Virginia Polytechnic Institute and State University

The Charles E. Via, Jr. Department of Civil Engineering

STRUCTURAL ENGINEERING

INNOVATIVE FLOOR SYSTEMS FOR STEEL FRAMED BUILDINGS

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Submitted to

American Institute of Steel Construction
Chicago, IL

Report No. CE/VPI-ST 90/03

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Research Report

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CHAPTER I

INTRODUCTION

1.1 History of Floor Systems

Optimizing the use of building materials has always been one of the primary goals of engineers throughout history. It is a constant challenge to seek innovative methods to build lighter weight structures. Sometimes it is achieved through the development of new building materials, other times it can be accomplished by creating entirely new types of structural systems. Often lightweight structures can be more aesthetically pleasing because of their streamlined appearance. However, in general the motivating factor in building lightweight structures is to reduce the overall cost. One portion of a structure which offers tremendous potential for weight reduction is the floor system. The floor system has always been one of the heaviest components in typical steel framed buildings. A reduction in the dead load of this component may result in a subsequent reduction in the total weight of the building structural system. The objective of this investigation is to create or identify innovative lightweight floor systems that can effectively reduce the overall cost of steel framed building construction.

For many years, the most common type of floor system used in steel framed buildings was a four inch thick concrete slab with the supporting beams completely encased in concrete. On top of this was a four inch topping slab which also contained conduits and electrical wires [Dellaire 1971]. In all, these eight inch thick floor slabs accounted for a substantial portion of the dead load of the total structural system of a building.

One of the earliest methods of reducing floor system weight was the use of ceramic slabs. In this type of floor system the supporting beams were spaced relatively close, i.e. 3 to 8 ft. Long cellular ceramic blocks were then placed parallel to and resting on the bottom flanges of the supporting members. The elements were made monolithic by a cast in place concrete slab [ASCE 1980]. These systems were popular in the early 1900's for use in multi-story buildings.

One of the pioneers in the field of ceramic slabs was the National Fireproofing Company. This company manufactured a number of different floor systems using terra cotta cellular blocks. The block were generally wedge shaped or had sloped sides such that when a series of blocks were placed between the supporting beams, they formed either a flat or curved arch, Figure 1.1. These systems were developed not only for their lighter weight, but also for their excellent fire proofing, and sound proofing qualities. The blocks generally weighed from 20 psf for a six inch depth to 50 psf for a fifteen inch deep block [Sweet's Catalog 1906].

A number of other innovative floor systems were also invented during the early 1900's. The Columbian System, Figure 1.2, developed by Columbian Fireproofing Company, New York, NY, consisted of ribbed steel bars spanning between girders. The bars were either framed into the girders by angles or suspended from the girders in stirrups resting on the top flanges. The bars, as well as the girders, were then completely embedded in the cinder, slag or stone concrete. The concrete and steel ribs acted compositely in resisting loads. It was

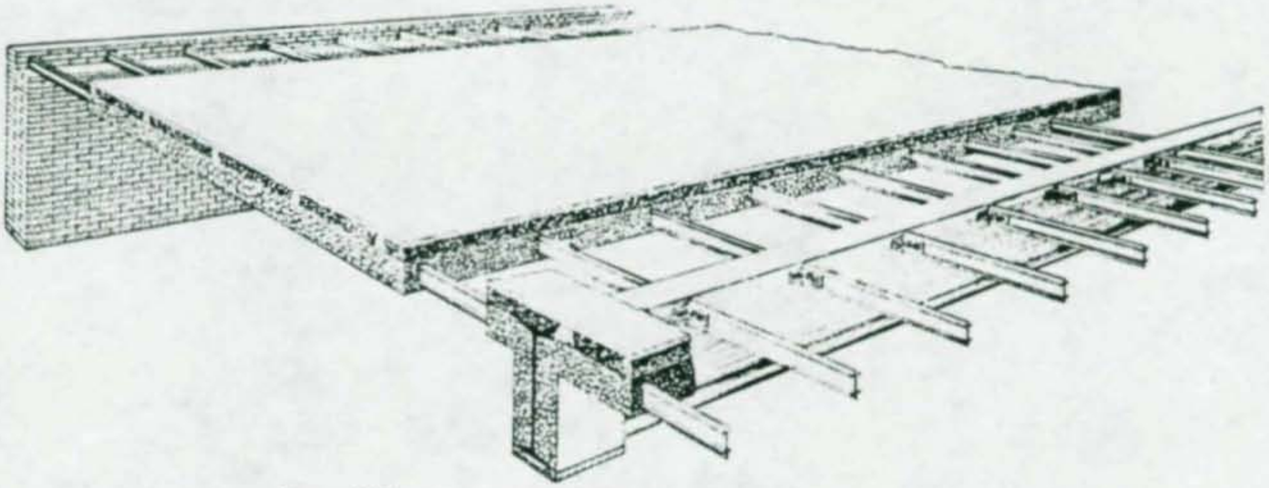


Fig. 1. VIEW OF FLOOR WITH HEAVY BARS FRAMED TO GIRDERS

This plan of connecting bars adapted to long spans and large bars

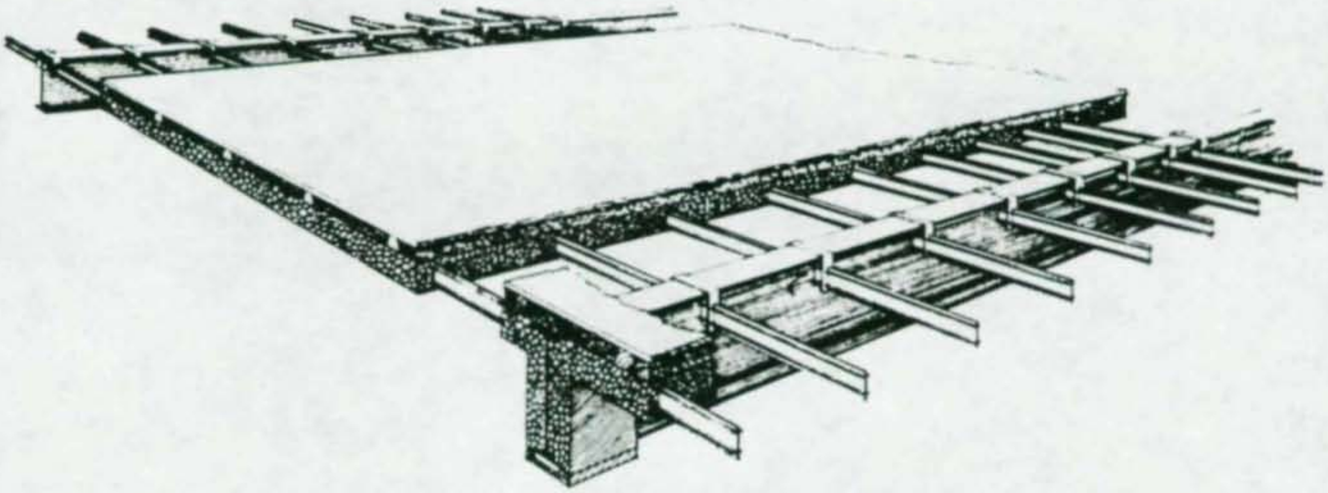


Fig. 2. VIEW OF FLOOR WITH HEAVY BARS SUSPENDED IN STIRRUPS

This plan of connecting bars adapted to short spans and small bars

Figure 1.2 Columbian System Floor Construction (Sweet's 1906)

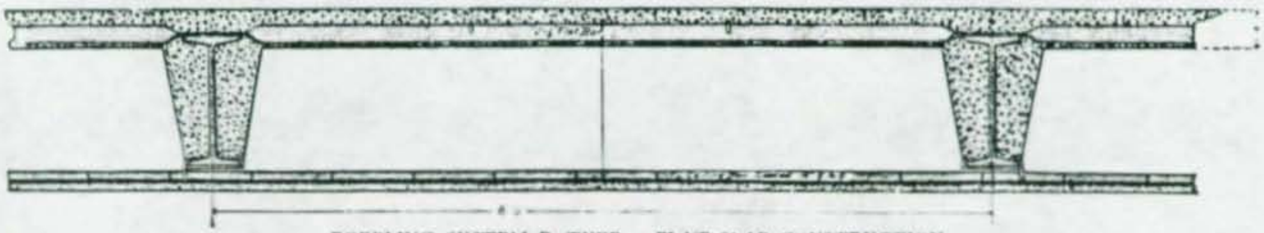
also believed at the time that the concrete completely insulated the steel ribs making the system impervious to fire.

Another floor concept similar to the Columbian System was the Roebling System B developed by Roebling Construction Co., Figure 1.3. The main difference between these two systems is that in the Roebling floor the steel ribs consist of flat bars with a 1/4 turn at each end. The bars rested on the top flanges of the girders. The bars and steel girders of this system were also totally encased in concrete.

Both the Roebling System B and the Columbian System were somewhat of a variation on the reinforced concrete slab concept. Each system was capable of spanning 16 to 20 ft, depending on the depth and spacing of the ribs. Some of the advantages of these floors were:

- Absolute protection from rust
- Rapidity of erection
- Saving in story height due to thin floor
- Saving in first cost of completed structure
- Level ceiling between girders
- Increased stiffness

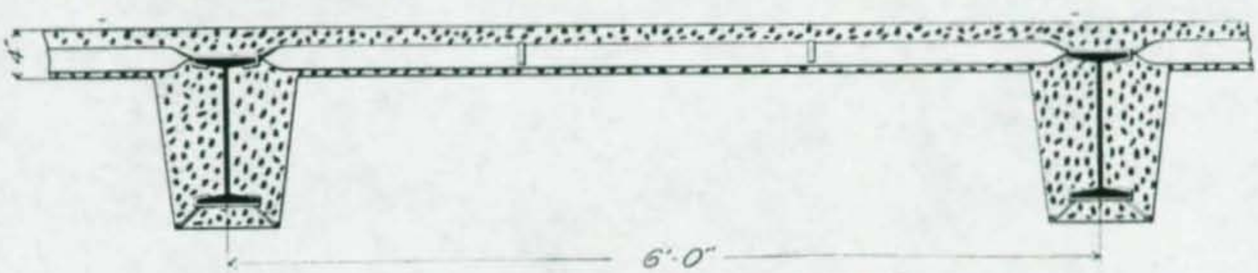
Two other floor systems advertised by the Roebling Construction Co. were the Roebling System A and the Rapp Fireproof Floor System, Figure 1.4. Both of these systems were constructed from a wire cloth arch resting between the girders. The arch is stiffened by steel rods in System A and steel tee ribs in the Rapp System. An additional characteristic of the Rapp System is that bricks were incorporated into the arch by placing them between the tee ribs. A cinder



ROEBLING SYSTEM B, TYPE 1, FLAT SLAB CONSTRUCTION
Adapted for Public Buildings, Offices, Theatres, Churches, Schools, Hospitals, Hotels, Residences, etc.
Capacity in 6 ft. spacing of beams, 1500 lbs. per square foot

TABLE OF WEIGHTS, SPACING OF BEAMS, ETC. ROEBLING SYSTEM B.

Type of construction	Spacing of beams	Depth of beams	Thickness of concrete	Weight per sq. ft. of concrete imbedded iron and wire	Weight of ceiling including plaster	No. of coats of plaster required
Type 1	8′ 0″	10″	4″	30 lbs.	10 lbs.	3
" 2	5′ 0″	10″	4″	35 lbs.	9 lbs.	2
" 3	7′ 0″	15″	4″	38 lbs.	10 lbs.	3
" 4	6′ 0″	8″	4″	28 lbs.	7 lbs.	2
" 5	up to 16′	15″ to 20″	5 1-2″	45 lbs.	7 lbs.	2



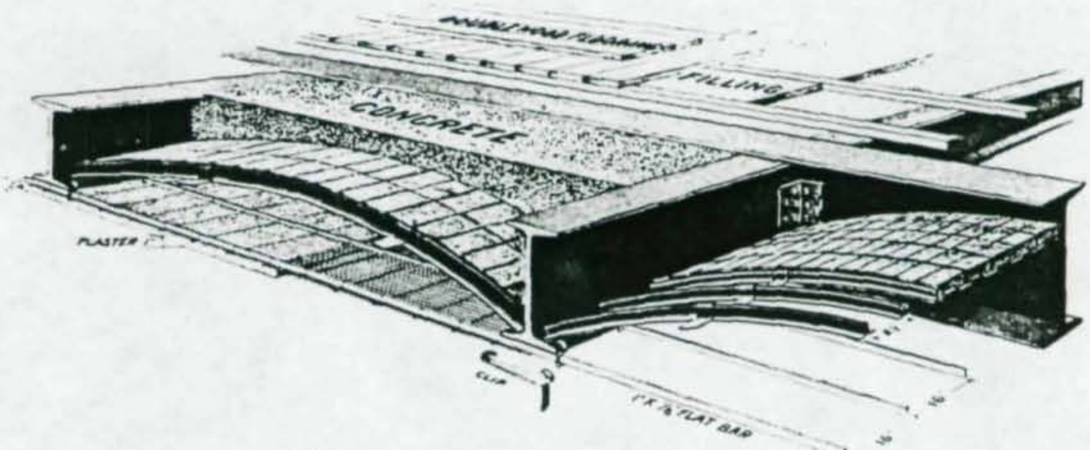
ROEBLING SYSTEM B, TYPE 2, PANELED CEILING
Adapted for Stores, Warehouses, Depots, Factories, etc.

Figure 1.3 Roebling System B Floors (Sweet's 1906)

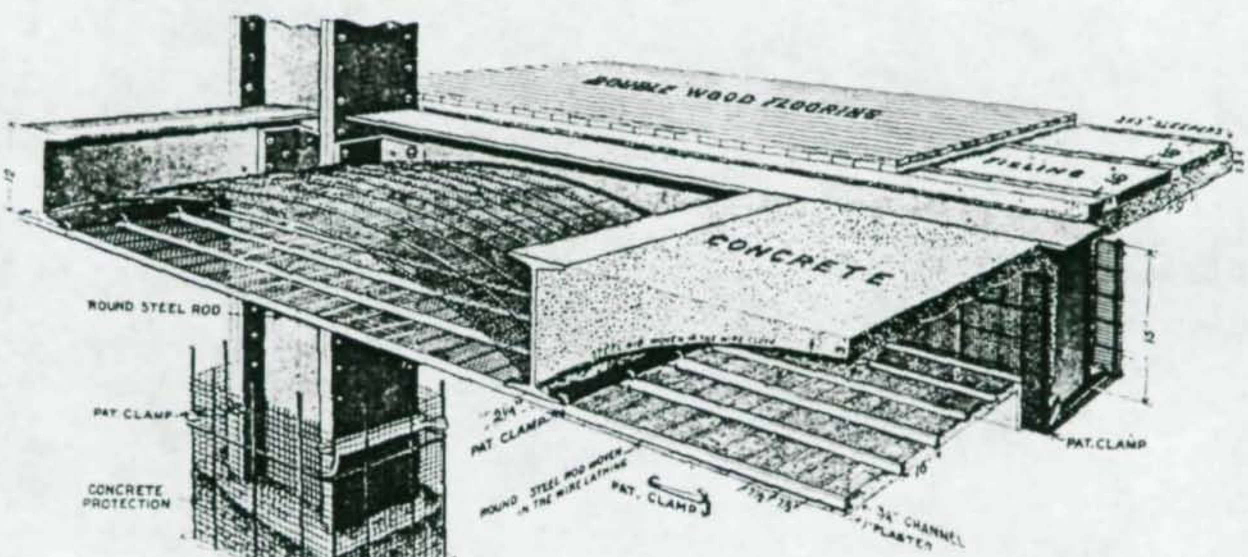
THE RAPP FIREPROOF FLOOR

THE RAPP FIREPROOF FLOOR SYSTEM, TYPE A—(Brick laid flat side down). With flat wire ceiling construction. Adapted for public buildings, offices, theatres, schools, churches, banks, libraries, hospitals, residences, etc.

Capacity in 10 ft. spacing of beams, 3,000 lbs. per sq. ft.



THE RAPP FIREPROOF FLOOR SYSTEM, TYPE A
(Brick laid flat side down)



ROEBLING SYSTEM A, TYPE 1, ARCH CONSTRUCTION.
Capacity in 6 ft. spacing of beams, 4,000 lbs. per square foot

Figure 1.4 Roebling System A and Rapp Fireproof Floor System (Sweet's 1906)

concrete fill was placed over the arch, flush with the tops of the beams. These systems were generally topped off with double wood plank flooring.

In all of the above floor systems: ceramic slabs, the Columbian system, the Roebling and the Rapp systems, the motivating factor in the development was fireproofing qualities. This was mainly fostered by the need to conform to the rules of the National Board of Fire Underwriters. However it is apparent that each company sought to make the systems as lightweight as possible in addition to obtaining the maximum fire rating.

Despite all the advantages of these innovative floor systems, they eventually fell into disuse. One of the primary reasons for their demise was the dwindling supply of cinders used to make the lightweight concrete fill. Other factors contributing to their downfall include increased labor costs and the onset of steel deck floor systems [Dellaire 1971]. Although none of these floor systems have been constructed for decades, their inventors should be admired for their imagination and creativity. Einstein was once quoted as saying "Imagination is more important than knowledge". It is entirely possible that with some modifications, these older systems may still have potential in floor systems today.

In the early 1920's engineers began to ask the question "...why should the weight of a floor system be so much greater than the live load it's designed to carry" [Dellaire 1971]. As a result it was in this time period that the first cellular steel floor was used in a Baltimore & Ohio Railroad Co. warehouse in Pittsburgh, PA. This cellular floor system was referred to as the "keystone beam", manufactured by the H.H. Robertson Co.. In these early steel deck floors, the steel deck was the only load-carrying structural element. The concrete slab was only used to provide a level surface and to obtain an adequate fire rating. It was not until around 1950 that Granco Steel Products began welding a wire mesh to the trapezoidal steel deck such that the concrete slab would act compositely with the steel deck [Dellaire 1971].

As the use of cold-formed steel deck increased, further improvements were made. In the 1960's deck manufacturers began to produce decking with embossments and depressions to provide a better bond for the concrete, Figure 1.5. This also facilitated the use of thinner gage steel for the decks. One of the most significant advances in the use of steel decks was the development of composite beam design in the late 1960's and early 1970's. Here composite action is developed between the concrete slab and the supporting beams by welding steel shear connectors through the deck to the beams, Figure 1.6. This composite beam action made it possible for design engineers to reduce the weight of steel beams in the floor systems by as much as 30% [Dellaire 1971].

Today, the most common types of floor systems used in steel framed buildings in the United States incorporate the use of cold-formed steel deck and concrete slabs, with or without composite beam action. In addition to their lighter weight these floor systems also have the following advantages [Dellaire 1971]:

- Speed initial construction
- Eliminate costly wood forms and shoring
- Provide flexibility in accommodating electrical and communications services
- Are constructed from readily available materials

Although profiled steel deck and concrete floors provide a lighter weight alternative to the thick concrete slabs of earlier years, little research has been conducted on developing new floor systems that could result in greater dead load

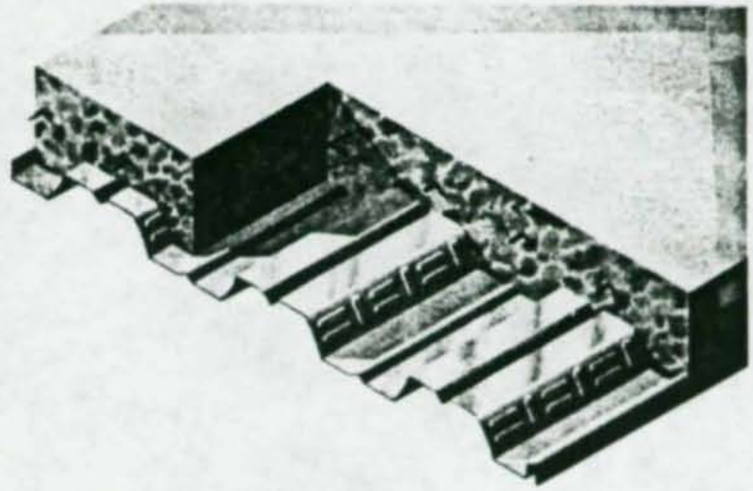


Figure 1.5 Composite Steel Deck/Concrete Slab

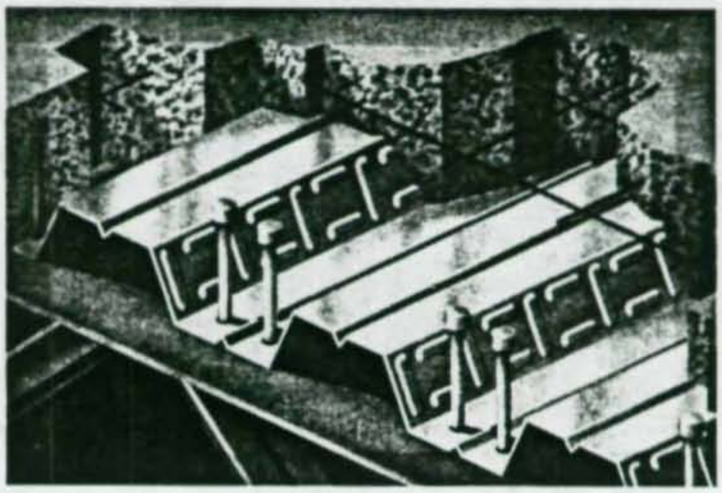


Figure 1.6 Typical Composite Beam Design

reductions. Subsequently this investigation is concerned with developing innovative alternatives for lightweight floor systems for use in steel framed buildings. However the materials in these systems are not limited to concrete and steel.

Floor systems used in steel framed buildings are generally composed of two distinct components, the framing members and the slab. An effort has been made to identify innovative concepts for both components. A few of the systems investigated rely on composite action between the frame and slab components to meet the various performance requirements. Not all of the proposed systems were investigated to the same extent. Some of the concepts would be difficult to develop analytically without experimental data. Discussions of the characteristics of the various innovative systems are presented in the subsequent chapters of this report.

1.2 Reference Floor System

As a basis of comparison for the innovative light-weight floors investigated, a series of reference floor systems were designed. In light of the advantages and the popularity of cold-formed steel decking, this configuration was chosen for all of the reference floor designs. A total of thirteen different systems were designed to allow a broader basis of comparison.

These floor systems were designed using both hot-rolled shapes and open web steel joists. All of the systems use normal weight concrete, 145 lb/cubic ft, placed on cold-formed steel decking. The steel deck is placed with the ribs running perpendicular to the supporting beams or joists. In addition, two of the systems using hot-rolled beams were designed using composite beam action in accordance with the <u>LRFD Design Specification for Structural Steel Buildings</u> [AISC 1986].

The design data used and the weights of the individual system are shown in Table 1.1. All of the floor systems were designed as a single 30 ft. by 30 ft. bay, taking into consideration both strength and serviceability limit states. The serviceability limit states included checking the live load deflections, as well as, the susceptibility of the floor system to annoying vibrations induced by human occupancy.

The vibration characteristics of the floor system were analyzed using the perceptibility criterion developed by Murray [1981]. This is done by using the following inequality ($D > 35\ A_o\ f_1 + 2.5$), where D = required damping, A_o = maximum initial amplitude of the floor system due to a heel-drop impact, and f_1 = first natural frequency of the floor system. For use in the mathematical model, the heel-drop impact is approximated by a linear decreasing ramp function having a magnitude of 600 lbs and a duration of 50 milliseconds. Based on the inequality developed by Murray, if the required damping is significantly more than 4%, then artificial damping may be necessary to make the floor system less susceptible to annoying vibrations. The required damping for each of the thirteen reference floor systems is also contained in Table 1.1.

Table 1.1 REFERENCE FLOOR SYSTEM DATA

SYSTEM	BEAM OR JOIST	S/t_s (ft)/(in)	WEIGHT TOTAL kips	WEIGHT %SLAB	VIBRATION RESPONSE f_1 (Hz)	A_o (in)	D_{REQD} (%)
RFS1C	W14x22	7.5/3.50	34.80	85.3	4.52	0.0161	5.05
RFS2C	W14x22	7.5/4.00	37.50	86.4	4.53	0.0146	4.81
RFS3N	W16x31	7.5/3.50	36.15	82.2	5.73	0.0150	5.52
RFS4N	W18x35	7.5/3.50	36.75	80.8	6.52	0.0140	5.70
RFS5N	W14x26	5.0/3.50	43.26	83.2	5.25	0.0130	4.88
RFS6N	W18x35	7.5/3.50	43.05	83.6	6.24	0.0113	4.96
RFS7N	W18x40	10.0/3.50	42.60	84.5	5.82	0.0136	5.28
RFS8J	18K4	2.0/3.25	34.96	85.0	5.64	0.0153	5.52
RFS9J	22K4	2.5/3.25	34.62	85.8	6.34	0.0150	5.84
RFS10J	22K6	3.0/3.25	34.54	86.0	6.37	0.0125	5.28
RFS11J	18K4	2.0/3.50	41.26	87.3	5.41	0.0111	4.60
RFS12J	20K5	2.5/3.50	41.00	87.8	5.76	0.0106	4.64
RFS13J	20K7	3.0/3.50	40.87	88.1	5.77	0.0124	5.00
RFSAVE			38.57	85.1	5.68	0.0134	5.16

RFS C - A36 Steel Composite Beam Design
RFS N - A36 Steel Non-Composite Beam Design
RFS J - Grade 50 Open-Web Steel Joists
S - Beam or Joist Spacing (ft)
t_s - Total Slab Thickness

All of the reference floor systems were designed using a superimposed live load of 70 psf as well as 8 psf for ceiling and mechanical loads. All the systems were designed using 3000 psi normal weight concrete. The vibration analysis for each system was performed considering dead load plus a superimposed live load of 11 psf.

In general the total weights of the reference floor systems ranged from 38.4 psf to 48.1 psf, with an average unit weight of 42.8 psf. One apparent characteristic of all of the floor systems is that 80 to 88 percent of the total weight can be attributed to the concrete slab. As a result it appears that the most significant weight reduction can be achieved by a reduction in the slab weight.

Although lightweight concrete could be used to reduce the weight of the reference floors, normal weight concrete has been specified because it is generally more available and less expensive.

As a final note, the same design loads and requirements used for the reference floor systems have also been applied to the innovative systems investigated.

CHAPTER II

CONCEPTUAL INVESTIGATION

2.1 Framing System

The first component of the floor system considered is the framing system. The most common type of structural member used for framing is the standard hot-rolled steel section. These are usually designed as simply supported beams arranged in orthogonal grids. Little change has occurred in this method of framing over the years for the following reasons: the manufacturing processes for the rolled sections are well established throughout the world, and these framing systems are easy to design, as well as erect. However new methods of structural framing are constantly emerging. The following is a discussion of some of the newer developments that may have potential for the framing of lightweight floor systems.

2.1.1 Open-Web Steel Joists

One of the more successful developments in the area of lightweight framing is the open-web steel joist, Figure 2.1a. These are usually manufactured using double angles for the top and bottom chords, with a web made of solid round or square bars. The webs can also be made of double angles or single angles crimped at the ends. The chords and webs are welded together into a Warren truss configuration. The design of these joists is governed by the load

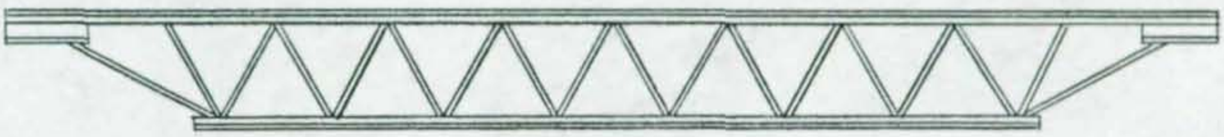


Figure 2.1a Open-Web Steel Joist

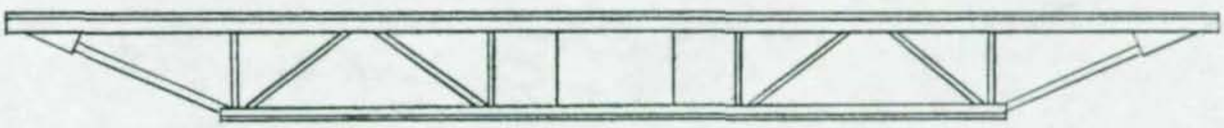


Figure 2.1b Vierendeel Lattice Girder

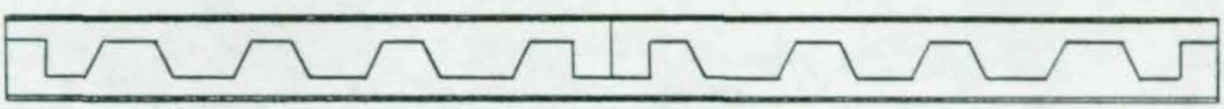
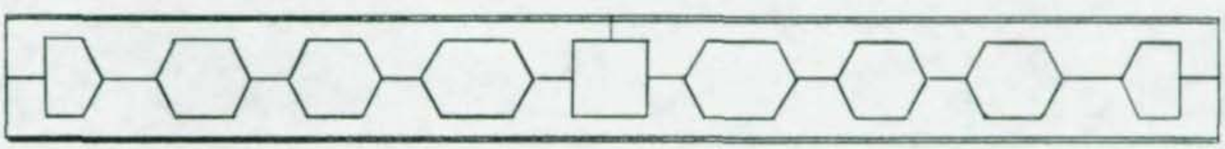


Figure 2.1c Castellated Beam



tables specified by the Steel Joist Institute. Optimally they are designed as simply supported beams carrying a uniformly distributed load.

Some of the advantages of steel joists are that they are easy to select, readily available, relatively lightweight and inexpensive compared to the cost per unit weight of hot-rolled sections. However steel joists are also subject to the following shortcomings [Galambos 1987]:

- They are designed optimally for only one loading case
- Out of plane slenderness is so small that they cannot support their own weight without lateral bracing
- They are very sensitive to joint eccentricities which can result during manufacturing
- They are sensitive to ponding, wind uplift and floor vibrations.

Steel joists have been around for many years and constitute one of the few economical and successful alternatives to standard hot-rolled shapes for floor system framing.

2.1.2 Lightweight Built-Up Sections

In addition to open web steel joists several other structural forms have been developed in recent years as a substitute for standard hot-rolled sections. Many of these have resulted from the demand for longer spans and to better facilitate placement of mechanical and electrical systems [Owens 1987]. Others have resulted from advances made in automated fabricating. Some examples of this are light plate girders and tapered beams. By using these types of sections, the engineer has greater freedom over parameters such as flange sizes, web thickness, depth and variation in depth [Owens 1987].

Plate girders are generally used in bridge construction when it is not economical to use a hot-rolled section, but seldom are they used in building construction. One area of building construction in which they have been successful is in pre-engineered steel buildings. Despite the extra expense in fabrication, light plate girders can still prove to be more economical in some applications due to their lighter weight and the lower cost of plate materials versus hot-rolled shapes [Owens 1987].

2.1.3 Vierendeel Lattice Girders

Another alternative that can be used as a framing component is the vierendeel lattice girder, Figure 2.1b. These resemble open-web steel joists except that a vierendeel panel is incorporated at the center allowing a shallower truss than the joists. Both joists and vierendeel lattice girders lend themselves well to accommodating service ducts. However the moment resisting panels in the vierendeel lattice girders make them heavier and more expensive to manufacture [Owen 1987]. The void left between the panels also makes it necessary to use larger angles for the top and bottom chords.

2.1.4 Castellated Beams

One interesting alternative to standard structural shapes is the castellated beam, Figure 2.1c. Castellated sections are beams with holes cut out of the webs. One of the more efficient ways of doing this is to take a standard hot-rolled section and cut a pre-described pattern across the web. The only requirement is that the pattern be of constant depth and antisymmetry about the axis of rotation.

The top half of the section is then cut at the centerline. The two top sections are then lifted and rotated 180 degrees. Once rotated the two top sections are welded back to the bottom half of the beam along the web. This results in a deeper beam of exactly the same weight as the original hot-rolled section. Another advantage is that the openings in the webs may accommodate service requirements. The size of the openings can be varied, which may be necessary to improve shear capacity at the beam ends [Owens 1987]. Despite the advantages, castellated sections are more costly to fabricate and subsequently have had limited use.

2.1.5 Stub Girder Floor Systems

The stub girder floor is a framing system which was developed in the early 1970's by J.P. Colaco to "...better utilize structural materials and to more efficiently integrate the electrical, mechanical, and structural systems within the building [Colaco 1972].". The stub girder, Figure 2.2, is constructed of a simply supported hot-rolled section, with short pieces of another steel section welded to the top flange. These short sections are called "stubs". Through the openings between the stubs it is possible to run the transverse floor beams continuously across the bottom half of the stub-girder. These secondary beams must be the same depth as the "stubs". Shear studs are then welded to the top flanges of the stubs, and a composite slab is poured in place.

The most commonly used sections for both the stub girders and the transverse beams are wide flange sections in the range of W12's to W16's. This type of framing is most economical for spans on the order of 35 to 50 feet. It is generally recommended that the stub girders be designed as simply supported beams, although continuous framing is possible [Bjorhovde 1985].

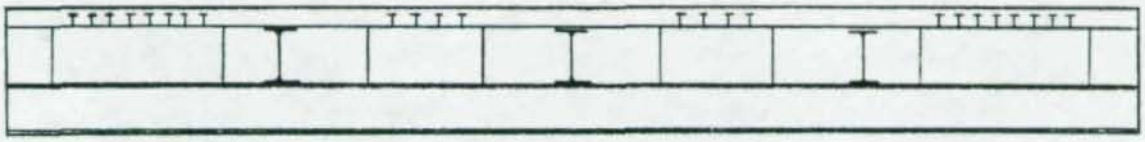
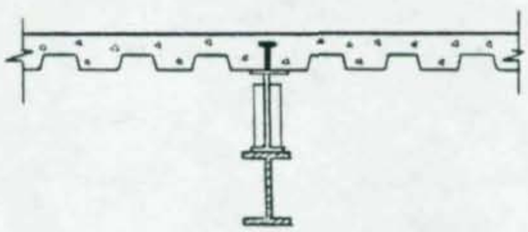


Figure 2.2 Stub Girder



CROSS SECTION

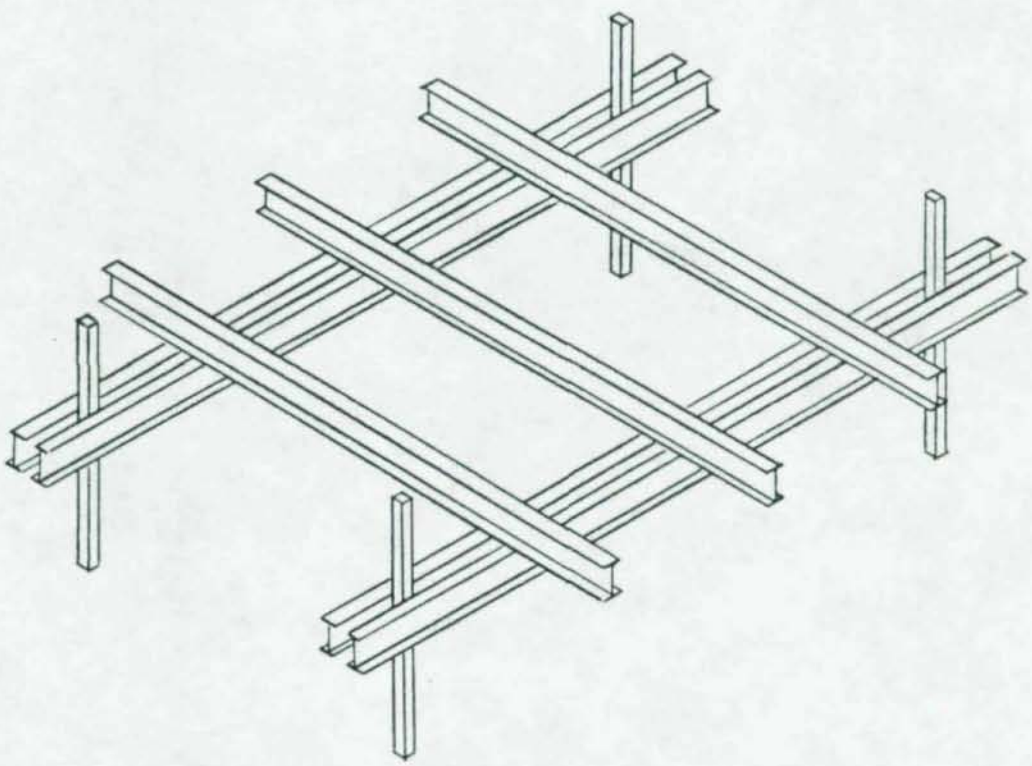


Figure 2.3 Dual Plane Grillage Framing System

The major advantage of the stub girder framing system is that the openings in the girders can accommodate service ducts and subsequently reduce the floor to floor height of the building [Nadaskay and Buckner 1985]. One of the disadvantages of this system is that it must be constructed using shored construction. This is necessary to prevent large deflections during erection procedures. However, service load deflections are usually not a problem since the design of stub girder systems is almost always controlled by strength rather than stiffness [Bjorhovde 1985].

A number of buildings have been constructed in recent years using this framing system. Although it may not be an economical alternative for all applications, according to Colaco, the use of a stub girder floor system can result in a savings of fifteen percent over the cost of conventional framing systems [Colaco 1972]. As a final note the stub girder framing system is already the subject of a design guide published by The American Institute of Steel Construction [Bjorhovde 1984].

2.1.6 Dual Plane Grillages

One method of continuous construction which warrants discussion is the dual plane grillage, Figure 2.3. In this system, twin sections are used for the primary beams so that they can bypass the supporting columns. The secondary beams then span across the tops of the primary beams, similar to the transverse beams in the stub girder system. As a result, continuity of the beams is achieved in both directions. A composite floor slab is then placed across the secondary beams. Furthermore the twin primary beams can be braced together.

This type of continuous framing offers several advantages such as a reduction in the bending moments and deflections, and a reduction in the number of beams to be handled and erected. This system also makes it possible to accommodate mechanical and electrical services in both directions. Like the stub girder framing system, this creates a potential for reducing the floor to floor height of the building. Alignment in construction is also less critical due to simplified connections thereby speeding erection [Owens 1987]. Some of the negative aspects of continuous construction are that the design is more complicated than for simply supported beams. Continuous construction can also result in the use of more slender members in which case lateral torsional buckling requires careful attention [Brett et.al. 1987].

2.1.7 Cold-Formed Steel Sections

In addition to profiled steel deck, cold-formed structural components can also be used as framing members. Some of the more commonly used cold-formed framing members used include channels, Z-purlins, angles, hat sections, T-sections, and tubular members. Cold-formed members can be manufactured in a variety sizes and forms. They can be used as secondary structural members or they can compose the entire structural system of a building. They can also be used as chord and web members in open web steel joists, space frames, trusses and arches. Some benefits of cold-formed steel members are that they are generally light-weight, have high strength and stiffness characteristics as well being easy to mass produce [Yu 1985].

2.1.8 Folded Trusses

One unusual application of cold-formed steel is in the construction of folded truss systems [Dannemann 1984]. Folded trusses have evolved as a somewhat obscure alternative to three-dimensional space trusses. In typical space trusses the axial members are connected together in groups of two or more at space nodes or hubs. The geometry control of these nodes is very important. The structural behavior of these trusses can be very sensitive to imperfections resulting from errors in either the manufacturing or erection processes. Furthermore the erection of these systems can be labor intensive. The folded truss system is much less susceptible to joint eccentricities than normal 3-D space trusses.

Folded trusses are characterized by the joining of two adjacent trusses at a biplaner joint. A biplaner joint results from the intersection of two planes in space. The behavior of these trusses is somewhat analogous to the concept of folded plates. As a result of the geometry, the adjacent trusses act in bracing each other against lateral translation. Thus no extra bracing is required as in the erection of typical open-web steel joists.

Several methods of constructing folded trusses from cold-formed steel members have been proposed [Dannemann 1984]. One method is to form a single section that can function as the top or bottom chord for two adjacent webs, Figure 2.4a. Another possibility is to fabricate two plane trusses separately and then join them at either the top or bottom chords during erection, creating double member chords, Figure 2.4b. Tubular cold-formed sections can also be incorporated into the trusses as web members.

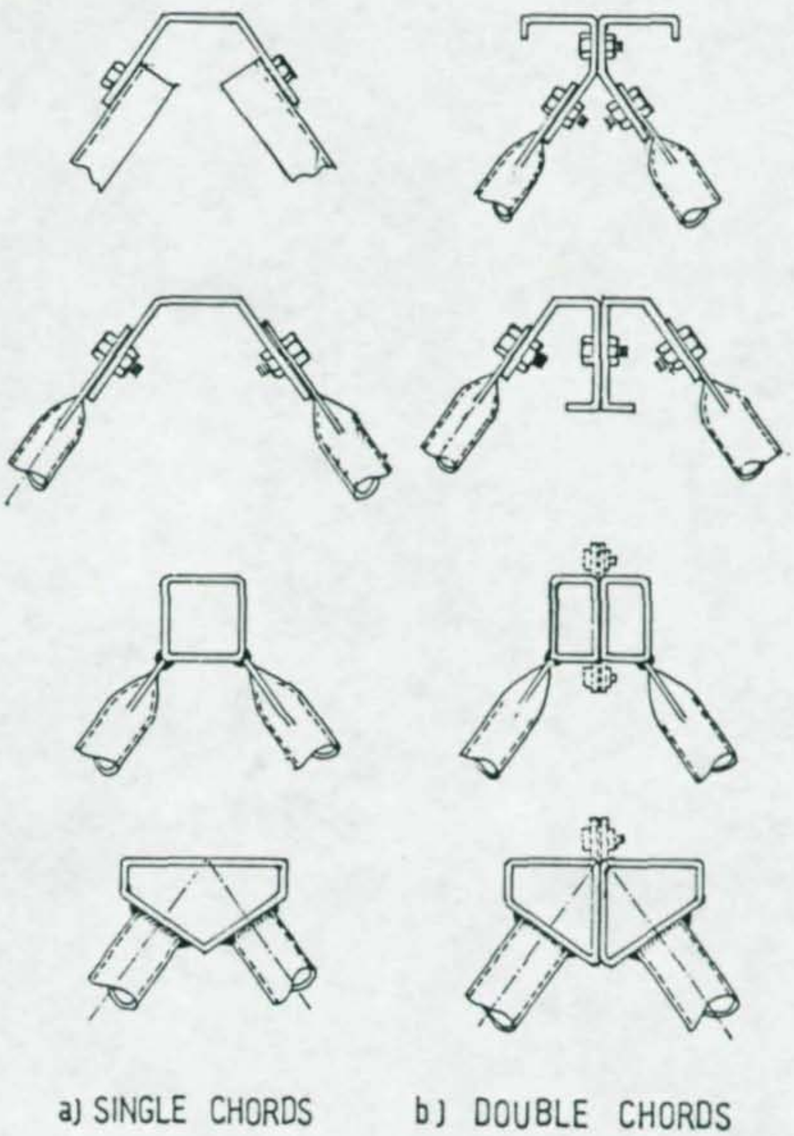


Figure 2.4 **Cold-Formed Steel Chords For Folded Truss Systems (Dannemann 1984)**

One example of a folded truss system is the Delta Joist System [Butler Manufacturing 1987]. This system was originally developed as a roofing system for buildings with masonry or concrete walls and a standing seam roof. Because a standing seam roof offers very little shear resistance to a building system, the framing must be designed to act as a diaghram to resist lateral loads. The Delta Joist System consists of open-web bar joists fabricated from single angle chords with circular bar webs welded at the junction of the angles legs. The joists are placed at 45 degree angles and the chords of the adjacent joists are bolted together creating double angle chords, similar to regular bar joists. A series of angles are also connected perpendicular to the bottom chords of the joists. This contributes to a certain amount of two-dimensional bending action.

All of the information available on folded truss systems advocates their use as roofing systems. It is not known wether or not this type of framing has ever been used in a floor system. However, because of its "self bracing" characteristics it does appear to offer an economical framing alternative that has advantages over both three-dimensional space trusses and typical systems of parallel open-web joists.

2.1.9 Tetrahedral Frames

One unique method of framing which has just recently been invented is a modular frame with tetrahedral nodes and cubic symmetry constructed from "puckered" rings. The method used to construct these "tetrahedral frames" was developed at the University of California at Berkeley, by John Gilman. Once constructed the rings form a framework "...analogous with the crystal structure of diamond [Gilman 1988]". In general the tetrahedral geometry results in one of the strongest and stiffest structures found in nature [Gilman 1988].

Tetrahedral geometry is based on the fact that the least number of struts required to fix a point in three-dimensional space is four. Whereas most three-dimensional structures are constructed with six members framing into each node, the tetrahedral frame only has four members framing into each node. This results in a structure with high specific strength and stiffness thereby optimizing the use of the building materials.

Tetrahedral space trusses have been around for many years. The novelty of Gilman's tetrahedral frame is in the method of construction that he developed for these structures. The normal method of construction is to connect a set of equal length struts to nodal hubs. The method proposed by Gilman incorporates the use of puckered rings. These rings are constructed with six equal length sides. The angle between any two sides is the tetrahedral angle of 109 deg. 28 min. Four rings can be connected to form a tetroid unit, Figure 2.5a. These units can then be connected together to form the tetrahedral frame, Figure 2.5b. The components can be connected with adhesives, mechanical connectors, welds, etc., depending the types of building materials used [Gilman 1988].

Unlike a tetrahedral space truss, these frames are capable of withstanding bending and shear stresses as well as axial loads. In addition to their lightweight and high specific strength and stiffness, these frames have other advantages. For example, since all of the puckered rings are usually the same for any given frame, they could be mass produced. They could also be easily stacked together for shipping and storage. Some of the disadvantages of these frames are that they could be difficult to analyze depending on the types of materials used. It may also

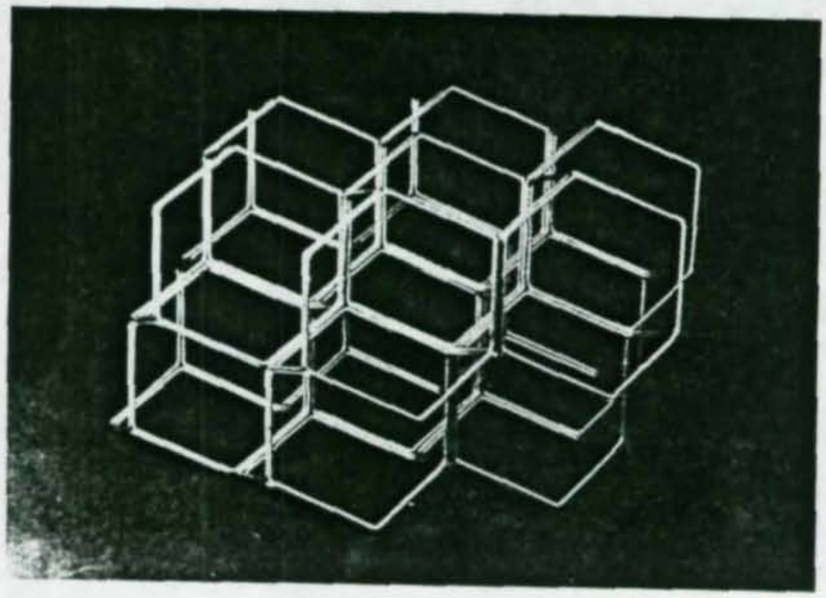


Figure 2.5a Four Puckered Rings Assembled Into Tetroid Unit (Gilman 1988)

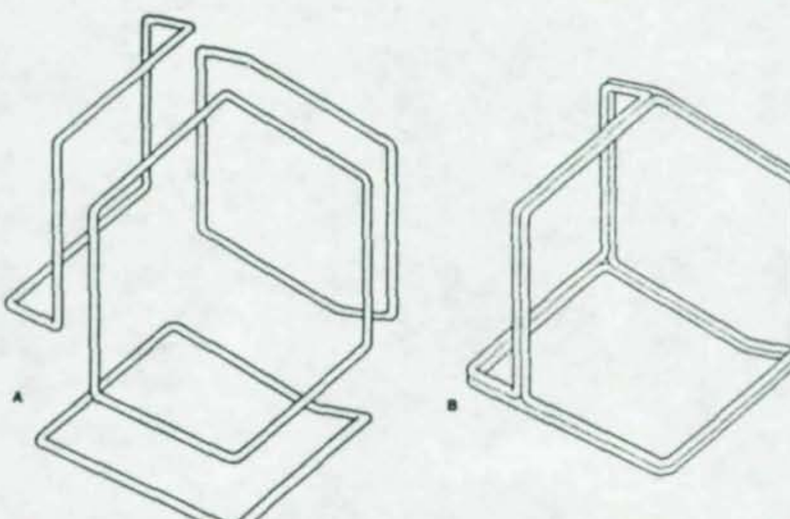


Figure 2.5b Model Of Tetrahedral Frame (Gilman 1988)

be difficult to establish acceptability within the building trades for such an unusual structural system.

Although the tetrahedral frames proposed by Gilman have not been incorporated into any known building structure to date, they could have tremendous potential. The key to their success in structural engineering applications depends wether or not they can be developed into a useful form. On one extreme the tetroid units could be used to form the skeletal structure for a multi-story building. On the other extreme it may be possible to manufacture a much smaller frame that could be used as reinforcement for a lower modulus matrix such as concrete. It may also have potential as a framing system for both roof and floor systems. Furthermore the frames could be manufactured from various types of building materials such as steel, concrete, fibre reinforced plastics, cold-formed steel, etc. The main objective of including tetrahedral frames in this report is to make other structural engineers aware of the existence of these systems and possibly stimulate some interest in developing some practical applications for building design and construction.

2.1.10 Summary

In the past, the framing system has generally comprised a substantial portion of a buildings total cost. Today however, the costs of other elements such as services and exterior cladding can equal or exceed the cost of the framing system. Because the cost of the frame has become a smaller percentage of the building cost, a lighter or less expensive frame may not result in the most economical solution with regard to the total building cost [Brett et.al. 1987]. Other factors that may govern the type of framing system used include how well the floor system can accommodate the service requirements. A substantial cost reduction may also result from a shallower framing system that can reduce the floor to floor height of the structure.

In addition to cost, there may be other factors that would make one framing system more advantageous over another. Some of these factors include serviceability requirements such as vibrations and deflections. In many instances fabrication and constructibility can also be a major concern. The required span length also plays an important part in determining what types of members would offer the most economical solution.

Given all these factors and the number of different alternative framing systems available it would be difficult to draw any definite conclusions as to which of these systems would have the most potential for use in a light-weight floor system.

2.2 Fibre Reinforced Plastics (Overview)

One material which has potential as a component in either the framing or deck system is fibre reinforced plastic or (FRP). FRP structural components have become widely used in the aerospace industry because of their high strength and light weight. They also offer excellent resistance to caustic environments. However their use in civil engineering applications has been very limited. One of the major reasons is cost. Another reason is the lack of design data and information [Green 1982]. In light of this fact a brief overview of FRP's and some of their civil engineering applications is warranted.

Fibre reinforced plastics belong to a group of structural components known as composite materials. A composite is essentially a combination of two or more materials acting together to resist a load. Composites have actually been utilized for centuries. A few examples are: the practice of placing straw in mud bricks, laminated iron-steel swords and gun barrels, particle board and concrete [Schwartz 1984]. Composite design is also used to refer to the interaction between a steel beam and concrete slab connected by shear studs. However for purposes of this chapter the term "composite materials" will refer mainly to fibre reinforced plastics.

Although there are hundreds, if not thousands of different composites manufactured in the world they all have a few characteristics in common. A brief summary of these characteristics is offered by [Cogswell 1988]:

> "Fibre reinforced composites contain four elements: the fibres, the matrix, the organization of the fibres in the matrix, and the interface between them. It is the reinforcing fibre which carries the load and so determines the stiffness and strength of the composite. The resin supports the fibre, particularly under compression loading, and is responsible for transferring load from one fibre to another. The resin also plays an essential role during fabrication since it is the medium by which elements of the structure are joined together to form a whole, and, in addition, it protects the relatively fragile fibres from abrasion."

Due to the orientation of the fibres, composites are by nature anisotropic, i.e. the strength and stiffness of the component varies with direction. This is elaborated on later.

The first major component introduced with regard to composite materials is the reinforcing fibres. As mentioned previously, the fibres generally determine the strength and stiffness characteristics of the composite. Fibres can take the form of whiskers, continuous fibres or bulk materials. "Fibres when incorporated into matrices, may range in length between 0.5 mm and several kilometers in continuously wound structures. They may be assembled into thin sheets in parallel orientation, or woven into fabrics with a variety of constructions and areal densities, or they may be used as parallel bundles or rovings [Bowen 1988]."

The most common types of reinforcing fibres used in fibre reinforced plastics are E-glass and S-glass. These are utilized primarily because of their low cost. Other high performance fibre reinforcements which can be utilized are: carbon fibre, boron fibre, aromatic polyamide (Kevlar), aromatic polyester, polyolefin (Spectra), silicon carbide, and alumina [Bowen 1988, Gosnell 1987]. Although these reinforcing materials are many times stronger and stiffer than glass fibres, the improvements in material properties are respectively apparent in their higher costs.

The second major component of composite materials introduced is the matrix. The matrix is essentially the binder that holds the fibrous reinforcing together. As such it must provide the transverse and shear strength to the composite material [DATA Inc. 1987]. The matrix for a fibre reinforced plastic usually consists of a thermosetting or thermoplastic resin. A resin as it pertains to FRP can be defined as follows: " A pseudosolid or solid organic material often of high molecular weight [DATA Inc. 1987]." The resins are generally isotropic.

The most common resins are the thermosetting polymers. These are plastics which are cured by applying heat. Once cured they are transformed into a substantially infusible and insoluble material [Schwartz 1984]. Some typical thermosetting resins include epoxy, phenolic, vinyl ester, and isophthalic polyester resins. One of the advantages of these resins is that they have a relatively low viscosity in the liquid stage which is beneficial in coating the fibrous reinforcement prior to curing [Cogswell 1988].

The other types of resins, which are used less frequently than thermosets in the manufacturing of structural plastics are thermoplastics. Some examples of thermoplastics include polyethylene (PE), nylon, polystyrene, polyvinyl chloride (PVC), and acrylics among others. Thermoplastic resins are characterized by their ability to be recycled by heating and cooling [Schwartz 1984]. Because of their linear chain molecular structure they have many advantages over thermosets. Some of these advantages are as follows [Cogswell 1988, Gosnell 1987]:

- No Refrigeration of prepregs
- Indefinite shelf life
- No lengthy curing required
- Recycle potential
- Cleaner production areas
- Higher impact strength

The use of thermoplastic resins is still somewhat in the developmental stages with respect to structural components. One thermoplastic resin presently being developed is polyether etherketone, or PEEK. One of the problems with thermoplastics is that they have a high viscosity in the liquid state. As a result the use of these resins has been described by the following analogy: "Technically the challenge of making thermoplastic matrix composites is equivalent to spreading a piece of chewing gum over the area of a small table [Cogswell 1988].".

One of the most common types of FRP used in civil engineering applications is the pultruded structural shape. This is a composite generally consisting of glass fibre reinforcing with a polyester or vinylester resin. These components are combined using the pultrusion process, Figure 2.6. This is a process where continuous glass rovings are pulled longitudinally through a resin bath where they are completely coated. Once coated they are pulled through a hot compaction die that cures the resin [Gosnell 1987].

Structural shapes manufactured by this process usually resemble hot-rolled steel sections. However there are significant differences in the material properties of FRP and steel sections. The main difference is that steel is usually assumed to be isotropic whereas the FRP sections are anisotropic. Consequently, even though the longitudinal flexural strength of the FRP is typically about 30 ksi, the transverse strength is considerably less. Likewise the transverse and shear moduli are significantly less than the flexural modulus of elasticity [Tepera 1982]. It should also be noted that FRP's generally display little or no yield at ultimate strength [Gosnell 1987].

Another major difference between steel and FRP is that the flexural modulus of elasticity is usually an order of magnitude less than that of steel. Because of this FRP beams will be more flexible in both flexure and torsion. Subsequently, deflections and member stability must be considered on a level equal to or higher than flexural stresses when designing FRP beams [Tepera 1982].

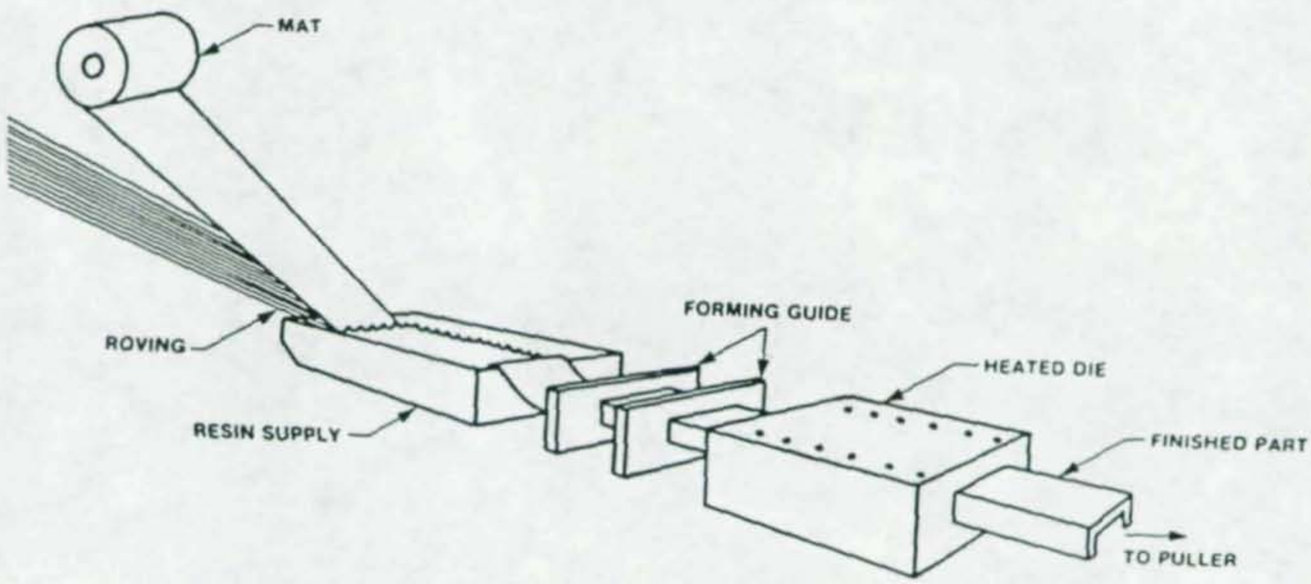


Figure 2.6 Fibre Reinforced Plastic (Pultrusion Process)

As stated earlier, pultruded FRP is one of the most common forms of composite structural components used for civil engineering applications. It is also one of the least expensive forms of FRP. There also exists a class of composites known as "advanced composites". Advanced composites refer to components manufactured using high performance resins and fibres [Gosnell 1987]. Consequently, these composites are considerably more expensive than a typical component manufactured from glass fibre reinforcing with a thermosetting resin. For example, using carbon fibres or aramid (Kevlar) fibres can result in a cost increase of 10 to 100 times the cost of a component manufactured with glass fibres [Bowen 1988].

Research is presently being conducted in advance composites at the United States Army Fort Belvoir, Development and Engineering Center. In conjunction with this investigation the Fort Belvoir Research Center was contacted to obtain information regarding the use of composites in military applications. The following is a brief synopsis of the work being done with advanced composites for use in military bridges [Kominos 1989]:

> Currently military bridge decks are manufactured primarily of aluminum. In addition, the deck is the heaviest component in the bridge. To combat this problem the US Army has developed two alternative decks using thermoplastic composites which result in a 30 to 40 percent weight reduction in the deck. One of the composite deck designs is a sandwich construction of graphite and glass reinforced Nylon-12 face sheet with a roahcell foam core, Figure 2.7. The other design option is a multi-hollow box beam made from a woven graphite/ultem (polyetherimide) prepreg.

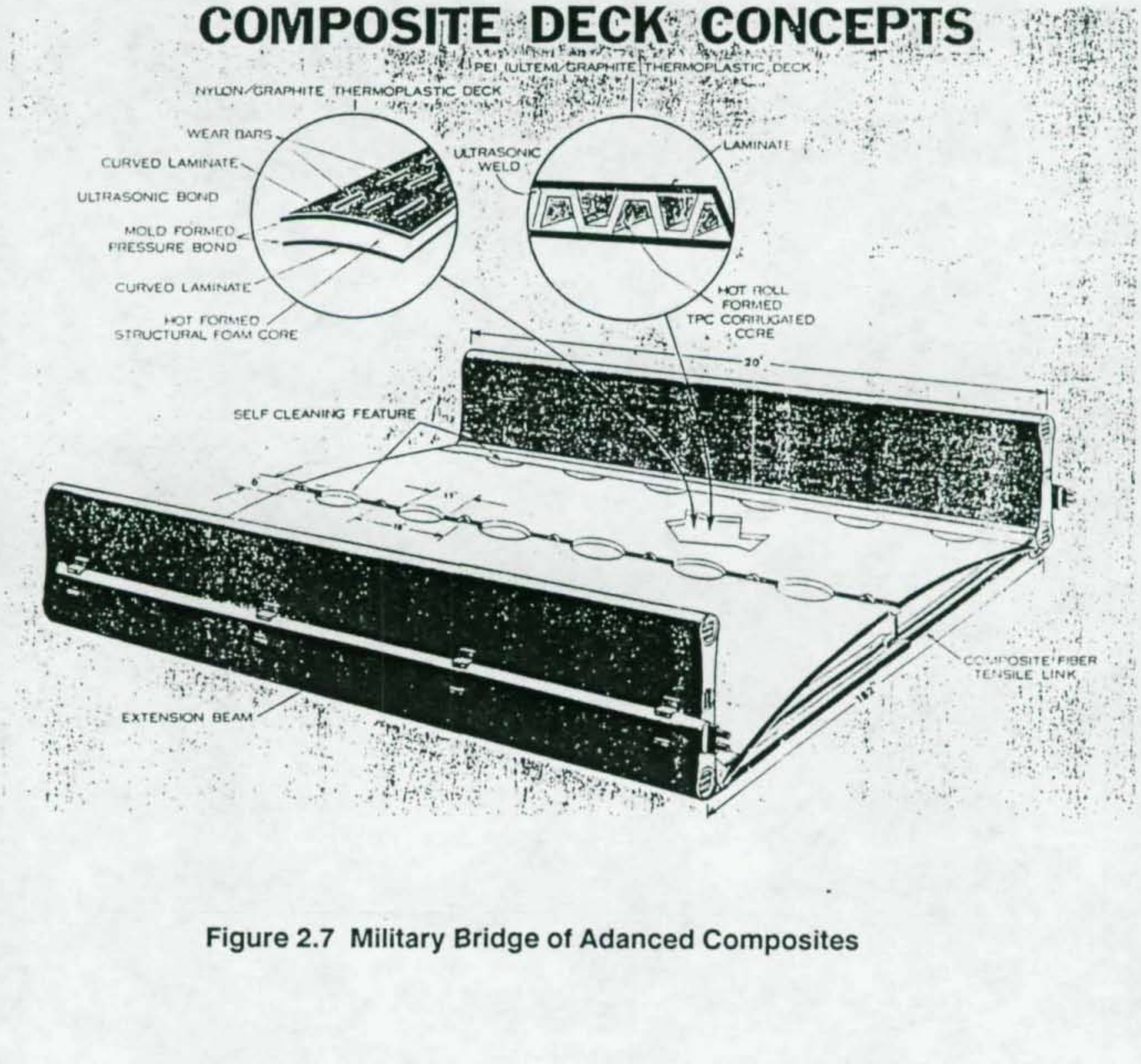


Figure 2.7 Military Bridge of Adanced Composites

The Fort Belvoir Research Center has concluded from their investigations that these composite bridge deck designs would be feasible and durable enough for military applications. However these composites are still far too expensive for use in typical steel framed buildings in the private sector.

In addition to plastics, composites can also be manufactured using metal matrices such as aluminum, titanium and magnesium alloys. These are commonly referred to as Metal Matrix Composites or (MMC's). Fibre reinforcements for these composites include Boron, Carbon, and Silicon Carbide. One example of a structural application of a metal matrix composite is the use of boron/ aluminum struts in the space shuttle orbiter [Bowen 1988]. There also exist products manufactured from ceramic matrix composites or (CMC's). However no civil engineering applications of CMC structural components have been discovered from this investigation. Both MMC's and CMC's are also considerably more expensive than standard glass/resin composites. It is not known when or if these types of composites would ever be feasible for civil engineering applications.

Finally, the most common composites used in civil engineering applications consist of a matrix of isotropic lignin reinforced with a fibrous anisotropic polymeric saccharide cellulose. This composite is generally 65 to 75 percent fibrous cellulose by weight [Gosnell 1987]. The material is usually referred to by civil engineers as wood. The potential for wood as a component in light-weight floor systems will be discussed in greater detail later.

2.2.1 Sandwich Construction

One type of composite structural component that can incorporate a variety of combinations of different building materials is the sandwich panel. Sandwich construction is characterized by the use of a relatively thick, low density core rigidly attached to thin face sheets of comparatively high density. Sandwich panels are an efficient form for applications requiring a light-weight structural component with high bending strength and/or stiffness. They can also be used in applications where there are other important requirements in addition to strength and stiffness, i.e. thermal insulation [ASCE 1978].

One of the earlier structural forms from which sandwich construction evolved is the stressed skin panels commonly used in timber design. Regardless of what types of materials are used to construct the sandwich panel, the structural behavior is the same. The high density face sheets act in resisting the tensile, compressive, flexural and shear stresses that act parallel to the plane of the panel. They also act in distributing any localized forces to the core. The core serves to separate the face sheets and provide a means of shear transfer through the cross-section, as well as stiffening the faces to prevent buckling. The face sheets and core are usually connected by some type of adhesive bonding.

There are two different methods which can be used to analyze sandwich panels. One method is based on Kirchhoff's assumption that plane sections remain plane in bending. This type of analysis is recommended for panels with shear rigid cores. The second type of analysis takes into account shear deformations and deflections in the core. This type of analysis is required for panels incorporating cores with low shear moduli, e.g. low density plastic foam.

The section properties for the composite panels can be calculated from transformed section theory. Often times fibre reinforced plastics are used as materials in sandwich construction. FRP have a tendency to creep under long term sustained loadings. When this is a consideration, it may be necessary to use the viscoelastic moduli for the various components in determining the composite section properties. Other considerations in the analysis of sandwich panels are:

- Local buckling of the faces
- Crushing of the core due to localized loads
- Debonding of the faces and core
- Stress induced by moisture and/or temperature

Further information pertaining to the design and analysis of sandwich panels as well as material properties for many different types of FRP are contained in the Structural Plastics Design Manual [ASCE 1978].

The following is a list of some of the more commonly used materials found as components in sandwich construction:

Cores

Nomex Honeycomb - constructed from aramid fibres dipped in phenolic, very light-weight, high cost
End Grain Balsa Wood - heavier than Nomex, low cost
Aluminum Honeycomb - very light-weight, low cost, susceptible to corrosion, susceptible to crushing from localized loads
Low Density Plastic Foam (polyurethane) - low strength and low shear modulus
Kraft Paper - dipped in phenolic, light-weight and low cost

Facings

S-Glass FRP
Graphite FRP
Aluminum
Plywood

Adhesives Polyester
Epoxy
Phenolic

One of the more popular applications of sandwich panels constructed from the above materials, where weight is a major consideration, is in aircraft flooring panels. In conjunction with this investigation one of the leading manufacturers of aircraft flooring panels, M.C.Gill Corporation, was contacted to obtain product literature and price information. M.C.Gill Corp. manufactures sandwich panels from nearly every combination of materials contained in the list above. It was found that for heavy loadings the most frequently used materials for cores in these panels are the Nomex honeycomb, and the end grain balsa wood. The polyurethane foam and the aluminum honeycomb are not recommended for heavy traffic areas due to the reasons listed above.

Some approximate price data is listed below based on various configurations of sandwich panels used for aircraft floors. The components of each particular panel are designated as follows, core material/facing material. All of the panels are designed to meet the same performance criteria, and have a total depth of 0.40 inches. The weights of the various panels are also listed in addition to their costs.

Balsa Wood/Aluminum	$ 7.25 / sq.ft.	0.79 psf
Balsa Wood/FRP (S-Glass)	$ 7.27 / sq.ft.	0.97 psf
Nomex H.C./FRP (S-Glass)	$12.00 / sq.ft.	0.56 psf
Nomex H.C./FRP (Graphite)	$21.00 / sq.ft.	0.52 psf

It should be evident from this data that the use of advanced composite materials such as Nomex honeycomb and graphite fibres can result in a dramatic increase in cost. It is possible that the floor panels listed above could perform satisfactorily in a floor system designed for a typical steel framed building, based on strength and stiffness requirements. Because the panels are so shallow, it would probably be necessary to limit beam spacings to 2 or 3 feet. Adequate design information is also available that would make it possible to develop larger and thicker panels that could accommodate greater beam spacings. However despite their extremely low weight, it appears that development of sandwich panel floor systems based on the materials used in aircraft floors would be to costly at the present time.

2.2.2 Fibre Reinforced Plastic - Pultruded Deck

The most commonly used FRP for structural applications is the pultruded section. One type of pultruded section that may have potential as a component in light-weight floor systems is fiberglass grating. This is a product that resembles steel bar grating, but was developed for applications where resistance to caustic environments is a primary consideration.

There are several manufacturers of FRP grating at present. Although the exact specifications for the gratings depend on the manufacturer, they are all constructed in a similar fashion. The gratings usually consist of a series of longitudinal pultruded I-beams or T-beams approximately 1 to 2 inches in depth. This arrangement of parallel beams, usually 1.5 to 2 inches on center, is then interconnected by continuous pultruded rods running through the webs. The connecting rods contribute to the transverse stiffness of the grating as well as providing the lateral bracing for the beams.

The original idea for use of FRP grating was to place the deck transversely across the supporting beams, similar to cold-formed steel decking. Thin sheets of either steel or FRP would have to be installed either continuously across the

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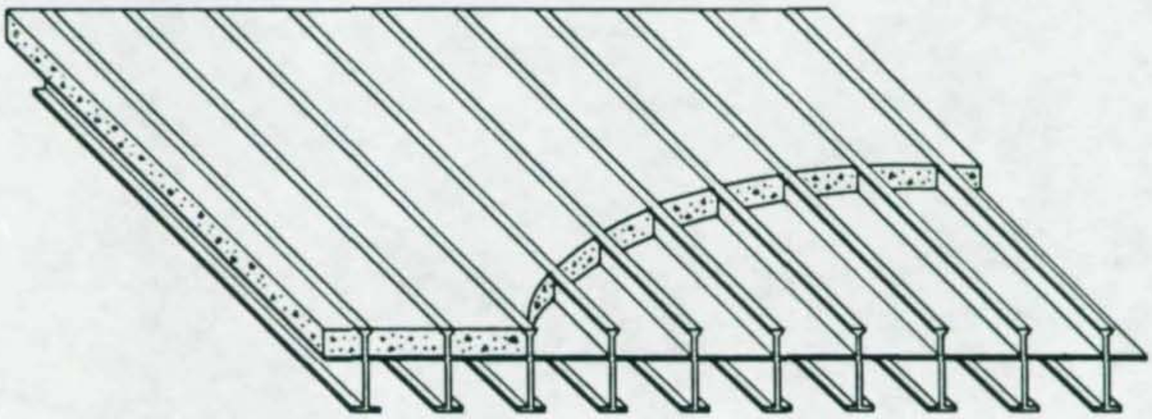


Figure 2.8 Fibre Reinforced Plastic (Proposed Pultruded Deck)

bottom surface of the deck or in narrow strips at approximately mid-depth between the pultruded beams. These strips could be placed directly on top of the interconnecting bars. A thin slab of concrete or other cementitious fill could then be placed on the deck. The FRP deck, would be the primary load carrying member of the slab, similar to a non-composite cold-formed steel deck.

The major drawback to the FRP gratings described above is that they are very expensive to produce, not only because of the material costs, but also because the process of interconnecting all of the individual pultruded beams is a labor intensive operation. Adding the thin sheets to act as form pans for the concrete would also result in substantial cost increase. The deck by itself, in presently available configurations, can cost as much as $10 per square foot.

It is apparent from these costs that of using existing fiberglass gratings is somewhat unrealistic. In an effort to reduce the cost of an FRP floor slab system, a new type of pultruded section has been developed in conjunction with this investigation. The proposed deck, Figure 2.8, consists of a series of three inch deep inverted T-beams on three inch centers. The beams are connected by intermediate flanges approximately one inch from the top flanges. The inverted T-beams and intermediate flanges are pultruded monolithically in two to three foot wide sections, eliminating the other costly manufacturing processes of connecting the beams and installing form pans. The intermediate flange is located so that when composite action is considered most of the concrete is above the neutral axis of the composite section.

A limited investigation was done with regard to the performance of this proposed deck. The section properties of the deck were calculated, as well as the transformed section properties assuming composite action with a one inch deep normal weight concrete fill. It was found that by assuming full composite action with the concrete, the stiffness of the section is at least doubled.

The composite deck configuration was then analyzed as a two span continuous beam, with equal spans of 7.5 feet. In addition to the dead load, a superimposed live load of 70 psf was used, similar to the loadings used for the reference floor systems. Although the stresses were found to be well within the acceptable range for both materials, deflections could become a problem because of the low elastic moduli of the materials, i.e. approximately 3000 ksi for both the concrete and the FRP.

Also investigated was the possibility of stiffening the proposed section by using light-gage steel wire reinforcements in conjunction with the glass fibres. Optimal use of the wire reinforcing fibres was sought by placing five 16-gage wires in the bottom flange, and three in the top flange. As a result the moment of inertia of the FRP deck by itself was doubled and the moment of inertia of the composite section was increased by fifty percent. This increase in stiffness is accomplished with only two percent of the gross area of the cross-section consisting of the steel wires. A similar increase could be accomplished by using other high modulus reinforcing fibres such as Kevlar or graphite in combination with the glass fibres. However, the use of small quantities of high modulus reinforcing materials can results in much higher stresses in these fibres, since they would carry a larger percentage of the load. Also of concern would be debonding of these fibres from the matrix.

The next portion of the investigation involved incorporating this composite deck configuration into a floor system. A single 30 x 30 foot bay was designed using five W16x31 beams on 7.5 foot centers. The floor system was then analyzed for susceptibility to annoying vibrations induced by human occupancy based on the model presented in Technical Digest No. 5 of the Steel Joist Institute [Galambos, Undated]. The increased stiffness of the slab attributed to the pultruded section appeared to have a favorable effect on the response of the system. Based on the inequality developed by Murray [1981], a floor system using the proposed FRP deck would have a required damping of approximately 7.1 to 8.1 percent. Whereas this is not necessarily within the acceptable range, it is not extremely bad either. In general it may be difficult to accurately predict the behavior of these systems with regard to vibrations without experimental testing.

One other characteristic which must be determined experimentally is the percentage of composite action developed between the concrete and the FRP. Most of the presently available gratings have a grit surface applied to the top flanges to provide a non-slip surface. It may be possible to apply this rough surface to the intermediate flanges to assist in shear transfer between the two materials. On the other hand, a certain amount of relative slip between the concrete and the FRP may contribute to the damping of vibrations in the floor system. Shrinkage cracks resulting from curing of the concrete may also be of concern.

With regard to weight, the proposed pultruded section indicated weighs approximately 3.3 psf. With the addition of a normal weight concrete fill the total weight of the slab is approximately 14.5 psf. Compared to the reference floor systems this corresponds to a weight reduction of anywhere from fifty to sixty percent. A further weight reduction can also be achieved by using a light-weight concrete or gypsum based concrete fill. By screeding the concrete across the top flanges of the pultruded section, it would also be possible to prevent ponding of the fill material, thereby more accurately controlling the dead load of the slab.

In conjunction with this investigation, several manufacturers were contacted to obtain product information on pultruded FRP structural components. Later in the study some of these companies were contacted again for feedback concerning the proposed pultruded deck section, including information pertaining to costs. The following is a summary based on conversations with the various manufacturers.

- It would be possible to manufacture the proposed pultruded deck section.

- The tooling costs to begin production would range between $34,000 and $40,000, including the preformers and dies.

- It would be possible to utilize steel wires as a small portion of the reinforcing fibres. Kevlar or graphite fibres could be used as well, but it is not recommended because of the high material costs.

- Possible controlling limit states to consider include local buckling of the web at concentrated loads or supports, as well as deflections induced by sustained loads. The importance of these limit states results from the low shear modulus and viscoelastic behavior exhibited by the thermosetting resins.

- The approximate retail costs, including production and marketing, would be $1.80 to $2.00 per lb. using standard resins or up to $3.00 per lb. using fire retardant resins. This translates into a cost of anywhere from $6.00 to $10.00 per sq. ft. for the proposed 3.0 in. deep deck.

One of the other limitations of pultruded sections, resulting from the manufacturing process, is element thickness. In general it is difficult to produce sections much thinner than 0.1 inches. Therefore the production of a much shallower depth cross-section would not result in a significant reduction in weight. Since the total cost of the section is usually governed by the costs of the raw materials, a deeper profile would result in a much more economical deck based on the specific strength, i.e. strength to weight ratio.

Another type of FRP grating that is presently available is called "Kordek". Kordek is an orthotropic grid manufactured from glass fibre reinforcement interwoven in either a square or rectangular configuration. The fibres are coated with a thermosetting resin and pressure molded into rectangular sections resembling waffles. One of the benefits of this type of FRP deck, is that two-way bending action is inherent because of the perpendicular orientation of the fibres. Another advantage is that the pressure molding process generally yields a slightly higher flexural modulus than a pultrusion process using the same materials.

Various configurations of Kordek grating were also investigated as potential light weight floor systems. For the most part these decks would probably perform similar to the pultruded decks, however there are several differences. Because the Kordek is an orthotropic grid, with equal depth beams running in both directions, the concrete fill would be confined in small square or rectangular cells rather than longitudinal strips. This would probably eliminate any problems with cracking of the concrete as well as enhance the diaghram action of the system.

Kordek gratings are presently available in either 1 inch or 1.5 inch deep grids. One of the drawbacks to this type of deck is that it may not be possible to mold the sections with intermediate flanges to act as form pans. However it is possible to attach a continuous sheet to either the top or bottom surface of the grating. Another possibility, would be to produce a deeper section, e.g. 3 inch deep grid, with a flat sheet attached to the bottom surface. The cells could be filled to mid-depth with a low density plastic foam. The top portion could then be filled with a concrete or other cementitious fill to provide a walking surface as well as increase the stiffness.

Another disadvantage of these orthotropic grids is that the production size is limited by the size of the mold, whereas the pultruded sections can be manufactured to almost any desired length. These grids are also relatively expensive. The presently available open grid configurations can cost as much as $9.00 per square foot.

Overall the use of fibre reinforced plastics in light-weight floor systems should not be discounted completely. The pultruded deck in particular may have great potential. Despite the high production costs, these systems have many advantages. However it may be difficult to develop a very generalized FRP floor system that could be implemented under a variety of beam spacings and loadings.

2.3 Lightweight Precast Panels

One method currently available to reduce the weight of the slab in a floor system is to use a light-weight subfloor. A subfloor consists of an unfinished floor that acts in transferring the loads to the supporting members as well as serving as a base for the finished floor surface. In general the subfloor is the primary load carrying component in the slab, although it can be enhanced, e.g. by using a concrete overlay.

One type of subfloor which is currently available for use in steel frame buildings is light-weight concrete planks. These consist of light-weight aerated concrete reinforced with steel wire mats and cast into panels small enough that they can usually be handled by one man. The edges are generally cast with tongue-and-groove joints so that the panels can be interconnected [Patton 1976]. One way in which the tongue-and-groove edges can be constructed is using a galvanized steel edging that is cast integrally into the concrete. One other characteristic of these panels is that they are nailable.

Only the planks with steel edges are recommended for use in floor systems. The reason for this is that approximately 80 percent of the load is actually supported by the steel edged frame. The concrete itself has a compressive strength of only 1000 psi and mainly serves to provide a level walking surface. Because of the complexity of the interaction between the steel edges, the concrete and the wire reinforcement, analysis of these panels can be rather complicated. As a result the design information tabulated by the manufacturers is usually determined experimentally. According to one manufacturer contacted, the allowable recommended design loads are only 25 percent of the ultimate loads based on full scale tests. This translates into a factor of safety of four. Furthermore, the design loads for floors are governed by a limiting deflection of span/360 [Martin Fireproofing Corp. 1989].

There are relatively few manufacturers of light-weight concrete planks for use as roofing or subflooring panels. Two manufacturers were contacted in conjunction with this project to obtain product and design literature pertaining to these planks. Based on this literature, two floor systems were designed using the same parameters as the reference floor systems presented earlier. Both floor systems consisted of 72 Steel Edge Creteplanks, Figure 2.9, manufactured by Martin Fireproofing Corporation. In addition a 1/2 inch gypsum-base floor overlay was used as recommended by the manufacturer. One system uses open web steel joists spaced at 2.5 ft. intervals. The other system uses hot-rolled W-shapes placed at 5 ft. intervals. The spacing interval of 5 feet is used as the maximum that is permitted by the design guides. Using a 5 ft. beam spacing interval, the planks can carry loads as high as 175 psf, with respect to both strength and stiffness. This is significantly higher than the 70 psf considered in this investigation. The total weights of the floor systems are 17,400 lbs. for the open-web joist floor and 19,900 lbs using W16x26 hot-rolled beams. This is a significant reduction in weight compared to even the lightest of the reference floor systems.

The concrete plank floor systems were also analyzed for susceptibility to annoying floor vibration due to human occupancy. From this analysis it was determined that the plank floor systems could be just as susceptible to floor vibration problems as the reference floors, if not more so. However the model used to analyze the floor for vibration characteristics may not be accurate for this type of floor system for the following reasons.

- It is not known what the actual modulus of elasticity is for the interconnected light-weight, steel edged panels.
- It is not known how much composite action is developed between the supporting beams and the planks.
- The additional stiffness attributed to any composite action between the planks and the overlay is neglected.

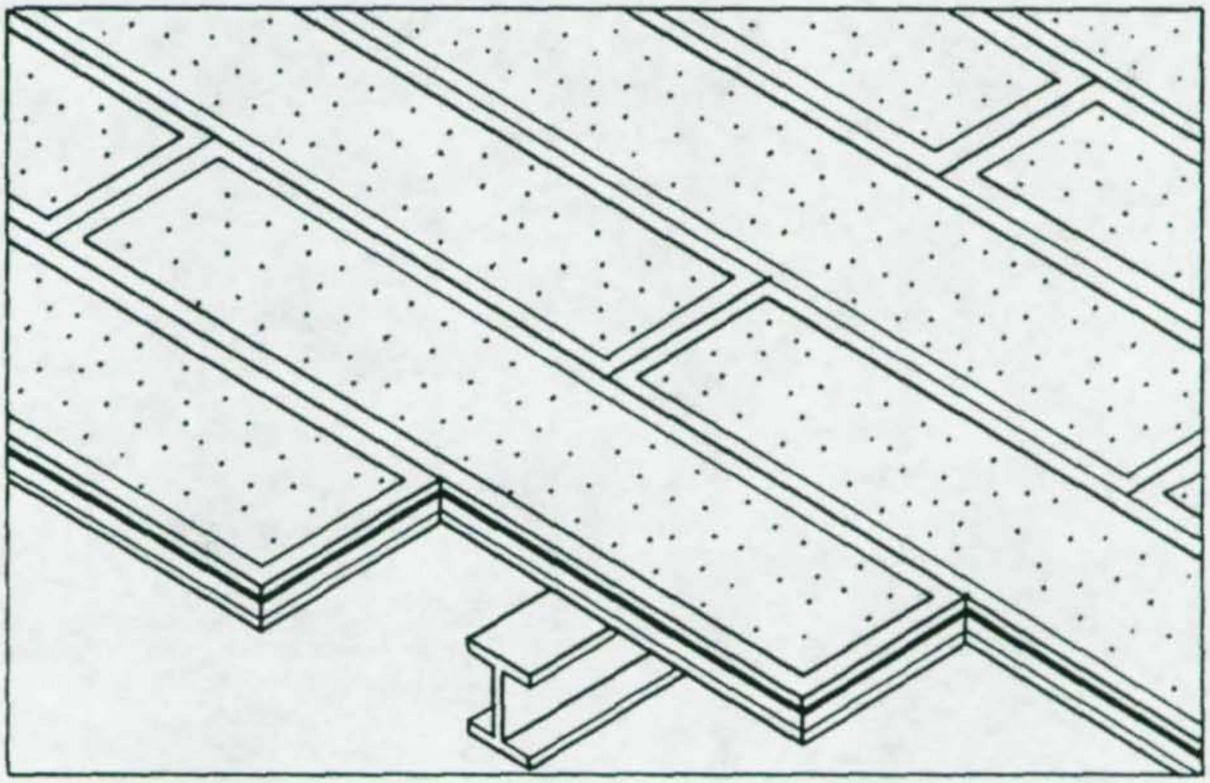


Figure 2.9 Lightweight Precast Concrete Planks

In addition to the items listed above, it may be possible that the friction between the tongue-and-groove joints in the planks might be an excellent source of damping against vibrations in the floor system.

One of the major advantages to light-weight precast planks, other than an overall weight reduction of a structural system, is that they can be installed quite easily. Despite all of the advantages of these panels, they still maintain only a relatively small market. This is in part due to their cost, which apparently is not competitive with typical floor systems used in steel framed buildings. The cost for the "Steel Edge Creteplank" is approximately $2.70/sq.ft. and the installed cost can run anywhere from $3.70 to $4.00 per sq. ft. Another factor that has contributed to their lack of popularity is the resistance of the building trades to accept an unusual form of building material.

2.4 "DRY" Lightweight Floors

A more recent development in the use of cold-formed steel structural components is a composite floor system composed of cold-formed steel decking and various types of plate materials such as plywood, oriented strand board, gypsum board, expanded plastics, or mineral wool. This type of floor system is known as a "dry" floor, Figure 2.10. Research has been conducted in the development of these floor systems in Europe [Konig 1981, Baehre and Urschel 1984, Herniland 1986, Wright et al 1989,].

The development of modular composite floor systems constructed from non-metal plate materials and cold-formed steel sections dates from 1967 [Caldwell and Cooke 1967]. Much of the recent work in the development of "dry"

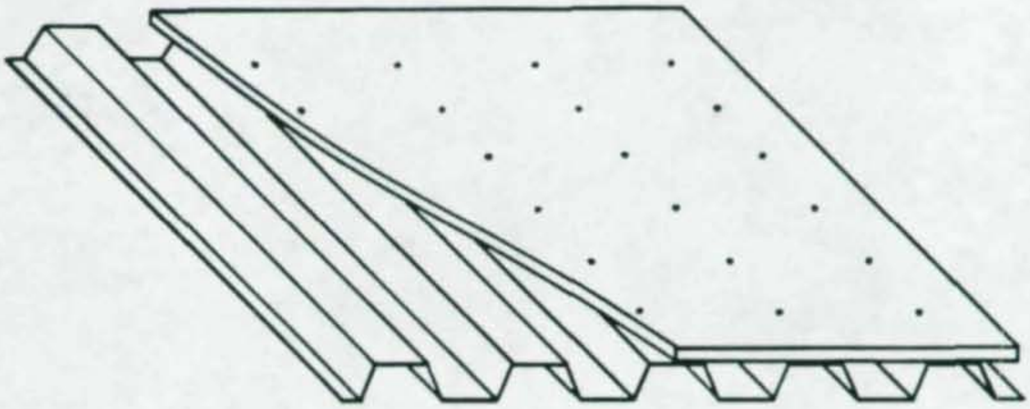


Figure 2.10 PROFILED STEEL DECK/DRY BOARD COMPOSITE FLOORS

floor systems is attributed to the research of Professor Rolf Baehre at the Royal Institute of Technology in Stockholm, Sweden [Konig 1981]. Baehre's research began by investigating the possibility of increasing the strength and stiffness characteristics of thin-walled steel members by combining cold-formed steel sections with different components of either organic or inorganic materials to form composite beams. The first cold-formed sections to be investigated were transversely loaded C-shaped sections oriented such that bending would occur along the weak axis. Later research includes the use of other cold-formed sections using profiled deck.

In general the compression elements of thin-walled steel sections are very susceptible to local buckling. As a result, the effective widths of the compression elements must be reduced to accurately determine the load carrying capacity of the entire member. Research has shown that although the board materials have relatively low elastic moduli with respect to steel, they still contribute to the section properties of the composite section in several different ways. One way is by their ability to sustain axial force due to their longitudinal stiffness. It has also been shown that attaching the board materials to the compression side of the cold-formed steel results in a stiffening of the compression zone and subsequent increase in the effective width with regard to buckling. It was also discovered, that if the composite beam has a high degree of asymmetry then it is possible to achieve extensive plastic yielding in the tension zone prior to failure of the section [Konig 1981].

As a demonstration of the behavior of these composite beams, it was found that by combining a cold-formed steel C-section with plywood the stiffness of the C-section was doubled. With regard to two-dimensional action, it was found that the use of wood or gypsum boards, combined with a profiled steel deck susceptible to buckling, could increase the stiffness of the section to a value corresponding to the stiffness of an unbuckled section [Baehre and Urschel 1984].

A profiled steel sheet/dry boarding composite floor system was investigated by Wright, Evans and Burt of Great Britain [1989]. They originally sought to develop the system as a replacement to existing timber joist floors used in lightly loaded office and residential buildings. One of the primary concerns, in addition to making the systems lightweight, was to reduce the floor-to-floor height of the building by making the floor systems shallower than typical timber joist floors. Another motivating factor was to increase the use of steel products in the residential building market.

Having done extensive research in the area of profiled steel sheeting the authors were well aware of the fact that, in general, most steel decks are designed to carry substantial construction loads by themselves prior to any composite action achieved once the concrete has cured. Many steel decks on the market are capable of supporting loads as high as 50 psf prior to any composite action. As a result, it was determined that a concrete slab would not be required to obtain the load carrying capacity desired. Instead the authors decided to use wood panels to provide a flat floor surface. Furthermore, by attaching the boarding to the deck with some sort of rigid or semi-rigid connection, it was anticipated that composite action could be achieved, thereby increasing the stiffness and load carrying capacity of the system.

As can be seen from the research that has been conducted in dry board floor systems one of the biggest difficulties in analyzing the systems is attributed to partial composite action between the steel section and the board material. Incomplete composite action results when the capacity of the board to sustain axial force and to stiffen the compression zone of the steel is only partially utilized [Konig 1981]. Subsequently the load capacity and stiffness of the composite section are decreased. Although there are mathematical models available to analyze partially composite beams, it is necessary to know the stiffness of the connection to make use of them. In addition, the stiffness of the connection is dependent on both the type of connector as well as the type of boarding material used, and must be determined experimentally [Wright, et al 1989].

Based on the research done at The Royal Institute of Technology, Stockholm, the following list of limit states has been compiled with regard to the composite beam action of cold-formed sections and boards [Konig 1981].

1. Failure of the sheeting panel

a) the yield stress is reached in the top flange
b) the yield stress is reached at the junction of the top flange and web in conjunction with the buckling of the flange (local buckling; buckling of the stiffener
c) the yield stress is reached in the bottom flange

2. Failure of the board component

a) compression failure due to normal stresses in the longitudinal direction of the panel (compression force and bending moment)
b) bending failure due to bending in the transverse direction
c) the tensile strength is exceeded in a direction perpendicular to the face of the board component (lamellar tearing)
d) buckling of the board component

3. Failure of the bonded joint

a) shear failure
b) tensile failure

Other important considerations in the development of dry floor systems include installation of services, insulation, noise transmission, installation, fire rating and susceptibility to vibrations [Wright, et al. 1989]. With regard to vibration characteristics, it is likely that the flexibility evident in the connections could provide an excellent source of damping for the system. With regard to other considerations Baehre and Urschel [1984] have proposed a double skin floor system for high sound and fire protection requirements. This system incorporates several layers of insulating materials as well as boarding composed of both wood and gypsum to enhance the noise transmission and fire rating requirements of the system. These additional materials could also enhance the damping qualities of the floor system with regard to vibrations.

One other problem introduced by using wood products in these floor systems is the effect of creep. Creep is essentially the increase in deformations over time due to sustained loads, which also results in a lowering of the board stresses and subsequent increase in steel stresses. Konig suggests that this has little effect on the load bearing capacity of the system, and can be accounted for by using a lower modulus for the board material. Shrinkage and swelling in the board can also result in a variation in the stresses in the composite beam. Here again the effects are considered to be very small [Konig 1981].

As indicated earlier, designing dry board floor systems without additional experimental data could prove to be somewhat inaccurate. However based on the work done by Wright, Evans, and Burt, as well as some preliminary rough calculations it seems possible to develop a light-weight floor system using steel profiled deck and wood boards. Using a 3 in. deep profiled deck with 3/4 in. structural plywood it should be possible to construct a single span floor system with a span of approximately 10 to 12 ft. based on the light loadings being considered in this investigation. The weight of a floor system with these components would be 6 to 7 psf, compared to approximately 30 psf for the reference floor systems using concrete slabs. This figure does not include the weight of the supporting beams or fire protection materials. However the addition of fire proofing materials would not cause a substantial increase in the weight.

All of the work done to date on these systems advocates the use of self-tapping screws or adhesives as the best means of connection between the two mediums. A combination of connectors, e.g. nails and elastomeric adhesives, is common practice in timber floor construction. It may prove interesting to investigate the combination of adhesives and self-tapping screws as the means of connection for dry board floor systems. This might provide a considerable increase in the stiffness of the connection medium, and subsequently provide a higher degree of partial composite interaction.

Other variations could be considered in addition to the means of connection. One of these is to investigate the use of cellular profiled steel deck. This is a type of deck available from several manufacturers that consists of a standard profiled deck with a continuous light gage sheet spot welded across one entire surface of the deck. By orienting the deck with the continuous sheet in the tension zone it may be possible to increase the strength and stiffness of the composite beam. Another variation might be to attach board materials to both the top and bottom surface such that the deck could be run continuously over the supporting beams.

The evolution of profiled steel decking has also resulted in the development of long span roof decks that are capable of spanning 30 to 36 ft.

[Bryan 1979]. The decks are usually about 4.5 to 8 in. deep. There has been some research conducted in Sweden with regard to using these long span decks in a dry board floor system [Herniland 1986].

One of the advantages of using long span decking is that it may be possible to span the entire floor bay eliminating the need for secondary beams. Although the weight reduction resulting from the elimination of the secondary beams could be offset by the weight increase caused by using a heavier steel deck, the system would have other advantages. For instance, the cells of the long span deck could be used to accommodate service requirements and possibly act as ventilation ducts for mechanical systems. Also, the depth of the floor system and the floor to floor height of the building could be reduced.

Despite the weight of the long-span deck, the entire floor system would still be considerably lighter than typical floor systems. Some of the negative aspects of this system are that the long span decks are relatively expensive. In addition, use of the profiles acting by themselves is generally governed by deflections for long spans. Since the increase in stiffness is not as great as the increase in load bearing capacity of dry board composite systems, it would be necessary to develop a fairly stiff connection medium to achieve a high degree of partial composite action.

2.5 Long-Span Deck/Concrete Slab Composite Floors

Another floor system developed in conjunction with this investigation is a long-span deck and composite slab floor system. This system consists of 7.5 in. deep, 14 gage cold-formed steel hat sections placed side by side with a shallow concrete slab poured above the top flanges, Figure 2.11. The concrete itself is

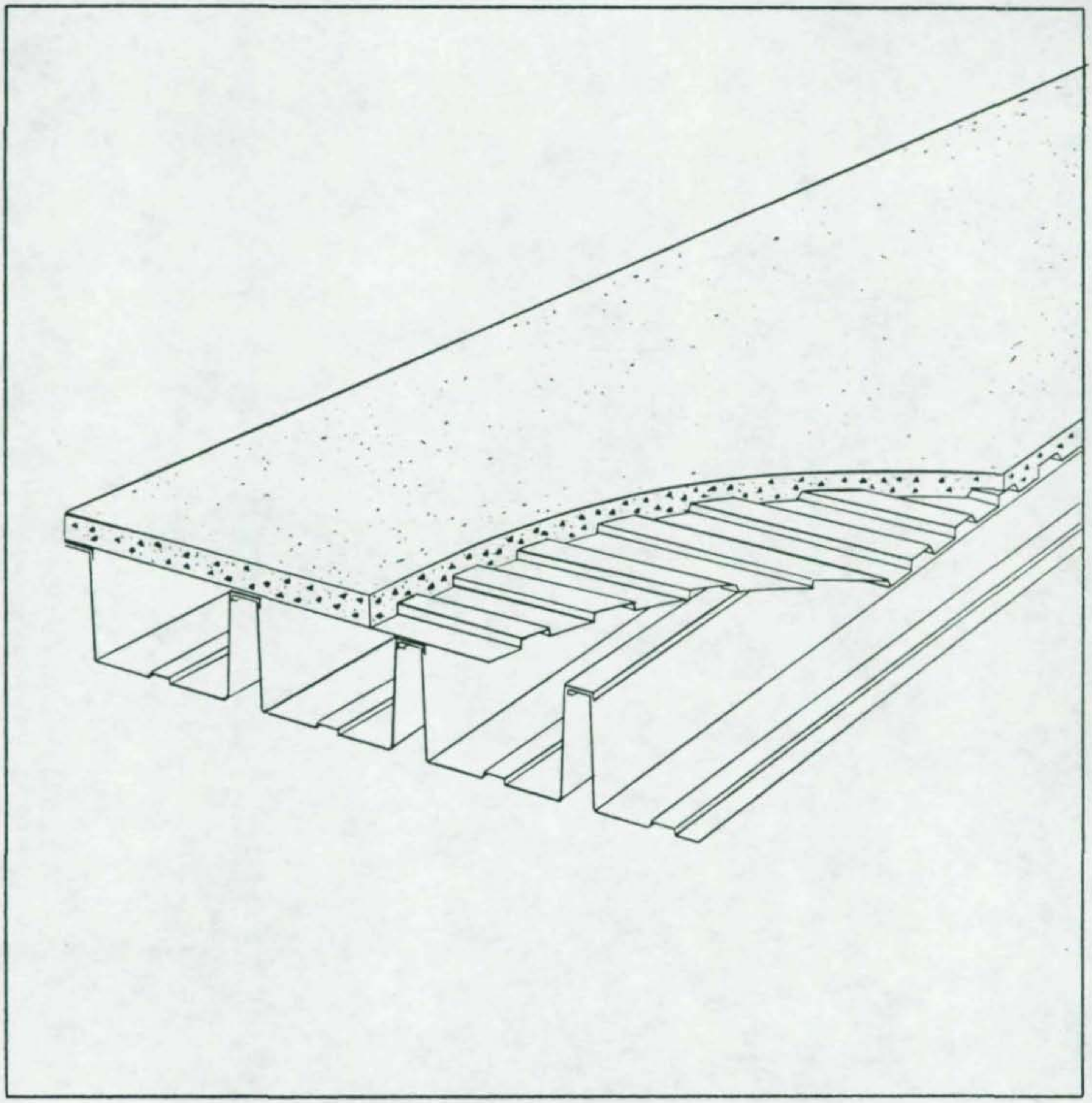


Figure 2.11 Long-Span Cold-Formed Deck/Concrete Slab Composite Floor System

placed on top of a very light gage, shallow steel deck which is laid transversely across the top of the long-span deck and rigidly attached by either self-tapping screws or spot welds. The shallow profiled steel deck must also be designed so that shear forces can be transferred between the steel and concrete components.

As with the dry board floor systems, the use of long-span deck makes it possible to span 30 ft. between supports. This system also offers the advantage of reducing the floor-to-floor height of the structure as well as using the cells of the deck to accommodate service requirements. The advantage of using concrete over dry board sheeting materials is that the concrete offers significantly greater strength and stiffness characteristics. A concrete slab would also be less susceptible to shrinkage and swelling due to moisture.

With respect to serviceability, the performance of this type of system may be difficult to predict analytically. The susceptibility of the system to annoying vibrations is highly dependent on the stiffness of the configuration perpendicular to the long-span deck. If the top flanges of the long-span deck contribute significantly to the transverse stiffness of the slab, then the magnitude of vibrations in the system may not be too severe. However if this is not the case, then this system may be quite uncomfortable to human occupants.

In all, long-span deck/concrete slab composite floor systems offer some tremendous advantages over typical systems. In addition, materials are presently available that make it possible to fabricate a prototype of this system, as discussed in greater detail in Chapter III.

2.6 Steel Grid Floor Systems

Steel grid decking is another promising alternative to cold-formed steel deck and concrete slabs. In this type of deck an orthotropic steel grid is fabricated by using shallow steel I-beams, 4.25 in. deep, positioned at close parallel spacings of about 4 to 8 in. Shallower secondary beams, usually rectangular bars, are then inserted through horizontal slits in the webs of the primary grid beams. After the secondary beams are inserted they are rotated 90 degrees into the upright position and welded in place. The space between the steel beams can then be fitted with form pans and filled with concrete.

Steel grid decking was originally developed in the 1920's for use as a lightweight but high strength bridge deck alternative to thick reinforced concrete slabs. One of the earliest examples is the 10th Street Bridge in Pittsburgh, PA. Constructed in 1932. This bridge is still in service, demonstrating the durability of these decks [Gilmore 1987]. Since that time steel grid bridge decks have evolved into a number of different types. In the 1950's an intermediate flange was added to the shallow I-beams and a light gage steel form pan inserted. By doing this, only the top half of the grid is filled with concrete resulting in a significant reduction of the dead load [Gilmore 1987]. This also facilitates a more economical use of the concrete. For example, in grids where the concrete is poured full depth, the concrete below the neutral axis is in tension, and subsequently does not contribute to the strength or stiffness of the section.

The most recent innovation in steel grid bridge decking is the "Exodermic" deck developed by Neal Bettigole in the early 1980's [Stefanides 1985]. The exodermic deck consists of an orthotropic steel grid with a composite slab placed on top of the grid. The slab is connected to the grid by shear studs and an elevated grid of rebars [Gilmore 1987]. These units are usually prefabricated complete with the concrete already cast in the deck. This type of decking was originally developed for an addition to the Driscoll Bridge on the New Jersey Garden State Parkway. The use of the exodermic panels resulted in a weight savings of over 50% without a reduction in strength [Stefanides 1985].

In general, the major advantages of steel grid bridge decking are; reduced dead load, durability, the potential for composite action with the supporting steel, reduced erection time due to prefabrication, and lower overall cost installed. The four most common types of steel grid bridge decking presently available are listed below [Gilmore 1987], Figure 2.12.

1. Full-depth grid reinforced concrete decks
2. Half-depth grid reinforced concrete decks
3. Exodermic Bridge decking
4. Open steel grid bridge flooring (welded or riveted)

One of the largest manufacturers of steel grid bridge floor systems is IKG Greulich. In addition to the orthotropic grids, IKG Greulich also manufactures a 4-way grid which incorporates diagonal secondary bars. The major difficulty in using steel grid bridge decks in lightweight floor systems for steel framed buildings is that they are designed for AASHTO loadings. As a result these decks are stronger and heavier than required for the loadings encountered in commercial and residential buildings. However in a scaled down configuration they may have great potential.

In conjunction with this investigation a preliminary floor system was designed using a scaled down version of the bridge decks. The scaled down

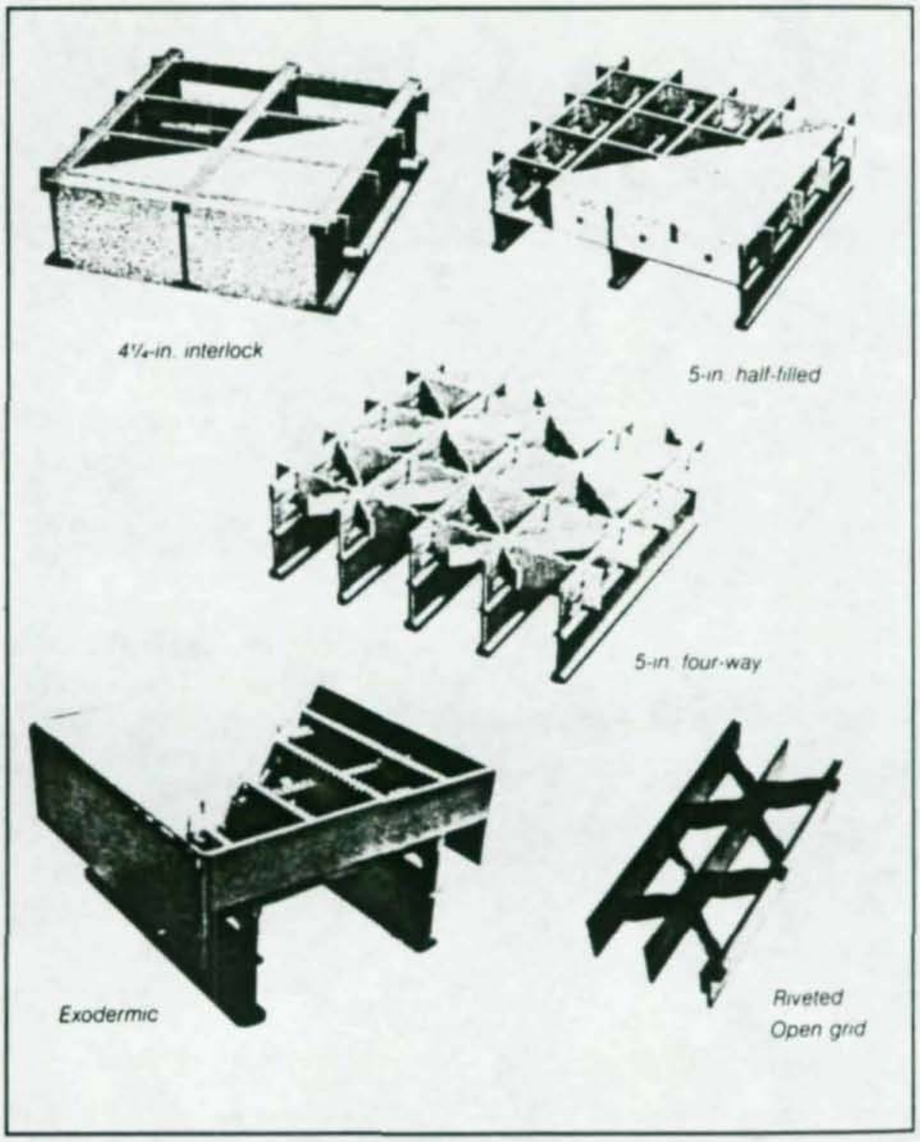


Figure 2.12 Typical Orthotropic Steel Bridge Decks

model consists of 3.5 inch I-beams on 6 inch centers. The secondary beams are 1 in. by 3/16 in. rectangular bars on 6 in. centers. This grid also contains an intermediate flange on the I-beams which supports light gage steel form panels. Consequently the grid can be filled with 1.25 inches of concrete to provide a level walking surface. The entire configuration, including a normal weight concrete fill, weighs 23.3 psf. This translates into a 38 percent weight reduction in comparison to the average reference floor system weight.

The proposed grid was analyzed as a two span continuous beam with equal spans of 7.5 feet. All calculations were based on the transformed section properties, assuming full composite action with a 1.25 inch deep normal weight concrete fill. Using a superimposed live load of 70 psf, the stresses and deflections for this configuration were found to be well within allowable limits. In reality it may be possible to span as far as 15 ft. assuming a two span continuous deck. Due to the low stresses in the concrete it may also be possible to use other lightweight fill materials resulting in a further reduction in weight.

Despite the light weight and excellent performance characteristics, even the scaled down version of the orthotropic deck may not be a feasible alternative to typical profiled steel deck concrete slab floors. Although lighter than presently available decks, the scaled down model still contains more than 8 psf of steel compared to approximately 2 psf for a sheet of cold-formed deck. In addition, the orthotropic decks are very expensive to manufacture due to numerous fabrication and welding processes.

In an effort to reduce the cost an alternate form of steel grid floor system was developed in conjunction with this investigation, Figure 2.13. The proposed system differs from present grid decks in several aspects. For example, the

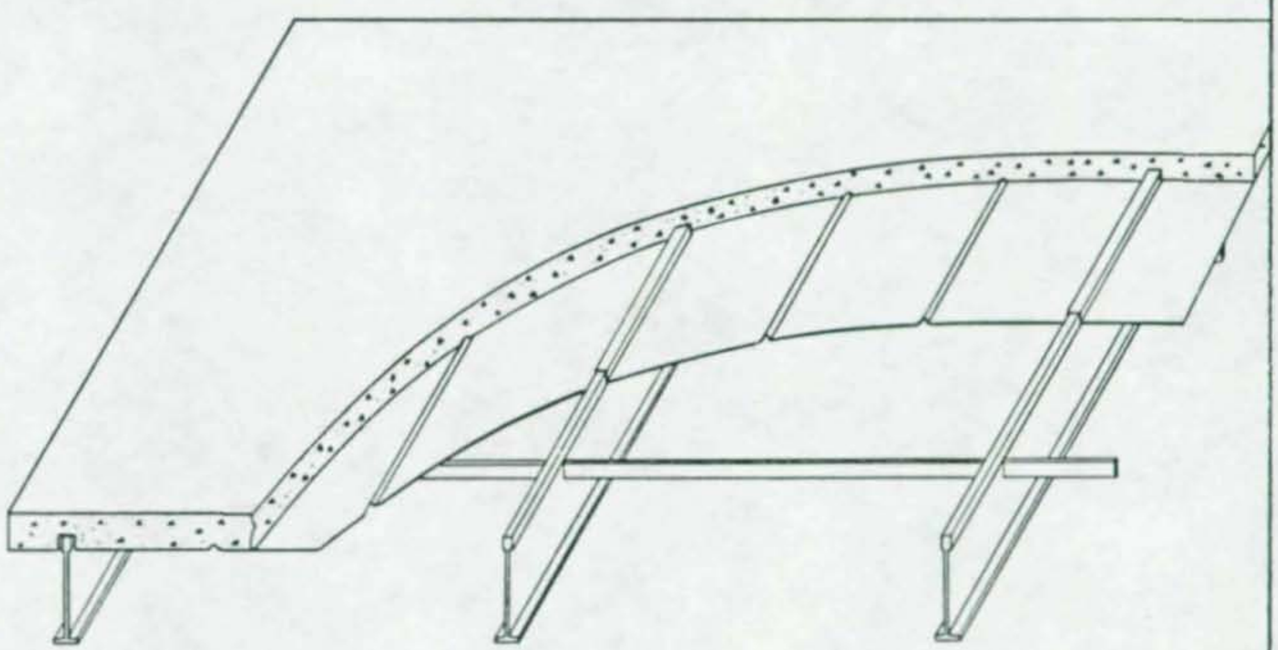


Figure 2.13 Proposed Steel Grid w/Continuous Profiled Deck
& Concrete Slab

spacing on the primary bearing bars is increased to 18 in. rather than the usual 6 to 8 in. The spacing on the secondary bars is also increased to 12 in. This results in a much more open grid which in turn requires considerably fewer welds to fabricate. The primary bars are the 4.25 in. I-beams presently manufactured. The secondary members are 1 in. by 3/16 in. rectangular bars. The secondary bars are installed such that the top of the bars are 1/2 inches from the tops of the 4.25 in. primary beams.

The biggest difference in the proposed deck however, is in the type of form pans used. Rather than install small square panels between the members of the grid, a continuous, light-gage cold-formed steel deck is placed with the ribs parallel to the 4.25 in. I-beams. The profiled deck is then supported both longitudinally by the top flanges of the I-beams as well as transversely by the tops of the secondary bars. The deck can either be installed during the fabrication process, or during erection. Regardless of when it is installed, the continuous steel deck would be a cheaper alternative than individual form pans.

Finally a 1.5 in. deep lightweight concrete slab is placed on the profiled steel deck. The entire configuration weighs 18.7 psf. It should also be noted that this deck uses 5 psf of steel in comparison to the more than 12 psf in the lightest regular bridge deck. The reduction in steel weight, along with lower fabrication costs, could make the proposed grid deck floor system feasible for buildings. this system also presents a significant reduction in the weight relative to typical floor systems.

To further study the concept, a preliminary floor system, 30 ft. by 30 ft. was designed using the proposed continuous steel sheet grid deck. Again a superimposed live load of 70 psf was used. The framing members consist of 4 each W16x36 beams at 10 ft. intervals. The deck was analyzed as a three span continuous beam, with equal spans of 10 ft. The stresses and deflections in the deck were found to be well within allowable limits. From this analysis it was also determined that composite action between the 4.25 in. I-beams and the concrete slab was not necessary for either strength or stiffness requirements.

The system was also analyzed for susceptibility to annoying vibrations induced by human occupancy and appeared to have a response at least equal to the average reference floor system. However because of the unusual configuration, the vibration response of the system should to be verified experimentally. It should be noted that although composite action between the components is not required, it should have a favorable effect on the susceptibility of the system to annoying vibrations.

Another variation of the steel grid floor system was also investigated, Figure 2.14. This system is similar to the previously discussed deck, except that the primary beams consist of a custom, hot-rolled shape that resembles an inverted T-beam rather than an I-beam. The beams are 7.0 in. deep with a small top flange and a large bottom flange, Figure 2.14a. The beams are placed on 6 to 12 in. centers, and interconnected with 1-1/2 in. by 3/16 in. rectangular bars. A light gage profiled steel deck is inserted continuously between the beams supported by the secondary bars. Once again a lightweight concrete slab with a total depth of 2.0 in. is placed on top of the deck. Shear forces are transferred between the concrete and the steel beams by providing deformations in the top flanges of the steel beams similar to those found on typical reinforcing steel.

The primary benefit of this system is that it is capable of spanning 30 ft. as a simply supported beam. This eliminates the need for any secondary framing

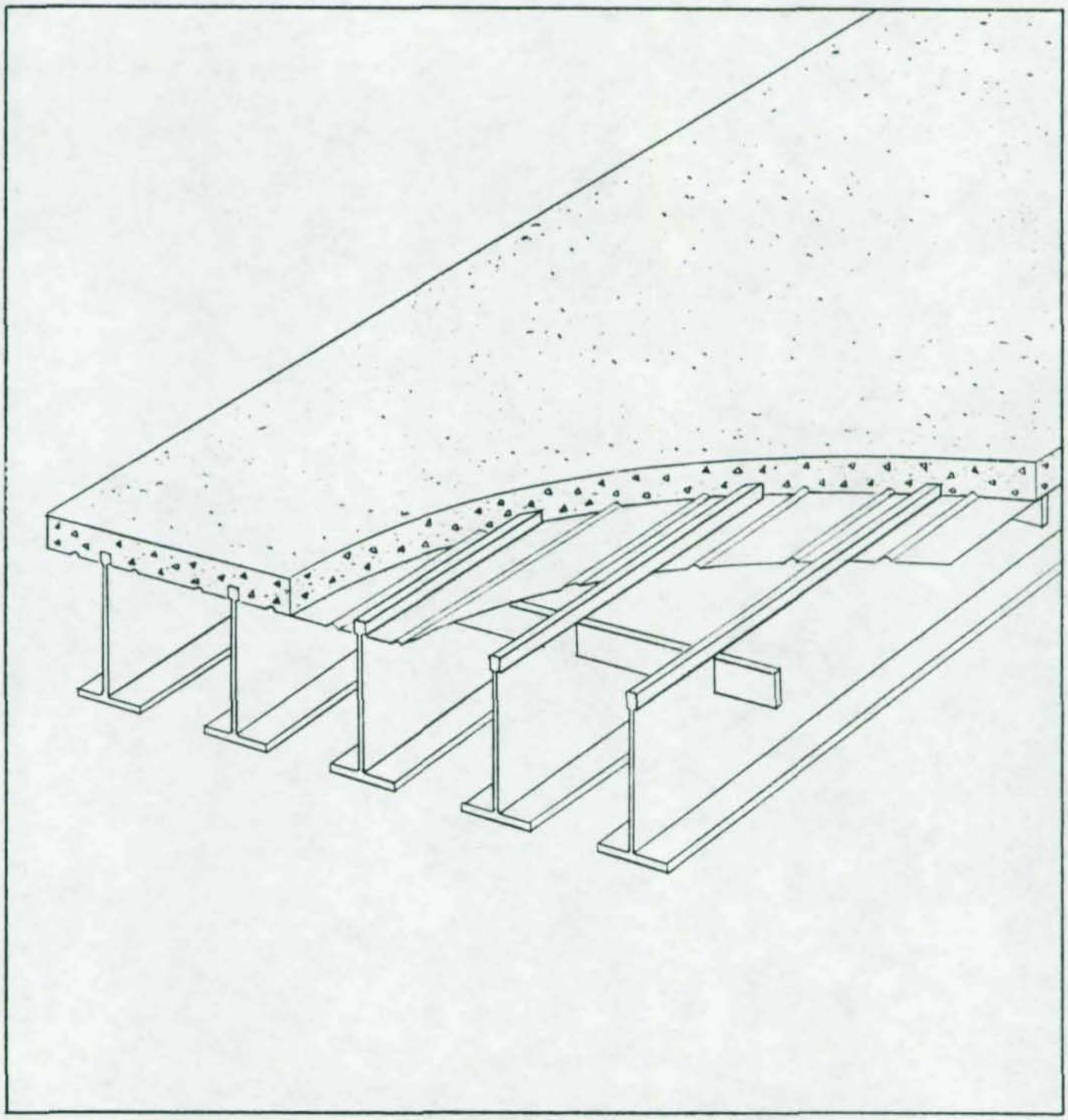


Figure 2.14 Proposed Long Span Steel Grid Floor System

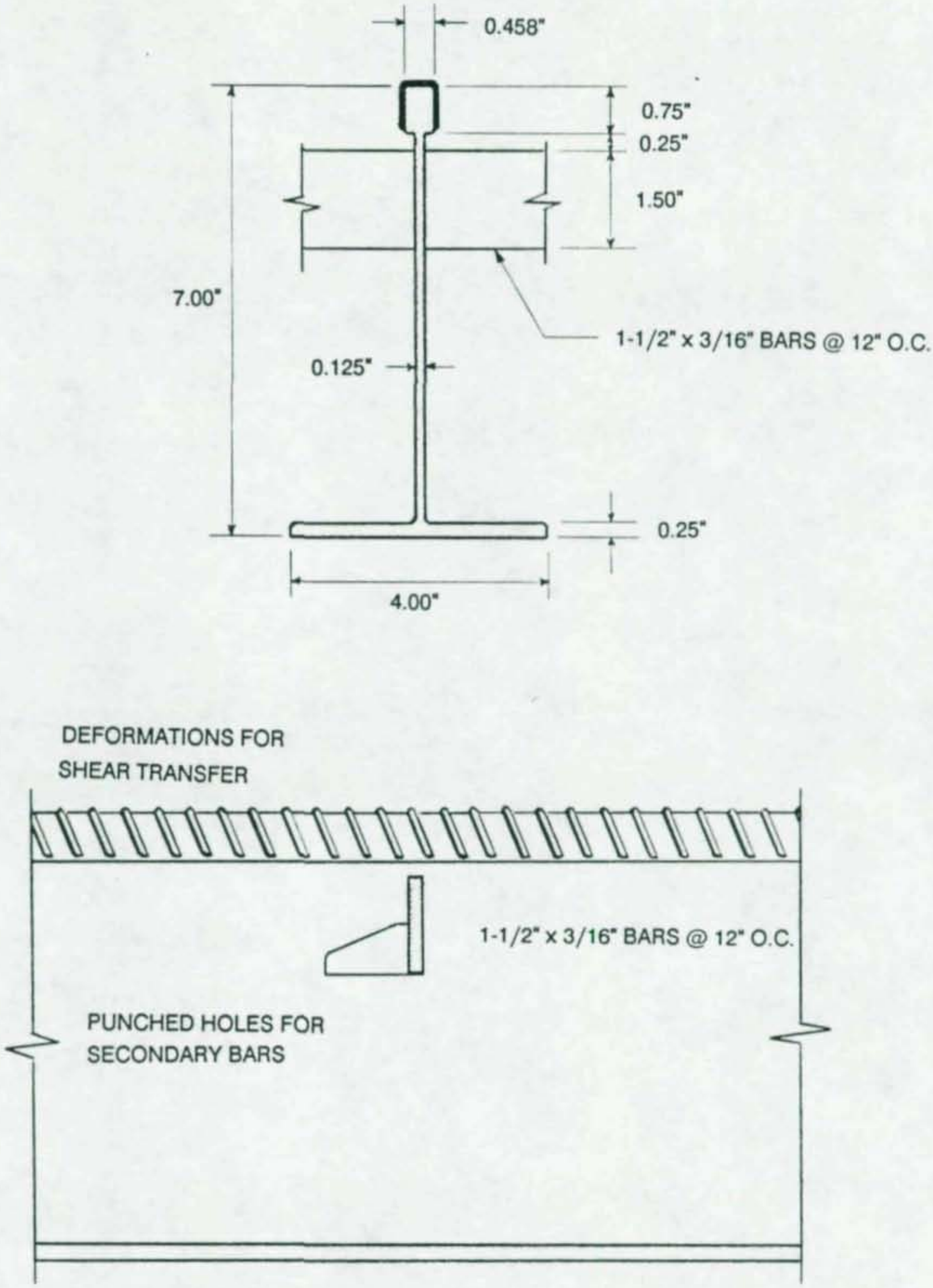


Figure 2.14a Proposed Cross Section for Long-Span Steel Grid

members, and because the entire configuration is only eight inches deep it could result in an overall reduction in the floor to floor height of the structure. Finally, by using normal weight concrete and increasing the slab depth, it would be possible to increase the spacing of the beams without a substantial increase in weight. This should result in a lower cost for the floor system.

2.7 Conclusions of Conceptual Investigation

2.7.1 Summary

If strength were the only requirement, designing lightweight floor systems would be a fairly simple task. However floor systems perform a great many functions in addition to merely sustaining gravity loads without some sort of catastrophic failure. Subsequently, a host of behavioral characteristics have to be taken into consideration to successfully design and implement any innovative lightweight floor system.

Baehre and Urschel [1984], in their extensive research in the area of lightweight floor systems, emphasize that, "The floor is not only a space-enclosing and load-bearing member, but rather a technical subsystem subjected to functional requirements...". These functional requirements are:

I Load carrying capacity
II Stiffness
III Durability
IV Fire Protection
V Sound Insulation
VI Climatic Protection
VII Room Environment
VIII Servicing Facilities

Requirements I through V are self explanatory. Included under item II, is the susceptibility of the system to vibrations induced by human occupancy. This is one of the most important functional requirements for lightweight floors and often one of the most difficult to quantify. Requirement VI, climatic protection, refers mainly to the thermal insulation properties of the floor system. These properties may be of concern depending on the utilization of the space above and below the floor. Item VII, room environment, pertains to "...those qualities that are necessary for the physiological well being of the users." Some of these qualities include surface treatment and sound absorption. Finally, item VIII is concerned with the ability to accommodate mechanical and electrical services within the system. This requirement can have a significant effect on the cost competitiveness of the floor system.

Baehre and Urschel also state that the necessity for functional requirements I through III is independent of the type and use of the building. In general, all floor systems must meet certain minimal requirements. These requirements are usually defined in the various building codes and are somewhat dependent on the intended use of the building. Additional demands to improve the quality of the floor may be specified by the users or the owners of the building [Baehre and Urschel 1984]. No matter what the total demand on the floor might be, it should be evident that no single set of standards can be defined that could apply to every floor system in every building.

One other factor that is essential to the development of any innovative lightweight floor system is that the system be cost competitive with presently available means of construction. This not only includes material costs, but fabrication, shipping and labor costs for construction. Even if the floor system is more expensive, the total cost of the structure may be reduced as a result of an overall weight or height reduction. Here again, the feasibility of a particular system is dependent on the type of building being constructed.

Finally, two other factors that affect the success of a new floor system are versatility and constructibility. Versatility is important in that the system should be able to accommodate a number of different span lengths and loadings. Secondly, if the system is difficult to construct, it will be very difficult to gain acceptance within the building trades. Both of the characteristics are very important in promoting the use of a particular floor system with both designers as well as contractors.

Because of the numerous factors that determine the performance of a particular floor system, it is difficult to rate the potential of the various concepts investigated in any numerical order. However some conclusions can be drawn based on the characteristics of the different systems. These conclusions are presented starting with the framing systems.

* * *

As stated earlier, the most economical framing system for any given floor, may not be the one with the lightest overall weight. There are many other factors which play an important part in determining the optimum floor framing plan for a given building. These include, but are not limited to:

- Fabrication costs
- Constructibility
- Stiffness (i.e. deflections)
- Susceptibility to annoying vibrations
- Ability to accommodate services
- Span length
- Overall height of floor system

It was also discovered in designing the reference floor systems used in this investigation, that the framing members compose only a small portion of the dead load of the entire system. On average the beams or joists only account for approximately 15 percent of the total weight of the floor. Assuming this is characteristic of most buildings, it can be seen that even a substantial reduction in the weight of the framing members, would only result in a minor reduction in the total weight of the structure.

This is not to say that innovative methods of framing should be completely disregarded. As the weight of the floor system decreases, strength requirements may become less predominant in design. Live load deflections and serviceability requirements could play a more important role in the selection of the framing system. With the implementation of a lighter slab component, the optimum framing system for a particular building may be redefined.

A fair amount of information is available on most of the framing members previously discussed. However two framing systems which may warrant further investigation are the folded truss concept, and the tetrahedral frames. Although the folded trusses have been used in roof systems, there doesn't seem to be any mention of them ever being used in a floor system. The tetrahedral frame is a concept that few structural engineers are even aware of. These frames may offer a whole range of potential for implementation into civil engineering structures, and should not be overlooked.

* * *

The slab component of a floor system can typically account for as much as 80 to 90 percent of the total dead load. As a result this is the component that offers the greatest potential for an overall reduction of the entire weight of a structure. All of the slab systems proposed in this investigation could have a significant weight advantage over typical profiled steel deck and concrete slabs. However it should be noted that any final determination regarding the performance of the various slab systems should not be made without experimental verification.

It may be difficult at this time to develop a lightweight floor system for steel framed buildings using fibre reinforced plastics. Unless advanced reinforcing fibres are used, deflections in FRP floors will be difficult to overcome. Many other factors could also inhibit the use of FRP components. For example, no matter what types of resins or reinforcing fibres are used, FRP's are still quite expensive. Plastics are also susceptible to degradation from ultra-violet radiation. Another problem is that the resins used consist of organic compounds which are classified as combustible materials. Subsequently achieving adequate fire ratings may be a problem.

Of all the systems constructed from fibre reinforced plastics, the pultruded deck may be the most feasible and versatile option. Although these decks would be costly to produce, they would still be less expensive than the FRP sandwich panels. The use of advanced composite materials at this time appears to be far to costly, even if they were weightless. Despite the high costs, FRP floor systems may still warrant further development for special purpose applications. These include situations where resistance to caustic environments or electrical conductivity is a primary concern.

Not much more can be said about lightweight precast panel floor systems except that they represent an innovative alternative that has been around for many years. These systems should not be discounted because of their limited popularity. In fact it may prove interesting to do further research into the strength and serviceability characteristics of these floors.

Dry board/profiled steel sheet floor systems offer a great range of potential for development of lightweight floor systems. On one extreme, double stress skin panels with a shallow steel deck core could be incorporated into joist floor systems. On the other extreme, long-span roof decking could be used to span the entire floor bay without intermediate framing girders.

One of the major draw backs to developing dry board floor systems involves obtaining an adequate fire rating due to the combustible materials used. Another difficulty arises in the analysis of these systems because of the partial composite action developed between the components. As a result development of these systems may require extensive experimental testing to verify the behavior. However, these systems offer some tremendous advantages in addition to their lightweight and should not be overlooked.

Long-span deck/concrete slab composite floor systems may be one of the better alternatives to typical floors in steel framed buildings. This concept is similar to the long-span dry board floors except that the concrete offers greater stiffness than the board components. In addition, if full composite action is developed between the concrete and the cold-formed deck, then the behavior of these systems should be fairly easy to predict. Again, one of the best advantages of this type of system is that it should be possible to construct an overall shallower floor system, thereby reducing the floor to floor height of the building. This characteristic could aid in making steel framed buildings more competitive against concrete frames utilizing thin post-tensioned flat slab floors.

Finally the development of steel grid floor systems appears to be another excellent concept. It may be possible with this type of floor, to obtain a substantial reduction in the dead load of the structure without compromising any of the performance characteristics of the system. The 4.25 in. deep grid with the continuous profiled steel sheet and concrete slab may offer the most feasible configuration for a steel grid floor. On the other hand, the long-span grid could make it possible to eliminate the need for secondary framing. As a result it would offer the same advantages as the long-span deck floor systems.

2.7.2 Recommendations

In general the slab components investigated have been developed somewhat independent of the type of framing members used for the floor system. The type of beam or joist used may be dependent on the most economical span length for the particular slab. As noted earlier, as the dead load of the entire floor system is reduced, alternate forms of framing members other than wide-flange beams and open-web joists may become more effective.

It should also be noted that the long-span cold-formed deck, and the long-span steel grid result in floor systems in which the framing and the slab act as a composite unit. Subsequently, secondary framing members are of no concern.

Table 2.1, provides somewhat of a quantitative comparison for some of the innovative floor systems investigated. The basis of comparison is the average reference floor system. The unit weights for the various systems include the weights of both the slab and framing components. The %SLAB value is the percentage of the total weight attributed to the slab component. The vibration characteristics for the proposed systems have been calculated using the mathematical models presented in Technical Digest No.5 of the Steel Joist Institute [Galambos undated] as well as the perceptibility criterion developed by Murray [1981]. The required damping for an acceptable floor system is shown as D_{REQD}

As indicated in Table 2.1, it is possible that some of the proposed lightweight floor systems may actually perform better than the reference floors with respect to vibrations. However the results of these vibration analyses should be verified by experimental testing to make an accurate comparison.

Overall, the systems which have the most potential for further development at this time, are the long-span deck/concrete slab concept and the steel grid floor systems. Both of these systems offer the advantage of lighter weight compared to present methods of construction. Although they do not necessarily represent the lightest alternatives investigated, it is believed that these two floor systems offer the best performance with respect to the various functional requirements.

Table 2.1 CHARACTERISTICS OF PROPOSED SYSTEMS

SYSTEM ANALYZED (Single Bay 30 ft x 30 ft)	WEIGHT PSF	WEIGHT %SLAB	VIBRATION RESPONSE f_1 (Hz)	VIBRATION RESPONSE A_o (in)	VIBRATION RESPONSE D_{REQD} (%)
RFS AVERAGE	42.86	85.1	5.68	0.0134	5.16
FRP Pultruded Deck	19.77	73.9	8.68	0.0190	8.14
FRP Deck w/2% steel reinf.	19.77	73.9	8.68	0.0150	7.14
Long-Span Deck/Conc. Slab	24.91	64.4	5.04	0.0190	5.84
Steel Grid (18in x 12in)	23.52	79.6	5.15	0.0150	5.13
Long-Span Steel Grid	36.01	80.2	4.20	0.0150	4.64

CHAPTER III

EXPERIMENTAL INVESTIGATION

3.1 Prototype Testing of Long-Span Deck/Concrete Slab Composite Floor

The initial phase of this investigation was to the development of conceptual ideas for innovative lightweight floor systems. The last phase addresses the experimental investigation of the floor system consisting of long-span cold-formed deck with a composite concrete slab.

The main purpose of the experimental portion of this research project was to build a full scale model of the floor system in order to evaluate its performance. The model was constructed as a single bay 30 ft. by 30 ft. floor, the same dimensions used throughout the investigation. This makes it possible to compare the experimental data more accurately with the results from the theoretical analysis.

The primary emphasis during the experimental phase was to examine the behavior with respect to serviceability criteria, namely elastic deflections and susceptibility to annoying vibrations. The intent was to load the structure to the design live load only. No attempt was made to actually load the floor systems to failure at this juncture.

3.1.2 Test Setup

The test floor to be constructed was the long-span deck/composite concrete slab system discussed in Section 2.5. The exact configuration of the system consisted of 30 sections of 7.5 in. 14 ga. long-span deck interlocked in an inverted hat position. Placed transversely across these sections was 900 sq.ft. of 9/16 in. 28 ga. form deck. Finally, on top of the form deck a concrete slab was placed with a total depth of 2 in. The concrete used for the slab was 4000 psi normal weight concrete. The entire floor system was simply supported on all four sides by 8 in. thick masonry walls with dimensions of 30 ft. by 30 ft. out to out, Figures 3.1 and 3.2. It was felt that this would more accurately simulate the continuity the floor system would have if it was actually constructed in a multi-bay building.

Once the masonry walls were completed the actual assembly of the floor system went quickly. All of the long-span deck sections were erected in five hours by two unskilled laborers. The placement of these sections was also complicated slightly by the fact that they were being assembled upside down of the way they were designed to interlock.

All of the 9/16 in. form deck was placed and attached by one laborer in a total of six hours. The deck was attached to the top flanges of the long-span deck sections using self-tapping, self-drilling screws. A total of 1085 screws were used. This unusually large number of screws was required to develop enough capacity to transfer the shear from the long-span deck sections to the concrete slab.

Most of the screws used were standard 12-14, 1-1/4 in. long. However the last six rows of screws at each end of the floor system were of the stand-off type. These screws consisted of a 12-14 2-1/4 in. long with a 1-1/4 in. steel sleeve

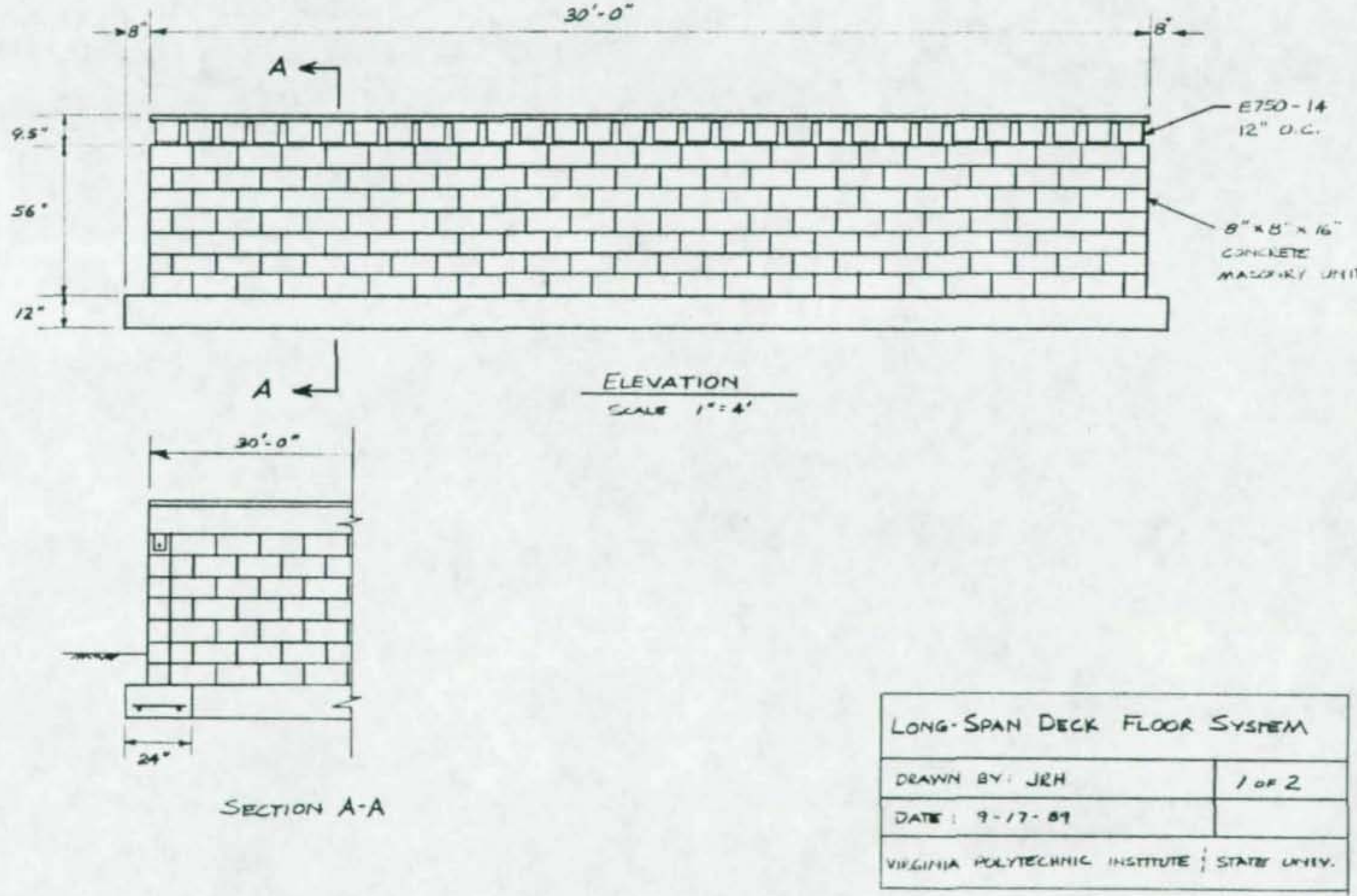


Figure 3.1 Test Setup (Elevation)

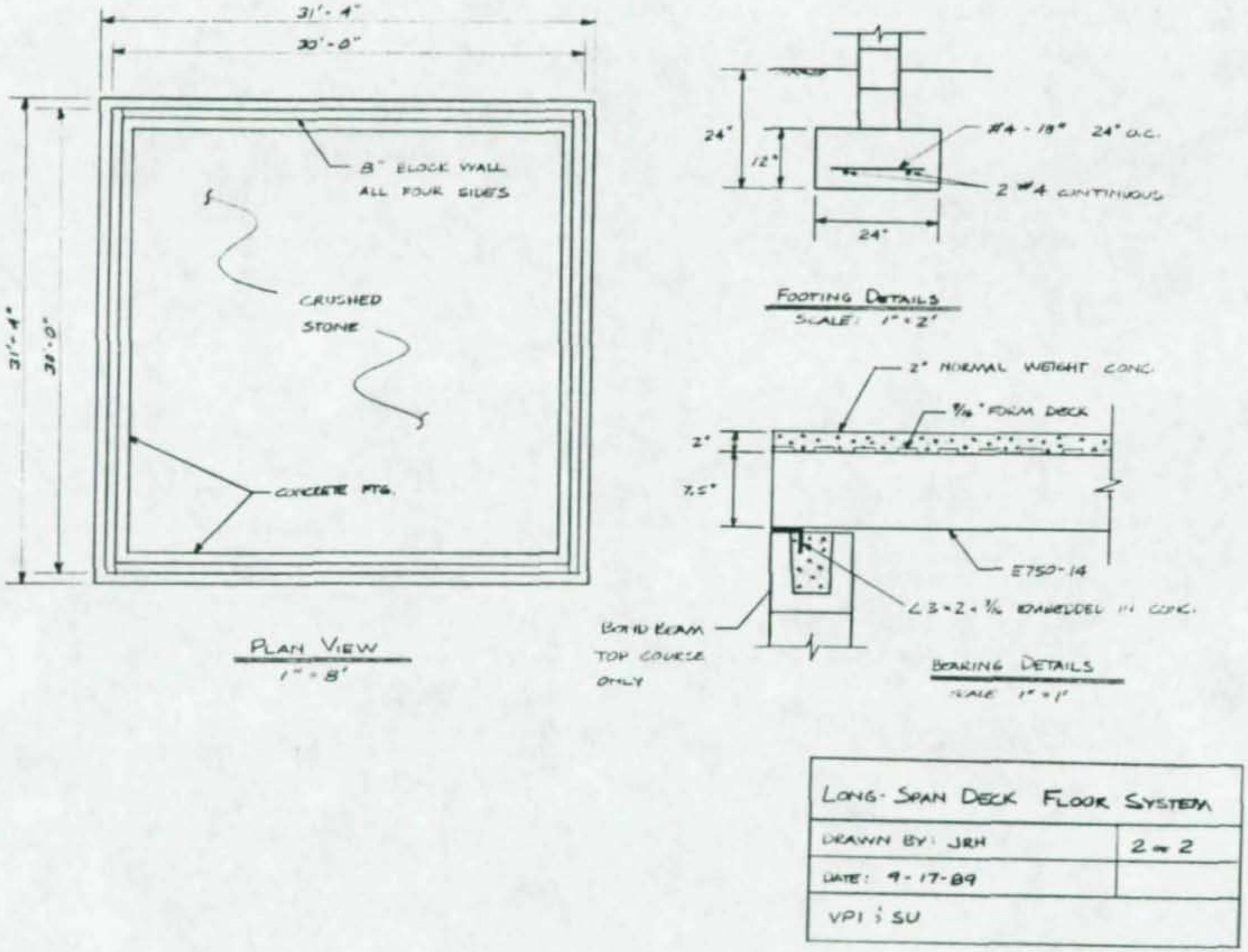


Figure 3.2 Test Setup (Plan View)

placed over the shank such that a significant portion of the screw sticks up above the form deck. As a result these screws are embedded in the concrete slab and behave as shear studs, Figure 3.4 . The spacing of the shear studs is as follows, starting from the edge of the pour stop and going in the longitudinal direction of the long-span deck:

- one 2-1/4 in. @ 3.25 in.
- one 2-1/4 in. @ 7.25 in.
- four 2-1/4 in. @ 10.0 in.
- ten 1-1/4 in. @ 10.0 in.
- two 1-1/4 in. @ 12.5 in.

Figures 3.3 thru 3.5 show photographs depicting the construction of the test floor during various phases.

The theoretical values for the midspan deflection of the floor system were calculated considering both one way and two way bending action. For one way action the live load deflection was calculated assuming a simply supported beam using the equation:

$$\Delta = \frac{5w\ell^4}{384EI}$$

Using this equation the midspan deflection for a 30 ft. span with a 70 psf superimposed live load was calculated to be 0.82 in. This is well below a limiting value of 1.0 in. considering span/360.

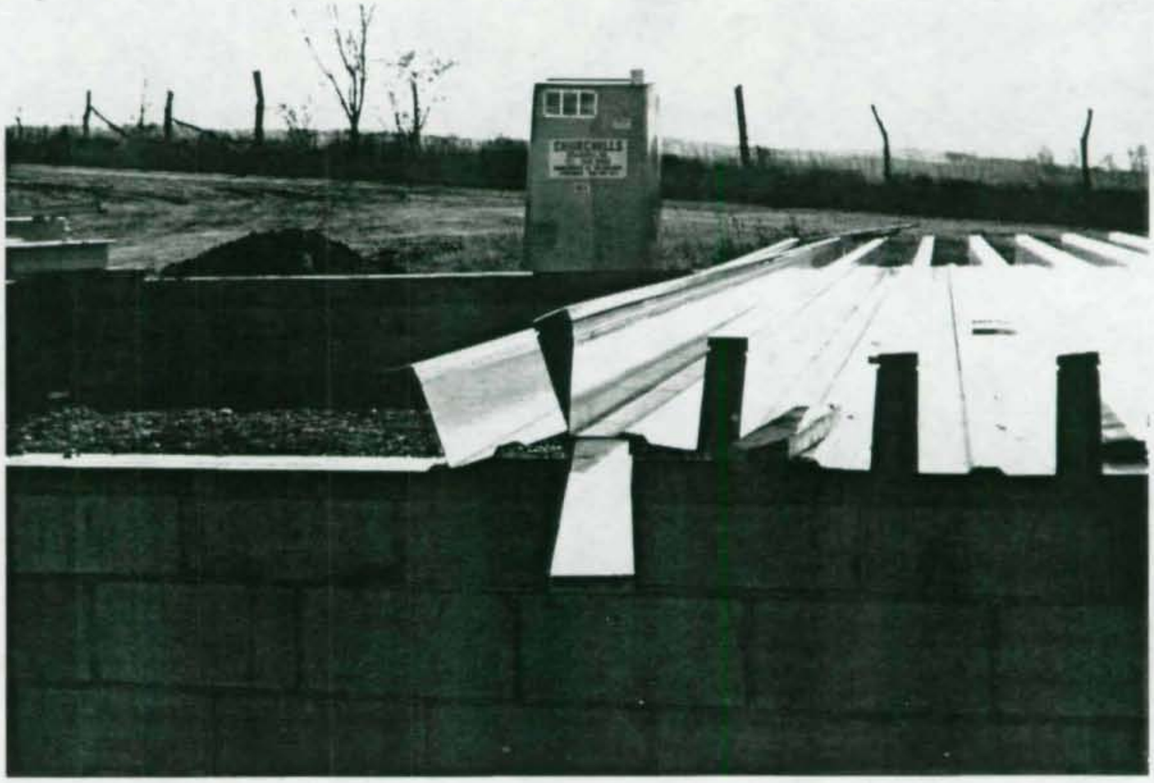


Figure 3.3 Erection of Long-Span Deck Sections

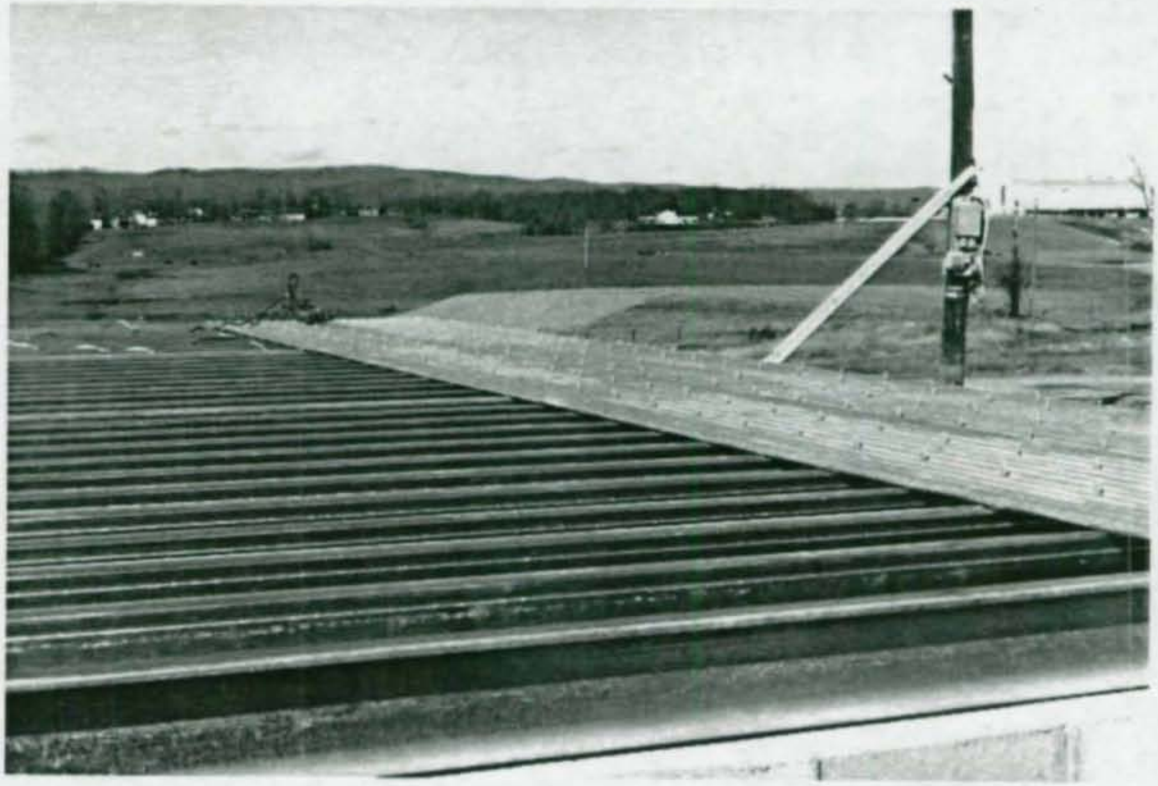


Figure 3.4 Construction of Form Deck

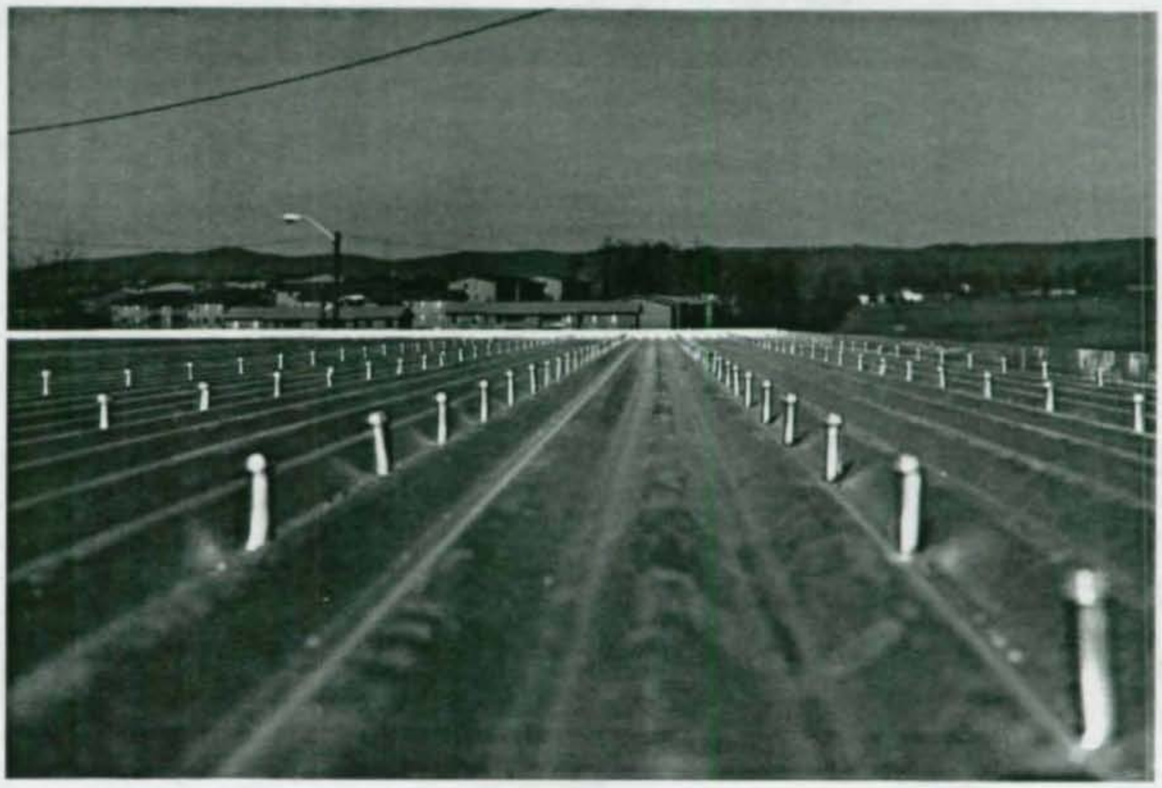


Figure 3.5 Self-Tapping Shear Stud Screws

The midspan deflection based on two way bending was calculated using Navier's solution for the deflection of a rectangular orthotropic plate, Figure 3.6, simply supported on all four sides. The equation is as follows [Szilard 1974]:

$$w(x,y) = \frac{16q_o}{\pi^6} \sum_m \sum_n \frac{\sin\frac{m\pi x}{a}\sin\frac{n\pi y}{b}}{mn\left(D_x\frac{m^4}{a^4} + 2B\frac{m^2n^2}{a^2b^2} + D_y\frac{n^4}{b^4}\right)}$$

Where $$D_x = \frac{E_c h^3}{12(1-\nu_{xy}^2)} + \frac{E_c h e_x^2}{(1-\nu_{xy}^2)} + \frac{E_{st} I_{ox}}{c_1}$$

is the flexural rigidity in the direction of the long-span deck

$$D_y = \frac{E_c h^3}{12(1-\nu_{xy}^2)} + \frac{E_c h e_y^2}{(1-\nu_{xy}^2)} + \frac{E_{st} I_{oy}}{c_1}$$

flexural rigidity transverse to the long-span deck

$$B = \frac{E_c h^3}{12(1-\nu_{xy}^2)} + \frac{G_{xy}}{6}\left(\frac{12\,A^2}{\Sigma\, b/t}\right)$$

torsional rigidity

The equation for torsional rigidity, "B", was modified assuming that the polar moment of inertia for the long-span deck sections can be calculated as a closed section rather than approximated as an open section.

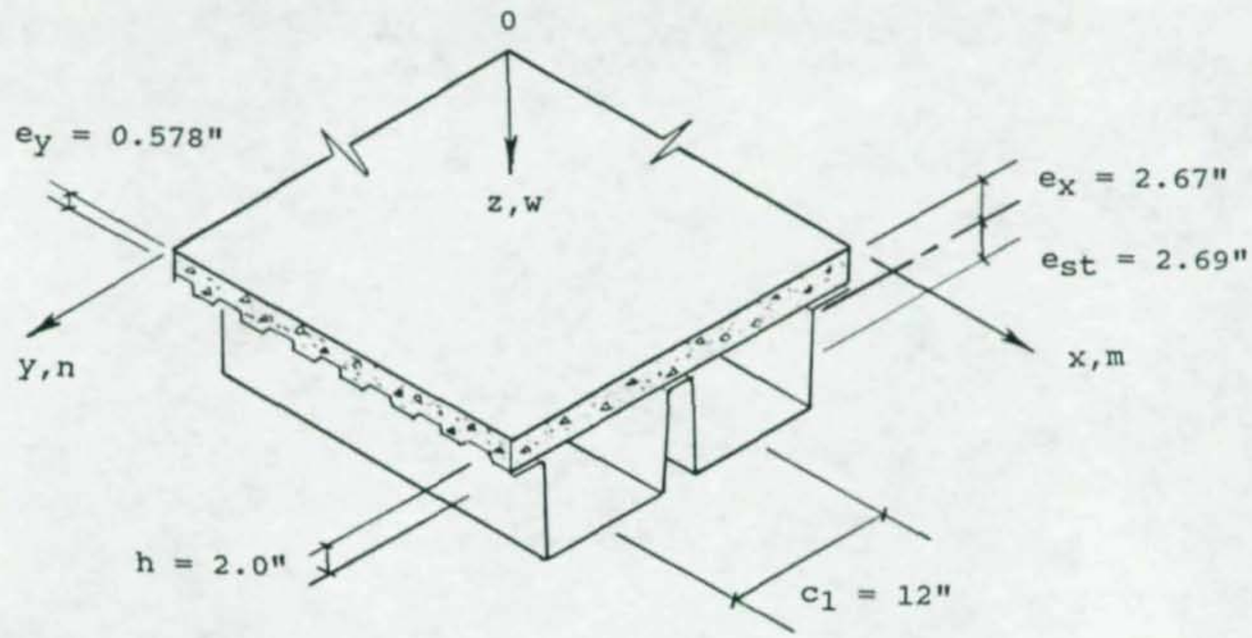


Figure 3.6 Model of Orthotropic Plate

From this two dimensional analysis the midspan deflection for the floor system was calculated to be 0.75 in. using the first 9 terms of the series expansion.

The theoretical vibration response of the system was calculated assuming the system acts as a single beam with a distributed mass. The theoretical natural frequencies of the system were also calculated assuming orthotropic plate behavior using the following equation [Szilard 1974]:

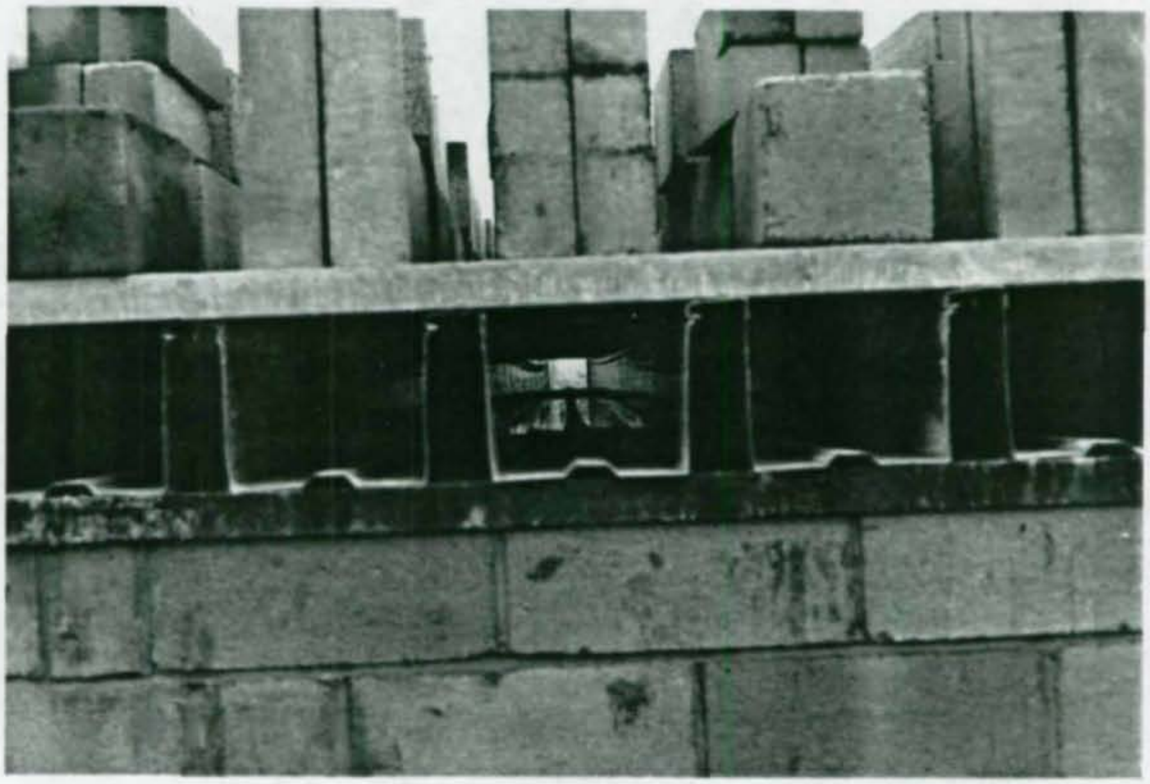
$$\omega_{mn} = \frac{\pi^2}{a^2} \sqrt{\frac{1}{m}} \sqrt{D_x\left(\frac{mb}{a}\right)^4 + 2B\left(\frac{mb}{a}\right)^2 n^2 + D_y n^4}$$

A more detailed presentation of the calculations used in designing the experimental floor system are contained in appendix B.

3.1.2 Test Results & Conclusions

On January 23, 1990 the experimental floor system was tested. The floor was loaded with 8"x8"x16" solid concrete blocks with an average unit weight of approximately 64 lbs, Figure 3.7. The blocks were applied in seven increments. Each load increment consisted of 140 blocks placed in a symmetrical pattern on the floor to simulate a uniformly distributed load. Each increment added an additional 10 psf to the floor. The only exception was the final load increment which had to be limited to 5 psf due to the amount of block available. The final load on the floor system was approximately 65 psf. This is just 5 psf short of the intended design live load of 70 psf.

Figure 3.7 Load Test of Experimental Floor



To monitor deflections during the loading four displacement transducers were installed below the floor system. The locations of the transducers are indicated on Figure 3.8 . The behavior of the system was found to be linear with some slight deviations. These deviations are most likely attributed to the weight of the individual blocks varying by as much as 14 lbs. Consequently every load increment may not have been exactly 10 psf.

The actual deflections were slightly greater than calculated for both beam action and plate action. This may be attributed to items such as variability in the modulus of the concrete or potential slippage between the deck and slab components. The experimental vs. theoretical values for the deflection at center slab can be seen Figure 3.9 . The final deflection at 65 psf was 0.80 in, which is just slightly larger than the predicted values of 0.696 in. for plate behavior and only 1/32 in. greater than 0.767 in. for beam behavior.

The other serviceability limit state checked during the test was susceptibility of the floor system to annoying vibrations induced by human occupancy. The vibrations were measured using a Wilcoxon Research Model 731 seismic accelerometer with a Model P31 amplifier. The digital signals were collected using a lap top computer. The method used to induce the vibrations was the "heel-drop" impact. This is performed by a man standing with his heels raised approximately 1.5 in. off of the floor then relaxing to impact the floor.

Four floor vibration measurements were taken after each load increment was completed. Two of these measurements were taken with the accelerometer placed at the center of the floor system (center bay). The other two were taken with the accelerometer placed at center span, 7.5 ft from the edge of the slab

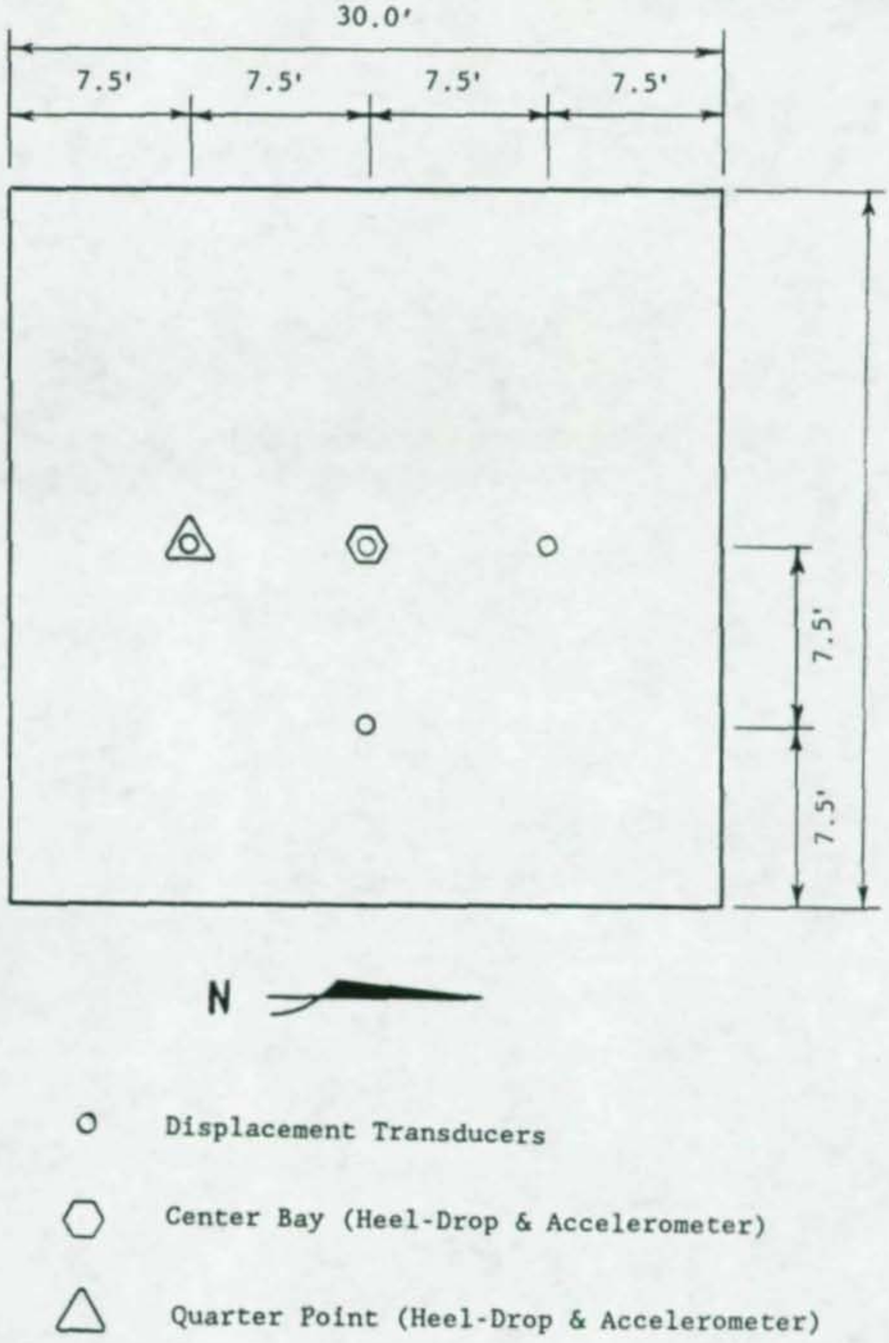
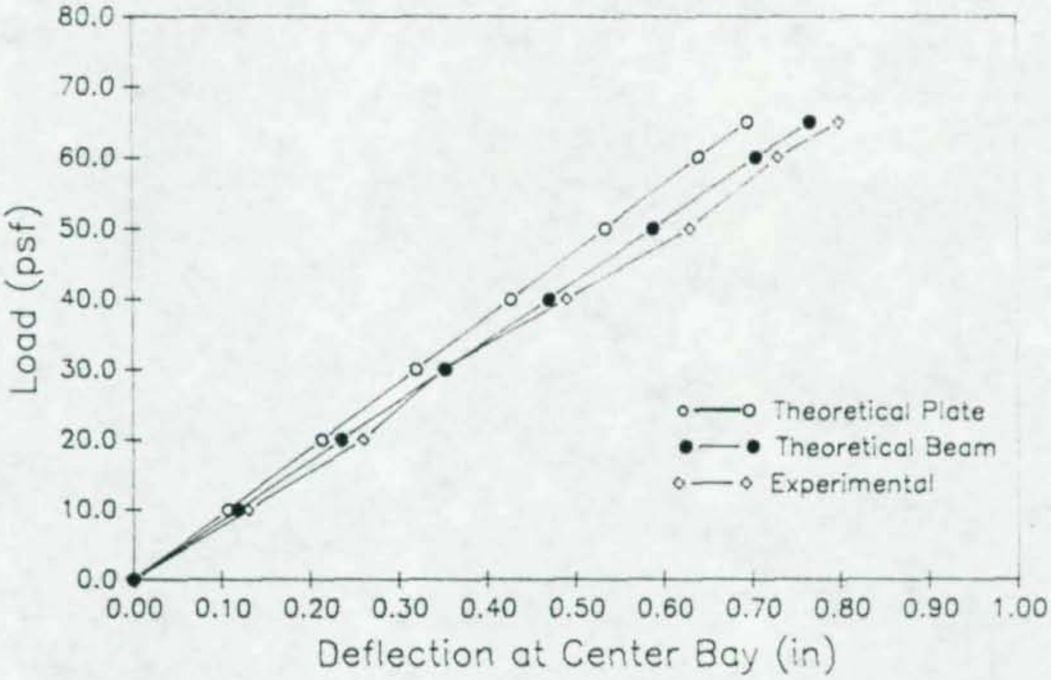


Figure 3.8 Layout of Data Acquisition Equipment



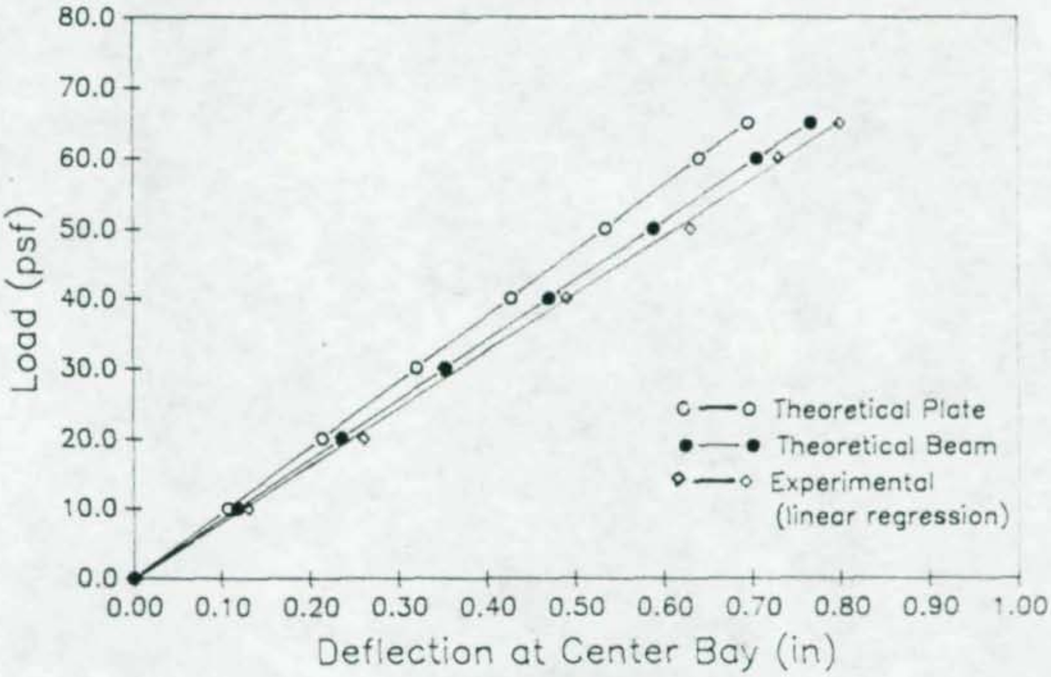


Figure 3.9 Center Bay Deflection

(quarter point). In all instances the heel-drop impact was performed directly next to the accelerometer, Figure 3.8 .

Once the vibration measurements were recorded the natural frequencies were determined by processing the data using a Fast Fourier Transform (FFT) algorithm. The experimental first natural frequency for each load increment is shown on Figure A.1 along with the first natural frequencies determined using both the SJI model and the orthotropic plate model. As seen on this plot the measured frequencies are relatively close to the theoretical values. As such it appears to be possible to predict the first natural frequency of this type of floor system with reasonable accuracy for use in vibration perceptibility analysis.

The data collected by the accelerometer gives acceleration vs. time. In order to analyze the floor using the perceptibility criteria, the accelerations must be converted to displacements, i.e. amplitudes. This is done using the peak accelerations for each cycle of the acceleration vs. time plots and integrating twice. However these peak values can be difficult to distinguish when seen on the original unfiltered plots, Figures A.2 to A.9 . To overcome this, the plots were improved by using the FFT program again to filter out all unwanted frequencies higher than the desired frequency.

Normally this works quite effectively for typical floor systems. However when viewing the power density spectrums, Figures A.18-A25, for the vibration measurements at center bay it is evident that this floor system actually has three distinctive frequencies contributing to the energy in the system. In fact it can be seen that with zero live load the higher frequencies are actually be more pronounced than the first natural frequency. Consequently these frequencies make significant contributions to both the accelerations and the amplitudes. This presents difficulties in that filtering out these higher modes, results in reducing the accelerations. As seen on the filtered traces, Figures A.10 to A.17, filtering the higher frequencies can reduce the accelerations by as much as five times. On the other hand if the higher frequencies are left in it is difficult to obtain a smooth filtered plot to use for calculating amplitudes and damping.

Although it may be possible to isolate the individual contributions of each prominent frequency, one problem still remains. The perceptibility criteria ($D > 35A_of_1 + 2.5$) is based on one distinctive frequency and one peak amplitude corresponding to that frequency. As such the inequality cannot accurately be used when more than one frequency is present.

The vibration characteristics were similar when the floor was impacted at the quarter point. The first frequencies matched the first frequencies when impacted at center bay. However the second frequency obtained by impacting at the quarter point was not evident when impacting at center bay. Moreover when impacted at the quarter point the second frequency was inevitably stronger for all load increments. Subsequently impacting at the quarter point presents the same problem as impacting at center bay. It is not possible to filter out the higher modes of vibration without significantly reducing the accelerations. Neither is it possible to obtain a clear plot with both frequencies present. This can be seen in the filtered plots coinciding with quarter point impacts as seen in Figures A.34 to A.41. In these plots only the frequencies above the second mode have been filtered out.

Despite the problems presented by the additional frequencies present, it is possible to draw some conclusions with regard to the vibration characteristics of this floor system. It was evident from both the unfiltered and the filtered plots that as the load increases, both the frequencies and the accelerations are reduced. In the inequality developed by Murray [1981], if both the frequency and the amplitude decrease, then the amount of damping required for the floor system would decrease with load.

It can also be seen that as the load increases the time required to damp out the vibrations is increased. This is due to the fact that the weight of the block provides more energy to the system that must be absorbed before the vibrations can subside [Lenzen 1966]. Lenzen also found that simple loads other than humans do not increase damping.

In conclusion, it would not be accurate to use presently available models to analyze this type of floor system with regard to vibrations. Although it is does appear to be possible to quantify the behavior of the floor, both theoretically and experimentally, the results cannot be compared to any available data base. The measurements obtained cannot justify wether or not the floor system will be acceptable to human occupants with regard to vibration. This can only be determined from a series of subjective evaluations.

3.1.3 Recommendations

Although it was possible to construct a prototype floor system from presently available materials, there are still a few details that must be investigated to develop this concept into a marketable floor system. The following is a discussion of some of these concerns.

- One of the crucial details necessary to make this a viable alternative to typical floor systems is the support conditions. It would be advantageous to the marketability of this floor system to develop connection details that are simple and exploit the shallow depth characteristics of this floor system. Two potential connection details are shown on Figures 3.10 and 3.11.

- Another consideration is ultimate strength. Before using this type of floor system it is inevitable that a series of load tests be conducted to determine the limit states.

- In conjuction with the ultimate strength tests, special consideration should be given to further development of the shear connection details used to provide composite action. The shear capacities of the screws and the form deck were not verified prior to construction of floor system. However, it appears that the screws in particular may have greater shear strength than anticipated. It may also be desirable to use steeper webs on the 9/16 in. form deck.

- A further strength consideration of this floor system would be to evaluate the ability of the system to provide diaghram action.

- It should also be noted that the deck sections used in this test were constructed upside down of their normal use. In the long run it may be advantageous to modify the shape of the section to simplify the erection of the deck. In doing this it may also be possible to further optimize the performance of the deck sections. For example by placing the interlocking edges in the bottom flanges it would be possible to locate more steel in the tension zone, which would be more efficient when considering composite

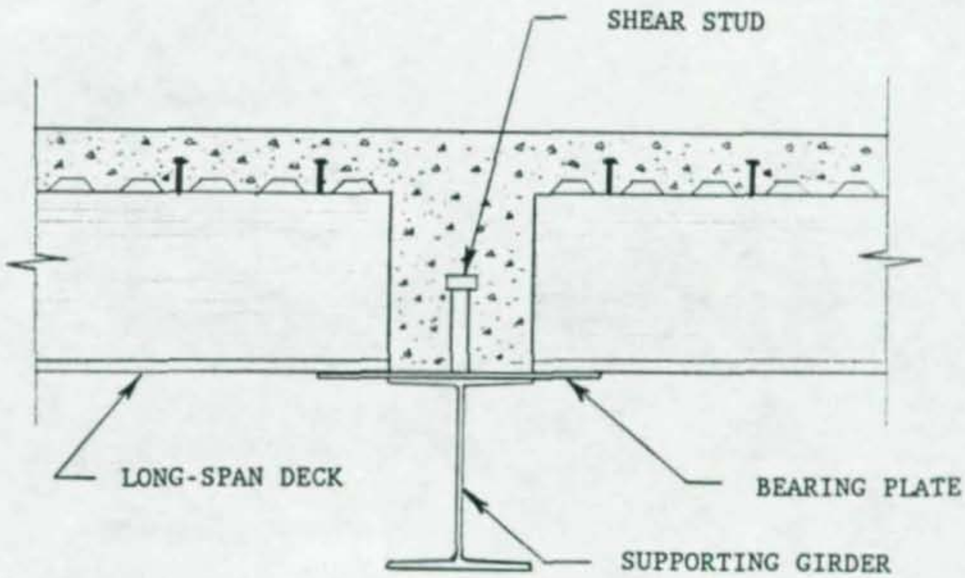


Figure 3.10 Proposed Connection Detail (Composite Girder)

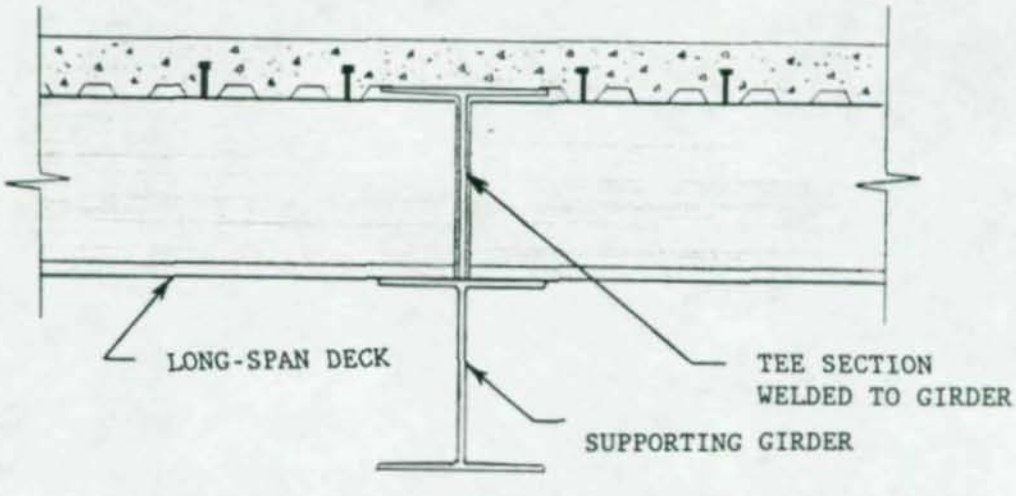


Figure 3.11 Proposed Connection Detail (Stub Girder)

action. The other advantage of interlocking the bottom flanges rather than the top is that the friction between the sections may offer more damping than when connected at the top flanges. Figure 3.12 shows one recommended alternative section to the long-span deck used in the test.

- It will also be necessary to determine the fire ratings of these floors prior to use in building construction. For this consideration it might be more desirable to use a lightweight concrete slab with a total thickness of 2-1/2 in. rather than the 2 in. normal weight concrete used in the test. This may also improve the vibration characteristics of the floor system.

- Finally an effort should be made to fully utilize the potential benefits of this type of floor system. This includes investigating the possibility of incorporating mechanical and electrical services into the floor system.

Even if this floor system exhibited no weight reduction, it would still have tremendous potential for building construction. With the use of this system it should be possible to construct a lighter structure with shallower story heights, both of which can add up to savings. The system is also easy to erect and requires no shoring. Finally, it may be the only system investigated that could result in a comprehensive floor system where a suspended ceiling would not be necessary to provide an aesthetically pleasing finish.

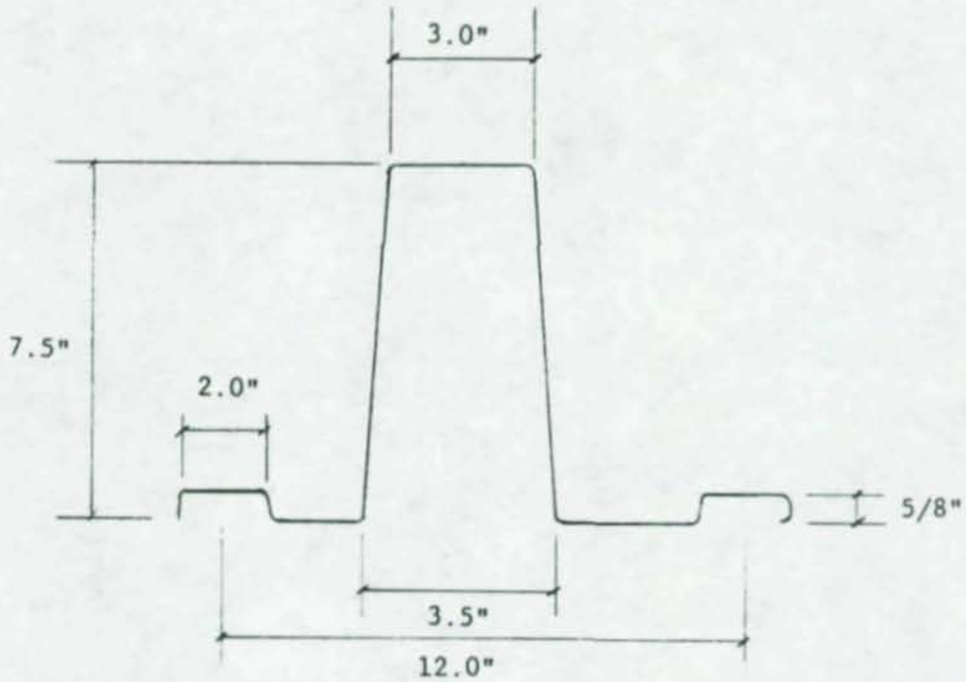


Figure 3.12 Recommended Deck Cross Section

CHAPTER IV

CHANGES ARE NOT PERMANENT, BUT CHANGE IS

4.1 Summary

Throughout history engineers have sought to improve the technology of their profession. Sometimes this is done by improving the strengths of the materials used and methods of fabrication. Others times it is accomplished by refining the methods of analysis available and by using computer applications to expedite the analysis.

Subsequently, the intent of this research was not just to investigate new lightweight floor systems. Rather the intent was to identify innovative alternatives to current building technology that would result in structures that are more efficient and economical. The goal was to accomplish this without making the systems more difficult to analyze or construct, and without compromising any of the requirements necessary in making the structural system safe and comfortable to human occupants.

The suggestions for future research are as limitless as the human imagination. For the most part this investigation has probably introduced more problems than it has solved. Several of the floor systems proposed warrant further investigation, particularly if they are ever to be incorporated into a building structure. An effort should also be made to continue to search for additional innovative concepts. No one individual holds a monopoly on creativity.

As a final note, floor systems are only one portion of building structures that can benefit from a conceptual investigation such as this. As long as the challenge exists to build lighter, taller or longer spanning structures, the engineer will be forced to investigate creative solutions to facilitate their design and construction. This investigation presents the ground work for some potential solutions to one of these problems.

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APPENDIX A

VIBRATION TEST RESULTS

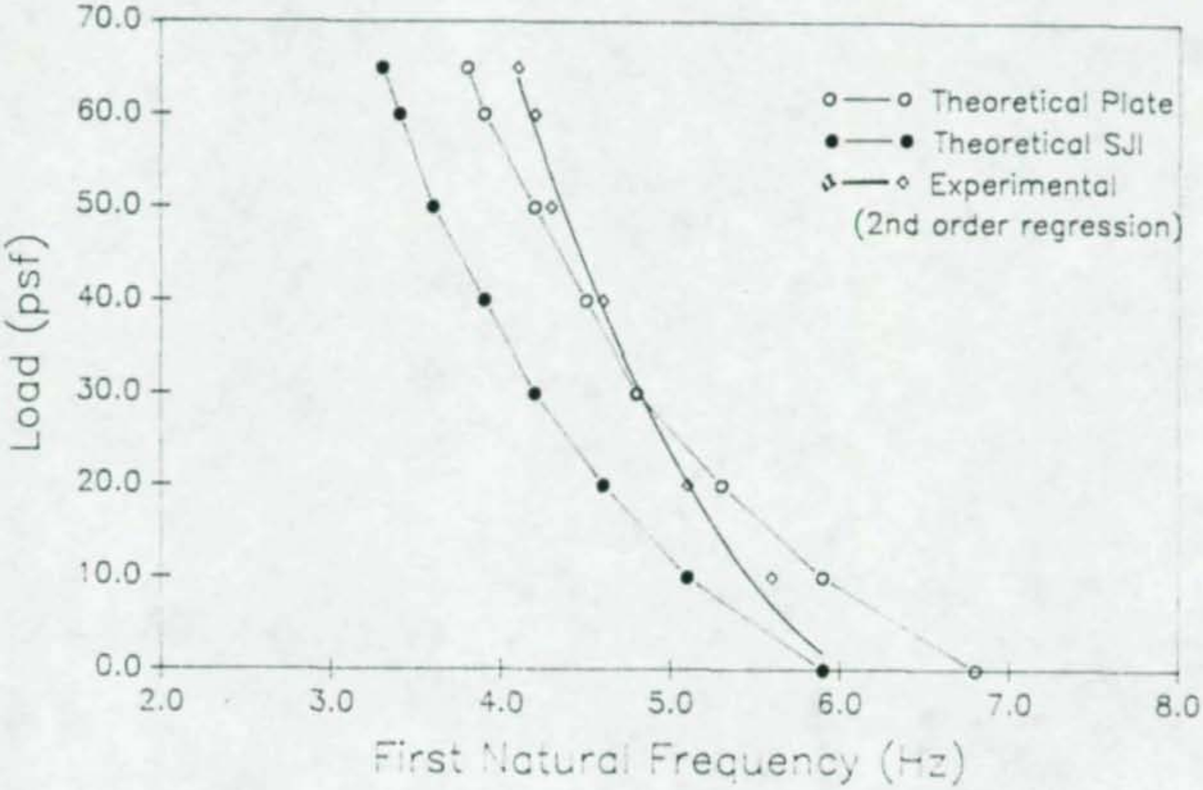

Figure A.1 First Natural Frequency vs. Load

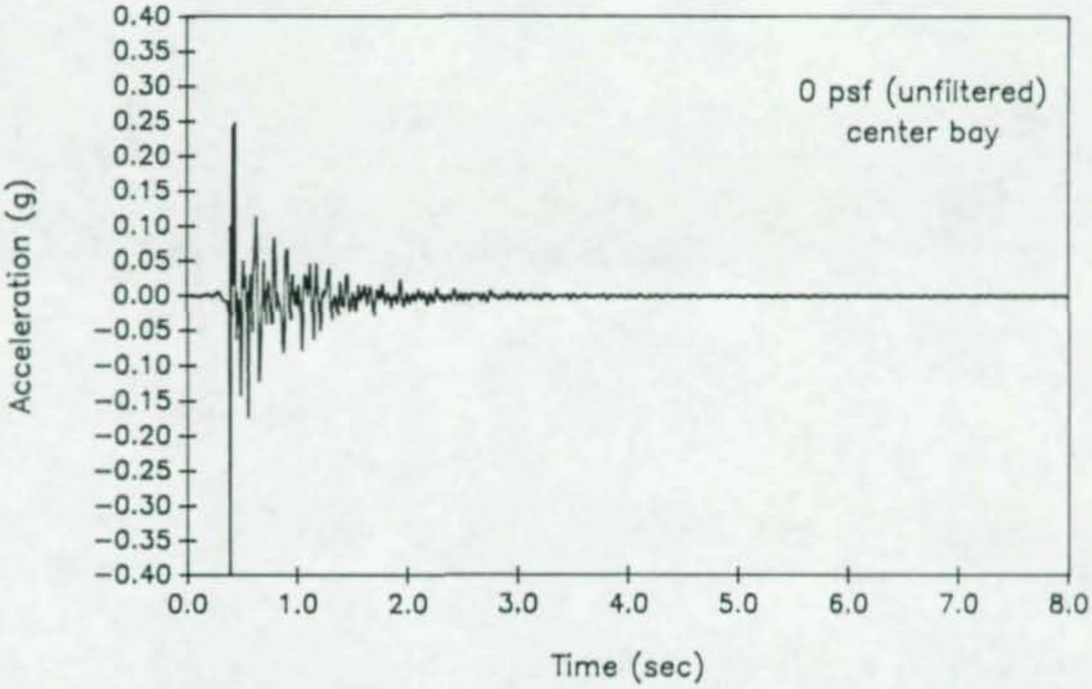

Figure A.2 Time vs. Acceleration @ 0 psf (unfiltered)

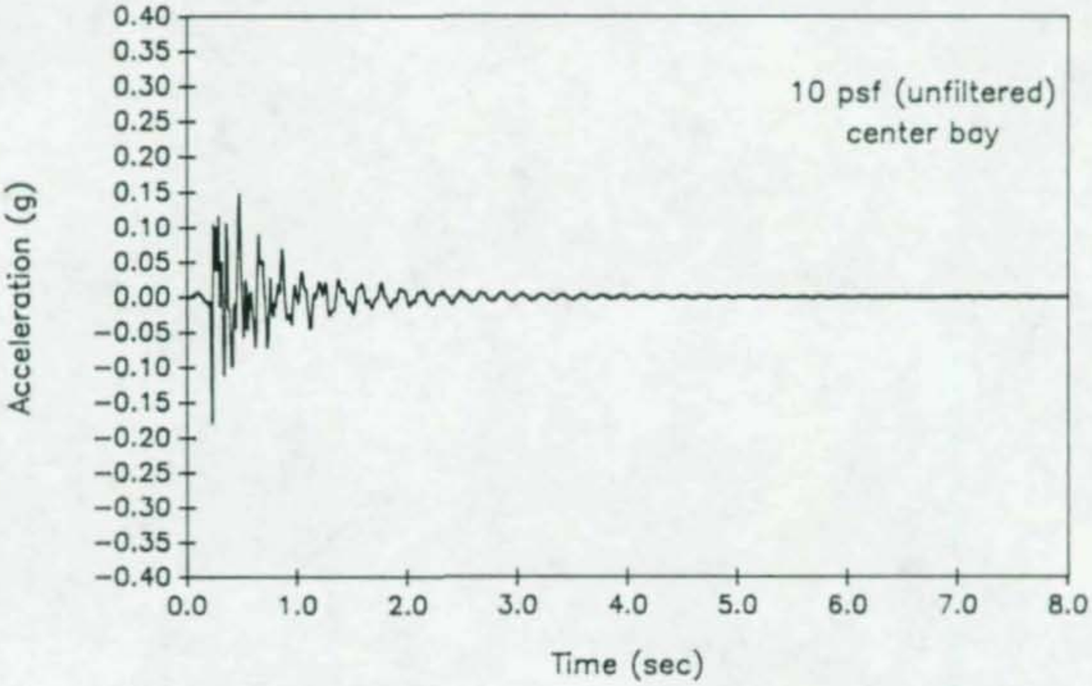

Figure A.3 Time vs. Acceleration @ 10 psf (unfiltered)

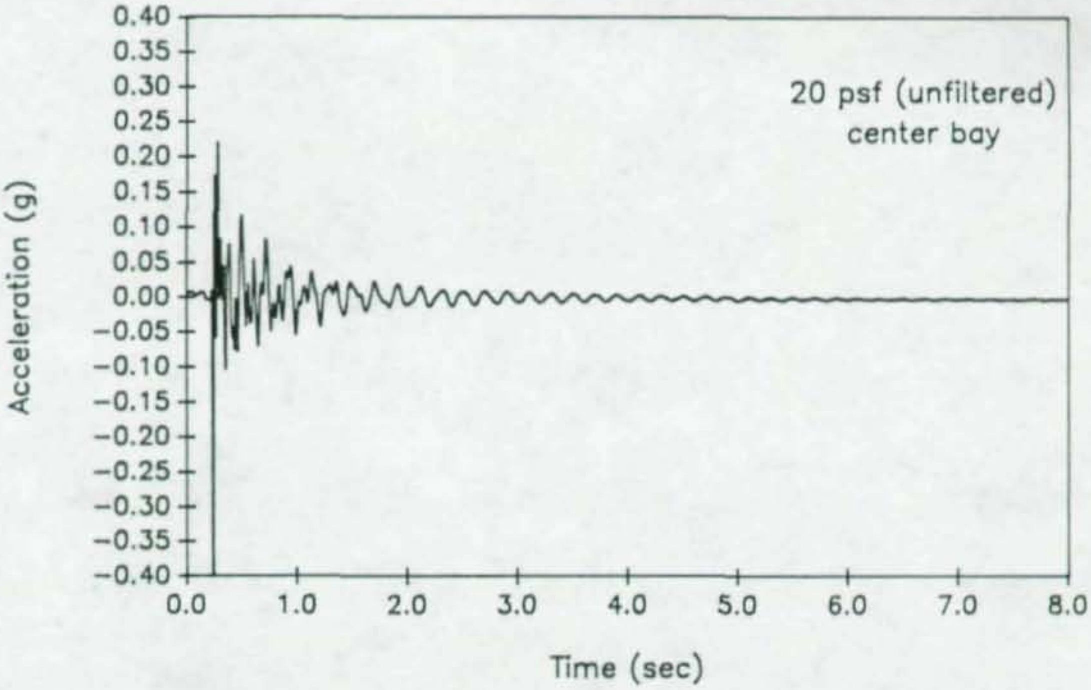

Figure A.4 Time vs. Acceleration @ 20 psf (unfiltered)

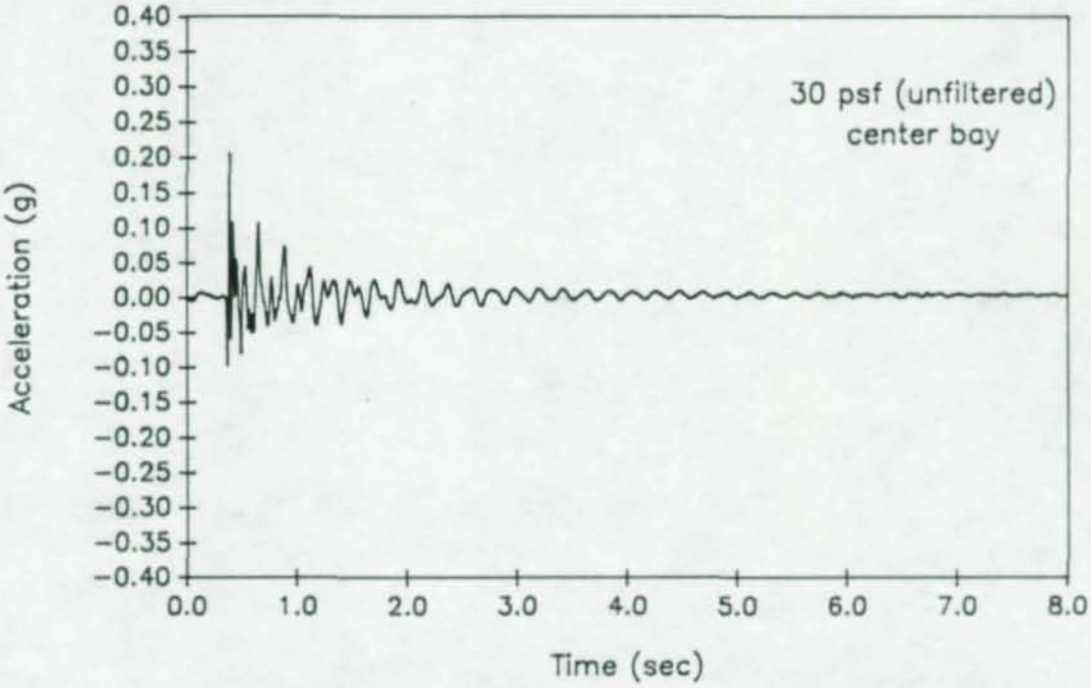

Figure A.5 Time vs. Acceleration @ 30 psf (unfiltered)

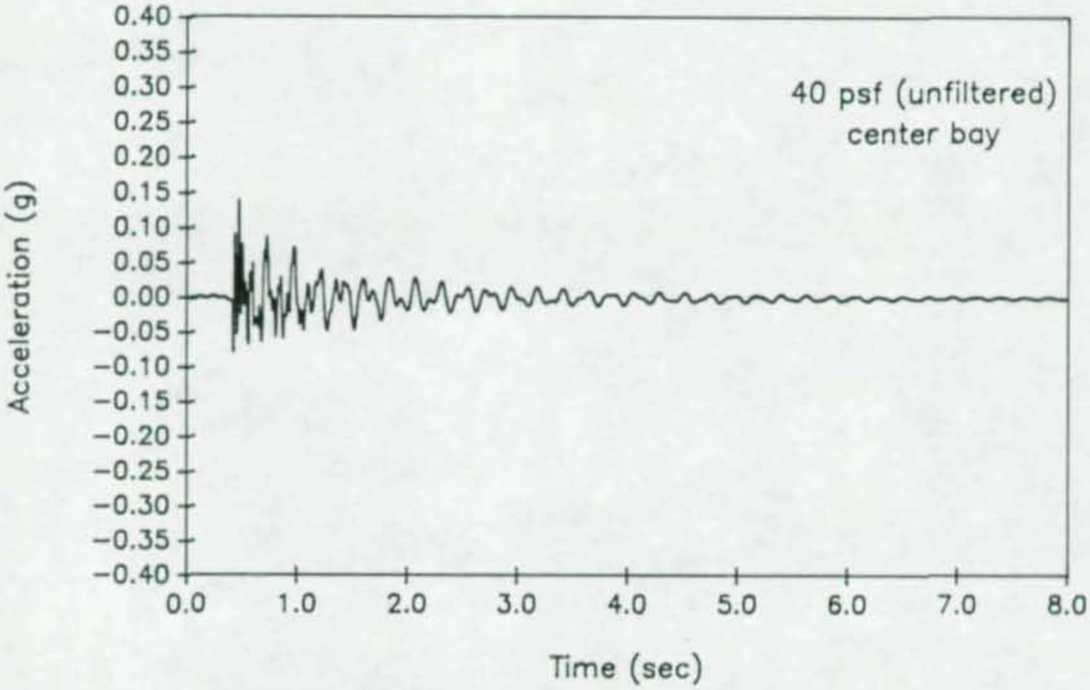

Figure A.6 Time vs. Acceleration @ 40 psf (unfiltered)

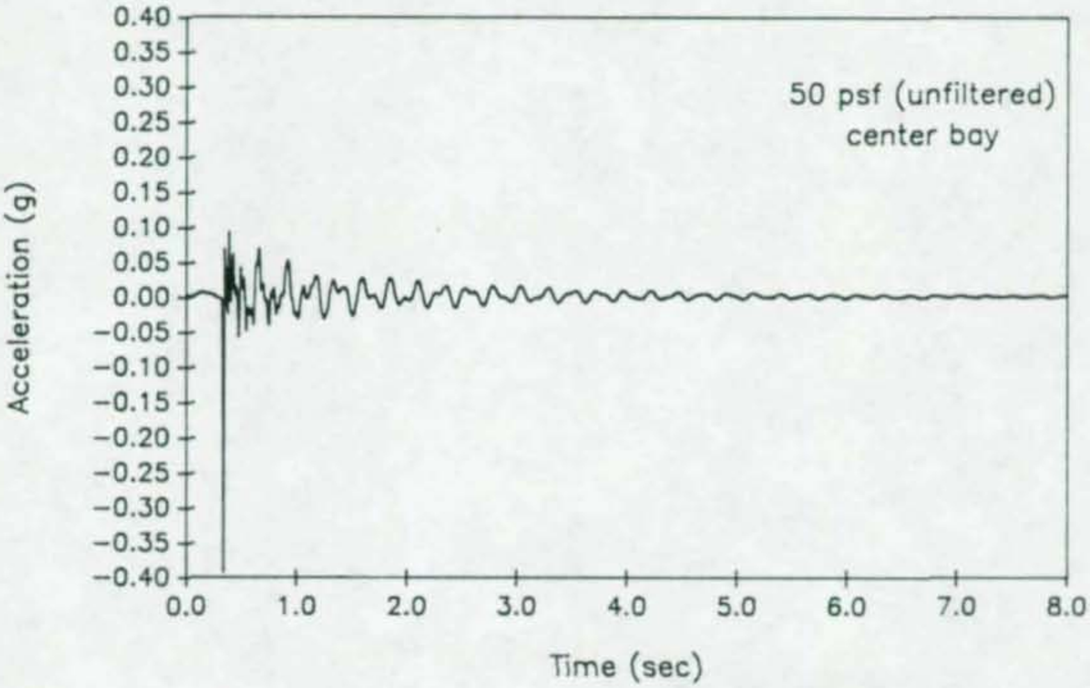

Figure A.7 Time vs. Acceleration @ 50 psf (unfiltered)

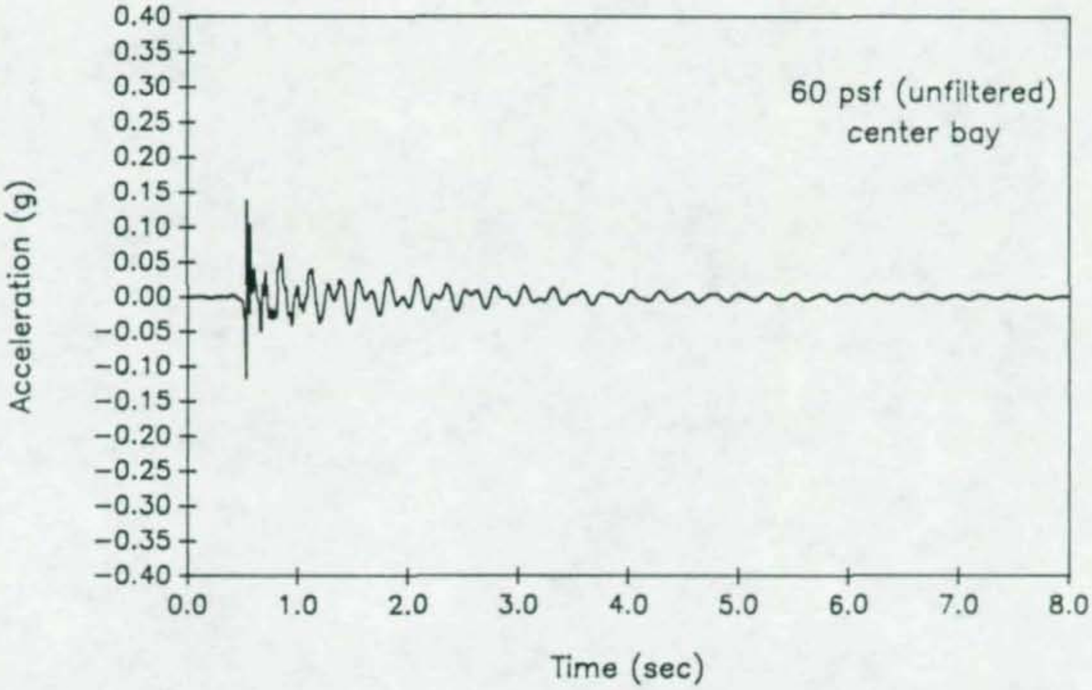

Figure A.8 Time vs. Acceleration @ 60 psf (unfiltered)

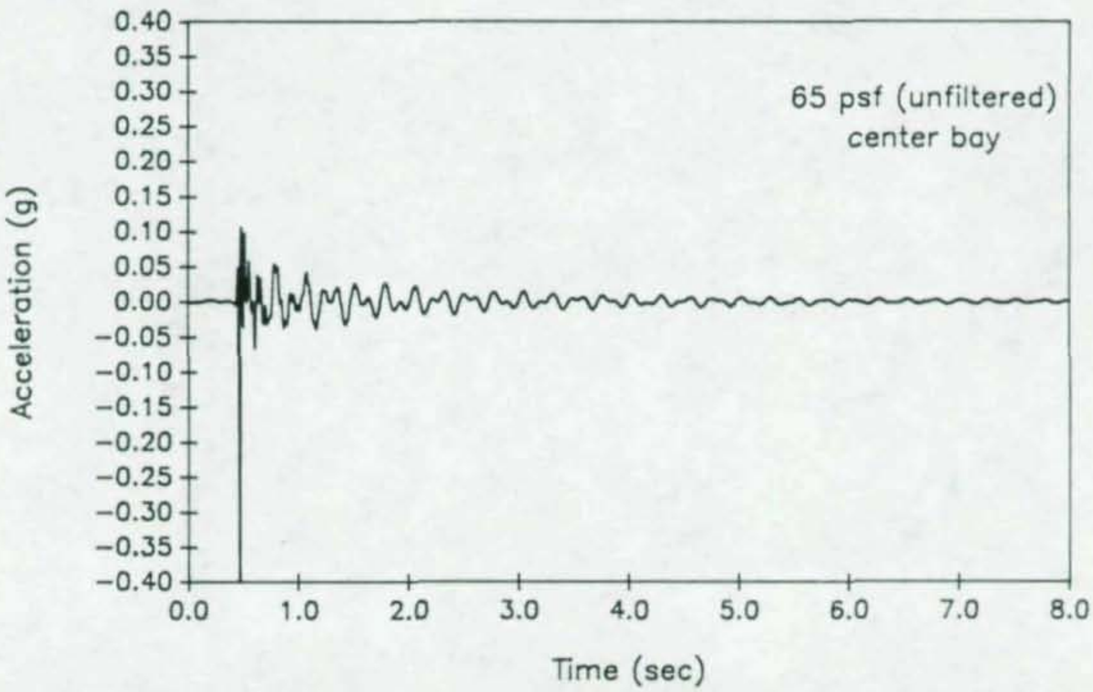

Figure A.9 Time vs. Acceleration @ 65 psf (unfiltered)

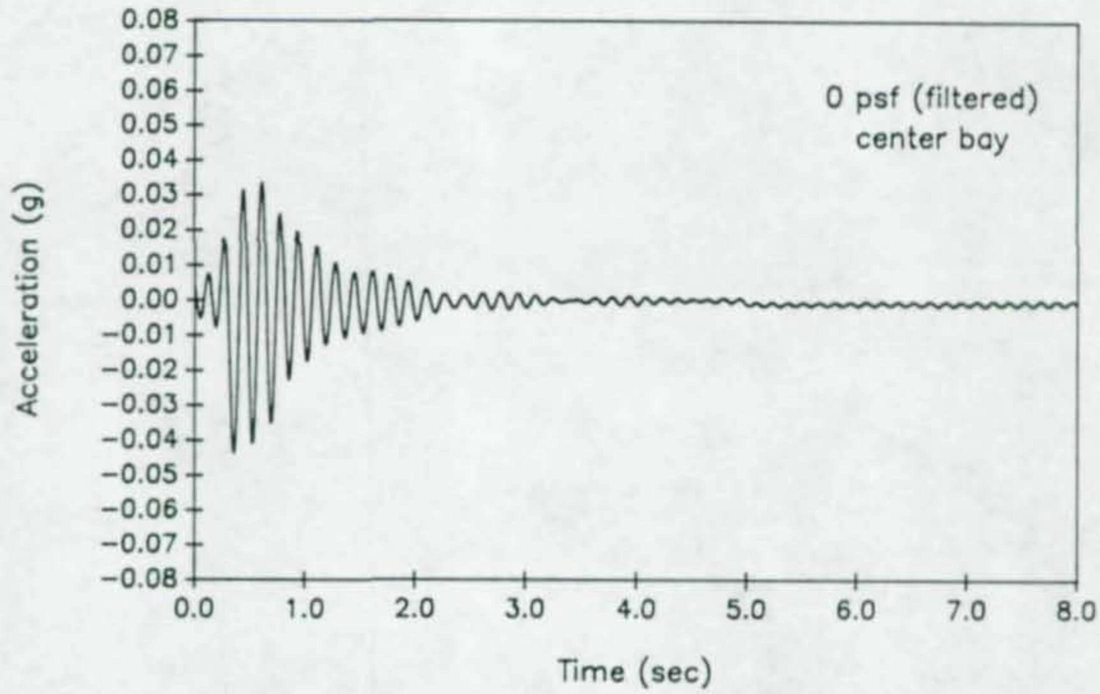

Figure A.10 Time vs. Acceleration @ 0 psf (filtered)

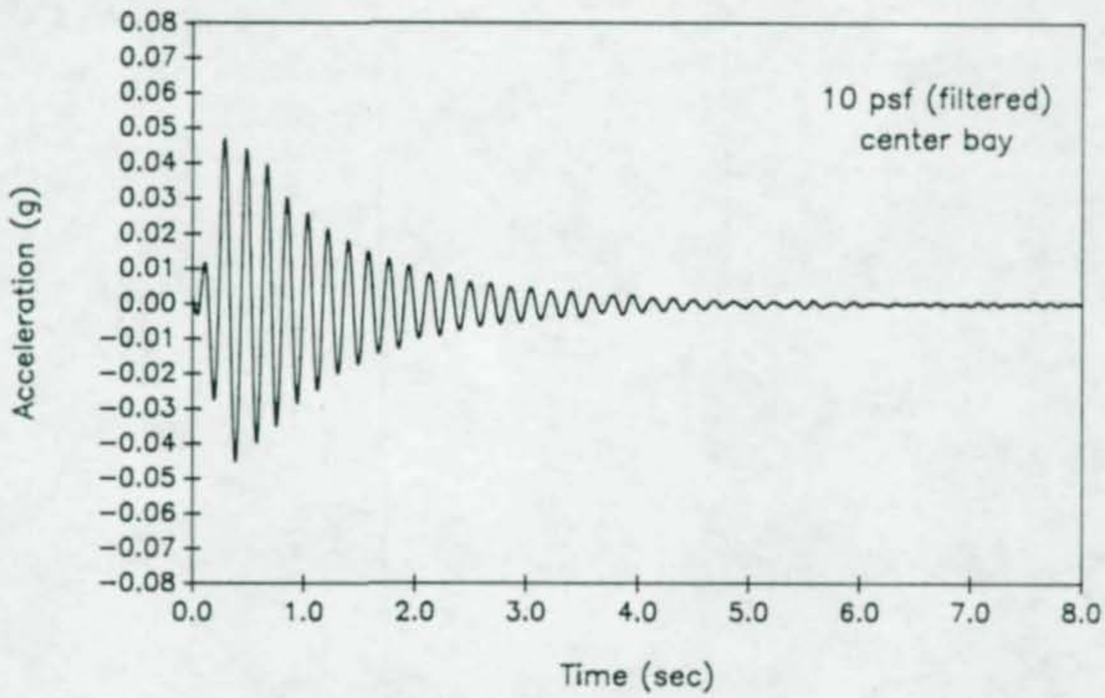

Figure A.11 Time vs. Acceleration @ 10 psf (filtered)

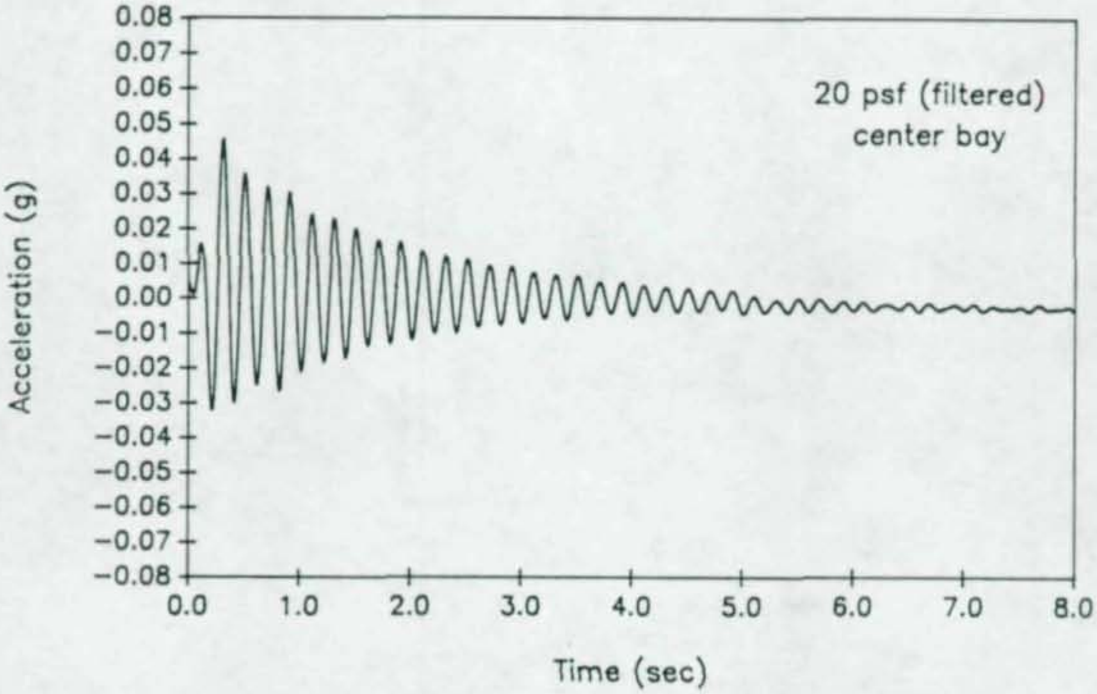

Figure A.12 Time vs. Acceleration @ 20 psf (filtered)

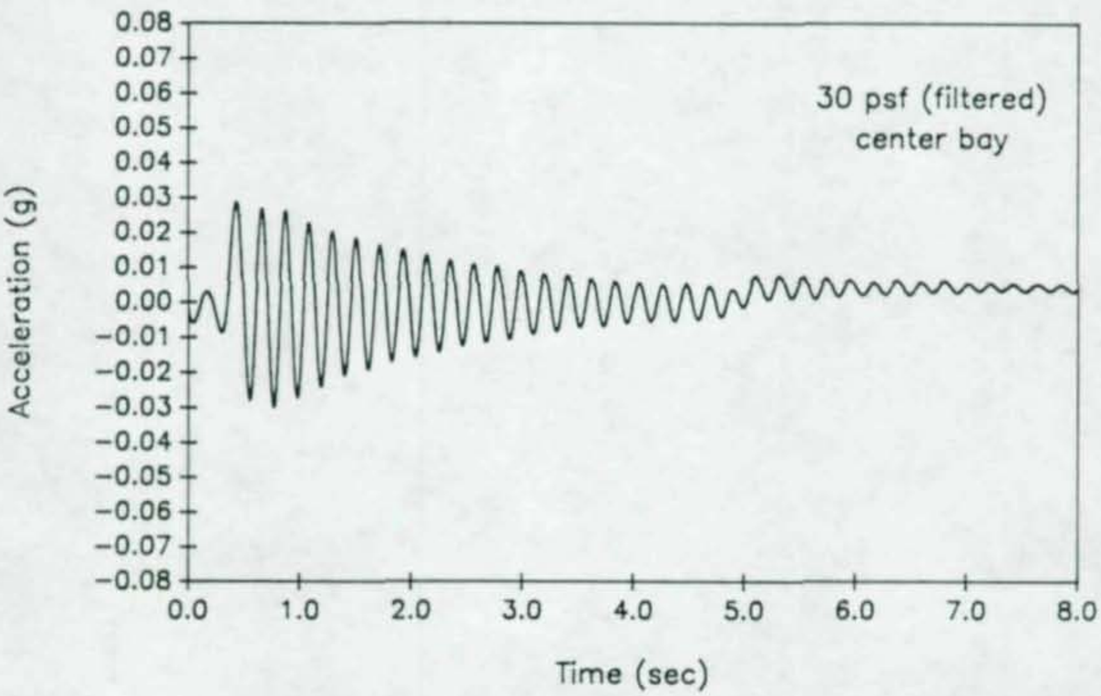

Figure A.13 Time vs. Acceleration @ 30 psf (filtered)

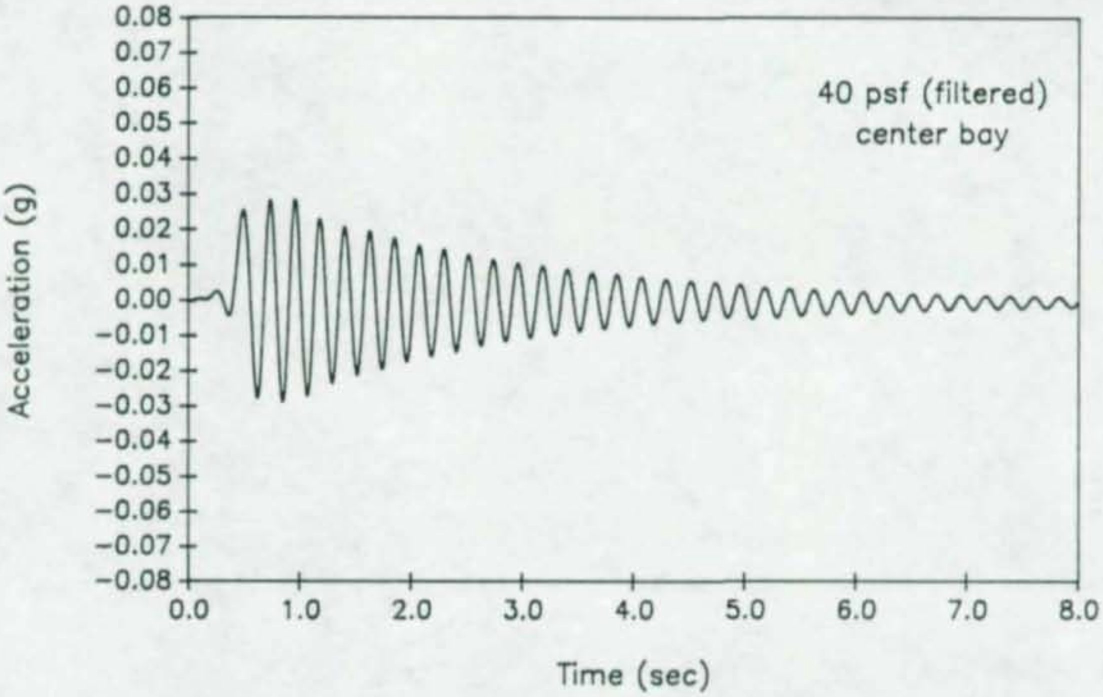

Figure A.14 Time vs. Acceleration @ 40 psf (filtered)

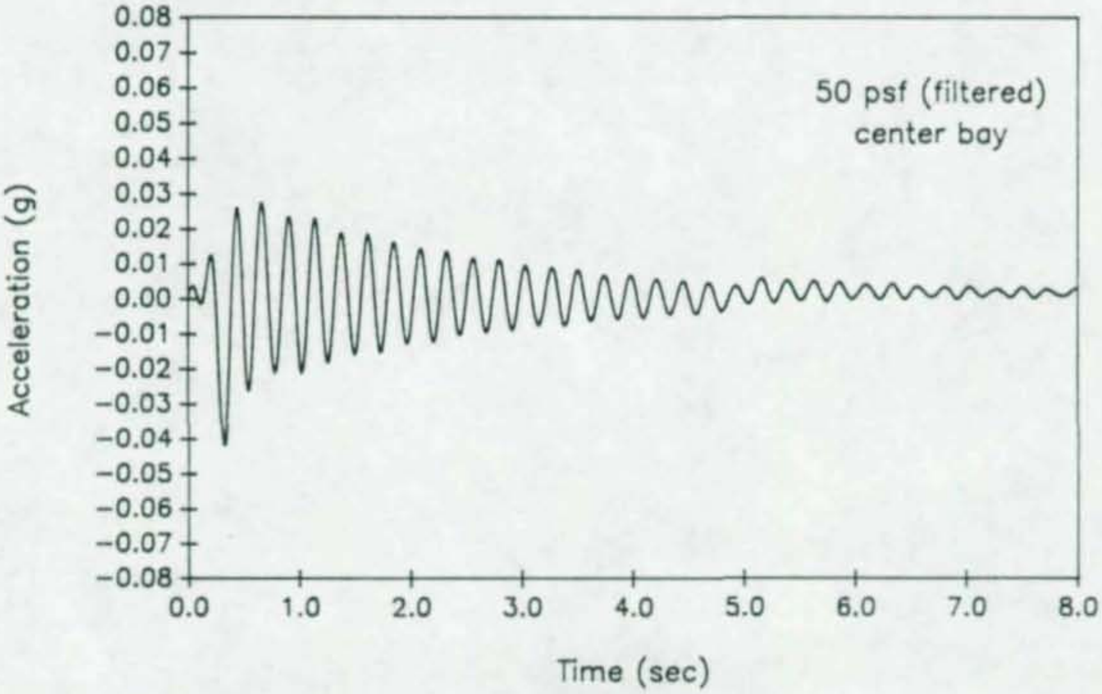

Figure A.15 Time vs. Acceleration @ 50 psf (filtered)

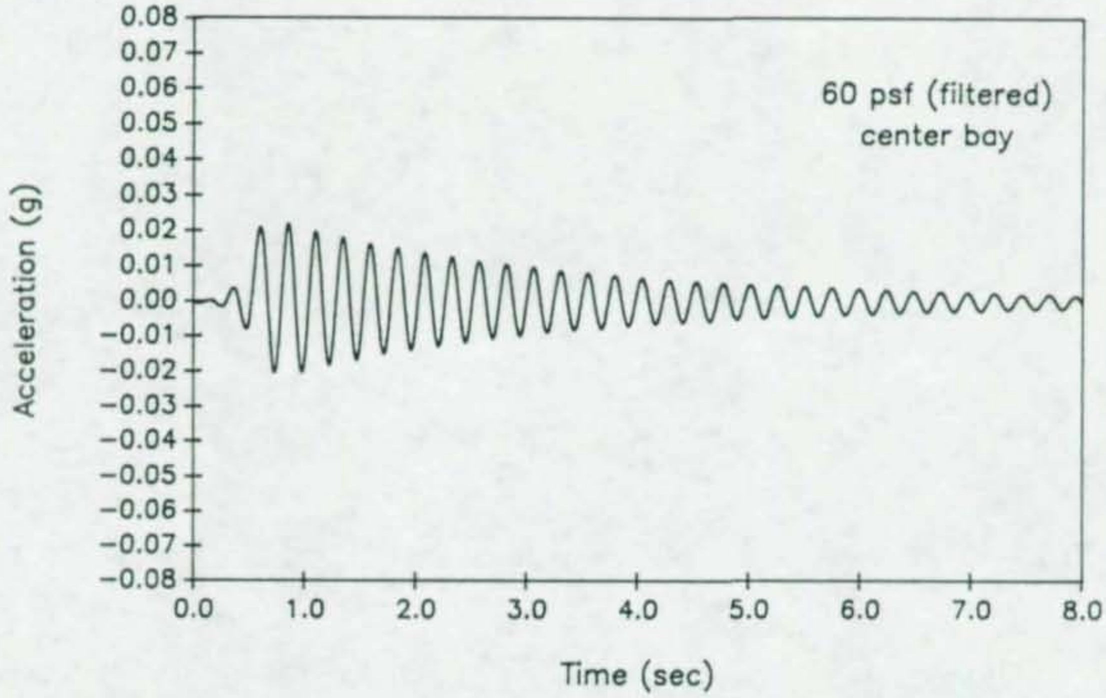

Figure A.16 Time vs. Acceleration @ 60 psf (filtered)

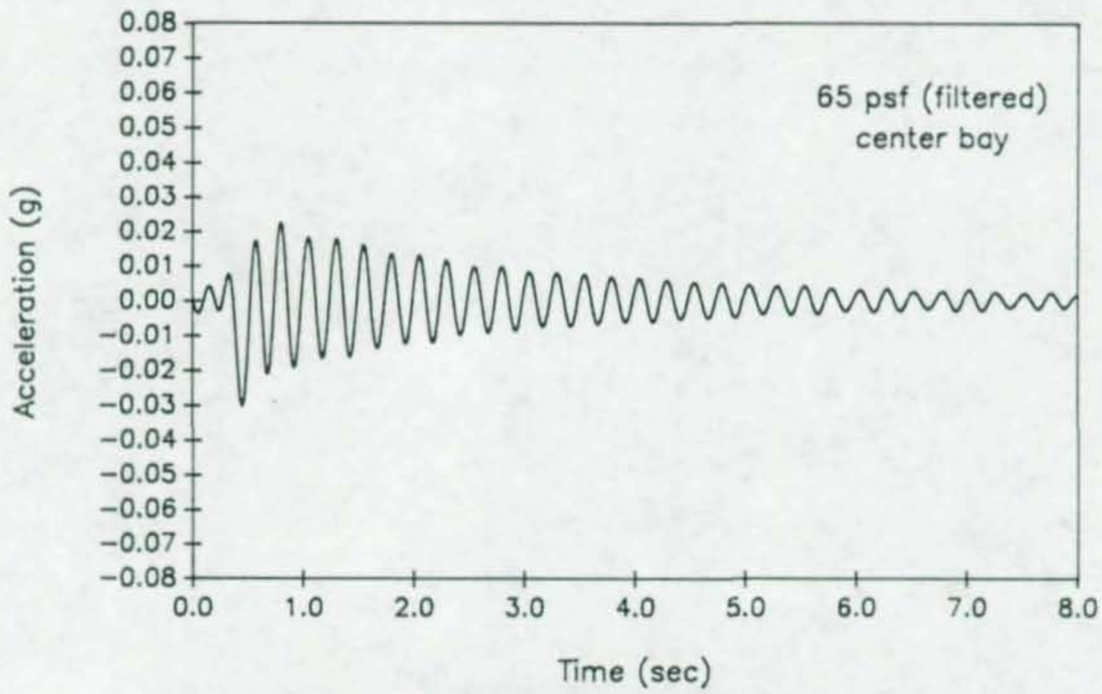

Figure A.17 Time vs. Acceleration @ 65 psf (filtered)

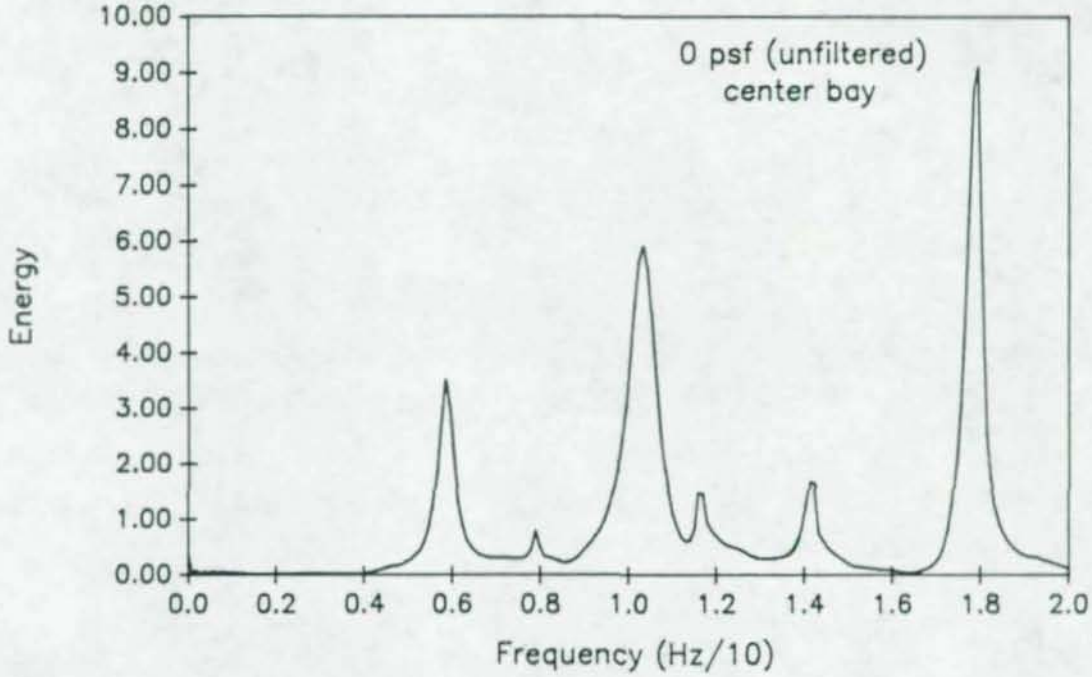

Figure A.18 Power Density Spectrum

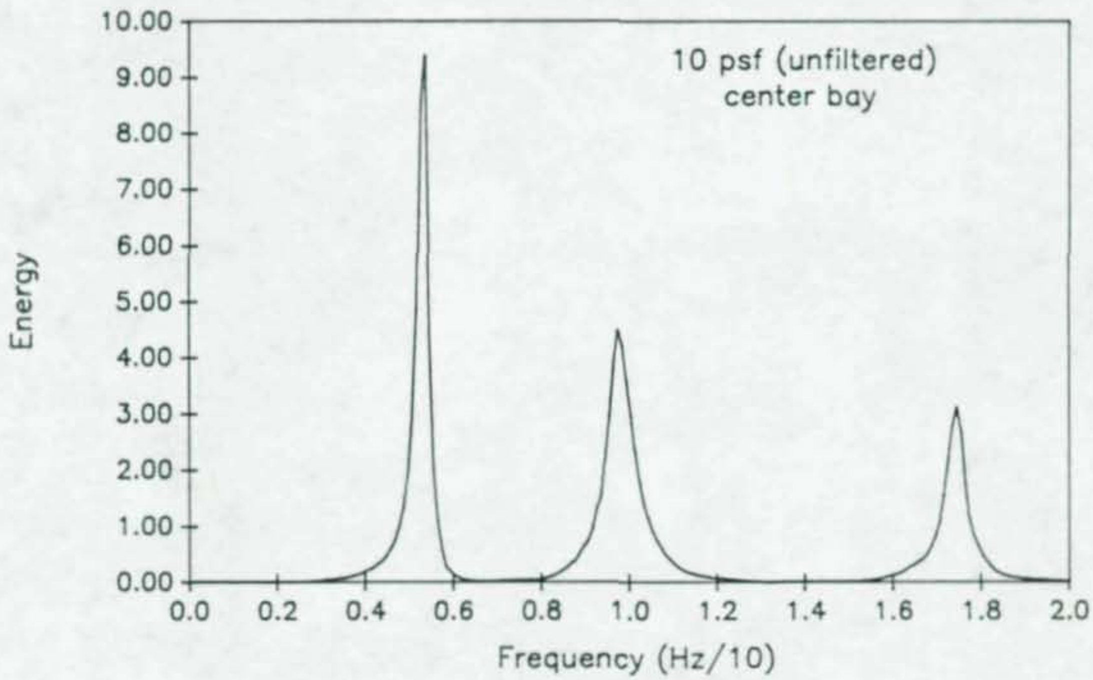

Figure A.19 Power Density Spectrum

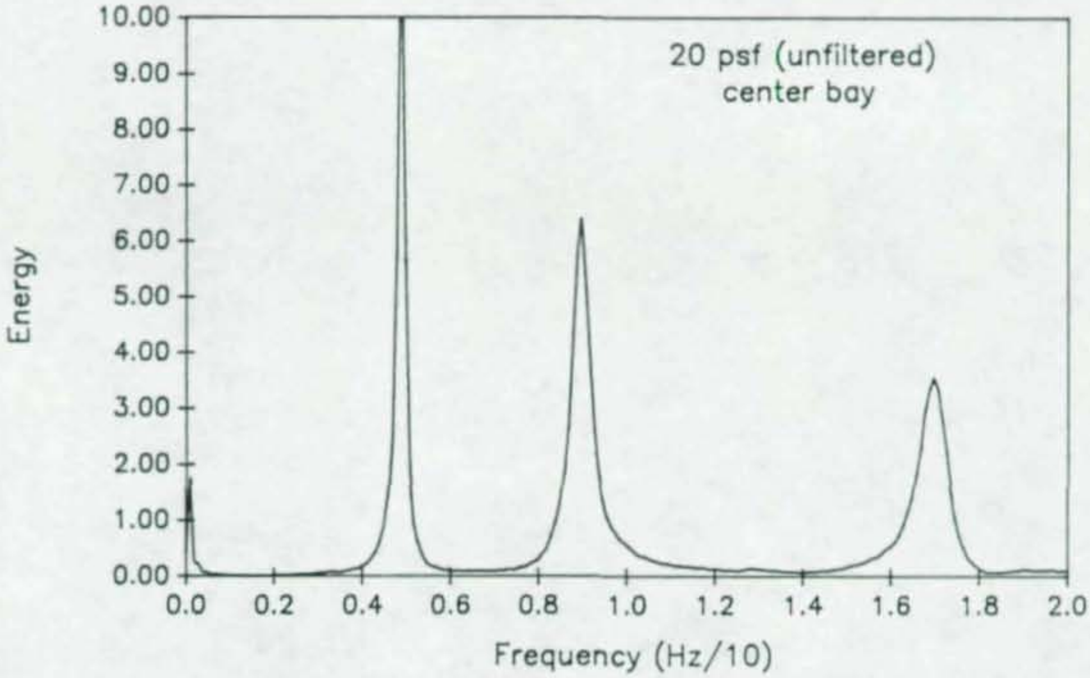

Figure A.20 Power Density Spectrum

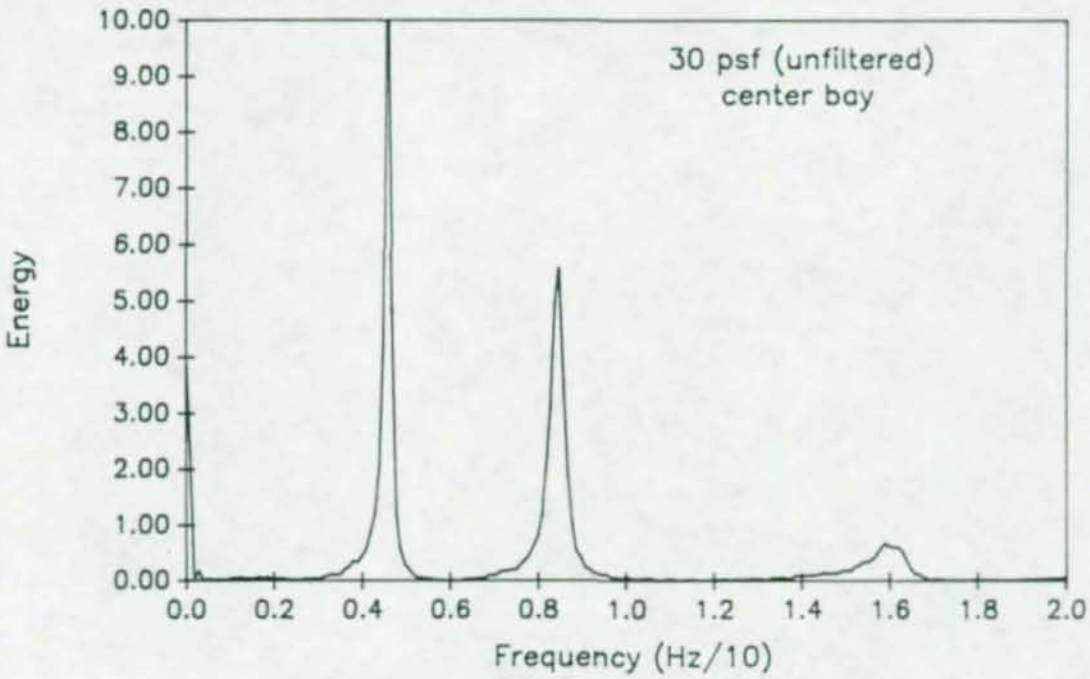

Figure A.21 Power Density Spectrum

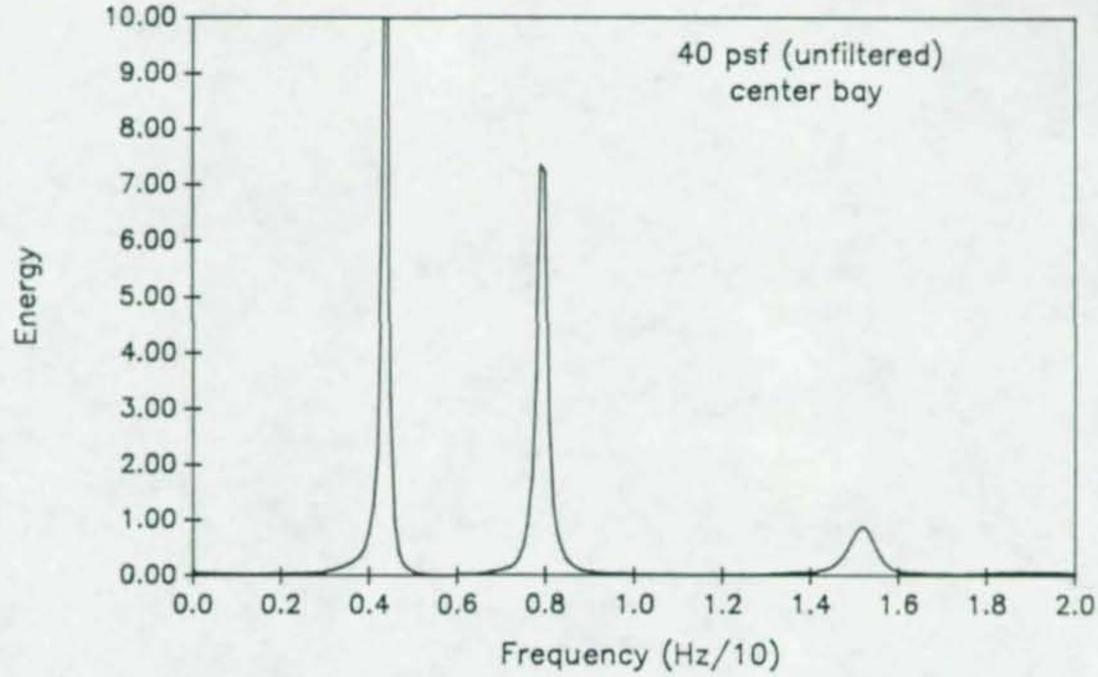

Figure A.22 Power Density Spectrum

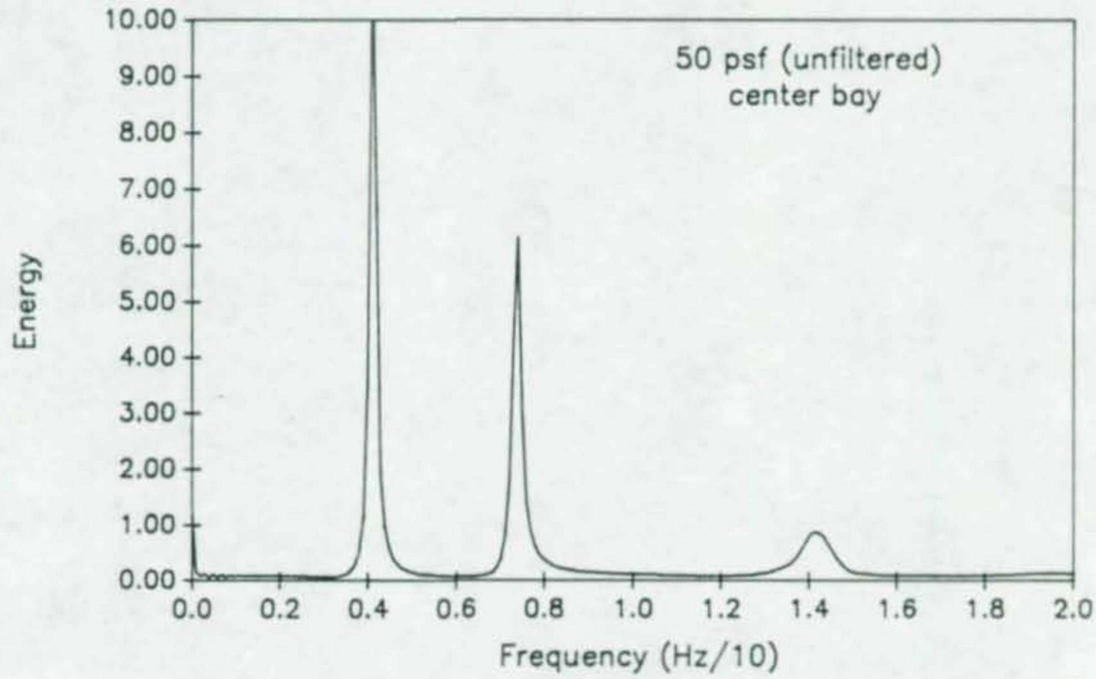

Figure A.23 Power Density Spectrum

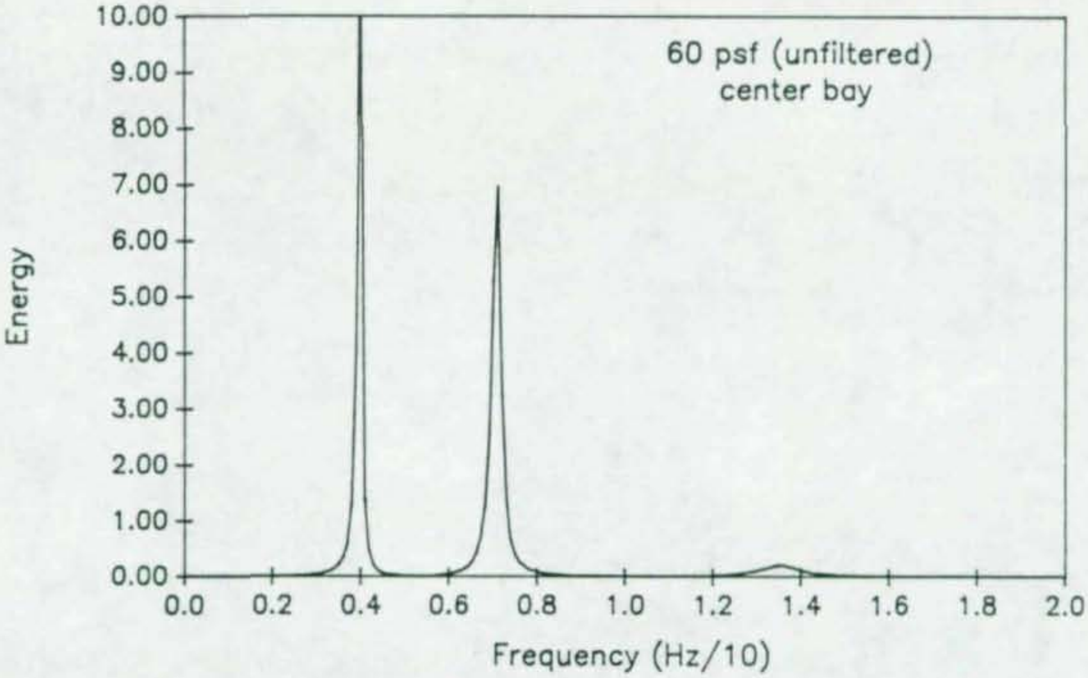

Figure A.24 Power Density Spectrum

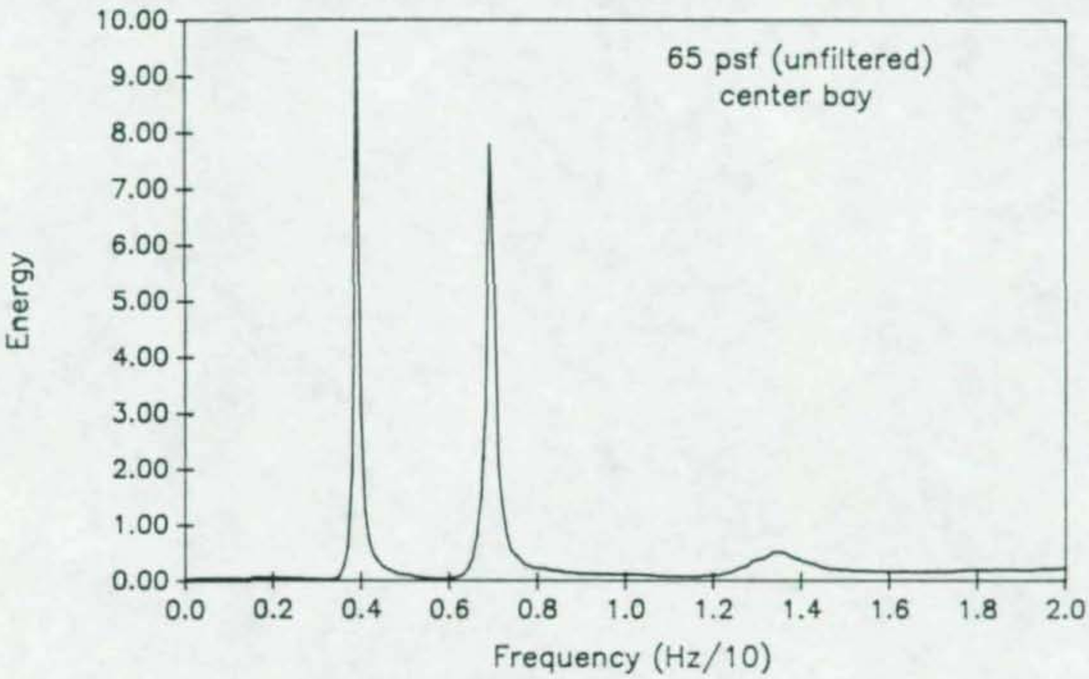

Figure A.25 Power Density Spectrum

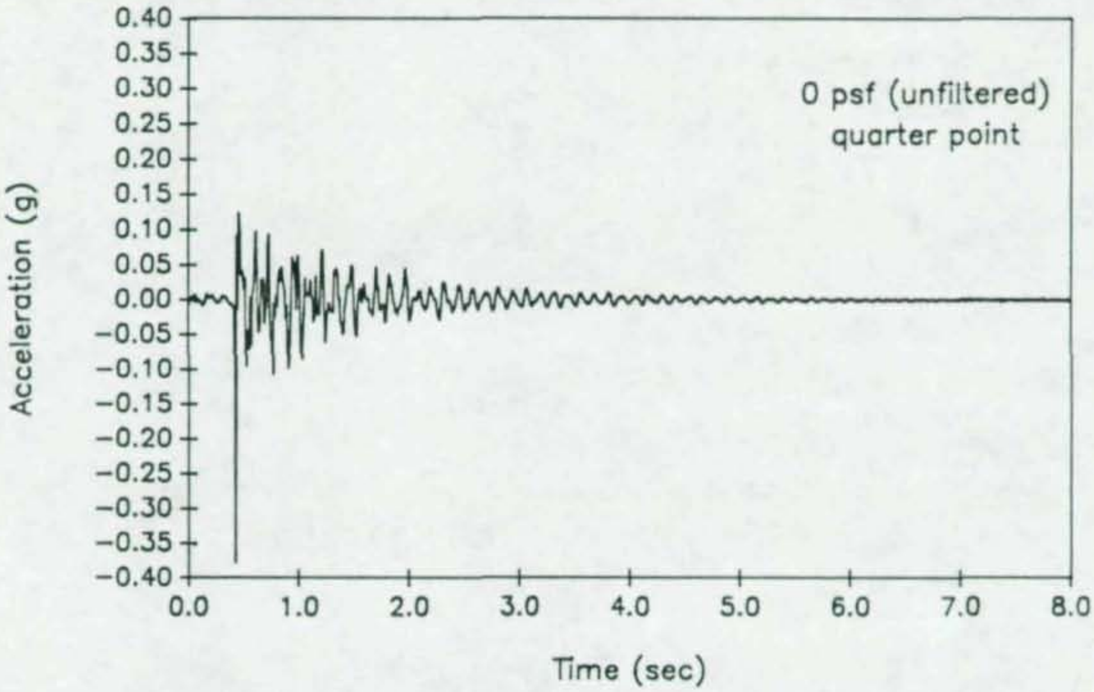

Figure A.26 Time vs. Acceleration @ 0 psf (unfiltered)

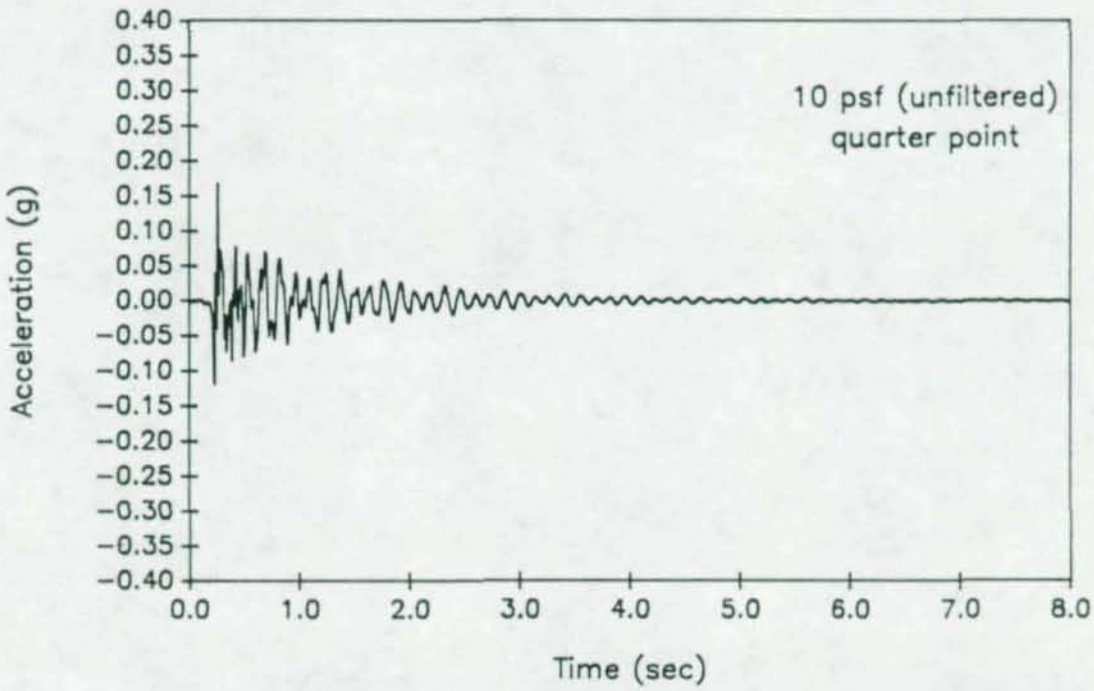

Figure A.27 Time vs. Acceleration @ 10 psf (unfiltered)

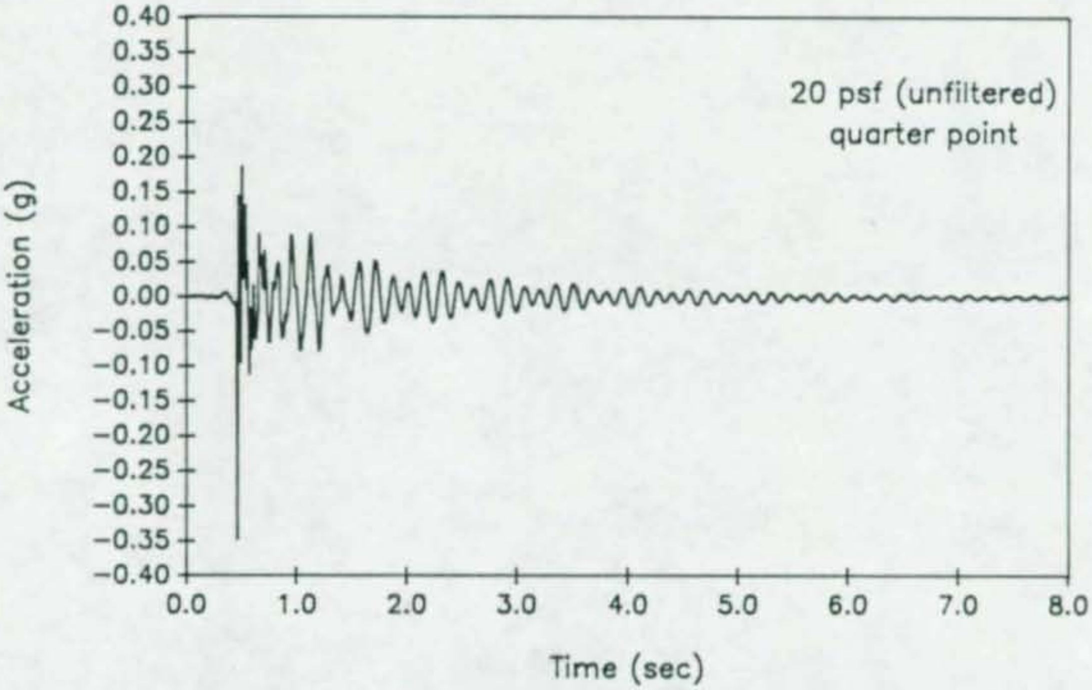

Figure A.28 Time vs. Acceleration @ 20 psf (unfiltered)

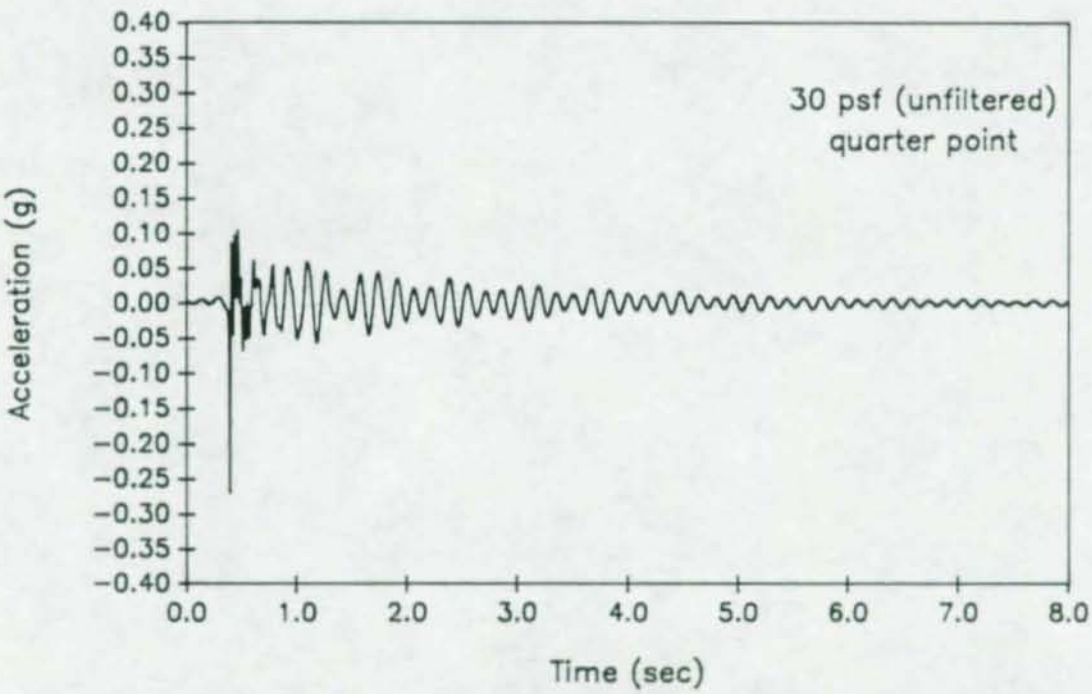

Figure A.29 Time vs. Acceleration @ 30 psf (unfiltered)

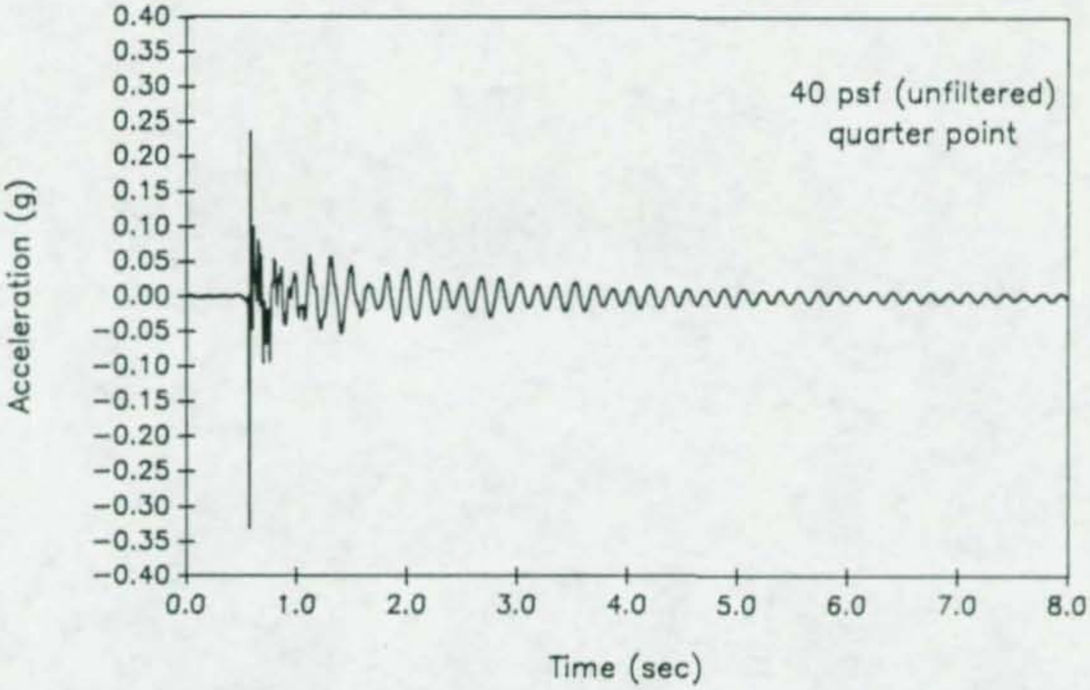

Figure A.30 Time vs. Acceleration @ 40 psf (unfiltered)

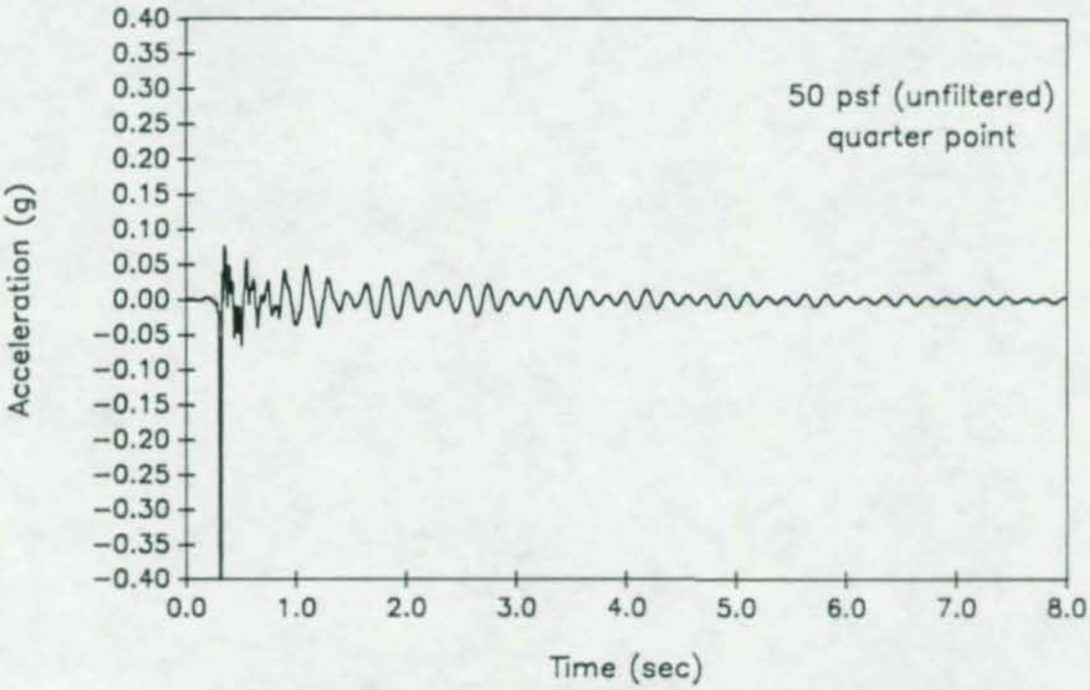

Figure A.31 Time vs. Acceleration @ 50 psf (unfiltered)

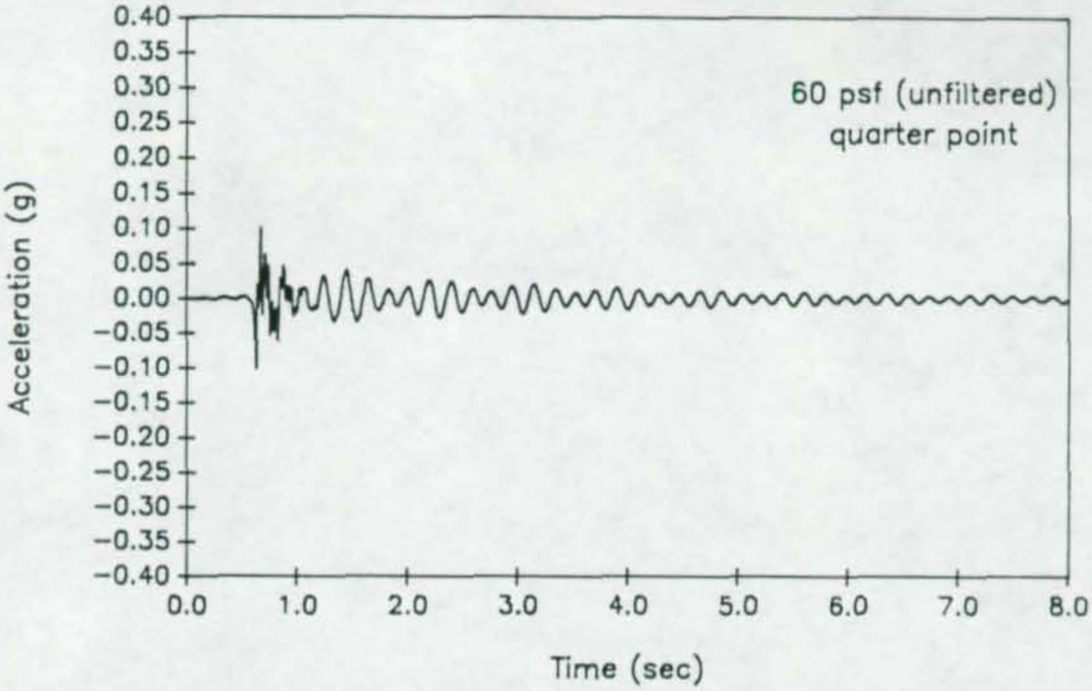

Figure A.32 Time vs. Acceleration @ 60 psf (unfiltered)

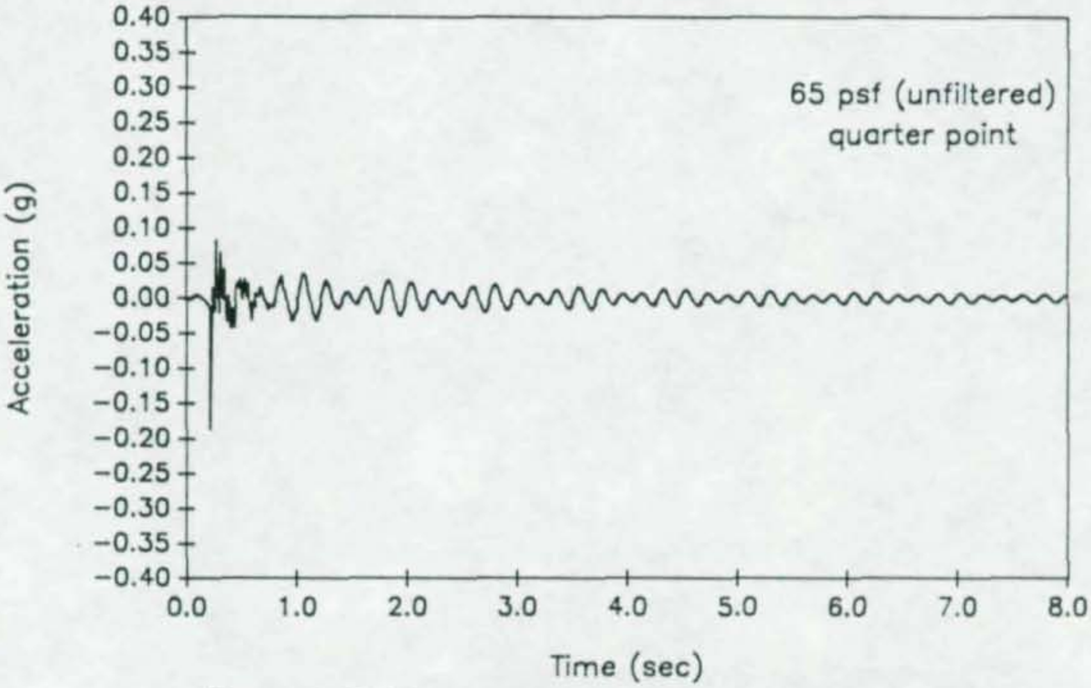

Figure A.33 Time vs. Acceleration @ 65 psf (unfiltered)

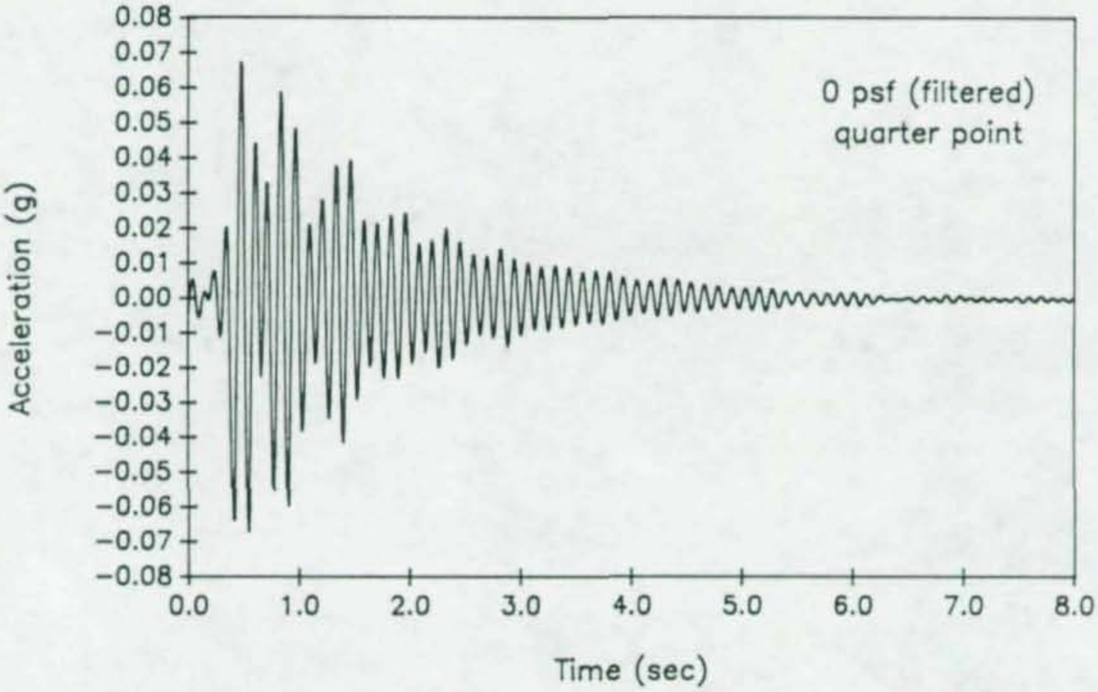

Figure A.34 Time vs. Acceleration @ 0 psf (filtered)

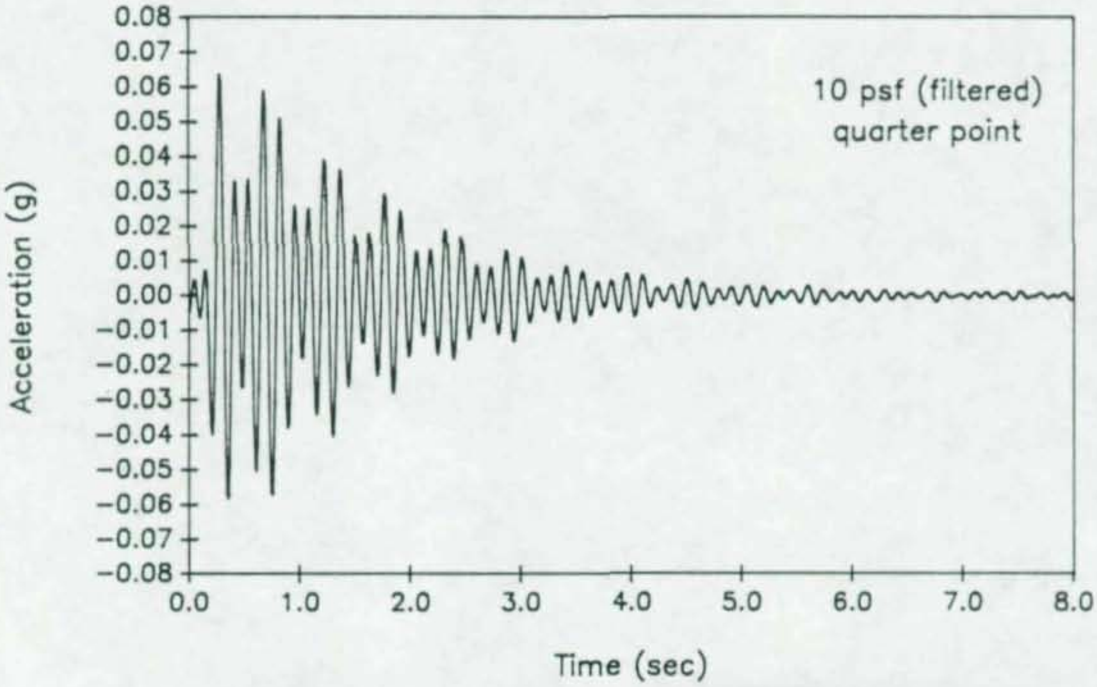

Figure A.35 Time vs. Acceleration @ 10 psf (filtered)

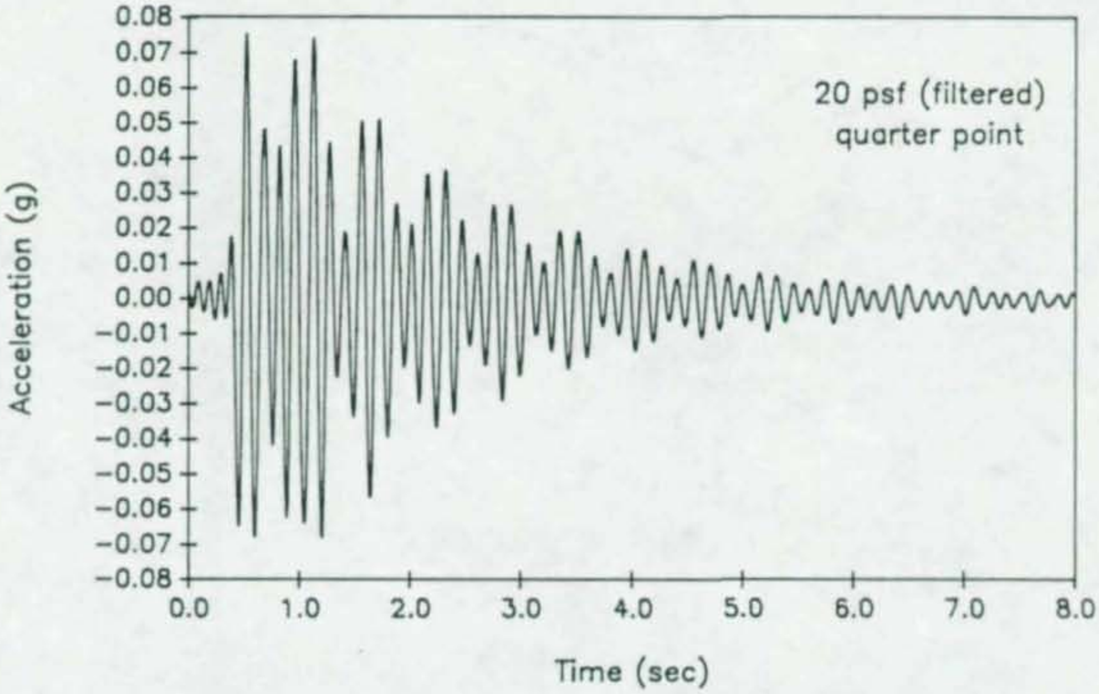

Figure A.36 Time vs. Acceleration @ 20 psf (filtered)

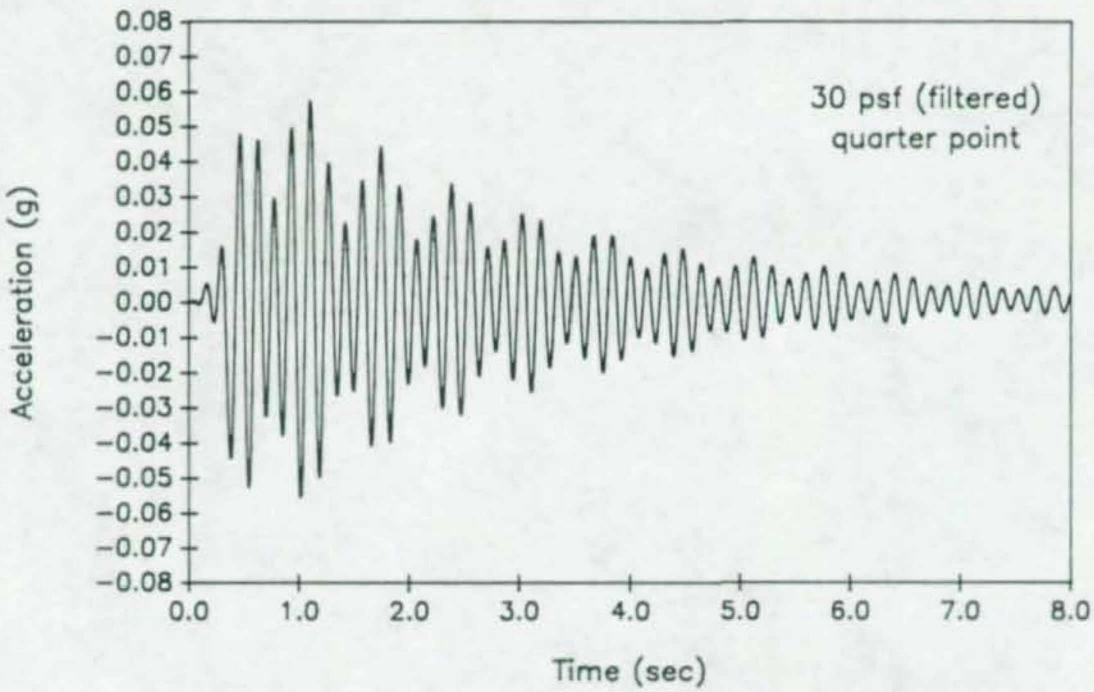

Figure A.37 Time vs. Acceleration @ 30 psf (filtered)

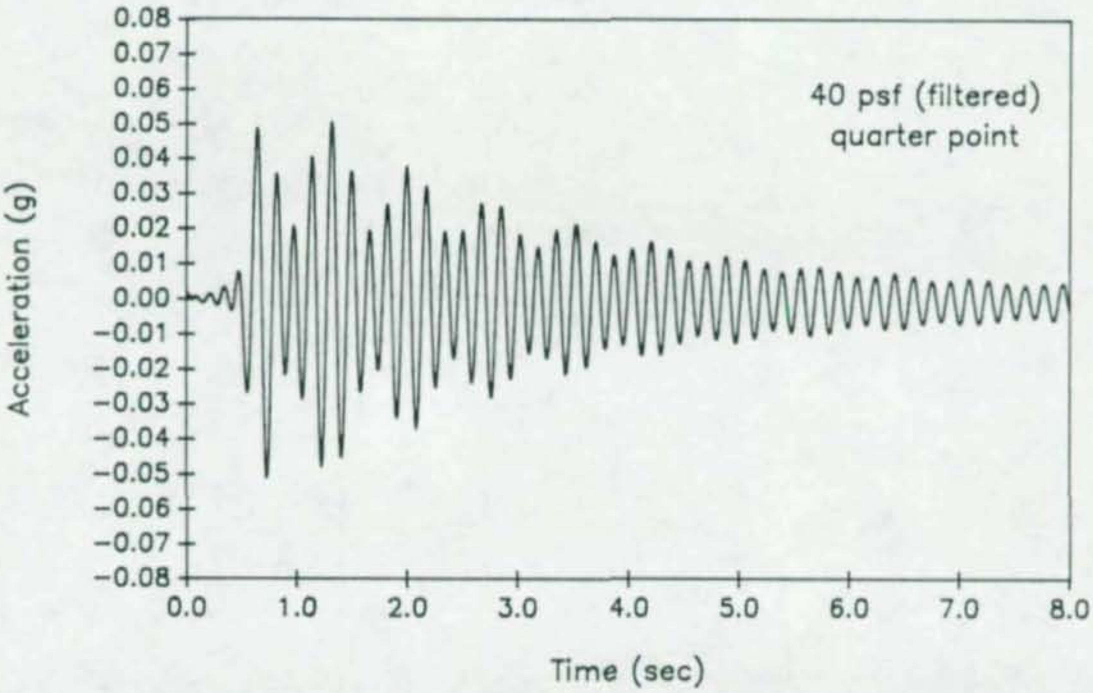

Figure A.38 Time vs. Acceleration @ 40 psf (filtered)

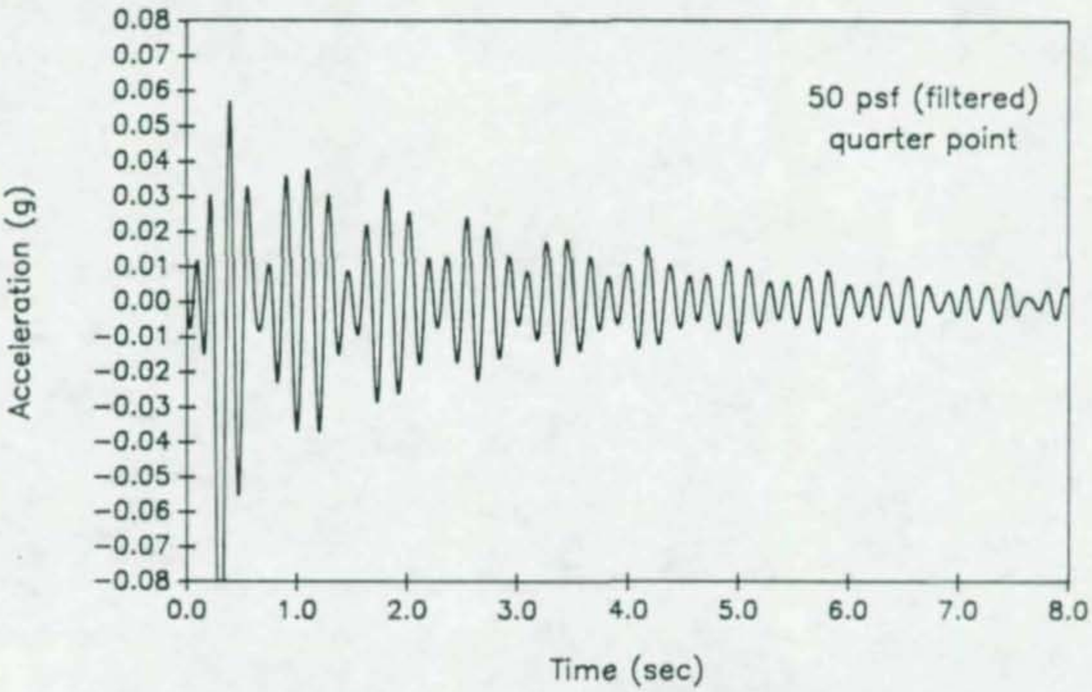

Figure A.39 Time vs. Acceleration @ 50 psf (filtered)

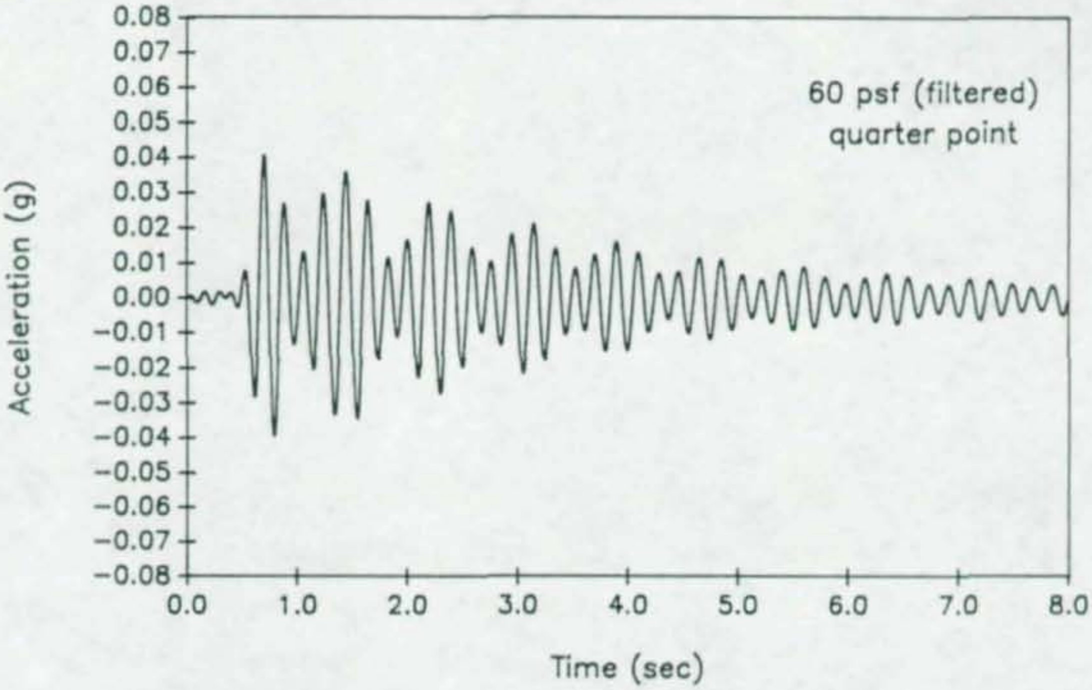

Figure A.40 Time vs. Acceleration @ 60 psf (filtered)

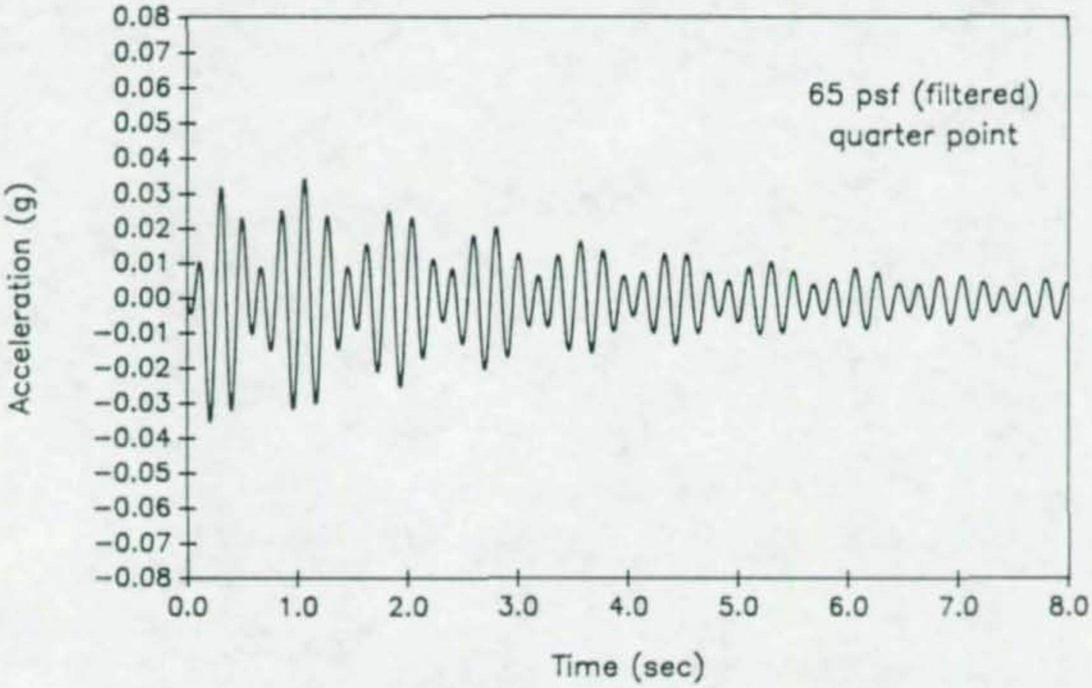

Figure A.41 Time vs. Acceleration @ 65 psf (filtered)

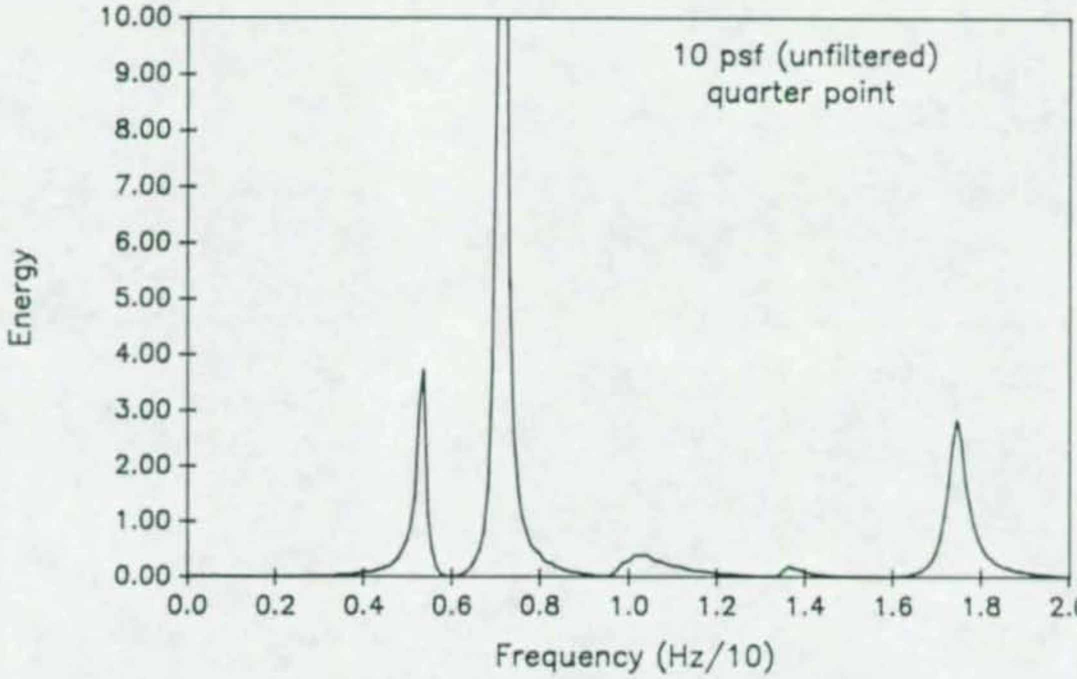

Figure A.42 Power Density Spectrum

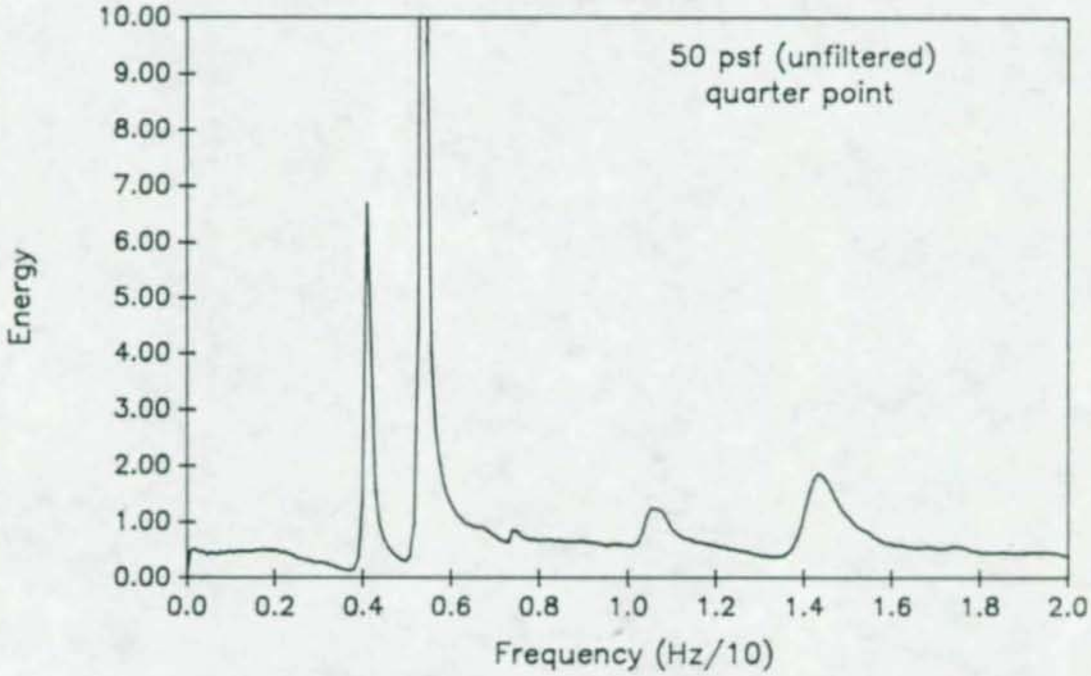

Figure A.43 Power Density Spectrum

APPENDIX B

CALCULATIONS FOR EXPERIMENTAL FLOOR SYSTEM

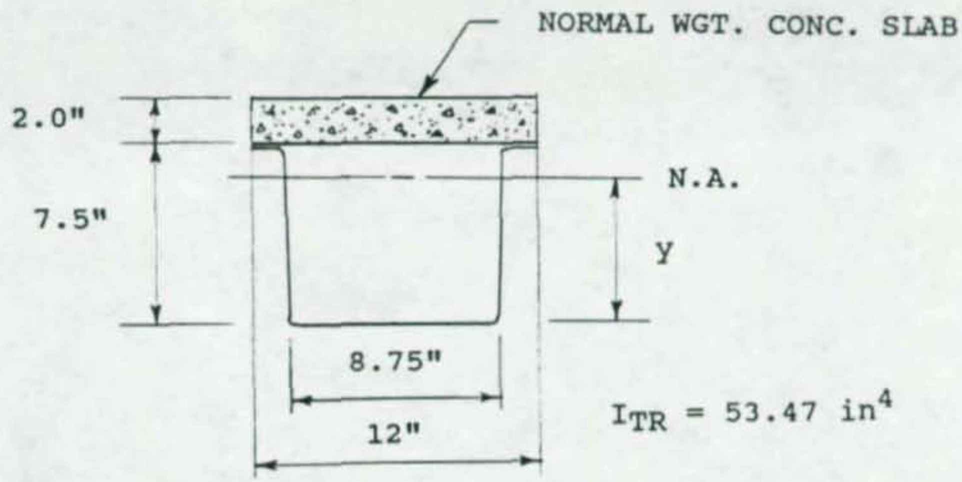

Figure B.1 Cross-Section of Experimental Floor

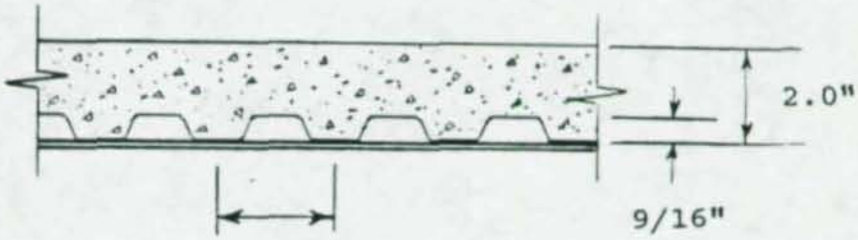

Figure B.2 Cross-Section of Slab

Steel Deck Properties

A	-	2.383	in^2	(area of steel cross-section)
I_x	-	20.849	in^4	(moment of inertia)
$\bar{y}$	-	3.750	in	(location of neutral axis)
S_x	-	5.121	in^3	(section modulus, top and bottom)
E_s	-	29000	ksi	(modulus of elasticity of steel)
f_y	-	33	ksi	(yield strength of steel)

Transformed Section Properties

f_c'	-	4000 psi	
n	-	E_s/E_c - 8	(modular ratio)
E_c	-	3605 ksi	(modulus of elasticity of concrete)
b_{eff}	-	12/8 - 1.5 in	(effective width of slab)
t_s	-	1.5 m	(effective slab thickness)
$\bar{y}$	-	6.08 in	(location of neutral axis)
I_{TR}	-	53.47 in^4/ft	(transformed moment of inertia)
$S_{TR\text{-}t}$	-	37.66 in^3	(section modulus of steel-top)
$S_{TR\text{-}b}$	-	8.79 in^3	(section modulus of steel-bottom)
$S_{TR\text{-}c}$	-	125.1 in^3	(section modulus of concrete-top)

Loads

14 ga. steel deck	8.1 psf
28 ga. steel deck	0.76 psi
145 pcf concrete slab	21.25 psf
Total dead load	30.0 psf
Superimposed live load	70.0 psf

Forces

$$M = \frac{w\ell^2}{8}$$

ℓ = 30 ft.

M_{DL} = 40.5 kip. in

M_{LL} = 94.5 kip. in

Check Concrete Stresses (ACI A.3.1a)

$$\sigma_{MAX} = \frac{M_{LL}}{S_{TR\text{-}c}} = \frac{94.5}{125.1} = 0.755 \text{ ksi}$$

$$\sigma_{ALLOW} = 0.45\ f_c' = 0.45(4.0) = 1.80 \text{ ksi} > 0.755 \text{ ksi} \quad \therefore \text{OK}$$

Check Steel Stresses (AISI C3)

Top $\quad \sigma_{MAX} = \dfrac{M_{DL}}{S_x} + \dfrac{M_{LL}}{S_{TR\text{-}t}} = 10.4 \text{ ksi}$

Bottom $\quad \sigma_{MAX} = \dfrac{M_{DL}}{S_x} + \dfrac{M_{LL}}{S_{TR\text{-}b}} = 18.6 \text{ ksi}$

$$\sigma_{ALLOW} = 0.6\ (f_y) = 0.6(33) = 19.8 \text{ ksi} > 18.6 \text{ ksi} \quad \therefore \text{OK}$$

Check Deflections

$$\Delta_{LL} = \frac{5w\ell^4}{384EI} = 0.82 \text{ in}$$

$$\Delta_{ALLOW} = \frac{\ell}{360} = 1.00 \text{ in} > 0.82 \text{ in} \quad \therefore \text{OK}$$

Vibration Analysis (Steel Joist Institute [Galambos undated])

I_{TR} - $53.47\ in^4/ft$ (moment of inertia of concrete slab)

I_{SLAB} - $2.028\ in^4/ft$ (moment of inertia of concrete slab)

D_x - $2.028\ E\ (k.\ in^2)$ (transverse stiffness)

D_y - $53.47\ E\ (k.\ in^2)$ (longidutinal stiffness)

ϵ - $(D_x/D_y)^{0.25} - 0.441$

x_o - $\frac{3\sqrt{2}}{4}\epsilon\ell - 14.04$ (half width of effective floor)

N_{eff} - $1 + 2\Sigma\cos\frac{\pi x}{2x_o}$ (number of effective beams)

N_{eff} - 17.86

w - $(30\ psf + 11\ psf)/12 - 3.417$ (weight per unit length)

b - $386\ in/sec^2$

$$f_1 - \frac{\pi}{2\ell^2}\sqrt{\frac{EI_{TR}g}{w}} - 5.073$$

$$j - 2\pi f_1(0.05) - 1.594$$

$$t_o - \frac{1}{\pi f_1}TAN^{-1}j - 0.063 < 0.05$$

$$A_o - [\frac{2(606)\ell^3}{\pi^4 EI_{TR}N_{eff}}][\frac{\sqrt{2(1-j\sin j-\cos j)+j^2}}{j}]$$

A_o - 0.016 (maximum amplitude)

D_{REQD} - $35A_of_1 + 2.5$ [Murray 1981]

D_{REQD} - $35(0.016)(5.073) + 2.5 - 5.26\%$

Shear Capacity

$S_r = \frac{VQp}{I}$

Q - first statical moment

p - connector spacing

12 - 14 x 1 1/4 in. screws maximum S_r - 0.783 kips

12 - 14 x 2 1/4 in. stand-off fasteners maximum S_r - 1.045 kips

APPENDIX C

NOMENCLATURE

NOMENCLATURE

E_{st} = 29000 ksi	(modulus of elasticity of steel)
E_c = 3605 ksi	(modulus of elasticity of 4000 psi concrete)
G_{xy} = 11200 ksi	(shear modulus of steel)
D_x = 125451 k in/ft	(flexural rigidity in longitudinal dir. of deck)
D_y = 4930 k in/ft	(flexural rigidity in transverse direction)
B = 20334 k in/ft	(torsional rigidity of plate)
I_{oy} = 0.854 in^4/ft	(transverse moment of inertia of slab)
I_{ox} = 37.0 in^4/ft	(longitudinal moment of inertia-steel only)
υ_{xy} = 0.25	(Poisson's ratio)
h = 2.0 in	(height of slab)
c_1 = 12.0 in	(spacing of long-span deck sections)
A = 67.5 in^2	(area enclosed by deck cross section)
b = (in)	(widths of elements in long-span deck)
t = (in)	(thicknesses of elements in deck sections)
e_x = 2.67 in	(distance from N.A. of composite section to centroid of slab)
e_{st} = 2.69 in	(distance from N.A. of composite section to centroid of long-span deck section)

VITA

John Robert Hillman was born in Grand Forks, North Dakota on April 20, 1963. He received his Bachelor's Degree in Civil Engineering from The University of Tennessee, Knoxville in 1986. Prior to pursuing graduate studies he worked for Figg & Muller Engineers for two years as a field engineer on the construction of the James River Bridge in Richmond, Virginia. He entered the graduate program in the Structures Division at Virginia Polytechnic Institute and State University in 1988.