USE OF CATENARY CABLES TO PREVENT PROGRESSIVE COLLAPSE OF BUILDINGS

FINAL REPORT

By
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COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA AT BERKELEY

Report to the Sponsors: General Services Administration / Skilling Ward Magnusson Barkshire

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.

USE OF CATENARY CABLES TO PREVENT PROGRESSIVE COLLAPSE OF BUILDINGS

FINAL REPORT

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This document presents the results of the tests of a mechanism that can be used to prevent progressive and catastrophic collapse of steel structures in the event of a blast attack and elimination of one of the exterior columns. The concept that was tested and verified is proposed by the Skilling, Ward, Magnusson, Barkshire (SWMB), Structural and Civil Engineers, Seattle to be used in a new building. The concept consists of placing horizontal cables in the floors and on top of the top flange of the girders along the exterior column line. By using Catenary action of the cables, the load of the eliminated column can be transferred to the rest of the structure.

The test specimen was full-scale representative of one floor of a typical steel building structure with its floor slab, steel deck, supporting beams, girders and columns. The specimen was designed by the SWMB and the test set-up was designed jointly by SWMB and UC-Berkeley engineers and researchers. The size of the specimen was 19'x60'x6'. The test plans consisted of constructing the specimen inside the UC-Berkeley, Civil Engineering laboratory in Davis Hall, adding instrumentation to the specimen, removing a middle column, pulling the bam-column joint of the removed column down, observing and collecting data on performance of the structure after removal of the column. A total of four tests were conducted which were 19.8-inch, 21-inch, 24-inch and 35-inch drop of column. The tests indicated that after removal of the column the Catenary action of the cable-supported floor was able to support 110 kips, 140 kips, 160 kips and 190 kips of column load respectively for 19, 21, 24 and 35 inches of drop of the column joint. These load values were based on using hydraulic actuator to apply the load. The corresponding gravity loads that could be applied to the column were 85, 108, 123 and 146 kips.

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ABSTRACT

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ACKNOWLEDGMENTS

The study reported here was supported by the General Services Administration (GSA) of U.S. Federal Government and by Skilling Ward Magnusson Barkshire (SWMB), Structural and Civil Engineers, Seattle. The authors wish to express their appreciation for the support as well as technical advice and input received from the GSA and SWMB engineers. Particularly, the support, technical input and comments received from Willie Hirano and Bela I. Palfalvi of GSA and Brian Dickson, Kurt Nordquist and Jon Magnusson of SWMB were very useful, essential to success of this project and are sincerely appreciated. However, the opinions expressed in this report are those of the authors and do not necessarily represent the views of the sponsors, the University of California or other agencies and individuals whose name appear in this report.

The tests reported here were conducted in the Department of Civil and Environmental Engineering of the University of California at Berkeley. The research was administered by capable and professional staff of the Institute of Transportation Studies, an Organized Research Unit of the University of California at Berkeley. Their support was very valuable and appreciated. William Mac Cracken, Dr. Lev Stepanov, Chris Moy, Frank Latoya, Larry Baker, Richard Parsons, Douglas Zulaica, Mark Troxler, all staff of the Department laboratories participated in development and in conducting the test as well as in data acquisition. Without their dedication and professional work, this project would not have been possible. Qiuhong Zhao, doctoral student, was involved in various aspects of this large scale testing. Her efforts are appreciated. Professor Yongkuan Zhao of Zhaoqing University of People's Republic of China who was a visiting scholar to Professor Astaneh-Asl also contributed to this project significantly. Some members of the research team as well as Sponsors' engineers are in Figure 0.1.

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, project engineer with Herrick Corporation are sincerely appreciated. The steel deck used in the specimen was donated by Verco Manufacturing Co. The shear studs and their installation were donated by Nelson Stud Co. These generous donations are acknowledged and appreciated.

Figure 0.1. Members of Research Team with Sponsors' Engineers

(From Left: Y. Zhao, B. Dickson, W. Hirano, Q. Zhao, W. Li, C. Moy, W. Mac Cracken, E. Madsen and A. Astaneh-Asl)

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CHAPTER 1

Introduction

1.1. Introduction to Blast Effects on Civilian Buildings

Car bomb attacks during the past several years by terrorists against public and privately owned buildings have brought about a new awareness in the structural engineering community and construction industry: How to design structures to survive terrorist attacks with possibly no loss of life and minimal injuries. Many military structures have been designed to resist explosive attacks. However, considering blast loading in the design of civilian building structures in the United States is a relatively recent idea. Three levels of activity can provide protection against a terrorist attack. These are shown in Figure 1.1 and are: (a) Intelligence gathering to identify the threat and prevent it from materializing, (b) limiting access and using barriers to prevent the explosives (e.g. car bombs) from being delivered to within proximity of the building and (c) designing the structure to prevent loss of life, serious injuries and progressive collapse.

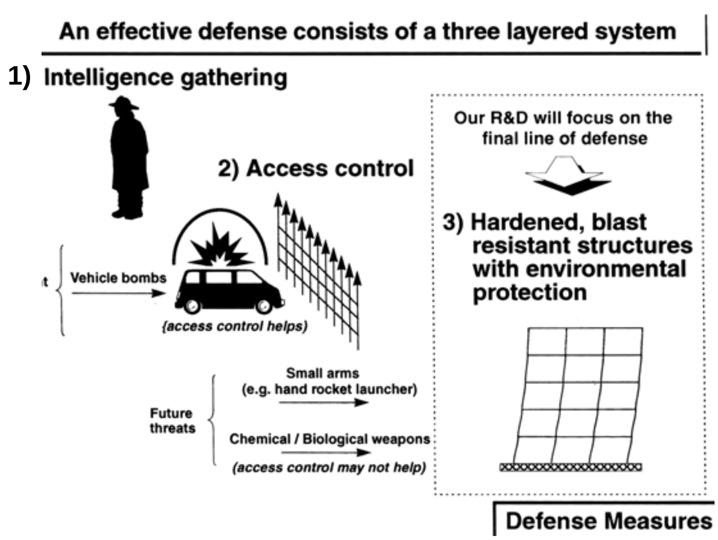


Figure 1.1 Three Layers of Protection Against a Terrorist Attack with Explosives

While the type of loading may be the same for a military structure and a civilian office building, both being external blast load, the magnitude of the load as well as architectural and economic aspects of a blast-resistant military structure are quite different from a civilian structure. In most cases, with a military building, the operations being performed inside the structure must not be stopped in the event of an attack. Therefore protecting the structure against an explosives attack is vital, and the structure is designed accordingly to be heavy, solid, massive bunkers with few openings.

With civilian buildings, however, these military-style attributes are in almost all cases totally unacceptable to architects and users. By its nature and function, a military building has a higher probability of being exposed to an explosives attack than a typical civilian building. As a result, the explosion force can become the governing design force for some highly exposed military installations particularly human and equipment shelters. Also in many cases, military equipment and personnel need to be protected inside a shelter that can withstand explosions from the outside. In these military cases, one can afford to spend necessary funds to design the structure to protect the contents against external explosive attack. On the contrary, in civilian facilities the probability of an explosive attack is quite low and usually is in the form of a terrorist attack. Obviously, considering the very low probability of a particular civilian structure being exposed to terrorist attack, one cannot afford to design all civilian facilities to withstand attack with explosives. Therefore, the first step is to identify certain buildings and facilities that are critical for the functioning of society and its vital organizations such as the government, infrastructure and public facilities. In recent years, most Federal buildings in the United States and overseas have been categorized to be “hardened” to withstand terrorist attacks.

The number of current buildings that need to be evaluated and new buildings that need to be designed to withstand terrorist attack without collapse is still very high. This poses a challenge to the structural engineering and construction community to refine available technologies and to develop new technologies to ensure that the “progressive collapse” of a building attacked by a terrorist bomb can be prevented.

The mechanisms to prevent collapse should:

1. Be economical and easily constructible.
2. Be architecturally acceptable and if possible be invisible altogether.
3. Add to strength and resistance of the building to other loads such as seismic effects.
4. Be proof-tested before its use to ensure that it will actually work when it is called upon to prevent progressive collapse.

In developing a collapse prevention mechanism, the structural engineer is faced with many challenges. With the actual threat of any given building being subjected to a bomb blast being very low, architects and owners quite often choose aesthetics over protection from an event deemed unlikely. Thus, the basic performance criterion for a civilian building is one of protecting life, by limiting the effects of the explosion to a small area and by allowing for quick evacuation and rescue. The design engineer must be willing to accept some damage while designing the structure such that partial or total collapse is prevented. Of course if one desires and is willing to bear the costs, similar to military buildings, civilian buildings can also be designed and constructed such that they can withstand a well-defined explosive attack with minimal damage to the building and minimal impact on the occupants and contents.

The actual effects of an explosion on a building are very complex. The amount of damage sustained is a function of the explosive size, distance from the building or location within the building, and the layout of any surrounding buildings. With an explosion occurring outside a building, the windows, doors, and façade on the perimeter are the first to be damaged, followed by external beams and columns. When a portion of the exterior fails, such as windows breaking, the blast pressure is allowed into the interior where more structural damage can occur. The failure of structural members will create, at the least, localized structural collapse and may even propagate progressive collapse of the structure. The Oklahoma City bombing was a case where failure of just a few structural members caused structural collapse of a large portion of the building.

Terrorist bombings can also occur inside a structure since security in most commercial buildings is quite lax. Internal explosions create much more structural damage and injury than external explosions. The effect of the blast being confined causes larger pressures as the blast waves reverberate through the structure. This creates a longer duration loading that can cause much more damage. Bombs may also be placed closer to key structural members from the inside than from outside a building. However, with today's security measures in critical buildings the likelihood of internal intentional blasts is very low.

1.2. Performance Based Blast-Resistant Design

In performance-based design, structures can be designed to perform in a predefined manner when subjected to a predefined loading or displacement. Of course the reliability of the design will depend on the reliability of the loads and/or displacements applied to the structure. In case of blast-resistant design, the loads and displacements imposed on a structure during and after a blast are not well known. The explosion, being a highly dynamic phenomenon depends not only on the character of the source of explosion but also on the dynamic interaction of the structure, its support and the explosion effects. Also, due to security reasons, the information on actual forces and other parameters related to explosions are not openly available to the general public. Therefore, without knowing the forces that a structure will be subjected to, it is difficult at this time to create a full-fledged performance-based blast-resistant design. One frequently used approach is to design structures for typical code specified design loads such as gravity, wind, seismic and then evaluate the designed structure for progressive and catastrophic collapse. Then, if such possibilities exist, devise concepts and technologies that can prevent progressive collapse.

In summary, in today's economy and considering the relatively low probability of a given civilian structure being subjected to a terrorist attack over its expected life, the blast-resistant design is focused on *preventing progressive collapse*. By avoiding progressive collapse it may be possible to limit the deaths and injuries to a relatively small area of the structure adjacent to the explosion.

1.3. Trends in Blast Resistant Design

While there are relatively detailed design guidelines for military structures and embassies to resist blast loading, the information for blast-resistant design of civilian facilities is very limited. To reduce the effects of an external bomb blast an easy way is to increase the distance between the structure and the explosion. In addition barriers, disguised as planters and artistic features can be used to block some of the blast pressure. However, large buffer zones are often not practical in crowded urban cities where space is tight. Therefore, the design engineer must expect some structural failure to occur while preventing overall collapse.

Some engineers have proposed that designing a structure for seismic zone 4 earthquake standards will provide enough ductility for the structure to withstand an explosion. However, this is probably not correct. Studies have shown that the Oklahoma City Federal Building would have failed even if designed for seismic zone 4 criteria. Blast loading is different enough from earthquake loading that a separate analysis and design procedure is required to protect against an explosion.

Careful attention must be given to all aspects of design to withstand a bomb blast. The first line of defense against a bomb blast is the exterior of the building. The façade, including windows, should be strengthened to reduce the chance of failure while still maintaining the aesthetics of the building. Exterior walls may need to be designed as structural members, not just cladding. Energy absorbing material may be used to protect structural elements. Main structural members should be designed to provide ductility especially in areas susceptible to explosions such as the lower floors and underground parking garages. Columns need to be designed to withstand the large bending forces created by blast pressures. This is especially important for gravity columns designed only to carry axial load. Columns must also be designed for a possible tension load resulting from upward pressure acting on the floor slab above. Slabs should be designed to carry this upward blast force in addition to a downward force caused by suction following an explosion. Detailing should be sufficient to prevent punching shear and slab failure around columns resulting in an increased unbraced length, which could possibly cause buckling failure of the column. Of course, a detailed cost analysis must be preformed to determine whether these strengthening methods are cost effective to the owner given the rarity of such an attack.

1.4. Current Research in U.S.

Most research into blast resistant design to date has focused in the areas of a better understanding of blast loading, more advanced analysis methods, and strengthening structural elements of a building. Much of this research has been analytical with the few experimental projects focusing on blast effects of small parts of the building such as high strength shatterresistant glass and energy absorbing façade materials. While the military has extensive data on blast effects of military-style structures, very little data exists on blast effects on civilian commercial buildings.

1.5. Objectives of the Tests at UC-Berkeley

The main objectives of the test series were:

1. To test the ability of proposed cable system to support imposed loads.
2. To test the ability of critical connections to carry imposed loads
3. To establish performance of the cable connections to the beams and columns
4. To establish rational design criteria for the catenary cable system.

CHAPTER 2

Geometry and Properties of Test Specimen

2.1. Test specimen

The test specimen is shown in Figure 2.1. It was designed by Skilling Ward Magnusson Barkshire to ensure that the specimen is a realistic full-size representative of a portion of the first floor of a building where this system is being implemented. More information about design of the test specimen is provided in Chapter 3. The specimen, shown in Figure 2.1 and 2.2 was representative of a portion of the first floor of a medium-rise steel structure.

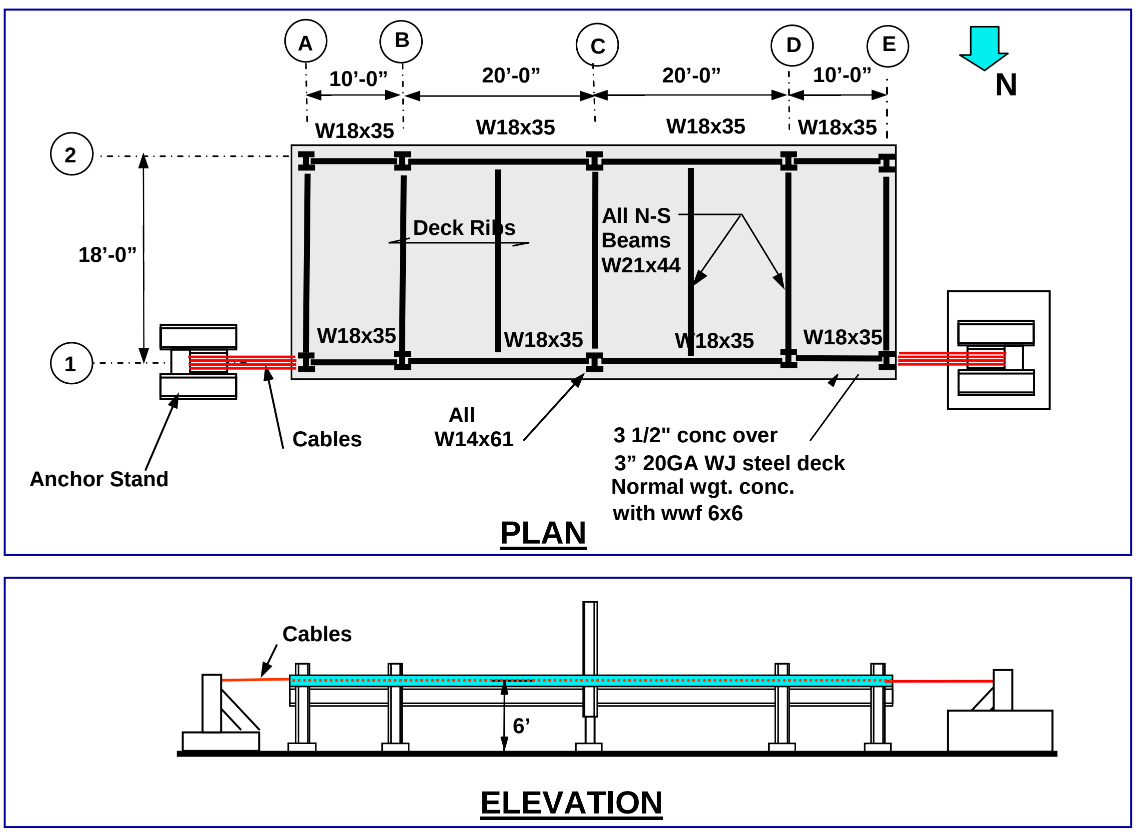


Figure 2.1 Test Specimens

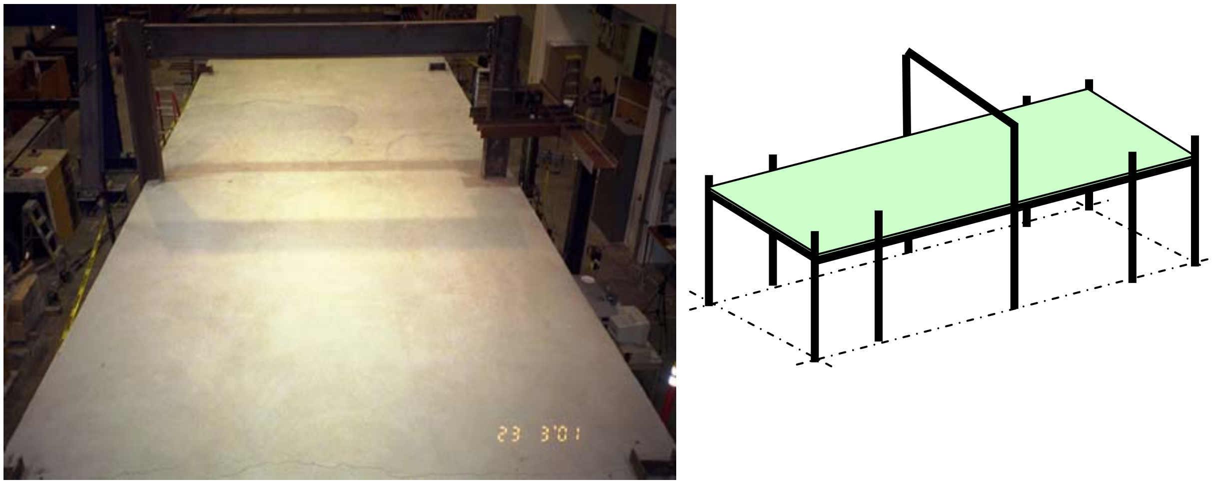


Figure 2.2. Specimen Representing a Portion of Actual Structural System

The specimen was a one-story structure with 18-ft bay in one direction and four bays (10'+20'+20'+10') in the other direction. The test structure had steel columns and beams and typical steel deck and concrete floor slab. The beam-to-column connections in E-W direction were seat angles bolted to bottom flange of the beam and web of column and a single angle bolted to the web of the column and beam. The connections in N-S direction were shear tabs with long slotted holes. The long slotted holes were designed to accommodate rotations of up to 0.60 radians during the tests.

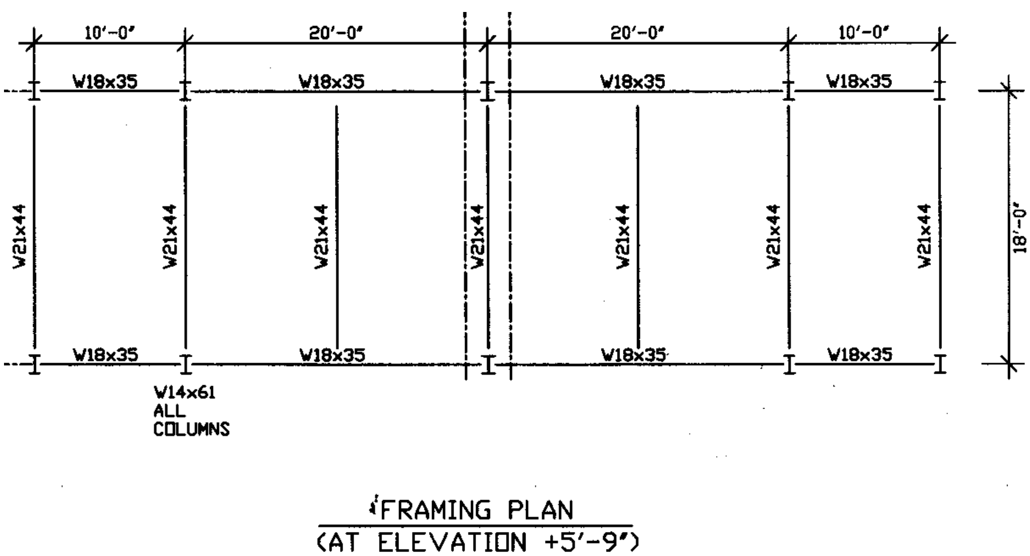


Figure 2.3. Framing Plan of Test Specimen

2.2. Details of Test Specimen

Details of the specimen are shown in Figures 2.3 and 2.4. Complete sets of structural drawings and shop drawings of the test specimen are provided in Appendix A.

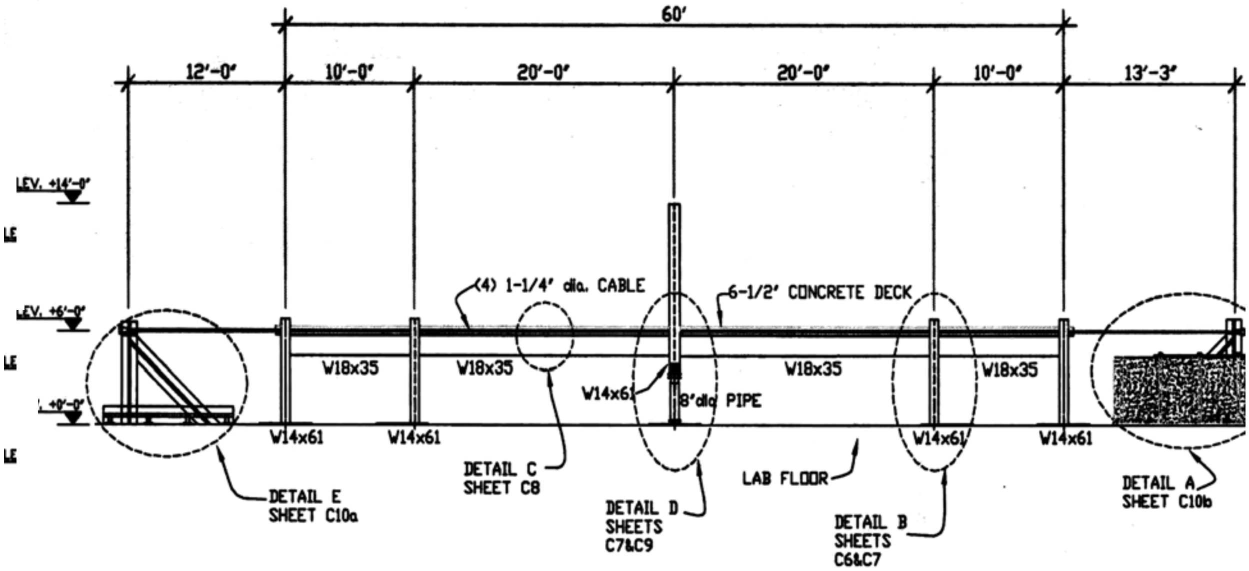


Figure 2.4. Elevation of the Test specimen

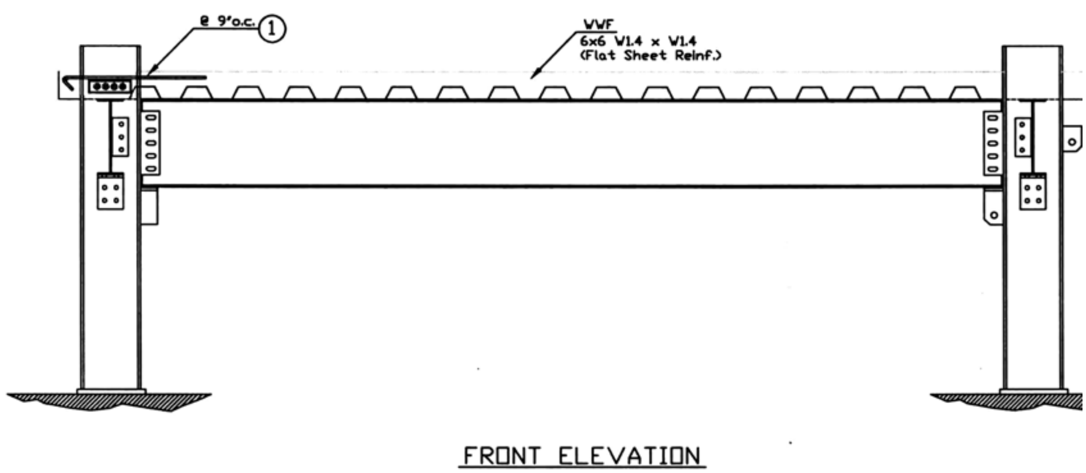


Figure 2.5. Cross section of Test Specimen

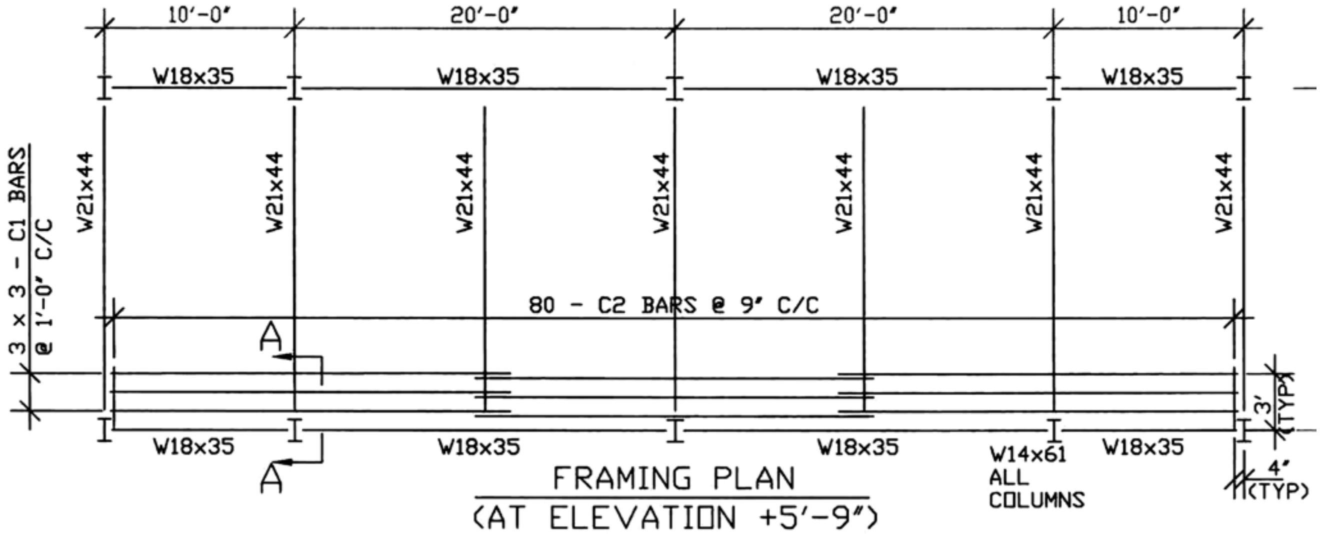


Figure 2.6. Plan of Shear Stud Placements

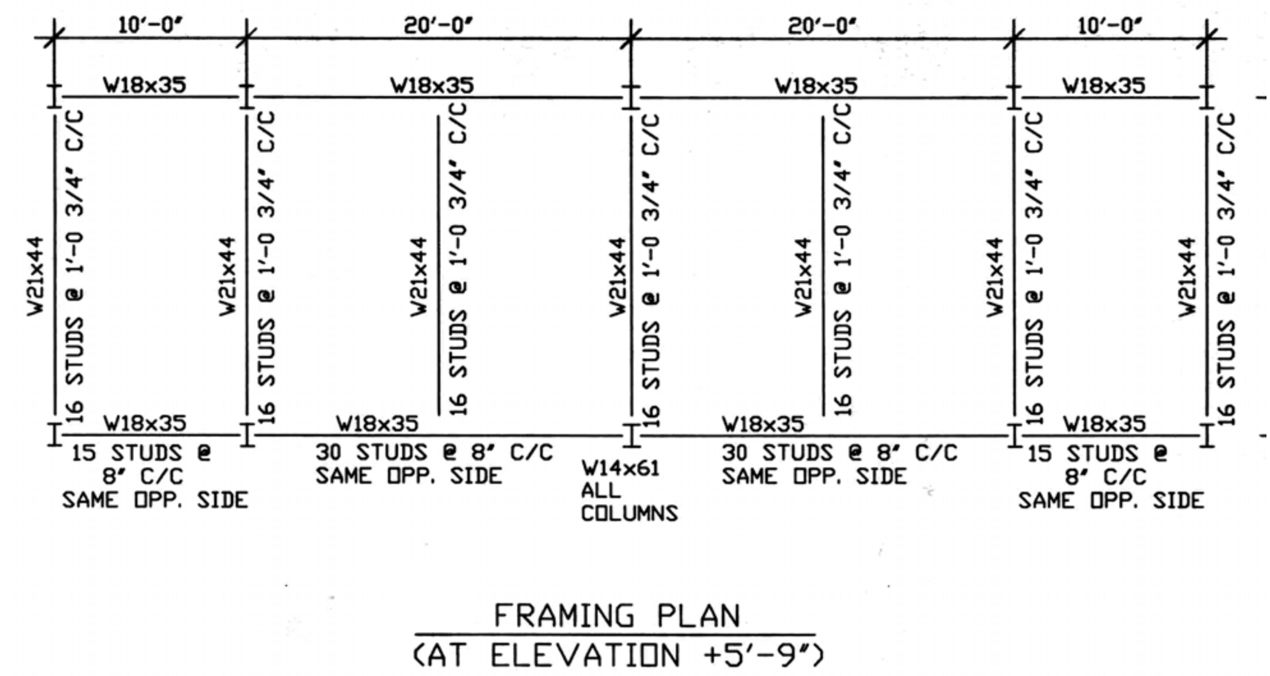
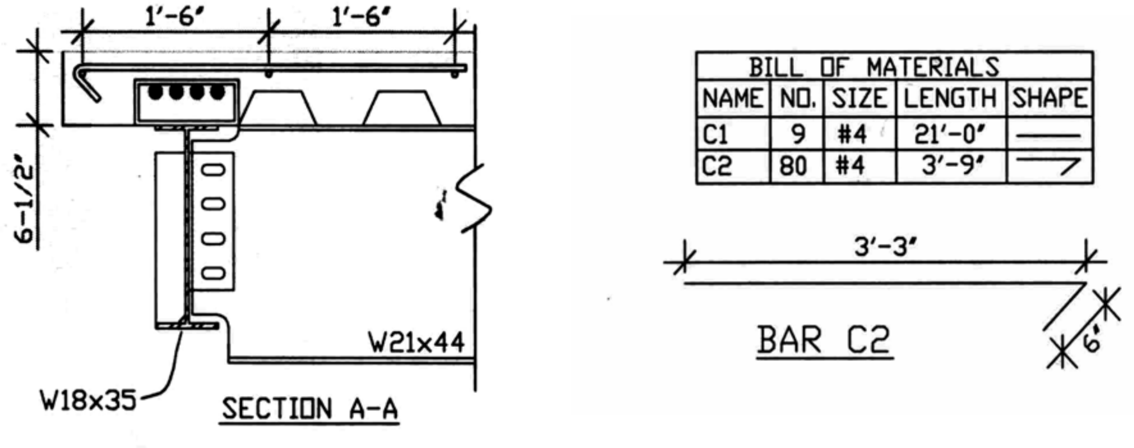


Figure 2.7. Plan of Rebar Placements



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

Figure 2.8. Details and Schedule of Rebars

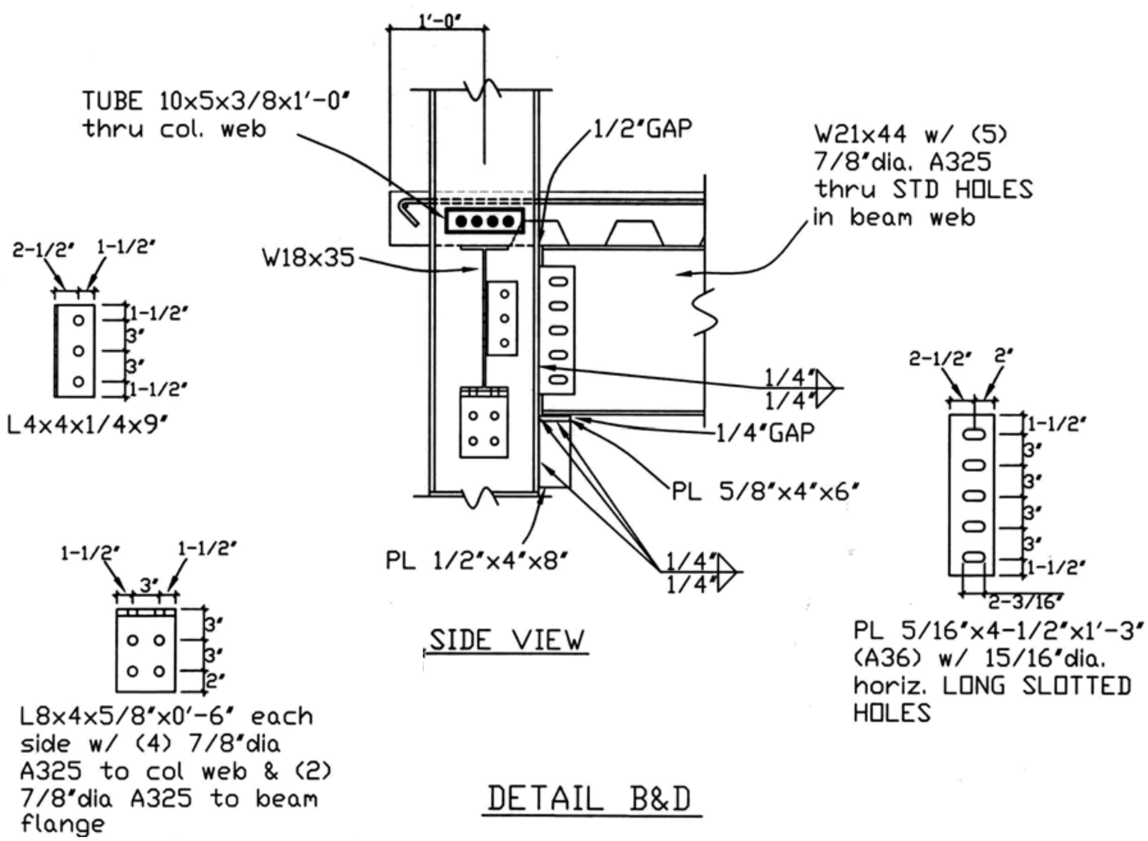


Figure 2.9. Details of Beam to Column Connections

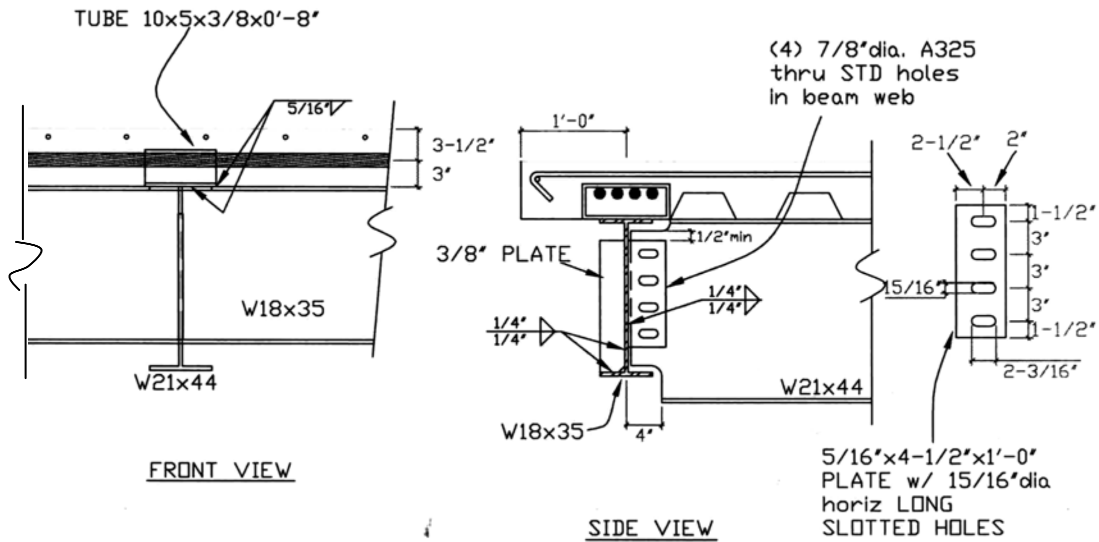


Figure 2.10. Details of Beam to Girder Connections

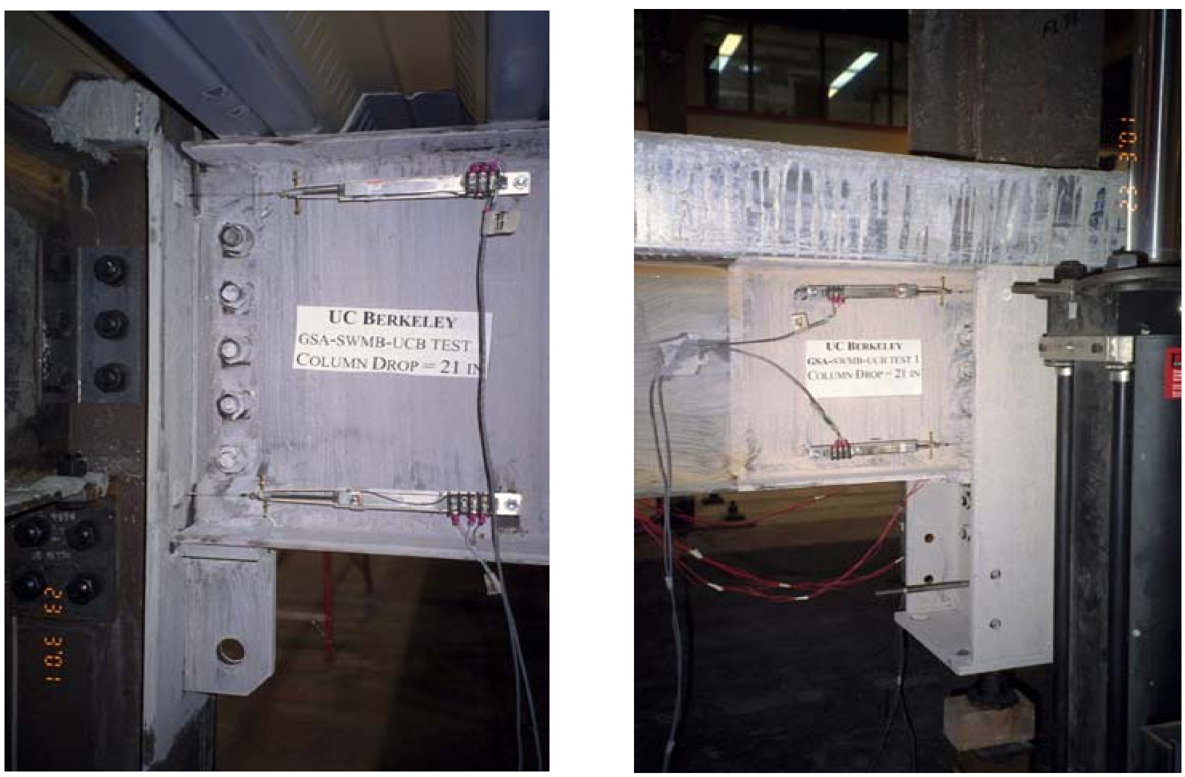


Figure 2.11. Close up of Typical Details of Specimen Prior to Testing

The specimen was a full-scale, realistic representation of a typical simply supported steel framed structure with typical steel deck and concrete slab. The columns were all W14x61 with their web in E-W direction. For safety reasons, the heights of the columns were selected to be 6 feet from the base to the centerline of the cables. This height represented the upper half of the columns from mid-height point of inflection to the connection. The columns were welded to one inch thick base plates and bolted to the laboratory reaction floor by a single one inch pre-stressing rod per base plate. More details on columns and base plates can be found in Appendix A. All girders in East-West Direction were W18x35 and all beams in North-South direction were W21x44.

2.3. Material Properties

2.3.a. Steel

The W14x61 columns, W18x35 girders and W21x44 beams were specified as A36 steel. The angle Mill Certificates for the steel, from Herrick, are provided in Appendix C. The saddles that housed the cables (listed as TS10x5x3/8 on shop drawings) were specified as A500-B steel.

Herrick Corporation of Stockton California fabricated all structural steel beams, columns, angles, shear tabs and anchorage systems as shown in the shop drawings of Appendix B. Upon receipt, all sizes were verified against the shop drawings. Upon inspection of fabricated steel, it was observed that a dent had occurred in the northwestern most flange of the north, central column. Despite this, no detrimental effects occurred during the testing, nor was the column further disturbed at the damaged point or anywhere else on account of the flaw.

2.3.b. Steel Decking

The project used Verco Structural Steel Decking (Verco, 2000), Gauge 20, Type W3 Formlok as displayed in Appendix D. A view of the steel deck of the test specimen is shown in Figure 2.12. The deck was 20-gauge steel painted deck. The selection of painted deck instead of galvanized deck was to avoid harmful gases that can be emitted inside the lab due to welding of galvanized steel. It is believed that structural behavior of painted decks is very similar to behavior of galvanized decks. As a result, there would be no effect of using painted deck on the test data.

2.3.c. Bolts

A325, 7/8-inch diameter bolts were used in the beam-to-column connections of the specimen structure. Length of bolts was either 2-1/4 or 3 inches. All lengths and respective locations are specified in the drawings in Appendix B.

Figure 2.12. Steel Deck (left) and Typical Bolted Connection

2.3.d. Shear Studs

Nelson Studs donated all of the necessary shear stud material and labor for the deck. All studs were 4-½ inches long. Layout of studs is shown in Figure 2.6. Standards, layout of studs and further information are displayed in Appendix B drawings.

2.3.e. Reinforcing Steel

All reinforcing bars were Grade 60 (F_y = 60 ksi), #4 (1/2 inch diameter) reinforcing bars. Wire mesh was 6-inch by 6-inch, W1.4 x 1.4 steel. Layout, size, length and quantity of reinforcement is shown in Figure 2.7 and in Appendix C.

2.3.f. Steel Cables

Cables were made of Grade 270 (F_u = 270 ksi) steel strand cable. Cables were 1-¼ inches in diameter and measured 85 ft.-6 inches in length, inside to inside of heads. Fused metal heads were attached to the ends of each cable and were used to brace the ends in the anchor stands. Cables were hand-tightened in the anchors to eliminate any visible sag in the cables. No significant pre-stressing force was applied.

Figure 2.13. Steel Cables As Delivered to the Laboratory

2.3.g. Concrete

A standard f'_c = 4 ksi concrete floor slab mix was placed on the decking. The concrete reached 4 ksi after 21 days of curing as shown in Figure 2.14. Stress and strain were calculated on the test day and are plotted in Figure 2.14. Average strength of the concrete on the first test day was 4200 psi. Concrete had aged 38 days before the initial series of tests. No tests were performed upon the concrete after the initial day of testing. The slump measured 4-½ inches on the day of placement, which was well within the design limits. All concrete tests were performed at the University of California at Berkeley.

(a)

(b)

(c)

(d)

Figure 2.14 (a) Slump Test, (b) Test Machine for Cylinder Test, (c) Cylinder test and (d) Cylinders after the Tests

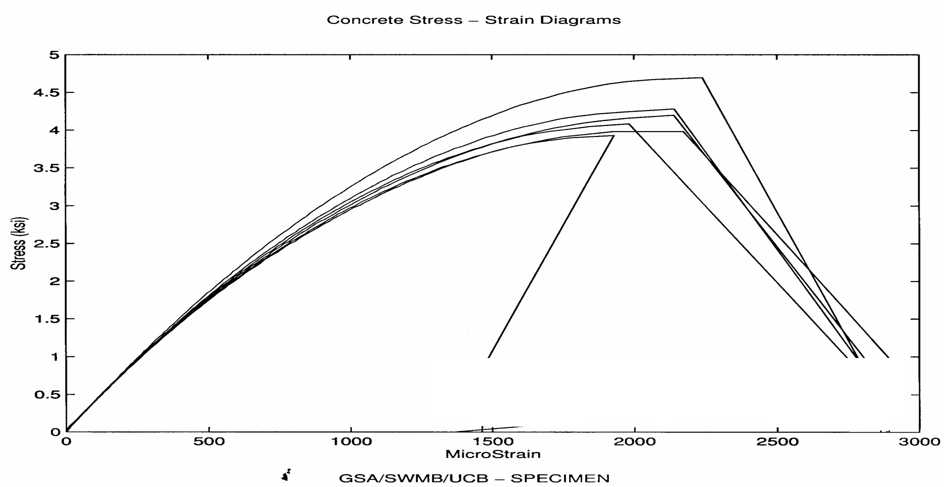


Figure 2.15. Stress-Strain Curves for Deck Concrete

CHAPTER 3

Design of Test specimen and Test procedure

3.1. Design of Test Specimen

The realistic condition for which the specimen was being tested was that an explosion has occurred near a column of the prototype building and the column is blown away. Then the floor supported by the column collapses under the gravity dead and live load. Obviously for safety reasons, the explosives could not be used in the lab to explode away the column and a mechanism had to be devised to remove the column supporting the floor. Again, for safety reasons, the floor could not be left to fall free under the gravity pull since such an uncontrolled crash of the full scale specimen in the laboratory could have resulted in unpredictable consequences in the form of injuries to researchers or damage to the test equipment or the structure of Davis Hall where the structural testing laboratory is located.

To conduct a safe yet realistic test, it was decided that the column, which was supporting the floor, would be designed in two pieces so that the lower portion can be pulled out to simulate the conditions of the column being blown away by the explosion. The removal of the column in this way, although not quite realistic, was a good approximation of loss of axial load capacity of the column in a real situation due to excessive bending and/or buckling.

The next problem in testing was how to control the free fall of the floor as well as create realistic dead and live load in the column. In real buildings, the dead load of the floor and live load of occupants and contents of the floor will be present when the explosion and loss of the supporting column occurs. In the specimen, the only gravity load was the weight of the floor and steel framing which was a good portion of dead load but not all. The dead load of non-structural elements was not present in the test specimen. Obviously, no live load would be allowed on the floor slab of the test specimen during testing. To add the portion of dead load and total live load that was not present in the specimen, vertical actuators were attached to the joint on top of the column to be removed. Then, during testing, when the column was removed, the vertical actuators could apply the additional dead and live loads not present in the specimen and pull the floor downward simulating the gravity pull. The procedure seemed very safe, controlled and doable. However, there was a problem. The oil-based actuators used nowadays in structural laboratories are usually controlled by displacements and not acceleration. In other words, it became clear that with the properties and servo-valve capacities of the actuators, it was not possible to pull the floor down with the constant acceleration of gravity, the conditions that will occur in a real building when it collapses. It turned out that the available actuators could push the floor down with a constant velocity of two inches per second instead of constant acceleration of 32.2 inches/second/second. Then the question was: how realistic is the test procedure? Of particular concern was if the floor were being pushed down with a constant velocity instead of constant acceleration, would the cables be subjected to the same forces in the test specimen, as they will be in an actual building? To answer this question and to develop a reliable relationship between the forces in the cables of the test specimen and an actual building, a series of non-linear finite element analyses of the specimen pushed down by the actuators and dropped in a free fall by gravity acceleration were conducted. These studies, conducted using Lawrence Livermore National Laboratory's powerful hardware and software are summarized in the next section. As shown in the following section, the studies showed that using the actuators to push the floors down instead of letting them freely fall under gravity load still can provide the same forces and displacement field in the cables, provided that the impact factors established by the analysis are applied to the load in the column. More detailed information on this is provided below.

3.2. Analyses to Verify Validity of Test Procedures

During the early phase of the project, a series of non-linear analyses of the specimen was conducted. Initially, the plan of testing included piling up concrete blocks on top of the test floor, removing the middle column and letting the floor fall under the free pull of gravity. After further study of this scheme safety concerns were raised about possible injuries to researchers or damage to equipment and the laboratory structure when the 60ft x 20 ft floor loaded with 60 large concrete blocks collapses inside the second floor of Davis Hall Laboratory. The uncontrolled collapse under gravity's pull not only was not deemed sufficiently safe, but the test would have been a one-time effort and one single test. Therefore, it was decided to use hydraulic actuators, attached to the top of the middle column, to pull down the floor instead of using gravity load. The actuator is shown in Figure 3.1. The main goal of these tests was to verify that testing a specimen using actuators instead of concrete blocks is realistic and will render reliable data.

Figure 3.1. Actuator Used to Apply Additional Gravity Load to Floor

The model shown in Figure 3.2 was used in the analyses. The figure indicates main elements of the nonlinear model. The floor slab in the specimen was a 3 inches thick corrugated steel deck supporting a 6.5-inch floor slab. The ribbed floor system (steel deck and concrete slab) was modeled with a constant thickness R/C slab on top of a flat steel plate as shown in Figure 3.3.

The test specimen had a solid, reinforced concrete flat beam on the south edge of the slab where the cables were placed as shown in Figure 3.4. This 6.5 inch thick slab had reinforcing bars in it. The nonlinear behavior model of the flat edge beam is shown in Figure 3.4. Material properties used in the model of the edge beam are given in Figure 3.4 as well.

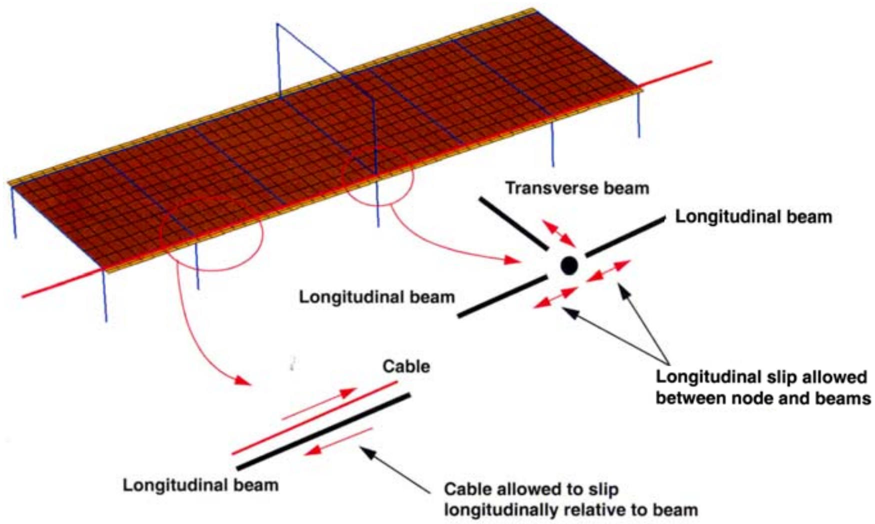
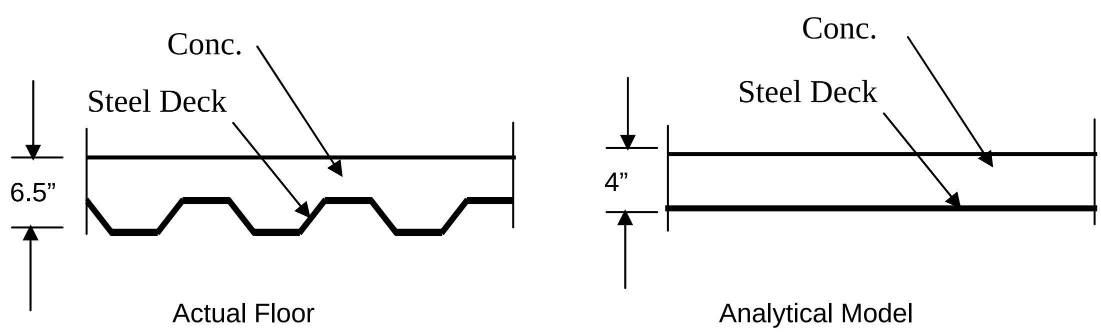


Figure 3.2. Finite Element Model of Specimen Used in the Analyses



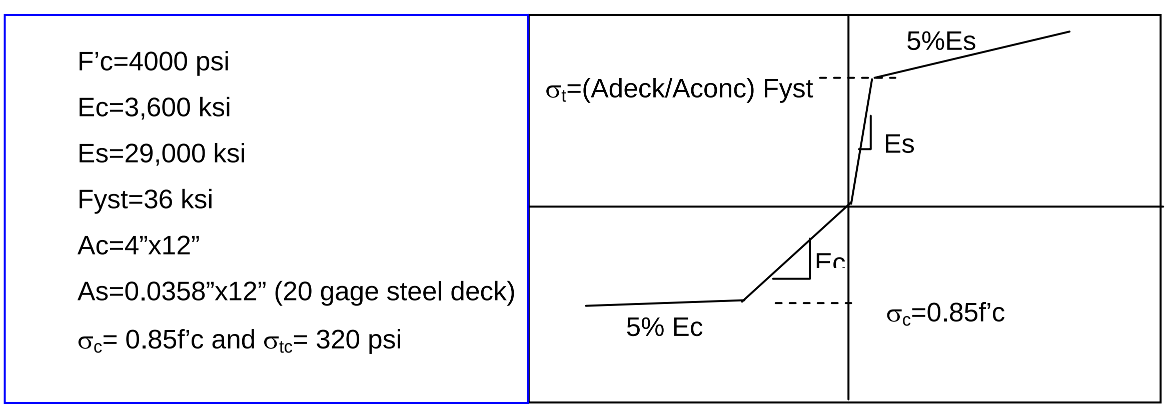


Figure 3.3. Nonlinear Model of Steel Deck/Conc. Slab Used in the Analyses

The nonlinear behavior model of the steel deck/concrete slab is also shown in Figure 3.3 along with material properties used in the deck/slab model.

The cables were modeled as bar elements with modulus of elasticity of 24,000 ksi as specified by ASTM A586-98 Standard (ASTM, 1998). The beams and columns were modeled as inelastic beam elements. The connections were modeled with elements that could slip to permit the slippage of bolts connecting the beams to their support.

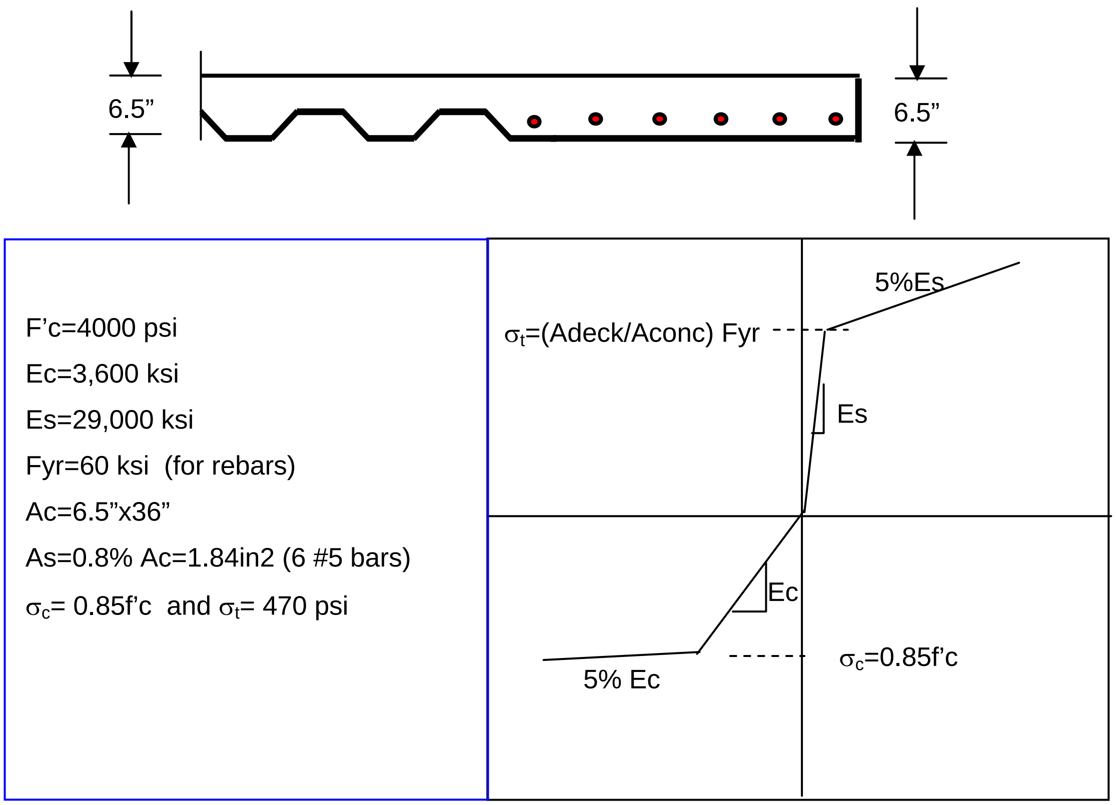


Figure 3.4. Model of Reinforced Concrete Edge Beam

3.3. Results of Analyses

As mentioned earlier, the main objective of the analyses was to compare the forces in the cable as well as displacement of floor for two cases of loading:

Case 1. Concrete blocks are placed on the floor to make the load, in the column to be removed, equal to the load due to a live and dead load acting on the floor. Then, by suddenly removing the middle column let the floor be pulled down by the gravitational pull. The floor in this case will be pulled down by the acceleration of gravity (g). Figure 3.5 shows concrete blocks placed on the specimen for this case.

Case 2. There are no concrete blocks on the specimen. However, to simulate the additional gravity load due to non-structural elements and live load, a hydraulic actuator is attached to the column. By removing the middle column and using actuator to push down the floor, the pull of gravity is simulated.

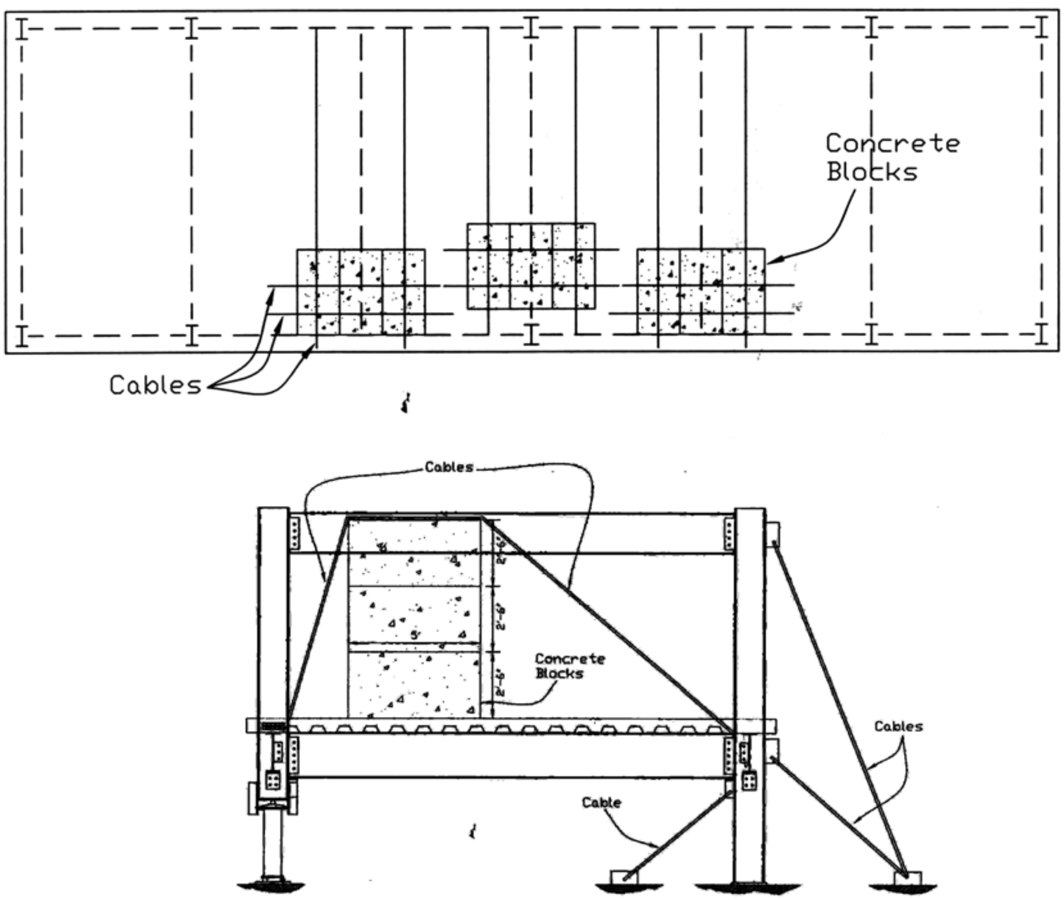


Figure 3.5. Concrete Blocks Placed on the Floor to Simulate Gravity Load

The first step of analysis was to apply initial post-tensioning in the cable by applying a small force to the cables to remove the sag. This was done in the laboratory also during the construction of specimen. Figure 3.6 shows the result of applying post tensioning which created about 2.87 kips axial load in the cables.

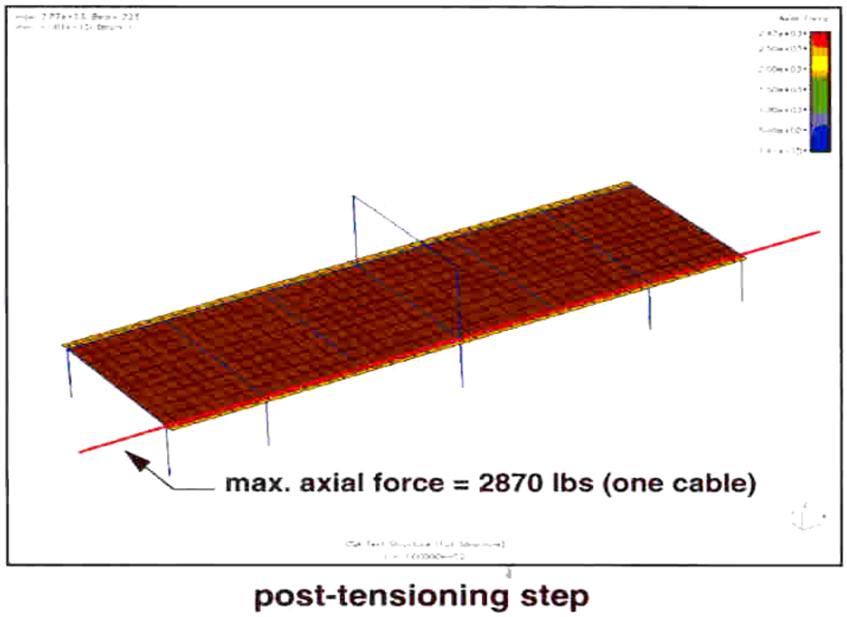


Figure 3.6. First Step of Analysis: Applying Cable Post-Tensioning Force

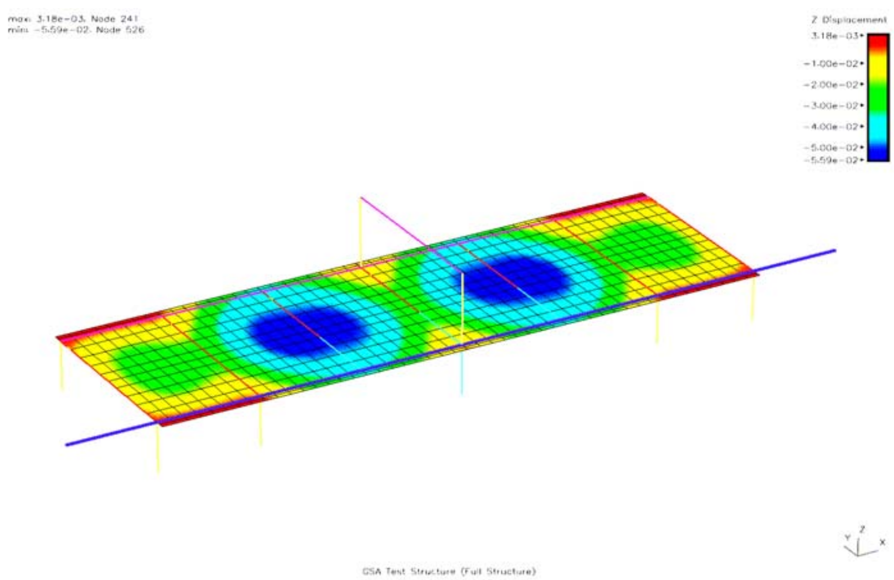


Figure 3.7. Displacement Due to Gravity Load in the Specimen

The next step was to apply gravity loads representing concrete blocks. Figure 3.7 shows vertical displacements of specimen floor due to the application of gravity loads of specimen. Figure 3.8 shows the vertical displacements when the weight of the concrete blocks is also added.

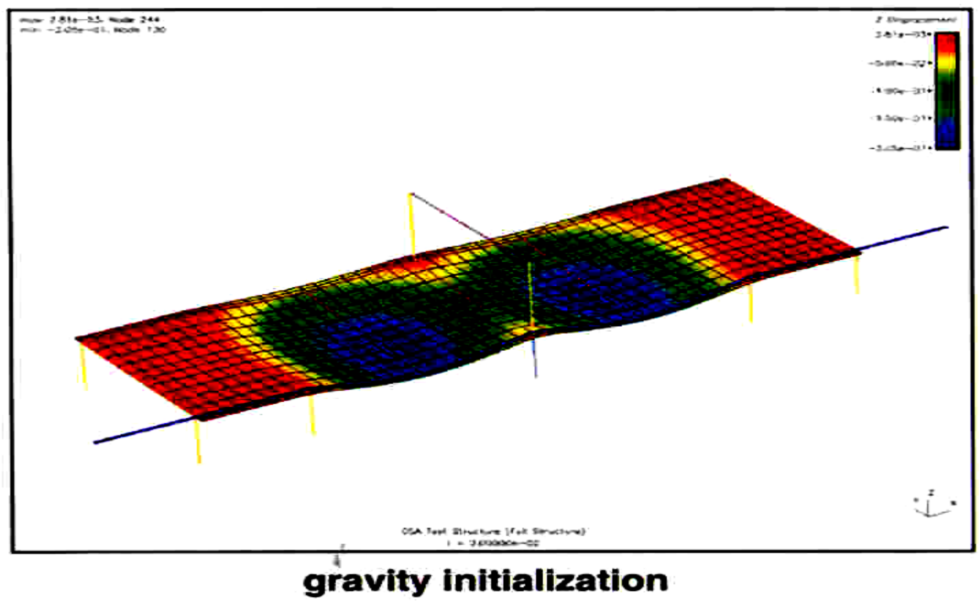


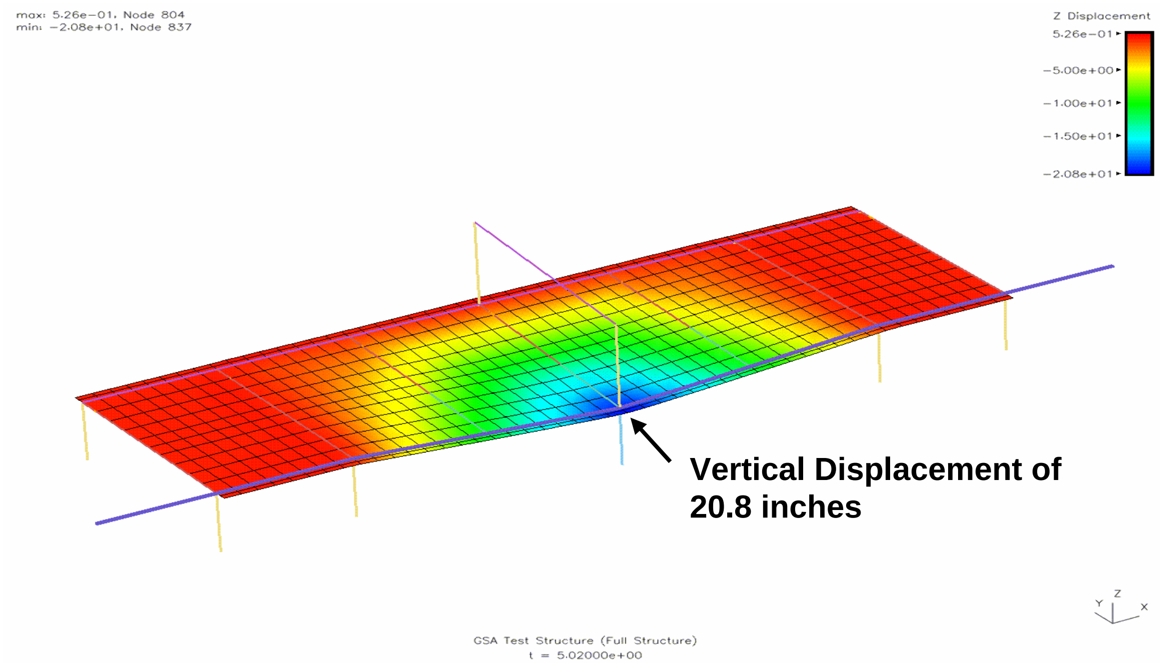
Figure 3.8. Vertical Displacement When Weight of Concrete Blocks is Added

The most important parameters in the analyses were the force in the cable and the vertical displacement of floor after removal of the middle column. Figure 3.9 shows a comparison of vertical displacement of the case with concrete blocks and the case with actuator pushing the floor down. The two cases are almost identical validating the use of the actuator to push the floor down instead of using concrete blocks.

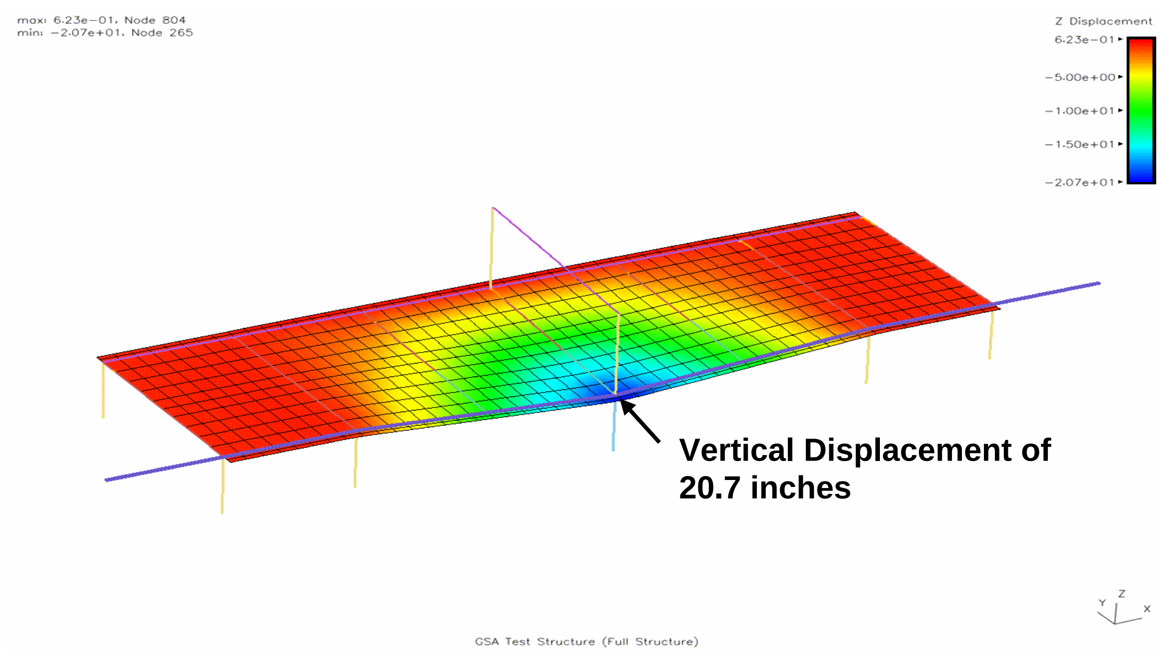
The most important findings of these analyses were:

1. Actuators can be used to simulate the gravity load effects in the columns and cables properly.

2. In order to generate correct axial force in the cables and column it is necessary to apply a load equal to 80 kips in the actuator. This will correspond to a displacement of about 20 inches of floor at the location of removed column.



Concrete Block Model



Actuator Forced Model

Figure 3.9. Comparison of Displacement Fields of Concrete Block and Actuator-Forced Specimens

Figure 3.10 shows a comparison of axial force in the column; first by using concrete blocks, then by using a hydraulic actuator to simulate the gravity load in the column. The results of analysis of two examples are very close indicating that using actuator-based system to do the test is appropriate.

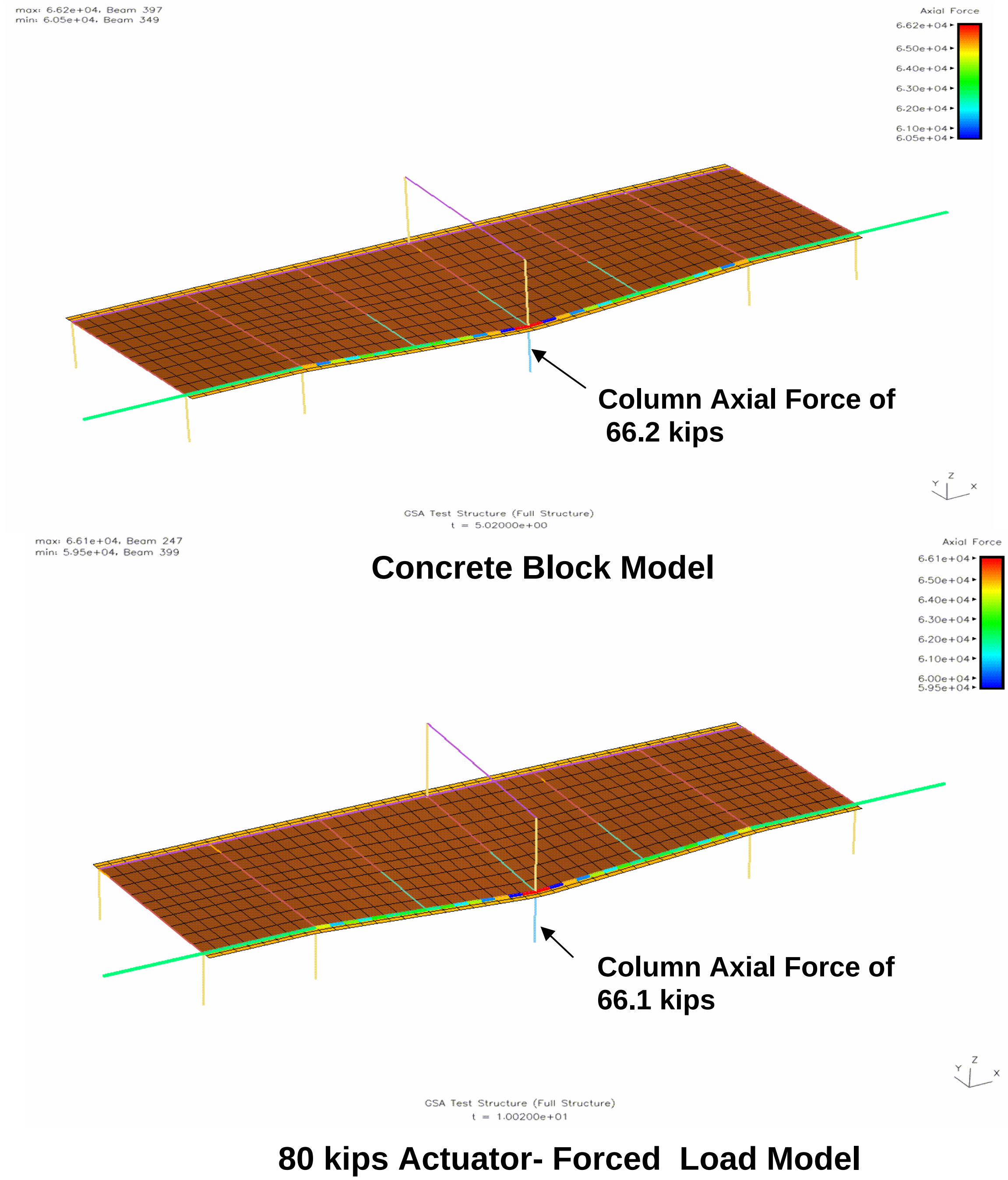


Figure 3.10. Comparison Analysis of Axial Load in the Column in Two Examples

3.4. Instrumentation of Specimen

Instrumentation was placed on and around the specimen to measure displacements and strains at the critical locations. The exact location and channel numbers for each instrument are given in Appendix B starting on page 113 of the report.

3.4.a. Displacement Transducers

As this full-size specimen was very large, and would undergo high displacements, displacement transducers (a combination of stick pots and wire pots) spread around the structure provided crucial information. The layout of the transducers is shown in detail in Appendix B. For the first set of tests where the specimen drop was 19, 21 and 24 inches, eleven of the twenty-two transducers were attached to the primary test column. Two of these, transducers DT 4 and DT 5, were wire pots used to measure vertical displacement at the test column. Transducers DT 6 through DT 11 measured rotation of the three adjacent beams.

Figure 3.11. Displacement Transducers Used to Measure Rotation of Beam End

Figure 3.11 shows the two transducers placed upon the transverse beam on the right. These transducers were each located 2 inches from the top and bottom of their respective flange and 16 inches from the connection to the column. The distance the pots were set back allowed the beam to undergo local deformations at the connection to the test column without greatly affecting the data collected. This way, data for global rotation of the beam could be gathered.

Displacement transducers DT 17 and DT 18 were attached to steel plates around the actuator. These were each 30-inch wire pots attached at an angle to the column. Since the column was going to go through a vertical displacement, measuring horizontal displacement required that the instruments follow the column through the vertical displacement. The instruments were attached to plates at the mid-point between the locus of zero displacement and the lowest point of displacement – 36 inches. Therefore, the wire would travel through the vertical displacement of the column, which was being measured by DT 4 and DT 5. Using these in tandem and from the geometry the horizontal displacement of the column could be calculated. Transducer DT 19 was also used in this manner, though it was not attached at the midpoint, but rather at a random point along the path of travel for convenience of instrumentation set-up. Displacement transducer DT 19 was also intended for rotational measurement of the column and a back up for DT 17 and DT 18.

At the ends of the structure along the cable side, transducers were attached at the centerline of the beam – column connection. These were DT 1 and DT 16. DT 2 and DT 15 were attached as seen in Figure 2.7 for the purpose of measuring vertical displacement of the opposite ends of the longitudinal beams connecting to the test column. DT 3 and DT 14 measured horizontal displacement of the columns used to support the resulting forty-foot bay. These latter four, DT 2, 3, 14 and 15 were all attached at the centerline of their respective member or joint.

On the opposite side of the structure, DT 20 measured the horizontal transverse displacement of the central W 21x44. Placement for DT 22 was decided upon as the west side of the structure aligned at the centerline of the southern series of girders. The reason for this distant placement was to find displacement of the whole structure.

Finally, a displacement transducer, DT 21, was attached to one of the cables to measure its extension during the testing, over the arbitrary length of 49 inches. By measuring the extension, the Young's Modulus could be used with the strain to determine the stress and consequently the force taken by the cable.

The displacement transducers placed above the deck-collected data for Tests 1-19, 1-21 and 1-24. For the final test, where the floor drop was 35 inches, DT 5 and DT 18 were not used. In the original design, DT 4 and DT 5 were below the test column. Since the tests were not intended to come close to the pots this was not an issue; but as the 35-inch test was proposed, this became a concern. While the pots could have been moved out far enough that they were not directly under the column, any side sway by the column when being pulled down to 35 inches would have caused an interference, and could possibly have destroyed the instruments. Therefore only one pot was used, placed on the north side of the column rather than the east and west.

The transducer DT 18 was a casualty of the new actuator design. The two actuators were placed where the angles that previously held DT 17 and DT 18 had been welded. With this avenue now closed for two pots, one solo pot was placed in between the actuators and attached to an angle at the midpoint. All other transducers remained in the same positions.

3.4.b. Strain Gauges

Strain gauges were also attached to the structure. Six gauges were placed on each of the angles connecting the test column to each adjacent W 18x35 as seen in Figure 3.11 and in drawings in pages 118 through 120. Three gauges were placed on each of the shear tabs connecting to the central transverse W 21x44. The main purpose of these gauges was to detect initiation of yielding in the angles and shear tabs. Each of these gauges was used for Tests 1-19, 1-21 and 1-24, and all were linear horizontal gauges. For test 1-35, only the three gauges on the shear tab not attached to the test column were used from the original eighteen; therefore, SG 16-18 became SG 15-17 for Test 1-35. Also for this new test, four new strain gauge rosettes and two linear gauges were attached underneath the steel deck as shown in Figure 3.12.

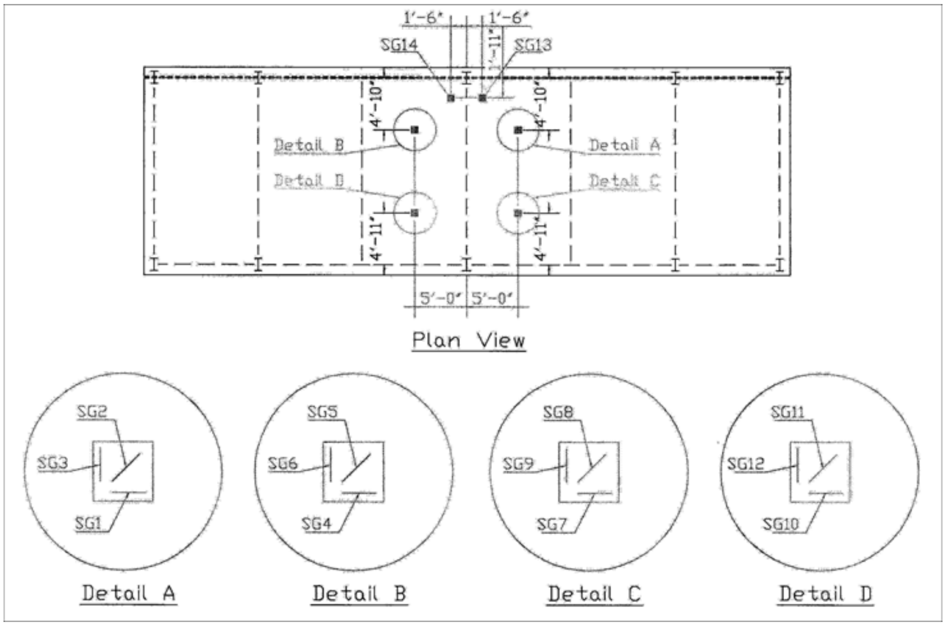


Figure 3.12. Partial Strain Gauge Layout, Test 1-35

Each gauge required a fair deal of preparation before application. Before the steel frame was erected, the angles and shear tabs were ground along their surface. A coarse sand grinder was used to remove the initial mill-scale from the steel. A size 36-grit sanding wheel then smoothed the surface further and removed any excess scale. Finally, a size 120-grit sanding wheel smoothed out the steel by removing scratches from the previous two sanders and also smoothed over any gouges, natural or accidental, until the steel had a very smooth, shiny surface. At this stage, 240-grit sandpaper smoothed the surface further, and was followed by 360-grit sandpaper. This last sandpaper was the final smoothing step. Occasionally, when the surface did not appear smooth enough, those performing the work would revert as far as the 120-grit sanding wheel, eventually advancing to the 360-grit sandpaper. The chemical cleaning could then be performed.

The surfaces were ground down prior to erection of the frame due to the inconvenient placement of the gauges. The bolts going through these connection pieces would have prevented grinding areas of the steel after erection. Once the structure was in place, the steel surfaces were then chemically cleaned according to a standard strain gauge application method. This method included:

1) Marking the centerlines of the gauge by using a ball-tip pen to mark a line on the steel. The ink was removed during the cleaning process; however, a faint indentation was made in the steel by the pen.
2) An acid bath combined with additional sanding using the 360-grit sandpaper followed. Degreaser was sprayed upon this area afterwards and the surface wiped clean.
3) Wiping the area first with an acid surface cleaner, then with an alkaline surface cleaner neutralized the area. If any dirt or colored streaks (often blue, yellow or brown) were found, the surface was deemed unclean and this step repeated.
4) When clean, quick drying, high-strength glue was applied to the face of the strain gauge and attached to the clean surface of the steel. The gauges were covered with duct tape to prevent any damage or interference from outside substances.
5) Researchers attached spade lugs to the end wires of the gauges to prepare for connection to the data acquisition system.

3.5. Data Acquisition

After the proper placement of each of the instruments, their respective wires were attached to the data acquisition boxes. These boxes each carried up to eight of the same type of instruments (DTs with DTs and Strain Gauges with Strain Gauges). The acquisition system used is called the Megadak. A description page can be found in Appendix C. Each instrument occupies one channel of Megadak. It has a maximum capacity of only 56 channels, but can record 50 data points per second. Due to the high speed of the Megadak and the relatively low number of channels, this machine was optimal for the project. The system recorded all information as the test proceeded.

In addition, a separate computer was used to control the actuators in this project. The computer had the ability to back up a small number of channels, and was utilized for several of the key measurements. DT 4, DT 5, DT 17 and DT 18 were connected to the computer for this purpose (only DT 4 and DT 17 during Test 1-35). This allowed a safeguard against problems with the initial recording system. All files were saved on disk and processed later.

3.6. Initial Measurements

Prior to testing, the specimen was measured and its respective instruments tested for accuracy. As noted earlier, the entire steel frame supplied by Herrick Corporation was measured and inspected upon its arrival at UCB. All members measured out to within 1/16 of an inch as intended, with the exception of the small, benign dent on the flange of the test column. The steel decking also proved to be within a fraction of an inch of its intended dimensions. Some of the instruments were measured to determine their initial wire lengths and the lengths of their respective connecting wires. The purpose of this was to determine, through used of Pythagorean theorem, the horizontal displacement of the test column. The resulting displacements are based upon these initial values and the results generated by their respective instruments.

All instruments were also calibrated before any testing occurred. This involved displacing each transducer a given distance, with the aid of special pre-measured calibration blocks, and confirming the offset on both the Megadak and the computer back up. Test 1-35 did not use DT 5 or DT 18, and therefore they remained as “dead” channels on the systems and collected no data. SG 4 and SG 10 for Test 1-35 were damaged and therefore also did not record any data. All other instruments used were reading properly on the systems prior to testing.

3.7. Preliminary Testing

Prior to Test 1-19, a quarter-inch low-level test was conducted on the specimen. The low-level test was done for the purpose of ensuring that all machinery and instrumentation was running and recording properly. Test 1-19 and Test 1-21 were both run on the same day, Friday March 23 as the low-level test. Test 1-24 occurred on Monday, March 26. Test 1-35 occurred on Monday, June 4. A quarter-inch low-level test was also run prior to Test 1-35. In all cases, the low level tests indicated that all instruments and machinery were working properly allowing the tests to commence as intended. Data for these low-level tests were processed prior to conducting the actual test. The results bear no significance upon the overall outcome of this project and have not been included in this report.

CHAPTER 4

Behavior of Specimen

A total of four tests were conducted on the specimen. The tests were:

1. Test Number 1-19 where after removal of middle column, the joint above the column was pushed down 19 inches. After completion of this test, the joint above the removed column was pushed up to its original position (zero displacement) and the removed column was put back in.
2. Test Number 1-21 where after removal of the lower portion of the middle column, the joint above the column was pushed down about 21 inches. After completion of this test, the joint above the removed column was pushed up to its original position (zero displacement) and the removed column was put back in its place.
3. Test Number 1-24 where after removal of the lower portion of middle column, the joint above the column was pushed down about 24 inches. After completion of this test, the joint above the removed column was pushed up to its original position (zero displacement) and the removed column was put back in its place.
4. Test Number 1-35. Since in the previous three tests, the cable-supported floor had performed well with no apparent damage to cable system, it was decided to conduct a last test on the specimen and push it down as much as the test set-up will permit. This was 35 inches of drop of floor since the distance between the bottom of joint and lab floor was 36 inches with one inch of space left after drop of floor. In this case, since it was felt that larger force would be needed to push down the floor above the removed column, a second actuator was added to make the capacity of actuators 240 kips instead of 120 kips used in the previous three tests. After removal of the lower portion of middle column, the joint above the column was pushed down about 35 inches. After completion of this test, the joint above the removed column was pushed up to its original position (zero displacement) and the removed column was put back in its place.

4.1. Behavior of Specimen During Test 1-19 (19 inches of Floor Drop)

Test 1-19 occurred on March 23, 2001. This was the first test conducted on the specimen. The drop of floor at the location of removed column was 19 inches. A quarter-inch low-level test was run beforehand to check that instrumentation is working properly. Shortly thereafter, the test commenced. The data collected from all instrumentation are given in Appendix C1, page 122 of this report. Test 1-19 was performed on the virgin specimen. After each test, the specimen was pushed up by actuators and was returned to its original zero displacement, and broken bolts were replaced with new bolts; however, the specimen was otherwise un-restored after each test and the cracks in the concrete were not repaired. Such cracks were marked with different colored pens, one color for each test.

Photograph documentation was taken throughout the test and the test was filmed on video. Column designations for the test specimen are shown in Figure 4.1.

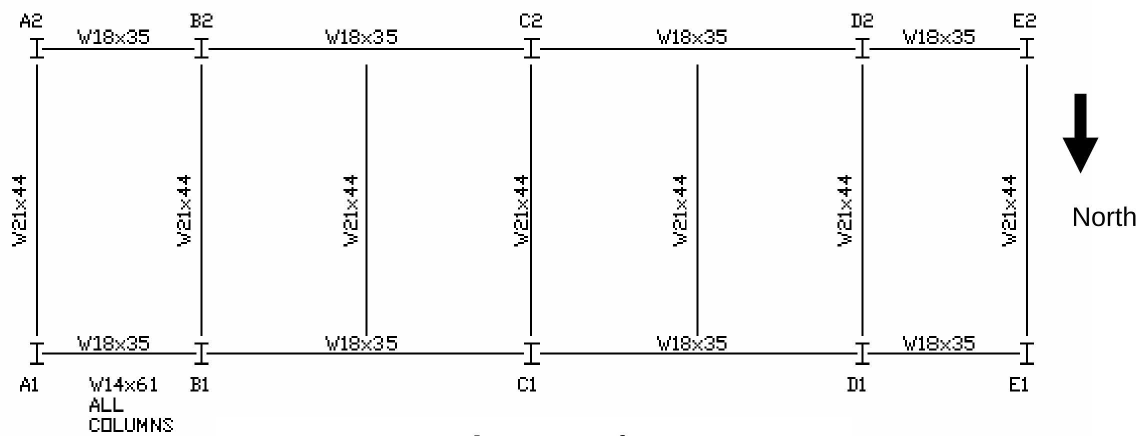


Figure 4.1. Plan View of Specimen

Prior to the test, the deck was inspected and cracking was found on the slab along the centerline of transverse beams, in between the three pairs of interior columns. All other instrumentation and properties of the specimen were as noted previously.

After inspection was completed, the specimen was raised up 0.07 inches by the actuators to remove the supporting jack from underneath the test column. At this point the actuators were carrying a compressive load of 11.0 kips. This value was taken as the gravity load of the specimen initially carried by the test column. After the jacks were removed and all members of the research team were at a safe distance from the specimen, the Chief Electronic Engineer lowered the actuator at the speed of approximately one inch/sec (the maximum speed possible for the actuator). As the structure lowered, there were a series of groans, pings and pops from the structure. These could possibly be attributed to separation between the concrete slab and the metal decking, cracking of the concrete, and bending of the entire slab. The test column dropped a total of 19.8 inches before the test was stopped. The test was terminated when the actuator load neared 100 kips, and had exceeded the design displacement. The specimen was held at maximum load briefly before being released down to zero load in the actuator. Displacement at zero load was 15 inches. The entire test took approximately 30 seconds from start to finish. The actual drop of specimen over 19.8 inches took about 11 seconds. Figures 4.2 and 4.3 show the specimen at the end of Test 1-19.

Figure 4.2. Specimen at the end of Test 1-19

Figure 4.3. A View of the Underside of Specimen at the End of Test 1-19

Behavior of various components of the structure is listed below. For location of components, the reader is referred to Figure 4.1.

4.1.a. Behavior of Structural Steel Framing System During Test 1-19

Columns A1 and A2:

No damage or noticeable plastic deformations occurred in the columns, their connections or any of the beams framing into the columns.

Columns B1:

No deformations were noticed in the column itself. The transverse beam and the beam connecting into Column A1 were also undamaged. However, the girder connecting to the test Column C1 did undergo yielding in several places as shown in Figure 4.4. The whitewashed

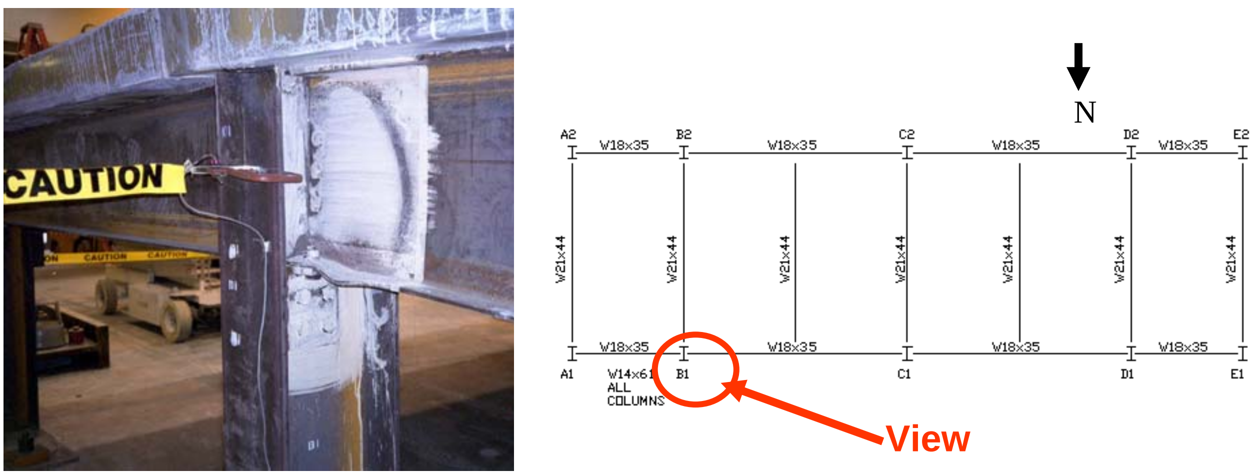


Figure 4.4. Local Buckling of Beam Web and Bottom Flange

areas of the web showed a crescent pattern of yielding due to local buckling of web of the beam. The bottom flange of the beam experienced it's own local buckling. Yielding was seen on the web near the bolts as well. Both bolts in the beam seat remained intact, as did all bolts in the angle.

Column B2:

No damage or noticeable plastic deformations occurred in the column, its connections or any of the beams framing into the column.

Column C1:

No deformation was noticed in the column itself, though the areas behind the angles where the bolts attached were not visible and potential for yielding was much more likely there. Figure 4.4 shows a view underneath the slab on the east side of C1. Clearly, the transverse beam on the left has shifted in the shear tab and the bottom bolt has slipped along the slots exposing areas of the beam previously unexposed to the whitewash. The shear tab seemed to be unaffected by the test. The angle at center, however, underwent serious deformations. The center of the angle is very shiny. That area was previously ground down for strain gauging, and this area was now exposed as it yielded and the whitewash flaked off. Yielding is also clear at the base of the angle and also on the longitudinal beam it attaches to.

The seat angles on both sides of the column web deformed away from the column. The angle attached to the web also pulled away from C1 as the bare face of the column web was exposed from behind it in Figure 4.5. The beam seat stayed connected to the longitudinal beam by two vertical bolts. However, of the four horizontal bolts connecting the longitudinal beam seats to the column, the top two bolts failed as seen in Figure 4.5. Yielding occurred in horizontal streaks across the bolt lines. Both the west and east beam seat at C1 underwent heavy deformations.

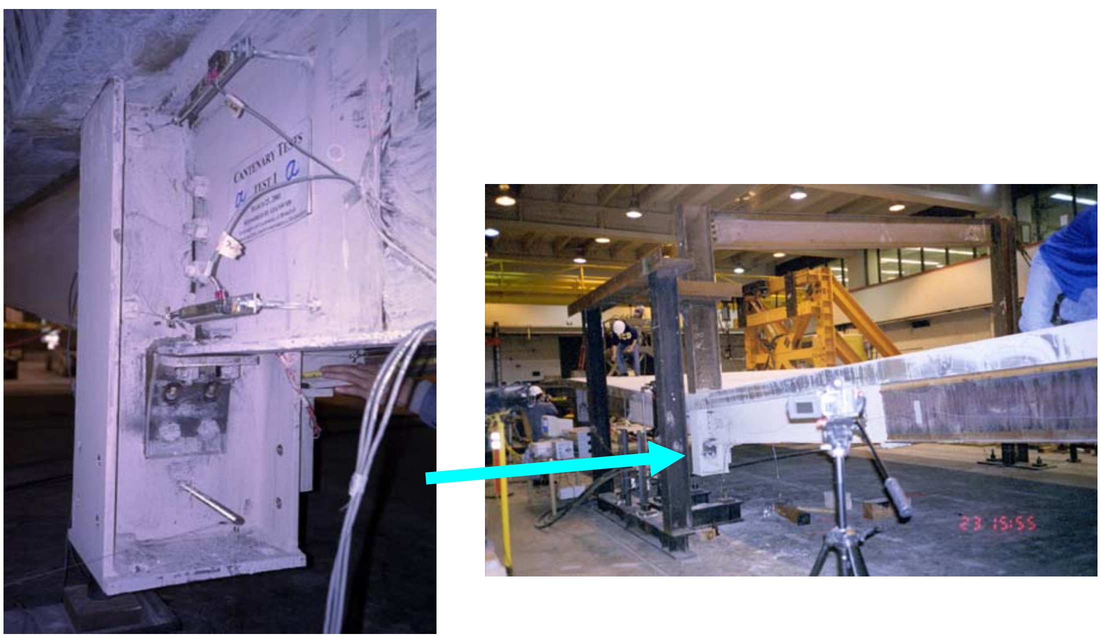


Figure 4.5. Bending of Seat Angle and Bolt Failure, Connection at top of Removed Column

Column C2:

No deformation was seen in the longitudinal beams framing into the column, or in the column itself. At the shear tab connections of the transverse beam the bolts slipped inside the horizontal long slots in the shear tab. The shear tab itself underwent slight deformations, if any, while the transverse beam buckled slightly around the beam seat. The web of the transverse beam also exhibited yielding around the base of the shear tab. This yielding was not as noticeable on the other side of the girder. Figure 4.6 shows this detail.

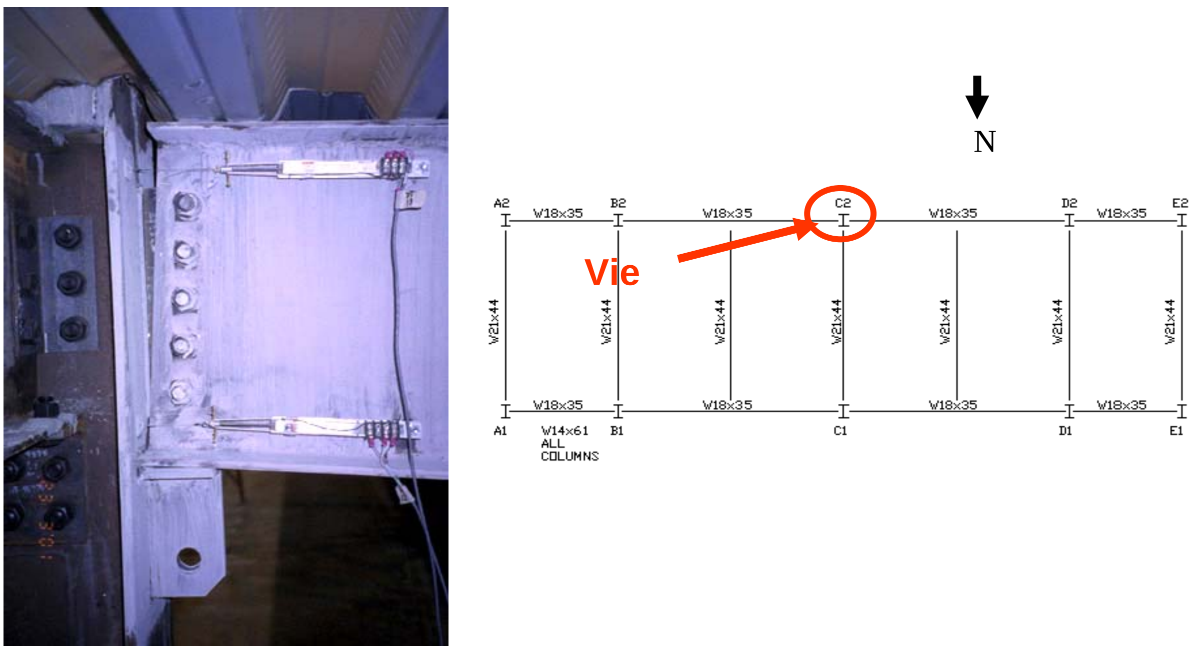


Figure 4.6. Rotation of End of Transverse Beam at Column Location C2

Column D1:

Column D1 experienced very similar results to Column B1. The noticeable differences were that both vertical bolts connecting the beam seat to the girder sheared off. The distortion between the seat and the girder was large enough that the bolts were irreplaceable after the test.

Column D2:

Behavior was the same as Column A1.

Column E1:

Behavior was the same as Column A1.

Column E2:

Behavior was the same as Column A1.

4.1.b. Composite Floor Deck

As expected, the concrete on the composite deck underwent a high degree of cracking with compressive crushing occurring above the transverse beam up to three feet away from the test column. Tension cracks occurred in a semicircular pattern around the test column often framing from sides B1 and D1 and flaring out towards C2. The concrete separated away from column C2 and cracked at least 1 inch deep for several feet on either side. Over columns D1 and B1, the concrete appeared to crack off the top of the saddle boxes. The deck also split and bowed out away from the structure at C1 as the concrete crushed slightly. Figure 4.7 shows the damage at this location on top of the deck.

Figure 4.7. Slight Damage Top of Floor Deck at Location of Removed Column

Despite the high level of cracking observed, the deck appeared to have retained a good degree of structural capability and was intact. Little damage, virtually no cracking, was found in either of the end bays between column lines A and B and between column lines D and E, Figure 4.1.

4.2. Behavior of Specimen During Test 1-21 (21 inches of Floor Drop)

Test 1-21 was performed on March 23, 2001 shortly following Test 1-19. No low-level test was necessary beforehand. The specimen had been restored to zero displacement following data collection of Test 1-19, and the jack replaced beneath the test column. Data collected from instrumentation is provided in Appendix D2.

Again the specimen was raised slightly, and the jack that was supporting the middle column was removed from beneath the specimen. Once all members of the research team were clear, the test commenced. The starting load on the actuators was 42.2 kips compression. As the structure lowered, there were again a series of groans, pings and pops from the structure. These could possibly be attributed to separation between the concrete slab and the metal decking, cracking of the concrete, and bending of the entire slab. Test 1-21 achieved a 21” displacement at about 105 kips, approximately the same force in the actuator as Test 1-19. This is not too surprising since the specimen had already been deflected 19 inches from the previous test. Also, the vertical beam seat bolts at D1 were irreplaceable.

Damage to the structure during the 21-inch test was minimal. There were slight extensions of the yielding that had occurred during Test 1-19, but the steel frame did not experience any other notable deformations. The concrete deck experienced further cracking. Most of the cracks were extensions of cracks already created, and a few new hairline cracks appeared, but again, nothing of much mention.

Following the test, the specimen was left at its position of zero load while data was gathered, then returned to the point of zero displacement. The researchers placed the supporting jack back beneath the test column.

4.3. Behavior of Specimen During Test 1-24 (24 inches of Floor Drop)

Test 1-24 was conducted on March 26, 2001. The goal for the test was to use the actuator’s full capacity of 120 kips upon the structure. No low-level test was performed beforehand. The load did reach 120 kips and the structure achieved a displacement of 24 inches. This test was the final test to be run with one actuator and at the maximum speed for the actuator and the highest recording speed for the Megadak data recording system.

The specimen incurred much more noticeable damage during Test 1-24 than it had during Test 1-21. Columns A1, A2, B2, D2, E1 and E2 and the members framing into them still experienced little if any noticeable yielding. However, the yielding that began at Columns B1, C1, C2 and D2 continued further and caused tearing in one of the angles connecting to C1. In the following sections specific new damage that occurred during this test is explained.

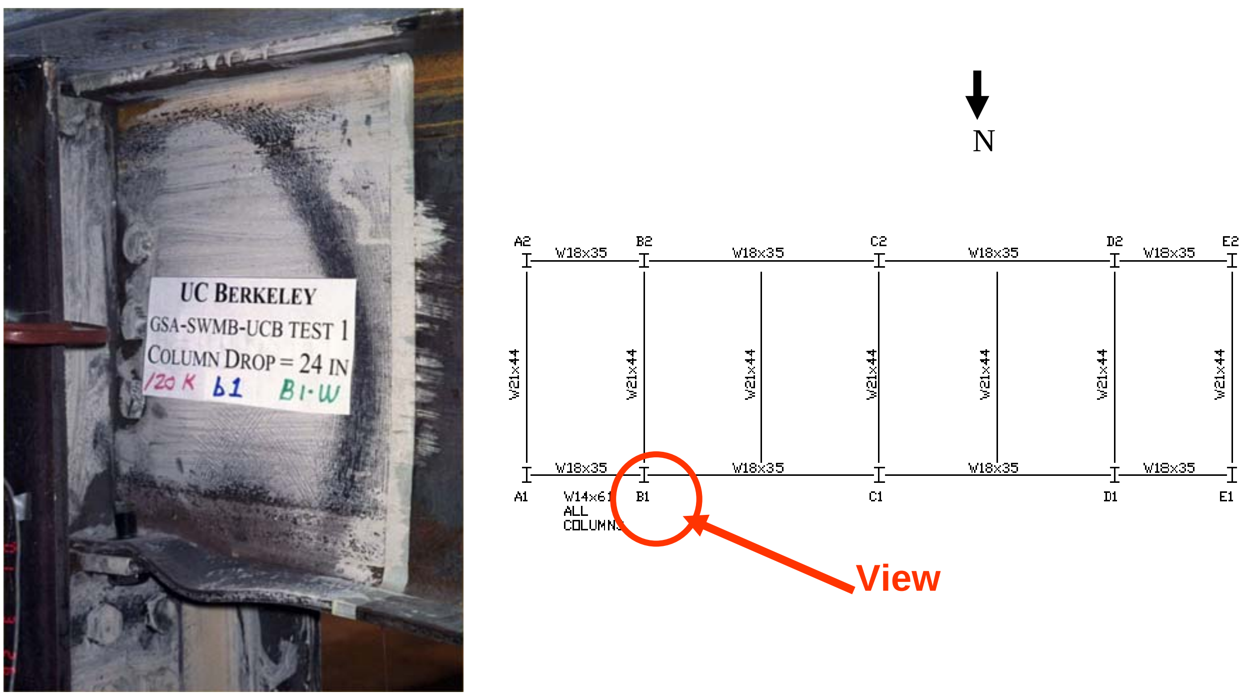


Figure 4.8. Girder Web and Flange Buckling at Location of Column B1
After 24 inches of Floor Drop

Column B1

Starting at connection B1, the yielding in the bottom flange due to the beam seat has clearly increased significantly. There is also considerable yielding in the web because of the heavy flange deformations, which is indicated by the dark spot just above the bottom flange and continues with criss-crossing striations up towards the center of the web. Although it is a little difficult to view, the web has actually buckled outward towards the camera between the long vertical crescent yield and the angle, predominantly underneath the sign. The area of a web between the bolts and the column has also been stressed as the dark spot shows. Finally, there has also been much more deformation in the beam seat as the whitewash has come off in the horizontal line along the top horizontal bolts in the seat. All these deformations are even more noticeable when compared with the picture from Figure 4.8 of the same connection at Column B1 just after Test 1-19.

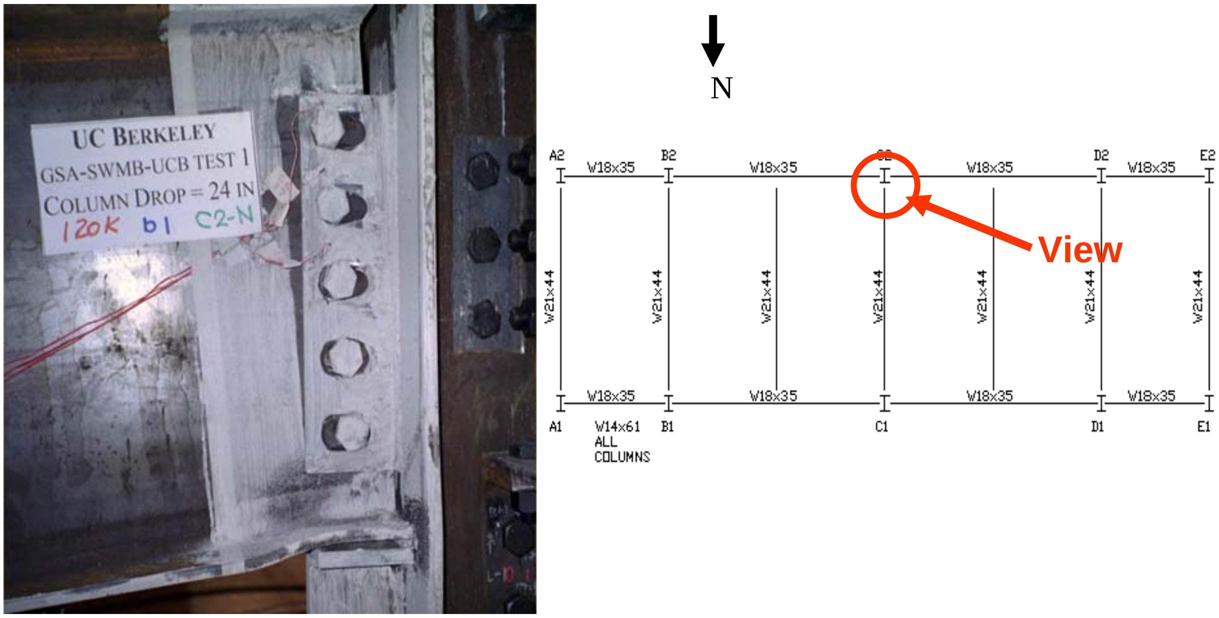


Figure 4.9. Rotation of Beam at Location of Column C2

Column C2:

The area around Column C2 also underwent some deformations. With the extra displacement downward, the bottom flange on the transverse beam between C1 and C2 yielded around the beam seat attached to C2 as seen in Figure 4.9. The web also yielded at the base of the shear tab. Further deformations are noticeable in the shear tab itself. The transverse beam pulling heavily on the top bolt stretched the shear tab around the bolt. We can see in Figure 4.7 also, how the rotational force created by the pressure of the beam upon the beam seat has also slightly bent the column flange inwards.

Column C1:

As before, yielding occurred to a great degree in the angles and the beam seats bolted to the test column (the beam seat welded to the column experienced little if any yielding). However, due to the additional rotation and force applied by the 24” displacement of the test column, the connecting angle on the east side of the column tore as shown in Figures 4.10 and 4.11 below.

On Figure 4.10 we can see the fracture traveled halfway up the angle. Excessive yielding had occurred in the angle in Test 1-19 well before this test. As can be seen in the data in Appendix D3, two of the gauges on the angle, SG7 and SG8, failed close to the time of the failure of the angle. SG7 was at the top of the angle and therefore it is apparent that the localized strain caused the failure of that gauge and not a separation of the steel underneath it.

Figure 4.10. Close up of Web Connection

Figure 4.11. Connection at Location of Column C1

From the figures, there is also definite yielding on both the longitudinal beam and the column. A light series of striations can be seen on the column underneath the angle. Heavier yielding is also noticeable just to the right of the bottom set of bolts on the beam seat. Note also that practically no whitewash remains on the beam seat at this point. Yielding in the beam is much more defined than the column, and is very visible at the base of the angle. It is also important that the vertical bolts on the seat remained intact on the east side depicted above. This clearly helped to deform the angle to failure by allowing the beam to take more load in catenary action. The vertical bolts on the west side failed during the test. The angle went through extreme deformation, however, no tearing or separation of the steel was visible.

The transverse beam also underwent slight deformations. The most noticeable yielding occurred in the shear tab, where the bottom bolt created a semi-circular deformation at the edge of the tab and yielded the steel in between. A larger quantity of un-whitewashed steel was exposed from underneath the shear tab in Test 1-24 than during previous tests. This indicates that since the level of deformation was very low in both the shear tab and the transverse beam, the slotted connection was very effective in allowing rotation and transmitting force without damage to the structural system.

Column D1:

Greater deformation occurred in the bottom flange of the beam caused by the beam seat. However, because the beam no longer was connected to the seat angle, the deformations were not as large as in Column B1.

Steel Deck and Concrete Floor:

Cracking was greater between Tests 1-21 and 1-24, (as could be expected), than between Tests 1-19 and 1-21. The existing cracks expanded further and many new tension cracks were formed upon the deck. Compression crushing of concrete was quite extensive along both the north and south sides of column C1. The north side can be seen in Figure 4.12.

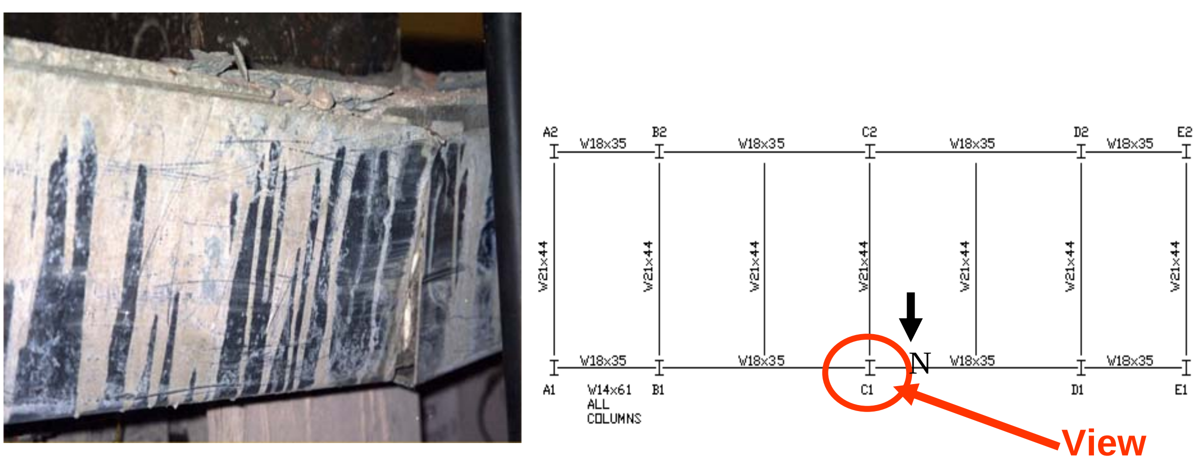


Figure 4.12. Deformation of Edge Beam on Top of Removed Column

Here the steel decking has bowed outwards and split (this happened in Test 1-19, but was not as extreme) and the concrete was broken loose and crushed. The slab pulled farther away from Column C2 than previously and the gap spread longitudinally along the top of the slab.

4.4. Behavior of Specimen During Test 1-35 (35 inches of Floor Drop)

Test 1-35 was conducted on June 4, 2001. A quarter-inch warm-up test was performed beforehand. All working instrumentation tested properly.

The test, of course, was intended to displace the test column 35 inches as previously mentioned. In order to meet this displacement, it was clear some changes to the testing were necessary. Since the last test, Test 1-24, had already maximized the actuator's loading capacity, it was clear that the loading capacity would have to be markedly increased in order to achieve an extra eleven inches of displacement. Therefore, a new testing apparatus was conceived, this time with a second 120-kip actuator.

Several alterations were also made to the instrumentation. Some of the strain gauges attached to the angles and the shear tab on the test column had failed already, or had already registered that the steel beneath them had gone through severe plastic deformation. Since these were either no longer working or necessary they were disconnected. New gauges were attached to the steel decking as mentioned, and the three gauges on the far shear tab remained intact and were used for this final test. All results from the instrumentation can be found in Appendix D4.

While several of the connecting elements around the test column did fail during this test, the structure itself did not fail and remained otherwise intact and able to bear the load. The structure did reach a displacement of 35 inches and an actuator tensile force of 190 kips as seen in the results of Figure 3-X, but at no point did its ability to take load decrease.

Here are the significant visible effects upon the structure:

Column A1:

Despite the heavy loading, no deformations were noticeable upon the column, any members framing into it or on any connections.

Column A2:

The behavior was similar to Column A1 as described previously.

Column B1:

At B1, considerable deformation was found. First, was the high level of damage incurred by the longitudinal girder between C1 and B1 as shown in Figure 4.13. Both bolts were lost during the experiment. The bolt pictured was not used as a functioning bolt during the experiment. Heavy bending of the beam flange and yielding of the web is evident. Yielding can be seen also on the web as well as all around the beam seat. In Figure 4.13 (right), this yielding can be seen more clearly.

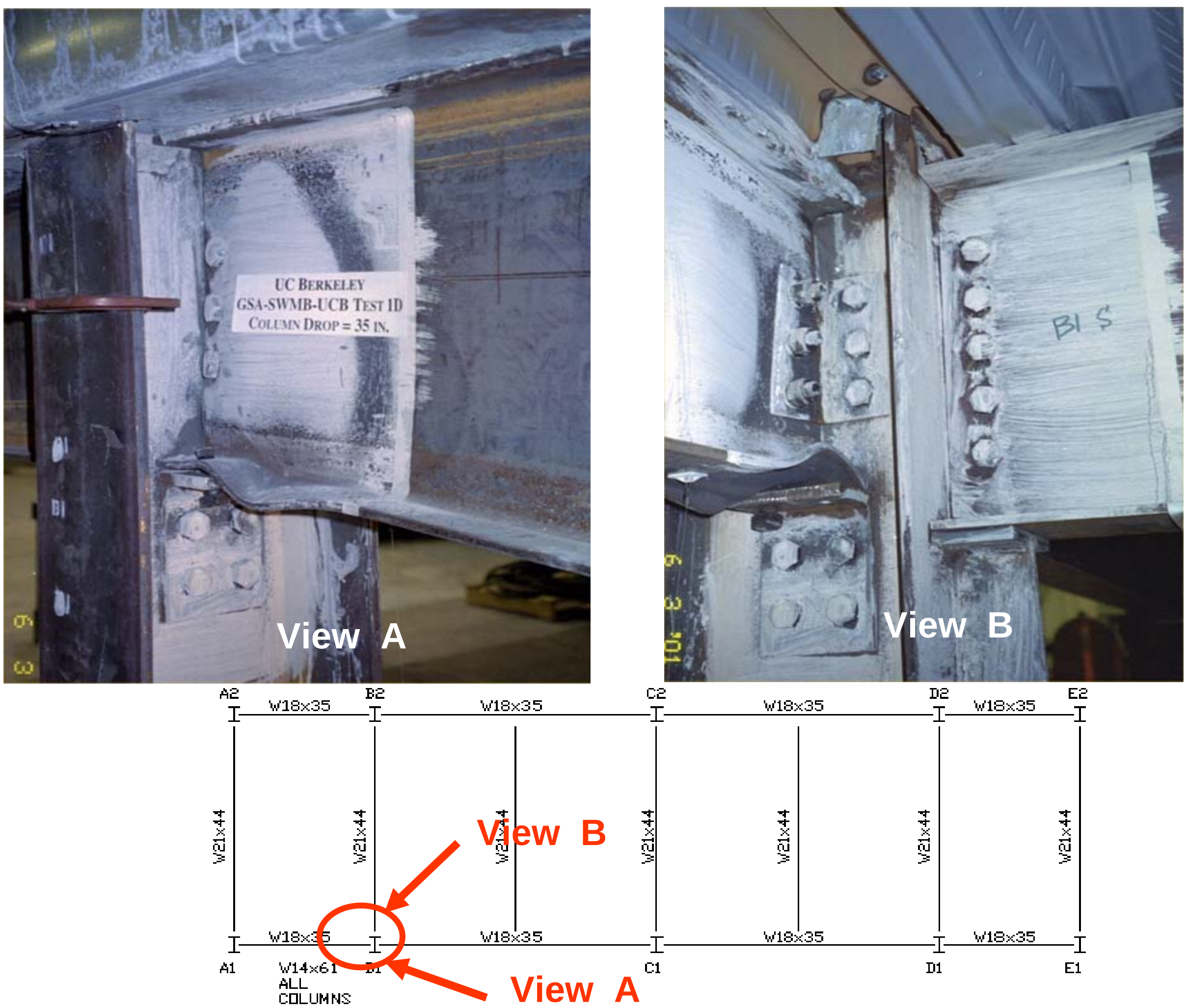


Figure 4.13. Two Views of Connection at Location of Column B1

The influence of the angle upon the web deformation is better illustrated here. We can see heavy yielding as the whitewash has flaked off at least an inch in every direction from the angle on the beam. No deformations occurred on the transverse beam pictured or its respective beam seat. However, above the beam there is clear deformation to the steel decking. This appears to be the hinge point for the decking which bent at the line of the transverse beam. There appeared to be horizontal striations on the south side of the south column flange nearest C1 indicating compression yielding had occurred. The striations extended from the flange edge for approximately three inches. No deformation was visible on the east side. Yielding can be seen also not just on the beam seat but on the web as well all around the beam seat.

Column B2:

Same as Column A1

Column C1:

In Test 1-24, the angle connecting the east longitudinal beam to the test column tore halfway. Test 1-35 completed this fracture and separated each steel connection from the test column, leaving the east beam connected only through the deck. Through close examination in Figure 4.14, the angle is seen to be higher on the right than on the left where the fracture occurred.

Heavy yielding has clearly occurred from the bottom right corner of the angle on the web as well as at both the top and bottom of the angle on the column web. During the test, all horizontal bolts on the seat angles fractured. However, they did not pop before some yielding of the column web on the sides and below the seat angles had occurred. The shear tab underwent large deformations around the bottom bolt as seen on the left in Figure 4.14. The area all around the bolts yielded and pulled outward extending the slot even farther. There also appears to be some yielding at the edge of the top flange there as well. One further note is that there appears to be some yielding at the base of the column in the center just above the bottom plate. Most likely, this is due to compression traveling downwards through the flanges and being transmitted through the plate. Conversely, the beams were pulling up through the web with a very high tension force from both sides. They appear to have met at the bottom of the web at the face of the plate

From the other side of the connection (View B in Figure 4.14) a little more of the connection is seen to be intact. The angle in Figure 4.14 (View B) did not separate from the column, though tearing began (only an inch in the center of the angle pulled apart). Therefore, this angle was still able to transmit stress to the beam. Yielding patterns on the west longitudinal beam seem similar to those on the east beam. There is very heavy yielding stemming from the bottom corner of the angle, and additional yielding above the top of the angle. The patterns on both sides of column web are the same, just mirrored. An interesting

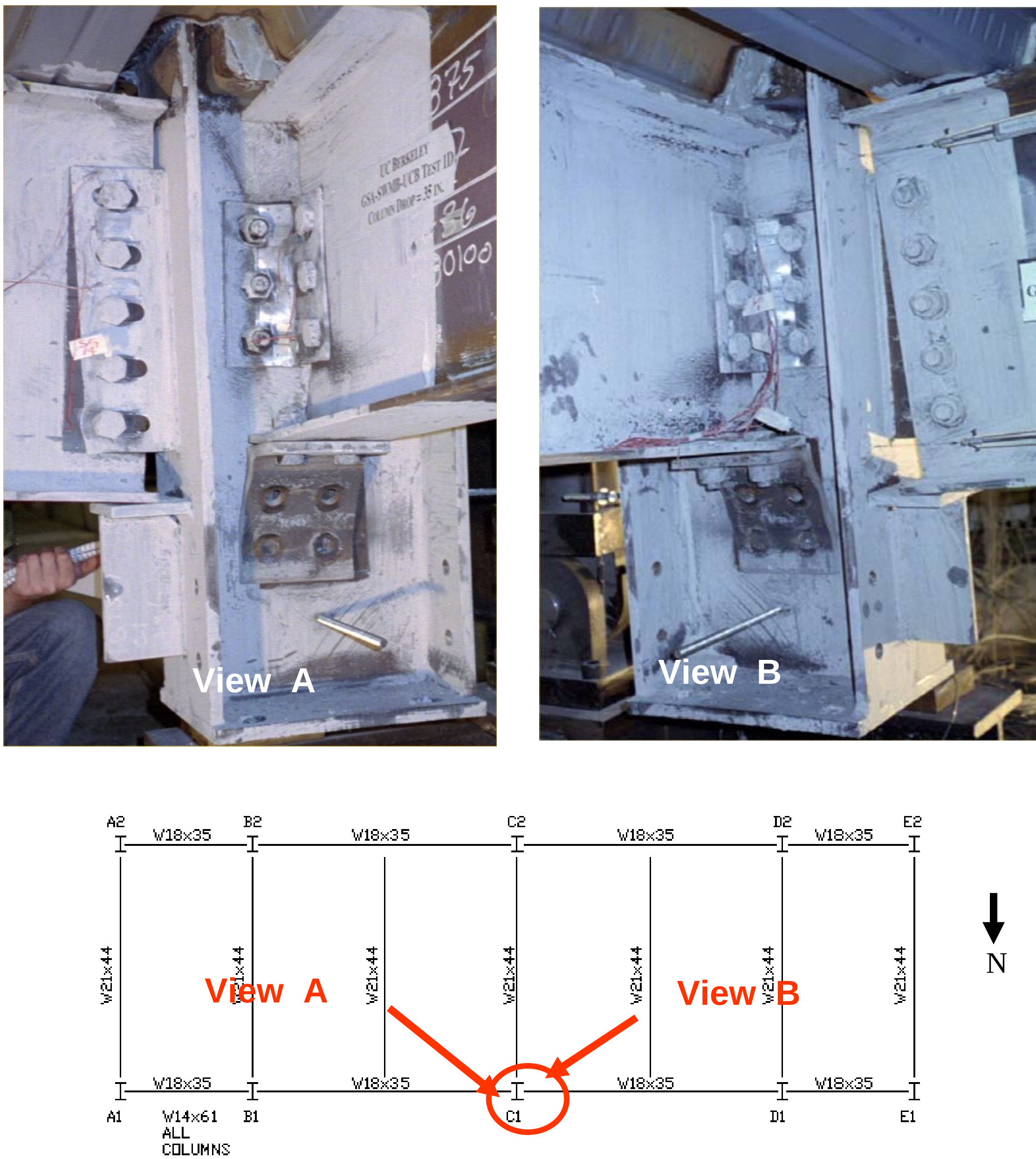


Figure 4.14. Views of Connection at Location of Removed Column

note is how little yielding is seen on the transverse beam from this side view. Other than deformation around the bottom bolt, very little yielding if any can be found to exist on the beam.

Column C2:

The transverse beam, in Figure 4.15 below, appears to have undergone much larger deformations at this end then Column C1. Yield lines extend horizontally in both directions from the bottom bolt and down to the flange. The flange itself has bent around the seat beneath it, which is clearly apparent on the other side in Figure 4.15 (View B). The shear tab underwent opposite, but similar deformations to its counterpart at the opposite (North) end. Here the top bolt pulled the tab outwards to allow more rotation in the connection.

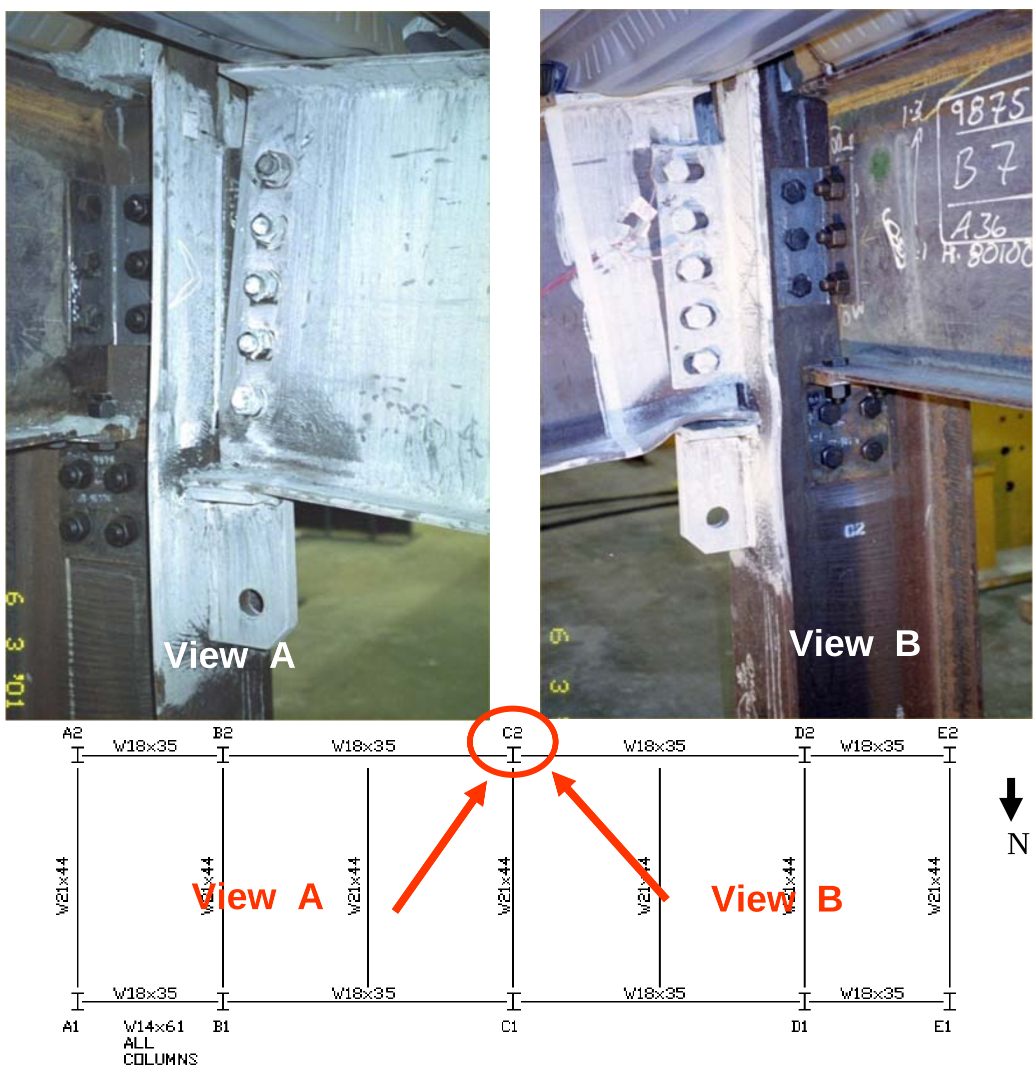


Figure 4.15. Views of Connection at Location of Column C2 at the End of Test 1-35

Column C2 also suffered some yielding. The flange was clearly deformed at both ends of the beam seat, and large yielding patterns can be seen around it at the base of the seat, especially in Figure 4.15 (View B). Close observation of column web indicated that it had yielded due to compression. Horizontal striations run all down the web underneath the longitudinal beam seat. Figure 4.15, (View B) shows a white streak that had run down the web and it is systematically missing at each striation line. Finally, the deck crushed down on the transverse beam.

Column D1:

The connections at Column D1 experienced similar failure to Column B1, only to a lesser degree. A crescent-shaped yielding pattern was seen on the north side of the longitudinal girder-web framing into C1. The flange at the base of the girder buckled around the beam flange, the bolts had been lost in Test 1-19. The web was strained also on the south side completely around the connecting angle. Some yielding of the column flange similar to Column B1 occurred as well.

Steel Deck and Concrete Floor:

Heavy compression crushing of the concrete was seen stemming from Column C1 towards Column C2 in the center of the structure. The crushing extended for approximately four to five feet from Column C1. Crushing was also very heavy on the overhang of the structure.

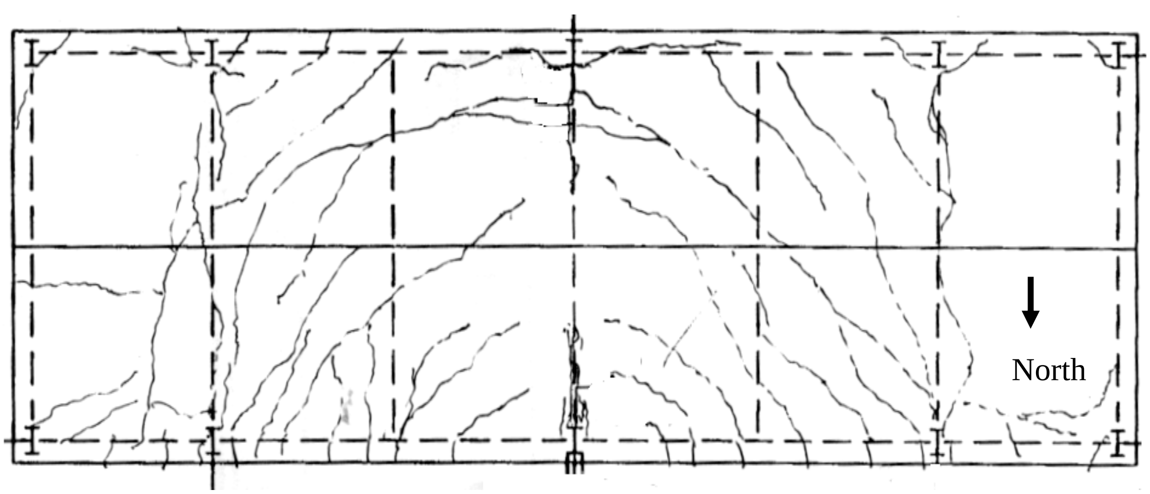


Figure 4.16. Plan View of Cracks on the Floor Slab at the End of All Tests

The area of concrete between Columns B1, C1 and D1 and the edge of the steel deck experienced extensive crushing. The vertical edge of the steel deck bowed outwards separating completely from the concrete for at least a quarter of a span in both directions of each column. The concrete was heavily damaged and spalled loose from the rebar. Another crack spanning orthogonal to the compression area at C1 ran longitudinally along the slab. This crack was unique because it was not in line with any tension or compression line, but was actually just over three feet from the edge of the slab – the approximate distance where the top reinforcement ended. This indicates that the reinforcement most likely carried quite a bit of force, and that cracking and spalling would have been much greater had it not been for the rebar. Figure 4.16 shows crack pattern at the end of tests.

Tension cracks were much greater than before. The slab pulled away from Column C2 all the way down to the steel deck for at least an inch on each side of the column. All previous cracks appeared to be both widened and lengthened. Many new cracks were exposed, and the slab appeared to be weakened, though certainly not incapacitated. The slab otherwise, held together quite well.

CHAPTER 5

Analysis and Discussion of Test Results

5.1. Summary of Data Obtained From the Tests

A complete set of all recorded data is provided in Appendix C of this report. With the data collected from instrumentation combined with the analysis of results a fuller and more precise picture of the actual behavior of the specimen was reached. All instrumentation data was processed and plotted and are given in Appendix C of this report starting on Page 121. Additional graphs derived from the recorded data are given at the end of Appendix C starting on Page 183 of this report.

The data given in Appendix C are categorized as: (1) Instrumentation data collected during the tests and; (2) Other data derived from the instrumentation data. In the following, the two sets of data given in Appendix C are explained.

1. Instrumentation data (starting on Page 122):

a. Data collected during Test 1-19
b. Data collected during Test 1-21
c. Data collected during Test 1-24
d. Data collected during Test 1-35.

The data presented in each of four above sections in a sequential manner consists of:

a. Structural load: This plot shows variation of total load in Column C1 versus time. The load includes the gravity load of the slab and the additional load applied by the actuator. The gravity load of the slab transferred to this column was measured by the actuator before the test started and the actuator was pushed down. Before starting the test, the actuator was acting as a support for the slab, thus, measuring the load transferred to the removed column.

b. Actuator Load: The load the actuator carried during the test. This is the only data that is not zeroed at the beginning of the respective test. The value shown at the time of zero (beginning of the test) indicates the gravity dead load of the floor supported by Column C1 before it was removed.
c. Actuator Displacement: The displacement, zeroed from start, was measured and recorded by a displacement transducer within each actuator.
d. Displacements DT1 through DT22: these graphs show the displacements recorded by displacement transducers DT1 through DT22. The transducers were mounted on the specimen at various locations to measure relative displacement of critical points on the specimen. The locations of transducers D1 through D22 are shown in Appendix B starting on Page 113.
e. Strains recorded by Strain Gauges SG1 through SG18: The graphs, starting on Page 160 of Appendix C give strains measured by Strain gauges. In these graphs, the vertical axis indicates “SGx (η STRAIN) “ is the strain. The strain gauges were zeroed at the beginning of the first test and read zero value at the start of the test. The strain gauges were also zeroed at the beginning of the Tests 1-21 and 1-24. However, to incorporate the permanent strains that might have occurred during Test 1-19, their initial values were adjusted to have the values at the end of Test 1-19. Test 1-35 has zeroed data for gauges SG1-SG14, and gauges SG15-SG17, previously SG16-SG18, follow the same pattern as they did in Tests 1-21 and 1-24 (i.e. they are zeroed and initial values from Test 1-19 subtracted off). For location of strain gauges, see Appendix B starting on Page 118.

2. Other graphs derived from above instrumentation data (starting on Page 183)

After presenting data recorded by strain gages (Part 1e above), the following graphs are presented:

a. Combined Force vs. Displacement- The force in Column C1 versus its displacement for all four tests are given. This is followed by a graph of actuator load versus actuator.
b. Cable Force vs., Cable Displacement- This graph (Page 184) shows variation of force measured in the cable versus the vertical downward drop of Column

C1. In order to measure the force in the cable, a displacement transducer was mounted on one of the four cables to measure its elongation. Then by multiplying the elongation by modulus of elasticity of cable given as 24,000 ksi, the axial force in the cable was estimated.

c. Concrete Stress vs. Strain- This graph (Page 185) shows the results of compression tests of concrete cylinders.
d. Horizontal Displacements of Specimen- Graphs in Pages 186 through 190 of Appendix C show variation of horizontal movements of points on the specimen as measured by displacement transducers.
e. Beam End Rotations- Graphs in Pages 190 through 193 show variation of end rotation of beams adjacent to Column C1. These rotations are calculated using measurements of displacement transducers near the top and bottom of the beam webs divided by the distance between the transducers.
f. Beam Displacements- Graphs in Pages 195 through 197 show variation of vertical displacement of beams adjacent to Column C1.

5.2. Discussion of Data

A. Load Displacement Data:

In the following four sections, load-displacement plots for all four tests are provided and discussed.

Test 1-19 (19 inches of Floor Drop)

The data obtained from Test 1-19 is critical, especially the force-displacement plot. The reason is that this test was performed upon the virgin specimen and initial behavior occurred here that could not be repeated in subsequent tests.

The force-displacement plot for Test 1-19 is shown in Figure 5.1. The force in the vertical axis is the load in the top portion of Column C1 after removal of its lower portion. The displacement in Figure 5.1 is the downward movement of the joint on top of the removed column (Column C1). On this plot, it is apparent that there were six noticeable jumps during the loading. Due to the large size of the test specimen, and the difficulty of viewing all portions of it at once, determining how those six "jumps" (in the plot) occurred was difficult. However, from the video gathered and witness observation, some possibilities can be put forth. Bolt failures at the center column are mostly verifiable, other causes though are hard to discern with certainty.

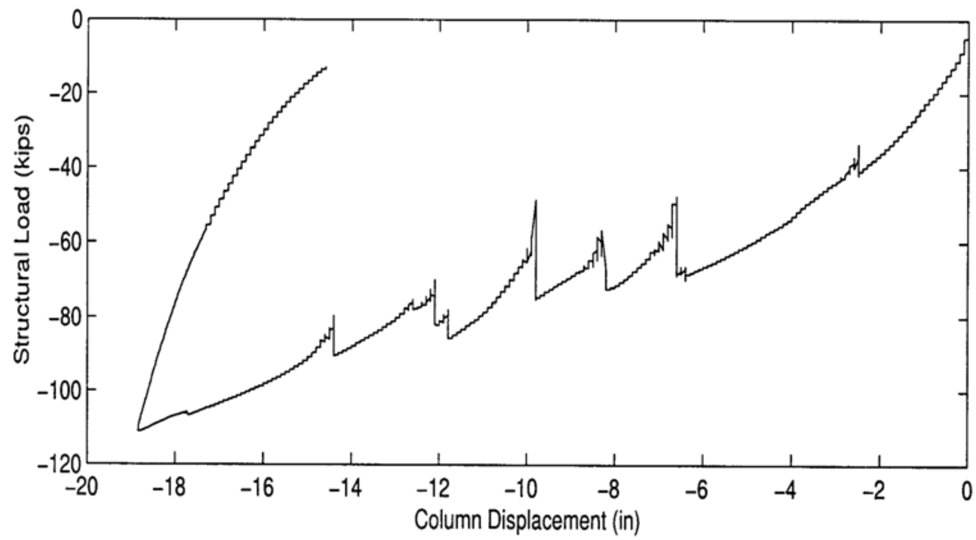


Figure 5.1. Variation of Force in the Column and Its and Displacement, Test 1-19 (Virgin Specimen)

The first discontinuity occurred very quickly in the test, and was most likely either local buckling of web of Beam B1-C1 at its B1 end and/or local buckling of web of Beam C1-D1 at its D1 end. It is also possible that slippage occurred at the horizontal or vertical shear planes of the bolted connections. For clarity, the column labeling is reprinted below in Figure 5.2.

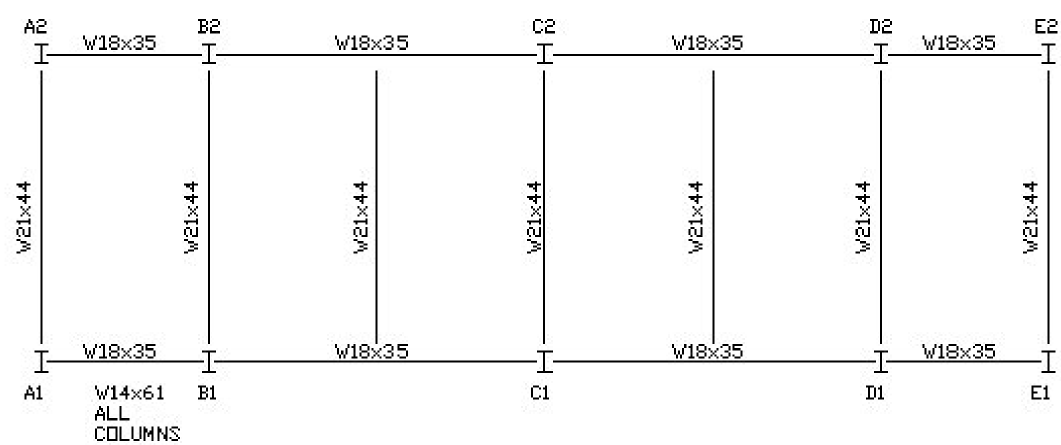


Figure 5.2. Plan View of Specimen with Location of Columns

In subsequent tests, no failure occurred this early; therefore, an initial, unrepeated buckling or slippage (or yielding) somewhere in the structure is the most likely event. This notion is furthered by the fact that the jump is very small

The second, third and fourth discontinuities (steps in the curve) in Figure 5.1 all appeared quite sharp. These occurred within a total of five seconds of each other. By studying the video of the test, it seems most likely all three were due to bolt failures. The second and fourth seem to have come from the beam seat area and bolt failure there. The noises in the video seem to indicate the third one overall was at the beam seat of C1.

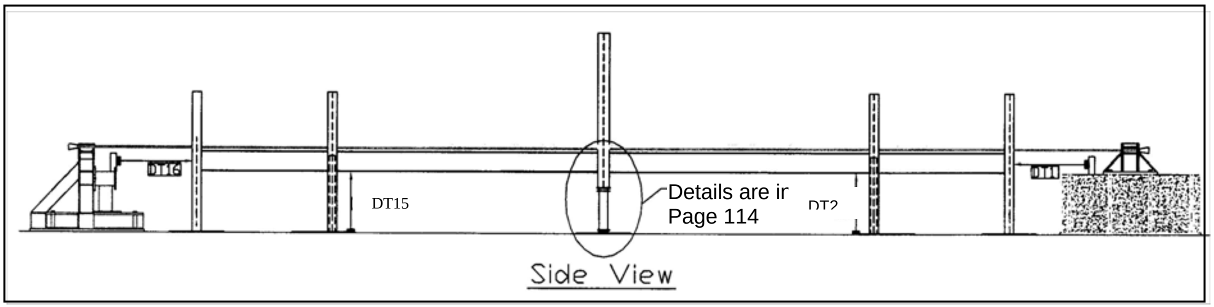


Figure 5.3. Transducers at Location of Column C1 (Middle Column)

This assessment comes from a careful analysis of the instruments at column locations B1 and D1. The layout of displacement transducers at these locations is shown in Figure 5.3 and in Pages 113 through 117 (Appendix C of this report). Displacement transducers DT3 and DT14 were measuring horizontal movements of columns B1 and D1. These transducers both registered heavy action at jumps two, four and six, indicating that whatever occurred in the structure shook both transducers B1 and D1 horizontally, therefore, the action most likely occurred at C1. Going then to the other two transducers at B1 and D1, which were transducers DT2 and DT15 respectively, one can observe that the third spike was registered at transducer DT2 but not at transducer DT15, seen below in Figures 5.4 and 5.5. Even more visible, was that the fifth spike registered very heavily upon transducer DT2 and did not return to its original path until the sixth spike. Therefore, bolt failure at location of column D1 was the most likely scenario for the fifth spike.

Bolt failure at D1 was not caught on camera and was obstructed from the view of the observers, however, the camera nearest D1 shook violently twice coinciding with loud noises in the structure near it. These most likely are the bolt failures, and perhaps the heavy local buckling at D1. The multiple jumps at the fifth spike may be due to several factors including the potential loss of bolts. Heavy yielding in both the angle and the beam seat at C1 immediately following the bolt fractures is one very likely option. Another option is sliding of the bolts in the slotted shear tab in the transverse beam. The final jump is smaller and with a high degree of certainty is not bolt failure or simple engagement of the deck. The cause for this could be any of the other potential options mentioned before, or perhaps could be something concerning the deck itself, a yielding perhaps of the steel decking and a subsequent breaking of the concrete above it. Crushing of the concrete at the face of the flange at C1 was occurring at this time as well, more so on the north side of the test column than the south. The north area, of course, was confined between the steel column and the steel decking.

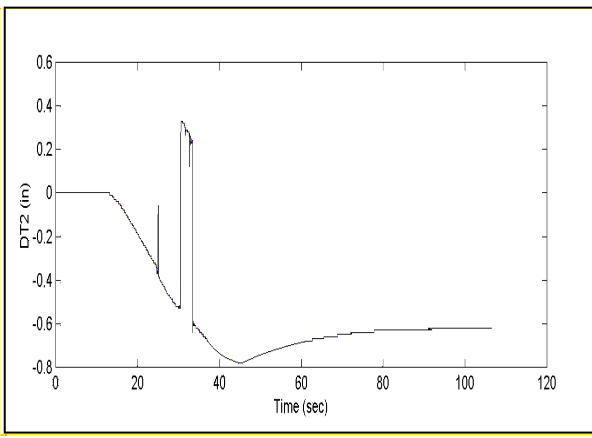


Figure 5.4. Plot of Disp. DT2 vs. Time

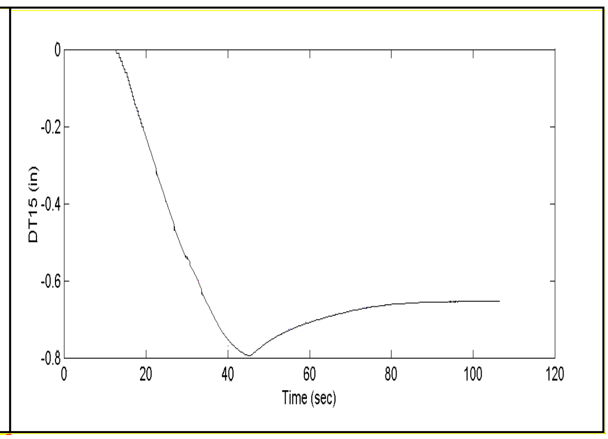


Figure 5.5. Plot of Disp. DT5 vs. Time

<u>b. Test 1-21 (21 inches of Floor Drop)</u>

Figure 5.6 shows load versus displacement plot for this test. Three discontinuities in the data occurred during Test 1-21, all very close together and all following the equivalent time of the fourth discontinuity, in Test 1-19. From evidence in the video, the bolts lost at joint C1 were most likely the causes of the first two discontinuities. A loud bang was heard at the time of the third discontinuity. The incident was off camera and may have been due to sliding of the bolts in the shear tab or a sharp buckling of the web of beams at location B1, C1, C2 or D1.

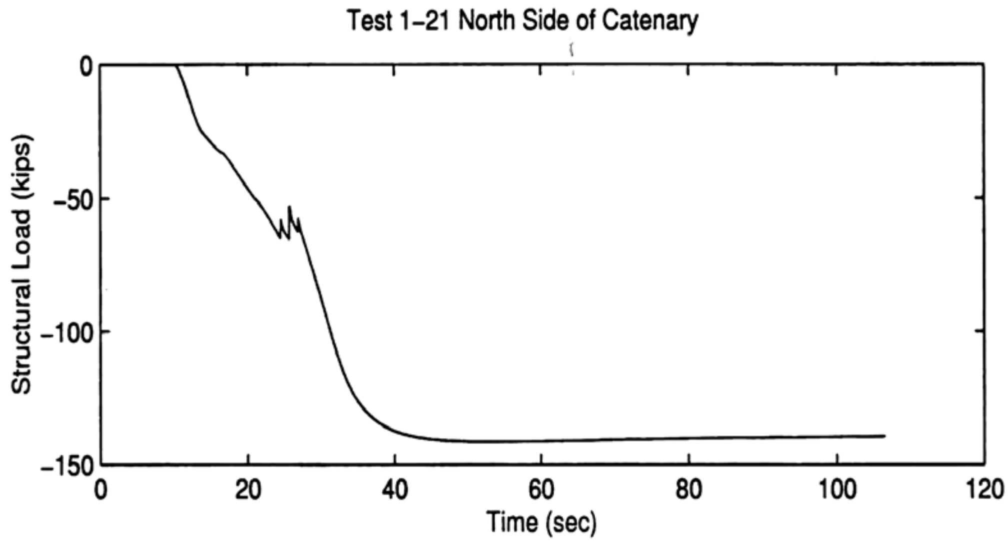


Figure 5.6. Variation of Force in the Column and Its and Displacement, Test 1-21

c. Test 1-24 (24 inches of Floor Drop)

Figure 5.7 shows force versus displacement for this test. Only one major spike occurred during this experiment. The video was quite good on this test, and in fact, it was descriptive enough to show that the north horizontal bolt failing followed shortly by the south bolt. The structure deformed with little load increase until shortly after this failure, when the stiffness was picked up by the structure again.

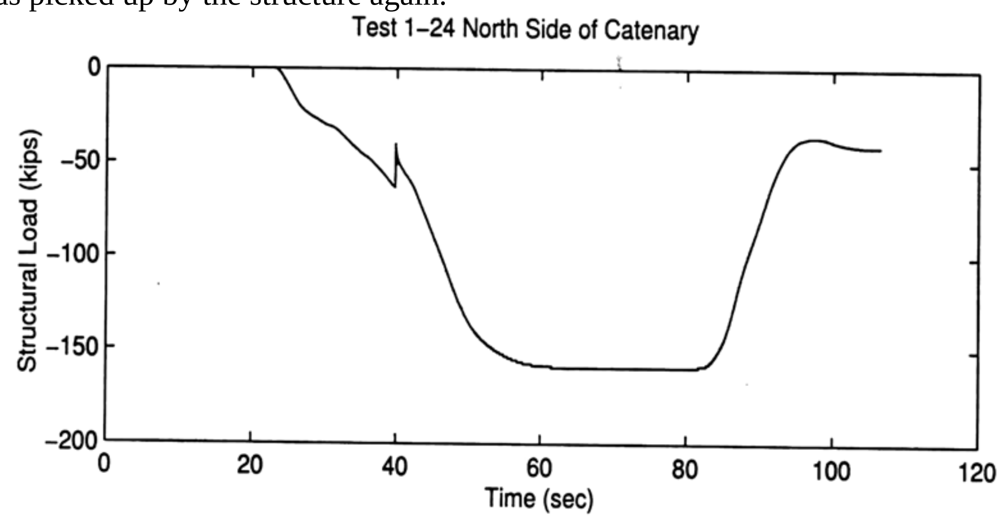


Figure 5.7. Variation of Force in the Column and Its and Displacement, Test 1-24

d. Test 1-35 (35 inches of Floor Drop)

Figure 5.8 shows force versus displacement for this test. As the structure groaned and stretched plastically back to the last point of deformation, the top row of horizontal bolts in the beam seat failed creating the first major discontinuity, seen in Figure 5.8.

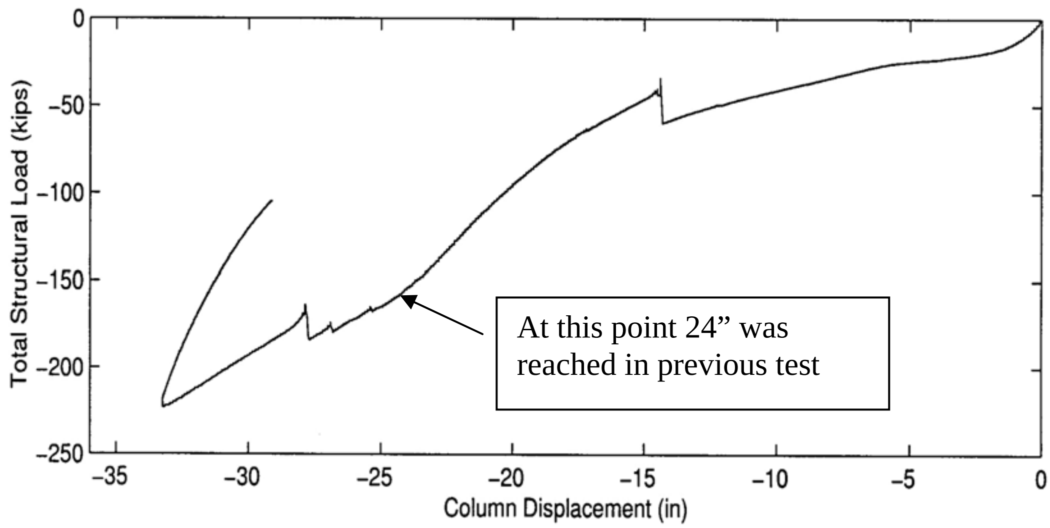


Figure 5.8. Variation of Force in the Column and Its and Displacement, Test 1-35

Following this event, the structure began to pick up the stiffness again. The gain in stiffness can be attributed to the fact that after failure of top row of bolts in the seat angles, the angle legs bent and created a path for additional Catenary action.

Despite several pops, a large break and dropping of the deck, the load-displacement remained relatively linear until just after 25 inches. Three discontinuities occurred between 25 and 29 inches of displacement. The failure of the bottom bolts in the beam seat angle at location of Column C1 is clearly one of these actions. The shearing of the east connecting web angle at C1 may be another, and the failure of the vertical beam seat bolts at B1 may be another possibility.

The structure still was maintaining a good degree of stiffness and strength at the actuator load of 190 kips and a displacement of more than 33 inches. Therefore, though the cables were taking more load, and the frame less, the structure as a whole was still accepting load at a good level of strength and stiffness.

B. Cable and End Bay Transducer Data:

During the tests, most instruments seemed to register logical data, i.e. the structure pulled downward and the instruments responded exactly as expected. However, data obtained from a few instruments were slightly intriguing. The end transducers DT1 and DT16 and the cable transducer, DT21, provided interesting results. Their layout is seen here again in Figure

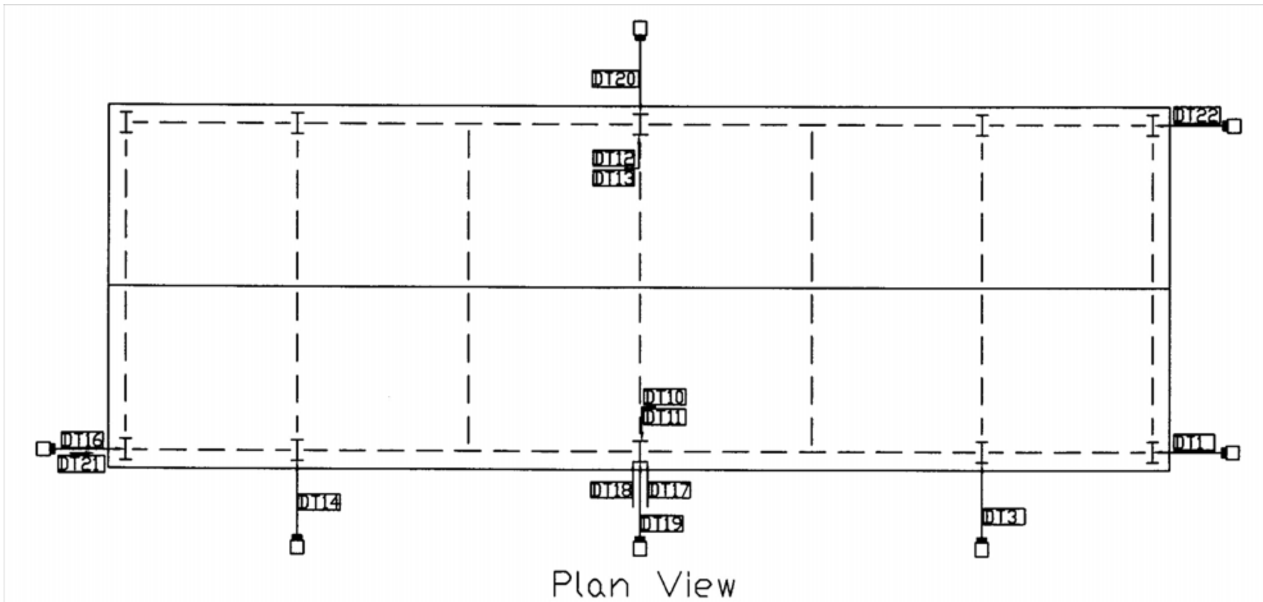


Figure 5.9. Plan View with Displacement Transducers

Displacement transducer DT21 was the instrument attached directly to the cable for measuring elongation of the cable over a given length, specifically 49 inches. From this, the strain was easily calculated, and the resulting force calculated by multiplying this strain, times the modulus of the cable, which is 24,000 ksi (ASTM, 1998), times the area of the 1-1/4 inch diameter cable. Figure 5.10, shows four curves obtained from this instrument on the force measured in the cable during each of the four tests. The curves for Test 1-21, 1-24 and 1-35 are rather straightforward. They show the elastic progression of loading on the cable as the system displaces. Clearly, the structure appears to be taking a greater percentage of the load at the beginning; however, after approximately twenty-five inches of displacement and reduced capacity of the structure, the cables begin to take the load more quickly.

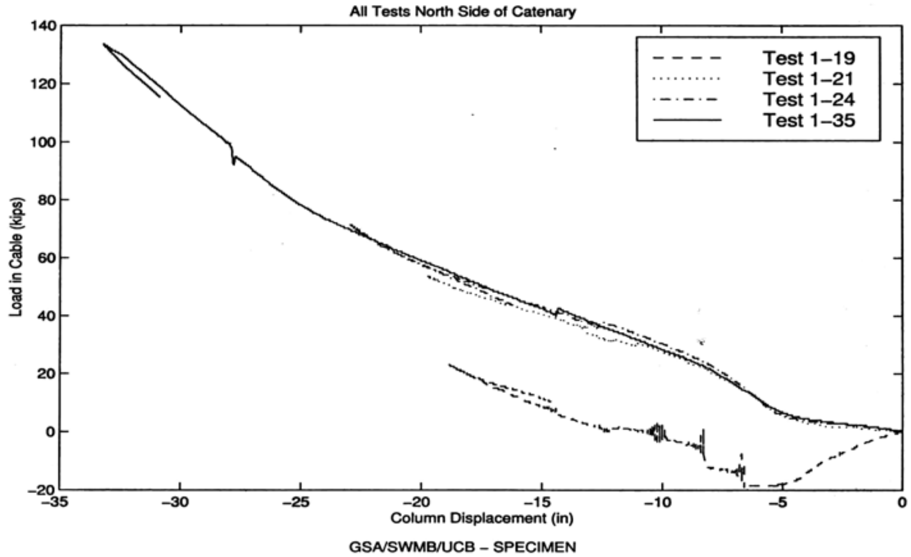


Figure 5.10. Variation of the Force measured in the Cable for All Four Tests

It is interesting to note that in Figure 5.10, the curve corresponding to Test 1-19 shows compression force developed in the cable up to a column displacement of about 5 inches. This can be because initially as the column started going down, the system was acting in bending. At this early stage and in this first test of virgin specimen, the cables due to bond between the cable and concrete, were acting as reinforcement within the slab and developing compression. When the column displacement reached about 5 inches, apparently, the bond between the cable and concrete broke and cable started developing Catenary axial tension independent of stresses in the concrete. During the subsequent tests since the bond between the cable and the concrete was not there, the cable did not develop compression, see Figure 5.10.

Displacement transducer DT22 received only “noise” during Tests 1-21 and 1-24 and did not return any clear data. During Test 1-35 it appeared to be pulled inward as expected, then paused and retracted before pulling away quickly shortly after the structure achieved 21 inches of displacement. The cause for this sudden retraction is unknown, though perhaps the failure of the load path through the deck and the frame reduced the resistance taken through E2.

C. Rotational Data:

Transducers DT6 through DT13 measured the rotational data of the three beams connecting at C1 and the transverse beam connecting to C2. This data was used to find the joint rotation of the beams, and also as a back up for the displacement measurements. The moment –rotations measured during the tests are given in Appendix C, starting on page 190. Very few of the rotations, when multiplied out to discern the vertical drop, matched up exactly. This might have been because none of the connections was idealized pin connections, and often rigidity of connection may have affected the rotation.

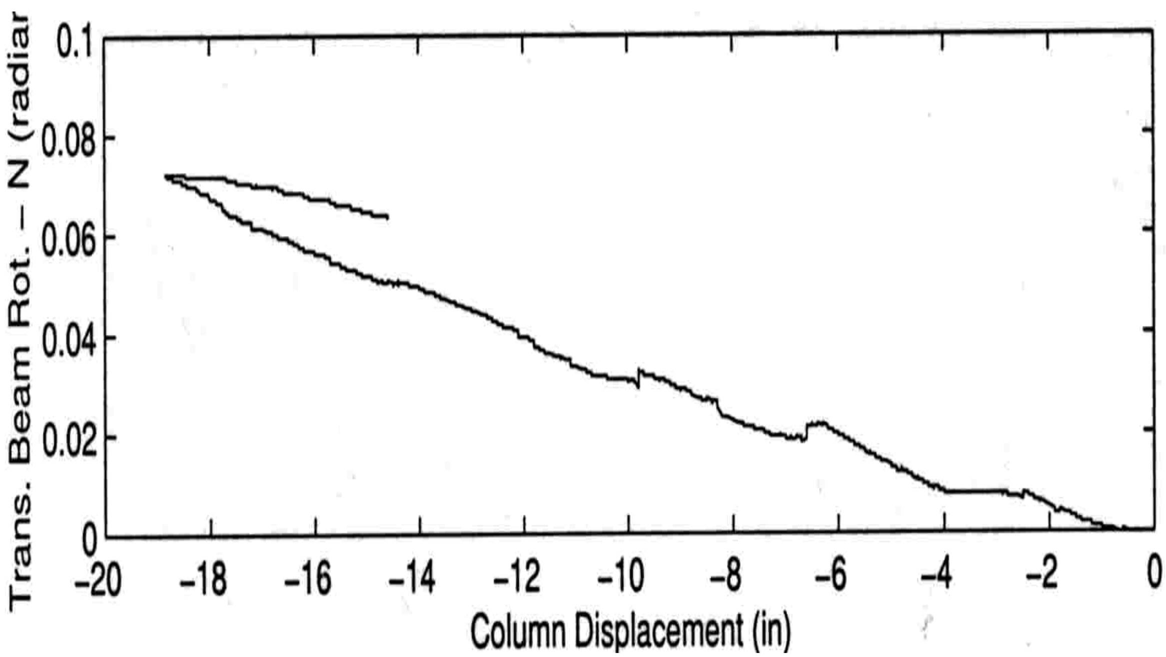


Figure 5.11. Variation of Transverse Beam End Rotation versus Time for Test 1-19

One of the interesting findings of these tests was that the long-slotted bolt holes provided at both ends of transverse beam C1-C2, worked very well and permitted almost free rotation of the transverse beam as the column was going down. Figures 5.11 show end rotation of Beam C1-C2 at its C2 end for Tests 1-19 and 1-35. As can be seen, the rotation of beam end was almost linear which indicated that end rotation took place primarily by bolt slipping and traveling inside the long slot of the shear tab connections. The long-slotted connections were able to rotate up to about 0.13 radians without any visible damage to the bolts, shear tabs or the beam web. At the end of test 1-35, slight bearing deformations were visible at the long-slotted holes.

D. Horizontal Displacement Data in the Transverse Direction:

Using the data collected from displacement transducers DT17-19, calculations were made to find the horizontal displacement of the test column. Since all three of these instruments collected data at an angle, their wire length along with the vertical displacement from transducer DT4 have simply been factored into a Pythagorean theorem to determine the horizontal distance. This distance was zeroed at the beginning of the test. The data are displayed in Appendix C starting on Page 186.

The test column appears to have pulled slightly towards the actuators during the first test, but less than an inch at the centerline of the beam-column connection for the final three tests. All graphs except for Test 1-35 show that the base of the column displaced less than an inch. The top of the column attached to transducer DT19 appears to have displaced less than two inches for all tests, and for Tests 1-21 and 1-24 roughly about one inch.

DT20 shows the horizontal displacement at C2. It appears that during Test 1-19, C2 was pulled just over an inch towards the test column. Most likely, the separation of the slab from column C2 and other local yielding kept the displacement to approximately half an inch for the final three tests.

The structure was braced at C2 to simulate the extension of the structure beyond C2. Therefore, this information proves that if the column is loaded vertically from above, there should be little lateral displacement of the column. Very importantly, the rest of the structure then is not inclined to collapse inwards towards the failed column.

E. Strain Gauges:

In the following sections, data obtained from strain gauges mounted on the structure are discussed. All the data are given in Appendix C starting on Page 160.

E.1. Structural Steel Connection Gauges

As previously mentioned, Tests 1-19,1-21 and 1-24 all shared the same instruments. The gauges for these tests were all attached to structural steel angles. The purpose of these angles was largely to help discern the yielding point of the angles. The yield strain of the steel was simply the design yield stress, 36 ksi, divided by the Young's Modulus of Elasticity for the steel, 29,000 ksi. This yield strain, in micro-strain (μStrain), is 1,241. If the probable assumption was made that the steel actually yielded above 40 ksi, this yield strain would increase to only 1,379 μStrain at 40 ksi. Results are in Appendices C.

The gauges on the angles at C1 indicated yielding in Test 1-19 at or around five seconds, and all within ten seconds. Some went nearly as high as 50,000 μstrain. Since they all yielded in the first test, the subsequent data for Test 1-21 was not very relevant, though some gauges achieved greater than 60,000 μStrain in 1-21 and 1-24 before failing and going out-of-range. Since the gauges may not have been restored completely to their initial position at the end of Test 1-19, the strain recorded in 1-21 and 1-24 can be considered as the minimum strain in the steel before failure.

Strain gauges SG13 - SG18 were attached to the shear tabs, with SG13 atop the tab at C1 and SG 16 atop the tab at C2. The gauges are arranged as shown in Figure 5.12. From the data it could be said that both gauges in the middle of the shear tabs did not yield in Test 1-19, however, both gauges at the tops of the tabs did. Although SG14 in the middle at C1, came very close to its design yield strain, its actual yield strain was most likely just high enough that it remained elastic. However, it probably yielded during Test 1-19 when it achieved a strain higher than 1500 micro-strain. Strain gauge SG17, on the middle shear tab gauge at C2, yielded in Test 1-21. Strain gauge SG18 did not yield apparently until during Test 1-35.

E.2. Steel Deck Gauges:

The first fourteen gauges in Test 1-35 were all attached to the deck. It was clear that the deck was taking a good deal of stress during the test, but exactly how much was very uncertain. The purpose of gauging the deck was to determine what load was being applied to the steel decking. The first twelve gauges were part of four rosettes. Strain gauges SG4 and SG10 were faulty and their results are not included in discussion below. Strain gauges SG11 and SG12 were linear gauges.

The layout of these gauges is depicted below in Figure 5.10. The minimum yield stress of the decking sheet steel was 38 ksi, and the modulus was 29,000 ksi (Verco, 2000).

Therefore, by similar computations as before, the maximum yield strain for the steel deck was 1,310 micro-Strain.

Strain gauges SG1 – SG12 show that none of those areas of the deck carried a force large enough to yield the steel. Strain gauges SG1 through SG6 registered stresses between 8.1 and 18.9 ksi. Strain gauges SG7 through SG12 showed much higher strains despite the fact that they were further from the column. They ranged from 8.7 ksi to 35.4 ksi. It appears that the area closer to the test column was under heavier compression, and therefore did not experience high tensile stresses. Strain gauges SG13 and SG14 were very close to the test column as seen in Figure 5.10. Strain gauge SG13 experienced 29.3 ksi; while strain gauge SG14 yielded and achieved 2600 micro-Strain, nearly double the minimum yield strain for the decking. These areas were under very high compression seen from the concrete crushing above and the steel deformations below, and most likely yielded in compression. The area around these gauges was quite deformed, and the corrugation of the steel deck flattened out as it was pressed against the transverse beam between columns C1 and C2.

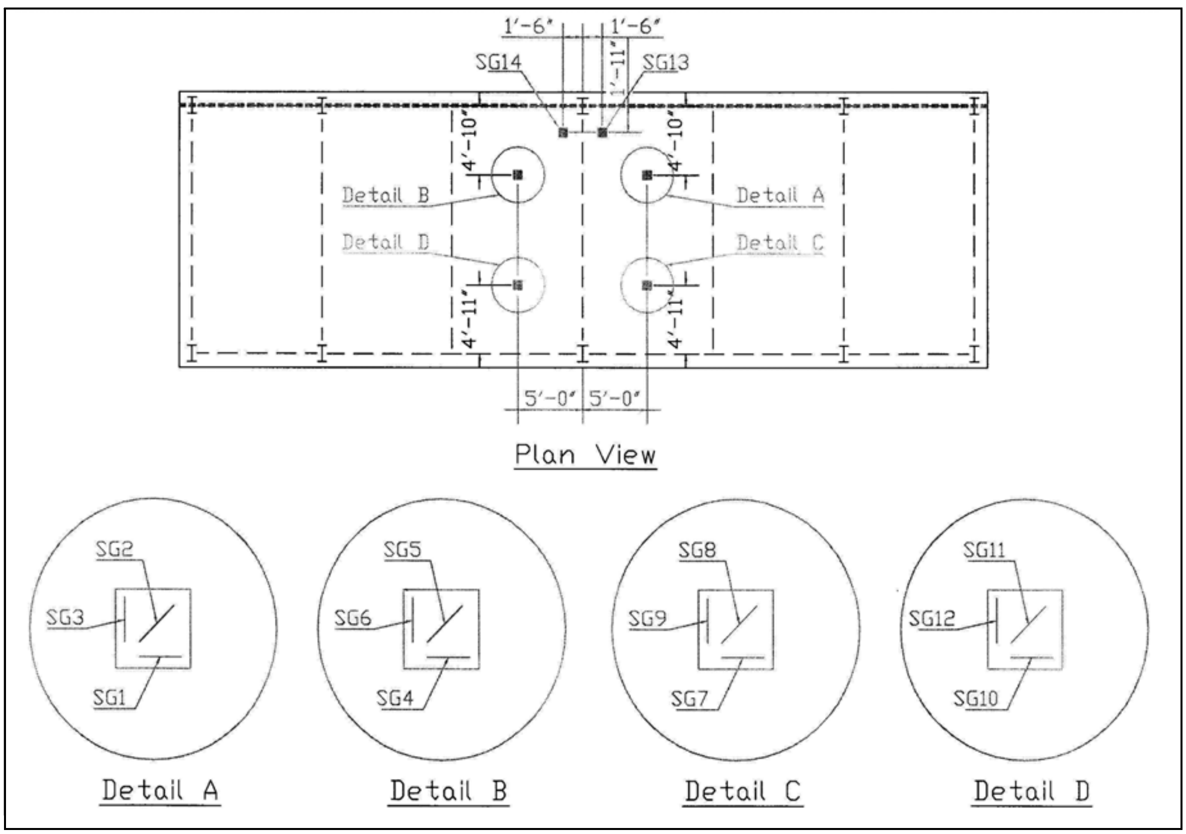


Figure 5.12. Deck Strain Gauge Layout, Test 1-35

CHAPTER 6

Summary, Conclusions and Design Recommendations

6.1. Summary

A full-scale one story steel structure was tested to study performance of a cable-based system to prevent progressive collapse of the structure in the event of a terrorist attack and removal of a column. The cable-based system consisted of four 1-1/4 inch diameter steel cables placed in the concrete floor slab along the exterior column line. The role of the cables was to redistribute the load of the eliminated column to the rest of the structure and to prevent progressive collapse. Figure 6.1 shows schematic view of the test specimen before and after the removal of the middle column.

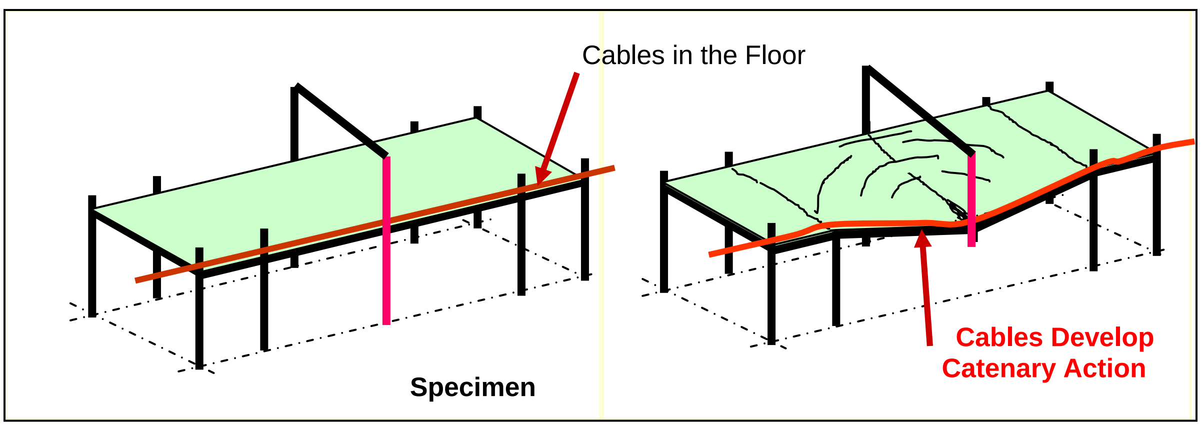


Figure 6.1. Schematic View of Test Specimen Before and After the Tests

Prior to the tests, realistic analytical studies were conducted to understand the behavior as well as to develop a safe yet realistic way of testing the specimen. The analytical phase of the project consisted of building a realistic model of the test specimen and subjecting the model to two types of loading: (a) the weight of the specimen and concrete blocks placed on top of the floor slab to simulate total dead and live load and; (b) the weight of the specimen and the force applied to the column by hydraulic actuators. The main objective of analytical studies was to compare test results under above conditions (a) and (b) and to find out what actuator load applied with what velocity will create the same effects in the specimen that the concrete blocks placed on the floor slab would create. For safety reasons, it was preferred to do the test and apply gravity effects in a very realistic manner using hydraulic actuators instead of free fall of the concrete blocks placed on the floor slab.

After confirming that using hydraulic actuators will result in reliable and relevant data, the test set-up and test procedures were developed and four tests were conducted on the specimen. All tests consisted of removing the bottom portion of the middle column, Figure 6.1, and pushing down the remaining top portion of the column a certain predetermined amount. The tests denoted as 1-19, 1-21, 1-24 and 1-35 had a maximum floor drop of about 19.8, 21, 24 and 35 inches respectively. Figures 6.2 and 6.3 show the appearance of specimen during the tests.

Figure 6.2. Appearance of Slab at the End of 19-Inch Drop Test

The main damage to the specimen was in the form of cracking of the deck concrete (minor cracks), failure of the bolts connecting the beams on frame line 1 to the joint that was dropped, and buckling of the web and flange at the far end of these beams. Table 6.1 shows maximum displacement and maximum force at that displacement for all four tests. The results of the four tests in terms of load in the middle column versus the drop of the top joint of that column are given in the following Figure 6.4. Also shown in Figure 6.4 is an envelope curve of all four tests (thick dashed line).

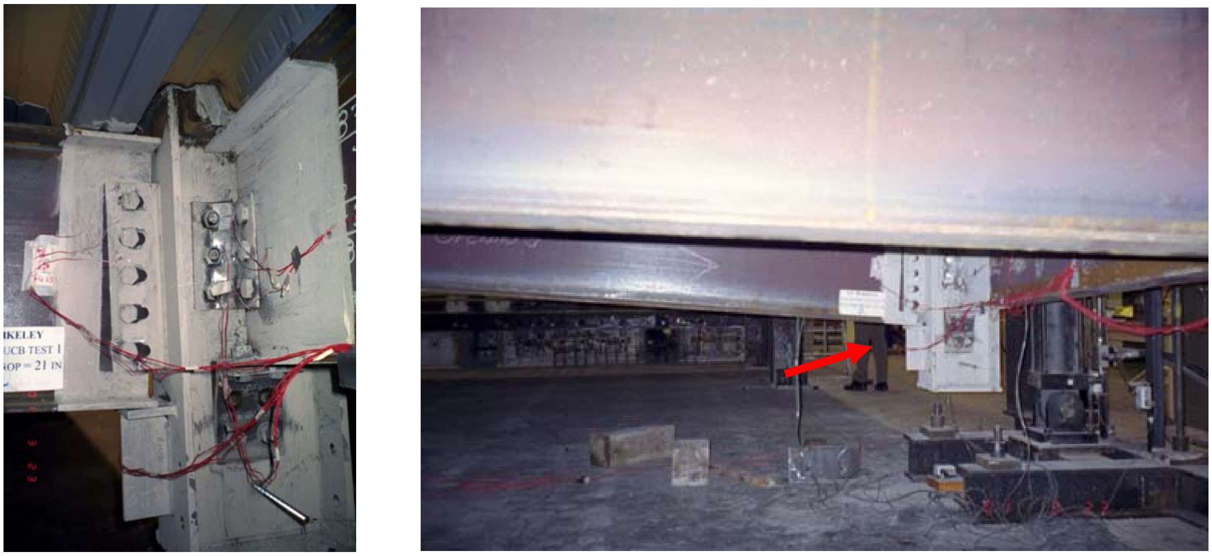


Figure 6.3. Appearance of Connection at Top of Dropped Column

Table 6.1. Maximum Force and Corresponding Displacement of Removed Column

Test Number	**Maximum Force (kips)**	**Maximum Displacement (in.)**	**Major Damage to Components**
1	100	19	Top 2 bolts on seat angle of middle column broke. Local buckling of beam web and bottom flange.
2	110	21	Top 2 bolts on seat angle of middle column that were replaced broke. End bolts on beams sheared.
3	125	24	Same as Test 2.
4	190	35	All four bolts on the bottom seat failed. Web angle in middle connection fractured.

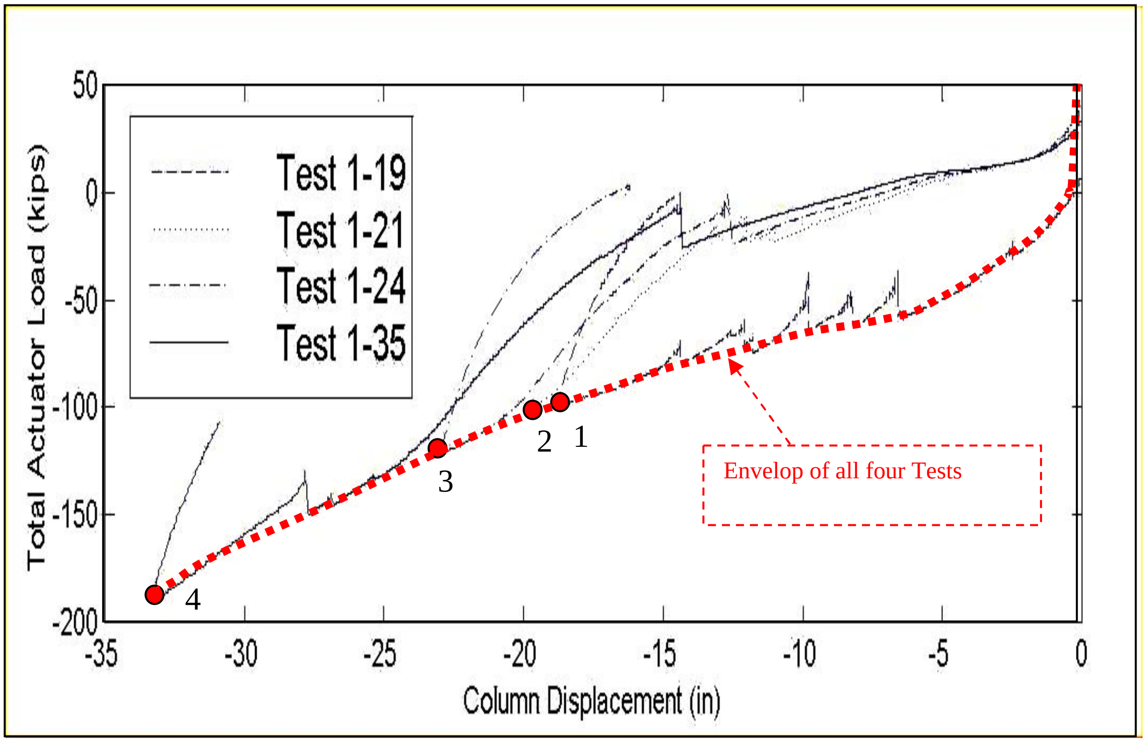


Figure 6.4. Force-Displacement Plots for Four Tests and the Envelop of Tests

6.2. Conclusions

Based on the results of these tests the following conclusions were reached:

1. The analyses indicated that hydraulic jacks could be used to simulate the effect of gravity on the floor. The load applied by the hydraulic jacks should be about 30% higher than the gravity load to simulate dynamic effects of the gravity load. This is shown in Figure 6.5. The tests reported herein were conducted on the North side frame of the specimen where four 1-1/4 inch cables were placed in the slab. For more details of structure, the reader is referred to Appendices A and B of this report. As a continuation of the tests done on the North side, a series of similar tests were conducted on the South side frame of the specimen. These tests were partially funded by the American Institute of Steel Construction and the results are reported in Reference (Astaneh-Asl et al, 2001). In Figure 6.6, the envelop of the load-displacement curves for both, North and South side frames, are shown.

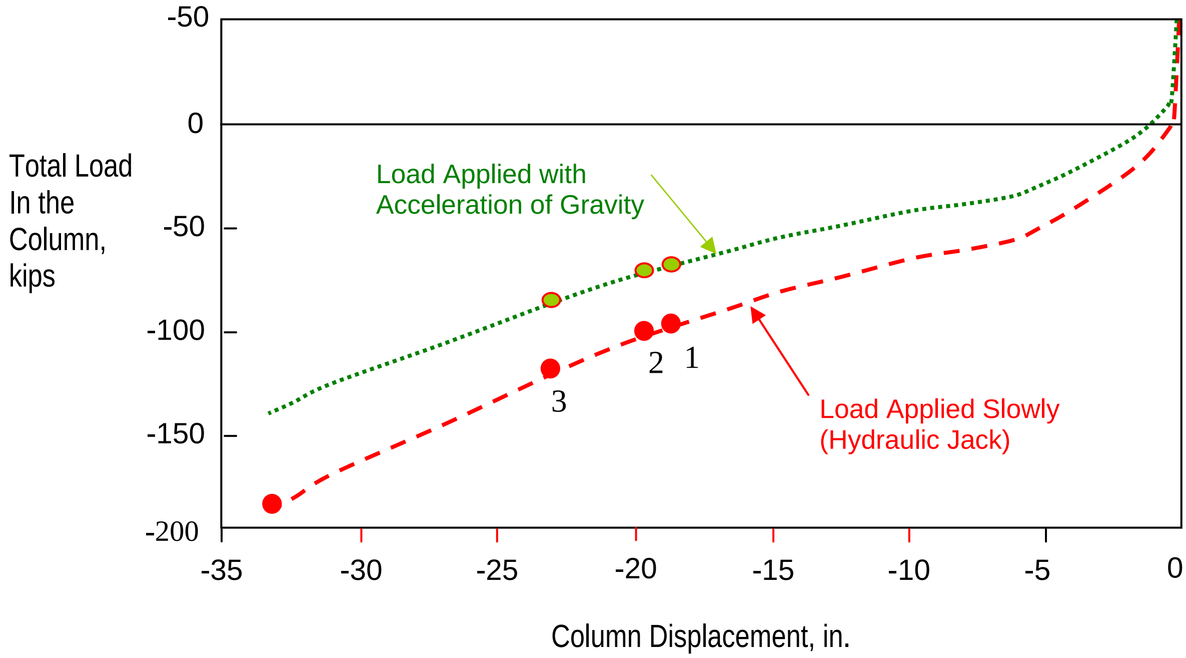
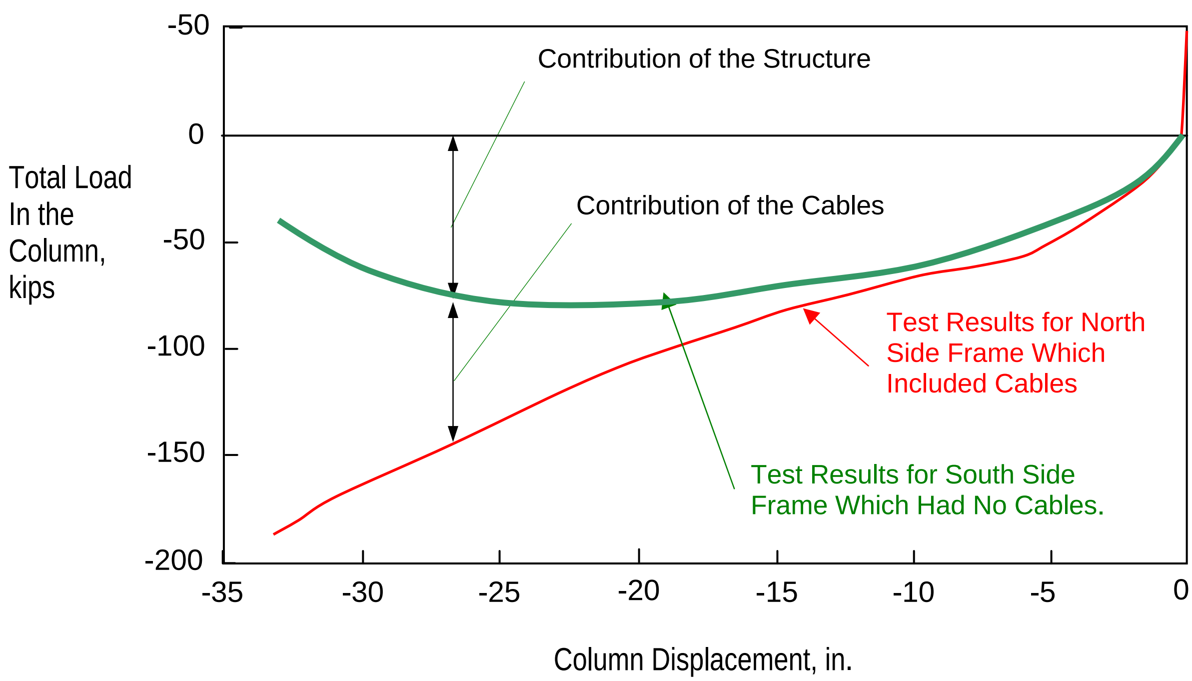


Figure 6.5. Comparison of Performance of Specimen Using Hydraulic Jacks with Direct Application of Gravity Load



The steel frames on the North and South side of the specimen were identical except for the steel cables and the reinforcement of the floor slab around the cables, which were present in North side frame only. Figure 6.6 shows contribution of cables and the structure to the overall strength and stiffness of the system. Initially, up to a column load of about 60 kips corresponding to a drop of about 10 inches, the resistance to column drop primarily was provided by the structure itself. Beyond this point, when the top two bolts in the seat angle supports fractured, some of the resistance to column load came from the cables as well. As the column load increased, the contribution of the structure decreased and the cable carried most of the load. Beyond about 27 inches of column drop, the cable was carrying more than half of the column load as can be seen in Figure 6.6.

2. The system performed well and the damage was limited to fracture of the bolts connecting vertical legs of the seat angles to the middle column, relatively minor cracks in the slab and local buckling of the beam webs.
3. The bolted seat angles performed as expected. During the tests. First, the two bolts (on the top row) on the vertical leg of the angle fractured. Later, as the load in the column increased the two bottom row bolts also fractured. This of course left the web single angles the only elements to carry shear in the connection. The web single angles fractured through the fillet during the final test (35 inch drop).
4. The long-slotted shear tab connections on the transverse beams (North-South beams) performed well and as intended. The bolts traveled the length of the slotted holes on the shear tab and the end of the beam was allowed to rotate more than 0.14 radians without serious damage to the connection.
5. When the transverse beam rotated large amounts, the bottom flange of the beam was bearing on the stiffened “safety” seats. These safety seats were placed under the transverse beams, with a ¼” gap, to prevent the beam from collapse in case the shear tab connections fail. The excellent performance of shear tabs indicated that these safety seats might not be needed.
6. Other than during the very early stages of first test (virgin specimen), it appeared that there was not any significant bound between steel cables and the concrete floor slab.

In other words, almost all the tension force in the cable was transferred to the end anchors without any significant amount transferred to the floor slab and deck.

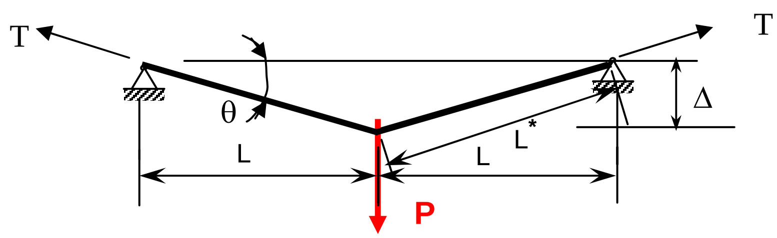
7. The connection of cables to the columns and top of the beams performed well and there was no sign of any distress or cracking of concrete in these areas.
8. About two feet wide strip of the floor slab on the North edge of slab (where the cables also were placed) had reinforcement. This area performed well and there was only hair cracks visible on the slab on this strip.
9. The steel deck performed well. There was very small amount of permanent deformation of the deck visible.
10. It was clear that after removal of the middle column, initially the beams and bolted seat angles were supporting some of the floor load by developing Catenary action. However, after reaching a column load of about 60 kips, when the bolts on the seat angle fractured, the cables were the primary elements supporting the load. A sister project (Astaneh-Asl et al, 2001) conducted on the south side of the specimen (which did not have the cables) indicated that indeed the steel structure alone could support about 60 kips in the column before the bolts on the seat angle fractured.
11. The cables were connected to two anchors outside the test specimen. The anchors performed well and indicated that in actual structure, attention should be paid to design of mechanisms that can transfer the Catenary tension of the cables.

6.3. Design Recommendations

Based on findings of this research project, following design-oriented suggestions and recommendations have been formulated and proposed.

6.3.a. Catenary Force in the Cables

A reliable estimate of the Catenary tension force in the cable can be obtained by applying classic Catenary equations, given by Popov (1990), to this case. Popov (1990) presents the basic case of Catenary action and equations governing the applied lateral load, Catenary tension and the displacement. The case treated by Popov (1990) and the equations are shown in Figure 6.8.



$$\Delta = L \tan\theta \text{ and } L^* = L / \cos\theta$$

$$\frac{TL^*}{AE} = L^* - L \quad \text{and} \quad T = AE(1-\cos\theta)$$

Figure 6.7. Equations for Catenary Action by Popov (1990)

Following the same approach as in Popov (1990) the equations governing Catenary action for the cables used in the specimen structure are established in the following section. In driving the equations, the contributions of the structure itself are not included. The contribution of structure itself, which includes, contribution of the deck, reinforcement in the slab, shear studs, connections and the beams is very complex and a simple but reliable mathematical equation could not be established. Added to the complexity is the fact that high level of material non-linearity occurs in the structure as the floor deforms and the column drops.

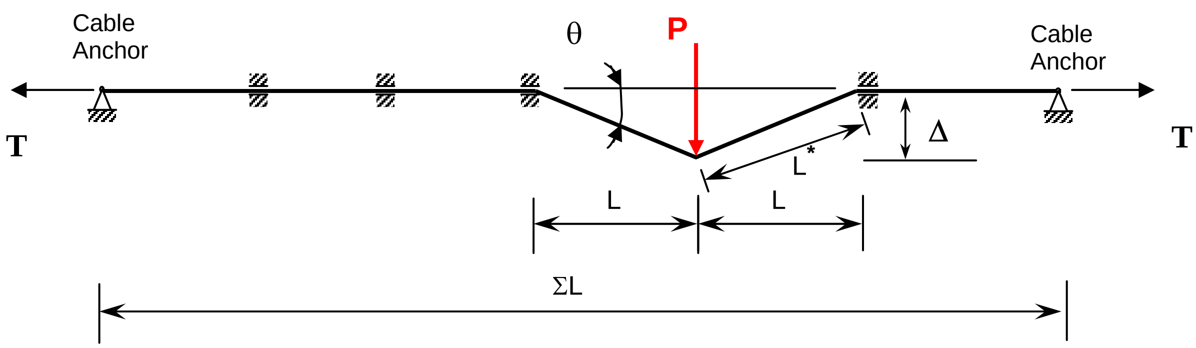


Figure 6.8. Model of the Cables Placed in the Floor Slab of a Typical Structure

Considering the cables only and with reference to Figure 6.8 and to Popov (1990), the relationships that govern this case can be written as:

$$P = 2T \sin\theta \tag{6.1}$$

$$\Delta = L \tan\theta \text{ and } L^* = L / \cos\theta \tag{6.2}$$

$$\frac{T(\Sigma\ L)}{AE} = 2(L^* - L) \tag{6.3}$$

$$T = \frac{2LAE(1-\cos\theta)}{(\cos\theta)(\Sigma\ L)} \tag{6.4}$$

The terms in above equations are shown in Figure 6.8. Using Catigliano's first theorem, (Oden and Ripperger, 1981), one can obtain similar equations for catenary force. This is done and following equations are obtained.

$$P = \sqrt{\frac{4T^3(\sum L)}{AEL}} \tag{6.5}$$

$$\Delta = \sqrt[3]{\frac{P(\sum L)(L)^2}{2AE}} \tag{6.6}$$

$$T = \sqrt[3]{\frac{P^2 AEL}{4(\sum L)}} \tag{6.7}$$

Equations 6.1 through 6.4, based on Popov's derivations and Equations 6.5 through 6.7 based on Castigliano's first theorem give almost identical values and both are based on equilibrium and conservation of energy.

Figure 6.9 shows the contribution of cable, predicted by above equations, superimposed on the test results. Curve OA in the figure shows variation of the force in the column versus drop of the column for South frame test where there were no cables in the floor slab. This curves provides very valuable information on how much resistance was provided by steel frame itself. Curve OB shows variation of the force in the column versus drop of the column as predicted by the above catenary force equations. This curve is a good representation of resistance of cables alone. If one adds curve OA and OB, a reasonably accurate estimate of total capacity of the system can be obtained. This is done and the result is Curve OC (full line in the figure). Curve OD is the test results for the North frame test where there were four 1-1/4 inch diameter cables inside the slab. Comparing curves OC and OD and their closeness, it appears that Equations 6.1 through 6.7 are predicting contributions of the cable quite well. Figure 6.10 shows force in one cable measured during the tests and as predicted by Catenary equations 6.1 through 6.7

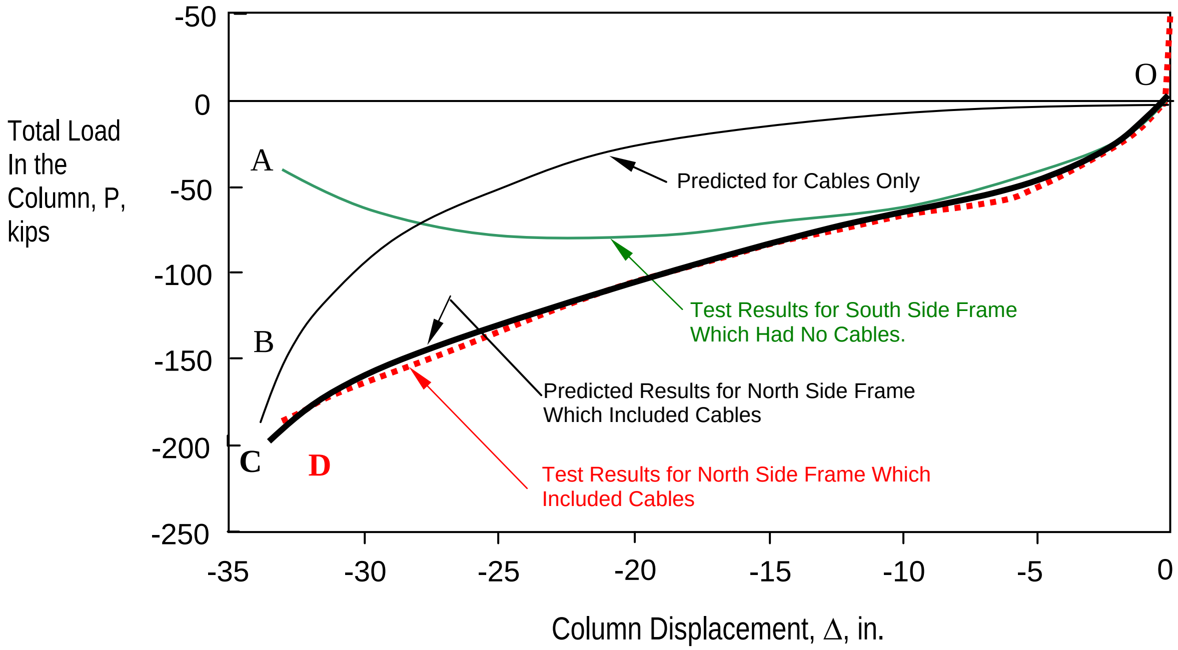


Figure 6.9. Predicted Cable Contributions and the Contributions of the Structure

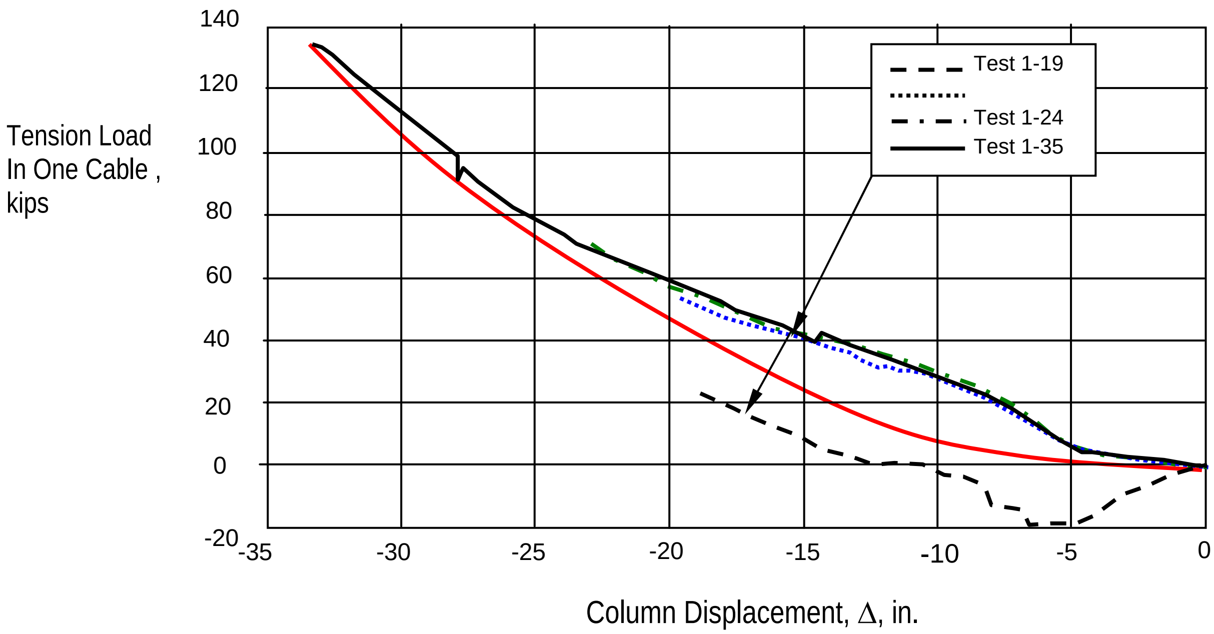


Figure 6.10. Predicted (Full Line Curve) and Measured Force in One Cable

6.3.b. Design of Connections

Based on performance of the test specimen it appears that the connections performed well. The only modification to design appears to be in design of seat angles. In the test specimen, top row of bolts connecting the seat angle to column flange failed. It is suggested that the seat angle be designed such that when the horizontal leg is pulled away from the column, yielding of angle leg be the governing mode and not the tension failure of the bolts. This can be done by increasing diameter of these bolts and decreasing the thickness of the seat angle.

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