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In-Situ Structural Evaluation of a Steel-Concrete Composite Floor System

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IN-SITU STRUCTURAL EVALUATION OF A STEEL-CONCRETE COMPOSITE FLOOR SYSTEM

By

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The application of steel joists to floor construction can be traced back more than 100 years to the use of a sheet steel joist in the State of New York Bank Building in 1855. Since that time various forms of joists have been developed and exploited. As a result, two general types of joists are now on the market: a) Solid web joists; b) Open web, or truss type, steel joists.

In order to determine the strength, stiffness, and behavior of these structural sections under load, representative open web steel joists have been tested at the University of Miami, School of Nursing Building (building about to be demolished).

Using two hydraulic jacks to apply the load at eight different locations along the strip, the assessment of the ultimate structural performance of the floor system to positive moments in correspondence of selected strips was possible. After analyzing the data collected from the sensors through the data acquisition system, it was concluded that the results obtained from the Finite Element model were consistent compared to the results obtained from the experimental approach, helping to understand better the behavior of this structural system. A recommendation for further study is enclosed.

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER I

INTRODUCTION

1.1. THESIS OUTLINE

Starting with the introduction, the following chart (Figure 1) describes the different parts of the body of this thesis. Each main title represents a chapter and under each one there are the different sections of that corresponding chapter.

The purpose of this outline is to help the reader to understand better the organization of this thesis and the way the author formatted it, so the reader can easily go through the different parts.

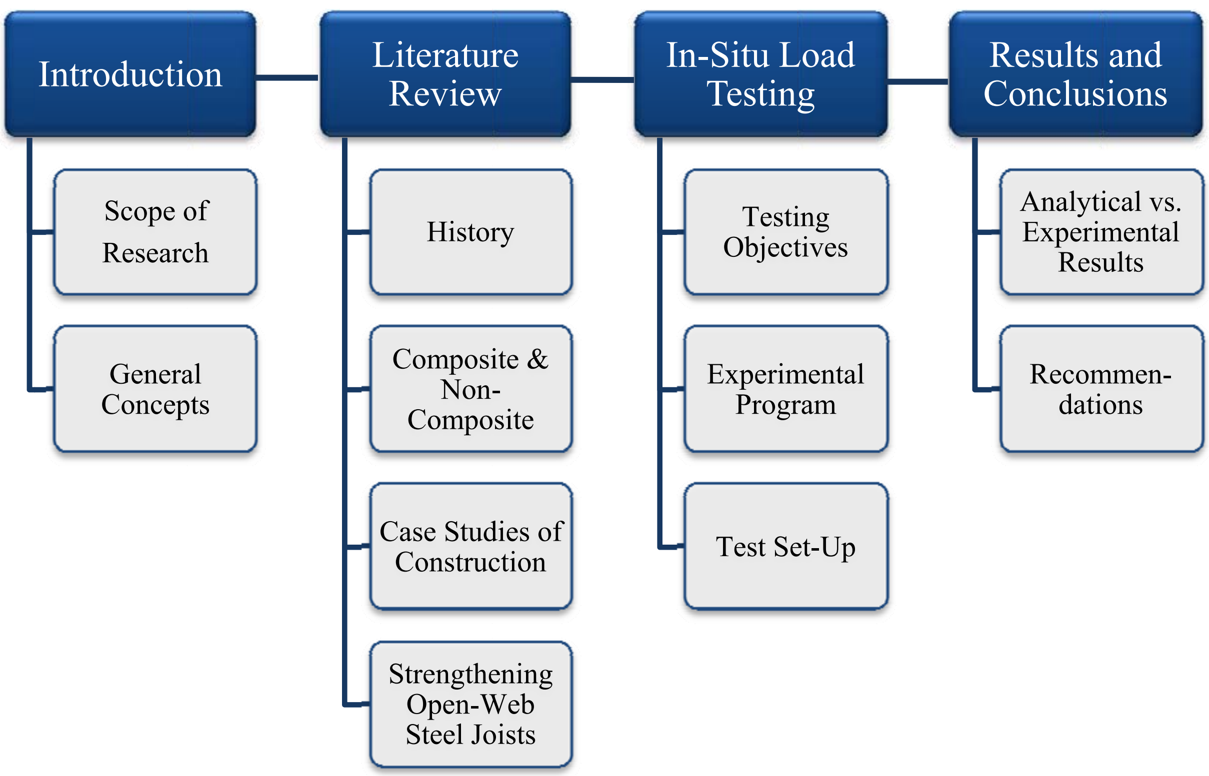


Figure 1 – Thesis Outline

1.2. Scope of Research

Open web steel joist systems have been recognized for a number of years as one of the most economical systems for constructing building floors. This study investigates the structural behavior of open web steel joist floor strips subjected to in-situ load testing.

1.3. Definitions

The following terms are presented with a brief description of each (SJI, 2005).

Chord: The top or bottom member of a joist, usually two angles separated by a gap between them (Figure 3).

Cold-formed: The process of forming a structural section by bending sheet or strip steel in roll-forming machines without the use of heat.

Hot-rolled: Structural steel sections which are formed by rolling mills from molten steel which can be angles, channels, W Shapes, S Shapes, etc.

Web: 1) The vertical or diagonal members joined at the top and bottom chords of a joist or joist girder to form triangular patterns or 2) The portion of a structural member between the flanges.

Shear Studs: Steel connectors that are welded to the top flange of a beam or top chord of a joist to achieve composite action with the concrete slab (Figure 2).

Metal Deck: A floor or roof covering made out of cold-formed metal attached by welds or mechanical fasteners to joists, beams, or other structural members that can be galvanized, painted, or unpainted.

Figure 2 – Steel Floor Deck Connected to the Joist (Havel, 2005)

Figure 3 – Steel Joist Connected to its Bearing on a Steel Girder (Havel, 2005)

CHAPTER II

LITERATURE REVIEW

2.1 INTRODUCTION

When performing a structural design of any type of structure or structural component, the designer uses an interactive process of applying engineering mechanics and engineering judgment to create a functional, economic and, most importantly, a safe structure for use by the public. By using structural analysis techniques and conforming to design specifications, the design engineer works to create a solution that is to everyone's benefit.

Through advances in theory, computational tools, and construction materials and techniques, structural design has evolved to its modern stage. Nowadays, economic factors help drive the development and the design of longer span floor systems, by taking advantage of more reliable and lighter materials and structural systems.

Figure 4 - Light-Weight Floor System (Hopleys, 2006)

Lightweight steel construction (Figure 4) has long been recognized as one of the most economical systems for constructing building floors. Three options have evolved over the years to meet the requirements for building height limitations and the need to accommodate complex heating, ventilating, electrical, and communication systems: (1) composite wide flange beams with web openings; (2) stub girders; and (3) open-web steel joists and joist girders.

Figure 5 – Steel Joist – Column Connection (Hopleys, 2006)

The term open web steel joist refers to an open-web, parallel chord, and load carrying member suitable for direct support of floors in buildings, utilizing hot-rolled or cold-formed steel (Figure 5), including cold-formed steel whose yield strength has been attained by cold working.

Open web steel joists are prefabricated lightweight flat trusses. They are typically spaced between 3ft and 7ft apart depending on the type of decking used. These are commonly used in conjunction with suspended ceiling systems. The two main types of open web steel joists are (Figure 6):

- **"S" type:** 8in to 24in deep; 4ft to 48ft spans; where the bottom chords consists of two rectangular section solid steel bars.

❖ **"L" type:** 18in to 48in deep; 25ft to 96ft spans; where the bottom chords consists of two "L" section angles.

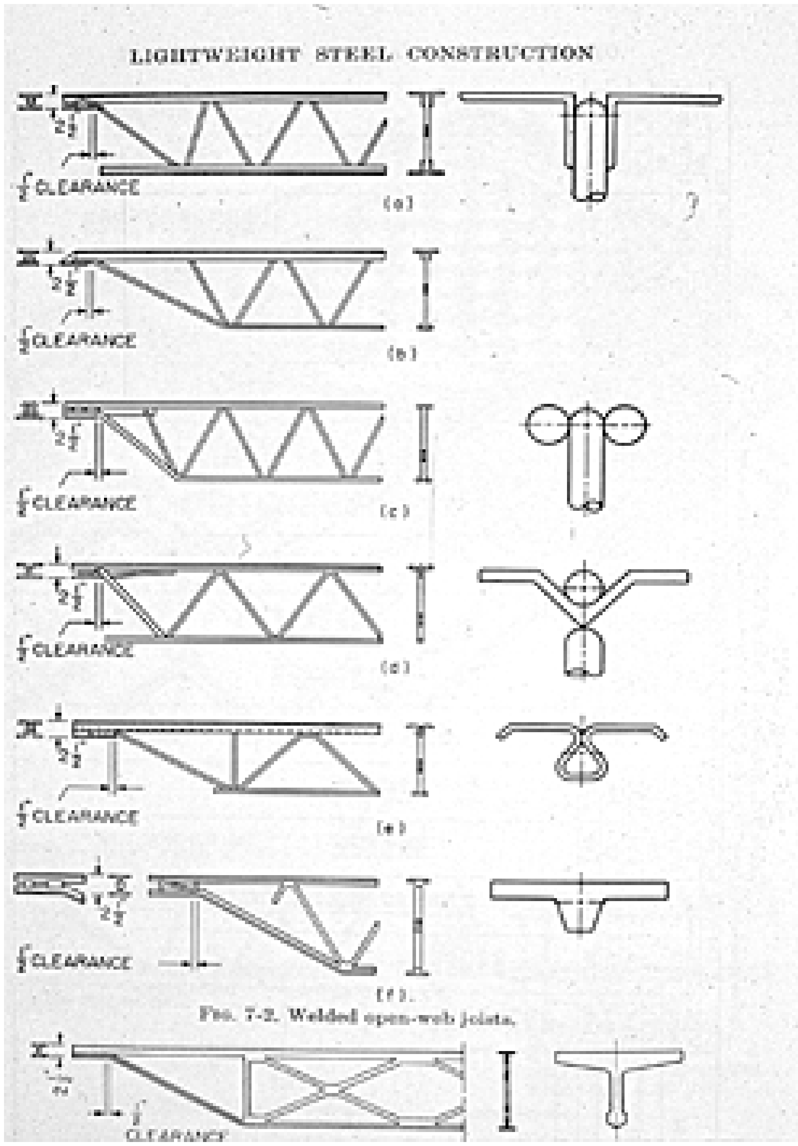


Figure 6 – Open Web Steel Joist Types (McGill, 1980)

Steel deck and open-web steel joist combination is a roof and floor framing system widely used in many types of buildings. The most efficient application is in warehouses, distribution centers and super stores, where the repetitive configuration of bay sizes takes most advantage of the nature of joists – mass production. In North America, this system is very popular in virtually all kinds of commercial, industrial, and institutional buildings. In the residential sector, this system finds its applications in condominiums. Today, most steel joist fabricators also produce steel decks, and steel joists and decks are often bid and shipped in a single package.

2.2 Scope of Review

This literature review will discuss, first of all, the history behind light-weight floor systems, second, the two major classifications of composite floor systems, which are:

a) Dry floor construction: Non-composite action
b) Composite steel joists with shear studs

Then, it will discuss the existing criteria used to evaluate dynamic and static response of light-weight floor systems and methods to predict the structural behavior of a floor system to certain distribution of concentrated loads.

The last part of this literature review will focus on both case studies of construction including details of the method of erections of the steel joists, and typical methods for the strengthening of open-web steel joists.

2.3 Background

2.3.1 Historical Review

Composite construction (Figure **7**) typically consists of a concrete slab placed upon a rolled steel beam or girder, and when interconnected with shear connectors form a composite section with composite action. These shear connectors are designed to resist the horizontal shear that develops during bending. Composite floor systems have been used in buildings for over thirty years (Salmon and Johnson 1996).

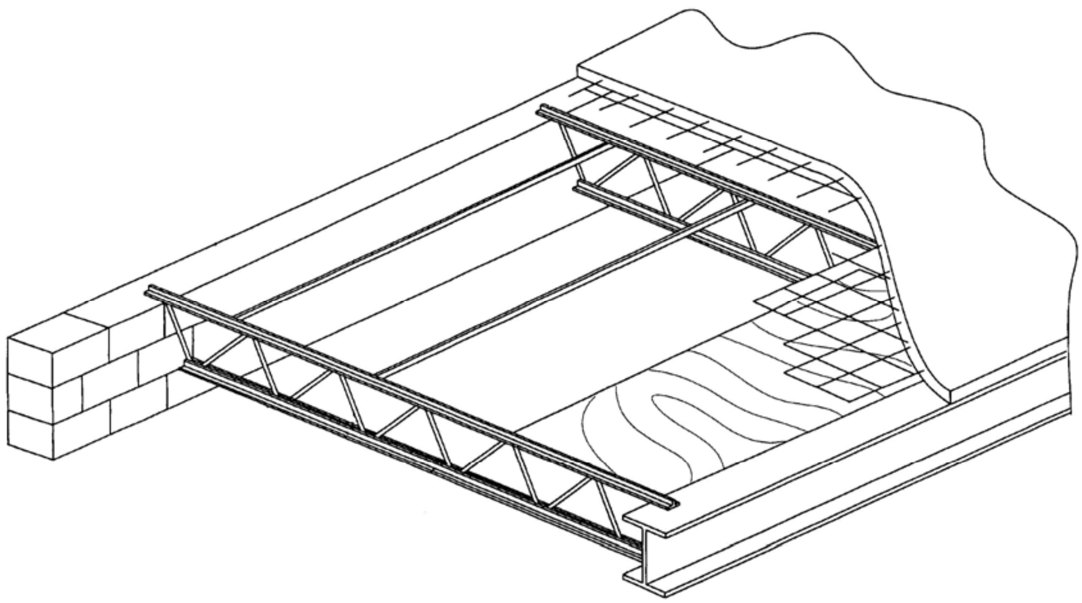


Figure 7 – Composite Construction (Newman, 1966)

Composite joists, which mean the use of a steel joist with a concrete slab on top, are relatively new in the field of structural engineering. In composite joist construction, open-web steel joists are used as an alternative to rolled steel sections. Joists are standardized parallel chord trusses economically fabricated using established techniques and standards. The chords are typically light double angle sections and the web members are usually round bars or small angles. The web openings in joists accommodate ductwork, electrical conduit and piping (Figure 8). Joists are also much lighter in weight than rolled steel sections.

The first open web steel joist, fabricated in 1923, was a Warren truss configuration. The top and bottom chords were round bars with web of the joist formed with a continuous bent bar.

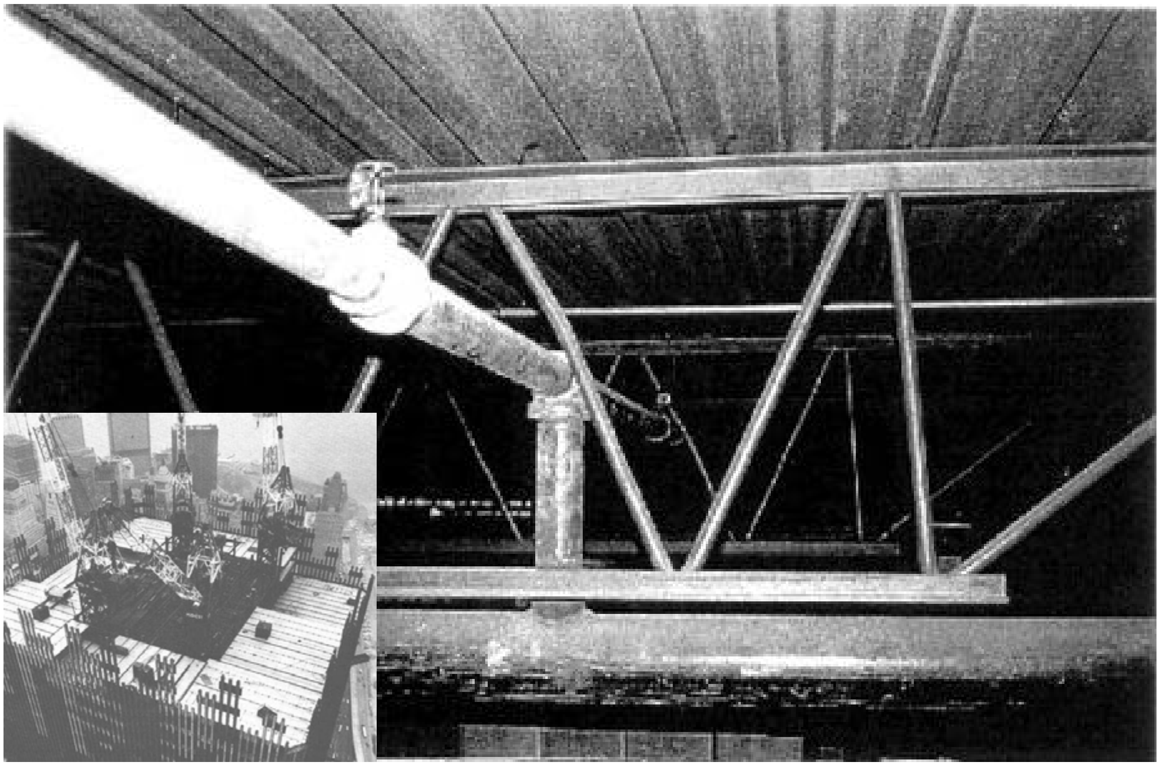


Figure 8 – Open-Web Steel Joist: WTC, New York, 1960

The first edition of Standard Specifications for Steel Joists was adopted by Steel Joist Institute (SJI) in 1928. The definition of steel joist in this edition was: “Any steel beam or truss shaped steel member suitable for supporting floors and roofs when used for floor filling between the main supporting girders, beams or walls, shall be known as a steel joist. Such joists may be made of rolled shapes or strips of sheet steel, round bars, angles or specially rolled bars riveted or welded together, or by expanding rolled shapes, or by any other method complying with the requirements of this article.” This edition covered two types of joists: “(a) Steel joists having solid webs shall be designed as beams;” and “(b) Steel joists built up of bars or other sections and those fabricated by expanding rolled sections shall be designed as trusses.”

In 1933, 1941 and 1946 revisions of the specifications, the definition of steel joists was changed as: “Any steel member suitable for supporting floors and roofs between the main supporting girders, trusses, beams or walls when used as hereinafter specified shall be known as a ‘steel joist’. Such steel joists may be made of hot or cold formed sections, strip or sheet steel, riveted or welded together, or by expanding.” Both beam type and truss type were covered by these revisions.

The 1949 revision kept the same definition of steel joist, but dropped the solid web, beam type joists from its scope and changed the name of the specifications as “Standard Specifications for Open Web Steel Joist Construction.” Since then, the term “steel joist” in SJI domain has been implicitly interpreted as Open-Web Steel Joists (OWSJ). OWSJ is also frequently referred to as bar joists.

The first load table for OWSJ was adopted by SJI in 1929. In this very first version of load tables, the joist span was limited to 32 feet, and depth to 16 inches. In 1952, the depth was expanded to 20 inches, and spans were extended to 40 feet.

The introduction of long-span steel joists in 1953 is a conceptual breakthrough to the traditional open web steel joists. The maximum span for this kind of light weight steel structure was expanded to 96 feet, with depths ranging from 18 inches up to 48 inches. Today, these long-span steel joists are designated as LH-series joists. In 1970, in response to the demand for longer spans and higher capacity, the deep long-span steel joists were introduced. The maximum span was expanded to 144 feet, with depths ranging from 52 inches to 72 inches. These deep long-span steel joists were designated as DLH-series, and this designation remains the same to date.

The introduction of joist girders in 1978 marked yet another milestone for steel joists. The SJI Standard Specifications for Joist Girders brought steel joists into the domain of primary structural members. Joist girders are designed to support secondary structural components like open web steel joists, long-span steel joists, etc. Joist girders can be designed either as simply supported members; or as members with continuity forming rigid frames.

In the mean time, the capacity of "traditional" open web joists kept growing. In 1959, the span was increased to 48 feet, and depth to 24 inches. In 1972, maximum span was expanded to 60 feet, with depth ranging from 8 inches to 30 inches. These joists are known today as K-series open web steel joists. In 1994, KCS joists were introduced as a part of the K-series Specification in response to the need for a joist with a constant moment and constant shear.

Today, steel joists are much more versatile and sophisticated than initially invented. The boundary between joists and structural trusses is no longer clear cut. The profiles of joists have been expanded from traditional parallel chords to various geometries, such as scissor, arched chords, bow string, and gable, etc. The maximum span has been expanded from the initial 32 feet to 144 feet in SJI load tables. Some joist manufacturers have further extended standard load tables. For example, Nucor Vulcraft's super long-span SLH-series joists have a maximum span 240 feet in the load table. Even longer spans are possible through careful engineering and fabrication.

2.4 DRY FLOOR CONSTRUCTION: NON-COMPOSITE ACTION

An early attempt to develop an innovative steel floor system, using open web steel joists to compete with flat plate concrete slab construction, was the Dry Floor System (Figure 9). A Dry Floor System was proposed to address problems in high-rise apartment construction including: structural borne sound, noise transmission, impact noise, interior partition cracking, seasonal limitations, labor-material balance, and economics. For a quantitative description on the performance of the Dry Floor System in these categories, refer to Newman (1966).

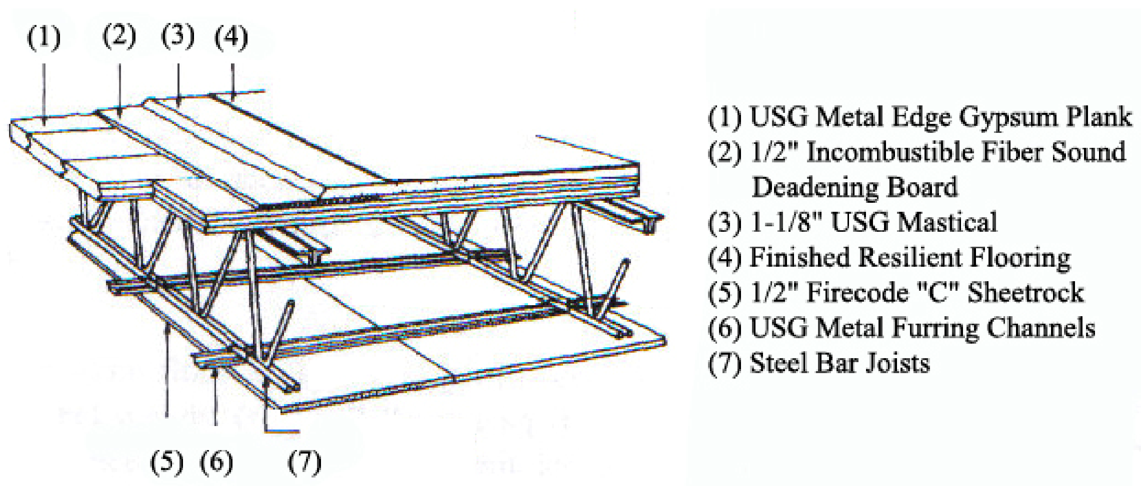


Figure 9 – Dry Floor System (Newman, 1966)

Full scale testing of a two-bay portion of floor area (between two columns lines), in a typical apartment building, was conducted at the U.S. Steel Applied Research Laboratory (Fang 1968). The gypsum planks used were precast units of 2-in. thick, 15-in. wide, and 10-ft. long. The edges of the planks were reinforced with 22 ga. galvanized steel tongue and groove edges to form mating joints (Figure 10). The results of the testing concluded that a very small magnitude of deflections resulted from the testing of the gypsum planks in place. Furthermore, it was apparent that the gypsum deck provided nearly all of the resistance to horizontal movement with very little shear contributed by stiffness of the frame.

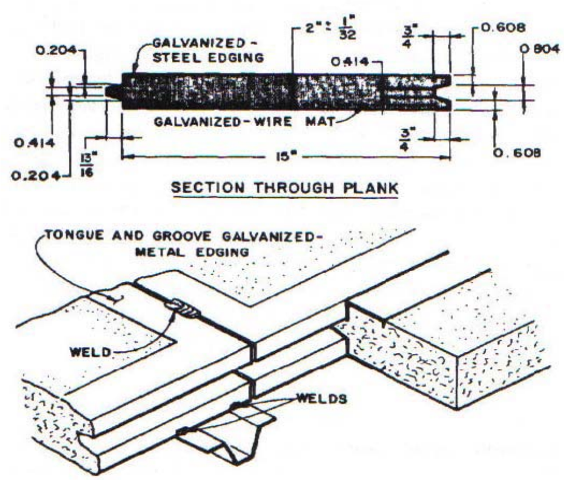


Figure 10 – Gypsum – Plank Details (Fang, 1966)

2.5 COMPOSITE STEEL JOISTS WITH SHEAR STUDS

With an increase in the availability of steel decking, concrete slabs, supported by ribbed steel decks bearing on the joists, became the mainstream open web steel joist floor system. In an attempt to further minimize floor-to-floor heights, the concept of a composite steel joist system has been introduced by the joist industry (Figure 11).

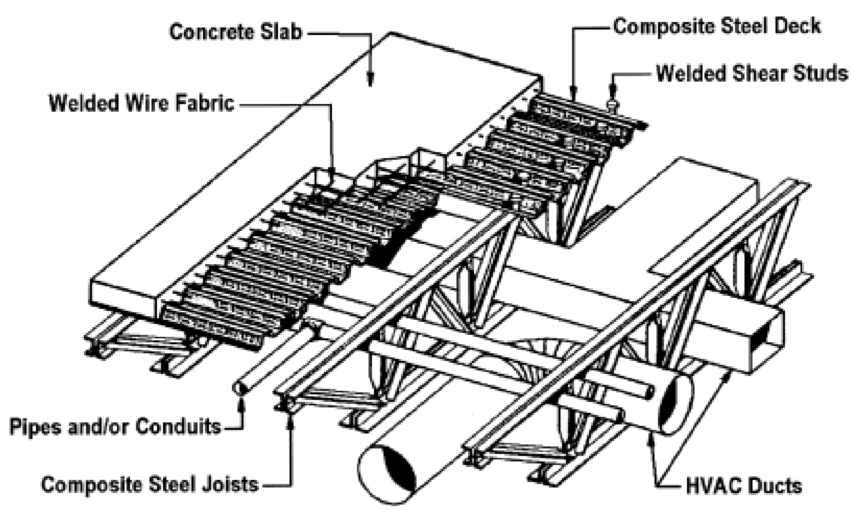


Figure 11 – Composite Steel Joist System (Samuelson 2002)

The term “composite” implies that the joist top chord and the overlaying concrete slab will act as an integral unit once the concrete has cured. The main components of the system are the steel joist, the metal deck, and the concrete slab (encasing welded wire fabric). Welded shear studs or specially designed truss top chords must be provided to ensure adequate transfer of shear; this allows the concrete slab to act as a compression flange.

2.6 CURRENT ACCEPTABILITY CRITERIA

2.6.1 OHLSSON’S CRITERION

Ohlsson performed more than ten years of research on light-weight floor systems and developed a design guide for the Swedish Council for Building Research. Although he performed most of his research on timber floor systems, the design criterion is presented for use with any construction material and structural configuration. The design procedure is broken down into three distinct parts.

The first part is a static load test. Floor systems must undergo no more than 1.5mm (0.059in) deflection when subjected to a 1kN (225lb) concentrated load located at the mid-span of the floor system. Deflection of a simply supported joist is then given by (Equation 1):

$$\Delta=\frac{PL^3}{48E_jI_j} \quad (m) \tag{1}$$

where,

Δ	=	Vertical Deflection at the Mid-span of the Floor Joist
P	=	Concentrated Point Load at Mid-span (N)
L	=	Span of the Floor Joist (m)

E_j = Modulus of Elasticity of Joist (Pa)

I_j = Moment of Inertia of Joist (m^4)

Floor systems that fall within the "better" range must go through a third and final design phase. If the floor system contains unobstructed room lengths less than 6–7m (20–23ft) or has a span length greater than 4m (13ft), then the response to a continuous loading must be performed.

2.6.2 AUSTRALIAN CRITERION

The Australian Standard Domestic Metal Framing Code (1993) analyzes the serviceability of cold-formed steel joist floor systems. Since this criterion adopted much of Ohlsson's guidelines, the two criteria are very similar. For a floor system to be considered acceptable, it must first have a fundamental frequency above 8Hz. There are only two other conditions that must be satisfied for a floor to be considered dynamically acceptable.

The first is a static deflection test which requires that the floor deflect no more than 2mm (0.0787in) for a 1kN (225lb) concentrated load placed anywhere on the floor. Equation 2 is used to obtain the mid-span deflection of the floor system. The single joist deflection factor, k_d, is used to account for more than one joist supporting the 1kN concentrated load:

$$\Delta = \frac{k_d p l^3}{48 e_j i_j} \quad \text{(m)} \tag{2}$$

where,

$K_d = 0.833 - 0.34 \log_{10}\left[\left(\frac{K_c}{K_b}\right) + 0.44\right]$ or from Figure 12

$$K_c = \frac{E_f\, t_f^3\, L}{12\, s^3} \qquad \text{(N/m)} \qquad \text{(for Joist Only Systems)}$$

$$K_b = \frac{E_i I_j}{L^3} \qquad \text{(N/m)}$$

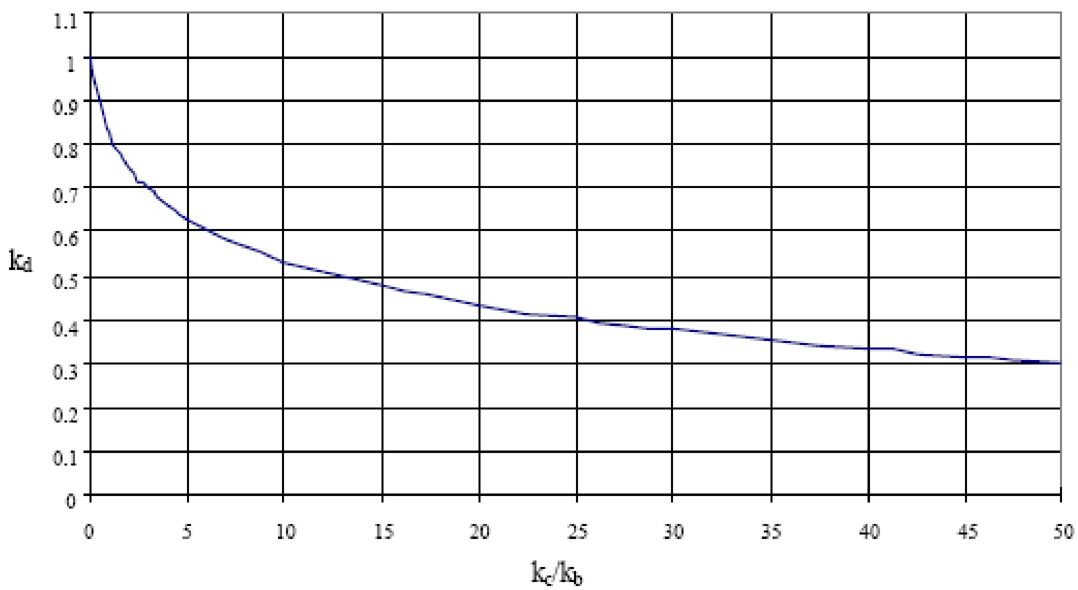


Figure 12 – Deflection Factor, K_d

The second requirement is based on the application of a unit impulse load of 1.0N-s (0.225lb-s) located anywhere on the floor system.

Although the specification states that the static load test will generally govern for short spans and impact velocity will control for long spans, it does not distinguish between the two span lengths.

2.6.3 Canadian Criterion

Onysko (1985) proposed a criterion based on the results of an extensive survey and testing program that included the assessment of 646 wood floors of different types in the 1970's. The occupants were provided with a well planned questionnaire about their perception of the behavior of the floors in their home and then vibration measurements were taken of each floor.

It was found that the best correlation to perceived acceptability was dynamic response due to an impact load and floor deflection due to a static load test. He initially planned to develop a design criterion based on the dynamic response of an impulse load on the floor systems. Since this required the knowledge of the damping values which are dependent on the use of the floor system and are usually unknown, he chose to use the deflection due to a static load test in his design procedure.

A criterion developed from his research was adopted into the 1990 National Building Code of Canada as a reference for the span tables of solid sawn wood floors (Onysko 1995). Onysko has since refined his criterion and the relationship is now (Equation 3):

$$y \leq \frac{8}{L^{1.3}} \quad \text{(mm)} \tag{3}$$

where,

y	=	Central Deflection of the Floor (mm)
L	=	Span Length of Floor (m)

The central deflection is due to a static load concentrated load of 1kN (225lb) placed at mid-span. In addition to this, floors with a span length less than 3.0m (9.8ft) must not deflect more than 2mm (0.0787in) under a 1kN static load placed at mid-span.

2.6.4 Murray's Criterion

Murray's design criterion was developed from the results of 91 steel joists or steel beam concrete slab floor systems (Murray 1979). He compared the subjective reaction of these floor systems to four previous design guidelines used in practice and found their results to be inconsistent and inaccurate with respect to the test floors. Murray developed another design procedure that relates damping, frequency, and peak displacement due to a heel drop impact. Plotting the product of frequency and measured amplitude of each floor system versus damping.

The initial dynamic displacement of a floor system, Δ_{static}, can be found by calculating the static displacement of the girder or beam due to a 600 lb. concentrated force (initial value of the decreasing ramp function) applied at the center. Since forces applied dynamically to a floor system can have a displacement of almost twice the static displacement of the same force, the static deflection calculated must be multiplied by a dynamic load factor (DLF).

Since the centerline static deflection will be carried by more than one joist in a floor system, the number of joists effectively carrying the load of the floor system must be determined. This will be fully discussed in the next section of this report and it is assumed that the number of effective joists, N_{eff}, is known. Knowing the above quantities, the actual displacement due to a heel drop can be calculated using Equation 4:

$$A_0 = \frac{A_{0t}}{N_{eff}} \quad (4)$$

where,

A_{ot} = Single Tee-beam Initial Amplitude Due to a Heel Drop Impact

N_{eff} = Number of Effective Joists

If the floor joists are supported by a beam or joist-girder, the above criterion must be checked for the combined results of both, the joist floor system beam and/or girder as well as the individual joist floor system.

The system deflection due to a heel drop is calculated using equation 5 (Murray 1991):

$$A_0 = A_{0b} + \frac{A_{0g}}{2} \tag{5}$$

where,

A_o	=	System Displacement Due to a Heel Drop
A_{ob}	=	Beam or Joist Displacement Due to a Heel Drop
A_{og}	=	Girder or Joist Girder Displacement Due to a Heel Drop

2.6.5 Johnson's Criterion

Johnson (1994) originally applied Murray's acceptability criterion to wood floor systems. After comparing Murray's criterion to 86 *in-situ* floors under construction, he decided to abandon this approach and proposed a criterion of his own for wood floor systems. He proposes that if the fundamental frequency of the floor system is above 15Hz under normal construction loading, then it is acceptable.

Johnson concluded from the results of numerous static deflection tests that the effective sheeting width was found to be negligible. Thus the moment of inertia of a single joist can be used instead of using the transformed moment of inertia. Shue (1995) tested an additional 78-floor system under the same conditions as Johnson and found Johnson's criterion to be acceptable for unoccupied floor systems with regard to vibration produced by human footfalls.

2.7 Number of Effective Joists

As stated above under Murray's criterion, a force applied to a floor system will distribute itself over a certain number of joists in the floor system. Lenzen and Dorsett (1969) denoted this as the effective floor width. Effective floor width is twice the measured distance perpendicular to the floor joist, x_o, from the centerline of the floor system to where the deflection due to a concentrated load at the centerline reaches zero. This is shown in Figure 13. They proposed a mathematical equation to find x_o (Equation 6):

$$x_0=1.06\ \varepsilon\ L \quad (ft) \tag{6}$$

where,

X_o	=	Distance from the Center of the Floor to the Point of Zero Deflection (in)
ε	=	$(D_x/D_y)^{0.25}$
L	=	Span Length of Floor Joists (in)

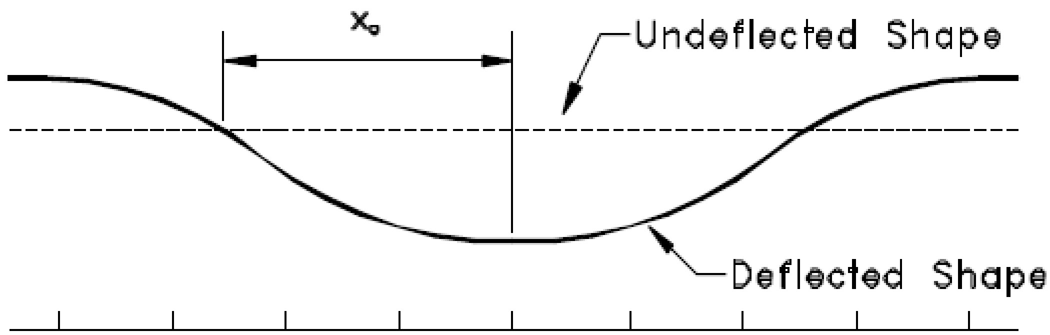


Figure 13 – Deflection Profile across Center Of Floor (Lenzen 1969)

After comparing the effective floor width to actual floor systems, Lenzen and Dorsett (1969) found the prediction equation was close for joists spaced at 24in but not for joists with a spacing of 48in. They then decided to express the effective floor width in terms of the joists and their spacing. The number of joists that will resist a concentrated load is known as the effective number of joists in a floor system. The joist directly under the concentrated load is considered to be fully effective with the joists on either side contributing less to the resistance of the load. The effectiveness of a joist is a ratio of the deflection of the joist under consideration to the deflection of the joist acting individually under a given loading. The following sections discuss three procedures used to determine the number of effective joists in a floor system.

2.7.1 SJI and AISC Equations

Lenzen and Dorsett (1969) approximated the two points of zero deflection using a sine curve. The joist locations are then superimposed on the approximate deflected shape of the sine curve and their contributing values are directly related to the location of the joist. The number of effective joists is then the summation of the effective contribution of the floor joists between the zero points of deflection and is approximated by Equation 7:

$$N_{Eff}=1+2\sum\left(\cos\frac{\pi\, x}{2\, x}\right) \qquad \text{for } x \leq x_0 \tag{7}$$

where,

x	=	Distance from the Center Joist to the Joist under Consideration (in)
x_o	=	Distance from the Center Joist to the Edge of the Effective Floor
	=	$1.06\ \varepsilon\ L$ (in)
ε	=	$(D_x/D_y)\ 0.25$
D_x	=	Flexural Stiffness Perpendicular to the Joists
	=	$\frac{E_f\, t_f^3}{12}$ (lb-in)
D_y	=	Flexural Stiffness Parallel to the Joists
	=	$\frac{E_j\, I_j}{S}$ (lb-in)

E_f = Modulus of Elasticity of Flooring (in^4)

E_j = Modulus of Elasticity of Joist (in^4)

t_f = Thickness of Flooring (in)

I_j = Transformed Moment of Inertia (in^4)

s = Joist Spacing (in)

Since the Steel Joist Institute (SJI) funded this research, Equation 7 is known as the SJI equation for N_{eff}. The SJI equation is intended to be used with floors with joist spacing up to 30 in. Saksena and Murray (1972) designed 50 steel beam-concrete floors based on typical office design loads, spans, spacing, and slab thickness. They used a computer program developed by Ohmart (1968) to determine the dynamic amplitude due to a heel drop for each floor system. Having the predicted amplitude from Ohmart's program and hand calculations for the predicted frequency of the Tee-Beams, Saksena and Murray (1972) computed the N_{eff} of each floor system by the following relation (Equation 8):

$$N_{Eff} = \frac{A_{0t}}{A} \tag{8}$$

where,

N_{eff} = Number of Effective Joists

A_{ot} = Maximum Dynamic Amplitude of a Tee-Beam Subjected to a Heel Drop Impact (in)

A_o = Maximum Dynamic Amplitude of the Floor System Due to a Heel Drop Impact (in)

A multiple linear regression analysis was performed on the values obtained to arrive at the following formula (Equation 9):

$$N_{eff}=2.967-0.05776\left(\frac{S}{d_e}\right)+2556\times10^{-8}\left(\frac{L^4}{I_t}\right)+0.0001\left(\frac{L}{S}\right)^3 \quad (9)$$

where,

d_e	=	Effective Slab Depth (in)
S	=	Beam Spacing (in)
L	=	Beam Span (in)
I_t	=	Transformed Moment of Inertia of the Tee-Beam (in^4)

Equation 9 has the following limitations (Equation 10):

$$15\leq\left(\frac{S}{d_e}\right)\leq40 \quad \text{and} \quad 1\times10^6\leq\left(\frac{L^4}{I_t}\right)\leq50\times10^6 \quad (10)$$

This research was sponsored by the American Institute for Steel Construction (AISC) and will be called the AISC equation for N_{eff}. This equation is to be used for floor systems with beam spacing greater than 30in, although it was developed for floors with beam spacing between 60in to 180in.

2.7.2 KITTERMAN'S EQUATION

Shamblin (1989) found that the SJI and the AISC equations do not converge at the 30in spacing. She also noted that the AISC equation did not consider open web joist floor systems. She developed an equation to be used at any spacing with either beam or open web joist floor systems with a concrete slab. A finite element analysis was performed on 240 floor systems designed using traditional strength criteria. ABAQUS (Hibbitt, et al. 1984) was used to determine the dynamic response to a heel drop impact. After Neff was calculated, the same procedure as by Saksena and Murray (1972), a multiple linear regression analysis was performed to generate her proposed equation.

Kitterman (1994) planned to verify Shamblin's proposed equation by comparing her results with the SJI and AISC equations. He found that Shamblin's equation predicted larger frequencies than the previous equations. Kitterman then tested the 240 floors Shamblin used with another finite element program, SAP90 (Wilson and Habibullah 1992). The number of effective tee-beams for each floor system was found by dividing the amplitude of a single tee-beam by the predicted floor amplitude. He then performed a multiple linear regression analysis to obtain (Equation 11):

$$N_{eff}=0.4898+34.19\frac{d_e}{S}+8.99\times10^{-9}\frac{L^4}{I_t}-0.000593\left(\frac{L}{S}\right)^2 \qquad (11)$$

with the limits of (Equation 12),

$$0.018\leq\left(\frac{d_e}{S}\right)\leq0.208 \quad \text{and} \quad 4.5\times10^6\leq\left(\frac{L^4}{I_t}\right)\leq257\times10^6 \text{and} \quad 2\leq\frac{L}{S}\leq30 \qquad (12)$$

After plotting the values predicted by the other equations and the values from his equation, he concluded that his equation calculated a smaller N_{eff} than the others for floor systems with a spacing less than 60in. This means that Kitterman's equation would predict larger system amplitude than the other equations for floors with spacing of less than 60in. He also states that there is little change in the system amplitude for floor systems with a large spacing. Kitterman recommends his proposed equation be used in conjunction with the Murray criterion to evaluate both existing and designed floor systems.

2.8 CASE STUDIES OF CONSTRUCTION

2.8.1 HOPLEYS OPEN-WEB STEEL JOISTS

Made from galvanized steel they won't rust, shrink, warp or twist and come complete with pre-punched, fully engineered brackets to make installation easy from the installation professional to the home handyman.

Hopleys steel joists are manufactured from light-weight galvanized steel (Figure 14) for durability and require little or no maintenance. Their obvious advantages over timber products are stability without warping or shrinking, lack of costly off cuts and ease of handling during construction (Figure 15). These characteristics afford the ease of modular design for enhanced strength plus a pleasing appearance when exposed.

Figure 14 – Hopleys Industrial and Commercial Flooring (Hopleys, 2006)

Figure 15 – Hopleys Domestic Flooring (Hopleys, 2006)

2.8.2 REAGAN HIGH SCHOOL, HOUSTON, TEXAS

This challenging project will upgrade the existing high school facility by retaining the distinctive existing main building, demolishing the additions and replacing them with a new classroom building, gymnasium building, parking garage, and cafeteria building. The classroom building and the gymnasium will be 2-story steel-framed buildings using open-web steel joists and a concrete deck for the floor system (Figure 16), and joists and beams with a metal deck for the roof framing; whereas the cafeteria building will be a tall single story steel-framed structure also framed with open-web joists and steel beams and columns. The 2-story parking garage will be a specially designed and detailed precast concrete framed structure with recessed connections as to enable them to be grouted and free of maintenance.

Figure 16 – Reagan High School, Houston, Texas

2.9 CASE STUDIES OF ASSESSMENT

2.9.1 UNIFORM-LOAD TESTING OF OPEN WEB STEEL JOISTS

A technique involving the use of an inflatable rubber bag to apply uniform load to a single open web steel joist, which could also be used in the testing for any similar two-dimension structural member, is described and compared to the method of applying a dead load of dead weigh.

The tests were conducted utilizing the test floor of the structures laboratory at The University of Kansas. The initial tests were conducted by connecting two 24-ft joists 24in apart by welding steel angles at six locations on the top chord. The ends of the joists were supported by steel bearing plates and Baldwin load cells. Wire ropes with turnbuckles inserted for adjustments were attached to these angles and to upright rectangular frames rigidly mounted on the test floor on either side of the joist pair to provide lateral support.

A wooden load deck was placed on the joists, and concrete cylinders of known weight were then stacked on the load deck in increments until failure. Figure 17 shows the dead-load test fixtures.

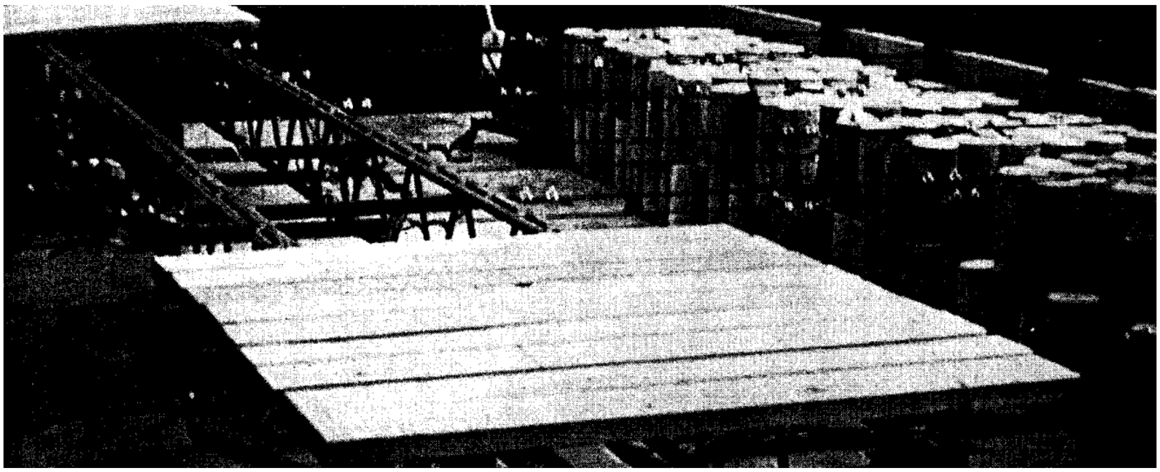


Figure 17 – Dead-Load Test Fixture (Lenzen, 1968)

The principal load method or “air-bag” technique consisted of the following procedure. A 12-in channel was placed flange down and centered between two rows of access holes. Rectangular frames constructed of angle sections were bolted to the channel every three feet. Steel outriggers were then fastened to the top of the frames and to the test floor in an A-frame configuration, providing an extremely rigid lateral support system for the test joist.

To prepare a test, the joist was placed though the rectangular frames and centered both longitudinally and laterally (Figure 18). Rollers were clamped to the joist at each frame on the top chord and at alternate frames on the bottom chord to provide the necessary lateral support.

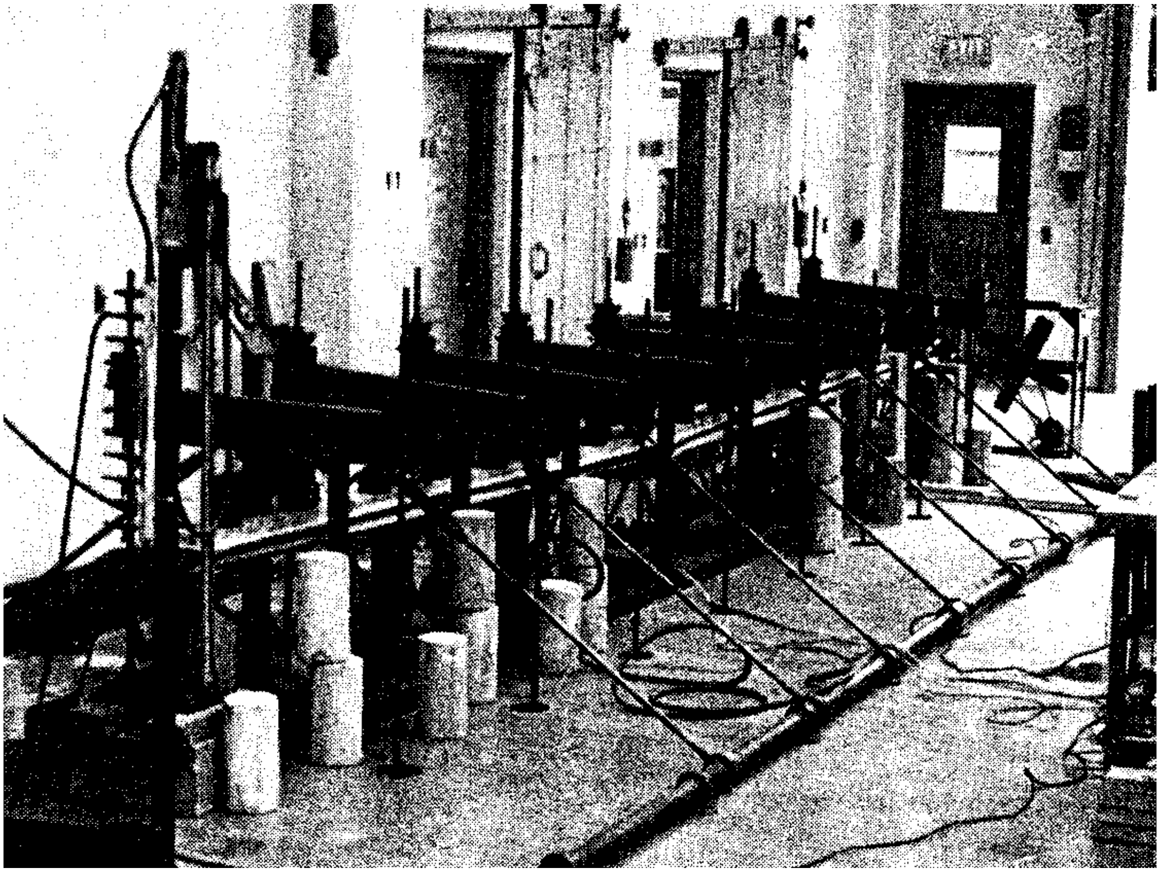


Figure 18 – "Air-Bag" Test Fixture (Lenzen, 1968)

Load-strain curves were plotted using the experimentally measured strains and incremented loads in the linear region of these curves were used to calculate the actual loads, the moment about the longitudinal axis of the joist, M_x, and the lateral moment, M_y, at each location to provide a comparison of the two loading methods.

The joists tested had oversized bottom chords to insure failure (buckling) of the top chord. Failure stresses were calculated using the axial load at mid-span produced by the total uniform load applied at failure. In addition, a computer analysis, which included the effects of joint eccentricity and local and total deformation, was used to obtain the theoretical axial load and moment about the x-axis at each location.

The test results indicated clearly that the two methods of uniform load are equivalent. The joist failure stresses and the axial loads, which provide the most obvious means of comparing the two methods, agreed quite closely. The moments about the x-axis, M_x, though reflecting a rather larger percentage of variation, generally were of the same magnitude for each location on corresponding joists and thus support the conclusion that the loading methods are equivalent.

However, with the air-bag method, it is possible to provide the degree of lateral rigidity necessary to duplicate actual conditions. Conversely, since room had to be allowed for loading and unloading the load deck during the dead-load tests, it was not feasible to provide the necessary lateral rigidity.

The air-bag technique with its advantages of (1) enabling a single joist to be tested, and (2) the ease and efficiency with which joists may be tested, had proved superior to the dead-load method and makes it feasible to test with an uniform load which more closely corresponds to actual use than does concentrated-load testing.

2.9.2 The Strength of Light Steel Joists

In order to determine the strength, stiffness, and behavior of two general types of joists, solid web joists of I-beam section, and open web steel joists; joists of both types were tested at the University of Wisconsin (Figure 19). Messrs. C.J. Held, L.A. Johnson, and H.H. Knuth tested 16 joists under uniform loading during the 1930-1931 term as a thesis project.

These tests were divided conveniently into three groups as follows:

I. Sixteen tests on individual joists under uniform loading (Figure 20),

II. Sixteen tests on individual joists under third-point loading, in which the loads were applied vertically and with 2 or 3 degrees of obliquity,

III. Sixteen tests on short lengths of solid web type joists; eight to determine their behavior in bearing under concentrated loads; and eight to study column action of the webs.

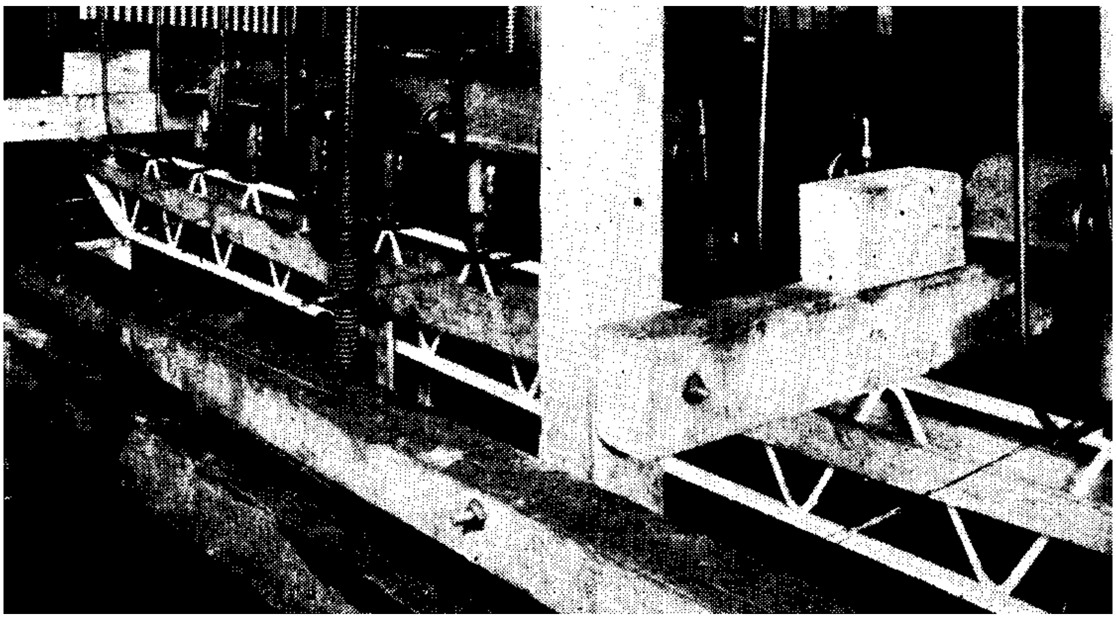


Figure 19 – 12-inch Truscon O.T. Joist (Held, 1930)

The device for producing approximately uniform loading on the joists consisted of a series of 1½ –ton hydraulic automobile jacks. These jacks were used in an inverted position. All jacks were connected by a single pipe line to an oil pump which was equipped with a pressure gage and a needle valve for precise regulation of oil pressure.

Lateral restraints of the joists was provided by horizontal steel wires, equipped with turnbuckles, which extended from each side of the top and bottom chords of the joists to a heavy timber frame fastened to the floor. The joists were supported by cast iron rocker supports, one of which rested on, an Olsen 50,000-pound reaction scale.

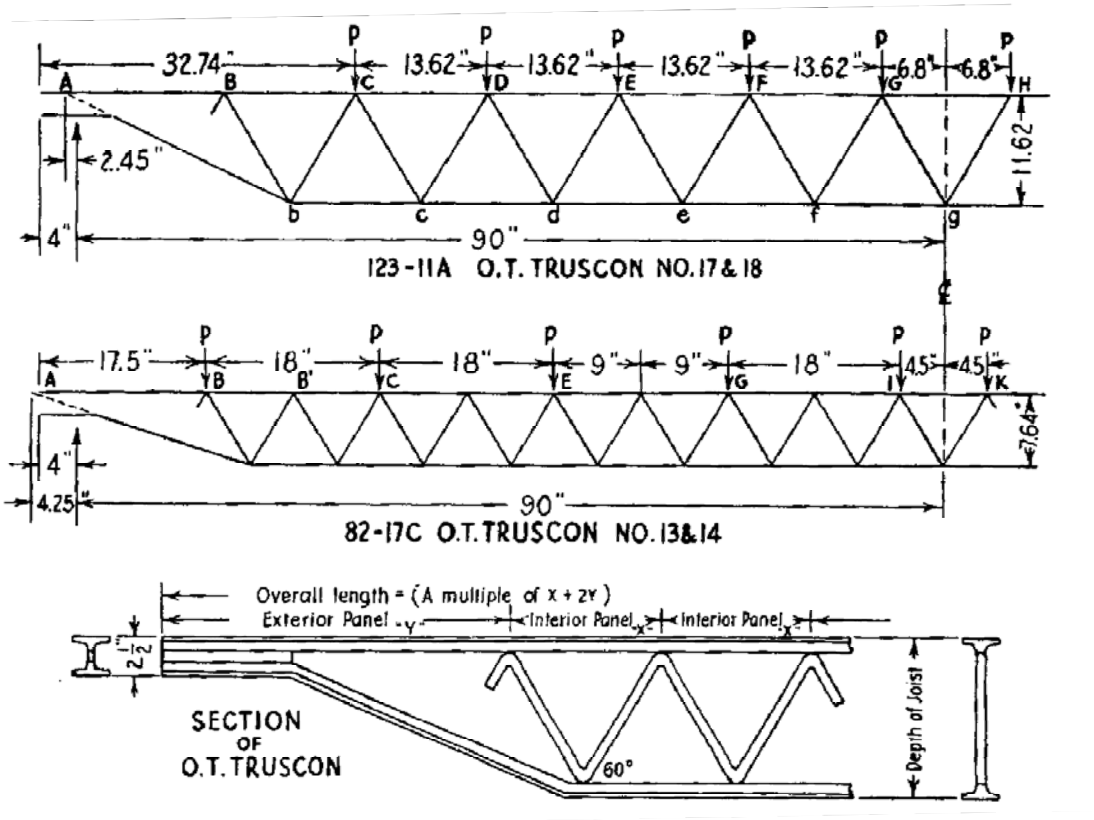


Figure 20 – Details of Joists (Held, 1930)

Each joist was placed in the machine so that the top chord was leveled. The span was adjusted to 15 feet. The top or bottom chords or both were stressed beyond the yielding point of the material before final failure occurred due to collapse of web members, vertical buckling of the top chords or lateral bowing of the compression flanges.

After a series of tests for different configurations it was concluded that:

I. The factor of safety of every open web joist based on the manufacturer's allowable load and tested with adequate bracing of the top flange but without a floor slab was under the specified value of 2.25.

II. All 8-inch open web joists failed to meet maximum ratio of deflection to length specification of 1/360 at allowable load, while all 12-inch open web joists and all solid web joists easily met the requirement.

III. The stress analysis of open web joists at loads up to allowable load can be made with sufficient accuracy by assuming that the joists are pin-connected trusses. Where such trusses have two diagonals per panel, the method of half shears gives reasonably good results.

IV. Small eccentricities of the end reactions with respect to the end panel points of open web joists cause serious bending of the end top chords and produce extremely high stresses, especially when the reaction falls outside the intersection of top and bottom chord axes. More attention must be given to the design of these joists to insure that the reactions will fall at or very slightly inside the end panel points.

V. In general for all classes of joists studied there is a clearly marked load at which both set and deflection curves for the same joist show a break or rapid change of curvature. It was proposed that this load s called Useful Limit Load, since it marks the maximum load beyond which set and deflection are likely to be excessive and objectionable.

VI. As series of tests of panels built with representative open web joist supporting a rigid slab would furnish much desirable information for proper design.

2.10 Strengthening Open-Web Steel Joists

Open-web steel joists are often the best value on the basis of cost per square foot, scheduling, useable space, and design flexibility; therefore, they are used extensively in floors and roofs in many types of structures throughout the United States and other countries. As a consequence, strengthening of open-web steel joists is often required due to the addition of any kind of extra loading such as rooftop units, snow drifting, and conveyors; or the need of lengthening or shortening them.

There are three basic methods of strengthening a steel joist for additional loading:

1. Load redistribution
2. Adding new joists or beams
3. Reinforcing existing joists

The first step in determining if a steel joist requires strengthening is to determine the existing joist capacity. The Steel Joist Institute has tables and specifications available that can be used to determine the existing joist capacity.

Whether information is obtained from records or from a field investigation, helpful and required information include the following (Fisher, 2004):

- ❖ Joist manufacturer
- ❖ Year of construction
- ❖ Structural documents
- ❖ Joist identification tag
- ❖ Joist configuration
- ❖ Loading on joist
- ❖ Joist span
- ❖ Joist spacing
- ❖ Bridging connections

❖ Actual condition of joist

2.10.1 LOAD REDISTRIBUTION

Load redistribution is a method of distributing concentrated loads to several joists in a floor or roof system (Figure 21).

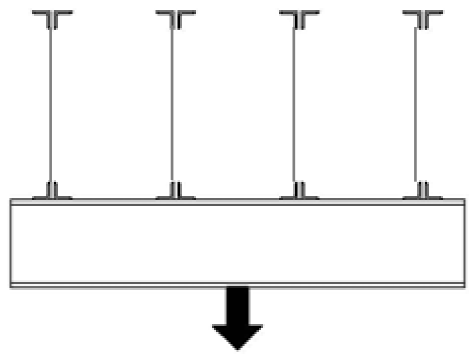


Figure 21 – Load Redistribution (Fisher, 2004)

2.10.2 ADDING NEW JOISTS OR BEAMS

Adding new joists or beams is a very good option when there are limited interferences such as piping, duct work, electrical conduits. Consideration needs to be given to camber and lateral stability of the top chord when a new joist is added. To insert a new joist or beam in place, "no camber" should be specified for the added element.

2.10.3 REINFORCING EXISTING JOISTS

When load redistribution or adding new joists of beams is not an option, strengthening the joist can be the solution. However, if the preliminary analysis shows that the top and bottom chords and the web are overstressed, it is often difficult to strengthen or reinforce the joists.

2.10.3.1 Chord Reinforcement

There are several different options available to reinforce the chord members. Shown in Figure 22 through Figure 26 are a number of different details that have been used successfully to reinforce top chords of joists.

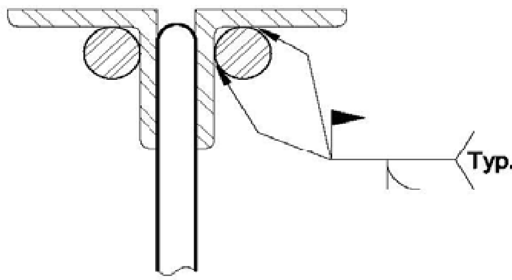


Figure 22 – Top Chord Reinforcement (Rods) (Fisher, 2004)

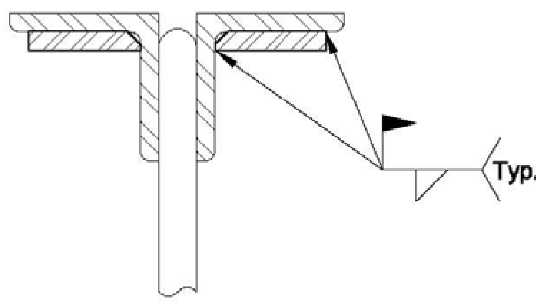


Figure 23 – Top Chord Reinforcement (Plates) (Fisher, 2004)

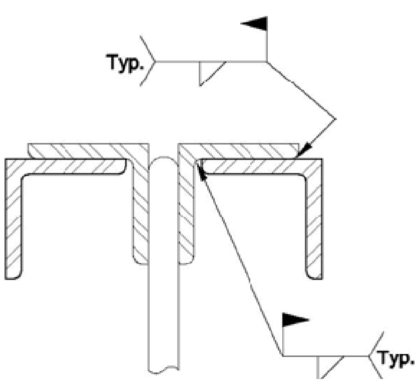


Figure 24 – Top Chord Reinforcement (Angles) (Fisher, 2004)

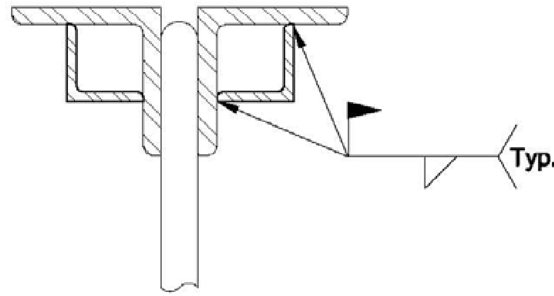


Figure 25 – Top Chord Reinforcement (Fisher, 2004)

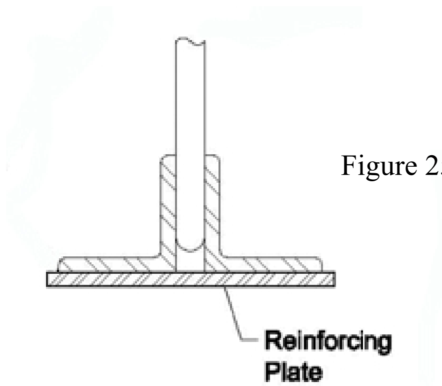


Figure 26 – Bottom Chord Reinforcement (Fisher, 2004)

2.10.3.2 WEB REINFORCEMENT

There are basically three types of webs used in most joists (Figure 27 through Figure 32):

- ❖ Rod web members placed between the chord angles
- ❖ Crimped web members placed between the chord angles that have the ends crimped to fit the 1-inch gap between the chord angles
- ❖ Double angle web members that are attached to the vertical legs of the chord angles

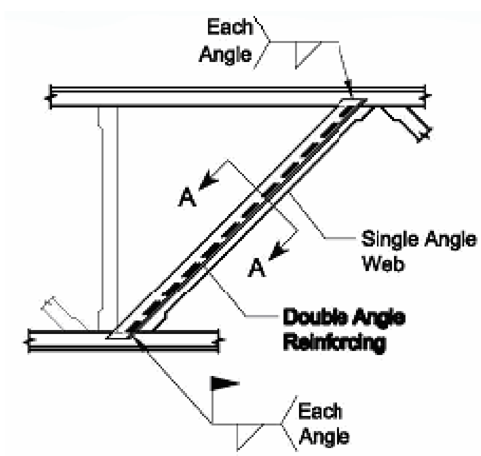


Figure 27 – Crimped Web Reinforcement (Fisher, 2004)

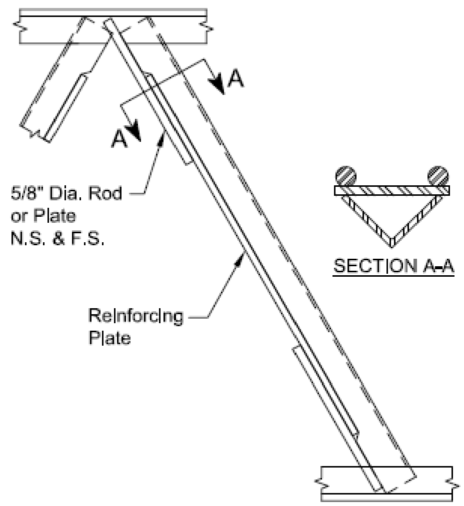


Figure 28 – Crimped Web Reinforcement (Fisher, 2004)

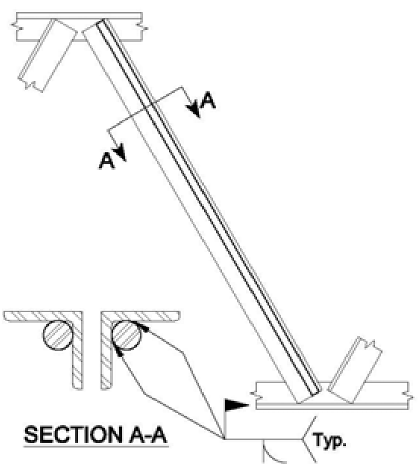


Figure 29 – Rod Reinforcing (Fisher, 2004)

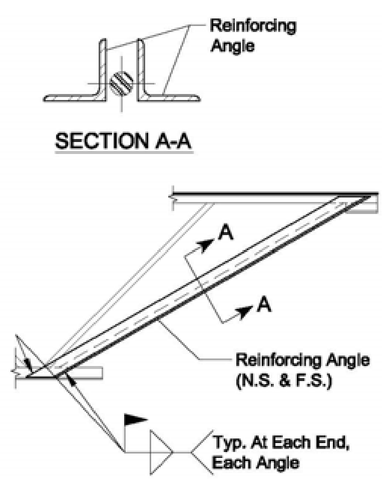


Figure 30 – End Diagonal Reinforcing (Fisher, 2004)

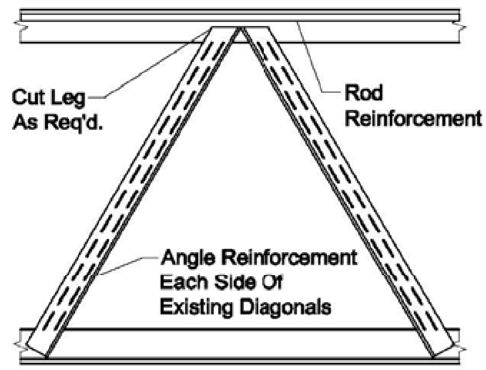


Figure 31 – Angle Reinforcement on Rod Web Joist (Fisher, 2004)

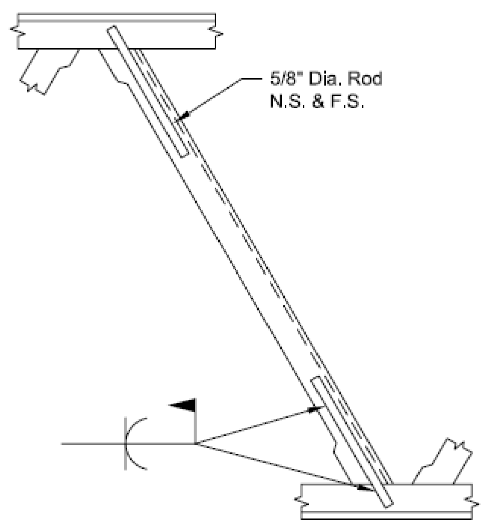


Figure 32 – Weld-Only Reinforcement (Fisher, 2004)

CHAPTER III

EXPERIMENTAL PROGRAM

3.1 INTRODUCTION

In January 2007 we knew that the old School of Nursing building was about to be demolished to make room for the new faculty townhouses. Therefore, with the idea of performing in-situ load tests in that building, we contacted Real Estate & Facilities Office of the University of Miami to know about the details. After the first visit to the building we started working on the preparations, which included instrumentation, cabling, data acquisition system, setup design, walls and ceiling demolition, and slab cutting. We decided to cut five 24-in wide strips of the floor system to test them to failure.

This chapter describes the in-situ load tests that were performed on a steel-concrete floor system at the former School of Nursing Building at University of Miami.

The aims of the load tests were:

1. To assess the ultimate structural performance of the floor system on a selected strip of the floor;
2. To evaluate the contribution of the concrete slab without shear studs on the floor system.-

The load testing procedure involved applying eight concentrated loads to the structural floor at pre-determined locations (Figure 33). The response of the structural member was monitored and used to evaluate that portion of the floor.

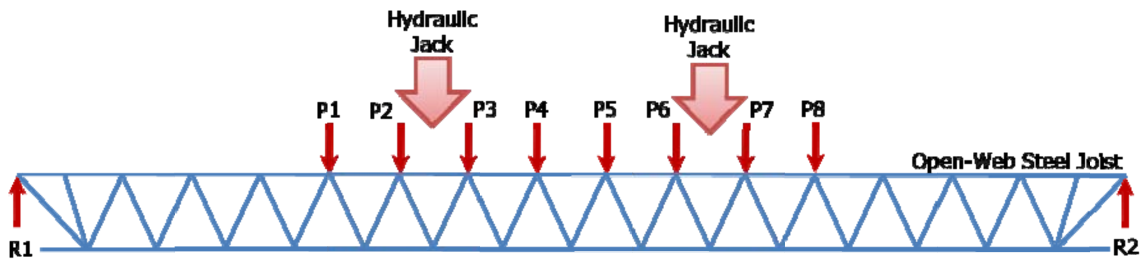


Figure 33 – Location of Concentrated Loads

3.2 TEST SITE: SCHOOL OF NURSING BLDG., UNIVERSITY OF MIAMI

The in-situ test took place in the School of Nursing building, a former fraternity house, located at 5801 Red Road, Coral Gables, Florida (Figure 34 and Figure 35). It was a two-story building, dating from 1957, that housed the school of nursing until 2006 when it moved to a new building in Coral Gables campus.

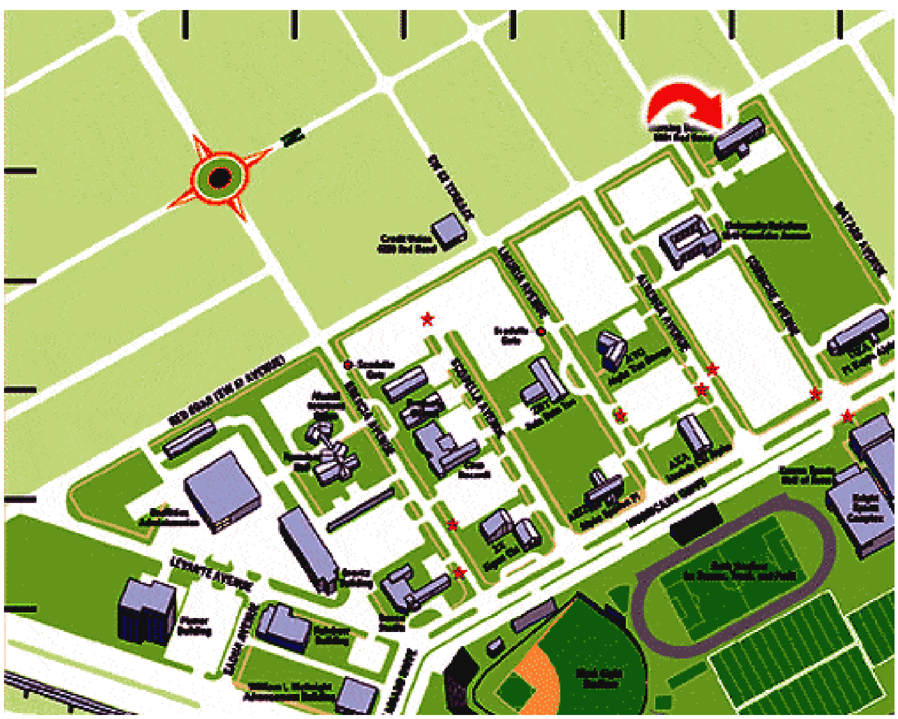


Figure 34 – Test Site: School of Nursing Location

Figure 35 – Test Site: School of Nursing Building

The building was constructed using open-web joists (Figure 36 through Figure 38) supported by masonry walls and, concrete columns and beams; the joists supported a 4”-thick concrete deck.

Figure 36 – Test Site: Open-Web Joists

Figure 37 – Test Site: Open-Web Joist Support

Figure 38 – Test Site: Open-Web Joist

Figure 39 – Test Site: Concrete Slab 2nd Floor (Strips)

3.3 Preliminary Investigations

The following summarizes the preliminary assessment of the structure and the sources for the information used in designing the load tests.

3.3.1 Structural Geometry

The structural geometry including member sizes were determined from the engineering drawings but mainly by inspection. The structural floor was a steel and concrete structural system formed by bonding a steel joist to a reinforced concrete slab in non composite action.

3.3.2 MATERIAL PROPERTIES AND CHARACTERISTICS

The material properties and characteristics were determined by taking samples of the steel and the concrete slab (Figure 40) as a result of lack of information from the drawings provided by Real Estate & Facilities Office of the University of Miami. The concrete slab is a 4-in thick reinforced concrete slab with a compressive strength (assumed), f'c = 4,000 psi; and the open-web steel joist is formed by 2 #7 rebar on the bottom, two 1-1/4" x 1-1/4" x 1/8" steel angles on the top and #6 rebar as diagonal members (Figure 41 and Figure 42). The tensile strength of the rebar was, Fy = 60,000psi and of the steel angles, Fy = 36,000 psi.

Figure 40 – Tensile Test on rebar

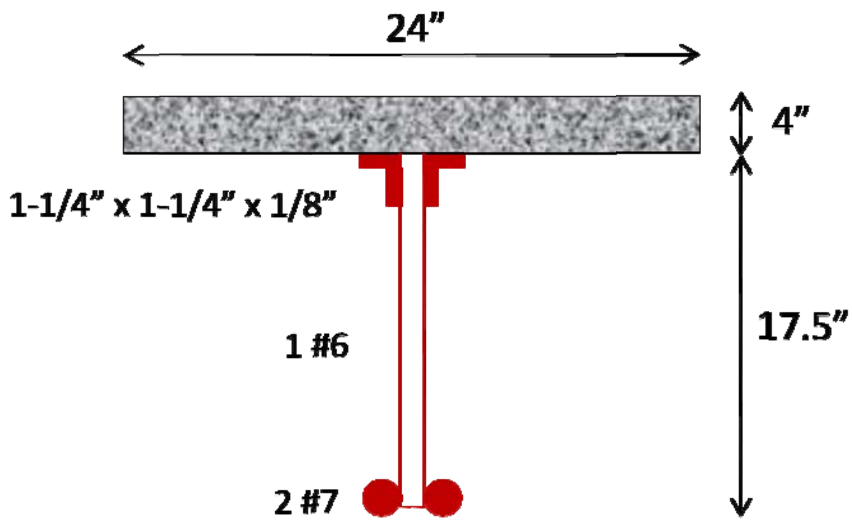


Figure 41 – Specimen No.1 Section

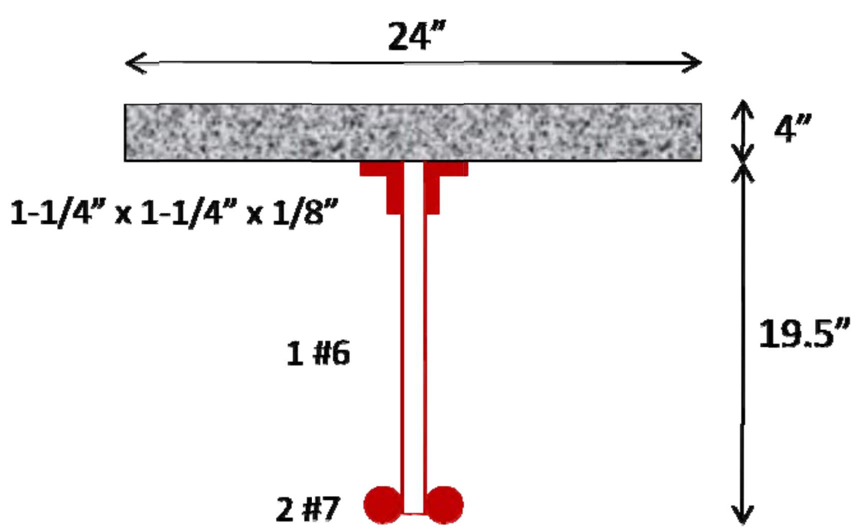


Figure 42 – Specimen No.2 Section

3.3.3 EXISTING STRUCTURAL JOIST CAPACITY

The joist capacity is typically determined using the Steel Joist Institute (SJI) Load Tables, which are contained in the SJI 75-year Steel Joist Manual (SJI, 2005). However, in our case the joist is a custom-made steel joist, which means it was fabricated for that specific job therefore it is not contained in the tables, for that reason the capacity of the joist had to be evaluated using structural principles.

In order to be able to perform a reliable analysis of the actual joist, we had to gather as much information about the joist system as possible. Helpful and required information included the following:

1. Year of Construction
2. Joist Configuration
3. Joist Span
4. Joist Spacing
5. Joist Depth
6. Type of Web Members
7. Type of Chords
8. Condition of Joists
9. Coupon Samples from Chords and Web Members to Determine Yield Strength
10. Coupon Samples from Concrete Slab
11. Type of Reinforcement of Concrete Slab
12. Determine the Existence of Shear Studs

3.3.3.1 STRUCTURAL ANALYSIS

A structural analysis (Figure 43 through Figure 46) was performed in order to determine the magnitude of the eight concentrated point loads that produce yielding on the bottom chord of the steel joist.

As a lower bound, a two-dimensional model of the main structural elements including the concrete slab as a dead weight, was used and then, compared to an upper bound model using plate elements to model the slab. The Finite Element model was implemented in commercial FEM software: Straus7.

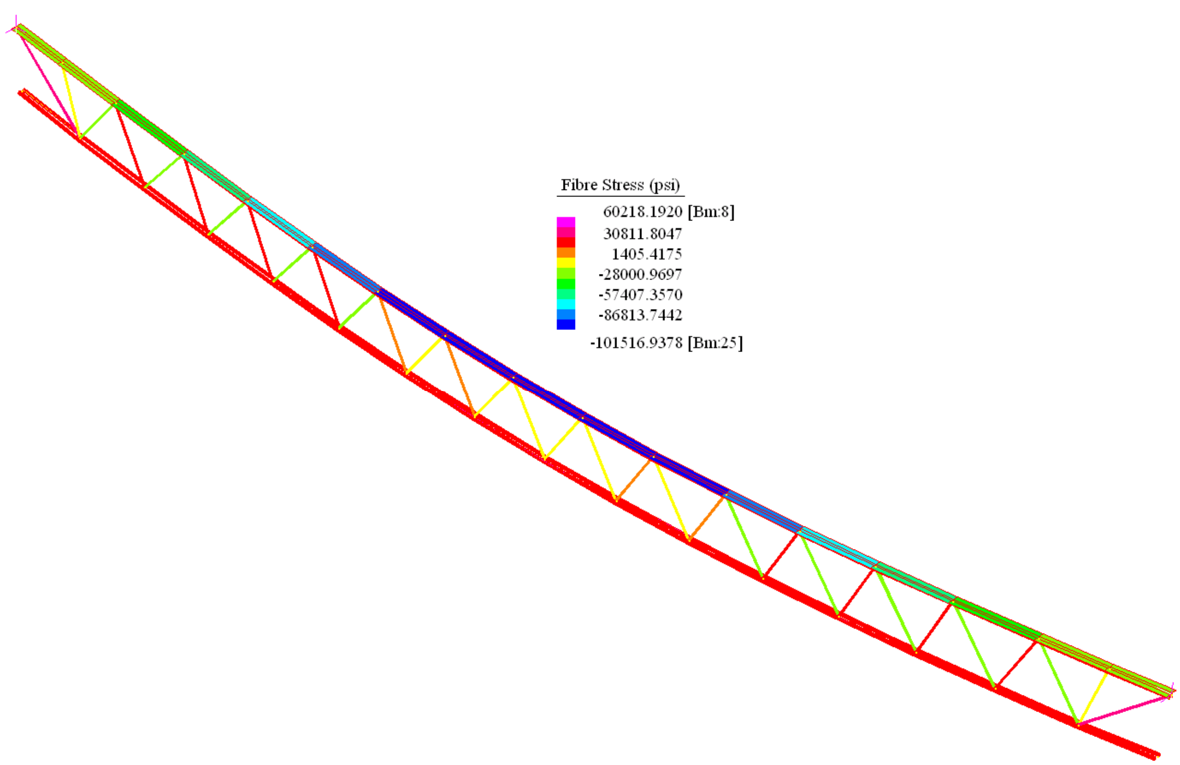


Figure 43 – Fiber Stress Specimen no.1 (Straus7)

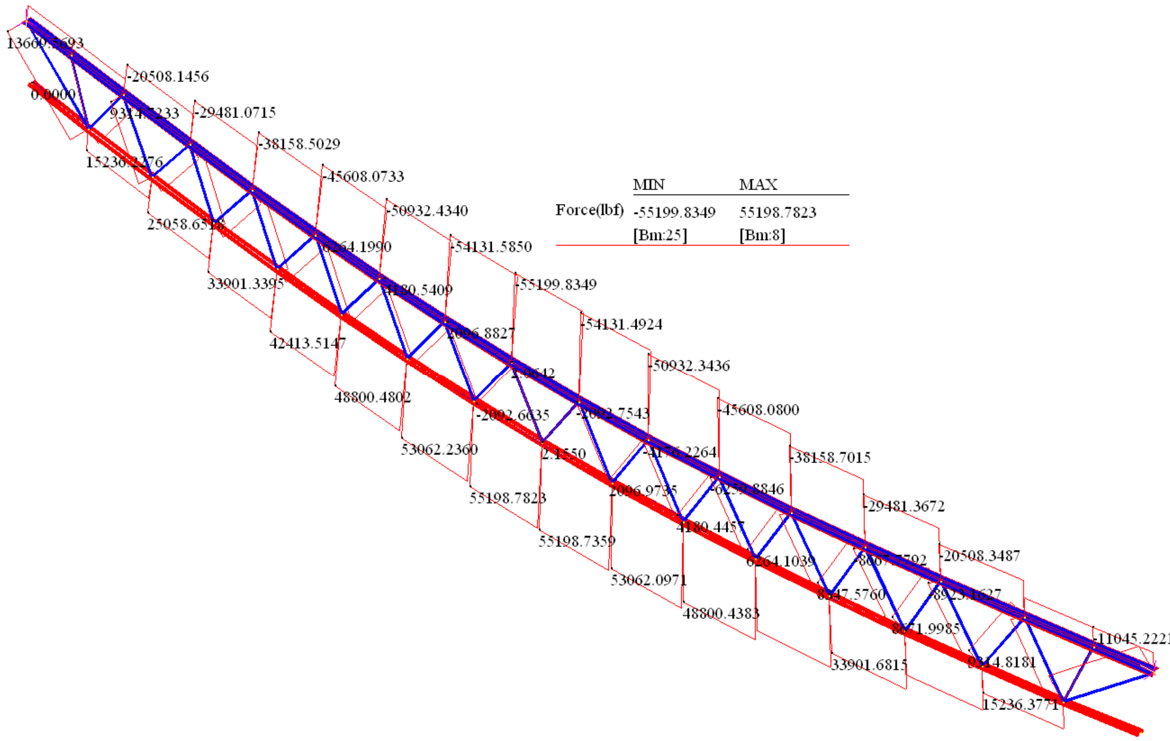


Figure 44 – Axial Loads Specimen no.1 (Straus7)

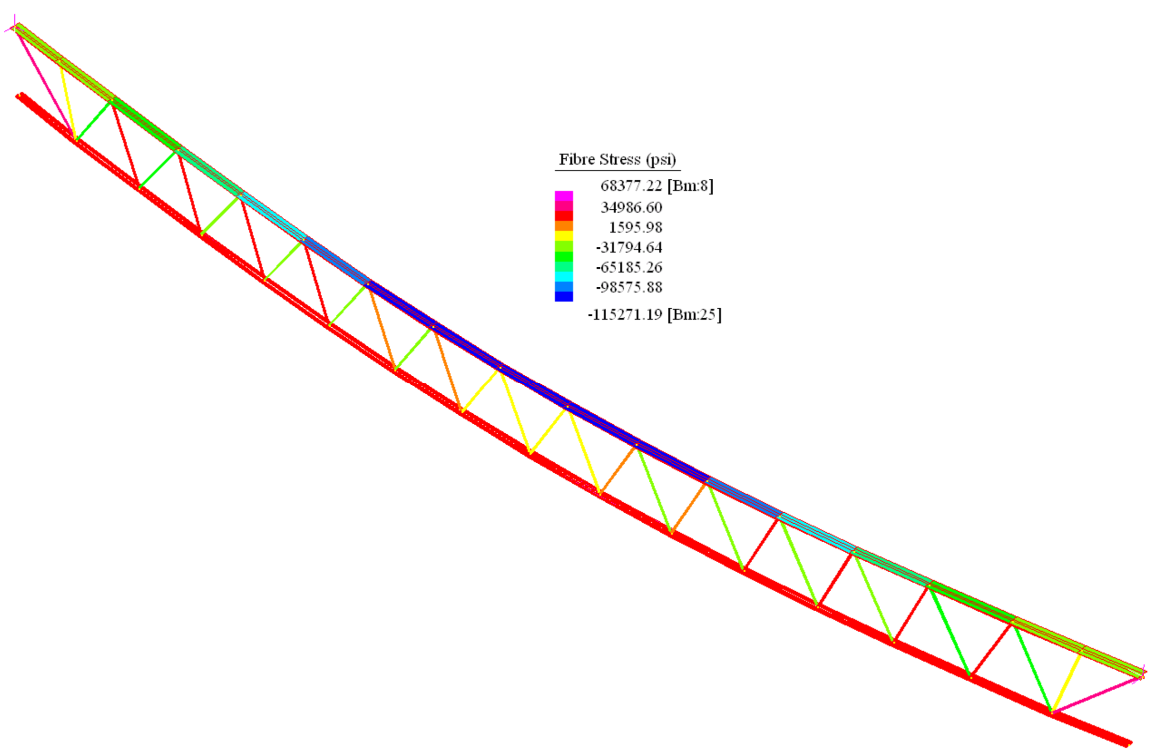


Figure 45 – Fiber Stress Specimen no.2 (Straus7)

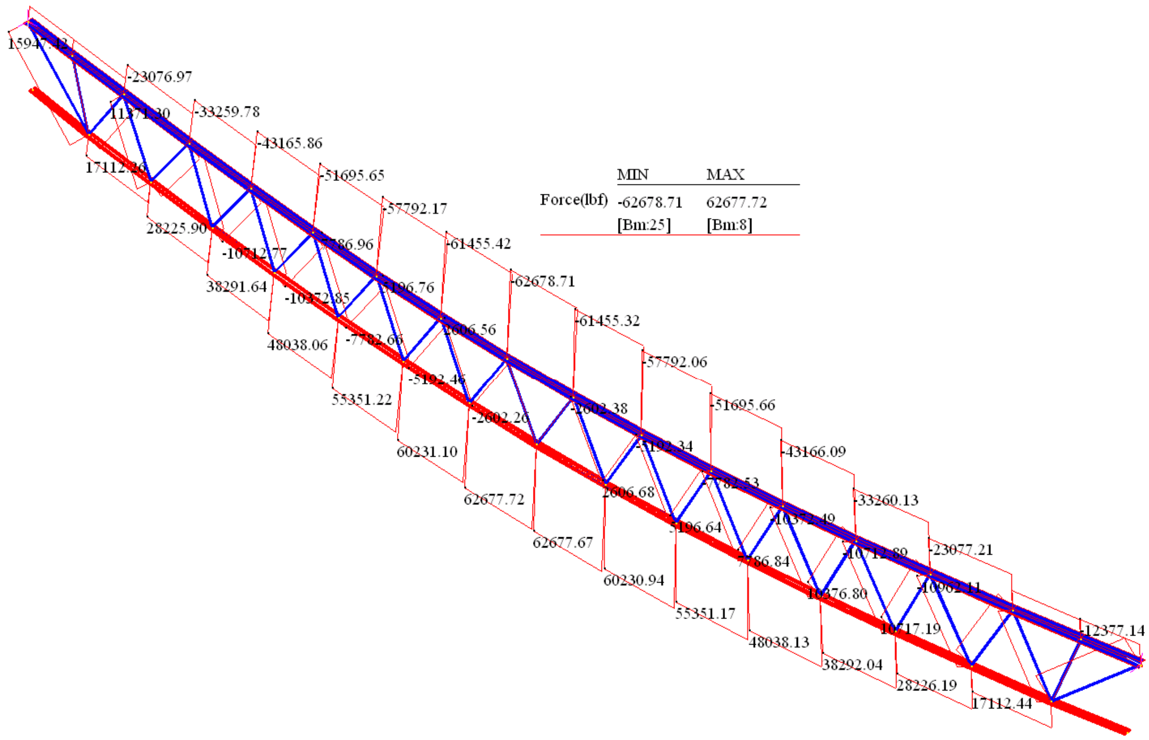


Figure 46 – Axial Loads Specimen no.2 (Straus7)

As a result of such analysis it was found that, using the loading layout given in Figure 64 and Figure 65, a maximum load of 6.40 kips on specimen no.1 and 7.24 kips on specimen no.2 should be applied at each loading point to reach the ultimate capacity of the strip.

Table 1 summarizes the findings in terms of point load P determined prior testing using the actual loading configuration for all the tests.

Test Label	P_{point} [kip]	P_{jack} [kip]	P_{Total} [kip]
Specimen 1	1.60	6.40	12.80
Specimen 2	1.81	7.24	14.48

Table 1 – Planned Point Load P Values

3.4 Description of the Load Test

3.4.1 Test Set-Up

The testing equipment to be used consisted of two 60-ton hydraulic cylinder jacks and one hydraulic hand pump (Figure 54) for applying the load, direct current differential transducers (DCVTs) for measuring deflections, two load cells of 25 kip each one, for measuring the applied load, and strain gages on both concrete and steel (Figure 64 and Figure 65). The DCVTs were mounted on frames for measuring the deflections from the second floor (Figure 50). This allowed having the data acquisition system at the same level of the loading jacks and to monitor the cracks during the test.

For applying the load we designed a setup using square steel beams to distribute the load (rigid enough to withstand the load), #8 thread bars from Dywidag Systems International, and Hilti rods (Figure 47) to anchor the whole setup to ground floor (Figure 49). The DSI thread bars (Figure 47) with a tensile strength of Fy = 150ksi were able to withstand a tensile force of approximately 118kips, much more than what we expected. For the anchoring system (Figure 48) we used 4 Hilti rods on each loading point with a total capacity of 8 kip, using a safety factor of 3.2.

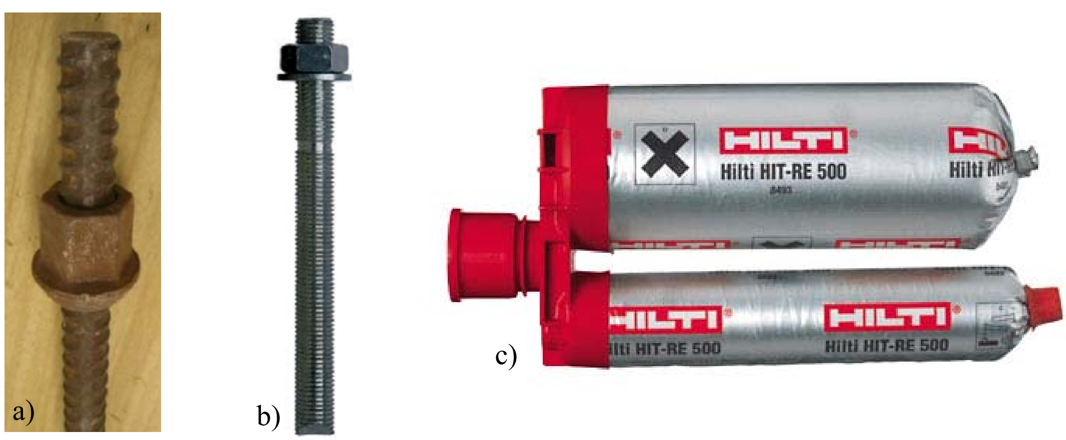


Figure 47 – a) DSI Thread Bar; b) Hilti Threaded Rod; c) Epoxy

Figure 48 – Anchoring System using Hilti Rods: Ground Level

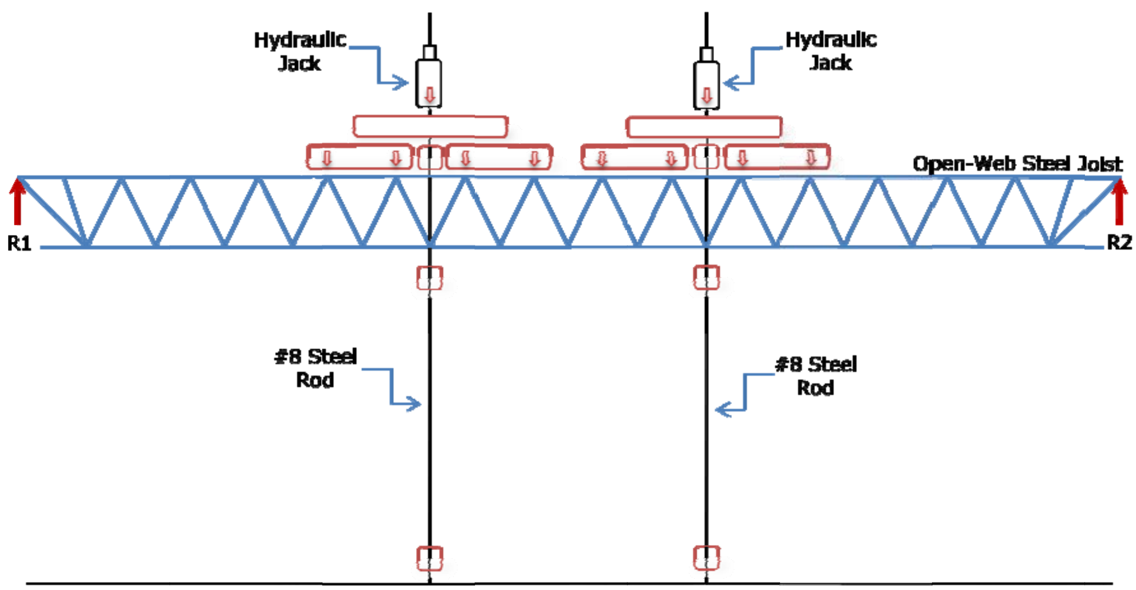


Figure 49 – Schematic of Test Setup Configuration

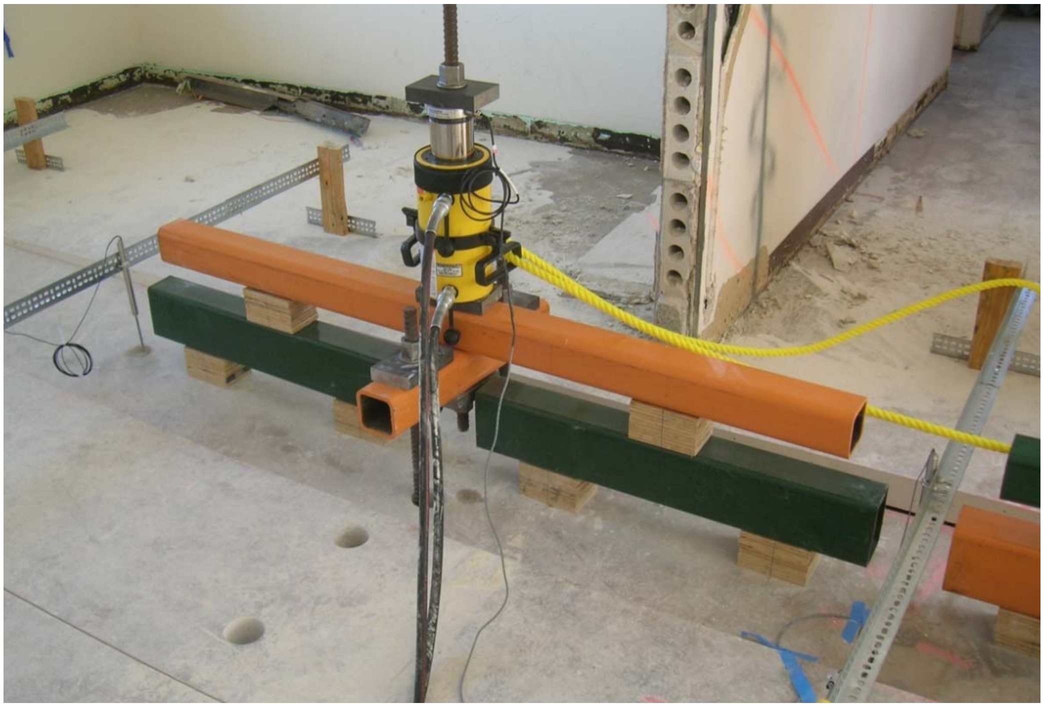


Figure 50 – Loading and Measuring Equipment

Figure 51 – Loading Setup: Steel Beams

Figure 52 – Loading Setup (Hydraulic Jacks)

Figure 53 – Loading Setup (1st Floor)

Figure 54 – Hand Pump

3.4.2 Data Acquisition System

As the data acquisition system we used one of the latest technologies from National Instruments. The PXI-1052 is an upgrade to the PXI-101, and works with all NI PXI controllers.

The portable data acquisition system with integrated signal conditioning enables the simultaneous use of 24 strain gages input channels and 32 channels for VDC signals. The unit is controlled and operated by a laptop/PC using a PCMCIA connection. The SC module includes:

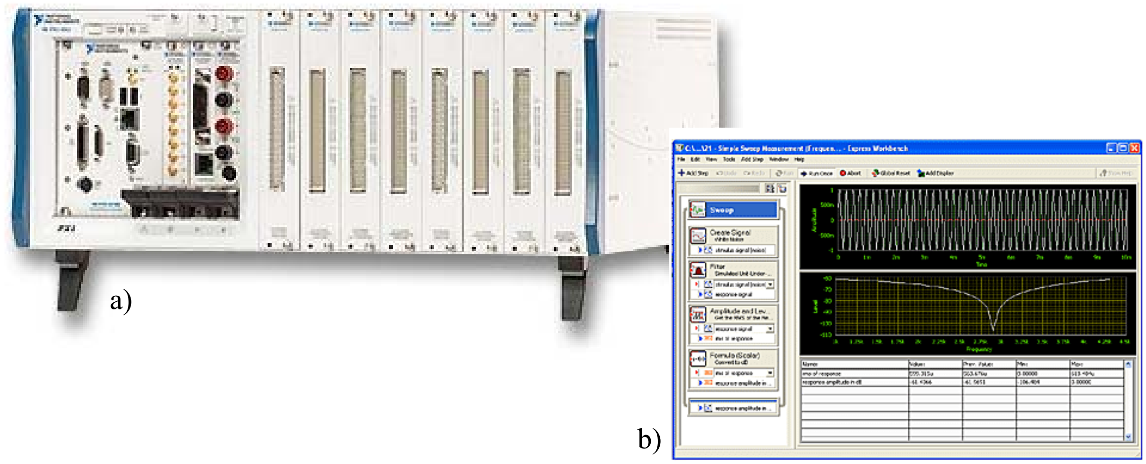
- Three 8-channel (total 24 channels) universal strain gauge analog-input modules with an independent 0-10 V programmable excitation source per channel, and quarter, half, and full-bridge completion capabilities. Typical strain-based sensors that can be interfaced with the module include electrical resistance strain gauges, load cells, strain-based displacement transducers;

- A 32-channel signal conditioning module for high-accuracy thermocouple measurements, complemented by a front-mounting terminal block. The module also can acquire millivolt, volt, 0 to 20 mA, and 4 to 20 mA current input signals. Typical DC-excitation analog-output sensors that can be interfaced with the module include DC-LVDT displacement transducers, draw-wire transducers, linear potentiometers, inclinometers, and pressure transducers.

DC power supply is provided by a triple-output (0-20 V, 0-6 V, 0- –20V) programmable DC unit that operates in conjunction with an auxiliary power source.

The data acquisition system (Figure 55 and Figure 56) was set to record data at a rate of 5Hz from all devices, displaying real time on a computer screen of the significant locations using LabVIEW Signal Express (Figure 56), a software developed recently by National Instruments and that provides an interactive measurement workbench for quickly acquiring, analyzing, and presenting data from hundreds of data acquisition devices and instruments, with no programming required.

Figure 55 – Data Acquisition System



a)

b)

Figure 56 – a) NI-PXI-1052; b) LabView Signal Express

3.4.3 Deflection Measurement

Deflection measurements were taken in many locations so that a significant portion of the strip was monitored during the entire load test. Deflection measurements were taken with 0.5in, 1in and 2in DCVT's (Figure 57) mounted on a frame anchored to the floor at the same level of the load tests. The layout of the DCVTs for the two load tests is shown in Figure 64 and Figure 65.

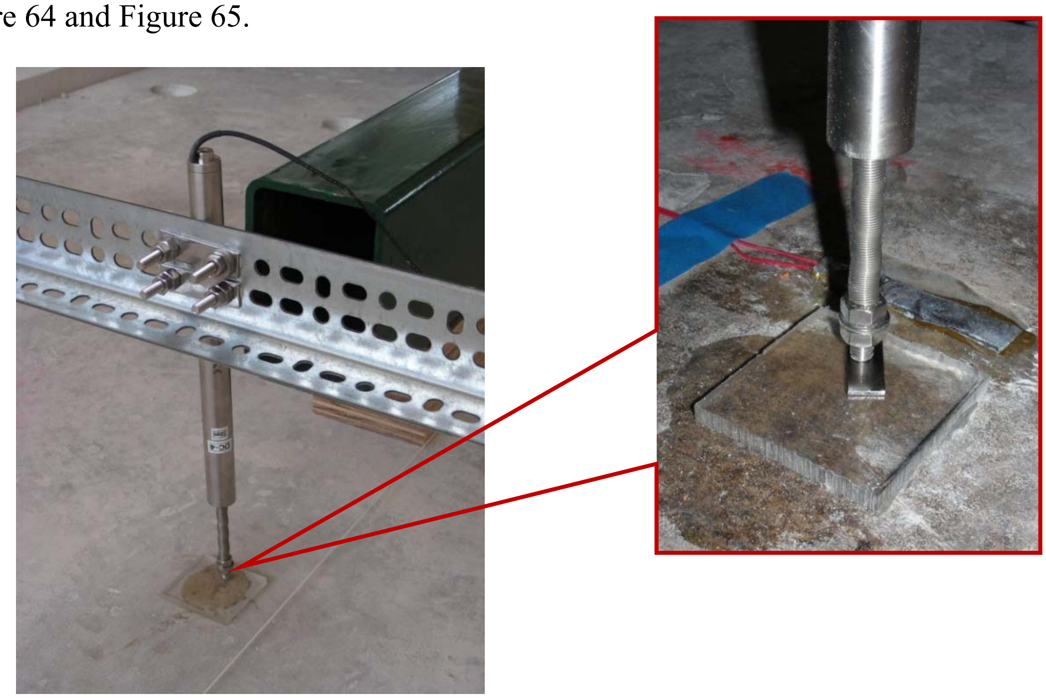


Figure 57- Differential Transducers (DCVTs)

3.4.4 LOAD MEASUREMENT

As mentioned before, in order to be able to measure the load applied we used donut shaped load cells from Cooper Instruments (Figure 58) with a total capacity of 25,000 lbs and an accuracy of ±5%. This type of load cell because of their rugged construction and design are an economical and ideal answer for application such as this test, which required a large through-hole design for applying the load.

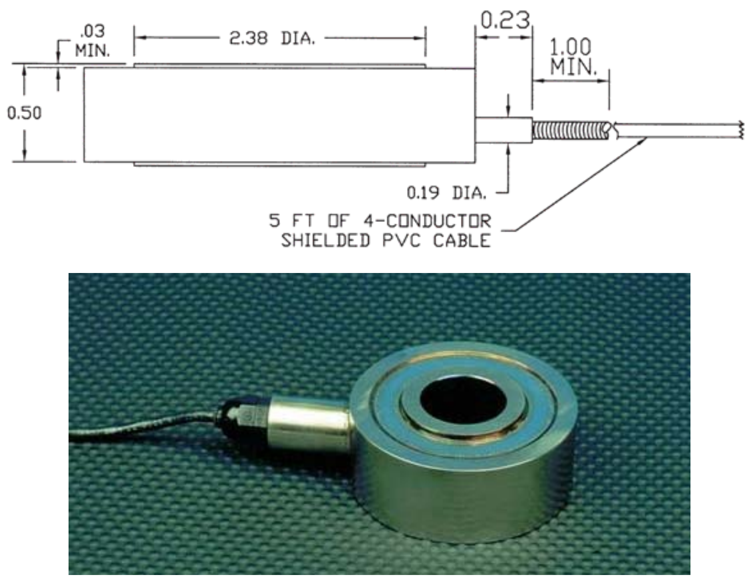


Figure 58- Donut Shaped Load Cell

3.4.5 STRAIN MEASUREMENT

For measuring the strain in both steel and concrete, we were able to use one of the most common devices to measure deformation of objects since 1938. For this test we used pre-wired strain gages (Figure 59 and Figure 60) from Omega Engineering with a total capacity of 5% and a length of 5mm for steel and 60mm for concrete; those strain gages were installed on the concrete and on the steel so we could measure the strain on different locations while applying the load. These strain gages were attached using a suitable adhesive, such as cyanoacrylate. As the object was deformed, the foil was deformed, causing the strain gage resistance to change; with this resistance change and using a quarter-bridge configuration and a gage factor of 2.1 we were able to read the actual strain of the different locations.

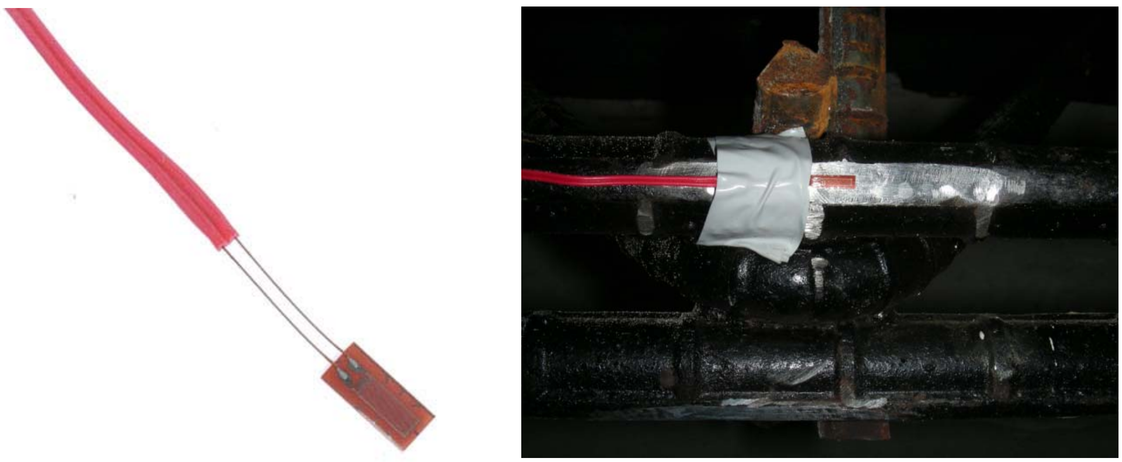


Figure 59- Pre-wired KFG Series Strain Gage: Omega Engineering

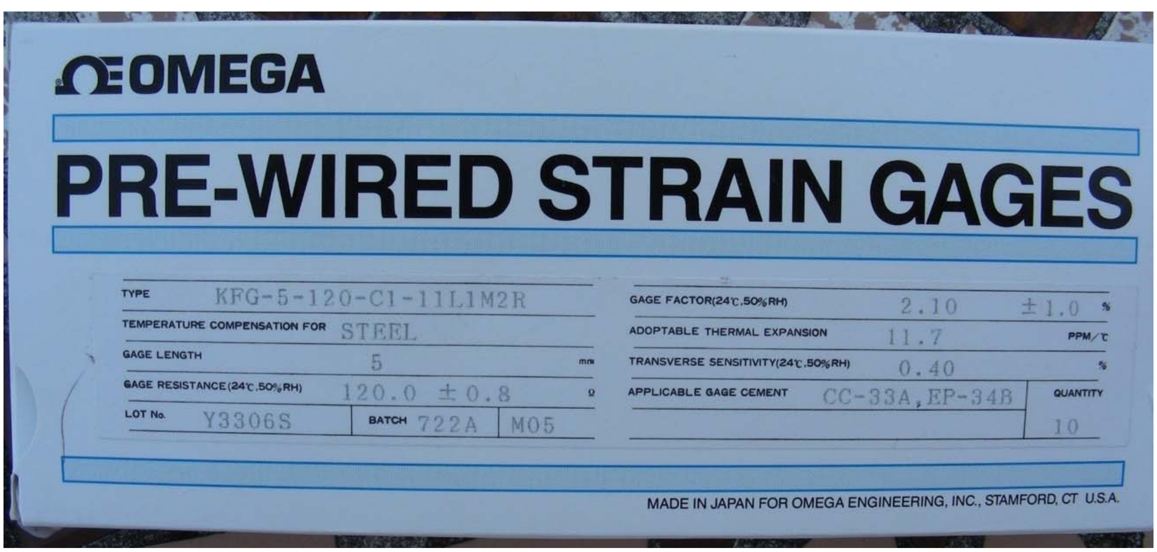


Figure 60- Pre-wired KFG Series Strain Gage: Omega Engineering

3.4.6 Load Test Configurations

The two load-tests were performed in a push-down method. In particular, two hydraulic jacks (one for each loading point) were used to provide the load that result in downward concentrated forces on the test member.

Figure 61 shows an overall schematic of the two tested areas.

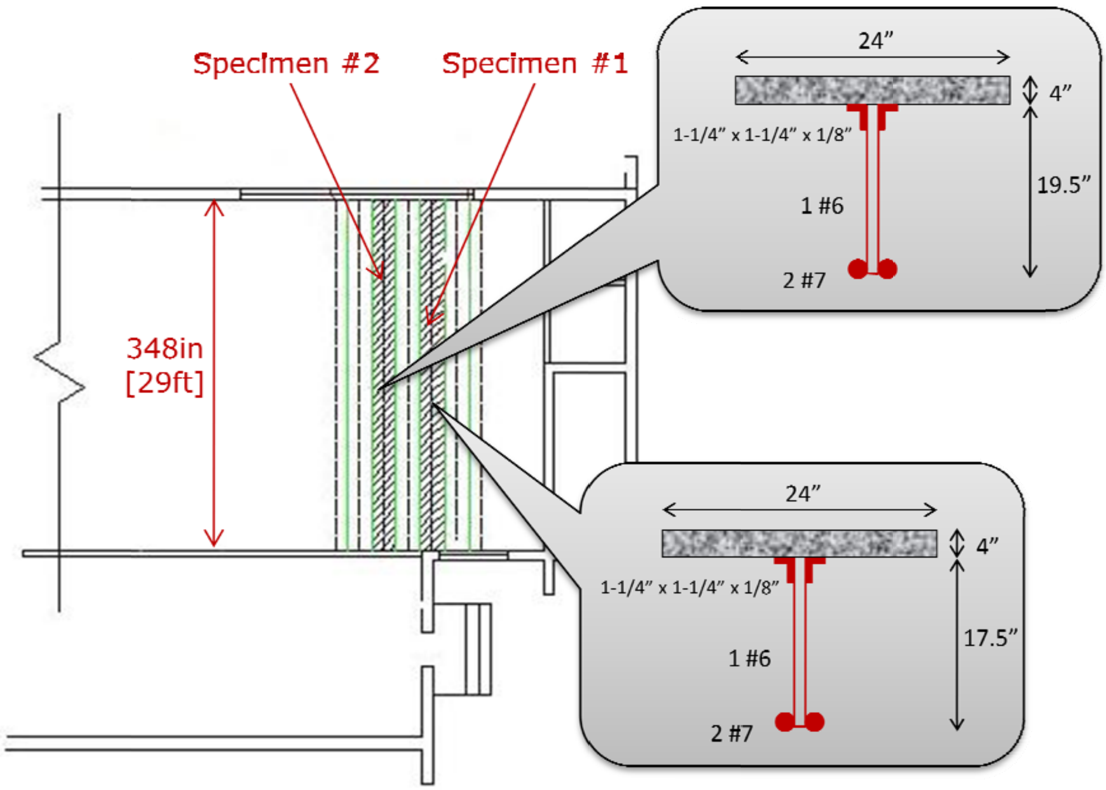


Figure 61– Schematic of the Load Test

Figure 62 -Schematic of Strips

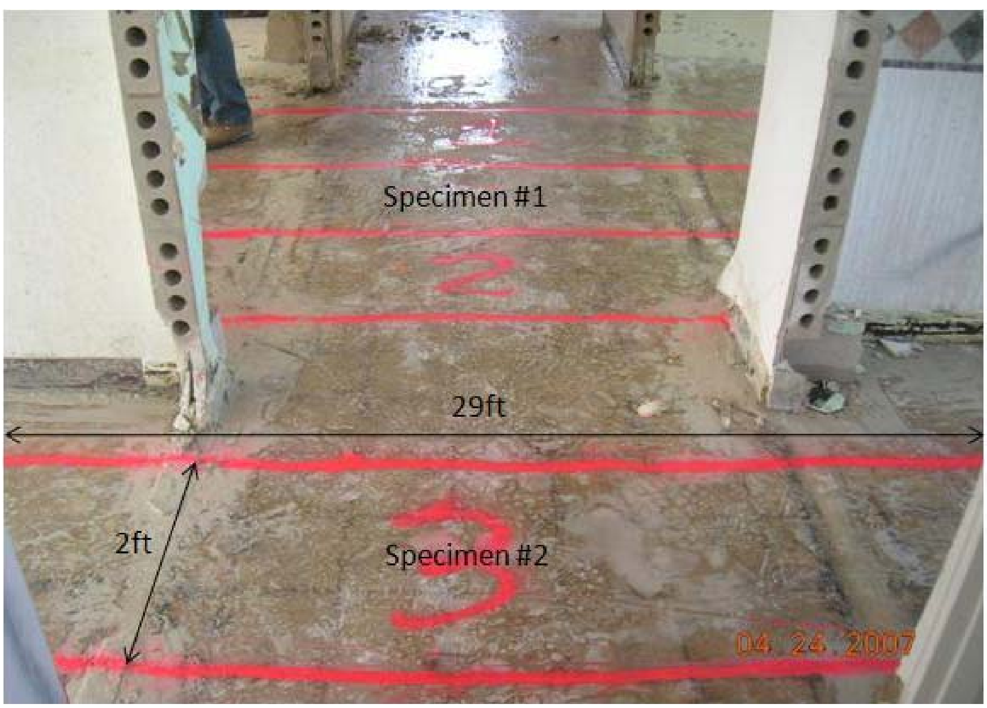


Figure 63-Schematic of Strips

The list of the instruments used for the load test for Specimen no. 1 is given in Table 2.

Sensor	Sensor ID	Measurement Reference
DCVT (±0.5 in)	DC0	Deflection
DCVT (±1.0 in)	DC1	Deflection
DCVT (±2.0 in)	DC3	Deflection
DCVT (±2.0 in)	DC4	Deflection
Electronic Inclinometer (±10 deg)	DC5	Rotation
Electronic Inclinometer (±10 deg)	DC7	Rotation
Strain Gage	SG0	Strain on Steel
Strain Gage	SG1	Strain on Steel
Strain Gage	SG2	Strain on Steel
Strain Gage	SG3	Strain on Steel
Strain Gage	SG4	Strain on Concrete
Strain Gage	SG5	Strain on Steel
Load Cell (25 kip)	LC6	Applied Load
Load Cell (25 kip)	LC7	Applied Load

Table 2 –Instruments to be used for Specimen No.1

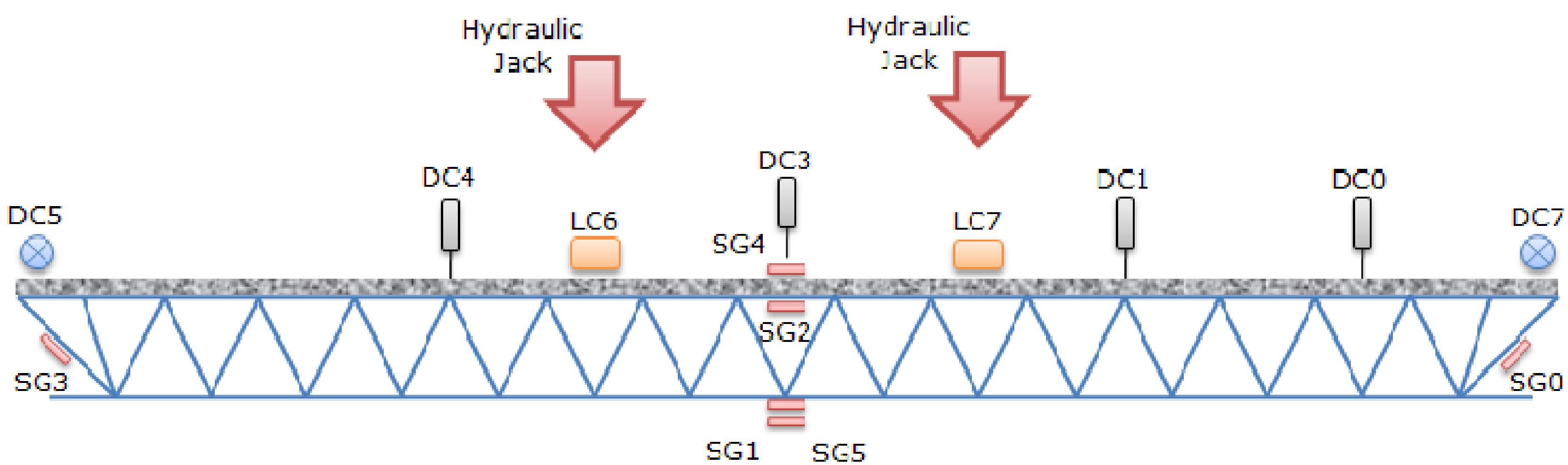


Figure 64–Sensor Distribution Specimen No.1

The list of the instruments used for the load test for Specimen no. 2 is given in Table 3.

Sensor	Sensor ID	Measurement Reference
DCVT (±1.0 in)	DC1	Deflection
DCVT (±2.0 in)	DC3	Deflection
DCVT (±2.0 in)	DC4	Deflection
Electronic Inclinometer (±10 deg)	DC5	Rotation
Electronic Inclinometer (±10 deg)	DC7	Rotation
Strain Gage	SG0	Strain on Steel
Strain Gage	SG1	Strain on Steel
Strain Gage	SG2	Strain on Steel
Strain Gage	SG3	Strain on Steel
Strain Gage	SG4	Strain on Concrete
Strain Gage	SG5	Strain on Steel
Load Cell (25 kip)	LC6	Applied Load
Load Cell (25 kip)	LC7	Applied Load

Table 3 –Instruments to be used for Specimen No.2

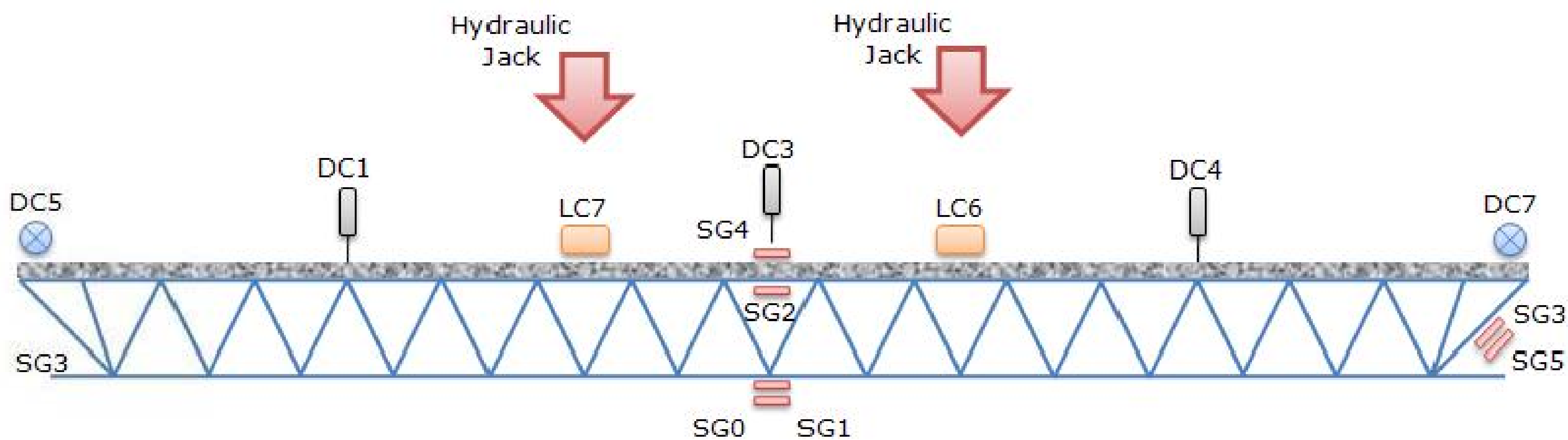


Figure 65 –Sensor Distribution Specimen No.2

3.5 WORK SCHEDULING

On the day prior to the test, the steel beams, accessories, and wood bearing plates were checked.

On the day of the test, all equipment was setup first for Load Test 1. The exact location of the instruments was determined. The instruments and the hydraulic cylinders and accessories were put in place on the second floor. DCVTs were extended at least one hour before testing to insure any creep of these stands is accumulated before testing began. Once the instruments were connected to the data acquisition system and the hydraulic cylinders were connected to the pump, a preliminary load was applied. The preliminary load was achieved by applying a minimum load of 1,000 lbs with the hydraulic jack. Following the preliminary load, the load test was executed. Each load cycle consisted of loading the structure in steps. Each load step was maintained for at least 1 minute. During this time the deflection of several location of the strip was monitored for stability. If the deflection at some point began to increase with a constant load, the system had past the elastic threshold and the test was halted. The peak load for each successive cycle was gradually increased to approach the maximum test load. Two cycles using the maximum test load were applied to verify repeatability of the measurements. The load test lasted 2-3 hours.

The actual load cycles varied slightly depending on the performance of the system as monitored during the test and the minimum load that has to be maintained to eliminate slack in the system.

Once the joist test was completed the equipment was dismantled and rearranged for the other tests, following identical procedures.

CHAPTER IV

ANALYSIS AND DISCUSSION OF TEST RESULTS

4.1 INTRODUCTION

In this chapter, data acquired from present study and those from the literature were combined together for a comprehensive study on the behavior of steel-concrete floor system at the former School of Nursing Building at University of Miami.

Two steel joists were tested applying 8 concentrated loads at specific locations using hydraulic jacks (Figure 66). As mentioned before, both specimens were instrumented with strain gages, displacement transducers and load cells.

Figure 66-Hydraulic Jacks

4.2 Test Results: Specimen No.1

The procedure of the load test on specimen no.1 consisted of the application of concentrated loads in a quasi-static manner, in 7 loading/unloading cycles (Figure 67 through Figure 72).

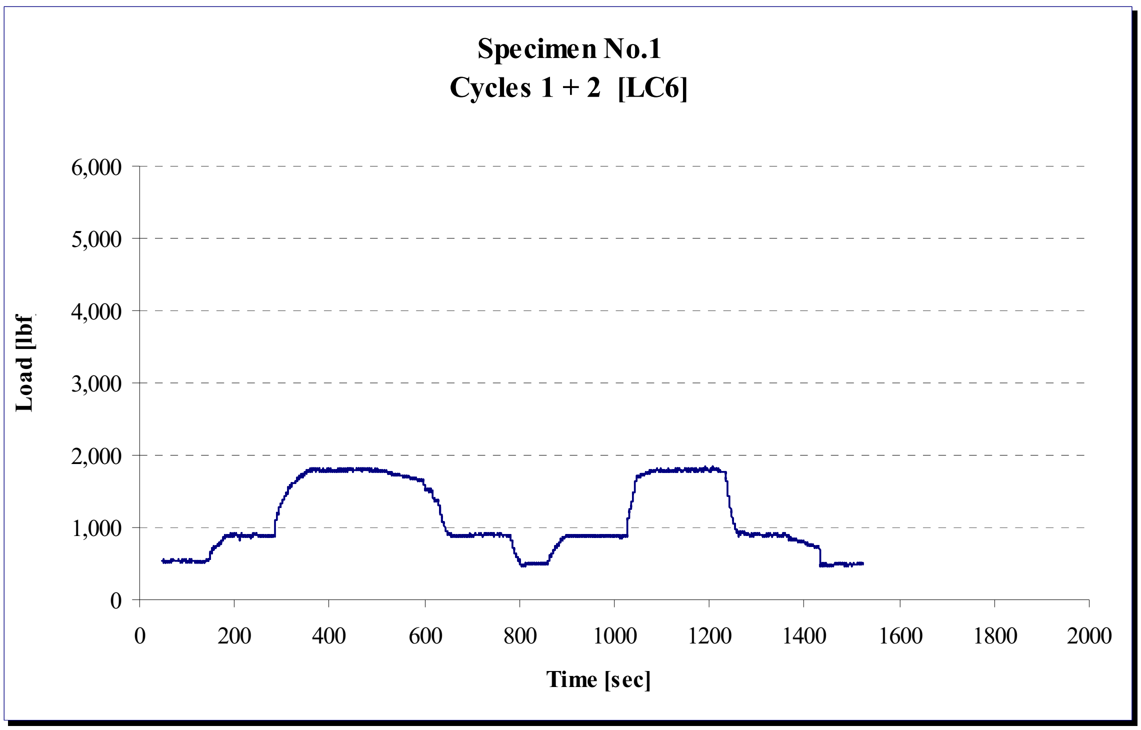


Figure 67-Cycles 1 and 2: Specimen No.1

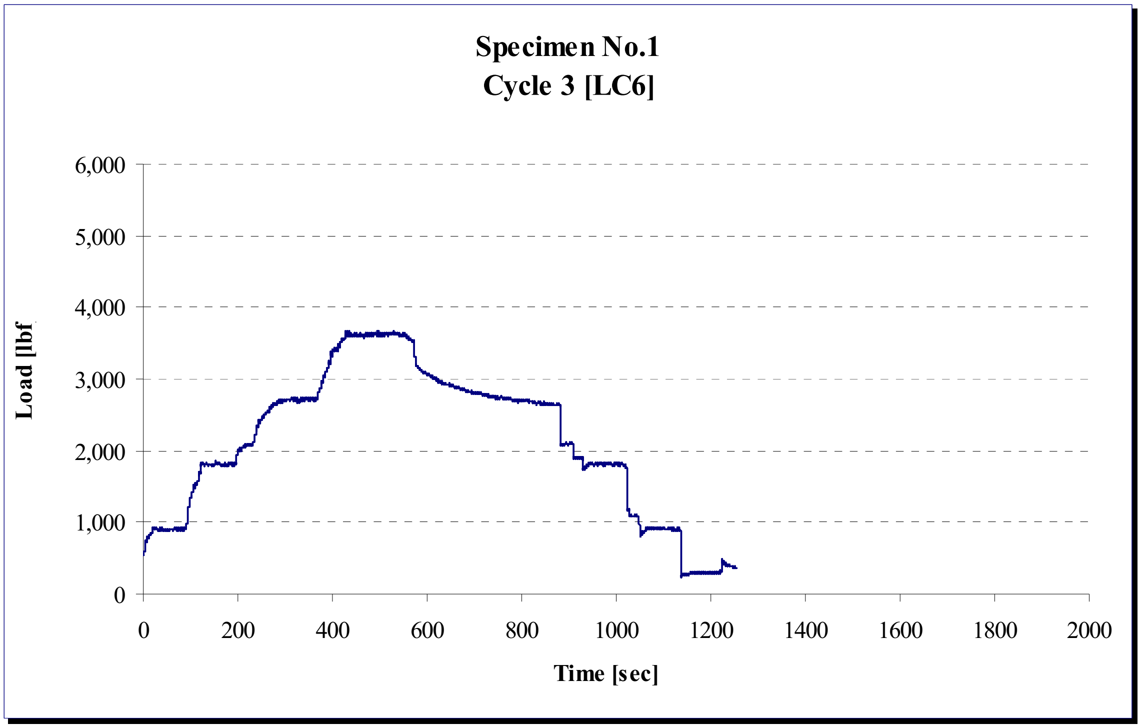


Figure 68-Cycle 3: Specimen No.1

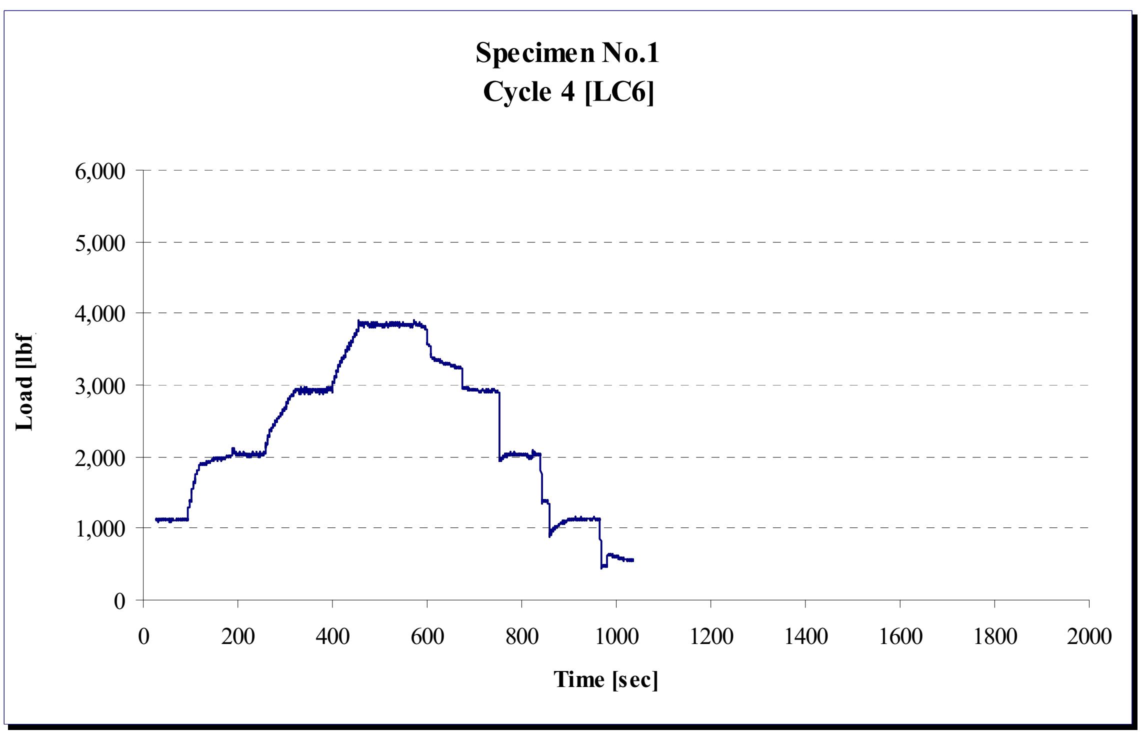


Figure 69-Cycle 4: Specimen No.1

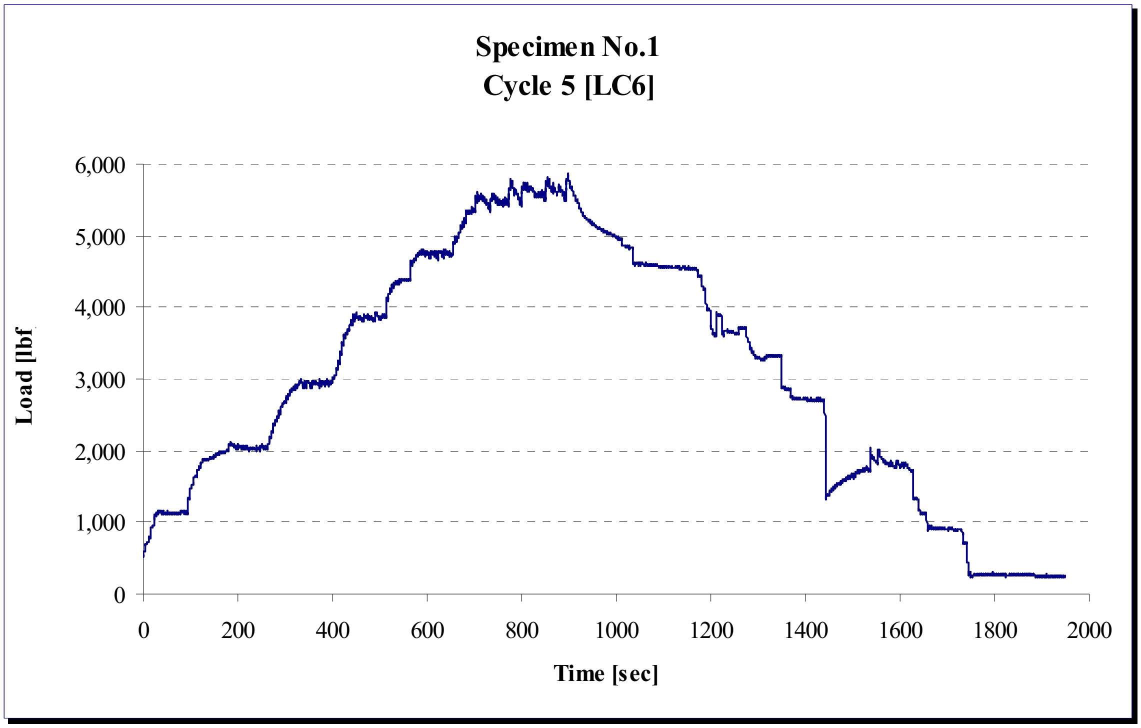


Figure 70-Cycle 5: Specimen No.1

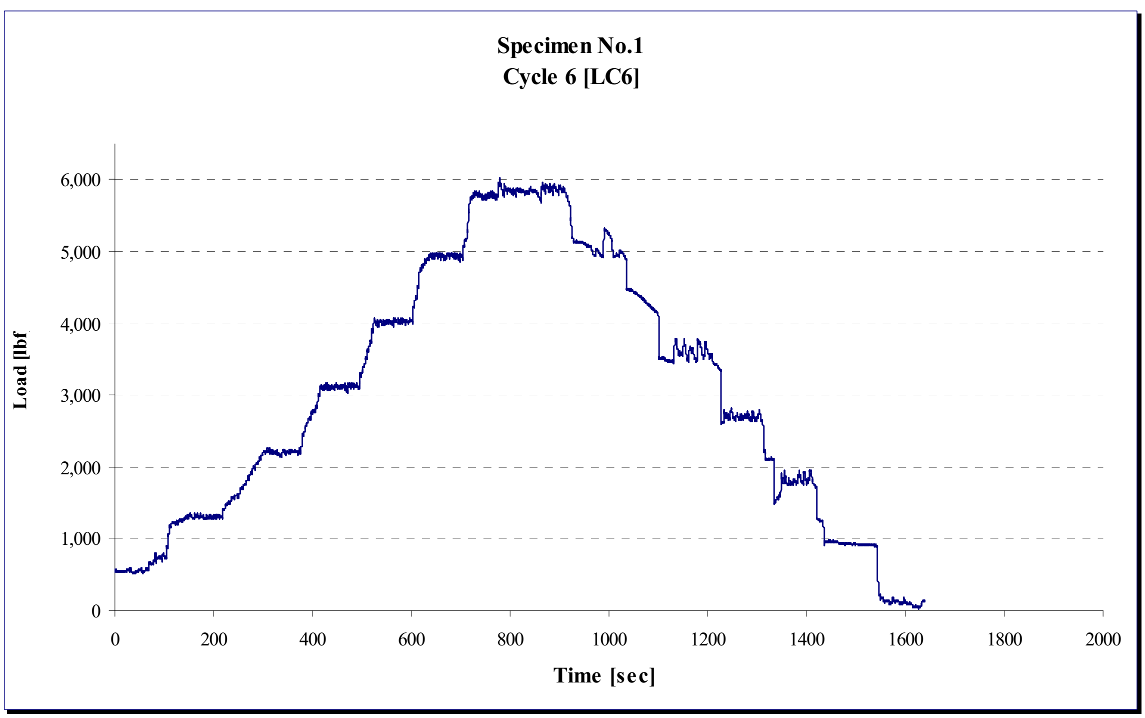


Figure 71-Cycle 6: Specimen No.1

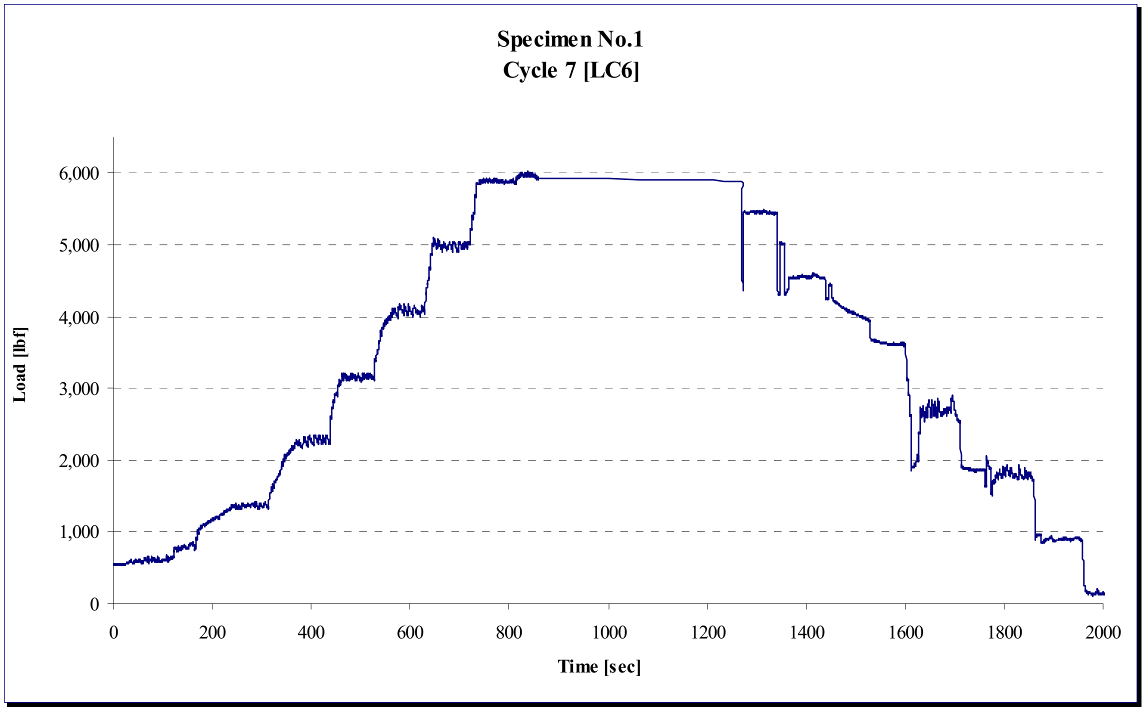


Figure 72-Cycle 7: Specimen No.1

Load-strain curves were plotted using the experimentally measured strains for each cycle. In this specimen the bottom chord was stressed beyond the yield point of the material, no buckling failure occurred (Figure 73 through Figure 75).

Figure 73-Failure Mode: Yielding Bottom Chord (Specimen no.1)

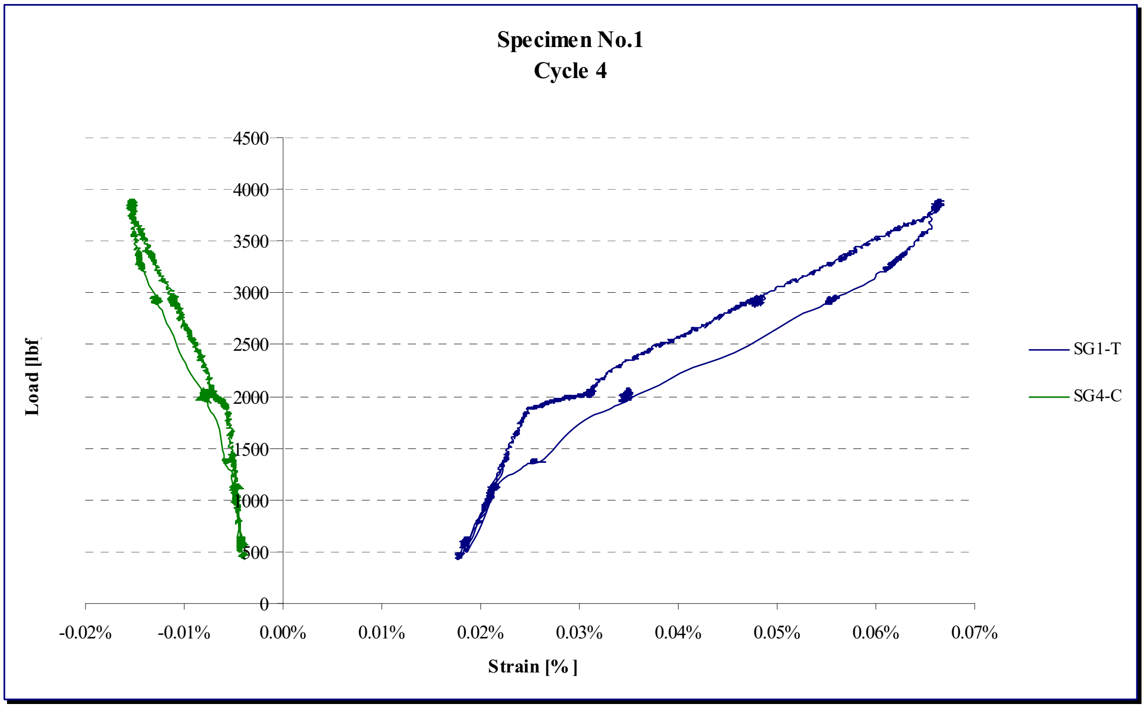


Figure 74 – Load-Strain Plot, Cycle 4: Specimen No.1

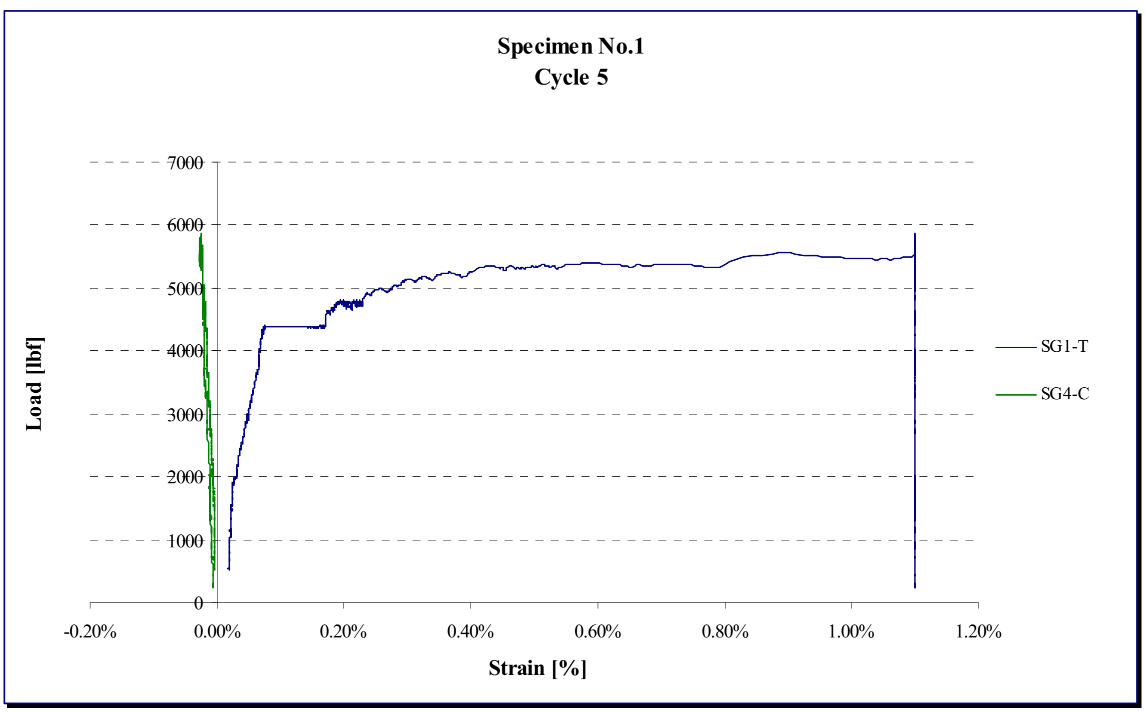


Figure 75 – Load-Strain Plot, Cycle 5: Specimen No.1

The load-deflection curve (Figure 76) shows that the curve begins to deviate from a straight line at approximately the same load. The load corresponding to break or rapid change of curvature of the load deflection curve is where the plastic behavior occurred.

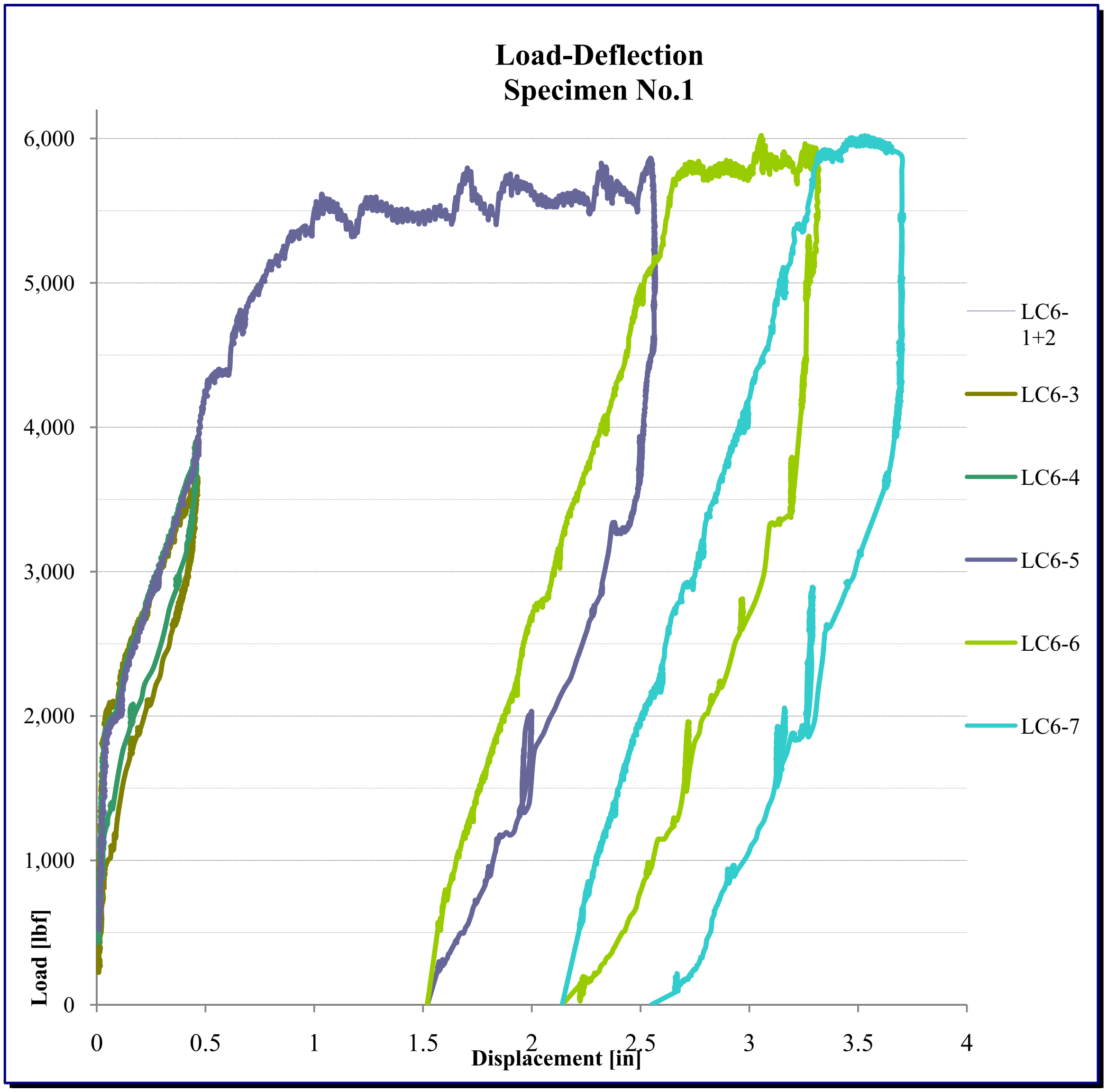


Figure 76 – Load-Deflection Plot: Specimen No.1

4.3 TEST RESULTS: SPECIMEN NO.2

The procedure of the load test on specimen no.2 consisted of the application of concentrated loads in a quasi-static manner, in 2 loading/unloading cycles (Figure 77Figure 67 and Figure 78). Unfortunately during the last cycle one of the jack was applying more pressure than the other one, causing a different behavior and hence a different mode of failure.

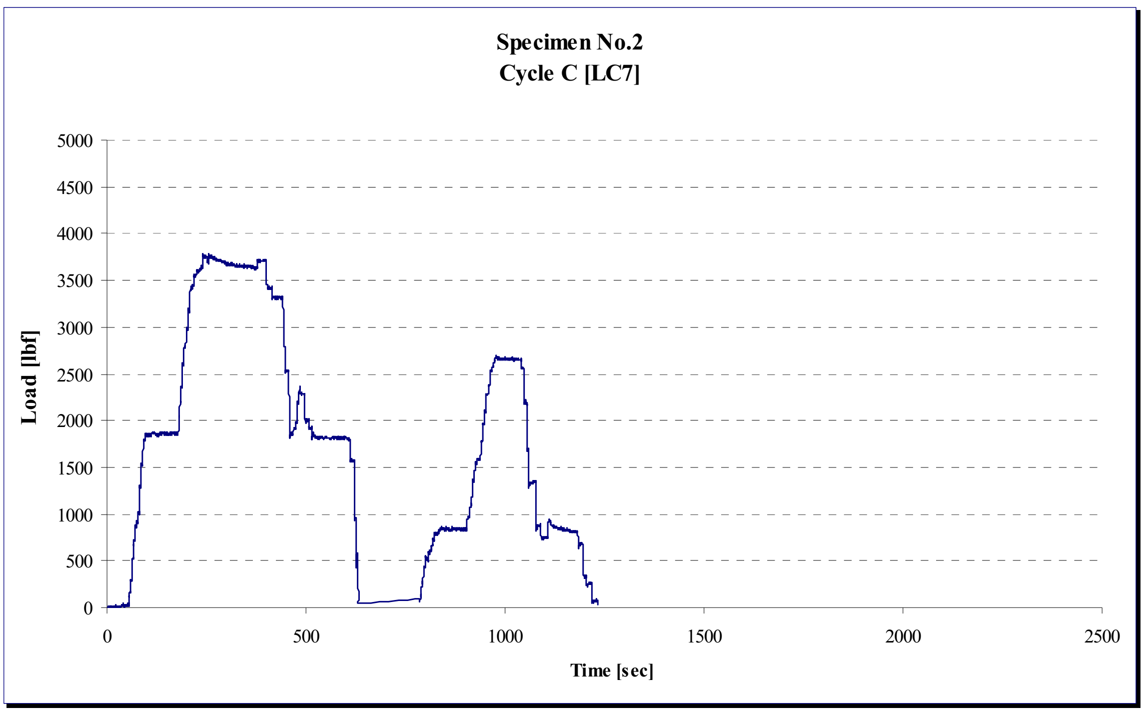


Figure 77 – Cycle 1: Specimen No.2

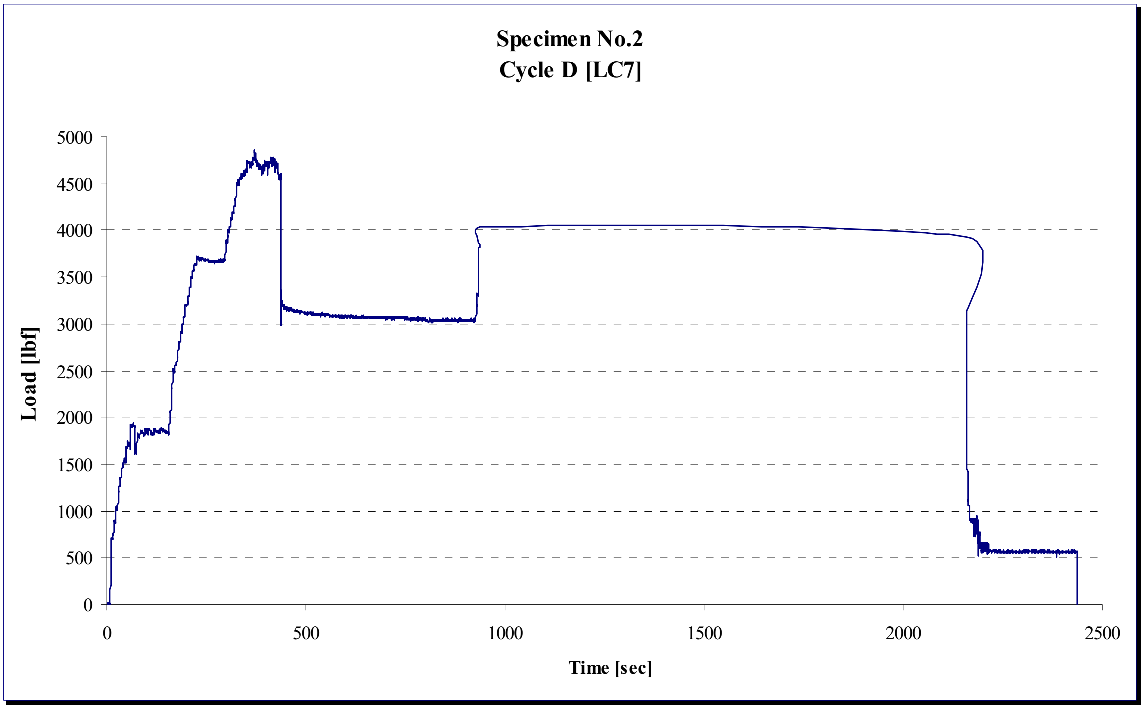


Figure 78 – Cycle 2: Specimen No.2

In specimen no.2 the bottom chord was stressed beyond yield point of the material, however, buckling failure occurred on diagonals (Figure 79 through Figure 81).

Figure 79 – Diagonal Bar Buckling: Specimen No.2

Figure 80 – Buckling Failure: Specimen No.2

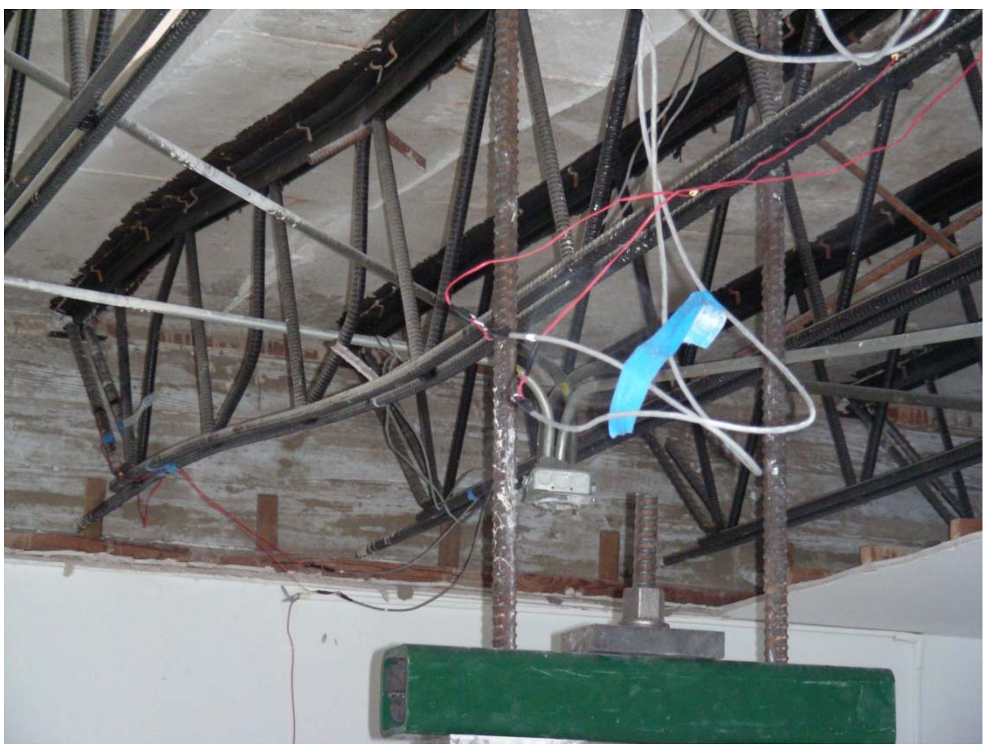


Figure 81 – Buckling Failure: Specimen No.2

The load-deflection curve (Figure 82) shows that the curve begins to deviate from a straight line at approximately the same load. The load corresponding to break or rapid change of curvature of the load deflection curve is where the plastic behavior occurred.

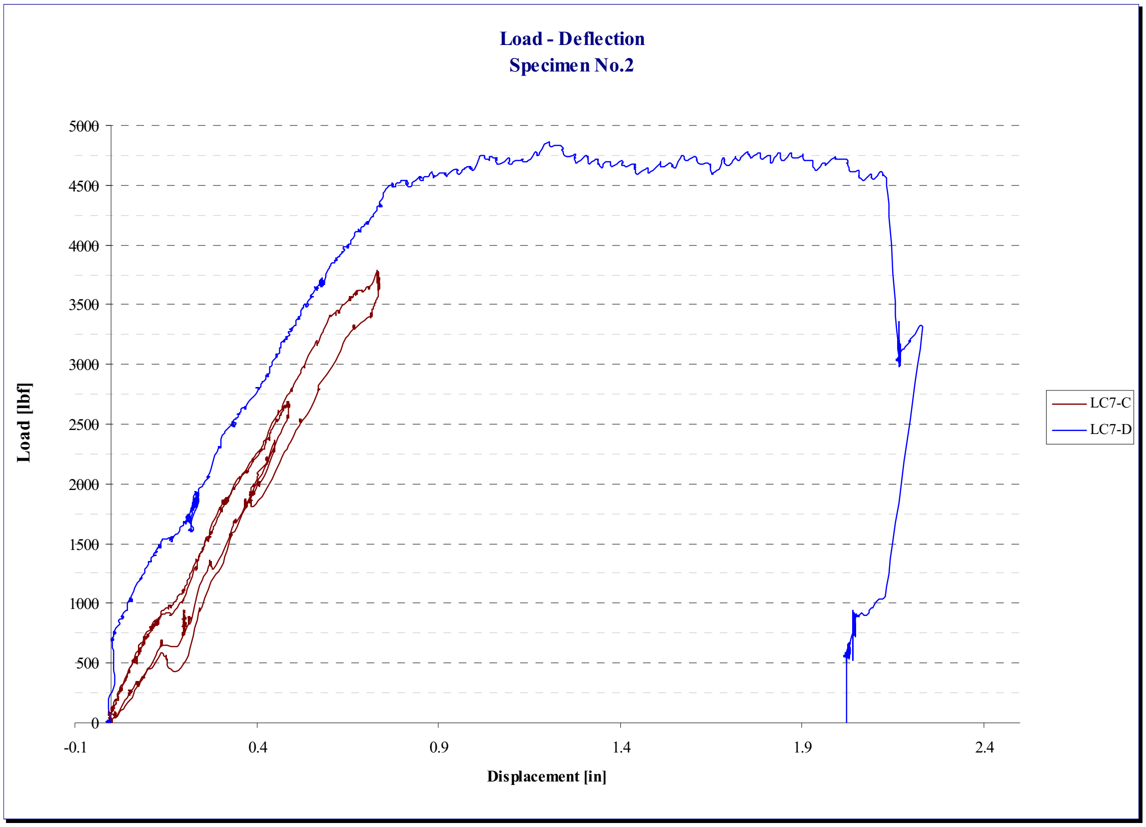


Figure 82 – Load-Deflection Plot: Specimen No.2

4.4 CORRECTION FACTOR

After processing and analyzing all the data collected from the tests, we found out that the test results from both tests were not similar to the analytical results, putting doubt in our minds about the data collected. First of all we started revising, over and over, the analytical model finding no difference, after knowing that the analytical approach was correct, we started testing the sensors again, finding that the sensors by themselves were working properly; however we realized that during the tests, on both specimen no.1 and specimen no.2, the load cells were not properly installed, meaning that the load, on one side of the load cell, was not applied on the loading surface of the load cell, creating a discrepancy. In an attempt to solve this issue, and with the unfortunate reality that we did not have the building anymore to try to test another specimen, we tested, on a small setup in the laboratory, the load cells and found a correlation between the correct and incorrect configurations (Figure 83 through Figure 87).

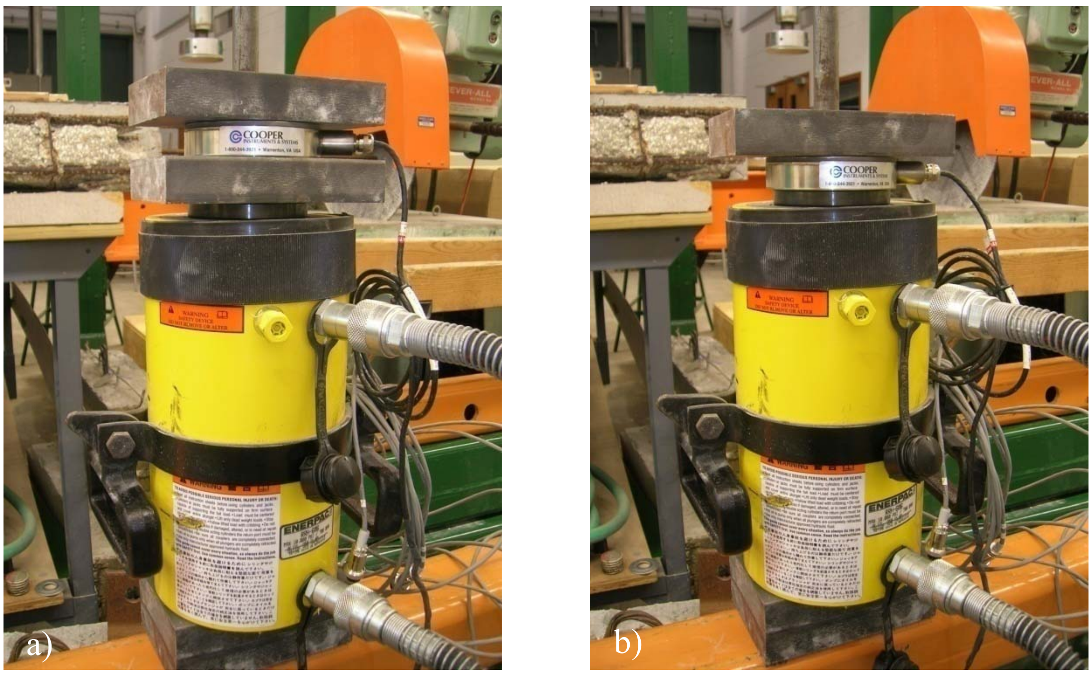


Figure 83 – a) Correct Configuration; b) Incorrect Configuration

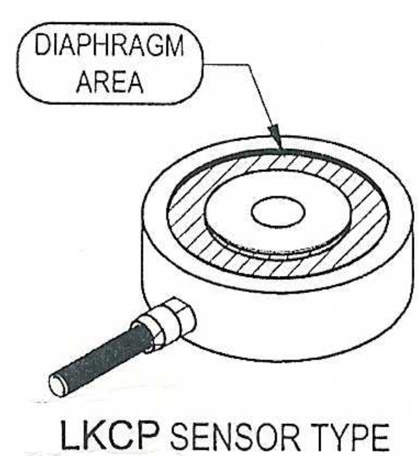


Figure 84 – LKCP Load Cell Detail

Figure 85 - Load Cell Loading Surface: Inner Ring

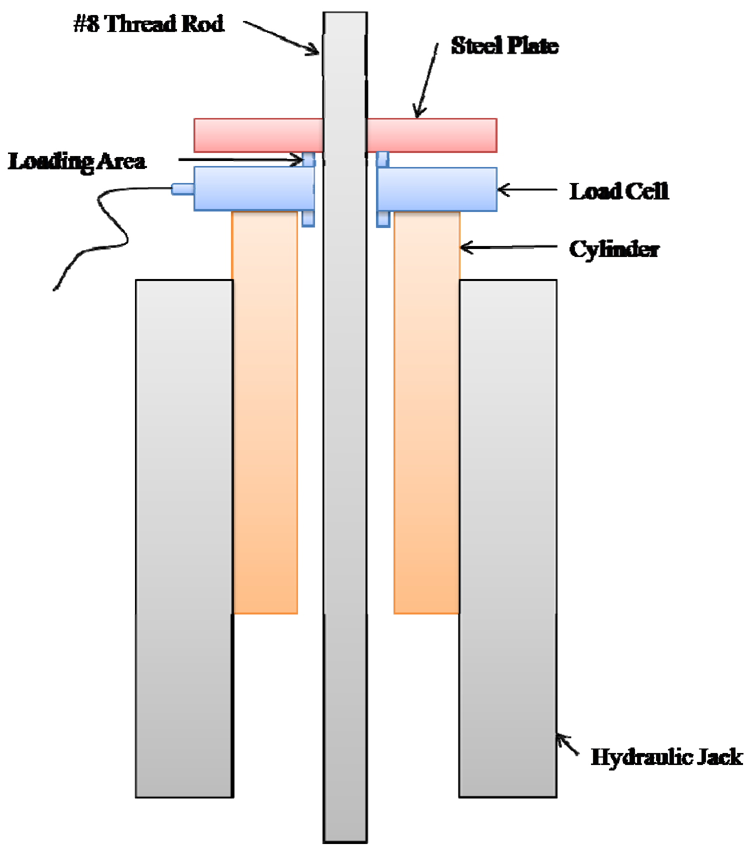


Figure 86 - Load Cell: Incorrect Installation

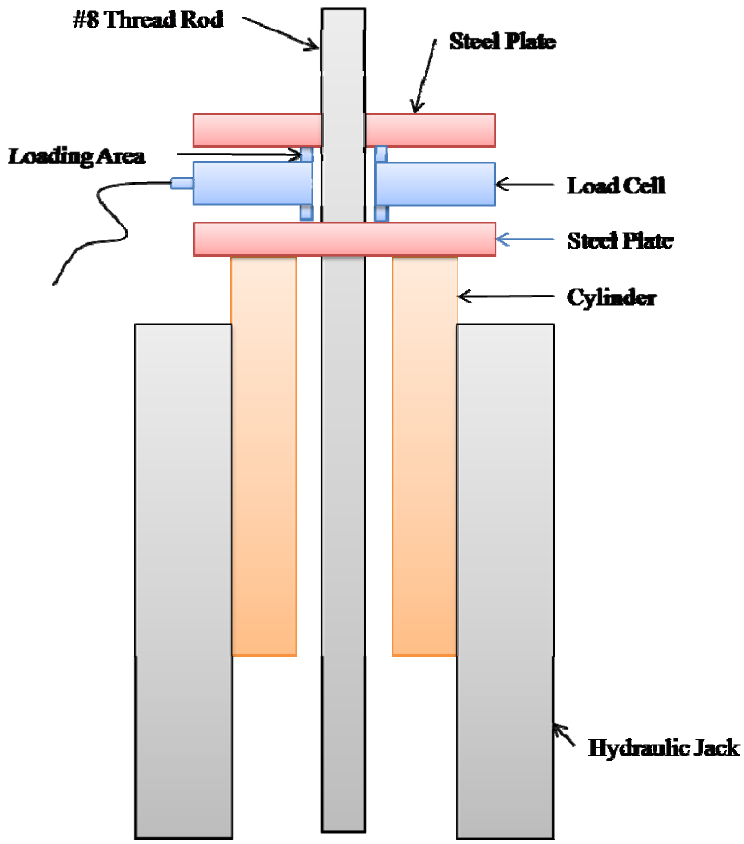


Figure 87 - Load Cell: Correct Installation

After sixteen tests, eight using the correct configuration and eight using the incorrect configuration (the configuration we used during the in-situ load tests), we found a correction factor of 2.27 that we used to correct the data collected. Table 4 shows the relation used to find that correction factor.

Test ID	Incorrect Configuration Single Plate		Correct Configuration Double Plate		Correction Factor
	Load Cell	Strain Gage	Load Cell	Strain Gage	
1	3,102.01	0.0006552	7,620.42	0.0006555	2.46
2	2,383.12	0.0004991	5,660.74	0.0005005	2.38
3	4,234.16	0.0008002	9,998.44	0.0008002	2.36
4	5,674.98	0.0010002	12,259.71	0.0010010	2.16
5	3,041.61	0.0006026	7,290.42	0.0006005	2.40
6	4,458.06	0.0008036	9,739.52	0.0008000	2.18
7	5,110.95	0.0008996	10,995.55	0.0009020	2.15
8	5,887.72	0.0010086	12,281.03	0.0010050	2.09
				Average	**2.27**

Table 4 - Correction Factor

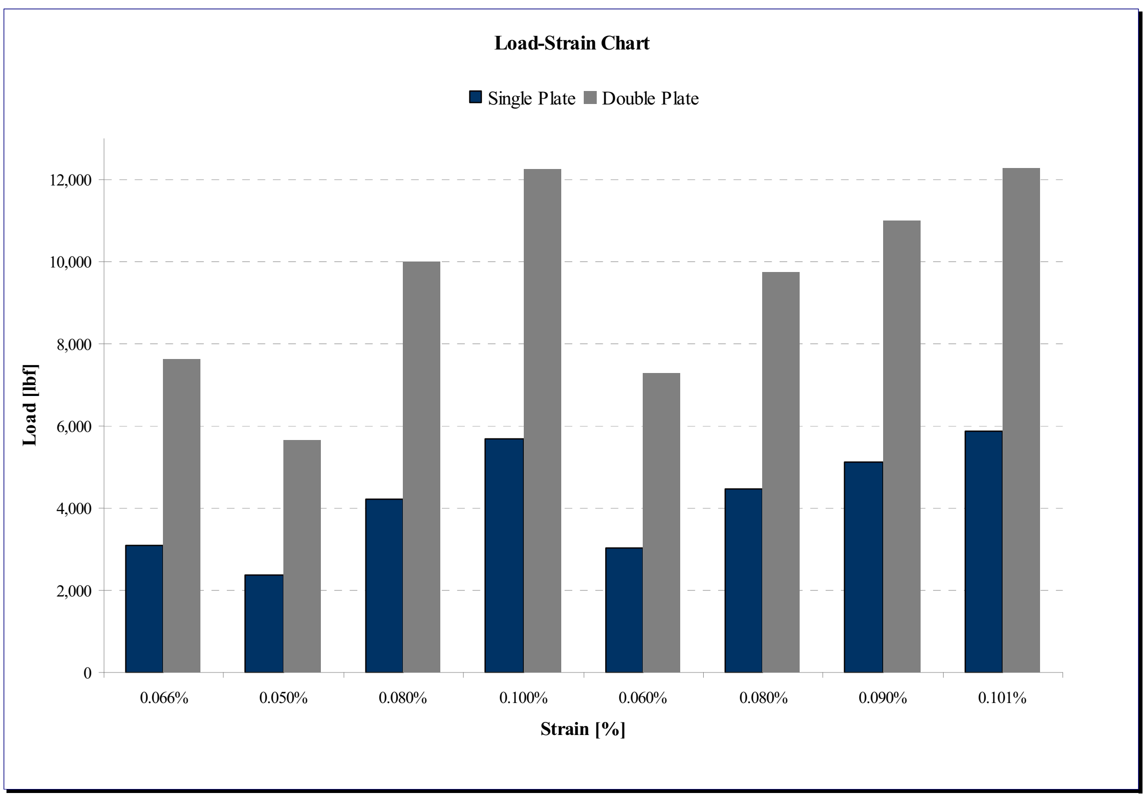


Figure 88 – Load-Strain Relation: Correction Factor

4.5 COMPARISON BETWEEN ANALYTICAL AND EXPERIMENTAL RESULTS

As mentioned before, the data collected was multiplied by a correction factor of 2.27 giving us the correct the data to compare to the analytical results.

4.5.1 SPECIMEN NO.1

Both analytical and experimental approach have been consisted in showing that the failure mode was yielding of the bottom chord at mid-span, with a minor difference in the maximum load applied to reach yielding. Table 5 shows that minor difference between analytical and experimental approach. As we were able to apply a balanced load along the specimen we did not see any buckling related failure, as expected from the analytical results.

Approach	**Load [kip]**	**Tensile Stress [ksi]**
Analytical	12.80	60
Experimental	11.96	60

Table 5 – Load-Stress Relation: Analytical and Experimental Results

4.5.2 SPECIMEN NO.2

As explained before, during the test of specimen no.2, an unexpected problem occurred with the jacks resulting in an unbalanced loading, which means one of the jacks was applying more pressure than the other one (Figure 90). Because of the high levels of internal forces buckling failure occurred on the diagonals bars (Figure 89).

Figure 89 – Buckling Failure

To take into consideration the unbalanced loading, we modeled the specimen, using Straus7, simulating the same conditions.

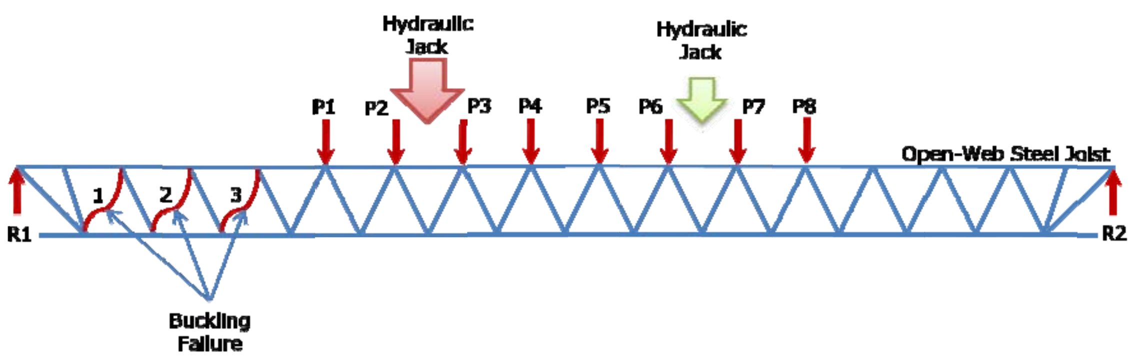


Figure 90 – Schematic of Buckling Failure: Specimen No.2

After analyzing the results from Straus7, local buckling was expected on the diagonals as a result of unbalance loading (Table 6).

The force to be applied on the diagonal members for buckling to occurred is given by Equation 13:

$$F = \frac{\pi^2 EI}{(KL)^2} \tag{13}$$

where,

F	=	Maximum or Critical Force
E	=	Modulus of Elasticity
I	=	Moment of Inertia
L	=	Unsupported Length
K	=	Effective Length (K=1.0 for pinned ends)

The critical force to be applied on the diagonal bar is (equation 14):

$$F = \frac{\pi^2 (29{,}000\text{ksi})(0.00931)}{(1.0 \times 22.1)^2} = 5.46\text{ kips} \tag{14}$$

Diagonal Bar No.	**Critical Force [kip]**	**Axial Force (Strauss7) [Kip]**
1	5.46	10.30
2	5.46	10.25
3	5.46	10.00

Table 6 – Force for Buckling to Occur on Diagonal Bars when Unbalanced Loading

In specimen no.2 local buckling on diagonal bars 1, 2 and 3 (Figure 90) occurred before yielding of the bottom chord, as a result of the unbalanced loading due to deficiencies of hydraulic jacks.

Chapter V

Summary and Conclusions

5.1 Summary

The behavior of steel joists under eight concentrated loads was investigated in this study. The research project was comprised of both experimental and analytical work.

The experimental work included the following:

1. Preparation of instrumentation, cabling and connections, by mean of selecting and purchasing the ideal sensors for this type of test, including load cells, DCVTs and strain gages; selecting and purchasing the ideal data acquisition system; preparing cabling and connections; and configuring the software to collect the data.
2. Design of setup: using steel beams to assure a system with a high stiffness to avoid any kind of influence on the behavior of the steel joist during the test.
3. Material property tests: steel samples and concrete cores were taken in order to know the tensile and compressive strength, respectively.
4. Flexural test: a total of two specimens were tested under eight concentrated loads.
5. This study examined only the static structural behavior of the specimens.

Test results from the present study, combined with collected data from the literature, were analytically evaluated with the following objectives:

1. To assess the ultimate structural performance of the floor system on a selected strip of the floor;
2. To evaluate the contribution of the concrete slab without shear studs on the floor system.-

5.2 CONCLUSIONS

The present study has resulted in a number of conclusions, as outlined below:

1. The stress analysis of open web joists at loads up to failure can be made with sufficient accuracy by assuming that the joists are pin-connected trusses.
2. Extended ends of the bottom chord, meant to support the false ceiling, after significant deflection, could touch the walls at both ends changing the boundary conditions and hence the behavior of the entire system.
3. The failure mode was affected by fabrication defects and/or differences on the geometry between the two specimens.

5.3 RECOMMENDATIONS

1. When using donut shaped load cells make sure that both top and bottom loading areas are in full contact with a rigid plate.
2. The author recommends further load tests of open web joists in order to understand better the behavior and ultimate capacity of this system. The load tests shall be made either by the application of a symmetrically distributed load upon two or more joists braced or by use of concentrated loads applied by means of calibrated testing apparatus upon one or more joists braced or laterally supported. The loads shall be arranged such that it does not provide load redistribution.

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