VIBRATIONS OF STEEL FRAMED FLOORS DUE TO RUNNING

by

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Abstract

Vibration has been a consideration in many types of structures, and as the advancement of technology has allowed steel and concrete sections to become lighter, vibration has become more of a consideration in the design of structures. This report focuses on occupant induced vibration of steel framed floors due to running as the vibration source. The history of vibration analysis and criteria in structures is discussed. However, lack of research and experimentation on running as the source of vibration exists; therefore, the history section focuses on walking as the source of vibration. The current design criteria for vibration of steel framed floors in the United States of America is the American Institute of Steel Construction (AISC) Design Guide 11: Vibrations of Steel Framed Structural Systems Due to Human Activity. This design guide discusses vibration due to walking, running, and rhythmic activities as well as gives design criteria for sensitive occupancies and sensitive equipment. In order to apply the Design Guide 11 analysis procedure for running as the source of vibration, the Kansas State University Chester E. Peters Recreation Complex is used as a case study. The recreation complex includes a 1/5-mile running track that is supported by a composite steel framed floor. Based on the Design Guide 11 criterion, the running track is deemed acceptable. Lastly, this report discusses remedial procedures in the case of annoying floor vibration specific to floors that have running as a source of vibration. In addition, areas of further research are suggested where running is a source of vibration on steel framed floors.

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Chapter 1 - Basics of Structural Vibrations

Vibration in structures has been a consideration in many types of structures. In the past, reports stated that soldiers should break step when walking across bridges even before analysis techniques to assess vibration existed. In recent years, vibration analysis has become prevalent because structures are becoming lighter with longer spans. Steel and concrete are now stronger due to the advancement of technology thus allowing smaller section sizes. Meanwhile, research on vibration design acceptance criteria has also made significant progress.

This report focuses on occupant induced vibration of steel framed floors due to running as the vibration source. The beginning of this report will provide an overview of the past and current analysis techniques and design criteria of occupant-induced vibrations for steel framed structures. The history section focuses mainly on walking as the source of vibration due to the fact that the research and testing for running as the source of vibration is lacking. The current design criteria for vibration of steel framed floors in the United States of America is the American Institute of Steel Construction (AISC) Design Guide 11: Vibrations of Steel Framed Structural Systems Due to Human Activity. The design guide discusses vibration due to walking, running, and rhythmic activities as well as gives design criteria for sensitive occupancies and sensitive equipment.

The later portion of this report addresses specifically the design criteria for vibrations in steel framed floors due to occupants running. The analysis procedure to determine the acceptability of a floor from AISC Design Guide 11 is discussed in detail. In order to apply the analysis procedure, the Kansas State University Chester E. Peters Recreation Complex is used as a case study. The recreation complex includes a 1/5-mile running track that is supported by a composite steel framed floor. The case study shows how to apply Design Guide 11 with modifications because the running track has a slightly different floor framing than as prescribed in the design guide.

Lastly, this report discusses remedial procedures in the case of annoying floor vibration specific to floors that have running as a source of vibration. In addition, areas of further research are suggested where running is a source of vibration on steel framed floors.

Overview of Vibration Theory

Before diving into vibrations of floor systems, some basic vibration theory must be introduced. The following terms are used throughout this report and are defined in this section to give a general background on vibration theory.

Vibrations. Vibration is a mechanical phenomenon where oscillation of a mass occurs. There are two types of vibration: free vibration and forced vibration. Free vibration occurs when a mass is disturbed from static equilibrium and is allowed to vibrate without an external dynamic load (Chopra, 2007). Forced vibration occurs when a mass is excited by an external dynamic load. Types of dynamic loading are explained in this section.

Frequency. The frequency of a system is the rate at which it vibrates freely when displaced and then released. In the case of structures, there are normally multiple frequencies of the system, which are all referred to as modal frequencies. The lowest modal frequency is termed the fundamental natural frequency and is usually of most concern (Murray, Allen, & Ungar, 2003). The lowest modal frequency is the smallest frequency of a floor at which it vibrates.

Dynamic Loading. Vibration in floors is caused by dynamic loading that is either applied directly to the floor or indirectly by the supports moving. Indirect dynamic loading is caused by traffic around the building and wind buffeting. These types of dynamic loading are not discussed within this report. Direct dynamic loading can be split into four groups: harmonic, periodic, transient, and impulsive. Harmonic loading is normally caused by rotating machinery. Periodic loading is caused by either machinery that generates repetitive impacts or by human activities that are rhythmic in nature such as dancing or aerobics. Transient loading is caused by the movement of people across a floor. Lastly impulsive loading is caused by human activities such as single jumps or heal drops (Murray, Allen, & Ungar, 2003).

Forcing Function. Each cause of vibration has a forcing function that shows a time history of the applied force and has a frequency of its own. For example, the frequency at which a person runs across the floors is the pace of their steps. This forcing function is discussed in this report.

Resonance. When the frequency of the forcing function equals the fundamental frequency of the structure, resonance occurs. Resonance occurs because with each successive load cycle the response of the system increases, thus increasing the amplitude of the vibration (Steel Construction Institute, 2016). Vibration can become of concern when resonance occurs within the structure. Fortunately, most structures have enough damping that resonance does not occur. Partial resonant build up, however, can occur when the force is applied for a short amount of time. The amplitude of the floor acceleration increases but it does not reach steady state acceleration.

Damping. Damping of a floor is a measure of the mechanical energy dissipated during a cycle of vibration; the more damping the system has the quicker the system will come to rest after it is displaced. For floor vibration analysis, the damping of a floor is normally given as a percent of the critical damping. Critical damping, shown as the solid line in Figure 1, is the amount of viscous damping at which a system does not oscillate when displaced. A system is overdamped when the amount of viscous damping is more than the amount at which a system does not oscillate when displaced, as indicated by the dashed line in Figure 1. A system is underdamped when the amount of viscous damping is less than the amount at which a system does not oscillate when displaced, as indicated by the dotted-dashed line in Figure 1. The difference between critically damped, overdamped, and underdamped systems and how they move after being displaced is shown in Figure 1. Within a structure, damping is provided by the structure itself, architectural finishes, people, and furniture.

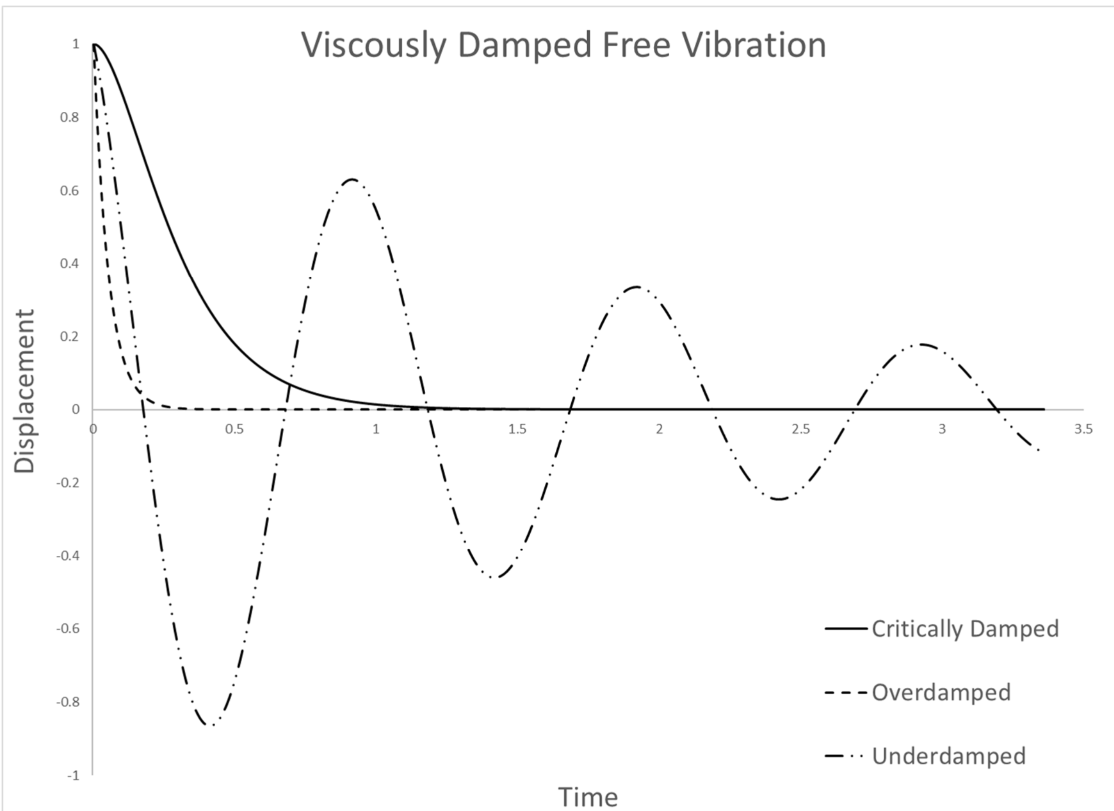

Figure 1: Viscously Damped Free Vibration

Harmonic. The harmonic multiple is an integer multiple of the frequency. The harmonics of a forcing function are important because even though the frequencies of the floor and the forcing function are not the same resonance can still occur if a harmonic of the forcing function is the same as the frequency of the floor.

Heel Drop Test. The heel drop test is a way of inducing a dynamic force into a floor system in order to understand the response of the system (Murray, Allen, Ungar, & Davis, 2016). A person puts all their weight onto the balls of their feet and raises their heels. Then the person quickly drops their weight onto their heels and thus onto the floor (Allen & Rainer, 1976).

Acceptance Criteria. In order to evaluate vibrations in floors, acceptance criteria must be established. The acceptance criteria have changed throughout the last few decades as more research has been conducted and researchers better understand the floor response. Currently, the AISC Design Guide 11 gives occupant comfort acceptance criteria in the form of an acceleration limit. If the acceleration of the floor, as a percent of gravity, is less than the limit given, then the floor is deemed acceptable.

Consequences of Vibrations

Fortunately for buildings, vibrations do not pose a large threat to the integrity of the structure. If the structure is designed to withstand the peak dynamic force, the only structural safety problems vibrations can cause are issues of fatigue. Fatigue cracks can occur, usually at connections, and then propagate through the structure. However, this is usually only a problem with very repetitive cyclical loading, such as in traffic over bridges (Steel Construction Institute, 2016). The largest consequence of vibrations is complaints from the occupants because the vibrations can cause alarm or discomfort. Thus vibrations are mostly a serviceability consideration in design.

Human Perception of Vibrations

Designing to eliminate the complaint from vibrations can be difficult because the perception of the vibrations can vary greatly depending on a person's perception and tolerance levels. In addition, the person causing the vibration and the person perceiving the effects are not usually in the same place. The Steel Construction Institute gives the following factors that play a role in the human perception of vibrations (Steel Construction Institute, 2016):

- Activity type
- Time of day
- Environment in which the activity is taking place
- Source of vibration
- Vibration amplitude and frequency
- Level of damping
- Duration of exposure

The “subjective nature of vibrations means that it is not possible to prescribe an exact limit that will guarantee an acceptable floor response” (Steel Construction Institute, 2016). For example, people in an aerobics class are more likely to accept floor vibrations than people who are sitting in a quiet office environment. Therefore, the limits given in design guides are the limits most acceptable to the majority of occupants. Even if a floor is designed to the limits within design guides, there is still the possibility for complaints to arise.

Chapter 2 - History of Floor Vibration Analysis

The analysis of structural vibration has changed over the years as more accurate types of analysis have emerged through research. This section of the report provides a brief overview of how the acceptance criteria and analysis methods for steel framed floors have evolved over the last 100 years. The acceptance criteria is used to determine if the floor system will be acceptable to most occupants, and the analysis methods are used to estimate the floor's characteristics in order to compare the actual floor vibration to the acceptability limits.

Acceptance Criteria History

This section of the report provides an overview of how the vibration acceptance criteria has changed over the last 100 years. The acceptance criteria take on different forms such as graphs/scales and inequalities as research has been conducted to improve our understanding of vibration in structural systems.

1931 – H. Reiher, and F.J. Meister. The Reiher – Meister scale was the first vibration acceptance criteria to be widely accepted and used. In order to determine the scale, Reiher and Meister subjected groups of people to steady state vibrations and recorded their responses (Murray, Allen, & Ungar, 2003). The vibrations ranged in frequency from 5 to 100 Hz and ranged in amplitude from 0.0004 to 0.40 inches. The subject's perception of the vibrations were categorized as "not perceptible", "slightly perceptible", "distinctly perceptible", "strongly perceptible", "disturbing", or "very disturbing" (Boice, 2003). Reiher and Meister then created a scale from their experiment based on the floor frequency and vibration amplitude using the categories listed above to determine if the floor system is acceptable.

1966 – Kenneth H. Lenzen. Kenneth Lenzen expanded upon the work of Reiher and Meister to determine the effect of floor vibrations on humans (Lenzen, 1966). Lenzen built composite steel and concrete test floors in order to determine how the floor frequency, amplitude, and damping would affect the occupants. Equation 1 was used to determine the floor's natural frequency.

$$f = 1.57\sqrt{\frac{gEI_t}{w_d l^4}}$$ **Equation 1**

Where,

f = natural frequency of floor (Hz)

g = acceleration due to gravity (in/s^2)

= 386.4 in/s^2

I_t = the moment of inertia of the composite section multiplied by the number of joists (in^4)

E = the modulus of elasticity of the composite section (psi)

w_d = the dead load of the floor system (lb/in)

l = the effective length of joist (in)

Lenzen determined that humans are more often subjected to transient vibrations rather than the steady state vibrations that the Reiher – Meister scale is based on. In this case, the main source of vibrations in floor systems is from the occupants themselves (Lenzen, 1966). Through experimentation, Lenzen found that the occupants of a floor are most effected by the damping within a floor. He states that "the main problem then is not one of frequency and amplitude such as encountered in steady-state vibrations but of damping. If floors can be damped before 12 cycles of oscillation, the effect of the oscillatory motion is reduced" (Lenzen, 1966). After 12 cycles of oscillation, the occupants respond similarly to that of steady-state vibration. Lenzen created the modified Reiher – Meister scale by scaling up the amplitude by a factor of 10 from the original scale as shown in Figure 2. Lenzen only changed the amplitude on the scale; the categories to determine the floor's acceptability remained the same.

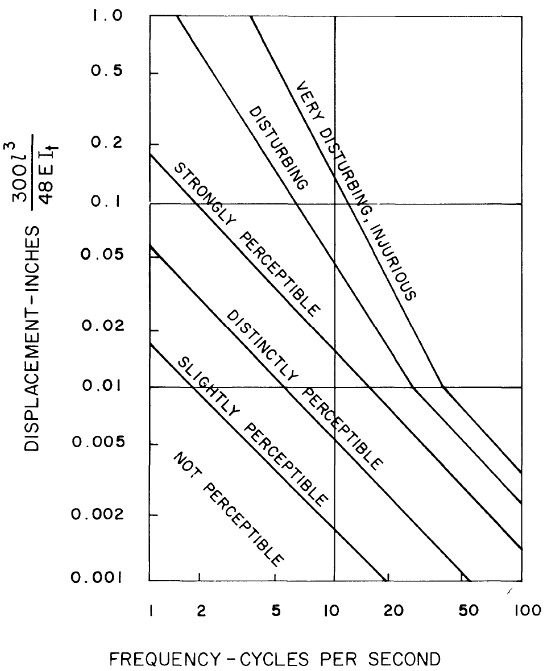

Figure 2: Modified Reiher - Meister Scale (Lenzen, 1966)

1976 – D. E. Allen and J. H. Rainer. Allen and Rainer determined acceptance criteria for long span steel beam floors with concrete decking in terms of floor acceleration and damping, expanding upon the work of Kenneth Lenzen (Allen & Rainer, 1976). Allen and Rainer examined floor test data from 5 different sets of researchers and experiments. The test floors were a mix of different floor systems. Joists, joists supported by girders, composite joists, composite beams, and non-composite beams were tested. The spans of the members ranged from 23 feet to 95 feet long. The effective concrete thickness ranged from 2.5 inches to 6.5 inches. Allen and Rainer then determined the criteria shown in Figure 3 for quiet spaces such as offices and residences with long span floor systems and used this scale to determine the acceptability of the test floors. The scale gives criteria for continuous vibrations, such as dancing or jumping, shown by the solid line and criteria for walking shown by the dashed lines. Based on frequency of the floor system, the peak acceleration limit can be determined based on the vibration source and floor damping. Allen and Rainer also noted that that steel beams and concrete slabs displayed composite action for vibration analysis even where shear studs were not present. This means that the transformed moment of inertia, based on full composite action, should be used in vibration analysis.

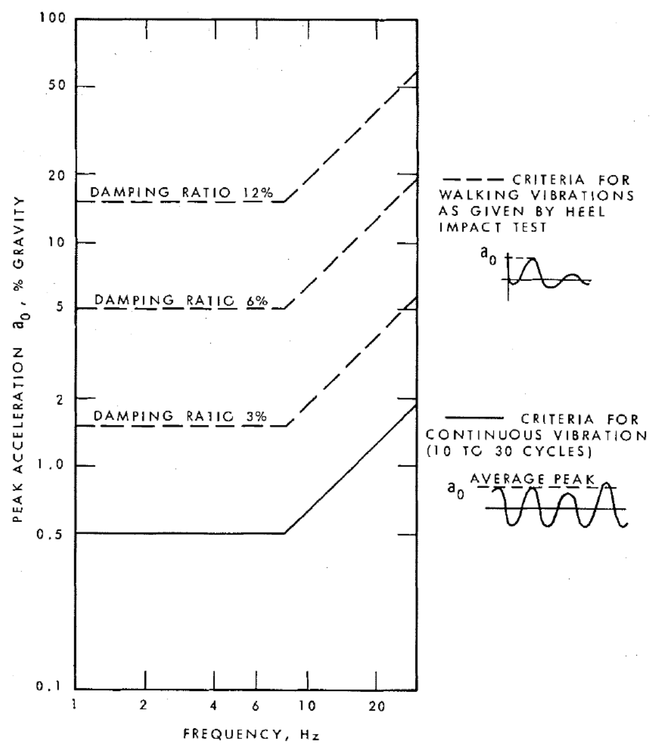

Figure 3: Allen and Rainer Scale for Walking (Allen & Rainer, 1976)

1981 – Thomas Murray. Thomas Murray field tested 91 steel joist or steel beam and concrete slab floors with the heel drop test to compare the existing vibration criteria at the time (Murray, 1981). Murray used five different existing systems to compare the criteria. The first system was a three story office building. The floor system was "a concrete slab over a cellular deck supported by welded steel beams and girders fabricated from 50 ksi yield plates" (Murray, 1981). System two was a large shopping mall with 1 ½” clay tile over a 2 ½” concrete slab on a metal that is supported by wide flange sections. The third system was a “laboratory mockup of a proprietary dry floor system for use in pre-engineered multi-story office buildings” (Murray, 1981). System four was a conventional high rise office building. The floor system consisted of a 5 inch lightweight concrete that was supported by W16x31 beams at 9 ft – 4 in on center that span 28 feet. The last system was similar to system four. “A 4 ½” lightweight concrete slab is supported by W16x36 A36 steel beams at 10 ft – 4 in o.c. and spanning 31 feet” (Murray, 1981).

Both the amplitude of the vibration and the frequency were obtained from each test, and each floor was evaluated by observers to be either acceptable or unacceptable. Murray noted a strong dependence of acceptable amplitude on damping as previously noted by other researchers. Murray compared 5 different acceptance criteria: the U.S. Department of Housing and Urban Development scale, the Canadian Standards Association scale, the International Organization for Standardization scale, the modified Reiher – Meister scale, and the Allen and Rainer scale, in addition to proposing new acceptance criteria. It was concluded that the criterion at the time was inconsistent and underestimated the dependence of the amplitude of vibration on damping. Therefore, the following equation was proposed as new acceptability criteria.

$$D \geq 35A_o f + 2.5 \qquad \textbf{Equation 2}$$

Where,

D = percent of critical damping (%g)

A_o = initial amplitude from a heel drop impact (in)

f = first natural frequency of the floor system (Hz)

1993 – D. E. Allen and T. M. Murray. Allen and Murray expanded upon their previous knowledge of floor vibrations to propose a new design criteria based on harmonic resonance of floor systems (Allen & Murray, 1993). The criteria and estimation of the floor parameters are similar to that used in the current AISC Design Guide 11: Vibrations of Steel-Framed Structural Systems Second Edition shown in Equation 3.

$$f_o \geq 2.86 ln \left[\frac{K}{\beta W}\right]$$ **Equation 3**

Where,

f_o = fundamental natural frequency of the floor system (Hz)

K = a constant that depends on the acceleration limit for the occupancy

β = damping ratio of the floor system

W = weight of the beam (lb)

The criteria prescribes the lowest acceptable frequency of the floor system for the satisfaction of the occupants. The analysis methods to determine the natural frequency of the floor system are described in the next section.

Analysis Methods History

This section provides an overview of the how the analysis methods for floor vibration has changed over the past 100 years. The analysis methods include how to estimate the parameters to determine the floor characteristics such as the frequency and amplitude.

1913 – Charles Tilden. Charles Tilden experimentally quantified the dynamic load effects of people on a structure. He investigated the increased load on a structure that people create by walking, standing, or jostling in addition to their body weight (Tilden, 1913). The load types were divided into two categories: vertical and horizontal. To test the dynamic vertical load, Tilden had his subjects stand rapidly from a crouching position. He measured the increase in load as the subjects stood up from the crouched position on an ordinary platform scale. Tilden also tested the subjects suddenly rising from the seated position for both vertical and horizontal forces. The last vertical test conducted was used to estimate the maximum vertical effect an individual could produce. The subject bent their knees slightly and quickly straightened them while jerking their arms and shoulders downward. Lastly, Tilden measured the horizontal forces exerted by a subject walking on a level floor and a subject running across a bridge.

Tilden made a few conclusions from his experiments. First, the test for the subject standing from a crouching position does not have practical applications; it verified the kinetic loading effect of a subject moving within a structure (Tilden, 1913). Second, Tilden concluded when a subject stands rapidly from a seated position has a practical application in an athletic stadium, such as when a sports team scores and the crowd stands at the same time. He found a 65% and 70% increase in vertical load when a subject stood from the seated position; however, Tilden concluded that the vertical load would most likely be within the 100 psf design load. The horizontal component would be an additional design load of 70 or 80 lb. Tilden also concluded that the horizontal effects from one subject walking across a floor was not of importance, except in the case of a crowd.

1975 – Thomas Murray. Dr. Murray set out to find an analysis procedure that would determine if an office floor would vibrate in a range that was both perceivable and annoying to the occupants (Murray, 1975). Murray tested over 100 steel beam and concrete slab floors to derive his procedure that used heel drop impacts as the source of vibration. At the time of the experiments, Murray did not have an accurate way of determining the amount of damping, rather he estimated the amount of damping within the floor. If the damping was estimated to be greater than 8-10%, Murray concluded the vibration analysis was not needed since the floor would not produce annoying vibrations. If the damping was estimated to be below that threshold, Murray presented a design procedure that used the floor frequency and amplitude along with the Modified Reiher – Meister scale to determine the acceptability of the floor system.

1993 – D. E. Allen and T. M. Murray. Allen and Murray expanded on their previous knowledge to create new design criteria and analysis methods similar to those used in the current edition of Design Guide 11 (Allen & Murray, 1993). The damping ratio of a floor system was determined to be linked to the non-structural components of a floor system. Allen and Murray found that previous values, found in 1975, for the damping ratio were overestimating the damping occurring in the floor system. The 1975 values were based on the vibration dispersion from a heel drop test that included frictional and material damping. Allen and Murray, through more testing, determined that the actual frictional and material damping were approximately half of the previous values.

When analyzing a floor system, Allen and Murray assumed the floor consisted of a concrete slab supported by steel joists that are either supported by walls or steel girders (Allen & Murray, 1993). In order to estimate the natural frequency of this floor system, Allen and Murray determined that the joist panel and the girder panel had to be analyzed separately before combining for the entire floor system. The frequency of the joist/girder panel is estimated using Equation 4.

$$f_{j/g} = 0.18\sqrt{\frac{g}{\Delta_{j/g}}} \qquad \textbf{Equation 4}$$

Where,

The j and g indicate the joist or the girder panel, respectively.

f = natural frequency (Hz)

g = acceleration due to gravity (in/s^2)

$= 386$ in/s^2

$\Delta_{j/g}$ = deflection of the joist or girder due to the weight supported (in)

The combined joist and girder mode's frequency then can be estimated using Equation 5. The combined frequencies of the two modes reduces the flexibility of the floor and thus makes the floor more susceptible to vibration annoyance.

$$f_o = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} \qquad \textbf{Equation 5}$$

f_o = natural frequency (Hz)

g = acceleration due to gravity (in/s^2)

$= 386$ in/s^2

$\Delta_{j/g}$ = deflection of the joist or girder due to the weight supported (in)

1996 – A. Ebrahimpour, L. Sack, A. Hamam, and W. N. Patten. Ebrahimpour, Hamam, Sack, and Patten experimentally investigated the effect of dynamic loads due to moving crowds (Ebrahimpour, Hamam, Sack, & Patten, 1996). The experiment used a force platform to measure the force of individuals and groups of people walking across the platform. For each group type, the subjects walked normally at a specific pace frequency as prompted by sound signals. "Tests involving one person were used to statistically characterize individual loads. Tests involving two people were used to quantify the coherency of motion. Four-people tests were used to verify the group load modeling and simulation results" (Ebrahimpour, Hamam, Sack, & Patten, 1996). Once the experiments were concluded, the researchers used a simulation software to simulate subjects walking across the floor to be able to run multiple simulations. "A Monte Carlo simulation program called "MCLSIM" (Moving Crowd Load Simulation) was written specifically to do this task" (Ebrahimpour, Hamam, Sack, & Patten, 1996). The simulated values and the experimental average root mean square (RMS) values matched closely; therefore, simulation software was used to simulate a crowd of people walking with the same step frequency. The researchers observed that the dynamic load factors for the same walking frequency for normal walking were less than that of those for walking prompted by the sound signals. It was concluded the prompted walking is the more severe loading case.

Current Acceptance Criteria and Analysis Methods

The current acceptance criteria and analysis methods build upon the historical criteria and methods summarized previously. For vibration analysis in the United States of America, AISC Design Guide 11: Vibration of Steel-Framed Structural Systems gives the most up to date criteria and analysis methods for many different types of vibrations in floors (Murray, Allen, Ungar, & Davis, 2016). In the next section, the acceptance criteria and analysis methods from Design Guide 11 are summarized for vibrations due to running.

Chapter 3 - Current Analysis Method of Floor Vibration for Running

The current edition of AISC Design Guide 11 provides analysis procedures for occupant induced floor vibration under different types of excitations. The design guide addresses vibrations due to walking, running, and rhythmic activities as well as gives design criteria for sensitive occupancies and sensitive equipment. This chapter introduces the analysis techniques and the acceptability criteria of floor vibrations due to a running excitation. Design Guide 11 applies to floors made of a concrete slab supported by joists or beams supported by steel girders, walls, or joist girders. In the design guide, the equations for joists and beams are the same; therefore, joists and beams are interchangeable within this report.

Design guide 11 provides equations to estimate the acceleration of a floor due to occupants running. In order to estimate the floor acceleration, the fundamental frequency, the effective panel weight, and the damping ratio must be estimated. How to apply the equations in Design Guide 11 to find these parameters is outlined in the following section. Once the floor acceleration is estimated, it must be compared to pre-determined acceptance criteria, outlined in the second section of this chapter.

Estimation of Parameters

To determine the acceptability of vibration for a floor system, the fundamental natural frequency of the floor, the effective panel weight, the damping ratio, and the magnitude of the floor acceleration must first be estimated. The equations to estimate these parameters as well as the explanation on how to apply these parameters are given in this section.

Fundamental Natural Frequency of Floors

The fundamental natural frequency of the floor's vibration dominates the floor's response. Therefore, the fundamental natural frequency is the only frequency used in vibration analysis, while the higher frequencies are neglected. To determine an entire floor's fundamental natural frequency, the vibration modes of the joists and the girders need to be analyzed separately and then combined. The fundamental natural frequency of a simply supported beam with a uniform mass is given by Equation 6 (Murray, Allen, Ungar, & Davis, 2016). This equation can be used for both the joists and girders of a floor system.

$$f_n = \frac{\pi}{2}\left(\frac{gE_sI_t}{wL^4}\right)^{1/2}$$ **Equation 6**

Where,

f_n = natural frequency (Hz)

E_s = modulus of elasticity of steel (ksi)

= 29,000 ksi

I_t = transformed moment of inertia (in^4)

L = member span (in)

g = acceleration of gravity = 386 in/s^2

w = uniformly distributed weight per length supported by the member (k/in)

The combined joist and girder panel natural frequency is calculated by Equation 7.

$$\frac{1}{{f_n}^2} = \frac{1}{f_j^2} + \frac{1}{f_g^2}$$ **Equation 7**

The uniform distributed load should be the actual loads that the beam or girder is expected to support, not the design loads for other limit states. In the case of vibrations, the design is unconservative if the dead and live loads are overestimated. The smaller the load, the larger the response of the system. Therefore, if the dead and live loads are overestimated, a floor may have undesirable vibrations once in use when the analysis indicated the floor was acceptable. In the case of floors under running excitations, the structural actual weight and the actual number of occupants should be used to estimate the loads when determining the total dead and live load on a floor system. If a live load fluctuates with the time of day or time of year, the lowest load should be used to estimate the worst vibration response. Since vibration is a serviceability concern, load factors should not be applied.

The transformed moment of inertia, using the full composite section, is used in the calculations. As long as the slab or deck is connected to the supporting member, even without structural shear connectors, the system exhibits composite action in dynamic loading (Murray, Allen, Ungar, & Davis, 2016). In addition, concrete is stiffer under dynamic loading than under static loading. "To account for the greater stiffness of concrete on metal deck under dynamic loading, as compared to static loading, it is recommended that the concrete modulus of elasticity be taken equal to 1.35 time that specified in current structural standards for calculation of the transformed moment of inertia" (Murray, Allen, Ungar, & Davis, 2016).

The following is an alternative procedure to estimate the natural frequency of a floor system with simply supported members. Equation 6 can be written as the following equations.

$$f_n = 0.18\sqrt{\frac{g}{\Delta}}$$ **Equation 8**

Where,

f_n = natural frequency (Hz)

g = acceleration due to gravity (in/s^2)

= 386 in/s^2

Δ = midspan deflection of the member relative to the supports due to the supported weight (in)

$$= \frac{5wL^4}{384E_sI_t} \qquad \textbf{Equation 9}$$

w = uniformly distributed weight per length supported by the member (k/in)

L = length of span (in)

E_s = modulus of elasticity of steel (ksi)

= 29,000 ksi

I_t = transformed moment of inertia (in^4)

If Equation 8 is used to determine the joist and girder frequencies, then the combined mode frequency is calculated by Equation 10.

$$f_n = 0.18\sqrt{\frac{g}{\Delta_j + \Delta_g}} \qquad \textbf{Equation 10}$$

Where,

f_n = natural frequency (Hz)

g = acceleration due to gravity (in/s^2)

= 386 in/s^2

Δ = midspan deflection of the member relative to the supports due to the supported weight (in)

Effective Panel Weight

Similar to the natural frequency, the effective weight of a floor system is determined by analyzing the joist and girder modes separately then combining the two. The effective panel weight for both joist and girder panels is determined by Equation 11 (Murray, Allen, Ungar, & Davis, 2016).

$$W = wBL \qquad \textbf{Equation 11}$$

Where,

W = effective panel weight of joist or girder (lb)

B = effective panel width (ft)

L = member span (ft)

w = supported weight per unit area (psf)

Again, the weight used in these equations are the actual loads the floor is expected to support, not the design loads. This includes the structural self-weight, actual dead load, and actual live load. Design Guide 11 provides a table that lists the live loads that should be used in vibration analysis. For example, when designing an electronic office, the typical design dead and live loads are 20 psf and 50 psf, respectively. The dead and live loads for vibration analysis would be 10 psf and 6 psf, respectively.

To calculate the effective panel weight for a joist, the effective width is given in Equation 12.

$$B_j = C_j \left(\frac{D_s}{D_j}\right)^{1/4} L_j \leq \left(\frac{2}{3}\right) floor\ width \qquad \textbf{Equation 12}$$

Where,

B_j = joist effective width (ft)

C_j = 2.0 for typical joists

= 1.0 for joists parallel to a free edge as in an edge of a balcony

D_j = joist transformed moment of inertia per unit width (in^4/ft)

$$= \frac{I_j}{S}$$

L_j = joist span (ft)

S = joist spacing (ft)

D_s = slab transformed moment of inertia per unit width (in^4/ft)

$$= \frac{12d_e^3}{12n} \qquad \textbf{Equation 13}$$

d_e = effective depth of the concrete slab, taken as the depth of the concrete above the deck plus one half the depth of the deck (in)

n = dynamic modular ratio

$$= \frac{E_s}{1.35E_c} \qquad \textbf{Equation 14}$$

E_c = modulus of elasticity of concrete (ksi)

$$= w^{1.5}\sqrt{f'_c} \qquad \textbf{Equation 15}$$

E_s = modulus of elasticity of steel (ksi)

= 29,000 ksi

I_j = transformed moment of inertia of the joist (in^4)

floor width = distance perpendicular to the joist spans where the framing is nearly identical

For girder panel modes, the effective width is given in Equation 16.

$$B_g = C_g \left(\frac{D_s}{D_g}\right)^{1/4} L_g \leq \left(\frac{2}{3}\right) floor\ length \qquad \textbf{Equation 16}$$

Where,

B_g = girder effective width (ft)

C_g = 1.6 for girders supporting joists connected to the girder flange with joist seats

= 1.8 for girders supporting beams connected to the girder web

D_g = girder transformed moment of inertia per unit width (in^4/ft)

= I_g divided by the average span of the supported joists

L_g = girder span (ft)

floor length = distance perpendicular to the girder spans where the framing is nearly identical

To combine the joist and girder panel effective weights Equation 17 should be used.

$$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g \qquad \textbf{Equation 17}$$

Where,

W = total effective panel weight (lb)

W_j = effective panel weight for the joist mode (lb)

W_g = effective panel weight for the girder mode (lb)

Δ_j = midspan deflection of the joist due to the weight supported by the member (in)

Δ_g = midspan deflection of the girder due to the weight supported by the member (in)

If the girder span is less than the joist panel width, then the combined mode is restricted, thus the combined panel weight can be modified by reducing the girder panel deflection as shown in Equation 18.

$$\Delta'_g = \frac{L_g}{B_j} \Delta_g \qquad \textbf{Equation 18}$$

Δ'_g = modified girder deflection (in)

L_g = girder length (ft)

B_j = effective width of joist (ft)

Δ_g = original girder deflection (in)

Damping Ratio

Recommended damping ratios given in Design Guide 11 estimate the actual damping in a floor. The pertinent values for floors that have running as a source of excitation are listed in Table 1: Damping Ratio Values from Deign Guide 11. To determine the damping ratio in a floor, the pertinent values from Table 1 are summed. For example, if a floor system supports ceiling and ductwork, but does not have full height wall partitions in the bay, then the damping ratio would be 0.01 plus 0.01 for a total of 0.02.

Table 1: Damping Ratio Values from Deign Guide 11

Component	Damping Ratio
Structural System	0.01
Ceiling and Ductwork	0.01
Full-Height Dry Wall Partitions in Bay	0.01

Floor Acceleration

For running on a level surface, such as a recreation center's running track, Design Guide 11 gives an inequality that determines the acceptability of a floor. The actual acceleration is shown in Equation 19. The estimated floor acceleration must then be compared to the established acceptance limits in Design Guide 11.

$$\frac{a_p}{g} = \frac{0.79Q(e^{-0.173f_n})}{\beta W}$$ **Equation 19**

Where,

a_p/g = ratio of peak acceleration to gravitational acceleration

Q = bodyweight for the running activity (lb)

= 168 lb for recreational runners to 250+ for some athletes

f_n = natural frequency (Hz)

β = damping ratio

W = effective weight of the floor (lb)

Equation 19 estimates the floor acceleration due to one runner. The weight of the runner can vary from 168 lb for recreational runners to 250+ lb for some athletes such as football players. Engineering judgement should be used to select the occupant weight that best matches the occupants using the running track.

Acceptance Criteria

Due to the lack of research on running on a level floor, the authors of Design Guide 11 used their engineering judgment to determine the acceleration limits (Murray, Allen, Ungar, & Davis, 2016). The acceleration limits are based on the baseline curve for human response to continuous sinusoidal accelerations, as shown in Figure 4. The natural frequency of the floor is determined in the previous section, and the acceleration limit is given by the curve for the floor occupancy. The acceleration limit determined by Figure 4 is then compared to the actual floor acceleration as shown in Equation 20. Based on Design Guide 11, Equation 20 is dependent upon the weight of one runner on the floor, the natural frequency of the floor, the damping ratio, and the effective panel weight. The following chapter is a case study that will show more in detail how to use the graph in Figure 4. In addition to giving acceleration limits, Design Guide 11 also suggests a minimum value for the natural frequency of floors where running occurs. Floors with a natural frequency of 1.6 to 4 Hz must be very heavy to ensure that there are not annoying vibrations; therefore, the minimum natural frequency is 4 Hz to keep floors more economical.

$$\frac{a_p}{g} = \frac{0.79Q(e^{-0.173f_n})}{\beta W} \leq \frac{a_o}{g} \qquad \textbf{Equation 20}$$

Where,

a_p/g = ratio of peak acceleration to gravitational acceleration

Q = bodyweight for the running activity (lb)

= 168 lb for recreational runners to 250+ for some athletes

f_n = natural frequency (Hz)

β = damping ratio

W = effective weight of the floor (lb)

a_o/g = acceleration tolerance limit

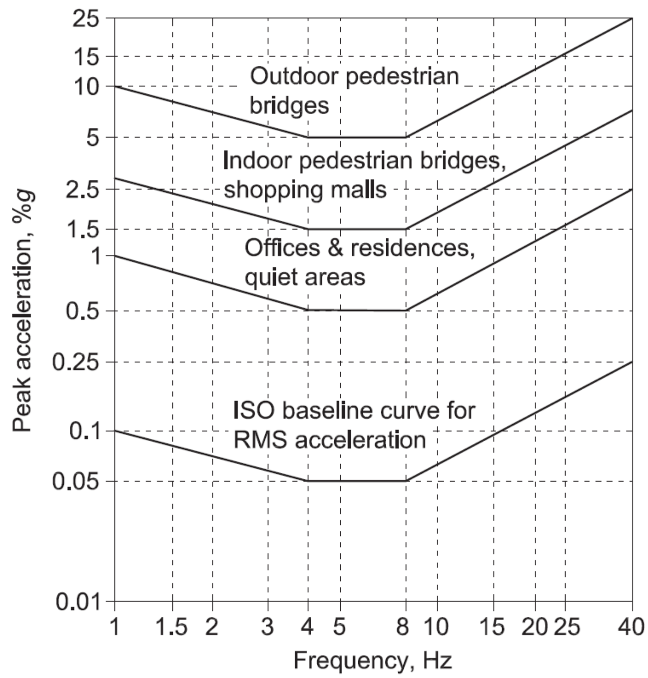

Figure 4: Recommended Tolerance Limits for Human Comfort (Murray, Allen, Ungar, & Davis, 2016)

Chapter 4 - Case Study – Running Track in the Chester E. Peters Recreation Center

The Chester E. Peters Recreation Center is Kansas State University's home for recreation services. The building was first constructed in 1980 and has since been renovated and expanded to encompass a total area of 257,000 square feet. The complex includes three gyms, squash courts, a weight room, cardio areas, a rock wall, and two indoor running tracks among other amenities. The 1/5-mile indoor running track serves as a case study for this report to apply the provisions given in Design Guide 11. The 1/5-mile track is on the second floor of the Chester E. Peters Recreation Center. Full floor plans and sections are in Appendix A. Partial plans for the first floor, second floor, and roof are shown in Figure 5, Figure 6, and Figure 7, respectively. A slab section is given in Figure 8. A straight portion of the running track, from grid line 1.5 to 5', is used to estimate the floor acceleration when an occupant runs across the track.

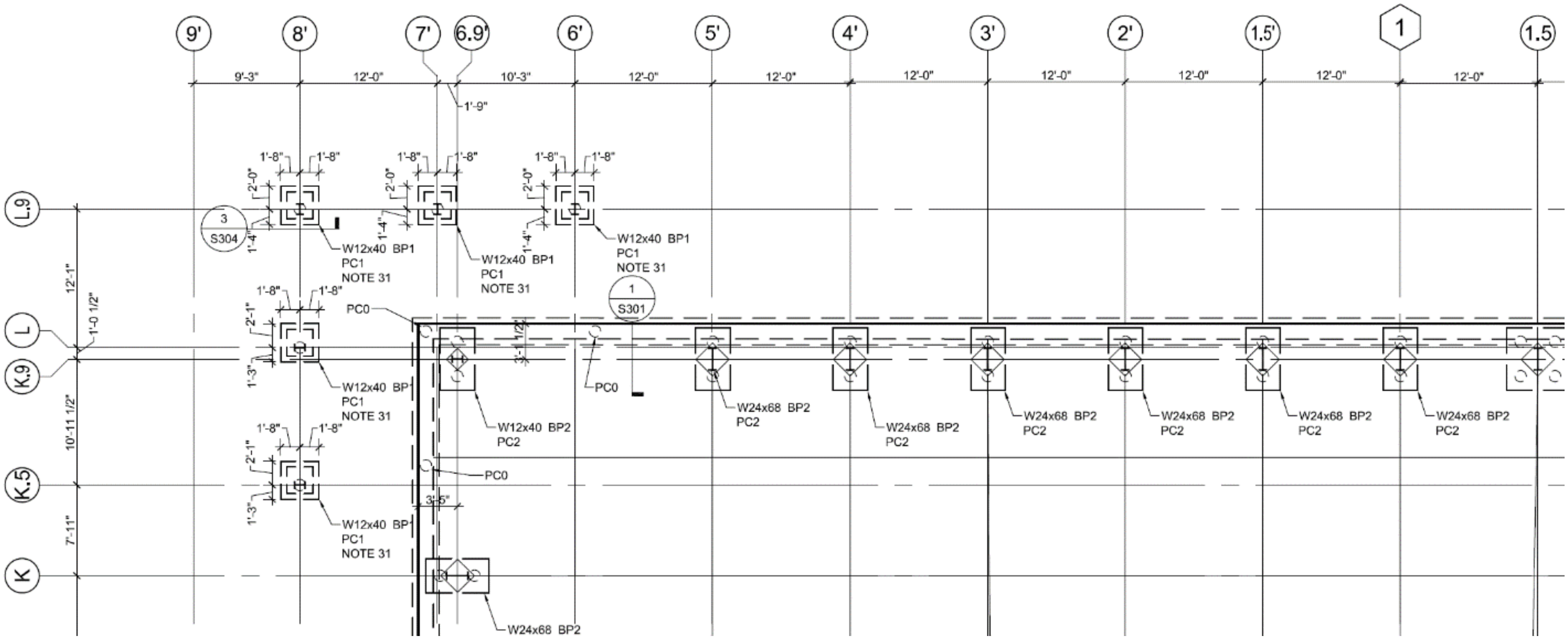

Figure 5: Partial First Level Floor Plan

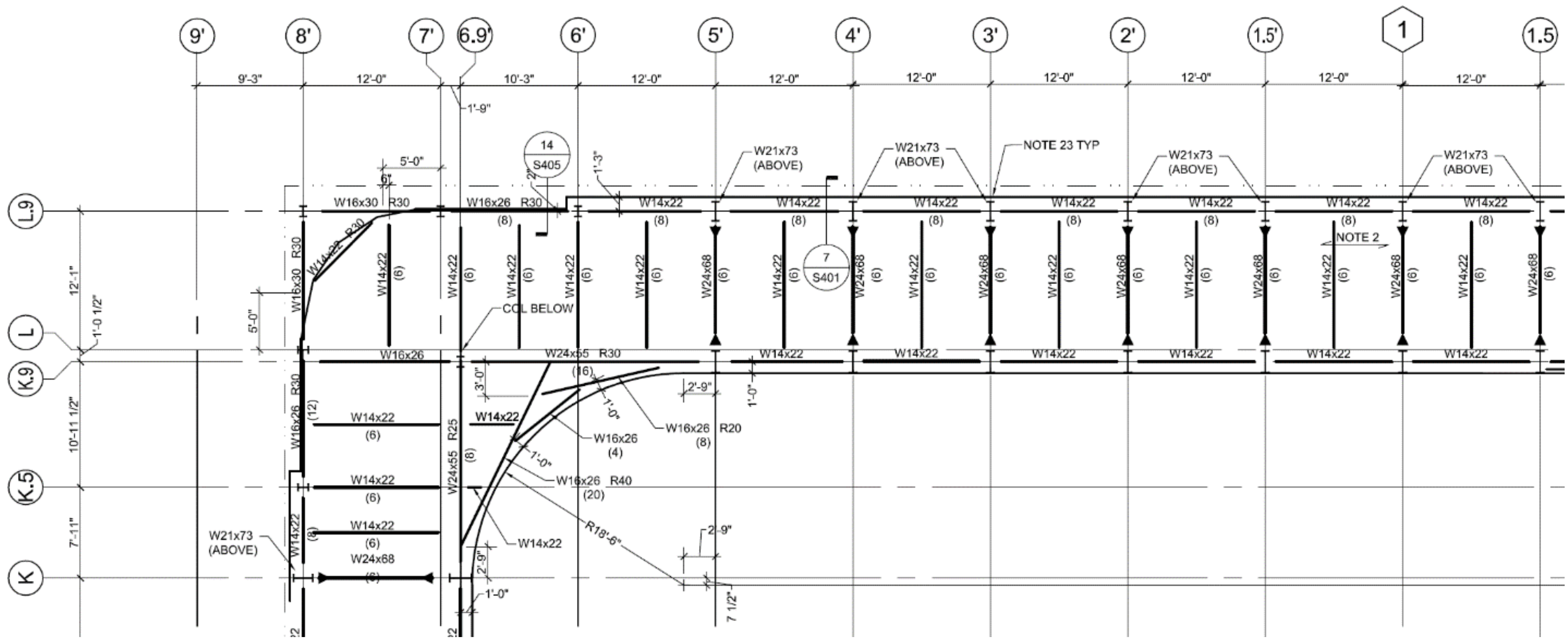

Figure 6: Partial Second Level Floor Plan

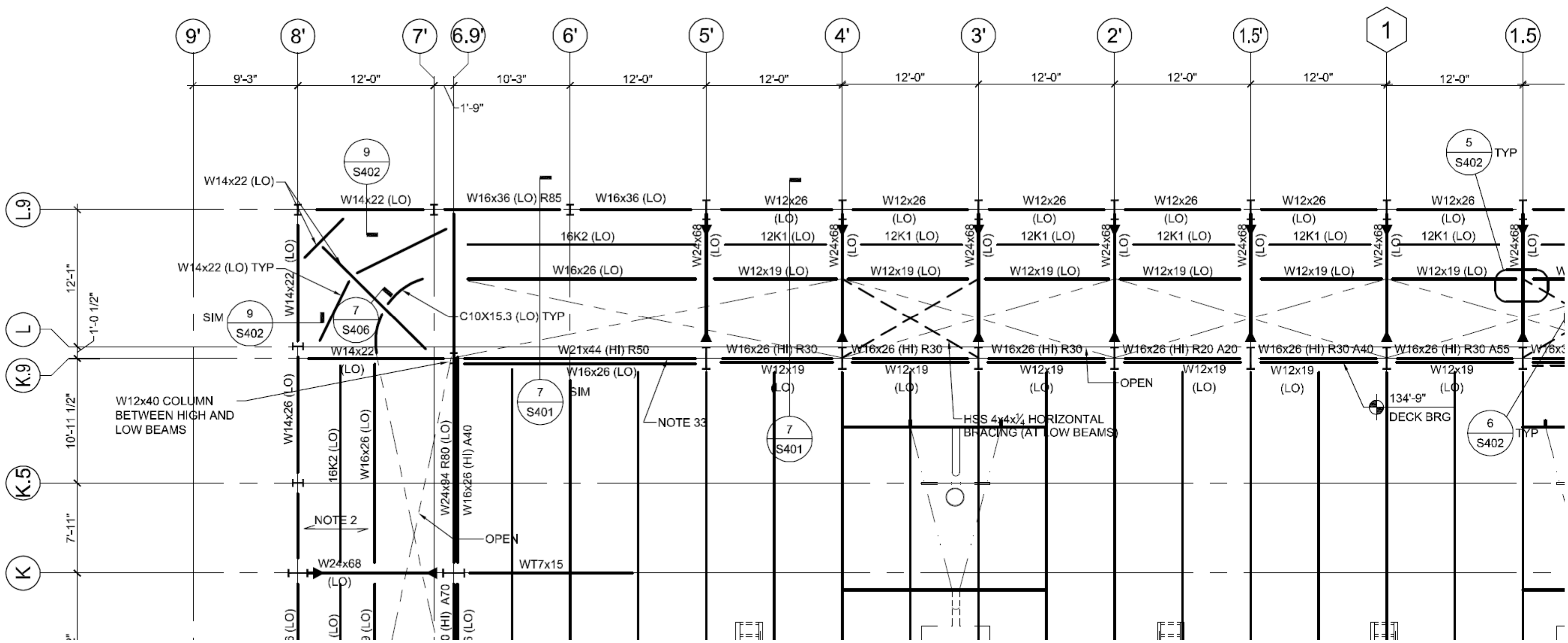

Figure 7: Partial Roof Plan

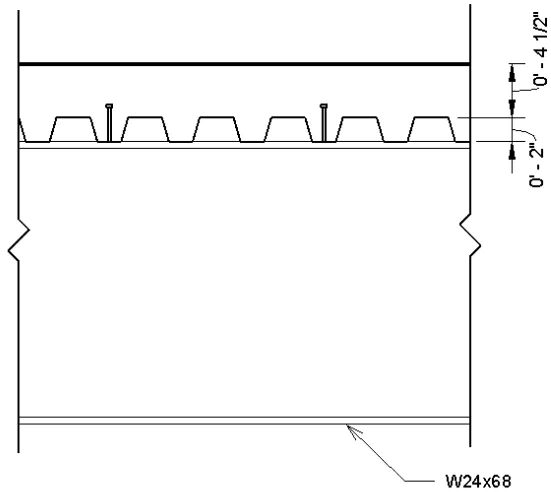

Figure 8: Slab Section

Section Properties

Material and section properties of the running track including properties for the steel beams, girders and concrete slab are introduced, and the floor's frequency is analyzed based on these properties.

Concrete Slab and Metal Deck Properties

The concrete slab is 4.5 inches of normal weight concrete (145 pcf) on top of a 2 inch deep 18 gauge composite steel deck for a total slab thickness of 6.5 inches. From the general notes of the Chester E. Peters Recreation Complex structural drawings, the 28-day specific compressive strength of the concrete is 4 ksi. The modulus of elasticity of the concrete is shown in Equation 21.

$$E_c = w^{1.5}\sqrt{f_c'}$$ **Equation 21**

$$= (145\ pcf)^{1.5}\sqrt{4\ ksi}$$

$$= 3492\ ksi$$

The modular ratio of steel to concrete is found by Equation 22.

$$n = \frac{E_s}{1.35E_c}$$

$$= \frac{29000\ ksi}{1.35(3492\ ksi)} \qquad \textbf{Equation 22}$$

$$= 6.152$$

The weight of the slab and the metal deck is shown below. Based on the metal deck profile, the concrete in the ribs of the metal deck is 50% of the volume if there were no ribs. The deck weighs 2.61 psf (Vulcraft, 2008).

$$w_s = \left(4.5\ in + \frac{2\ in}{2}\right)\left(\frac{1\ ft}{12\ in}\right)(145\ pcf) + 2.61\ psf = 69.07\ psf$$

Steel Properties

The beams are composite W14x22 and the girders are W24x68 with the section properties given in Table 2. All beams and girders are of A992 steel. For all beams and girders, a dead load of 10 psf is assumed to incorporate the hanging ceiling and mechanical equipment. The live load is assumed to be 0 psf because only the runner under consideration is assumed to be on the floor with no other live load.

Table 2: Beam and Girder Section Properties (American Institue of Steel Construction, 2011)

Section Size	A (in^2)	I_x (in^4)	d (in)
W14x22	6.49	199	13.7
W24x68	20.1	1830	23.7

Beam Moment of Inertia and Deflection

The first member analyzed is the beam highlighted in Figure 9. The effective slab width, as shown in Figure 10 and Equation 23, is determined by Chapter I of the AISC specification. The effective slab width is the sum of the effective widths for each side of the beam centerline, each of which shall not exceed: (1) one-eigth of the beam span, (2) one-half the distance to the centerline of the adjacent beam, or (3) the distance to the edge of the slab (American Institute of Steel Construction, 2010). The effective width equations from AISC Design Guide 11 shown in Chapter 3 are not used here because the floor has girders that are both perpendicular and parallel to the beams. Therefore, the floor framing does not match that prescribed in Design Guide 11.

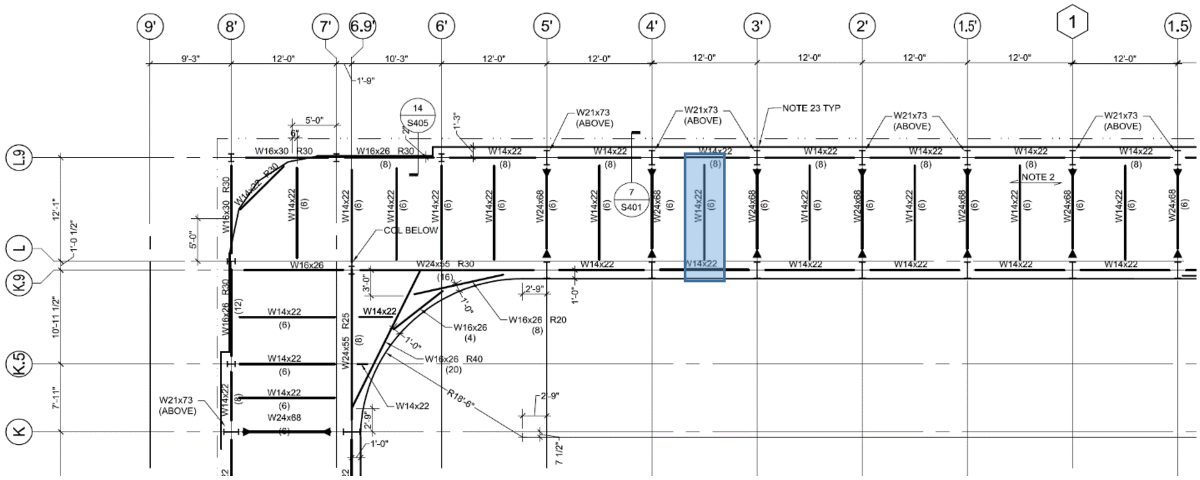

Figure 9: Beam

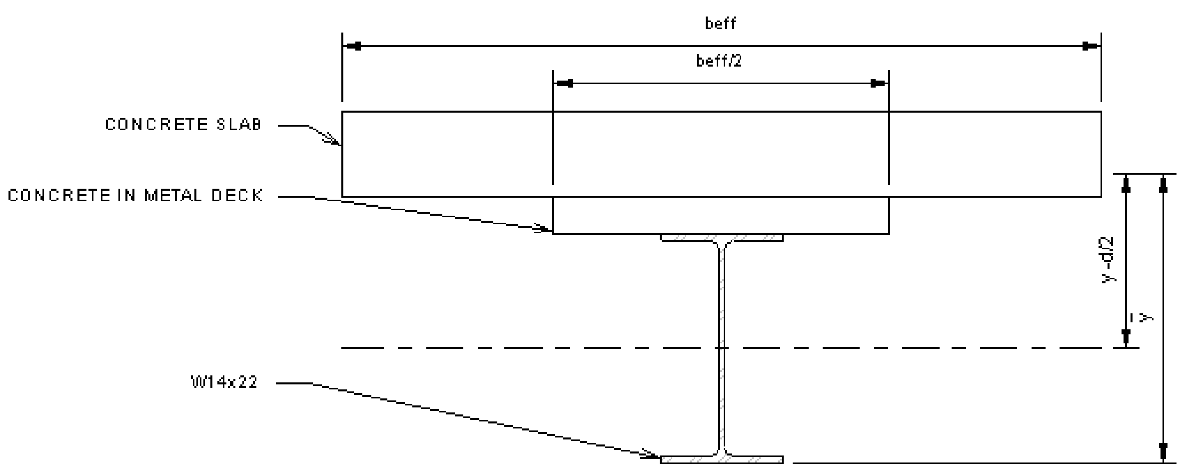

Figure 10: Beam and Slab Diagram

$$b_{eff} = min\left(\frac{1}{8}L_{beam}, \frac{1}{2}S, D\right) \quad \textbf{Equation 23}$$

Where,

b_{eff} = effective width of the beam (ft)

L_{beam} = length of the beam (ft)

S = distance to the centerline of the adjacent beam (ft)

D = distance to the edge of the slab (ft)

$$b_{eff} = min\left(\frac{1}{8}(12.08\ ft), \frac{1}{2}(12\ ft), N/A\right) + min\left(\frac{1}{8}(12.1\ ft), \frac{1}{2}(12\ ft), N/A\right) = 3.021\ ft$$

The effective width of concrete in the deck is half of the effective width of the concrete slab. It was assumed that the ribs in the metal deck take out half of concrete. The transformed areas of the slab and the deck are found by Equation 24.

$$A_{slab} = \frac{b_{eff}}{n}t$$ **Equation 24**

Where,

A_{slab} = area of the slab (in^2)

b_{eff} = effective width of beam (in)

n = dynamic modular ratio

t = thickness of slab (in)

$$A_{slab} = \frac{(3.021\ ft)\left(\frac{12\ in}{1\ ft}\right)}{6.152}(4.5\ in) = 26.52\ in^2$$

$$A_{deck} = \frac{(1.510\ ft)\left(\frac{12\ in}{1\ ft}\right)}{6.152}(2\ in) = 5.891\ in^2$$

The distance to the neutral axis is calculated by Equation 25.

$$\bar{y} = \frac{\sum A_i y_i}{\sum A_i}$$ **Equation 25**

$$= \frac{\left(26.52\ in^2\left(13.7\ in + 2\ in + \frac{4.5\ in}{2}\right)\right) + \left(5.891\ in^2\left(13.7\ in + \frac{2in}{2}\right)\right)}{26.52\ in^2 + 5.891\ in^2 + 6.49\ in^2}$$

$$+ \frac{\left(6.49\ in^2\left(\frac{13.7\ in}{2}\right)\right)}{26.52\ in^2 + 5.891\ in^2 + 6.49\ in^2}$$

$$= 15.61\ in$$

The transformed moment of inertia for the steel beam, concrete slab, and deck is determined by Equation 26.

$$I_{trans} = (I_{concrete} + A_{concrete}d_{concrete}^2) + (I_{steel} + A_{steel}d_{steel}^2)$$ **Equation 26**

$$= \left[\left(44.75\ in^4 + (26.52\ in^2(13.01\ in)^2)\right)\right.$$

$$\left. + (1.963\ in^4 + (5.891\ in^2)(9.760\ in)^2)\right]$$

$$+ (199\ in^4 + (6.49\ in^2)(8.760\ in)^2)$$

$$= 5794\ in^4$$

This first beam is a simply supported beam with a distributed load as seen in Figure 11. The distributed load along the beam consists of the self-weight of the beam, dead load, live load and the weight of the metal deck and concrete as shown in Equation 27.

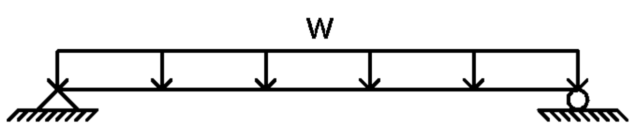

Figure 11: Simply Supported Beam with Distributed Load

$$w = (69.07\ psf + 10\ psf)(6\ ft) + 22\frac{lb}{ft} = 0.496\ klf$$ **Equation 27**

The deflection of the simply supported beam is given in Equation 28.

$$\Delta = \frac{5wl^4}{384EI_{trans}}$$ **Equation 28**

Where,

Δ = midspan deflection of the member relative to the supports due to the supported weight (in)

w = uniformly distributed weight per length supported by the member (k/in)

L = length of span (in)

E_s = modulus of elasticity of steel (ksi)

= 29,000 ksi

I_t = transformed moment of inertia (in^4)

$$\Delta_{beam} = \frac{5\left(0.496\frac{k}{ft}\right)(12.08\ ft)^4\left(\frac{12\ in}{1\ ft}\right)^3}{384(29000\ ksi)(5794\ in^4)}$$

$$= 0.0014\ in$$

Due to the fact that the first beam is supported by the perpendicular beams, the end reaction of the first beam must be found to know the force that transfers to the second beam. The end reaction is shown in Equation 29.

$$\sum F_y = -\left(0.436\frac{k}{ft}\right)(12.08\ ft) + 2R = 0$$ **Equation 29**

$$R = 2.634\ k$$

Girder One Moment of Inertia and Deflection

The second member analyzed is the girder highlighted in Figure 12. The effective slab width is shown below and in Figure 13.

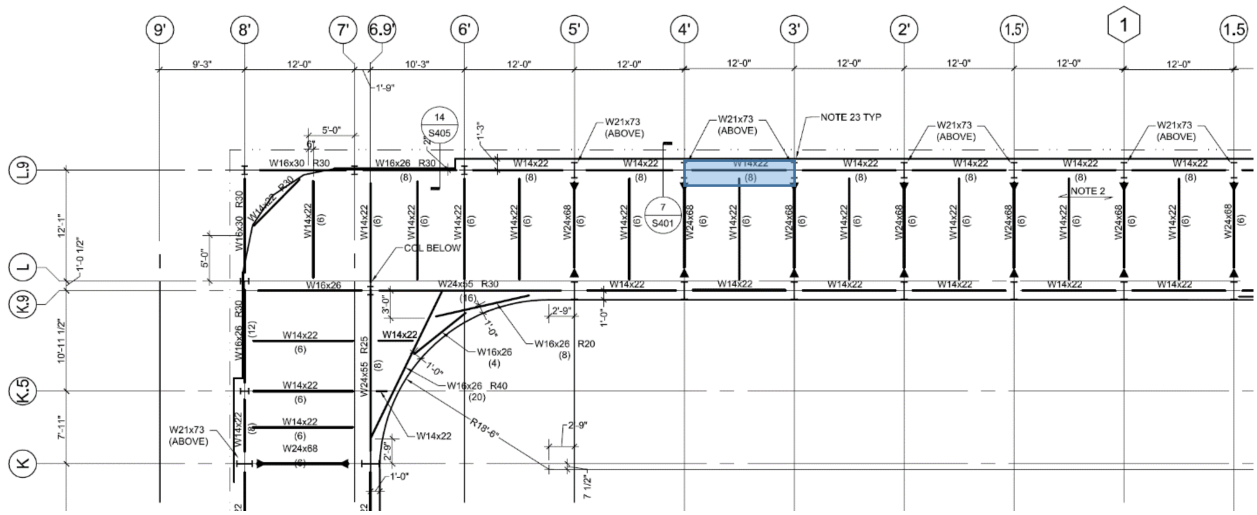

Figure 12: Girder One

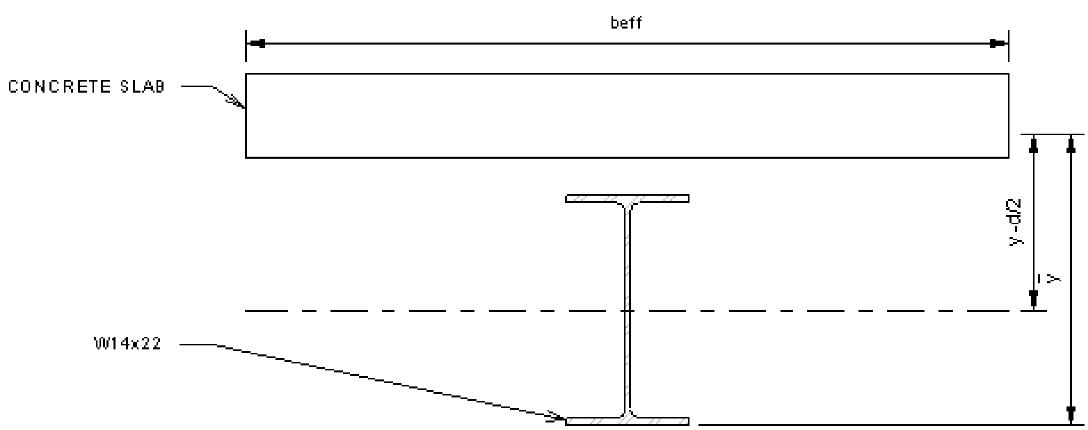

Figure 13: Girder One and Slab Diagram

$$b_{eff} = min\left(\frac{1}{8}(12\ ft), \frac{1}{2}(12.08\ ft), 1.250\right) + min\left(\frac{1}{8}(12\ ft), \frac{1}{2}(12.08\ ft), N/A\right)$$

$$= 2.750\ ft$$

The transformed area of the concrete is shown below.

$$A_c = \frac{(2.750\ ft)\left(\frac{12\ in}{1\ ft}\right)}{6.152}(4.50\ in)$$

$$= 24.14\ in^2$$

The distance to the neutral axis is calculated below.

$$\bar{y} = \frac{\left(24.14\ in^2 \left(13.7\ in + 2\ in + \frac{4.5\ in}{2}\right)\right) + \left(6.49\ in^2 \left(\frac{13.7\ in}{2}\right)\right)}{24.14\ in^2 + 6.49\ in^2}$$

$$= 15.60\ in$$

The transformed moment of inertia for the steel girder and concrete slab is shown below.

$$I_{trans} = (40.73\ in^4 + (24.14\ in^2)(13.00\ in)^2)$$

$$+ (199\ in^4 + (6.49\ in^2)(8.750\ in)^2)$$

$$= 4816\ in^4$$

The girder is a simply supported member with a point load from the beam at the midspan and a distributed load as seen in Figure 14. The distributed load along the girder consists of the self-weight of the girder, the weight of the metal deck and the weight of concrete that cantilevers from the girder.

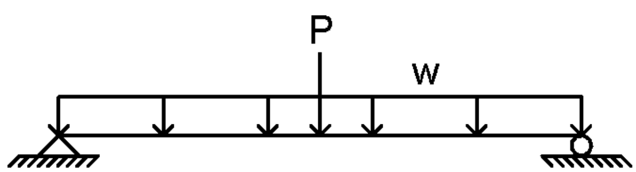

Figure 14: Simply Supported Girder with Distributed Load and Point Load

$$w = (69.07\ psf)(1.25\ ft) + 22\frac{lb}{ft} = 0.1083\ klf$$

The deflection of the girder due to the distributed load is found by using Equation 28 as shown below.

$$\Delta_{girder\_1,w} = \frac{5\left(0.1083\frac{k}{ft}\right)(12\ ft)^4\left(\frac{12\ in}{1\ ft}\right)^3}{384(29000\ ksi)(4816\ in^4)}$$

$$= 0.00036\ in$$

The deflection of the girder due to the point load is given in Equation 30.

$$\Delta_P = \frac{Pl^3}{48EI}$$ **Equation 30**

$$\Delta_{girder\_1,p} = \frac{(2.634\ k)(12\ ft)^3\left(\frac{12\ in}{1\ ft}\right)^3}{48(29000\ ksi)(4816\ in^4)}$$

$$= 0.00117\ in$$

The total deflection of the girder is given in Equation 31.

$$\Delta_{girder\_1} = \Delta_{girder\_1,w} + \Delta_{girder\_1,p}$$ **Equation 31**

$$= 0.00036\ in + 0.00117\ in$$

$$= 0.00153\ in$$

Due to the fact that the girder is supported by the cantilever girders, the end reactions must be found to be able to apply the force to the girders. The end reaction is shown in Equation 32.

$$\sum F_y = -\left(0.1083\frac{k}{ft}\right)(12\ ft) - 2.634\ k + 2R = 0$$ **Equation 32**

$$R = 1.967\ k$$

Girder Two Moment of Inertia and Deflection

The third member analyzed is the girder highlighted in Figure 12. The effective slab width is shown below and in Figure 13.

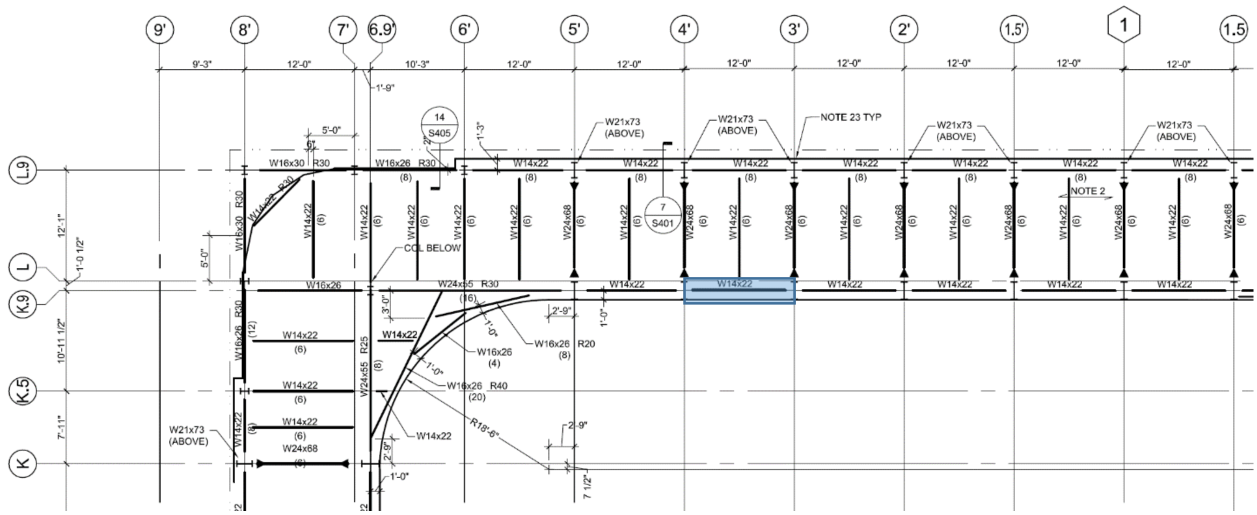

Figure 15: Girder Two

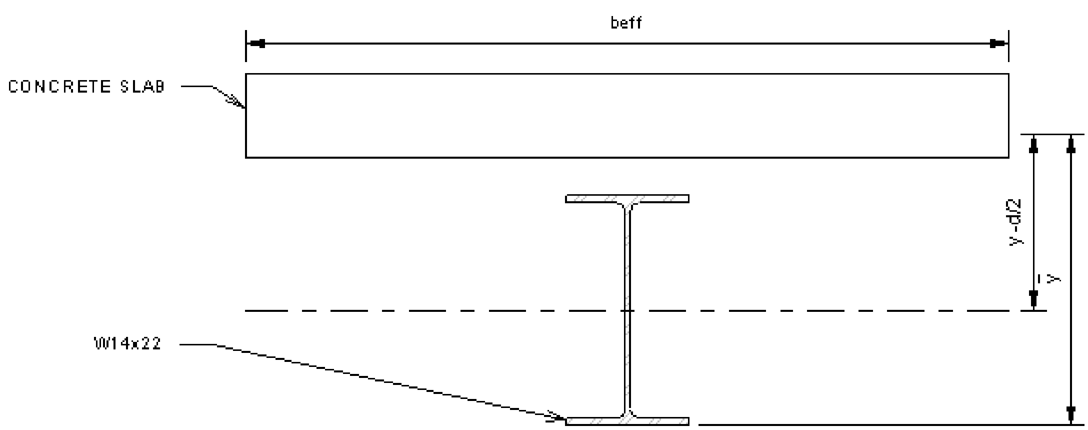

Figure 16: Girder Two and Slab Diagram

$$b_{eff} = min\left(\frac{1}{8}(12\ ft), \frac{1}{2}(12.08\ ft), 1.042\ ft\right) + min\left(\frac{1}{8}(12\ ft), \frac{1}{2}(12.08\ ft), N/A\right)$$

$$= 2.542\ ft$$

The transformed area of the concrete is shown below.

$$A_c = \frac{(2.542\ ft)\left(\frac{12\ in}{1\ ft}\right)}{6.152}(4.50\ in)$$

$$= 22.31\ in^2$$

The distance to the neutral axis is calculated below.

$$\bar{y} = \frac{\left(22.31\ in^2\left(13.7\ in + 2\ in + \frac{4.5\ in}{2}\right)\right) + \left(6.49\ in^2\left(\frac{13.7\ in}{2}\right)\right)}{22.31\ in^2 + 6.49\ in^2}$$

$$= 15.45\ in$$

The transformed moment of inertia for the steel girder and concrete slab is shown below.

$$I_{trans} = (37.65\ in^4 + (22.31\ in^2)(12.85\ in)^2)$$

$$+ (199\ in^4 + (6.49\ in^2)(8.600\ in)^2)$$

$$= 4401\ in^4$$

The girder is a simply supported member with a point load from the beam at the midspan and a distributed load as seen in Figure 14. The distributed load along the girder consists of the self-weight of the girder, the weight of the metal deck and the weight of concrete that cantilevers from the girder.

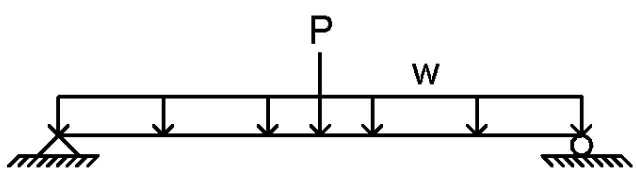

Figure 17: Simply Supported Girder with Distributed Load and Point Load

$$w = (69.07\ psf)(1.0\ ft) + 22\frac{lb}{ft} = 0.0911\ klf$$

The deflection of the gider due to the distributed load is found by using Equation 28 as shown below.

$$\Delta_{girder\_2,w} = \frac{5\left(0.0911\frac{k}{ft}\right)(12\ ft)^4\left(\frac{12\ in}{1\ ft}\right)^3}{384(29000\ ksi)(4401\ in^4)}$$

$$= 0.00033\ in$$

The deflection of the beam due to the point load is given in Equation 30.

$$\Delta_{girder\_2,p} = \frac{(2.634\ k)(12\ ft)^3\left(\frac{12\ in}{1\ ft}\right)^3}{48(29000\ ksi)(4401\ in^4)}$$

$$= 0.00128 in$$

The total deflection of the girder is given in Equation 33.

$$\Delta_{girder\_2} = \Delta_{girder\_2,w} + \Delta_{girder\_2,p}$$ **Equation 33**

$$= 0.00033\ in + 0.00128\ in$$

$$= 0.00161\ in$$

Due to the fact that the girder is supported by the cantilever girders, the end reactions must be found to be able to apply the force to the girders. The end reaction is shown in Equation 34.

$$\sum F_y = -\left(0.0911 \frac{k}{ft}\right)(12\ ft) - 2.634\ k + 2R = 0$$ **Equation 34**

$$R = 1.864\ k$$

Cantilever Girder Moment of Inertia and Deflection

The third member analyzed is the cantilever girder highlighted in Figure 18. The girders are cantilevered thus inducing a negative moment on the girder and slab causing the slab to be in tension; therefore, the composite moment of inertia cannot be used. Only the moment of inertia for the steel will be used to determine the deflection of the cantilever girder.

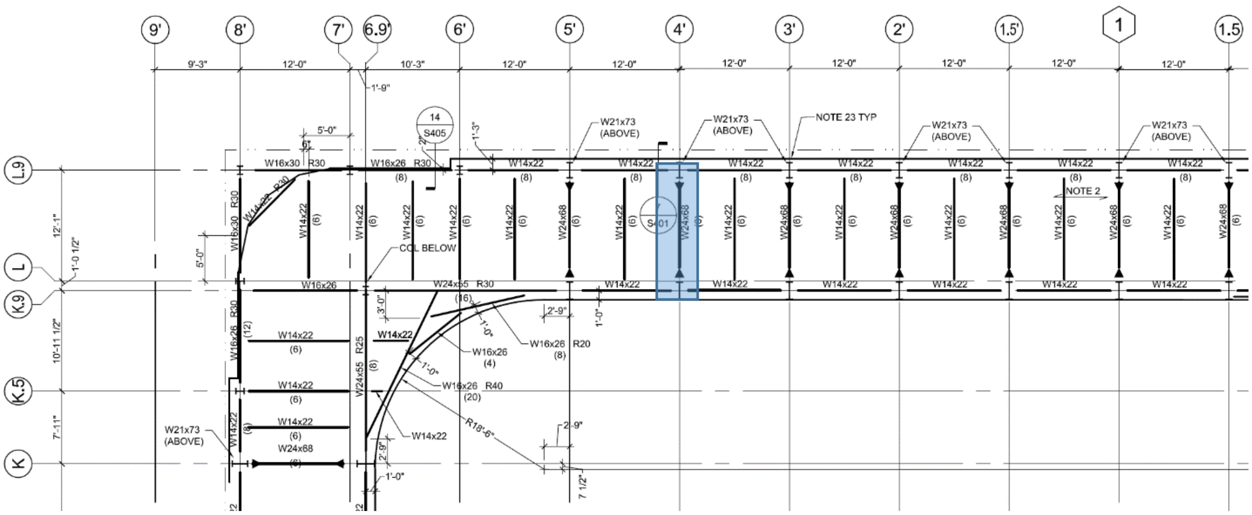

Figure 18: Girders

The girder, as stated above, is cantilevered from the columns along grid line K.9 and supports the columns along grid line L.9 that support the roof. The girder has a distributed load from the dead load, slab weight, self-weight and two point loads at the free end, one from the girder reactions and the second from the column reaction as shown in Figure 19 and below. The point load from the column reaction is assumed to be the entire load from the tributary area of that column to be conservative. The column is moment connected to the cantilever girder on the running track as well as the cantilever girder on the roof; therefore, frame action would cause some of the column load to transferred up into the cantilever girder in the roof system. Assuming the entire load is transferred onto the cantilever girder of the running track would cause more deflection on the running track therefore lowering the frequency of the track. The lower frequency would cause the floor to vibrate more; therefore, the assumption is conservative.

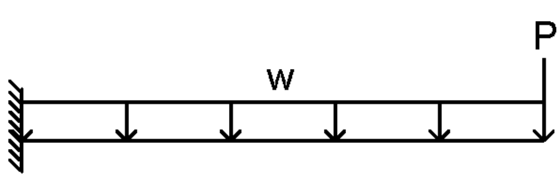

Figure 19: Cantilever Girder with a Distributed Load and Point Load

$$w = (69.07\, psf + 10\, psf)(6\, ft) + 68\frac{lb}{ft} = 0.5424\, klf$$

For the roofing material and the roof deck, a 10 psf load was assumed. The load from the roof and thus the column reaction is shown below.

$$P_{roof} = (6\, ft)\left(26\frac{lb}{ft} + 5\frac{lb}{ft} + 19\frac{lb}{ft}\right) + (6.0417\, ft)\left(68\,\frac{lb}{ft}\right) + (10\, psf)(72\, ft^2)$$

$$= 1.431\, k$$

The load from the girder reactions is shown below.

$$P_{girder} = 2(1.967\, k) = 3.934\, k$$

The total point load on the girder is shown in Equation 35.

$$P = P_{roof} + P_{girder} \qquad \textbf{Equation 35}$$

$$= 1.431\, k + 3.934\, k$$

$$= 5.365\, k$$

The deflection of a cantilever with a distributed load is given in Equation 36.

$$\Delta_{cantilever\_girder,w} = \frac{wl^4}{24EI} \qquad \textbf{Equation 36}$$

$$= \frac{\left(0.5424\,\frac{k}{ft}\right)(12.083\, ft)^4\left(\frac{12\, in}{1\, ft}\right)^3}{24(29000\, ksi)(1830\, in^4)}$$

$$= 0.01567\, in$$

The deflection of a cantilever with a point load is given in Equation 37.

$$\Delta_{cantilever\_girder,P} = \frac{Pl^3}{3EI} \qquad \textbf{Equation 37}$$

$$= \frac{(5.365\, k)(12.083)^3\left(\frac{12\, in}{1\, ft}\right)^3}{3(29000\, ksi)(1830\, in^4)}$$

$$= 0.1027\ in$$

The total deflection of the girder is shown below using Equation 38.

$$\Delta_{cantilever\_girder} = \Delta_{cantilever\_girder,w} + \Delta_{cantilever\_girder,P} \qquad \textbf{Equation 38}$$

$$= 0.01567\ in + 0.1027\ in = 0.1184\ in$$

Floor Frequency

The frequency of the floor is determined by Equation 10 from Chapter 3. This equation determines the floor's fundamental natural frequency by the total deflection of the bay. In Equation 10, the deflection of the joists or beam is added to the deflection of the girder because the bay is assumed to be simply supported with uniform framing. In the case of the Chester E. Peters recreation complex, the framing is simply supported and cantilevered. The total deflection of the bay is, therefore, the average deflection of the two girders plus the deflection of the beam and the cantilever girder. The deflection of the beams perpendicular to the girders includes the deflection of the girders. The total deflection of the bay is given in Equation 39.

$$\Delta_{bay} = \frac{(\Delta_{girder\_1} + \Delta_{girder\_2})}{2} + \Delta_{beam} + \Delta_{cantilever\_girder} \qquad \textbf{Equation 39}$$

$$= \frac{(0.00153 + 0.00161)}{2} + 0.0014 + 0.1184$$

$$= 0.1214\ in$$

Using Equation 10, the fundamental frequency of the floor is shown below.

$$f_n = 0.18\sqrt{\frac{386\,\frac{in^2}{s}}{0.1214\ in}}$$

$$= 10.15\ Hz$$

Floor Weight

Design Guide 11 gives equations, Equation 11-Equation 17 in this report, to determine the effective panel weight. These equations are for a simply supported bay that has uniform framing. Due to the nature of the floor system used in the Chester E. Peters Recreation Complex, the actual floor weight is used to determine the acceleration of the floor system as seen in Equation 40.

$$W = (deck + concrete\ weight + dead\ load)(Area) + (self\ weight\ of\ steel) \qquad \textbf{Equation 40}$$

$$= (69.07\,psf + 10\,psf)(12ft)(12.08\,ft + 1.25\,ft + 1\,ft) + \left(22\frac{lb}{ft}\right)(12\,ft) + \left(22\frac{lb}{ft}\right)(12.08) + \left(68\frac{lb}{ft}\right)(12.08\,ft)$$

$$= 14951\,lb$$

Floor Acceleration and Acceptability

The acceleration of the floor due to an occupant running across the floor is determined by Equation 19. The damping ratio is assumed to be 0.02% to account for the structural system, the ceiling, and the ductwork as shown in Table 1. The running track does have a synthetic running surface material; however, the actual damping effect is not known and therefore is not included in the damping calculations to be conservative. The occupant is assumed to be an average recreational runner; therefore, 168 lb is used.

$$\frac{a_p}{g} = \frac{0.79(168\,lb)\left(e^{-0.173(10.15\,Hz)}\right)}{(0.02)(14951\,lb)}$$

$= 0.0767 = 7.67\%g$

The acceleration limit is given by Figure 4, which is repeated in Figure 20 to show the acceleration limit of the floor with a 10.15 Hz frequency. Although the running track used in this case study is indoors, the outdoor pedestrian bridge acceleration limits are used. An occupant running on the track is moving quickly, compared to an occupant walking, and would normally have less objections to the floor vibrating. Also, an occupant in a recreation center may expect floor vibrations from an indoor running track. These reasons make the running track more similar to an outdoor pedestrian bridge over the indoor pedestrian bridge where an occupant is more likely to find the vibrations objectionable. Based on Figure 20, the acceleration limit is approximately 7.0%g, which is less than the actual floor acceleration of 7.67%g. The floor system, is therefore slightly unacceptable based on the current evaluation techniques in Design Guide 11.

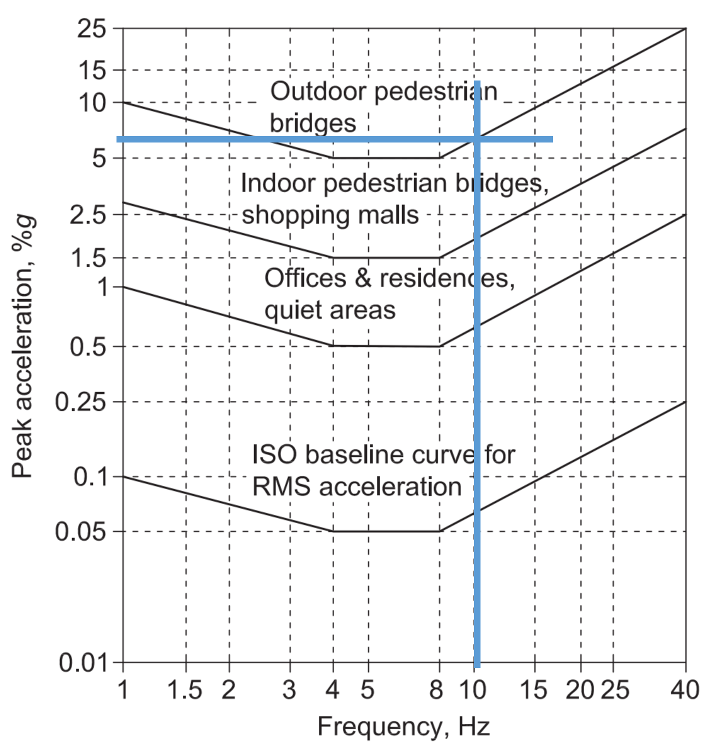

Figure 20: Design Guide 11 Recommended Tolerance Limit

Chapter 5 - Conclusion and Areas of Further Research

This report discussed the history and the evolution of vibration analysis and design criteria for steel framed floors due to occupant induced vibration. Walking as the source of vibration was covered in the history almost exclusively due to the fact that there is a lacking in the research and experimentation for running as the source of vibration. The current design criteria and analysis procedure for running from AISC Design Guide 11 was discussed and applied to the Kansas State Chester E. Peters Recreation Center.

This section of the report will summarize the analysis procedure of a steel framed floor system under running excitations based on AISC Design Guide 11. In addition, the evaluation of the Chester E. Peters Recreation Center's running track is briefly discussed. Lastly, areas of further research on this topic are suggested.

AISC Design Guide 11

Design Guide 11 outlines how to estimate the important parameters in determining the acceptability of a floor system in terms of structural vibration when the floor is subjected to occupant induced vibration. In addition, the design guide provides some remedial procedures to take if a constructed building does have objectionable vibrations.

Analysis Procedure and Acceleration Limits

AISC Design Guide 11 has the most widely adopted procedure for analyzing steel framed floors for human induced vibrations. The design guide provides evaluation procedures and acceleration limits for vibrations due to walking, running, rhythmic activities, and design for sensitive occupancies and sensitive equipment. The equations developed in Design Guide 11 apply to steel framed floors with a concrete slab or metal deck. In this report, the equations pertaining to occupant induced vibrations due to running were discussed in detail.

In order to estimate the floor's acceleration, the floor frequency, effective panel weight, and damping ratio must be estimated. The most important floor parameter is the floor's fundamental natural frequency. Design Guide 11 outlines how to calculate the frequency based on the total deflection of the floor system. The effective panel weight of the floor is determined by looking at the joist and the girder modes separately and then combining the two modes. The Design Guide 11 equations calculate the effective joist or girder panel weight based on the panel effective width, the member span, and the weight supported by the member. Design Guide 11 also provides estimates of the floor system's damping ratio depending on the occupancy and the type of architectural finishes used. Lastly an inequality is used to estimate the floor's acceleration and to compare it to the acceleration limit given for the occupancy type.

Remedial Procedures

In addition to evaluation techniques, Design Guide 11 also provides suggestions on how to fix floors that have objectionable vibrations (Murray, Allen, Ungar, & Davis, 2016). The suggestions pertain to all floor occupancies, not just floors that are subject to running, and the remedial procedures applicable to floors with running are summarized here. The first suggestion is to do nothing about the structural vibration itself, but to remove things that point to the vibration, such as architectural finishes that make noise when the floor vibrates. The second suggestion is to move the person who is bothered by the floor vibration. As discussed earlier in this report, vibrations are subjective and the effects can vary person by person. Complaints of vibrations are normally less when the person is near a column or support because the magnitudes of vibrations are the largest at midbay. Another suggestion is to stiffen the floor system, which in turn increases the floor's natural frequency. The structural components with the lowest natural frequency should be stiffened first. The last suggestion is to increase the damping of the floor system by adding architectural finishes. The lower the initial damping in a floor system the more the addition of damping will help the vibration problem.

The use of remedial procedures can be completely eliminated during the design phase of a building. Engineers can ensure that the floor acceleration is within the recommended tolerance limits. In addition, the location of the running in a building is an important design decision to ensure the occupancies next to the running track are not annoyed. For tracks within recreation centers, the areas around a running track are usually other workout areas; therefore, the occupants are not as easily disturbed because they are moving as well. The vibration can become a nuisance if the running track is located near office areas or more quiet activity rooms, or if the running track is included in a building that is not solely for recreational activities.

Chester E. Peters Recreation Center

The Chester E. Peters Recreation Center 1/5-mile track served as a case study in this report to apply the provisions in Design Guide 11. The running track consisted of columns that supported cantilever girders that in turn supported the girders and beam for the running track and columns for the roof system. A section of the straight portion of the track was analyzed to estimate the acceleration of the floor due to occupants running across the floor.

The floor's fundamental frequency was determined by calculating the maximum deflection of a bay using the equations for a cantilever span and a simply supported span. Due to the irregular layout of the bay, the equations for effective panel weight form Design Guide 11 were not used; the actual weight of the floor was used. The damping ratio was assumed to encompass the structural system, ceiling, and ductwork but not the running track surface material in order to be conservative. Assuming recreational runners would be using the track, the average body weight of 168 lb was used in conjunction with the floor frequency, weight, and damping ratio to estimate the acceleration of the floor. The acceleration limit for the floor was determined using Design Guide 11's recommended tolerance levels for human comfort based on the floor frequency and the assumption that occupants on the running track were similar to those on an outdoor pedestrian bridge. The running track was deemed slightly unacceptable because the floor acceleration was slightly higher than the acceleration limit in Design Guide 11.

Areas of Further Research

During the research phase for this report, it was determined that there is a lacking of research in the field of occupant induced vibrations due to running in steel framed floors. Design Guide 11 also confirms this:

> "There has been much less research on the response of floors to running than to walking, and there are no recommended acceleration limits in the literature for floors subjected to running. However, it seems reasonable to recommend the limits shown in Figure 2-1 using engineering judgment for selecting the appropriate category depending on the location of the running and the affected occupancy." (Murray, Allen, Ungar, & Davis, 2016)

The first suggestion for further research would be to determine if the tolerance limits in Design Guide 11 are adequate for running induced vibrations. In the case study of this report, it was assumed that the running track would behave as an outdoor pedestrian bridge, thus the acceleration limit is higher. Further research should be conducted to ensure that this assumption is appropriate and that the occupants find this limit acceptable.

The second suggestion for further research is for the vibration of cantilever floor systems. The equations in Design Guide 11 are for floor systems where the joists are supported by girders which are in turn supported by walls or columns. In the case study of this report, the floor system included cantilever girders. More research should be conducted to ensure that the equations given in Design Guide 11 hold for cantilever floor systems, especially for running tracks. Most recreation centers utilize the space above basketball/tennis/volleyball courts for the running tracks, which in most cases prohibits the use of a completely simply supported system. The floor system needs to be cantilevered from the columns or walls in order to not take up space on the court below.

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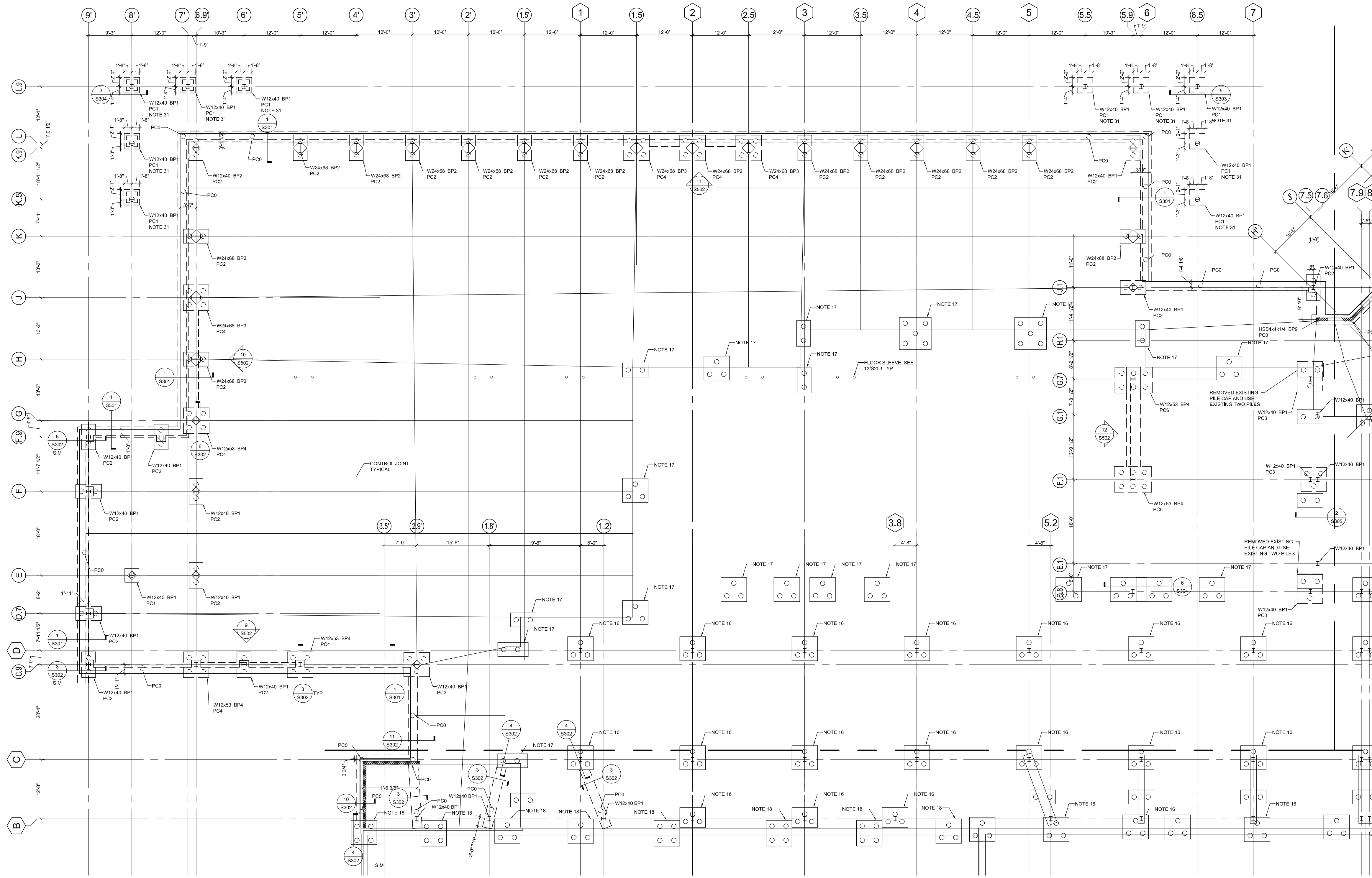
Appendix A - Chester E. Peters Recreation Center Structural Drawings



FLOOR PLAN NOTES

1. REFER TO GENERAL NOTES AND SPECIFICATIONS FOR ADDITIONAL INFORMATION.
2. SLAB ON GRADE IS 5" THICK REINFORCED WITH WELDED WIRE REINFORCING 6x6-W2.9xW2.9. OVER 15 MILS THICK VAPOR BARRIER (REFER TO SPECIFICATIONS) OVER 4" FREE DRAINING CRUSHED ROCK OVER 18" LOW VOLUME CHANGE MATERIAL (REFER TO SPECIFICATIONS). REFER TO GENERAL NOTES FOR MINIMUM CONCRETE STRENGTH REQUIREMENTS AND MAXIMUM WATER/CEMENT RATIO. TOP OF CONCRETE ELEVATION = 100'-0" TO MATCH EXISTING ELEVATION UNLESS NOTED OTHERWISE ON PLAN.
3. CIVIL DRAWINGS ELEVATION 1091.91' = DATUM ELEVATION 100'-0".
4. REFER TO SHEETS S201-S203 FOR FOUNDATION AND SLAB ON GRADE TYPICAL DETAILS.
5. REFER TO SHEET S203 FOR FOUNDATION SCHEDULES.
6. REFER TO SHEETS S501 AND S502 FOR BRACED FRAME ELEVATIONS AND DETAILS.
7. ALL FOUNDATIONS SHALL BE CENTERED UNDER THE WALL OR COLUMN THEY SUPPORT, UNLESS NOTED OTHERWISE.
8. N O T U S E D .
9. PROVIDE 2-#4x4'-0" ADDITIONAL SLAB REINFORCEMENT AT ALL RE-ENTRANT CORNERS.
10. PROVIDE THICKENED SLAB WITH ADDITIONAL REINFORCEMENT PER TYPICAL DETAIL 13/S202. REFER TO ARCHITECTURAL DRAWINGS FOR LOCATIONS OF STEEL STAIRS.
11. REFER TO ARCHITECTURAL DRAWINGS FOR EXTENTS AND DIMENSIONS OF RAISED OR DEPRESSED SLAB AREAS, SLOPES, AND DRAINS. REFER TO TYPICAL DETAILS FOR REINFORCEMENT REQUIREMENTS.
12. BUILDING EARTHWORK AND SITE PREPARATION SHALL BE IN ACCORDANCE WITH SPECIFICATIONS AND THE RECOMMENDATIONS OUTLINED IN THE GEOTECHNICAL REPORT.
13. REFER TO MECHANICAL, ELECTRICAL, AND PLUMBING DRAWINGS FOR SLAB PENETRATIONS AND UNDERGROUND UTILITIES.
14. CONTRACTOR TO FIELD VERIFY ALL EXISTING CONDITIONS AND DIMENSIONS AND TO REPORT ANY DISCREPANCY TO THE ARCHITECT AND ENGINEER PRIOR TO THE SUBMISSION OF SHOP DRAWINGS.
15. CONTRACTOR TO VERIFY ALL SLAB EDGE, STAIR, AND ELEVATOR OPENING DIMENSIONS WITH ARCHITECTURAL DRAWINGS PRIOR TO CONSTRUCTION.
16. EXISTING STEEL COLUMN AND PILE CAP TO REMAIN.
17. EXISTING PILE CAP AND PILES MAY REMAIN IF THE DISTANCE BETWEEN THE TOP OF PILE CAP AND TOP OF SLAB ON GRADE IS GREATER THAN OR EQUAL TO 1'-0". REMOVE CONCRETE AND REINFORCING STEEL AS REQUIRED TO ACHIEVE 1'-0" MINIMUM BETWEEN TOP OF PILE CAP AND TOP OF CONCRETE SLAB ON GRADE.
18. VERIFY THE LOCATION OF THE EXISTING PILE CAPS PRIOR TO CONSTRUCTION. EXISTING DRAWINGS DO NOT PROVIDE CONCLUSIVE INFORMATION REGARDING PILE CAP LOCATIONS IN THIS AREA.
19. TYPICAL FLOOR SYSTEM CONSISTS OF 4-1/2" NORMAL WEIGHT CONCRETE ON 2" DEEP, 18 GAGE GALVANIZED (G90) COMPOSITE STEEL DECK (6-1/2" TOTAL THICKNESS). REFER TO GENERAL NOTES FOR MINIMUM CONCRETE STRENGTH REQUIREMENTS. TOP OF CONCRETE ELEVATION = 100'-0" TO MATCH EXISTING UNLESS NOTED OTHERWISE ON PLAN.
20. REFER TO GENERAL NOTES FOR COMPOSITE STEEL BEAM NOTATION.
21. MINIMUM FACTORED REACTION SHALL BE 12 KIPS.
22. REFER TO SHEETS S204 AND S205 FOR TYPICAL FLOOR FRAMING DETAILS.
23. COMPOSITE SLAB SHALL BE REINFORCED WITH WELDED WIRE REINFORCING 6x6-W2.9xW2.9, UNLESS NOTED OTHERWISE. REFER TO DETAIL 14/S204 FOR WELDED WIRE REINFORCING SUPPORT BARS.
24. STEEL DECK SHALL BE PLACED WITH A TWO-SPAN CONDITION MINIMUM. NO SINGLE SPANS ARE ALLOWED.
25. PLACE #4x8'-0"@12" ACROSS STEEL GIRDERS BELOW THE WELDED WIRE REINFORCEMENT PER DETAIL 13/S204.
26. PROVIDE A CONTINUOUS 1/4" BENT PLATE WITH 1/2" DIAMETER x 8" LONG HEADED STUDS SPACED @ 2'-0" AT ALL SLAB EDGES AND OPENINGS, UNLESS NOTED OTHERWISE.
27. ALL STEEL THAT IS PERMANENTLY EXPOSED TO THE EXTERIOR SHALL BE HOT DIPPED GALVANIZED.
28. EPX DESIGNATES EMBEDDED PLATE TYPE, REFER TO 10/S205.
29. PORTION OF CONCRETE FLOOR SYSTEM TO BE REMOVED TO ACCOMMODATE NEW STAIR. CONFIRM EXTENT OF OPENING WITH ARCHITECT. REFER TO GENERAL NOTES FOR DEMOLITION NOTES.
30. EXISTING CAST-IN-PLACE REINFORCED CONCRETE FLOOR SYSTEM. 4 1/2" THICK SLABS SPANNING NORTH AND SOUTH BETWEEN CONCRETE BEAMS. OVERALL STRUCTURE DEPTH = 21" TYPICAL. BEAMS WIDTHS VARY. REFER TO EXISTING BUILDING DOCUMENTS FOR MORE INFORMATION. TOP OF EXISTING FLOOR FRAMING = 100'-0"
31. TOP OF PILE CAP ELEVATION = 98'-6"
32. THE CONTRACTOR SHALL CONFIRM THE DEPTH OF THE THICKNESS OF THE FLOORING SYSTEM WITH THE APPROVED MANUFACTURER BEFORE CONSTRUCTION AND

1 FIRST FLOOR FRAMING PLAN AREA E
1/8" = 1'-0"

N

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Revised Date

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Building Number 36700-00159

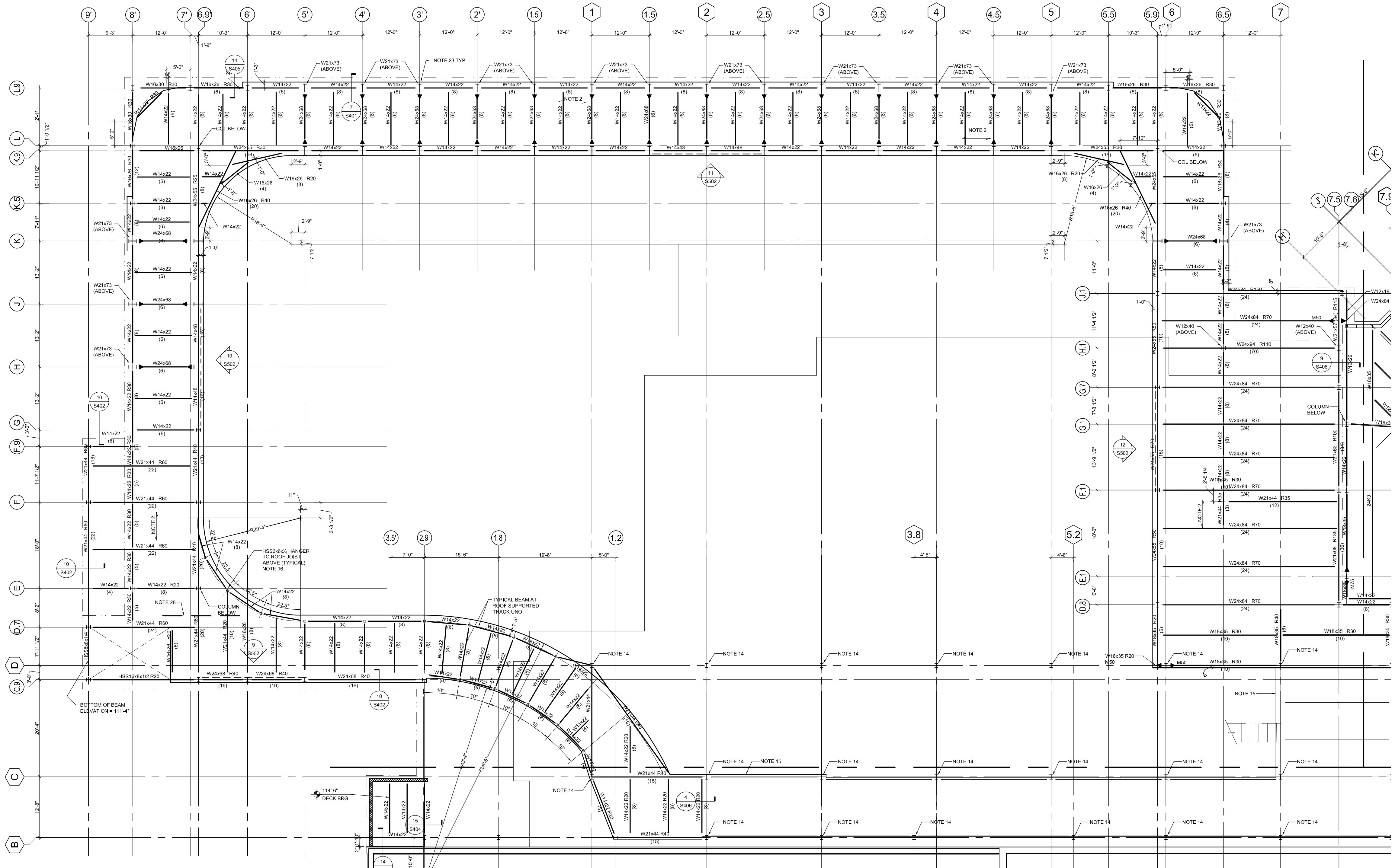
DATE: DRAWN BY: CHECKED BY: REV.:

SHEET CONTENTS
FIRST FLOOR FRAMING PLAN
AREA E

DFM #: A-011021

S101E

ORIGINAL CONTRACT DOCUMENTS



FLOOR PLAN NOTES

1. REFER TO GENERAL NOTES AND SPECIFICATIONS FOR ADDITIONAL INFORMATION.
2. TYPICAL FLOOR SYSTEM CONSISTS OF 4-1/2" NORMAL WEIGHT CONCRETE ON 2" DEEP, 18 GAGE GALVANIZED (G90) COMPOSITE STEEL DECK (6-1/2" TOTAL THICKNESS). REFER TO GENERAL NOTES FOR MINIMUM CONCRETE STRENGTH REQUIREMENTS. TOP OF CONCRETE ELEVATION = 112'-0" TO MATCH EXISTING UNLESS NOTED OTHERWISE ON PLAN.
3. REFER TO GENERAL NOTES FOR COMPOSITE STEEL BEAM NOTATION.
4. MINIMUM FACTORED REACTION SHALL BE 12 KIPS.
5. REFER TO SHEETS S204 AND S205 FOR TYPICAL FLOOR FRAMING DETAILS.
6. REFER TO SHEETS S501 AND S502 FOR BRACED FRAME ELEVATIONS AND DETAILS.
7. COMPOSITE SLAB SHALL BE REINFORCED WITH WELDED WIRE REINFORCING 6x6-W2.9xW2.9, UNLESS NOTED OTHERWISE. REFER TO DETAIL 14/S204 FOR WELDED WIRE REINFORCING SUPPORT BARS.
8. STEEL DECK SHALL BE PLACED WITH A TWO-SPAN CONDITION MINIMUM. NO SINGLE SPANS ARE ALLOWED.
9. PLACE #4x8'-0"@12" ACROSS STEEL GIRDERS BELOW THE WELDED WIRE REINFORCEMENT PER DETAIL 13/S204.
10. PROVIDE A CONTINUOUS 1/4" BENT PLATE WITH 1/2" DIAMETER x 8" LONG HEADED STUDS SPACED @ 2'-0" AT ALL SLAB EDGES AND OPENINGS, UNLESS NOTED OTHERWISE.
11. ALL STEEL THAT IS PERMANENTLY EXPOSED TO THE EXTERIOR SHALL BE HOT DIPPED GALVANIZED.
12. REFER TO MECHANICAL, ELECTRICAL, AND PLUMBING DRAWINGS FOR PENETRATIONS NOT SHOWN. REFER TO TYPICAL DETAILS FOR ADDITIONAL REINFORCEMENT REQUIREMENTS AT OPENINGS.
13. CONTRACTOR TO VERIFY ALL SLAB EDGE, STAIR, AND ELEVATOR OPENING DIMENSIONS WITH ARCHITECTURAL DRAWINGS PRIOR TO CONSTRUCTION.
14. EXISTING COLUMN TO REMAIN.
15. EXISTING FLOOR SLAB TO REMAIN.
16. HSS HANGER COLUMNS MUST BE SHORED BY THE CONTRACTOR UNTIL THE CONNECTION TO THE JOISTS ABOVE ARE COMPLETED. REFER TO NOTE 19 ON S121E.
17. REFER TO 13/S206 FOR CONNECTION REQUIREMENTS.
18. EXISTING CAST IN PLACE CONCRETE STAIR FROM LEVEL 2 T0 3 TO REMAIN.
19. VERIFY 12" CMU FROM LEVEL 1 TO ROOF ADJACENT TO STAIR. REINFORCED CELLS AT 30" ON CENTER MINIMUM.
20. EXISTING CAST-IN-PLACE REINFORCED CONCRETE FLOOR SYSTEM. 4 1/2" THICK SLABS SPANNING NORTH AND SOUTH BETWEEN CONCRETE BEAMS. OVERALL STRUCTURE DEPTH = 21" TYPICAL. BEAMS WIDTHS VARY. REFER TO EXISTING BUILDING DOCUMENTS FOR MORE INFORMATION. TOP OF EXISTING FLOOR FRAMING = 112'-0"
21. REMOVE EXISTING CONCRETE STAIR. REFER TO 10/S411 AND 13/S410 SIMILAR.
22. RAIL PER ARCHITECT DESIGNED PER SPECIFICATIONS.
23. CANTILEVERED COLUMN FRAME MUST BE SHORED BY THE CONTRACTOR UNTIL THE ROOF DECK IS COMPLETELY INSTALLED AND ALL BRACED FRAMES ARE COMPLETELY INSTALLED, SEE 7/S401.
24. BEAM IS DESIGNED FOR A FACTORED 1.0K LIVE LOAD AT MIDSPAN WITH A MAX 12" ECCENTRICITY. TV SUPPORT AND CONNECTION BY SUPPLIER. DESIGN END CONNECTION TO RESTRAIN BEAM IN TORSION. TOP OF STEEL ELEVATION = 111'-5 1/2
25. PROVIDE 3-#5 REINFORCEMENT BARS MID-DEPTH OF SLAB, PROVIDE CORNER BARS AT BENDS.
26. PROVIDE #4x16'-0"@12"OC

1 SECOND FLOOR FRAMING PLAN AREA E
1/8" = 1'-0"

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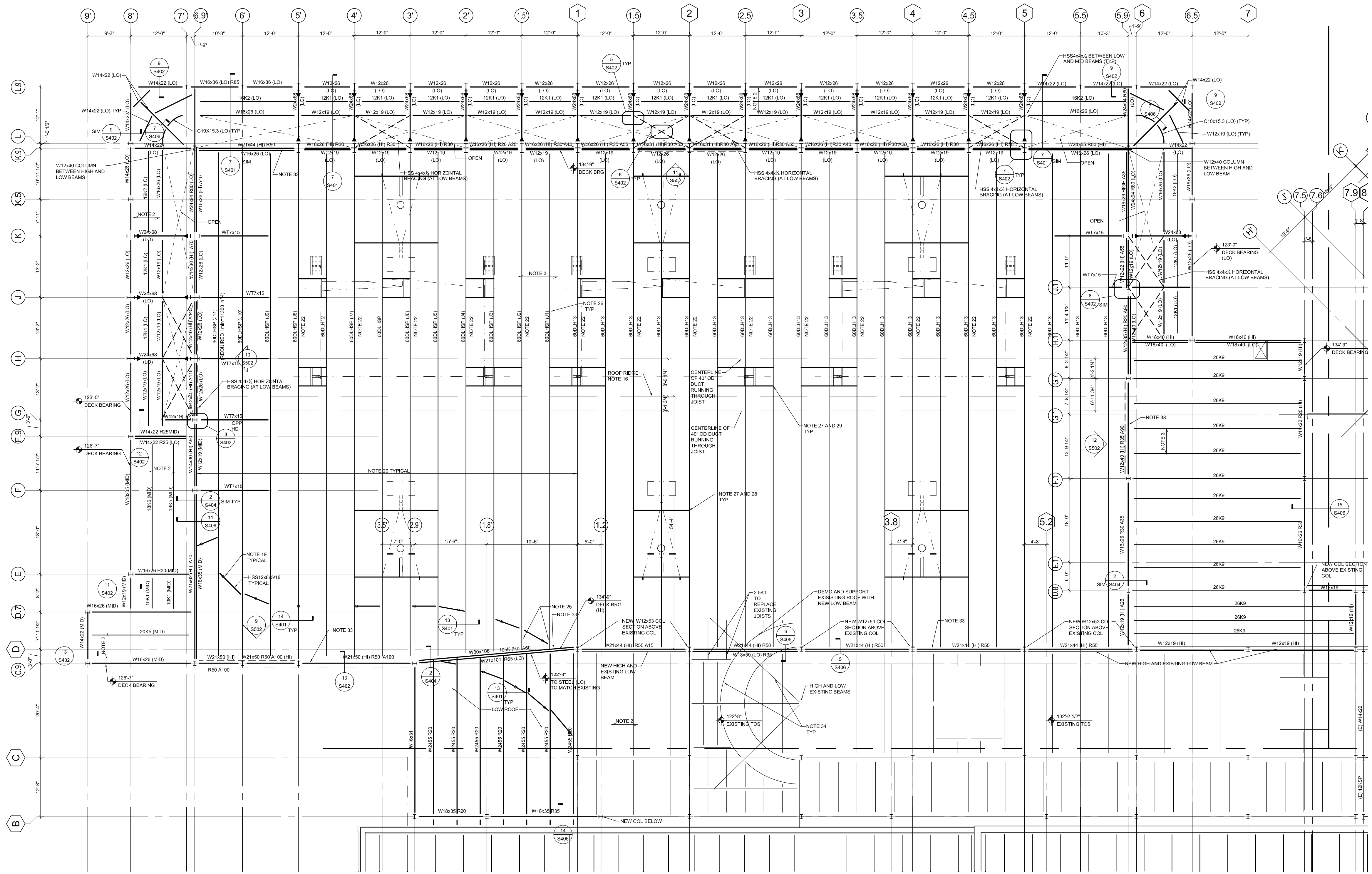
SECOND FLOOR FRAMING
PLAN - AREA E

DFM #: A-011021

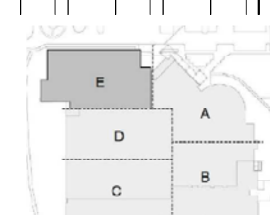
S102E

ORIGINAL CONTRACT DOCUMENTS



ROOF FRAMING NOTES

1. REFER TO GENERAL NOTES AND SPECIFICATIONS FOR ADDITIONAL INFORMATION.
2. ROOF DECK SHALL BE 1-1/2" DEEP, TYPE B, 20 GAGE GALVANIZED (G60) STEEL ROOF DECK. PROVIDE 36/5 FASTENER PATTERN WITH 3/4" DIAMETER PUDDLE WELDS AND (6) WELDED SIDELAP FASTENERS. REFER TO ARCHITECTURAL DRAWINGS FOR PAINT LOCATIONS AND/OR FIREPROOFING LOCATIONS. COORDINATE FINISHES AS APPROPRIATE.
3. ROOF DECK SHALL BE 2" DEEP EPIC ER2RA, 18 GAGE GALVANIZED (G60) ACOUSTICAL STEEL ROOF DECK OR EQUIVALENT. PROVIDE 24/6 FASTENER PATTERN WITH 3/4" DIAMETER PUDDLE WELDS AND #12 TEK SCREW @ 12" ON CENTER SIDELAP FASTENERS. ROOF DECK SHALL BE PLACED WITH A THREE SPAN CONDITION MINIMUM. REFER TO ARCHITECTURAL DRAWINGS FOR PAINT LOCATIONS AND/OR FIREPROOFING LOCATIONS. COORDINATE FINISHES AS APPROPRIATE.
4. REFER TO PLAN FOR DECK BEARING ELEVATIONS. TOP OF STEEL ELEVATIONS FOR BEAMS AND GIRDERS SHALL BE DETERMINED FROM JOIST SEAT DEPTHS AND DECK BEARING ELEVATIONS, UNLESS NOTED OTHERWISE.
5. REFER TO GENERAL NOTES FOR STEEL BEAM NOTATION.
6. MINIMUM FACTORED REACTION SHALL BE 12 KIPS.
7. —— INDICATES LOCATIONS OF BOTTOM FLANGE BRACING. REFER TO DETAIL 10/S206 FOR BRACING.
8. REFER TO SHEETS S204 AND S206 FOR TYPICAL FRAMING DETAILS.
9. ROOF DECK SHALL BE PLACED WITH A TWO SPAN CONDITION MINIMUM. NO SINGLE SPANS ARE ALLOWED.
10. PROVIDE A CONTINUOUS 1/4" BENT PLATE AT ALL ROOF EDGES AND OPENINGS, UNLESS NOTED OTHERWISE.
11. DESIGN ALL ROOF JOISTS AND BRIDGING FOR A NET WIND UPLIFT PRESSURE OF 20 PSF AWAY FROM ROOF EDGES AND 35 PSF WITHIN 10 FEET OF ROOF EDGE. PROVIDE JOIST BRIDGING PER STEEL JOIST INSTITUTE.
12. LH-SERIES JOISTS HAVE A 5" JOIST SEAT AND K-SERIES JOISTS HAVE A 2 1/2" JOIST SEAT, UNLESS NOTED OTHERWISE.
13. ALL STEEL THAT IS PERMANENTLY EXPOSED TO THE EXTERIOR SHALL BE HOT DIPPED GALVANIZED.
14. REFER TO MECHANICAL, ELECTRICAL, AND PLUMBING DRAWINGS FOR PENETRATIONS NOT SHOWN. REFER TO TYPICAL DETAILS FOR ADDITIONAL STEEL REQUIREMENTS AT OPENINGS.
15. CONTRACTOR TO VERIFY ALL DECK EDGE, STAIR, AND ELEVATOR OPENING DIMENSIONS WITH ARCHITECTURAL DRAWINGS PRIOR TO CONSTRUCTION.
16. ROOF JOIST ARE TOP CHORD DOUBLE PITCHED UNDERSLUNG WITH A MINIMUM OF 1/4" PER FOOT SLOPE. BOTTOM OF THE STEEL ELEVATION OF THE BOTTOM CHORD IS 129'-9". CAMBER JOISTS PER SJI RECOMMENDATIONS.
17. REFER TO 13/S206 FOR CONNECTION REQUIREMENTS.
18. PROVIDE A 5" JOIST SEAT
19. HSS HANGER CONNECTION TO JOISTS PER 13/S401 SHALL NOT BE COMPLETED UNTIL AFTER THE ROOF IS COMPLETE AND ALL ROOF DEAD LOADS HAVE BEEN APPLIED. THE COMPOSITE FLOOR FRAMING SUPPORTED BY THE HSS HANGER COLUMNS MUST BE SHORED BY THE CONTRACTOR DURING CONSTRUCTION.
20. JOISTS MUST HAVE A MINIMUM MOMENT OF INERTIA OF 6000 IN^4 OVER ENTIRE LENGTH OF JOIST .
21. DO NOT PRIME OR PAINT STRUCTURAL STEEL THAT IS TO RECEIVE SPRAYED ON FIREPROOFING.
22. DESIGN JOISTS FOR A FACTORED AXIAL TENSION LOAD OF 20 KIPS, SEE 6/S405 FOR CONNECTION.
23. DESIGN JOIST FOR A FACTORED UPLIFT OF 3500 LBS.
24. EXPOSED CANOPY STEEL IS PAINTED WITH HIGH PERFORMANCE PAINT, SEE ARCHITECTURAL DRAWINGS AND SPECIFICATIONS.
25. EXTEND JOIST BOTTOM CHORD AS REQUIRED FOR HANGER BEAMS.
26. SEE 5/S405 FOR JOIST LOADING DIAGRAMS.
27. THE BASKETBALL BACKSTOP SUPPLIER AND THE VOLLEYBALL NET SUPPLIER SHALL PROVIDE THE DESIGN, FABRICATION, AND ERECTION OF ALL MISCELLANEOUS STRUCTURAL STEEL REQUIRED FOR THE SUPPORT OF THE HANGING EQUIPMENT. IF THE BASKETBALL BACKSTOP SUPPLIER AND/OR THE VOLLEYBALL NET SUPPLIER EXCLUDES THIS WORK FROM THEIR BID, THE GENERAL CONTRACTOR WILL BE RESPONSIBLE FOR THESE SERVICES.
28. DESIGN JOIST FOR AN UNFACTORED LIVE LOAD OF 1000 POUNDS AT EACH CONNECTION POINT OF THE BASKETBALL GOAL TO A JOIST.
29. DESIGN JOIST FOR AN UNFACTORED LIVE LOAD OF 2000 POUNDS AT EACH CONNECTION POINT OF THE VOLLEYBALL NET TO A JOIST.
30. SEE ARCHITECTURAL DRAWINGS FOR LOCATION OF FANS SUPPORTED BY THE ROOF STRUCTURE. PROVIDE SUPPLEMENTAL STEEL TO SUPPORT FANS AS RECOMMEND BY THE FAN SUPPLIER.
31. DESIGN JOIST FOR A FACTORED LIVE LOAD OF 450 PLF NET UPLIFT.
32. PROVIDE JOIST EXTENDED END TYPE R1.
33. PROVIDE HSS PER 11/S206 BETWEEN JOISTS ALONG THIS BEAM LINE.
34. CONTRACTOR IS TO SHORE EXISTING BEAMS, CUT TO NEW LENGTH, AND INSTALL CONNECTION TO NEW LOW BEAM, SEE 6/S406.



1 ROOF FRAMING PLAN AREA E
1/8" = 1'-0"

N

HTK ARCHITECTS PA
9300 W 11OTH ST. STE. 150
OVERLAND PARK, KS 66210
P 913.663.5373
2900 SW MACVICAR AVE.
TOPEKA KS 66111
P 785.266.5373
WWW.HTKARCHITECTS.NET
WALTER P MOORE
WALTER P. MOORE AND ASSOCIATES, INC.
920 MAIN STREET, TENTH FLOOR
KANSAS CITY, MISSOURI 64105-2008
PHONE: 816.701.2100 FAX: 816.701.2200

CLIFTON M. GREENLEE LICENSED 12237 KANSAS PROFESSIONAL ENGINEER

Date
JANUARY 15, 2010

Revised Date

Project Number
HTK #: 0808.13
F&S #: 2008-20

DEPARTMENT OF ADMINISTRATION
DIVISION OF FACILITIES MANAGEMENT
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PETERS STUDENT RECREATION CENTER EXPANSION
Manhattan, Ks
Building Number 36700-00159

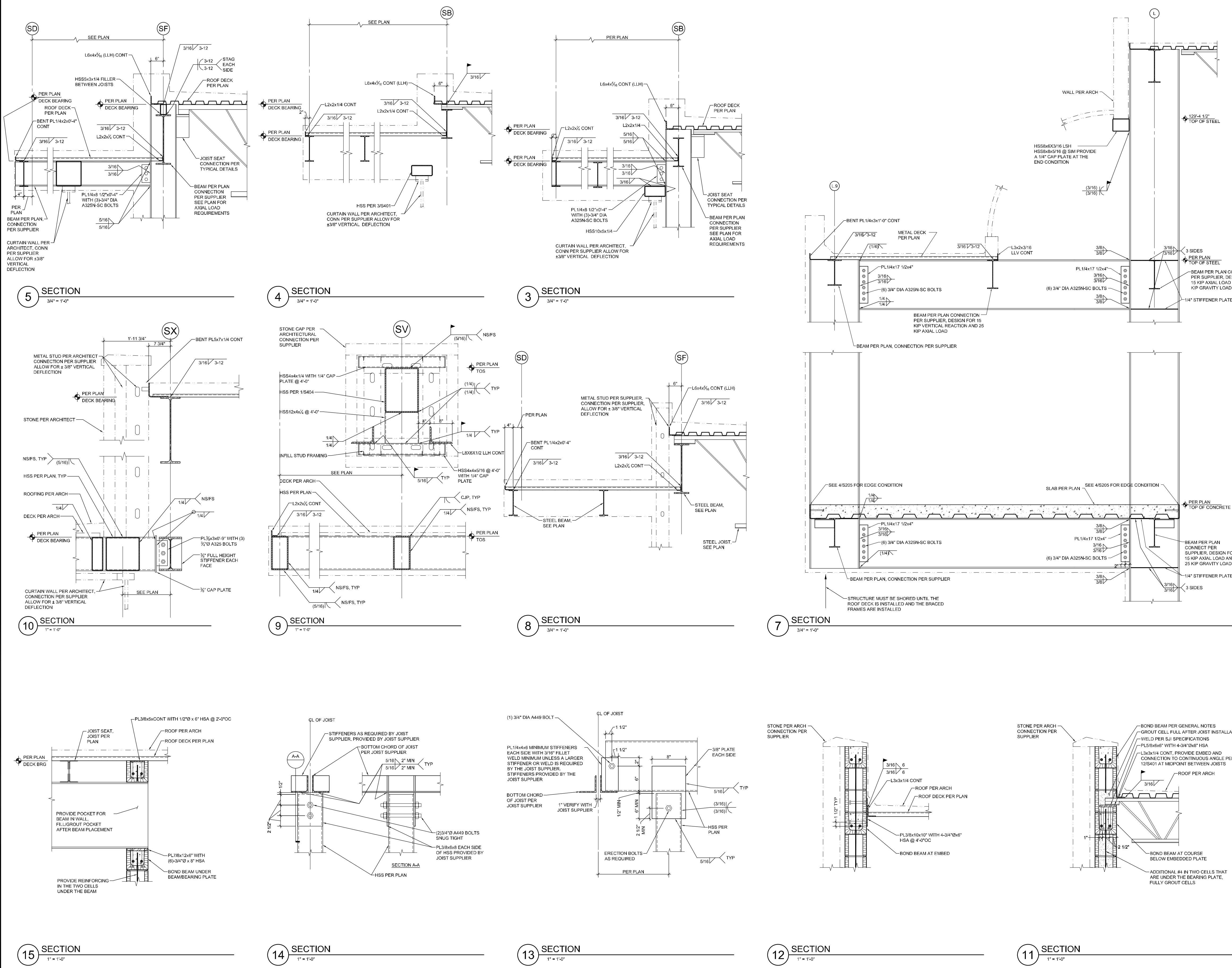
DATE: DRAWN BY: CHECKED BY: REV.:

SHEET CONTENTS
ROOF FRAMING PLAN
AREA E

DFM #: A-011021

S121E

ORIGINAL CONTRACT DOCUMENTS



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SHEET CONTENTS
■ BRACED FRAME
■ CONNECTION DETAILS

DFM #: A-011021

S401

ORIGINAL CONTRACT DOCUMENTS