the floor slab (Astaneh-Asl, Madsen, and Jung, 2001).

2.5.2. Strain Gauges

Strain gauges were installed to monitor the strain at locations of interest. Specific locations of the gauges are included in Figures 2.16, 2.17, 2.18, and 2.19. Four 45° rosettes were placed in a symmetrical fashion on the underside of the steel deck. Three linear gauges previously attached to an angle were monitored. The particular angle connected the test column to the longitudinal beam directly east of the test column. In addition, four linear strain gauges were placed underneath the steel deck in a single line, 30 inches east of the test column. In addition, seven additional gauges were placed on the web and flanges of the longitudinal beam directly east of the column 30 inches from the test column.

The specimen was carefully prepared before the strain gauges were bonded to the surface. Rust and mill-scale were removed from the steel beam and angle by grinding the surface with a stone-based grinding wheel. The surface was then progressively smoothed by using a cycle of sanding wheels beginning with a 36-grit wheel and ending with a 120-grit wheel. This process removed visible defects. Next, the surfaces were sanded by hand using 240-grit sandpaper followed by 360-grit sandpaper. The process of placing the gauges on the steel deck began at this hand-sanding step. The locations were then ready to be chemically treated

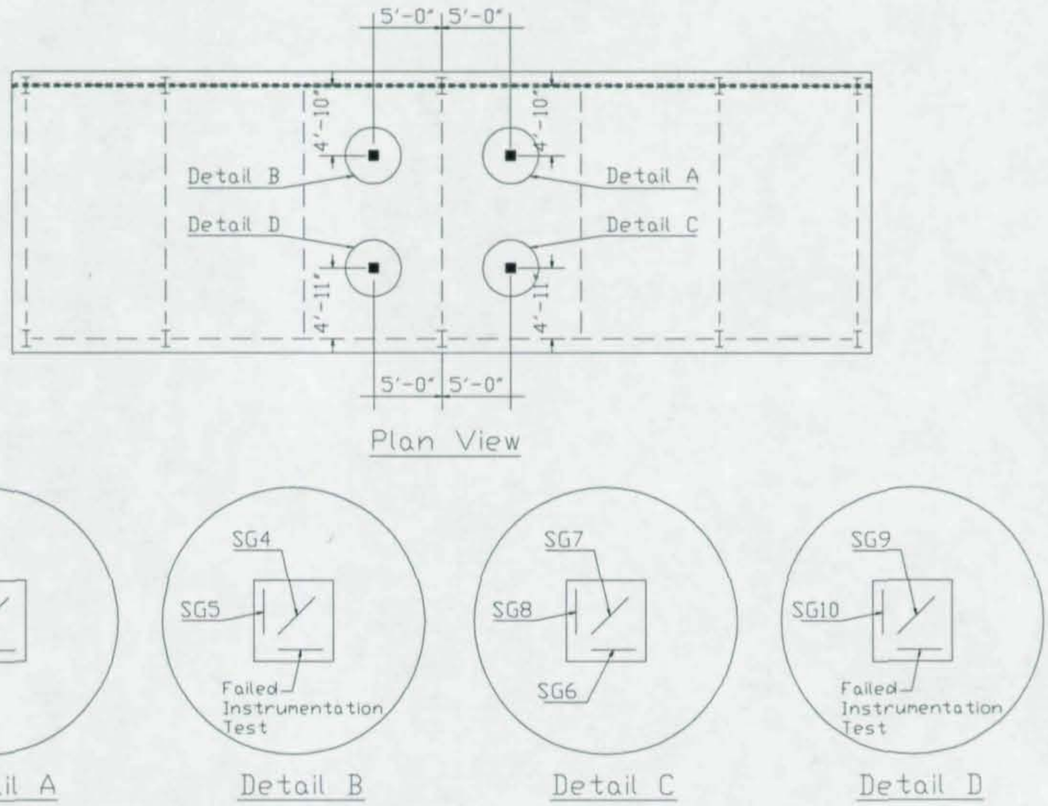


Figure 2.16. Layout of Strain Gauges 1 through 10.

Three linear gauges were placed on an angle connecting the test column to one of the longitudinal beams during testing of the opposite side, Figure 2.17. The gauges were once again monitored during testing. Significant and visibly noticeable yielding of the angle occurred during the prior testing. The three linear strain gauges were labeled SG 11 through 13.

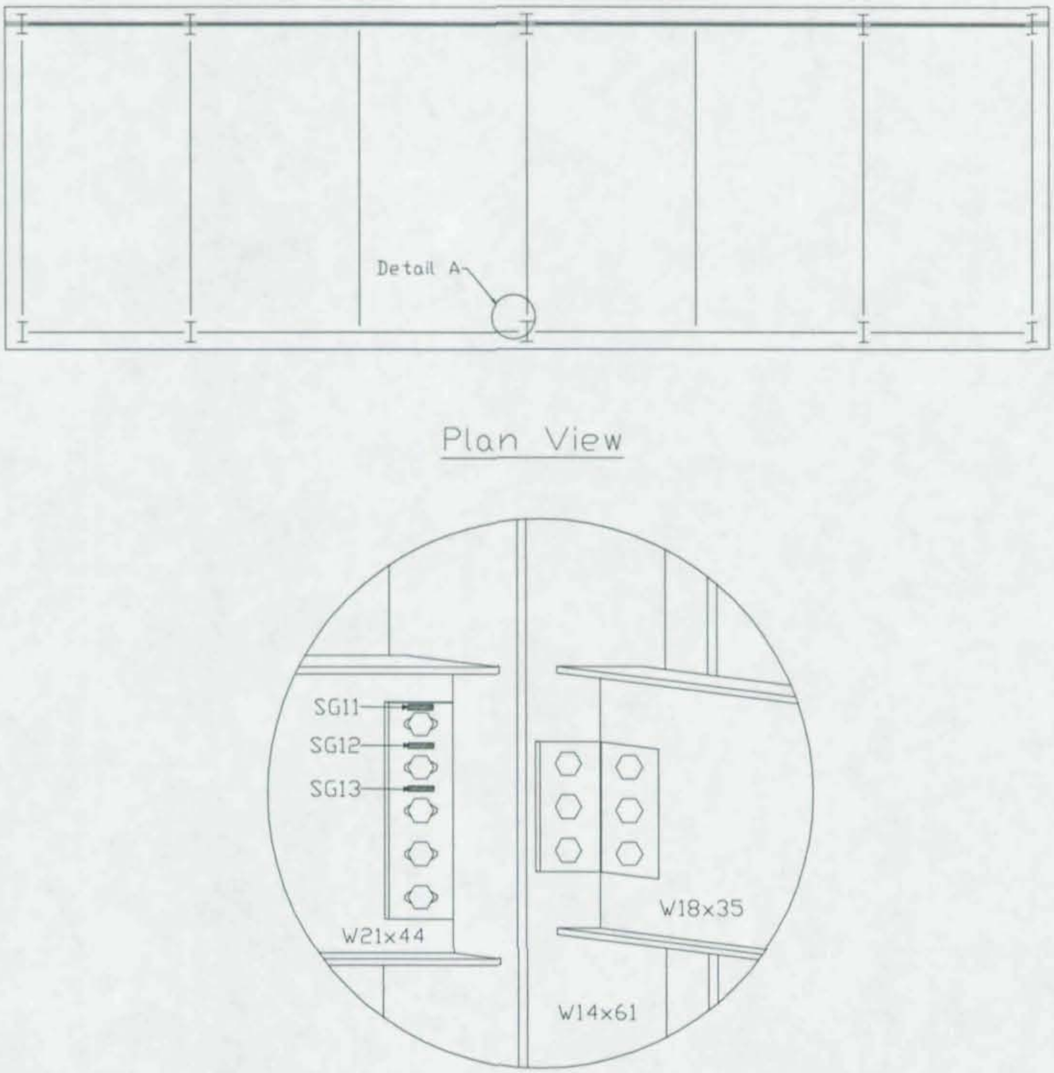


Figure 2.17. Layout of Strain Gauges 11 through 13.

Four 45° strain rosettes were placed on the underside of the steel deck in the bays adjacent to the displaced column, Figure 2.16. These gauges were placed systematically, with two residing on the west side of the test column, and two on the east side. During installation and inspection, it was determined that both longitudinal (east-west) gauge wires in the bay immediately west of the test column were damaged and did not provide the necessary resistance. For this reason, they were not attached to the data recording system.

No problems were encountered with the adjacent strain rosettes, placed in the bay immediately east of the test column. The ten functional gauges were labeled Strain Gauge (SG) 1 through 10.

Four additional linear strain gauges were placed beneath the steel deck, 30 inches east of the test column (Figure 2.18). The gauges provided data on the variation of stress in the steel deck. The four gauges were labeled SG 14 through 17.

Seven gauges placed on the longitudinal beam directly east of the column provided data that could be used to calculate the force in the beam throughout the tests (Figure 2.19). Two gauges were place on the bottom of the top flange and the top of the bottom flange, one on either side of the web. Three additional gauges were placed on the web of the beam facing the outside of the structure. These seven gauges were all located 30 inches east of the test column and labeled SG 18 through 24.

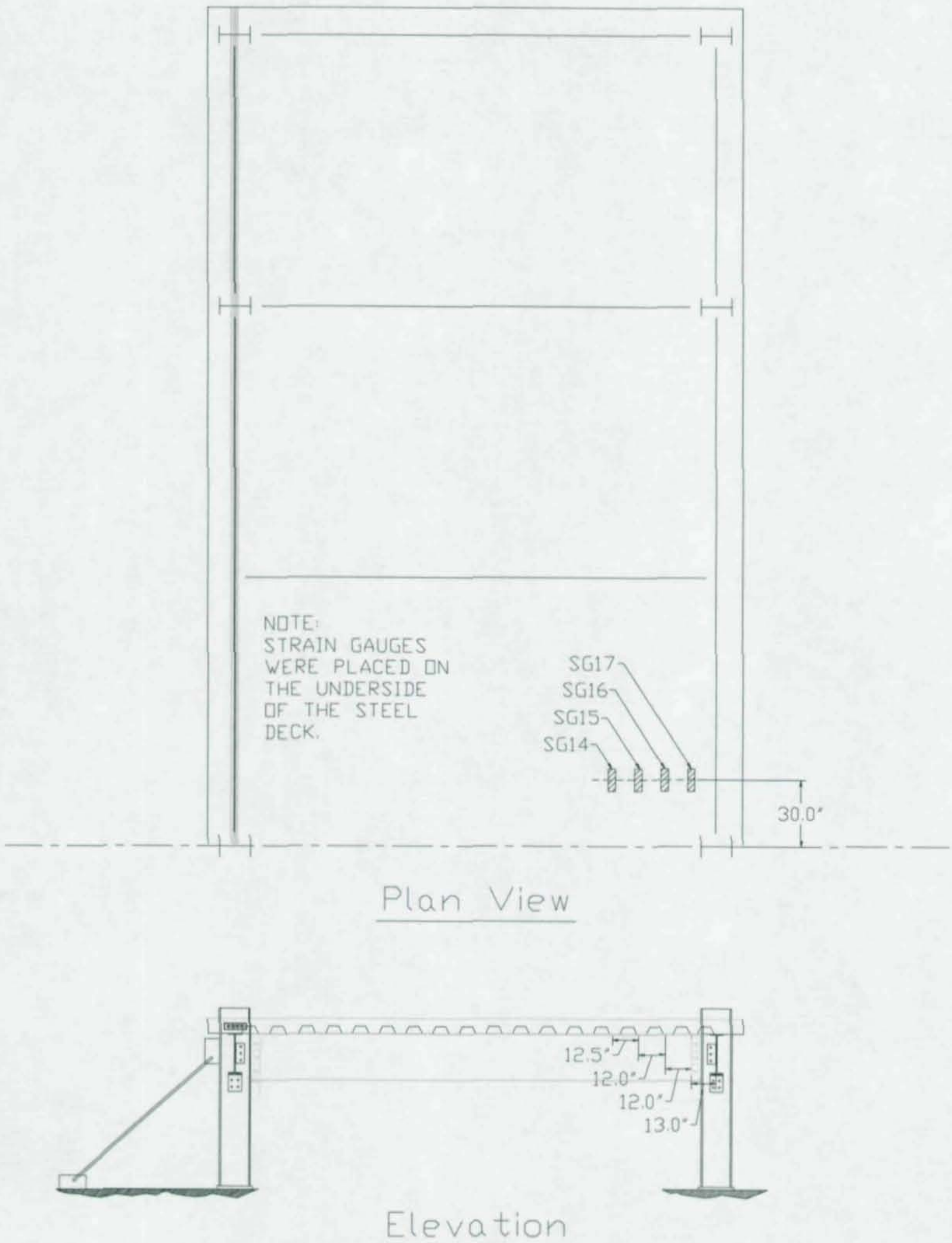


Figure 2.18. Layout of Strain Gauges 14 through 17.

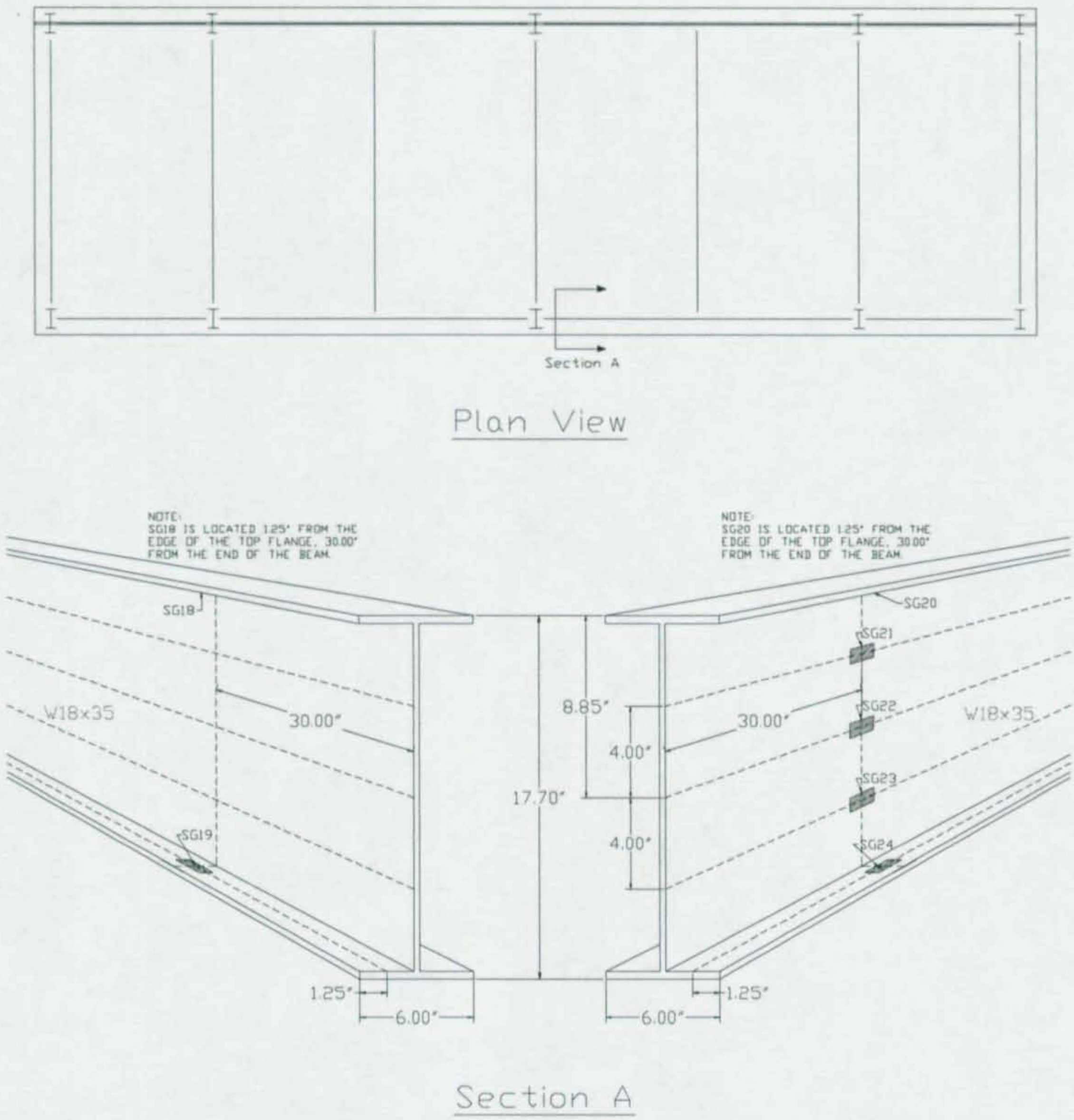


Figure 2.19. Layout of Strain Gauges 18 through 24.

2.6. Data Acquisition

The displacement transducers and strain gauges were attached to six data acquisition boxes. Each box was capable of monitoring eight channels. The instruments were attached such that only one of the two types of instruments led into each of the boxes. The acquisition boxes were subsequently connected to a Megadak data recording system. With a channel capacity of

56 and the capability to record 50 data points per second, the system provided sufficient data recording capabilities.

A second computer controlled the actuators during the experiments. The computer was capable of recording the force and displacement in each actuator, and a small number of additional channels. For this reason, several of the key instruments were also connected to the second computer to serve as back-up system. The data was recorded and processed later.

2.7. Problems Encountered in Instrumentation and Data Acquisition

Examination of the data revealed that the displacement transducers (DTs) numbered 4 and 19 both produced unexpected output. DT 4 was positioned just outside the test column and measured the vertical drop in the test column. The processed data plots of DT 4 reveal changes in the direction of movement. With the actuators displacing the column at a constant rate, the processed data was clearly unreliable. The transducer may have been incorrectly attached to the data acquisition system.

DT 19 was attached to the structure near the top of the test column, although the layout of the lab required that the transducer be secured to piping located on the ceiling of an adjoining sub-room. When securing the transducer to the overhead piping, it was believed to be sturdy enough to resist the small force needed to extend the wire. However, the processed data clearly shows that the transducer was displaced approximately 0.4 inches by this small force. This is evident because once the displacement of the test column was completed, the transducer showed a shortening of the connected distance by about 0.4 inches. Follow-up inspection of the pipes to which the transducer was attached confirmed the likelihood of the movement.

2.8. Test Procedure

Prior to the testing date, all instrumentation was installed, calibrated, and tested for proper operation. This included a check of the strain gauge resistances. The test consisted of three constant velocity loadings. The column was displaced at a rate of 0.25 inches/second to three different displacement levels. The column was displaced 19, 24, and 35 inches downward for the first, second, and third subtests, respectively. After each subtest, the bottom of the test column was returned to its original zero displacement position, 36 inches above the base level.

A complete time history of the displacement of the test column is included as Figure 2.20. The final displacement of each subtest was governed by the tests that were performed on the opposite side of the structure that contained the catenary cables. By performing the same tests on each side, one can evaluate the contribution of the catenary cables in resisting the increased load.

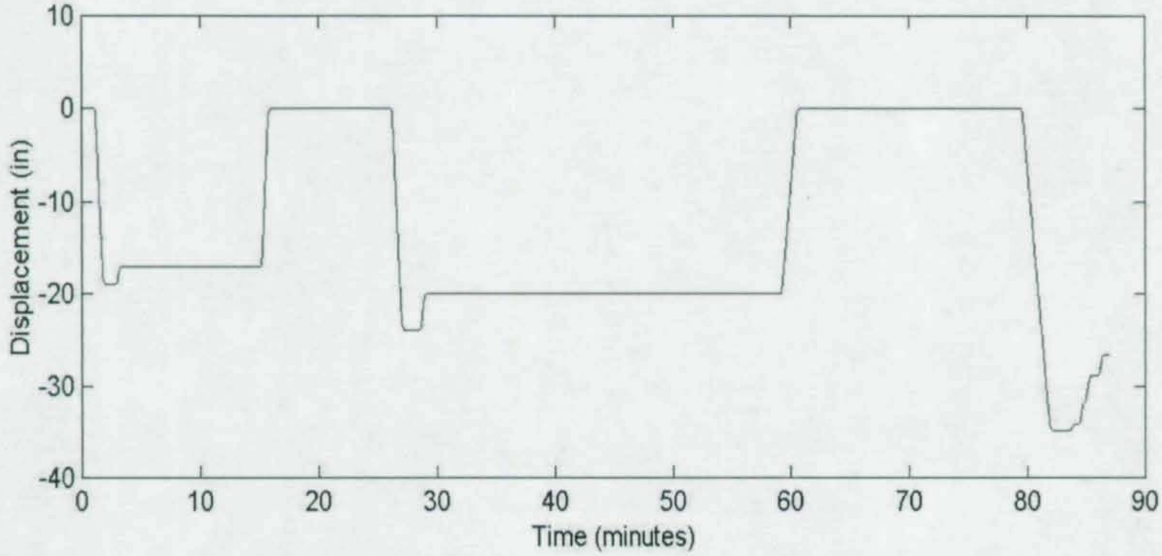


Figure 2.20. Recorded Time History of the Displacement of the Test Column.

The specimen was tested with the use of two 120,000-pound hydraulic-based testing machines located on the second floor of Davis Hall on the main campus of UCB. The tests were performed on June 14, 2001. A photograph of the test specimen can be seen in Figure 2.21 showing the loading actuators that were used to push the floor down. In order to remove the 36 inch support column, the actuators were raised a slight amount, leaving a gap slightly greater than 36 inch between the bottom of the column and the base of the floor. Once the column support was removed, the specimen was ready to be tested at the prescribed column displacements.

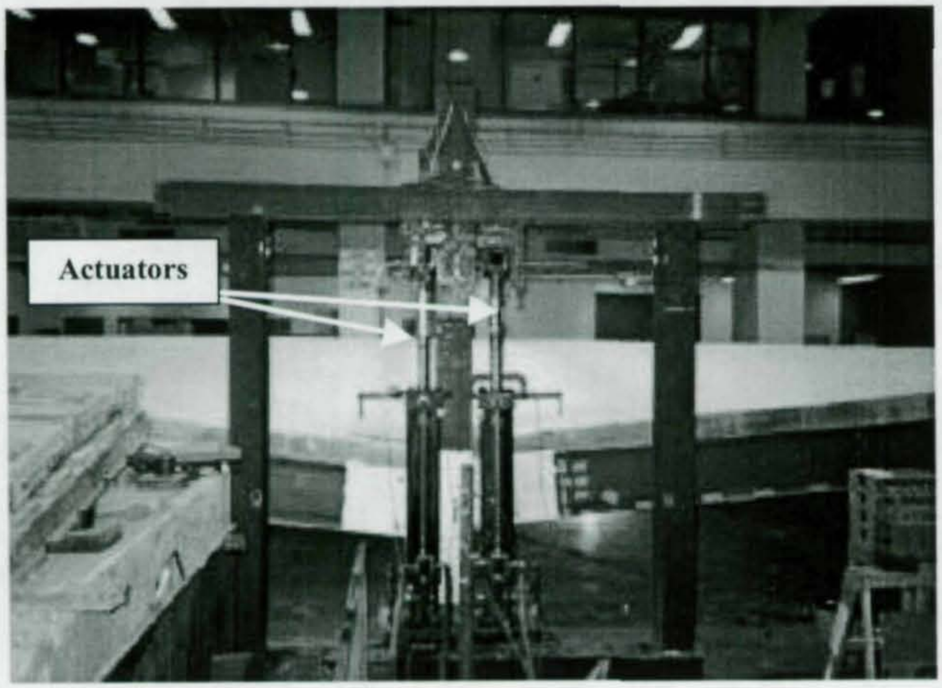


Figure 2.21. The Two Actuators and Loading Mechanism Used to Push the Column Down

CHAPTER THREE

Observed Behavior and Test Results

This chapter summarizes the test results and observed behavior of the test specimen. A complete set of test results and data recorded during the tests is provided in Appendix B.

3.1. Test 2A (0 to 19-inch Displacement)

The first test was conducted at 2:50 PM on June 14th, 2001. The test column was displaced 19 inches in this test, identified as Test 2A. The force time history and the force displacement plots can be seen in Figure 3.1. The total force was calculated as the sum of both actuator forces.

As the column displacement exceeded 14 inches, failure occurred of the top two bolts on the lower leg of the seat angle that holds the longitudinal beams. This is identified as spike "A" in both the force time history and force displacement plots in Figure 3.1. The end of the bolts, including the nuts, violently shot through the air while the head of the bolts remained in place. Four bolts held the angle to the displacing column, leaving only two bolts to resist the vertical load of the beams. As the bolts were failing, the seated angle itself was undergoing local yielding and deformation. See Figure 3.2 for a photograph of the local damage.

As the displacement approached 15 inches, at the other end of one of the longitudinal beams, west of the displaced column, the two bolts that secured the beam to the top of the seated angle also failed and "popped" (Figure 3.3). Spike "B" in Figure 3.1 clearly shows the loss of strength because of the bolt failures. The shear tab that supported the transverse beam also underwent local yielding.

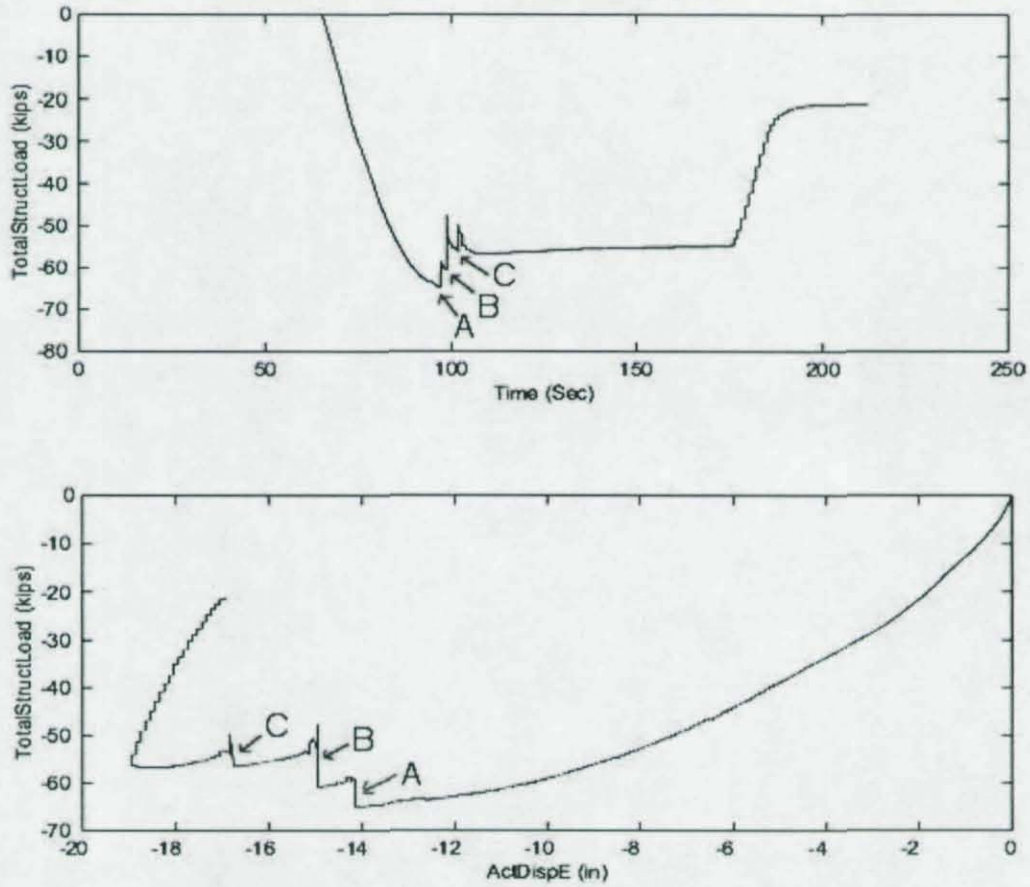


Figure 3.1. Force Time History and Force versus Displacement for 19-inch Drop Test

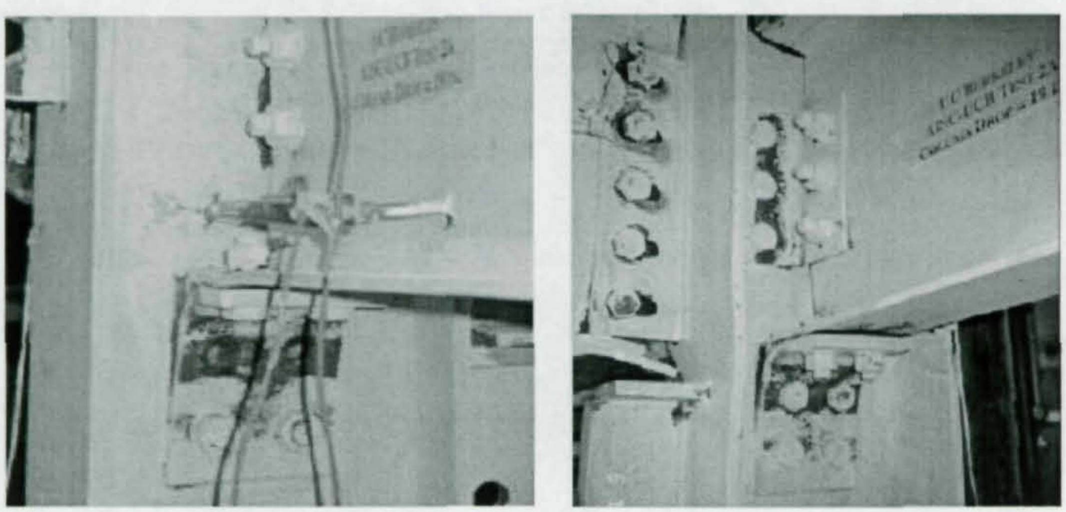


Figure 3.2. Failure of Top Two Bolts in the Seat Angle

As the displacement approached 17 inches, slippage at some point within the structure occurred. This is evident by examining spike "C" in Figure 3.1. Although the load carrying capacity of the structure was abruptly decreased at that point, the load gradually increased to the same value as before the spike. A failure would result in an unrecoverable decreased capacity, whereas slippage only resulted in a temporary decreased load carrying capacity.

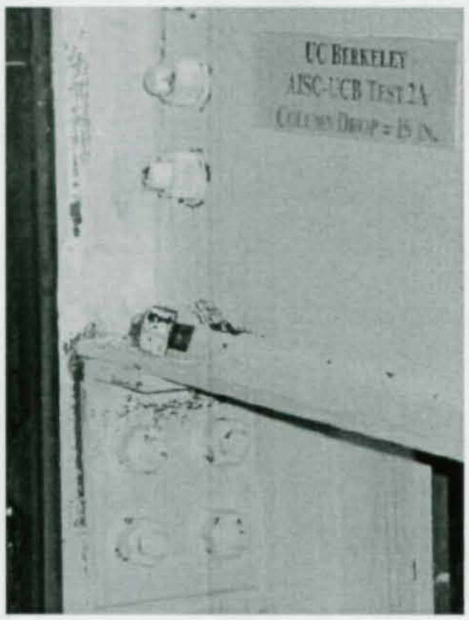
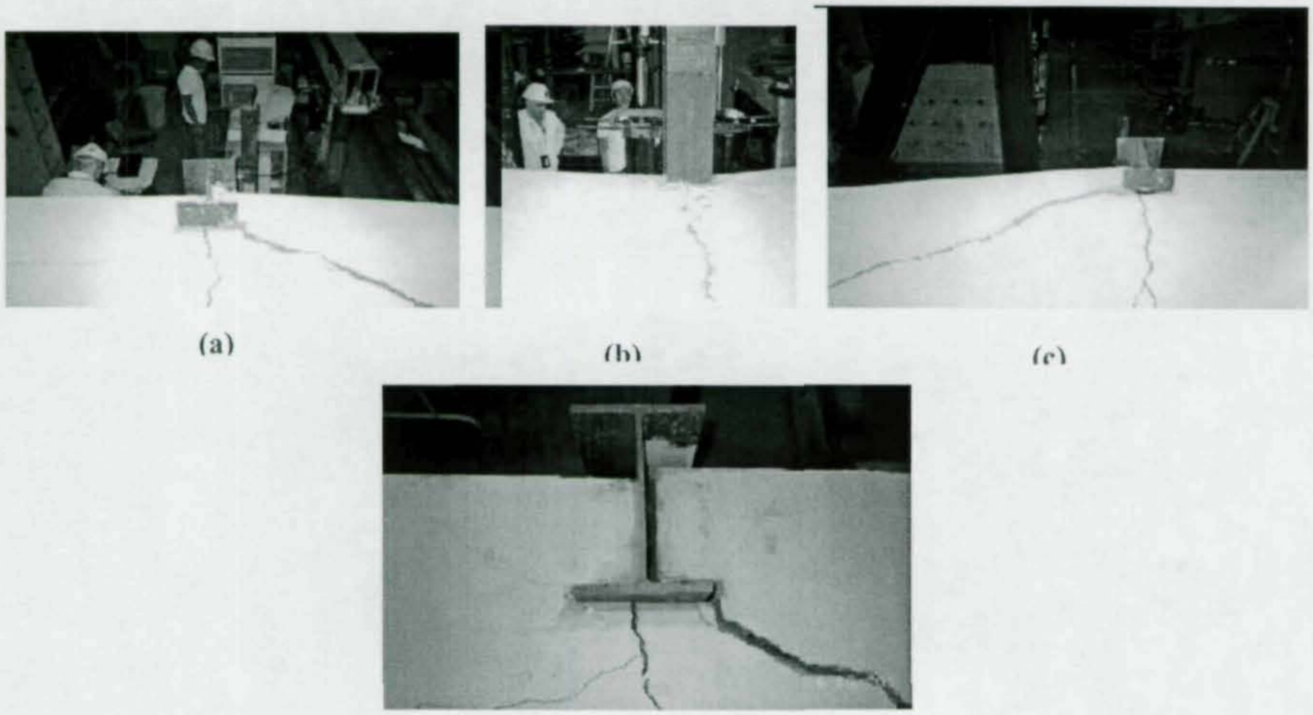


Figure 3.3. Failure of the Two Bolts on the Seat Angle

The concrete slab separated from certain columns as the displacement increased. At the final displacement of 19 inches, a gap of approximately 1 inch had opened between the slab and column B2. Photographs of the local damage surrounding columns B2, C2 and D2 are included as Figures 3.4(a), 3.4(b) and 3.4(c) respectively. Figure 3.4(d) shows close-up of the cracks around column B2.

After the column reached a displacement of 19 inches, the displacement was held constant for approximately one minute before returning to a displacement of 17 inches. The total sustained force, including dead load, at the final displacement of 19 inches was 54.0 kips. While the structure was displaced 17 inches, the research team inspected and made notes of the relevant deformations. Approximately 10 minutes later, the column was returned to the original height, corresponding to zero displacement.



(a) (b) (c)

(d)

Figure 3.4. Cracking of the Concrete Slab at Column B2 at the End of 19-inch Drop Test

Next, the two bolts that failed at the loaded column were replaced. Due to the seated angle deformation noted above, a ¼ inch gap remained along the shaft of the bolt after the nut was tightened.

The most notable behavior observed in the 19-inch displacement was the failure of two sets of bolts. The first to fail, located at the seated connection of the displacing column, revealed the limited rotational flexibility of the beam-to-column connections. The bolts failed when the longitudinal beam rotated 3.2° relative to the displaced column. Immediately following, the bolts also failed at the other end of the beam directly west of the displacing column. This second set of bolts connected the longitudinal beam to the seated connection. Although these bolts did not provide any direct vertical load carrying capability, the catenary action of the displacing frame was lost.

A maximum micro-strain of 1450 was measured on the steel deck during the displacement. The corresponding maximum stress was calculated to be 42.1 ksi. Four linear strain gauges were placed on the deck, 30 inches from the centerline of the structure. The highest strain was measured at the innermost strain gauge, with strains generally decreasing as location approached the edge of the structure. The notable difference was the second innermost strain gauge, which was placed about an inch from the end of a steel deck segment. Being so close to a free edge, the deck was not able to develop the relatively large strains of the adjacent strain gauge locations.

The outermost strain gauge recorded the lowest strain value. Post-experiment inspection of the structure revealed significant crushing of the rib on which the gauge was located. The local crushing was concentrated above the transverse beam connecting the two center columns. It is believed that this local crushing was responsible for relieving the stress in the rib containing the outermost strain gauge. As a result, the strain gauge, located over two feet away from the local crushing, recorded low levels of strain. Also, the high stiffness of the longitudinal beam located just 13 inches away may have reduced the strain in the outermost gauge.3.2.

3.2. Test 2C (0 to 24-inch Displacement)

The second test was conducted at 3:15 PM on June 14, 2001. The test column was displaced 24 inches in this test, identified as Test 2C. The force time history and the force displacement plots can be seen in Figure 3.5. The total load in Figure 3.5 was zeroed at the beginning of the test in the data processing stage.

As the column reached a displacement of approximately 19 inches, the two bolts that held the longitudinal beam, east of the loaded column, on the seated connection at column C2 failed. The dramatic drop in load is evident as spike "D" in Figure 3.5. Local yielding of the seated connection could also be observed. Also, cracking and tearing of the connecting angle between the loaded column and the longitudinal beam directly east had begun.

As a result of the two bolt failures, large eccentricities were created as the east actuator shot back to zero load while the west actuator was still providing 20 kips of load to provide the constant displacement. At a displacement of 24 inches, the west actuator was applying approximately 45 kips of downward force while the east actuator was applying approximately 10 kips of upward force.

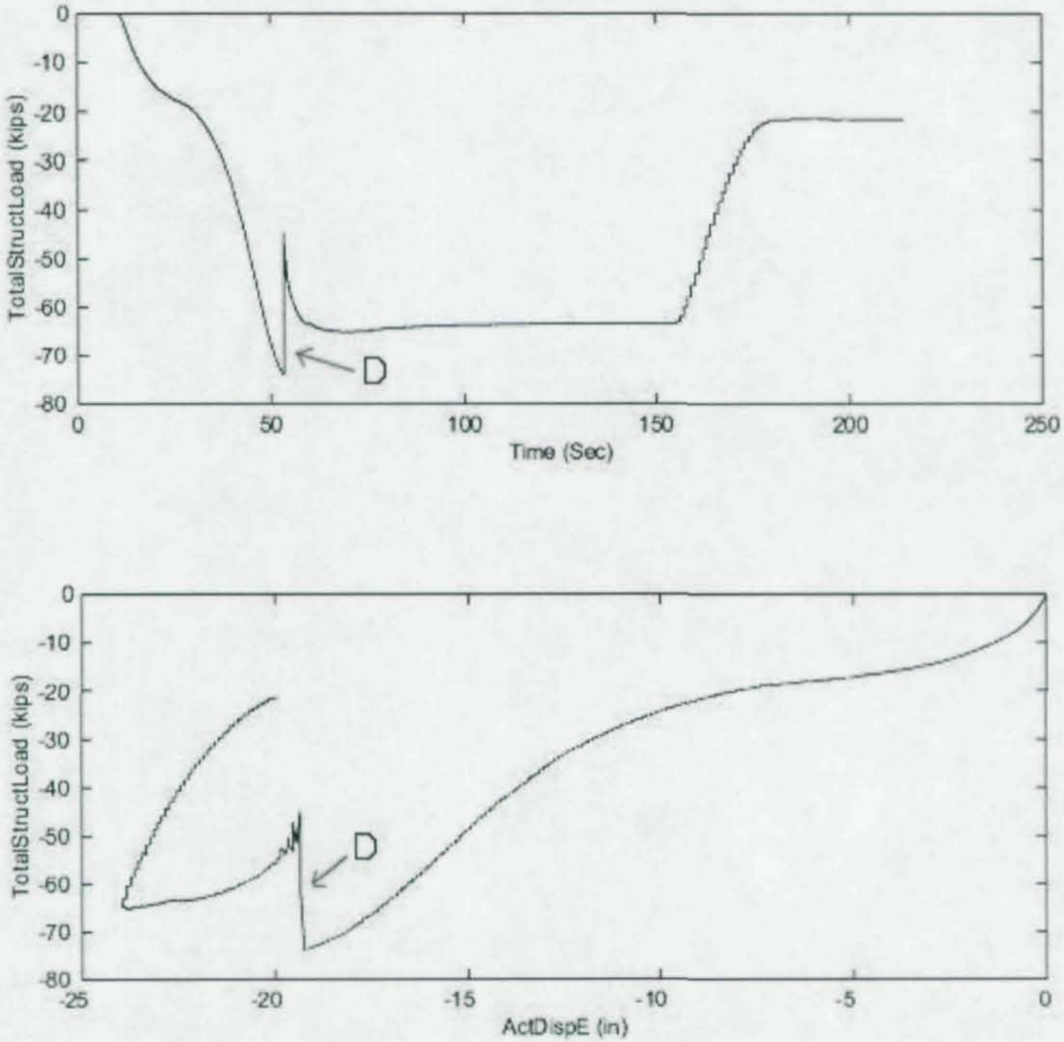


Figure 3.5. Force Time History and Force-Displacement Plot of 24-inch Drop Test.

The steel deck suffered noticeable damage. Major deformations were noted along the column line of the displaced column, with concentrated damage occurring near the beam to column connections. The steel deck was ripped near columns B2 and D2, while bending was widespread.

After the column reached a displacement of 24 inches, the displacement was held constant for approximately one and a half minutes before returning to a displacement of 20 inches. The total sustained force, including dead load, at the final displacement of 24 inches was 62.8 kips. While the structure was displaced 20 inches, the research team inspected and made notes of the relevant deformations. Approximately 10 minutes later, the column was returned to the original height, corresponding to zero displacement.

Figure 3.6. The Two Replaced Bolts on the Top Half of the Bottom Leg of the Seat Angle Remained in Place.

While attempting to replace the same top two bolts on the seated angle connection that were replaced before, it was noticed that the bolts did not fail, but underwent large deformations. The bolts were removed, but could not be replaced with new bolts due to the large deformations.

Only one failure occurred during the column displacement to 24 inches. The pair of bolts failed that secured the longitudinal beam directly east of the displaced column to the seated connection. Unlike the beam west of the column, this failure occurred at the end connecting into the test column. Both failures left the bottom flanges of the beams free to move at one end. As a result of the asymmetrical failure, the structure was no longer behaving in a symmetrical manner. The east actuator was actually applying a 10 kip upward force to the column even though the column was being displaced downward. Strengthening the bolts to eliminate their failure should produce a more balanced, well-behaved failure mechanism.

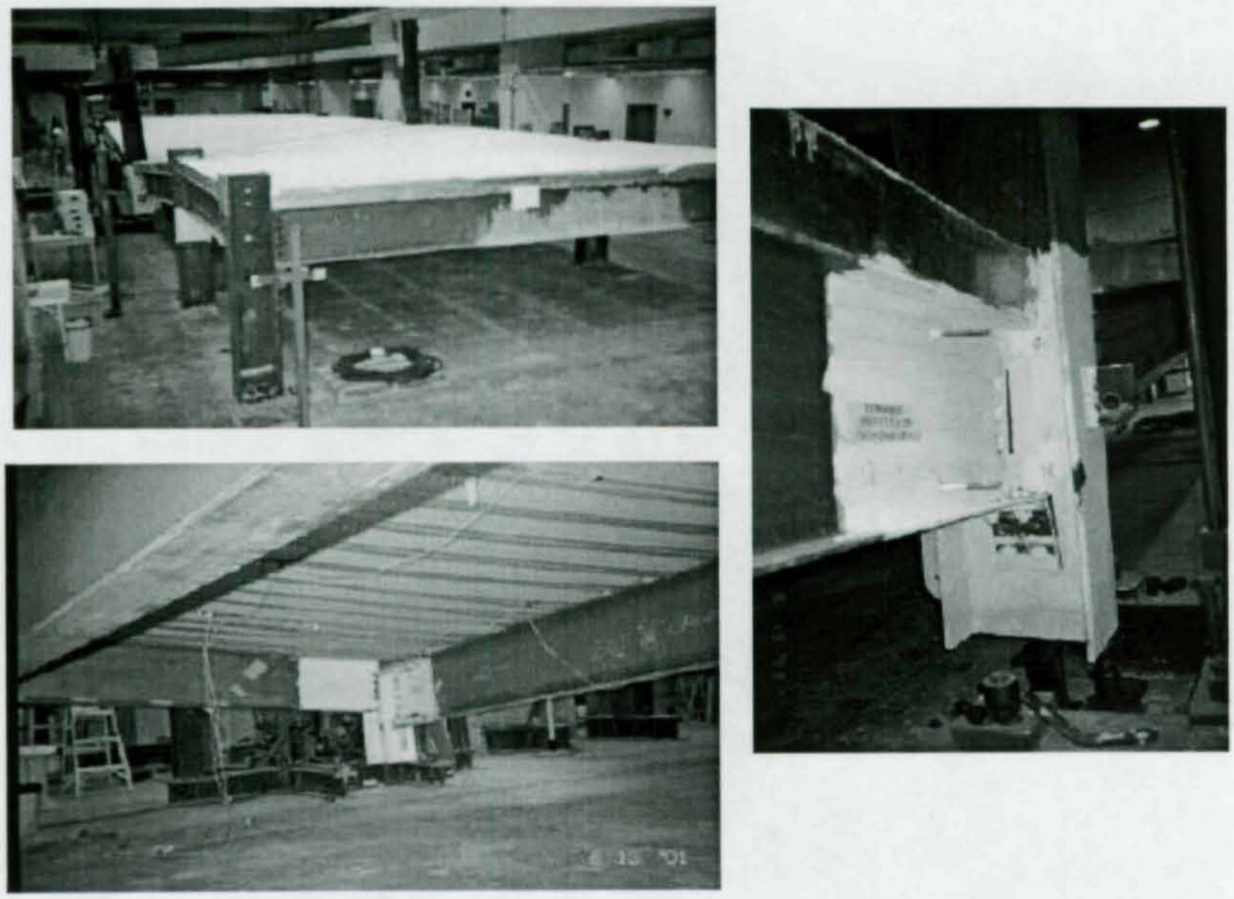


Figure 3.7. Specimen at the End of Test2C (24 inch Column Drop)

The force-displacement plot (Figure 3.5) shows a gradual stiffening of the structure once the displacement passes 5 inches. This results from the replacement of the two bolts in the seated connection at the displaced column. The ¼ inch gap that remained after the replacement affected the behavior of the structure. As the bolts adjusted to the gap and engaged themselves, the structure became stiffer.

A maximum micro-strain of 1560 was measured during the displacement. The corresponding maximum stress was calculated to be 45.2 ksi. The strains for this second subtest include any residual strains that existed within the gauges after the column was returned to a position of zero displacement. As before, the innermost of the four linear strain gauges placed on the deck recorded the highest strain. The strain generally decreased as the edge of the structure was approached with the exceptions noted above.

3.3. Test 2D (0 to 35-inch Displacement)

The third test was conducted at 4:10 PM on June 14, 2001. The test column was displaced 35 inches in this test, identified as Test 2D. The force time history and the force displacement plots can be seen in Figure 3.8. The total load in Figure 3.8 was zeroed at the beginning of the test in the data processing stage.

In addition to the above noted behavior, as the column displacement approached 26 inches, complete failure occurred at the connecting angle between the loaded column and the longitudinal beam directly east (Figure 3.8). Spike "F" in Figure 3.8 shows the sudden drop in load after the angle failure. The preceding spike, labeled "E," is the evidence of slippage within the structure, possibly within the angle before complete failure.

After the angle had failed, the concrete slab was the primary element transferring the applied force to the longitudinal beam directly east of the displacing column. Unable to maintain this force transfer, the slab failed in shear along the transverse beam connecting the two central columns. This is represented by spike "G" in Figure 3.8. Figure 3.8 shows specimen after failure.

The concrete slab and steel deck suffered a great damage during the 35-inch displacement. The steel deck continued to tear, opening up gaps both within and between the deck segments. The damage was concentrated along the column line of the tested column. There was a separation of at least one inch between deck segments along the transverse beam connecting the two center columns. Near column D2, the deck tore, opening up a hole that measured over three inches between the steel deck and concrete slab.

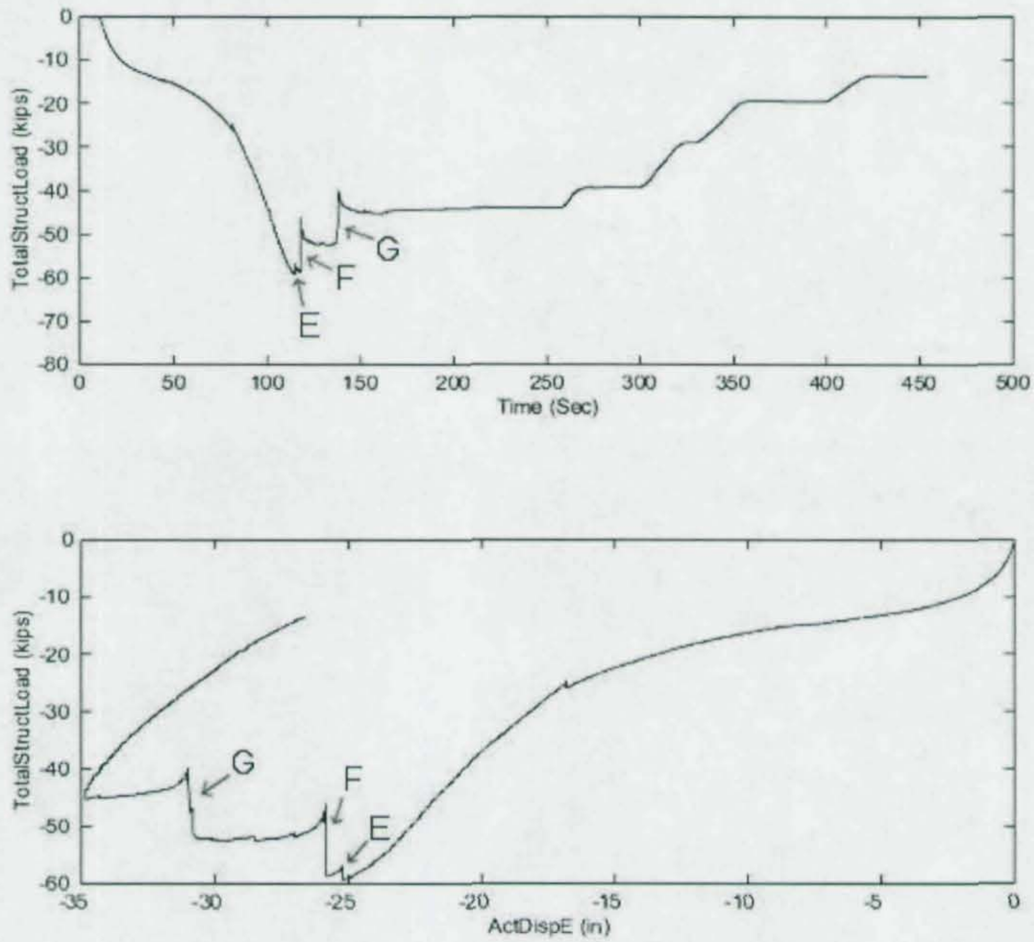


Figure 3.8. Force time history and force-displacement plot of Test 2D.

After the column reached a displacement of 35 inches, the displacement was held constant for approximately 2 minutes before returning to a position of approximately zero applied force. The total sustained force, including dead load, at the final displacement of 35 inches was 43.5 kips. The structure remained in the position of zero applied force, corresponding to a displacement of approximately 27 inches, for several days. While the structure was displaced 27 inches, the research team inspected and made notes of the relevant deformations. The following week, the column was returned to the original height, corresponding to zero displacement.

Additionally two component failures occurred during the 35-inch displacement. First failure was at the connecting angle between the test column and the longitudinal beam directly east. The concrete slab was left as the only remaining component to transfer the applied force of the actuators to the east side of the structure. Unable to provide this force transfer, the concrete slab failed in shear.

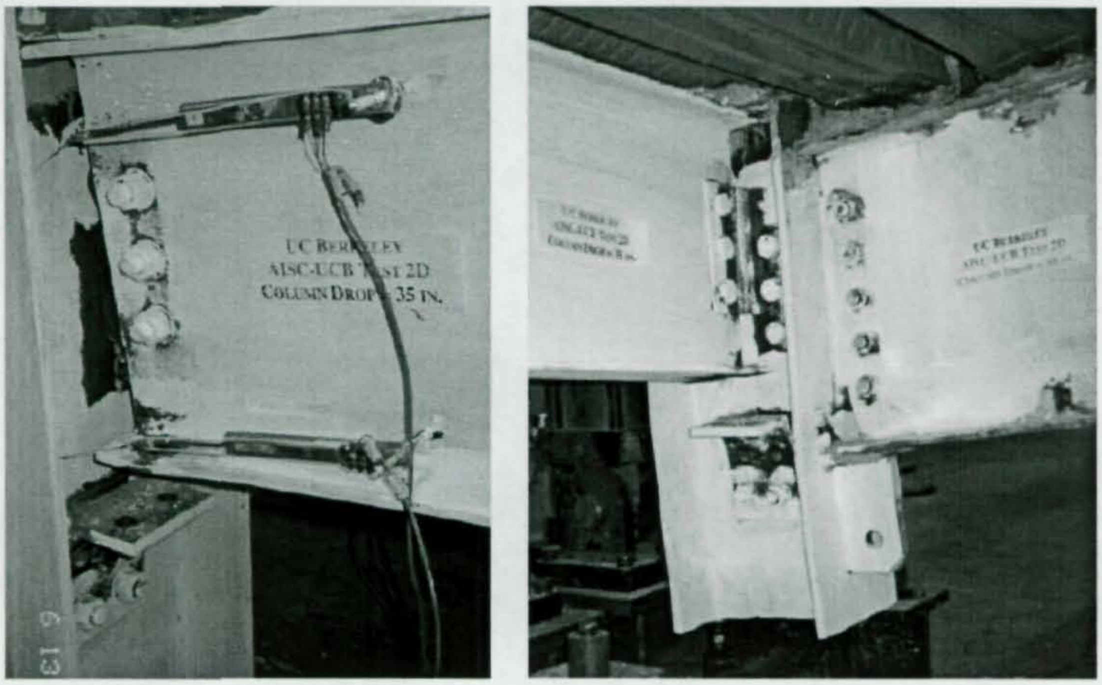


Figure 3.9. Complete Failure of the Shear Angle on the Web of Column

A maximum micro-strain of 1340 was measured during the displacement. The corresponding maximum stress was calculated to be 38.9 ksi. The gauge readings include any residual stress that remained in the deck after the column was returned to zero displacement following the previous test. Once again, the innermost of the four linear strain gauges placed on the deck recorded the highest strain. The strain generally decreased as the edge of the structure was approached with the exceptions noted above.

3.4. Summary of the Behavior and Results

Of the three subtests, the 24-inch displacement sustained the highest load of 62.8 kips. The 19-inch and 35-inch test sustained 54.0 and 43.5 kips of loading, respectively. The load-displacement plots of all three tests are included on one plot in Figure 3.10. As can be seen in the figure, a maximum load is reached at a displacement somewhere between 19 and 35 inches. Proximity of all five component failures can easily be seen, beginning with the initial bolt failures and ending with the complete transverse cracking of the floor slab. In addition, the stiffness degradation of the system is evident as the same load resulted in greater displacements in the following tests. The capacity of the structure was reached just before failure of the pair of bolts that secured the east beam onto the seated connection at the test column. This failure, occurring during Test 2c, resulted in the largest drop in applied load.

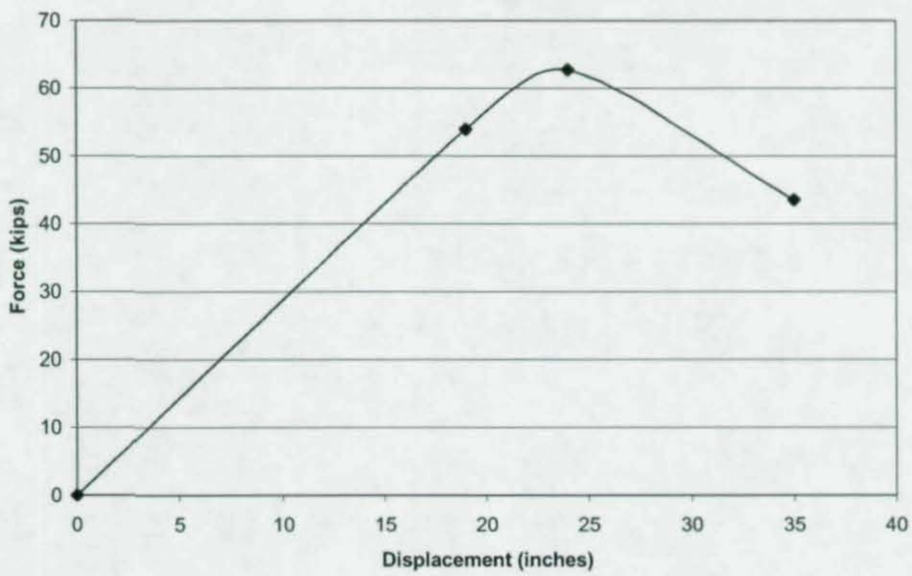


Figure 3.10. Force-Displacement Plot of the Three Tests.

CHAPTER FOUR

Conclusions and Recommendations

4.1. Conclusions

The following conclusions were reached by observing the behavior of the test specimen and by studying the test data.

1. The ultimate capacity of the structure following a loss of column is limited by the beam-to-column connections at the lost column. The catenary action of the beams and steel deck appear to be adequate to prevent a progressive collapse if connection bolts do not prematurely fail.

2. The steel deck is effective in redistributing the increased load resulting from the loss of a column. The maximum stress readings during the tests were mostly between 5 and 15 ksi, however, localized small areas of deck yielded. Failure of the steel deck through ripping or tearing was isolated and did not limit the strength of the system.

3. The combined Catenary action of floor steel deck and the simply supported girders was able to prevent the collapse of column with a load of about 63 kips in the column. This load corresponds to about 300 kips per square feet of tributary area of the floor. Considering a reduction factor of 0.5 due to impact, the floor load that was carried by Catenary action could be established as about 150 pound per square feet of tributary area.

4. This one test only established the potential of typical steel structures to resist progressive collapse in the event of sudden removal of a column. Further research is needed to

establish the parameters that affect this resistance and to develop appropriate design guidelines to take advantage of this phenomenon in preventing collapse of structures.

4.2. Comparison of Resistance of Steel Structure With and Without Cables

As mentioned in previous chapters, the tests reported here were conducted on a specimen that was also used to test the catenary action of cables placed in the floors. Four 1-1/4 inch diameter steel cables were placed in the slab along the south frame line. The south side was tested first and the results were reported in Astaneh-Asl et al., (2001). The north side, which did not have cables, was tested next and the results were reported in previous chapters of this report. Figure 4.1 shows a comparison of the behavior of two sides.

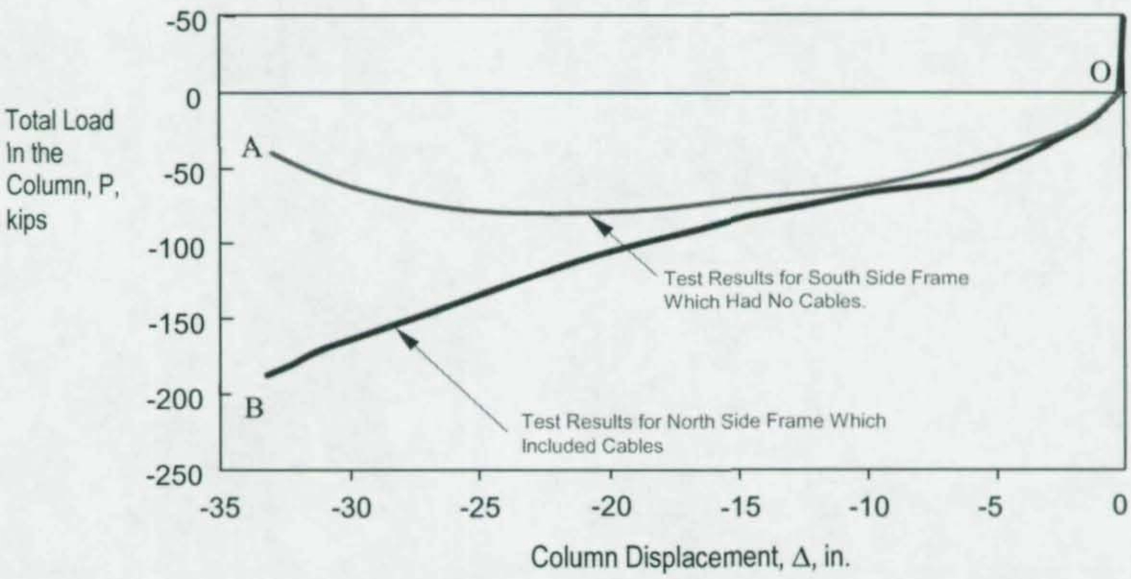


Figure 4.1. Comparison of Test of Specimen with and Without Cables Embedded in the Floor

4.3. Recommendations

The following recommendations are based on the observed behavior of the test specimen and by studying test data obtained. Each recommendation is directly related to one of the observed failure mechanisms.

1. Connection bolts should be strengthened to improve structure behavior in a loss of column scenario. The bolt failures observed in the experiments led to greater specimen

deformations and resulted in failure of a beam-to-column angle connection. High-strength bolts and/or larger sized bolts would improve the behavior of the system with little additional cost.

2. Placing reinforcement bars within the concrete slab could postpone and possibly eliminate complete cracking of the slab. The bars could be placed along the column lines of any frames susceptible to blast loads. The additional steel cross section should prevent complete shear fracturing of the floor slab.

3. The connections should be designed for combined effects of bending and axial load due to Catenary forces. Guidelines for design of such connections can be developed upon further research, experimental or analytical, and better understanding of magnitude of forces and rotations involved. The first author will continue efforts in that direction.

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