OPTIMIZING THE PERFORMANCE AND COST OF FORMWORK

by

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Abstract

Although it is a temporary structure, formwork is a major system that must satisfy several design and construction requirements. This thesis examines some of the issues involving formwork at the construction site. It looks at the factors that determine the quality, performance, and safety of formwork, and the methods used to optimize them. It also looks at some of the concepts dealing with formwork economy, and includes a study that reduces the construction costs of a cast-in-place concrete design through better formwork re-use. Finally, this thesis looks at some of the recent formwork innovations that aim at improving the cost and performance of formwork.

Thesis Supervisor: Professor Jerome Connor
Department of Civil and Environmental Engineering

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INTRODUCTION

Constructability is a major factor in determining the feasibility of a construction project. With steel structures, the steel connections are the main issues of constructability. The steel connections often determine the efficiency and profit margin of the job. With cast-in-place concrete structures, the main issue of constructability is *formwork*. Many design professionals continue to dismiss formwork as a simple, temporary means of achieving the final structure. However, formwork is a major system in itself, and its proper design and use are often a critical factor in determining the project cost and construction productivity.

Formwork must have the proper qualities to satisfy several critical requirements. It must be economical, since it is a major factor in the overall project cost. It must be safe and strong enough to support all of the imposed construction and lateral loads. It must be easy to erect and remove, and needs to maintain the same quality and shape during the course of the construction. Formwork is also the most important factor in determining the finished appearance of the concrete and must be able to produce the desired surface texture.

This thesis examines some of these issues involving formwork at the construction site. It looks at the factors that determine the quality, performance, and safety of formwork, and the methods used to optimize them. It also examines some of the concepts dealing with the economics of formwork. These concepts stress the importance of formwork in determining the overall cost and productivity of the project.

To test these concepts, two different cast-in-place concrete building designs were included in this thesis. One design was part of the 1997-98 Master of Engineering HPS Project. It incorporated a minimal amount of concrete and reinforcing steel to emphasize member efficiency. The other was designed for the purpose of this thesis, and emphasized optimal re-usage of formwork rather than material efficiency. The material and formwork costs and quantities of both designs were then compared, and the results confirmed the importance of formwork in the overall project cost.

Finally, this thesis examines some of the recent innovations involving formwork and formwork materials. These innovations aim at improving the quality and performance of formwork, as well as reducing the overall cost of the project.

1. FORMWORK QUALITY AND PERFORMANCE

1.1 INTRODUCTION

Concrete, when freshly mixed, is basically a plastic slurry that lacks physical strength and shape. However, it does impose considerable weight and fluid pressure. When poured in place on the construction site, the fresh concrete must be contained, shaped, and supported by some other means until it sufficiently cures to support itself and to attain its final form. Formwork serves as this temporary means of support. It must be strong enough to provide adequate support while preventing excessive deflection. Formwork must also resist lateral forces from wind, as well as from pressure exerted from the fresh concrete.

Formwork is an important factor in determining the finished appearance of the concrete. Proper stripping of the formwork after the concrete sufficiently cures is essential to avoid any damage to the concrete surface. The type of formwork material must be properly chosen to produce the desired surface texture of the concrete.

1.2 STRENGTH AND SERVICEABILITY

1.2.1 Loads

The basic consideration of formwork performance is strength, which is the ability to support all loads and forces imposed during construction. Horizontal forms must support dead loads based on the mass of the freshly placed concrete and the weight of the formwork itself. The forms must also support the construction live loads of the crew, the equipment, and any other loads applied during the construction. Formwork provides the initial support for the reinforcing bars, which must be maintained in the correct location during the pouring and curing of the fresh concrete. Correct location of the reinforcing bars is critical to ensure that the structural section will develop its proper design strength after curing.

Vertical forms must primarily resist lateral concrete pressures due to a particular height of plastic concrete. They must also resist the lateral forces due to wind and other forces, such as those caused by power equipment during construction. Adequate bracing must be provided to ensure lateral stability in all directions during and after the pour.

The American Concrete Institute Committee 347 recommends a minimum live load of 50 psf for workers and equipment and 75 psf if heavy equipment is to be used. A minimum dead load of 100 psf for concrete is recommended and 125 psf if heavy equipment is to be used. Wind forces can be based on local codes. However, a minimum of 15 psf is recommended by ACI [38].

1.2.2 Deflection

Each component of the formwork must provide adequate strength to support loads without exceeding deflection limits. Therefore, forms are designed to limit deflection as well as to carry loads safely. This is particularly important for areas where appearance of exposed surfaces is critical, and for walls and columns where the rate of placement of concrete is usually rapid.

Typical deflection limits for the various components are usually a maximum of $\frac{span}{360}$, but not to exceed $\frac{1}{16}$ inch for sheathing and $\frac{1}{4}$ inch for joists and beams [38]. These limits ensure that the resulting concrete sections will be straight once the forms are removed.

1.3 LATERAL PRESSURE

1.3.1 Hydrostatic Pressure

Freshly placed concrete initially acts like a liquid, exerting hydrostatic pressure against the vertical forms. Since hydrostatic pressure at any point in a liquid is the result of the weight of the fluid above, the lateral pressure acting on the formwork increases as the height of fresh concrete increases. The density of the concrete mix also influences the magnitude of the lateral pressure. The pressure, however, is only temporary and disappears when the concrete sets enough to support itself. Usually one or two hours must pass before the concrete begins to harden and stops pushing against the forms [6]. It is important to make sure that the concrete does not create pressure in excess of what the forms and ties are capable of resisting. Otherwise a breakout may occur, or forms may deflect to produce a pillow effect on the surface of the concrete.

The American Concrete Institute has developed several design formulas for calculating the maximum lateral pressure at any elevation in the form. They can be easily found in most construction textbooks [20, 31, 38]. The formulas are based on prescribed conditions of concrete temperature, rate of placement, slump of concrete, weight of concrete, and vibration.

1.3.2 Variable Conditions

Some of the variables that affect lateral pressure are the rate of concrete placement, the use of vibrators, variations in temperature, formwork stripping, and the choice of materials and release agents.

Rate of Placement

The rate of placement is one factor that affects lateral pressure. If concrete is rapidly placed in less time than it takes for the hardening to start, the height of fresh concrete will increase and add to the lateral pressure at the bottom of the form. With slower rates of placing, the concrete at the bottom of the form begins to harden before the upper part of the form is filled, and the pressure never gets a chance to build.

Placing concrete also creates an impact load on the formwork. The effects of impact during the concrete placing can vary depending on the height of the forms or the height of the concrete drop. The incoming concrete impacts the plastic concrete already in place, which transmits this additional lateral force to the forms. The faster the rate of placement, the greater the impact pressure is likely to be.

Forms and ties should have sufficient strength to withstand these impact loads. The rate of placing should not be greater than their safe load capacity. Drop chutes should be used to avoid segregation of aggregate and paste when placing concrete into high vertical forms. Free-fall distance should be limited to five feet or less [31].

The drop height of fresh concrete can be reduced by the use of elephant trunks or tremies. Another method is to simultaneously erect the forms and place the concrete. One side is usually formed to its total height while the second or opposing side is initially built only to the height of the first lift. As the placing starts the next lift of forms is erected and the placing operation follows closely behind. This method not only avoids long drops but also makes placing and vibrating easier.

Yet another method is to pump the concrete from the bottom of the form. However, it is important to fill the forms more rapidly. Slow pumping rates will allow setting to begin before the placement is completed, and excessive pressure will be produced inside the form.

ACI has developed formulas to calculate the proper pouring rates for given conditions. These formulas can be easily found in most construction textbooks [20, 31, 38].

Vibration

When a vibrator is inserted into concrete, it can temporarily liquefy all of the concrete that is within its full depth or radius of action. The result is an increase in lateral pressure. This increase needs to be accounted for and controlled. Extra care should be taken to prevent the vibrator from coming into contact with the form face, so that the release agents on the formwork remain intact. Care should also be taken to avoid damaging or breaking any of the ties when vibrating, since doing so would transfer more load to the other ties and load the forms unevenly. It is also possible to bulge and rupture any wall or column form by inserting the vibrator too deep into previously placed or partially set concrete. The vibrator should be allowed to penetrate the previously placed concrete about 1 inch, but not more than 8 inches [31].

External vibrations should be avoided if possible. External vibration with form vibrators increases loads on forms even more and causes fluctuations of lateral pressures. It also causes the formwork to oscillate enough to break up the film of the release agents so that the concrete hardens against the formwork itself and causes high stresses when stripped.

Temperature

The temperatures of both the concrete and the atmosphere affect the pressure since they affect the setting time. The rate of hardening of concrete is more rapid at high temperatures and slower at low temperatures, so the head of unhardened concrete in the formwork will vary with the weather. When temperatures are low, it is possible for greater lateral pressures to develop since larger amounts of fresh concrete can be poured before the previously placed concrete begins to set. For this reason most formwork manufacturers publish recommendations on placement rates for use at various temperatures. They also specify the use of proper heating and insulating equipment in cold weather. Heating and insulation functions to protect against damage from freezing and to establish an acceptable rate at which strength is gained. It also reduces pressures on formwork by allowing earlier setting.

In hot weather the main difficulty involves slump. An increase in slump has the effect of transmitting a higher pressure to the formwork. To overcome rapid slump loss at high temperatures some people are inclined to soup up the mix with additional water, often in violation of good practice. High temperatures need to instead be anticipated and accounted for during the design stage.

1.4 OTHER FACTORS

1.4.1 Stripping

One way to save money on a repetitive forming job is to cycle the forms as quickly as possible. However, there is a common tendency to produce bad surfaces by removing the forms too early. Surface damage is usually of two types: scaling, which occurs when patches of concrete stick to the forms; and edge damage or spalling of sharp corners, caused by shearing the formwork away from the concrete. Balancing such risks against cost savings is a key issue involving formwork stripping.

Although the forms must remain in place until the concrete has attained sufficient strength, prompt removal of forms is best where the surface will require treatment. Stripping should be started away from any corners, projections, or recesses, since these require working room and extra care. Crowbars or other objects with sharp edges or points should not be used against the concrete to loosen the forms. A flat object or wedge is usually best. A wooden wedge can be placed between the form and surface and tapped lightly to free the form. Hammering against the face of forms will leave marks in them and cause blemishes in the concrete the next time the forms are used.

1.4.2 Materials

The fresh concrete must be supported until it develops sufficient strength to support its own weight. Materials used as formwork must have sufficient strength to provide the necessary support during this critical period. They must also exhibit other qualities, such as durability, lightweight, and ease of assembly. The type of material is chosen according to the project requirements and design specifications. Proper selection of formwork material is essential for optimal productivity.

Wood

Wood in the form of dimensioned lumbar and plywood sheathing is the most widely used material for building forms, mainly because of its good strength-to-weight ratio, workability, relative low cost, and re-usability. Also, with its resistance to changing shape when wet and its ability to withstand rough usage without splitting, plywood is an excellent sheathing material. An additional advantage of using plywood as sheathing in formwork is its ability to bend, thus making it possible to produce smooth, curved surfaces. Wood that contacts the concrete should be non-staining and free from organic substances that may impart detrimental effects to the concrete.

Steel

Steel is widely used in formwork because of its strength and durability. Steel angles and bars are used extensively as supporting members for form panels faced with plywood sheathing. Steel corner pieces used in connecting panels are also popular because of their strength and re-usability. When large spans are encountered, structural steel sections can be used as framing members to support formwork sections. Steel-faced forms produce uniform color, but can sometimes cause blowholes to form in the concrete during vibrating. Unlike wood, which tends to absorb vibration, steel forms tend to reflect the vibrations and bounce them back. Rust-inhibiting oils should be used on steel-faced forms to avoid discoloration from iron oxides.

Plastics

Glass-fiber reinforced plastic is another material that has been used successfully as a forming material. It is lightweight and easy to handle, and its strength and toughness make it very re-usable. Another principle advantage is its ability to mold or take any designed shape and finish. Materials with a plastic surface provide more constant and predictable concrete surfaces. However, plastic sometimes is not stable under the heat of concrete pouring. Plastic forms are seldom built at the job site. They are usually manufactured under controlled shop conditions.

Aluminum

Although expensive compared to other materials, aluminum is used because of its lightweight qualities. It has the advantage of lighter shipping and handling, as well as a rust-free, prolonged life. Greater spans and loads can be supported by aluminum compared to wood or plastic. Aluminum shores have also been developed as an alternative to steel.

1.4.3 Release Agents

A form release agent must serve a number of purposes, such as: (1) to permit clean release of formwork from hardened concrete during stripping, (2) to protect the formwork for long life and extensive re-use, and (3) to prevent corrosion of metal forms and consequent staining of the concrete surface.

Release agents should be applied through spraying or rolling methods. Release agents could also be applied through brushing, mopping, or wiping, but doing so seldom produces a sufficient uniform film. A dipping method can also be applied, but it is not practical for use on the construction site. When dipped coatings are required for lumber or plywood, pre-dipping at the mill is the most practical solution.

Discoloration and blemishes due to the release agents can sometimes be a problem, and they occur usually over the entire concrete section or wall rather than in isolated areas. The difficulty may persist throughout the job, partly depending upon re-usage of forms. The type of blemish may vary as the job progresses since form materials begin to react differently with increased re-usage, exposure to the weather, and reapplication. For example, exposure to strong sunlight for a few hours can change the chemical characteristics of some release agents. Their application must therefore be timed accordingly.

To avoid these problems, form surfaces should be thoroughly cleansed, preferably before erection. Forms that are continually re-used are generally treated with the form release agent just after stripping and cleaning. Also, whenever possible, the application of the release agent should be timed so that it can dry or be absorbed into the formwork before the reinforcement is installed. This procedure prevents loose rust or dirt from the reinforcement from subsequently showing up as marks on the concrete surfaces. The release agent should also be applied carefully to avoid contacting reinforcement or adjacent construction joints.

Every form material reacts differently with form oils or other form release agents and its behavior may change between the first use and subsequent re-uses. Careful selection of form release agents should be made for the surface finish desired. Where the surface is critical, complete testing should be conducted under the conditions expected. It is much more economical to correct problems discovered through pre-testing than to correct them after a large investment in materials has already been made.

2. FORMWORK SAFETY

2.1 INTRODUCTION

Deadlines and financial pressures often result in safety being reduced to a bare minimum. However, the contractor, as well as the entire construction and design team, has the responsibility to make sure that quality and safety are not sacrificed while striving for better project economy.

Formwork failure is one of the safety issues that often results in serious or fatal consequences. Special attention has therefore been given to this aspect of construction safety. Form design is subject to local code requirements, as well as to a group of national regulations. Four documents that have established national applicability for formwork safety are: (1) Part 1926, Subpart Q of the Federal Construction Safety and Health Regulations (OSHA); (2) Chapter 6 of the American Concrete Institute (ACI) Building Code Requirements for Reinforced Concrete; (3) The American National Standards Institute (ANSI) A10.9, "Concrete and Masonry Work - Safety Requirements;" and (4) The ACI 347-78 standard, "Recommended Practice for Concrete Formwork."

2.2 SAFETY ISSUES

Experienced contractors and engineers often make their own lists of safety precautions. The following are some of the concerns that frequently appear.

Improper Stripping and Shore Removal

Forms and shores must be left in place until the concrete is strong enough to support itself. When schedules become tight, shore and form removal is often accelerated and can lead to disastrous results.

Re-shoring, the process of placing temporary shores under slabs or structural members after forms have been stripped, is a critical operation that must be carried out exactly as specified by the designer and must always have adequate bracing. Only a limited area should be stripped and re-shored at one time.

Inadequate Bracing

Diagonal bracing for shores and lateral bracing for wall and column forms are needed to resist wind and other lateral loads on the formwork system. Wind can come from any direction, and the bracing system must be capable of handling it at any time. If braces are positioned on only one side of the wall, they must be able to take tension or compression, and their connections must be able to do the same. The connections must be properly secured, especially nailed connections since vibrations from equipment, movement of workers, or even passing traffic can cause them to loosen. Other lateral loads that the bracing system must resist are those from cable tensions, inclined supports, concrete pouring, or impact from placing equipment.

The American Concrete Institute advises designing wall form braces for a minimum of 100 plf applied at the top, or 15 psf wind load, whichever is greater [21, 38].

Inadequate control of concrete placement

The rate and location of concrete placement should be controlled to avoid exceeding the design loads. Form specifications should be properly and carefully followed. When designing the formwork, the designer must be sure to use accurate values for the rate of placement and concrete temperature. Placing rates that are faster than anticipated by the designer can increase lateral pressure and cause a blowout. Temperatures lower than anticipated can also increase the pressure on the forms.

Lack of attention to details

The smallest mistake in assembly details can cause a local weakness and start a chain of events that can ultimately result in form failure. Simple things, such as insufficient nailing or failure to tighten locking devices on metal shoring, can lead to accidents or disasters. Other mistakes, such as skipping over a form panel when applying release agents, can slow down work or cause accidents when removing the panel.

Inadequate foundations for formwork

Formwork safety depends on all of the loads being properly transmitted to solid ground. Adequate mudsills must be placed under all shoring that rests on the ground. Furthermore, the bearing capacity of the soil must be great enough to support shoring loads. Many regulations prohibit mudsills from resting on frozen ground. Moisture and heat from concrete operations or changing air temperatures can thaw the soil and allow settlement that overloads or shifts the formwork. Site drainage must also be adequate to prevent a washout of soil supporting the mudsills. Surrounding excavations must be checked to ensure that formwork does not fail due to embankment failure.

3 FORMWORK ECONOMY

3.1 INTRODUCTION

In a sense, formwork is actually an entire temporary building that must be erected and demolished to produce a second, permanent concrete building. The cost of formwork is therefore a major component of the overall construction cost and is frequently more than the concrete itself. In a typical reinforced concrete frame, the concrete would cost about 20% of the total cost, including materials and labor. Reinforcement would cost about 25%, while formwork would comprise the remainder, anywhere from 40 to 60% [14, 31]. Labor for erecting and stripping the formwork would account for about three-quarters of this amount [27]. It is therefore imperative for designers and contractors to employ methods to cut the formwork and labor elements to a minimum.

However, design and construction have traditionally tended to focus on the permanent elements of the structure, and not necessarily on the ease of the building process. Designers often focus on the finished product and overlook formwork since it is a temporary element. For example, column sizes are usually designed to be smaller on the higher floors to increase their material efficiency, but doing so requires different sets of forms to be made. To design for optimal efficiency of the entire project, it is essential to ensure repetition. Repetition might result in less efficient permanent sections, but the financial reward is a net savings from lower formwork cost, faster construction, and more efficient labor.

Optimal project cost and constructability therefore depends on the ingenuity, efficiency, and planning of the design and construction team. If the architect and structural engineer work together with the contractor and the suppliers to create repetition on the job, productivity and job costructability will increase, and so will the profits.

3.2 ROLE OF THE DESIGN TEAM

Designers often try to reduce material costs by designing structural sections as small as possible. However, savings in permanent material do not always result in the most economical buildings. Because the labor and formwork are such large portions of the project, it is best to design for constructability. Designing the formwork to be easy to build and re-build will eventually save money.

The following are some of the things that the design team can do to help make the project more constructable.

Work together

The architectural drawings and structural design should be developed together at the same time. Doing so will bring about issues of constructability much earlier in the design process and could help avoid any possible costs or delays from re-designing later on.

Design Uniform Member Sizes

Designing for a single set of member dimensions will optimize constructability. The amount of reinforcement and concrete can be varied to achieve uniform sizes. The design should also consider readily available standard form sizes to avoid the costs of custom forms. The savings in formwork and shoring should exceed any possible cost increase in concrete or steel. The workers will also learn the job faster, which should increase output and decrease labor costs.

Avoid irregularities

Architects often design structures without considering how difficult it is to build them. In fact, 90% of design time is usually spent on the structural design, while as little as 10% is spent on detailing and analyzing how the structure will be built [27]. If the economics of the project is of greater concern than the architectural daring of the structure, then special or complex designs should be avoided. Columns, beams, and slabs should be kept as simple as possible.

Maintain uniform floor height

A uniform floor height should be maintained for as many floors as possible. If changes are necessary, the upper stories should be reduced in height to allow the forms to be cut and re-used.

Space columns uniformly

The same spacing between structural members should be used in order to maximize re-use and eliminate cutting and fitting of forms.

3.3 ROLE OF THE CONSTRUCTION TEAM

The following are some of the things that the construction team must do to improve the constructability of the project and to ensure maximum formwork usage.

Choose an Effective Formwork System

The contractor's first step towards better constructability is the careful selection of a formwork system that is suited to the size of the job, and which provides the greatest re-usage. Proper selection of the system is of major importance, and how the equipment is used is a deciding factor in achieving maximum economy. Other considerations involve the project location, time of year, project duration, availability of equipment, and the amount of hardware and miscellaneous items to handle.

Select only one formwork system

When possible, one formwork system should be picked and maintained throughout the project. Each formwork scheme adds costs for mobilization, new material, and the increased learning curve for the workers.

Effectively plan and control the re-use cycle

Overall site planning and control is foremost in constructability. They should include the careful planning of sequential, coordinated cycles or movement of formwork, components, and equipment. The cycle should be set up to maximize formwork re-use. It also needs to comply with specifications regarding the length of time that the forms and shoring must remain in place. The cycle should make formwork setting and stripping a daily repetition in order to increase worker productivity. The cycle should also keep all of the equipment in use at all times. Idle equipment can be very costly to the job.

Coordinate with other construction activities

Timing must be planned to avoid interference with the forming crew and the other site activities. Consideration should be given to analyze how forming operation will affect other operations such as material handling, steelwork, plumbing and finishing.

4. A STUDY IN OPTIMIZING FORMWORK COST

4.1 INTRODUCTION

The High Performance Structures team in the 1997-98 Masters of Engineering program was assigned the task of developing the preliminary design for a new MIT Civil and Environmental Engineering building. The proposed site of the new CEE building is located at the corner of Main Street and Vassar Street. It has a site dimension of 400 x 120 feet.

The design required 112,000 square feet of usable space for classrooms, offices, and laboratories. To enable maximum flexibility, a laboratory live loading of 200 psf was used in the design for each floor. A maximum height restriction of 120 feet was also factored into the overall design.

Group A of the HPS team proceeded with the conceptual design of a 60 x 360-foot building. An innovative exterior façade surrounded the building and was primarily separated from it. About two-thirds of the length of the building consisted of 5 main floors and a basement, while the remaining one-third consisted of 3 floors covered by an innovative membrane roof system. The total usable floor space, including the basement, was 114,000 square feet.

Four different structural schemes were developed for the main interior frame of the building and were compared for cost and performance. One structural steel scheme supported the floors with a braced steel frame, while another scheme hung the floors from large steel trusses supported by 100-foot exterior columns. Another scheme incorporated the use of huge core columns that were 20 feet in diameter.

One of the schemes ("Scheme C" of the project) was a concrete cast-in-place structural frame. Its design will be incorporated into this formwork study.

4.2 CAST-IN-PLACE CONCRETE SCHEMES

4.2.1 Minimum-sizing scheme

"Scheme C" of the project consisted of 6-inch, reinforced, one-way slabs with clear spans of 15 feet. The columns along the width of the building (east to west), as well as those along the length of the building (north to south), were spaced at 15 feet on center. This column spacing created a 15-foot length for the end-beams and the end-girders. The interior girders were spaced 15-feet on-center along with the columns, with an unsupported length of 60 feet. The columns each had an unbraced length of 20 feet, with the exception of the top floor, which had an unbraced length of 15 feet.

This concrete scheme was designed for member efficiency. In other words, the structural members were designed with minimal dimensions in order to use the minimal amount of materials.

The structural drawings and concrete schedules for this minimum-sizing scheme are presented in Appendix A.

4.2.2 Uniform-sizing scheme

Another concrete scheme was designed and developed for the purpose of this exercise. The same structural layout and loads were used as in the previous concrete structure. However, rather than being designed for minimum material sizing, this alternate scheme was designed for uniform member dimensions. The columns, beams, and girders were all designed to fit a single set of dimensions.

The structural drawings and concrete schedules for this uniform-sizing scheme are presented in Appendix C.

4.3 STRUCTURAL CONCRETE DESIGN

The two cast-in-place concrete schemes were designed using the Ultimate Strength Design process. The structural calculations are presented in Appendix B for the minimum-sizing scheme and in Appendix D for the uniform-sizing scheme.

The following is a description of some of the design factors and considerations.

Slabs

The one-way reinforced concrete slabs were designed using the ACI Moment Coefficient method presented by Nilson and Winter [30]. Deflection was controlled through a minimum thickness requirement. A clear cover of 3/4 inch, a bar radius of 1/4 inch, and a steel ratio of 0.0018 were used in the design. The slabs were designed to support the roof or live loads, as well as the self-weight of the concrete. Negative, positive, and transverse steel reinforcing was designed.

Beams

The rectangular edge beams were designed using the Ultimate Strength Design method presented by Nilson and Winter [30]. The beams were designed to support the loads from the slab, the dead load from the walls or cladding, and the self-weight. Negative and positive reinforcing was designed. In this preliminary design, web reinforcements were estimated as 10% of the total reinforcement. Deflection was controlled through the use of a steel ratio of $0.18\frac{f'c}{fy}$.

Girders

The rectangular girders were designed using the same procedure as the edge beams. The interior girders support the loads from the beams and the self-weight. The edge girders support an additional wall or cladding load.

Columns

The square reinforced columns were designed using the Ultimate Strength Design method presented by Nilson and Winter [30]. The columns were designed to support the load and moment from the girders and the edge-beams, as well as the load from the upper columns. The area of reinforcing steel was found with the aid of interaction diagrams for the factored design values of stress and moment. Web reinforcements were estimated as 10% of the total reinforcing steel.

4.4 COSTS AND QUANTITIES

Material quantities and contact areas were then taken for each scheme. The required formwork and materials were priced according to the unit costs listed in the 54th edition of Means Building Construction Cost Data [28]. The items included in the cost estimation were formwork, reinforcing steel, ready-mix concrete, curing, finishing, and placing. Walls, cladding, and other factors constant for both schemes were excluded for the purposes of this exercise. The members were to be poured monolithically, with an estimated pouring rate between 400 and 450 cubic yards per day.

The quantities and cost estimations are presented in Appendix E for the minimum-sizing scheme and in Appendix F for the uniform-sizing scheme.

4.5 RESULTS

The uniform-sizing scheme required an additional 457 cubic yards of concrete (+8%), which added another $52,139 (+7%) to the cost of concrete. It also required an additional 59 tons of reinforcing steel (+7%), which resulted in $26,303 (+3%) more to the cost of reinforcement. An additional 13,428 square feet of contact area (+10%) was required for formwork. However, despite this addition in contact area, the cost of formwork decreased by $201,175 (-30%). The results are summarized in the following table:

Material and Cost Summary	
Minimum-Sizing Scheme	*Uniform-Sizing Scheme*
Formwork: 264,447 SF $681,679	Formwork: 277,875 SF $480,504
Concrete: 5,813 CY $698,274	Concrete: 6,270 CY $750,413
Reinforcing Steel: 784 Tons $965,144	Reinforcing Steel: 843 Tons $991,447
Total: $2,345,096	Total: $2,222,363

These results confirm the importance of formwork re-use in project economy. The uniform-sizing scheme provided a better opportunity to utilize formwork re-use. Rather than only 1 to 2 re-uses attained in the minimum-sizing scheme, the uniform-sizing scheme was able to re-use its formwork 4 times. As a result, the cost of formwork decreased by a higher percentage than the increase in material cost, and a net savings was achieved for the total material cost of the project. This material cost savings, however, is only a small fraction of the total potential savings that can be realized through optimal formwork re-use. The increased repetition on the job site due to the formwork re-usage will result in improved labor productivity and shorter construction time. Although not investigated in this exercise, these factors will certainly produce much greater savings in project cost.

5 FORMWORK INNOVATIONS

5.1 INTRODUCTION

More and more construction and design professionals are realizing that formwork is a key element to any cast-in-place concrete structure. Only the most up-to-date products and methods will be acceptable in today's highly competitive market. As a result, greater attention is being placed on new formwork materials that enable faster, cheaper, and more efficient concrete construction.

Innovations in formwork materials usually increase the initial cost of the system, but they create significant benefits to the overall productivity of the project. In the UK, for example, the ratio of labor savings to material cost is 3 to 1 [14]. In other words, an improved formwork method that is initially 5% more expensive will result in 15% total project savings from better labor productivity.

The following is a look at some of the recent improvements in formwork development.

5.2 IMPROVEMENTS WITH PLASTICS

Designers continue to use the material advantages of plastic to improve form erection and stripping time. One recent example is a plastic formwork system called Plastiform [18, 37]. Plastiform is made of extruded high-density poplyethylene, which is only 1/3 the weight of conventional lumber and is therefore easier to handle. No nailing is required with Plasiform. Stapled into the ends of each form are molded plastic connectors that allow workers to simply snap the forms together for quick setup and unsnap them when stripping. Positioning and repositioning the forms are easy. Each form has a dovetail slot that runs the entire length of a side. Re-usable clamps can be inserted and locked anywhere along the slot. With these advantages, Plastiform is twice as fast as steel or wood for setting and three times as fast for stripping. Other advantages of Plastiform involve its flexibility and smooth texture. The plastic forms can bend in any direction to form inside or outside radii, and can be cut or trimmed to further enhance flexibility. They are also easier to strip, since the concrete generally does not adhere to the form's smooth finish. Plastiform can be rubbed clean or washed with water, and is flexible enough to allow dried concrete to be cracked off if necessary. Plastiform also has built in UV inhibitors and is re-usable for years.

5.3 CONTROLLED PERMEABLE FORMWORK (CPF)

Formwork has traditionally been a watertight container that kept the water and grout from escaping. If the water is not allowed to evaporate or escape while the concrete is fresh, it can lead to voids, which can have a significant effect on the appearance and durability of the concrete. A high water-to-cement ratio means lower strengths as well as increased porosity and permeability. Chlorides and alkalis are more likely to attack the reinforcement in the critical cover zone, which is the concrete between the surface and the reinforcement. A high water-to-cement ratio also means greater pore water pressure, which accounts for about 90% of the lateral pressure on the formwork [16].

One method of improving durability without special coatings or admixtures is to line the formwork with a specially designed porous membrane. This technique is known as Controlled Permeable Formwork [14, 16] and is applicable to almost all types and grades of concrete. CPF is a layer of micropores that acts as a filter. It retains the cement paste but allows trapped air and excess water to pass through and drain away. In other words, CPF allows the surface of the newly poured concrete to breathe during the early hydration and curing phases. By doing so, it creates an optimal level of hydration and maintains the correct moisture level during the curing process. The result is a denser and smoother concrete surface with increased durability and higher initial strength. This improved initial strength allows for earlier formwork stripping, while the surface quality requires significantly less touch-up work or repair.

Another result is a significant reduction in permeability. Tests show that the pores of concrete formed with CPF are on average six times finer, creating a very dense cover zone that dramatically reduces the damaging effects of carbonation, freeze/thaw cycling, and chloride ingress.

Tests also show that CPF exhibits four times less pore volume than equivalent, conventionally formed concrete, resulting in a substantial reduction in formwork pressures. This in turn leads to lighter formwork, and therefore higher productivity.

CPF is initially more expensive than the use of conventional formwork. However, these significant labor and productivity benefits, along with a longer re-use life, result in significant cost advantages in the long run.

Dupont has recently released a new high-performance CPF form liner called Zemdrain MD [19, 27, 36]. Zemdrain offers easier fixing, better economy, and more consistent results. The new liner consists of a latticed support grid that provides reinforcement, prevents stretching, and assists with drainage. The improved stiffness enables attachment to the formwork without the need for tensioning, which ensures a wrinkle-free surface.

5.4 IMPROVEMENTS WITH PLYWOOD AND RELEASE AGENTS

Release agents are applied to formwork to prevent the concrete from sticking to the form faces. Occasionally, release agents can cause problems, such as differential staining on the finished concrete surfaces or non-adhesion of decorative concrete finishes. In addition, some of the chemicals involved have raised concerns over workers' health and subsequent ground contamination. There are some parts of the world where legislation has been enacted to ban chemical release agents.

Simpson EnviroForm [27, 32] concrete forming panels, available through CSC Forest Products, are ideal for these situations. EnviroForm is a specially prepared plywood that eliminates the need for any release agents. Each panel features a "Dri-Strip" polymeric coating that protects the integrity of both the plywood and the concrete. Because no release agents are required, there is no residue left in the face pores, and the problems involving release agents are avoided. Furthermore, material costs are lower since the release agents, along with the handling, storing, and application of such agents, are no longer needed. EnviroForm panels are also smoother and more uniform since they are filed and sanded before the overlay is applied. Contractors can achieve between 20 to 40 re-uses of the panels without any re-treatment of the surface.

5.5 PERMANANT FORMWORK

The Taisei [24] company recently developed a polymer-impregnated concrete that is used as formwork and becomes part of the structure after the concrete is cast. This permanent formwork consists of a fiber-reinforced concrete board that is impregnated with polymer. The polymeric material fills the pores and cracks, reducing the porosity of the concrete by 85%. It also increases the concrete's performance regarding water-tightness, freeze-thaw durability, and corrosion resistance. Because the formwork does not need to be stripped, the product reduces labor costs while increasing constructability.

5.6 IMPROVEMENTS WITH ALUMINUM

SGB Formwork has recently produced a new aluminum shoring system called GASS [14, 15]. The GASS system is safer, stronger, and more versatile than any previously available system. It requires only 3 main individual components: an outer leg, an inner leg, and a ledger frame. This design results in less difficulty with identifying components, as well as greater adaptability and configuration options for the user. The ledger frames can be attached to the outer leg in 8 multiple directions for even greater adaptability. Each frame only requires 4 wedge connections, which results in faster installation.

The legs comprise a variable loading system with a capacity ranging from 40 to 130kN. An optional link between the outer legs ensures even higher loading capacity. This variation enables GASS to suit any application, from the simplest to the most demanding project. It also increases efficiency since the system is no longer subjected to a single load restriction over the entire system.

Other accessories provide GASS with even more versatility and efficiency. A swivel/rocking head plate enables the system to adapt to slope requirements of up to 15°. Extension legs allow multiple leg stack-ups when very high shoring is required. Saddle beams attached to the inner legs allow a second level of support where drop beams occur in slabs. A castor/trolley unit enables the system to move quickly and effectively around a site without the need to erect and dismantle every time.

The new safety features of GASS include a T-bolt and wedge connection that can only be in a fixed or unfixed position. This allows for easy visual checking with less confusion on site, and removes any doubt about bolt slipping. A square-tooth frame lock adds stability while an external latch mechanism firmly locks the leg without imposing any stress.

The initial material cost of the GASS system is about twice as much as a conventional shoring system. However, the new features of the GASS system result in productivity levels five times greater than previously available shoring systems. For example, the lightweight components greatly facilitate handling and erection, while the economical design significantly reduces the amount of equipment and assembly required. These improvements will translate into major savings in labor costs and construction time.

CONCLUSION

Formwork is a critical factor that determines the constructability of cast-in-place concrete structures. It can greatly affect the cost and efficiency of the project, as well as the final quality of the concrete. It is important that the formwork be safe and strong enough to support the imposed loads and pressures. Design teams should consider formwork as a primary factor when designing concrete members. Construction teams should carefully design, choose, and manage the formwork to optimize strength, service, and productivity.

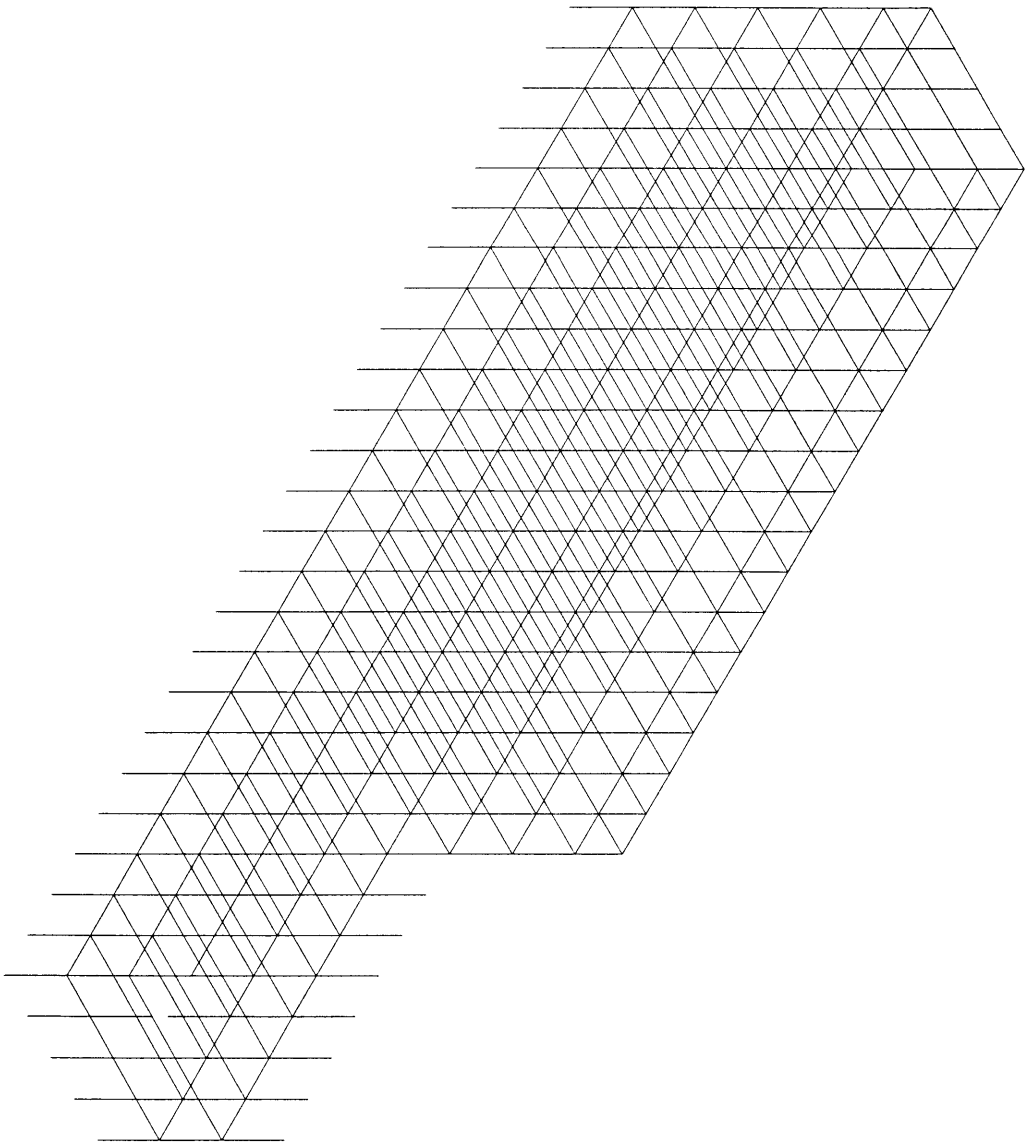
Appropriate use of innovations and technology can enable contractors to significantly reduce the cost of formwork and have a tremendous effect on the project's economy. More and more construction and design professionals are realizing that only the most up-to-date products and methods will be acceptable in today's highly competitive market. However, many contractors still focus on the initial cost of the formwork and concentrate on finding ways to procure the materials for the lowest cost. They decline on paying the extra initial 5 to 10% for the new technology, even if it means increasing productivity in the long run by a much greater percentage. Formwork manufacturers believe that the only way to steadily progress is to educate both the clients and the contractors about the long-term benefits that can be gained through formwork innovations. They must realize that by sacrificing innovation, they are compromising improvements in safety, productivity, cost-effectiveness, and competitiveness.

In any case, formwork is much more than just a temporary element. Rather than only focusing on what it costs to put formwork up and how long it takes to tear it down, construction and design professionals should always focus more on what formwork can add to the overall productivity of the project. Dismissing formwork as merely a simple, temporary structure can be a crucial mistake to any cast-in-place concrete design.

APPENDIX A:

Structural Drawings and Concrete Schedule

Minimum Sizing Scheme



Isometric View

Minimun-Sizing Scheme

Paper Size: 11" x 17"

CONCRETE SCHEDULE
Minimum Sizing Scheme

Members

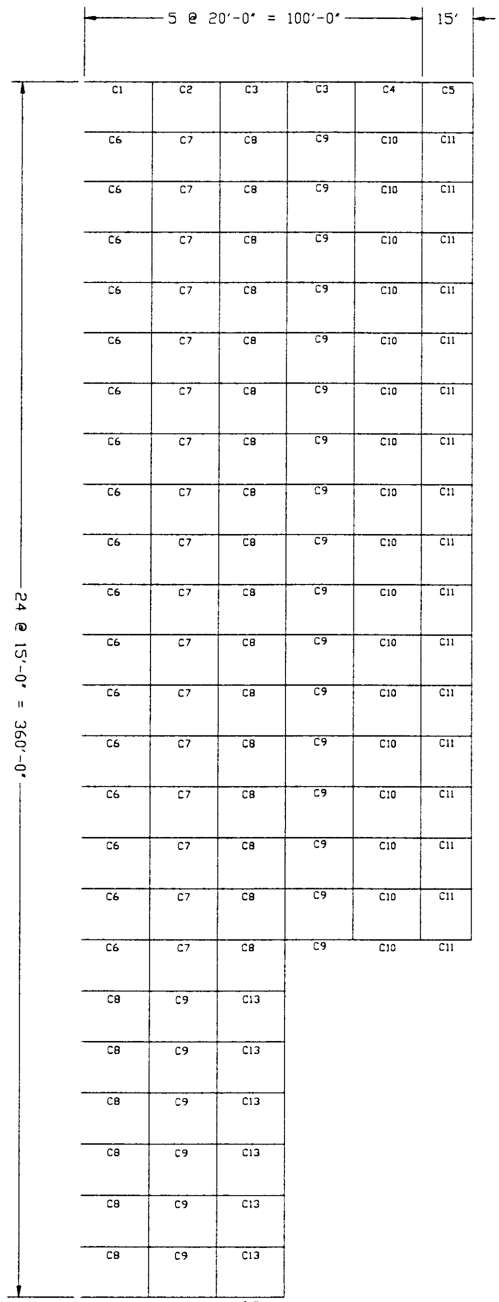
Member	b(in.)	d(in.)	l(ft)	Neg.As(in^2)	Neg.Reinf.	Pos.As(in^2)	Pos.Reinf.
B1	5	9	15	0.45	3No.4	0.36	2No.4
B2	7	10	15	0.76	4No.4	0.54	3No.4
G1	24	32	60	12.10	8No.11	9.51	10No.11
G2	6	14	15	1.08	6No.4	1.00	6No.4
G3	24	48	60	19.01	5No.18	14.90	12No.10
G4	10	16	15	2.16	5No.6	1.80	3No.7

Columns

Column	b(in.)	d(in.)	h(ft)	As(in^2)	Reinf.
C1	16	16	20	1.79	7No.5
C2	16	16	20	2.17	8No.5
C3	14	14	20	4.16	7No.7
C4	12	12	20	3.06	7No.6
C5	12	12	15	3.06	7No.6
C6	36	36	20	33.05	16No.14
C7	36	36	20	35.80	16No.14
C8	34	34	20	44.20	11No.18
C9	34	34	20	49.13	12No.18
C10	32	32	20	21.76	14No.11
C11	32	32	15	21.76	14No.11
C12	12	12	20	3.06	7No.6
C13	32	32	20	21.76	14No.11

Slabs

Slab	t(in.)	w(ft)	l(ft)	Neg.As(in^2)	Neg.Reinf.)	Pos.As(in^2)	Pos.Reinf.	Transv.As(in^2)	Transv.Reinf.
S1	6	15	60	0.17	No.3@8in.	0.11	No.3@10in.	0.13	No.3@10in.
S2	6	15	60	0.53	No.4@4.5in.	0.34	No.4@7in.	0.13	No.3@10in.

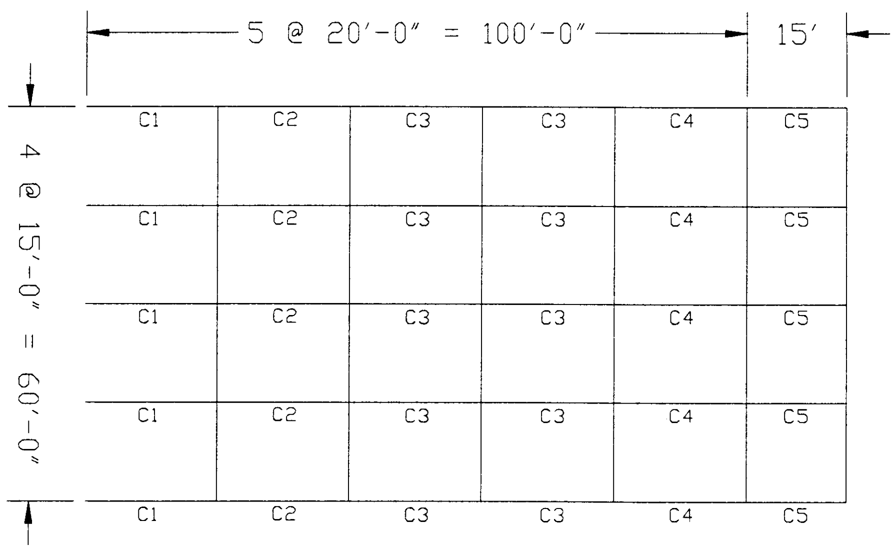


South Elevation

Scale: 1/32"=1'

Minimum-Sizing Scheme

Paper Size: 11" x 17"

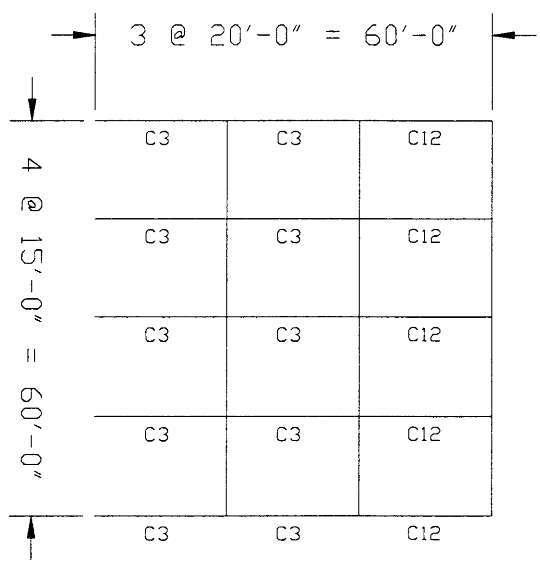


West Elevation

Scale: 1/16"=1'

Minimum-Sizing Scheme

Paper Size: 11" x 17"

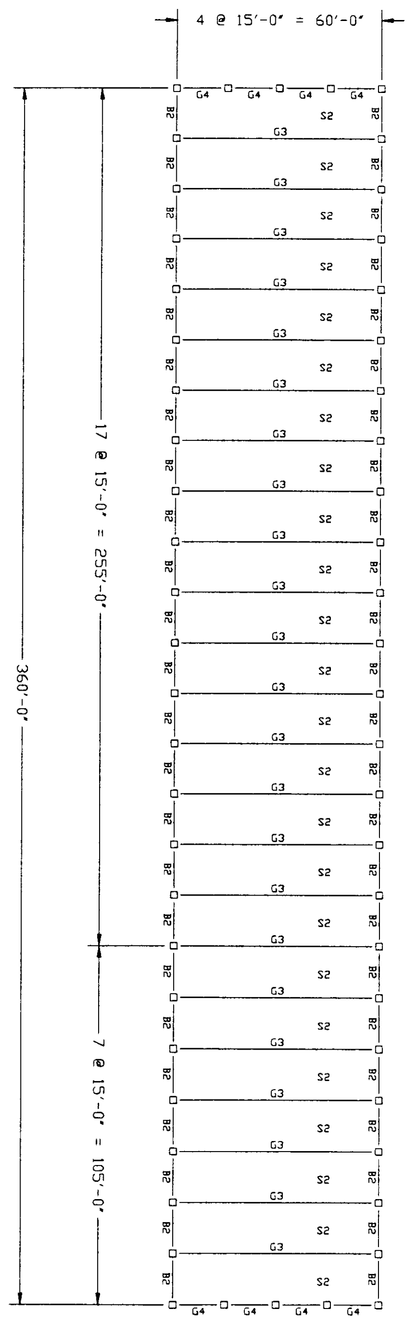


East Elevation

Scale: 1/16"=1'

Minimum-Sizing Scheme

Paper Size: 11" x 17"

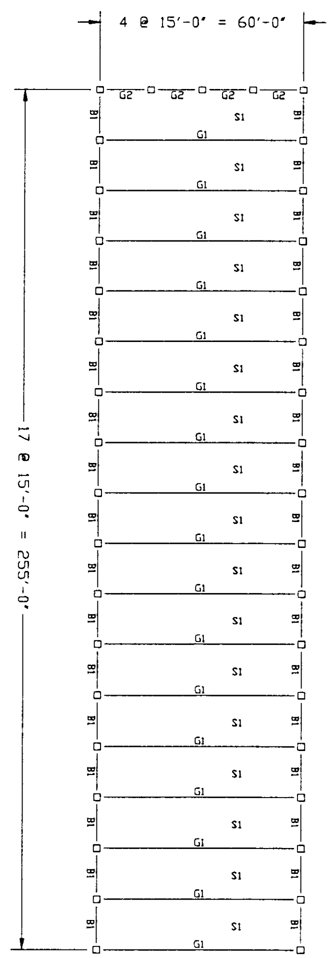


Scale: 1/32"=1'

Floor Plan

Paper Size: 11" x 17"

Minimum-Sizing Scheme



Scale: 1/32"=1'

Roof Plan

Paper Size: 11" x 17"

Minimum-Sizing Scheme

APPENDIX B:

Structural Calculations

Minimum Sizing Scheme

Preliminary Concrete One-Way Slab Design

Minimum Sizing Scheme

Slab S1

Clear span, l = 15ft
unit width = 1 ft
fy = 60 ksi
f'c = 6 ksi
γc = 150 pcf
steel ratio, p = 0.0018

Thickness

deflection control = 30

$$h = \frac{lx12}{30} = 6 \text{ in}$$

trial h = 6 in

Loads

$$\text{slab} = \frac{h}{12} \text{x } \gamma c = 75 \text{ psf}$$

live = 10 psf
snow = 30 psf
D.L. = 1.4 x 75 = 105 psf
L.L. = 1.7 x 10 = 17 psf
S.L. = 1.7 x 30 = 51 psf

$$\text{Wu} = \frac{DL + LL + SL}{1000} = 0.173 \text{ ksf}$$

Moment Coefficients

$$\text{positive(midspan)} = \frac{1}{14}$$

$$\text{negative(support)} = \frac{1}{9}$$

Negative Steel Reinf.

trial a = 1in
d-a/2 = 4.5 in

$$\text{As trial1} = \frac{M(12)}{0.9(fy)(d-a/2)} = 0.22 \text{ in}^2$$

$$a = \frac{As(fy)}{0.85(f'c)(1x12)} = 0.21 \text{ in}$$

d-a/2 = 4.90 in
As trial2 = 0.196 in^2
a = 0.1925 in
d-a/2 = 4.90 in
As trial3 = 0.196 in^2
a = 0.19 in

use bar No.3, 8"spacing

Effective Depth

clear cover = 0.75 in
bar radius = 0.25 in
d = h x 0.75 x 0.25 = 5 in

Shear

$$\text{Vu} = \frac{Wu(l)}{2} = 1.3 \text{ k/ft}$$

$$\text{vu} = \frac{Vu}{(1x12)(d)} = 0.0216 \text{ ksi}$$

$$\text{vc} = \frac{2(0.85)\sqrt{(f'c)(1000)}}{1000} = 0.13 > \text{vu ok}$$

Moments

$$\text{M positive} = \frac{1}{14}(\text{Wu})(\text{l}^2)(1\text{ft}) = 2.78 \text{ ft-k}$$

$$\text{M negative} = \frac{1}{9}(\text{Wu})(\text{l}^2)(1\text{ft}) = 4.33 \text{ ft-k}$$

Positive Steel Reinf.

a = 0.19 in
d-a/2 = 4.90 in

$$\text{As} = \frac{M(12)}{0.9(fy)(d-a/2)} = 0.126 \text{ in}^2$$

use bar No.3, 12"spacing

Transverse Steel Reinf.

Ag = h x (1x12) = 72 in^2
As = p x h = 0.1296 in^2

use bar No.3, 10"spacing

Slab S2

Clear span, l = 15ft
unit width = 1 ft
fy = 60 ksi
f'c = 6 ksi
γc = 150 pcf
steel ratio, p = 0.0018

Thickness

deflection control = 30

$$h = \frac{l x 12}{30} = 6 \text{ in}$$

trial h = 6 in

Loads

$$\text{slab} = \frac{h}{12} \text{ x } \gamma c = 75 \text{ psf}$$

live = 200 psf
D.L. = 1.4 x 75 = 105 psf
L.L. = 1.7 x 200 = 340 psf

$$Wu = \frac{DL + LL}{1000} = 0.445 \text{ ksf}$$

Moment Coefficients

$$\text{positive(midspan)} = \frac{1}{14}$$

$$\text{negative(support)} = \frac{1}{9}$$

Negative Steel Reinf.

trial a = 1in
d-a/2 = 4.5 in

$$\text{As trial1} = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.55 \text{ in}^2$$

$$a = \frac{As(fy)}{0.85(f'c)(1x12)} = 0.54 \text{ in}$$

d-a/2 = 4.73 in
As trial2 = 0.52 in^2
a = 0.51 in
d-a/2 = 4.74 in
As trial3 = 0.52 in^2
a = 0.51 in

use bar No.4, 4.5"spacing

Effective Depth

clear cover = 0.75 in
bar radius = 0.25 in
d = h x 0.75 x 0.25 = 5 in

Shear

$$Vu = \frac{Wu(l)}{2} \text{ k/ft}$$

$$vu = \frac{Vu}{(1x12)(d)} = 0.056 \text{ ksi}$$

$$vc = \frac{2(0.85)\sqrt{(f'c)(1000)}}{1000} = 0.13 > vu \text{ ok}$$

Moments

$$\text{M positive} = \frac{1}{14}(Wu)(l^2)(1ft) = 7.15 \text{ ft-k}$$

$$\text{M negative} = \frac{1}{9}(Wu)(l^2)(1ft) = 11.13 \text{ ft-k}$$

Positive Steel Reinf.

a = 0.51 in
d-a/2 = 4.74 in

$$As = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.335 \text{ in}^2$$

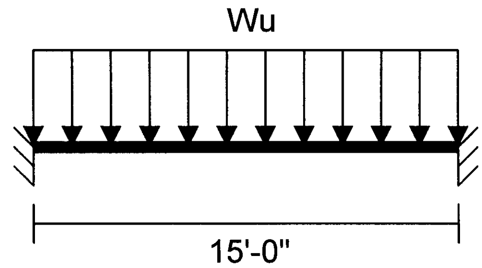
use bar No.4, 7"spacing

Transverse Steel Reinf.

Ag = h x (1x12) = 72 in^2
As = p x h = 0.1296 in^2

use bar No.3, 10"spacing

Preliminary Concrete Beam Design
Minimum Sizing Scheme



Beam B1
Beam length, l = 15ft
floor height, h = 15ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, $p = 0.18 \times \frac{f'c}{fy} = .018$

Loads
wall load = 20 psf
$W_{wall} = (20) \times h = 300$ ppf

$W_{beam} = \frac{bd}{144} \times \gamma c = 47$ ppf

$Wu = 1.4 \times (W_{wall} + W_{beam}) = 485.6$ ppf

$Pu = \frac{Wu(l)}{2} = 3642$ lbs

Negative Moment Reinforcement

Neg.Moment

$Mu = \frac{Wu(l)(l)}{9} \times 12 = 145688$ lb-in.

$bd^2 = \frac{Mu}{0.145 f'c} = 167$

Try
b = 5 in
h = 9 in
d = h-4 = 5 in

$bd^2 = 125$

Neg.Steel Reinforcement

As = p x b x d = .45 in^2
Use bar No.4
3 bars near supports
1 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$Mu = \frac{Wu(l)(l)}{14} \times 12 = 93656$ lb-in.

$bd^2 = \frac{Mu}{0.145 f'c} = 108$

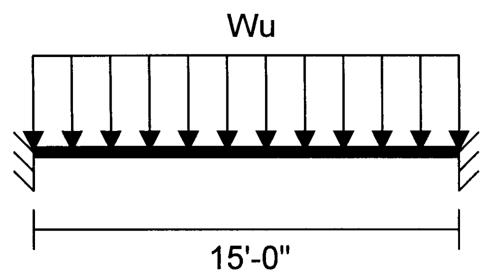
Try
b = 5 in
h = 8 in
d = h-4 = 4 in

$bd^2 = 80$

Pos.Steel Reinforcement

As = p x b x d = 0.36 in^2
Use 2 No.4 bars

Beam B2



Beam length, l = 15ft
floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy}$ = .018

Loads

wall load = 20 psf
W_{wall} = (20) x h = 400 ppf

$W_{beam} = \dfrac{bd}{144} \times \gamma c$ = 73 ppf

$Wu = 1.4 \times (W_{wall} + W_{beam})$ = 662 ppf

$Pu = \dfrac{Wu(l)}{2}$ = 4966 lbs

Negative Moment Reinforcement

Neg.Moment

$Mu = \dfrac{Wu(l)(l)}{9} \times 12$ = 198625 lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c}$ = 228

Try
b = 7 in
h = 10 in
d = h-4 = 6 in

bd^2 = 252

Neg.Steel Reinforcement

As = p x b x d = .756 in^2
Use bar No.4
4 bars near supports
1 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$Mu = \dfrac{Wu(l)(l)}{14} \times 12$ = 127688 lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c}$ = 147

Try
b = 6 in
h = 9 in
d = h-4 = 5 in

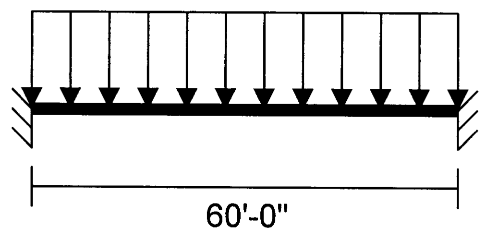
bd^2 = 150

Pos.Steel Reinforcement

As = p x b x d = 0.54 in^2
Use 3 No.4 bars

Preliminary Concrete Girder Design
Minimum Sizing Scheme

Girder G1



Girder length, l = 60ft
Trib width, s = 15ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, $p = 0.18 \times \dfrac{f'c}{fy} = .018$

Loads
slab design = 145 psf
$W_{slab} = (145) \times s = 1475$ ppf

$W_{beam} = \dfrac{bd}{144} \times \gamma c = 700$ ppf

$Wu = W_{slab} + (1.4 \mathrm{x} W_{beam}) = 3155$ ppf

$Pu = \dfrac{Wu(l)}{2} = 94650$ lbs

Negative Moment Reinforcement

Neg.Moment

$Mu = \dfrac{Wu(l)(l)}{9} \times 12 = 15144000$ lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c} = 17407$

Try
b = 24 in
h = 32 in
d = h-4 = 28 in

$bd^2 = 18816$

Neg.Steel Reinforcement

$As = p \times b \times d = 12.1\ in^2$
Use bar No.11
8 bars near supports
3 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$Mu = \dfrac{Wu(l)(l)}{14} \times 12 = 9735429$ lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c} = 11190$

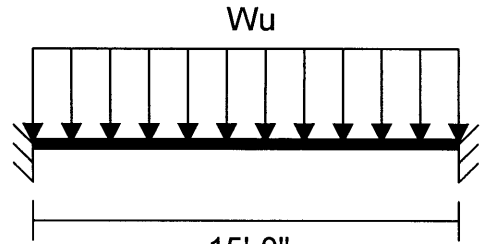
Try
b = 24 in
h = 26 in
d = h-4 = 22 in

$bd^2 = 11616$

Pos.Steel Reinforcement

$As = p \times b \times d = 9.50\ in^2$
Use 10 No.9 bars

Girder G2



Girder length, l = 15ft
Trib width, s = 7.5ft
Floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, $p = 0.18 \times \dfrac{f'c}{fy} = .018$

Loads
slab design = 145 psf
$W_{slab} = (145) \times s = 1088$ ppf
wall load = 20 psf
$W_{wall} = (20) \times h = 400$ ppf

$W_{beam} = \dfrac{bd}{144} \times \gamma c = 62.5$ ppf

$Wu = W_{slab} + 1.4x(W_{wall}+W_{beam}) = 1735$ ppf

$Pu = \dfrac{Wu(l)}{2} = 13013$ lbs

Negative Moment Reinforcement

Neg.Moment

$Mu = \dfrac{Wu(l)(l)}{9} \times 12 = 520500$ lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c} = 598$

Try
b = 6 in
h = 14 in
d = h-4 = 10 in

$bd^2 = 600$

Neg.Steel Reinforcement

$As = p \times b \times d = 1.08\ in^2$
Use bar No.4
6 bars near supports
2 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$Mu = \dfrac{Wu(l)(l)}{14} \times 12 = 334607$ lb-in.

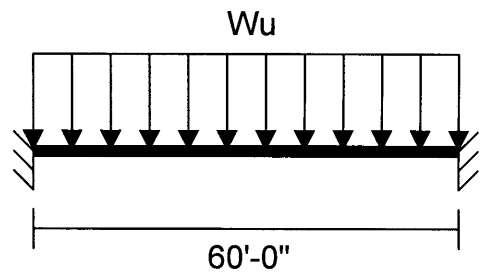
$bd^2 = \dfrac{Mu}{0.145 f'c} = 385$

Try
b = 7 in
h = 12 in
d = h-4 = 8 in

$bd^2 = 448$

Pos.Steel Reinforcement
$As = p \times b \times d = 1.008\ in^2$
Use 6 No.4 bars

Girder G3



Girder length, l = 60ft
Trib width, s = 15ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy} = .018$

Loads
slab design = 445 psf
$W_{slab} = (445) \times s = 6675$ ppf
$W_{beam} = \dfrac{bd}{144} \times \gamma c = 1200$ ppf
$Wu = W_{slab} + (1.4 \times W_{beam}) = 8355$ ppf
$Pu = \dfrac{Wu(l)}{2} = 250650$ lbs

Negative Moment Reinforcement

Neg.Moment
$Mu = \dfrac{Wu(l)(l)}{9} \times 12 = 40104000$ lb-in.
$bd^2 = \dfrac{Mu}{0.145 f'c} = 46097$

Try
b = 24 in
h = 48 in
d = h-4 = 44 in

$bd^2 = 46464$

Neg.Steel Reinforcement

$As = p \times b \times d = 19.0 \text{ in}^2$
Use bar No.18
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement

Pos.Moment
$Mu = \dfrac{Wu(l)(l)}{14} \times 12 = 25781143$ lb-in.
$bd^2 = \dfrac{Mu}{0.145 f'c} = 29634$

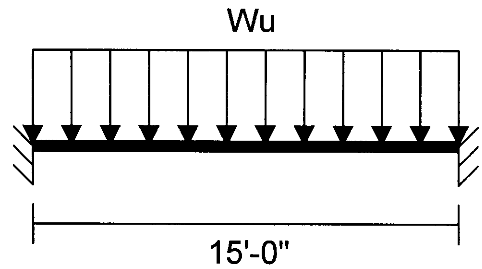
Try
b = 23 in
h = 40 in
d = h-4 = 36 in

$bd^2 = 29808$

Pos.Steel Reinforcement

$As = p \times b \times d = 14.90 \text{ in}^2$
Use 12 No.10 bars

Girder G4



Girder length, l = 15ft
Trib width, s = 7.5ft
Floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = 0.18 x $\frac{f'c}{fy}$ = .018

Loads
slab design = 445 psf
W_{slab} = (445) x s = 3338 ppf
wall load = 20 psf
W_{wall} = (20) x h = 400 ppf

$W_{beam} = \frac{bd}{144} \times \gamma c$ = 167 ppf

Wu = W_{slab} + 1.4x(W_{wall}+W_{beam}) = 4131 ppf

Pu = $\frac{Wu(l)}{2}$ = 30981 lbs

Negative Moment Reinforcement

Neg.Moment

Mu = $\frac{Wu(l)(l)}{9} \times 12$ = 1239250 lb-in.

$bd^2 = \frac{Mu}{0.145 f'c}$ = 1424

Try
b = 10 in
h = 16 in
d = h-4 = 12 in

bd^2 = 1440

Neg.Steel Reinforcement

As = p x b x d = 2.16 in^2
Use bar No.6
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement

Pos.Moment

Mu = $\frac{Wu(l)(l)}{14} \times 12$ = 796661 lb-in.

$bd^2 = \frac{Mu}{0.145 f'c}$ = 916

Try
b = 10 in
h = 14 in
d = h-4 = 10 in

bd^2 = 1000

Pos.Steel Reinforcement
As = p x b x d = 1.8 in^2
Use 3 No.7 bars

Preliminary Concrete Column Design
Minimum Sizing Scheme

Column C1

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	4967 lbs
upper column weight	5333 lbs
P_{2nd}	41280 lbs

Shear loads

Proof	13013 lbs
P6th	18905 lbs
P5th	38947 lbs
P4th	40030 lbs
P3rd	40030 lbs
P2nd	41280 lbs
Pu	192205 lbs

$$P'u = \frac{Pu}{0.7} = 274578 \text{ lbs}$$

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 198625 lbs-in

Mu = 1437875 lbs-in

$$M'u = \frac{Mu}{0.7} = 2054107 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 7.48 \text{ in}$$

$$\frac{e}{t} = .468$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .179$$

$$K\frac{e}{t} = .0836$$

$p_t m = 0.05$

Steel Reinforcement

As = .008(b)(t) = 1.792 in^2

Use 7 No. 5 bars

Column C2

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	4967 lbs
upper column weight	4083 lbs
P_{3rd}	40030 lbs

Shear loads

Proof	13013 lbs
P6th	18905 lbs
P5th	38947 lbs
P4th	40030 lbs
P3rd	40030 lbs
Pu	150925 lbs

$$P'u = \frac{Pu}{0.7} = 215606 \text{ lbs}$$

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 198625 lbs-in

Mu = 1437875 lbs-in

$$M'u = \frac{Mu}{0.7} = 2054107 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 9.53 \text{ in}$$

$$\frac{e}{t} = .595$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .140$$

$$K\frac{e}{t} = .0836$$

$p_t m = 0.1$

Steel Reinforcement

As = .008(b)(t) = 2.176 in^2

Use 8 No. 5 bars

Column C3

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	4967 lbs
upper column weight	4083 lbs
P_{4th}	40030 lbs

Shear loads

Proof	13013 lbs
P6th	18905 lbs
P5th	38947 lbs
P4th	40030 lbs
Pu	110894 lbs

P'u = $\dfrac{Pu}{0.7}$ = 158420 lbs

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 198625 lbs-in

Mu = 1437875 lbs-in

M'u = $\dfrac{Mu}{0.7}$ = 2054107 lbs-in

Try

b = 14 in
t = 14 in
d = t-2 = 12 in

e = $\dfrac{M}{P}$ = 12.97 in

$\dfrac{e}{t}$ = .926

$\dfrac{d}{t}$ = .857

K = $\dfrac{P'u}{(b)(t)(f'c)}$ = .135

K$\dfrac{e}{t}$ = .125

$p_t m$ = 0.25

Steel Reinforcement

As = .008(b)(t) = 4.165 in^2

Use 7 No. 7 bars

Column C4

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	13013 lbs
edge beam load	3642 lbs
upper column weight	2250 lbs
P_{6th}	18905 lbs

Shear loads

Proof	13013 lbs
P6th	18905 lbs
Pu	31917 lbs

$$P'u = \frac{Pu}{0.7} = 45596 \text{ lbs}$$

Moment

edge girder moment = 520500 lbs-in
edge beam moment = 198625 lbs-in

Mu = 719125 lbs-in

$$M'u = \frac{Mu}{0.7} = 1027321 \text{ lbs-in}$$

Try

b = 12 in
t = 12 in
d = t-2 = 10 in

$$e = \frac{M}{P} = 22.53 \text{ in}$$

$$\frac{e}{t} = 1.878$$

$$\frac{d}{t} = .833$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .053$$

$$K\frac{e}{t} = .10$$

$p_t m = 0.25$

Steel Reinforcement

As = .008(b)(t) = 3.06 in^2

Use 6 No. 7 bars

Column C5

Column height = 15 ft
Unsupported length = 11 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	13013 lbs
P_{roof}	13013 lbs

Shear loads

Proof	13013 lbs
Pu	13013 lbs

$$P'u = \frac{Pu}{0.7} = 18589 \text{ lbs}$$

Moment

edge girder moment = 520500 lbs-in
edge beam moment = 198625 lbs-in

Mu = 719125 lbs-in

$$M'u = \frac{Mu}{0.7} = 1027321 \text{ lbs-in}$$

Try

b = 12 in
t = 12 in
d = t-2 = 10 in

$$e = \frac{M}{P} = 55.26\text{in}$$

$$\frac{e}{t} = 4.60$$

$$\frac{d}{t} = .833$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .021$$

$$K\frac{e}{t} = .10$$

$p_t m = 0.25$

Steel Reinforcement

As = .008(b)(t) = 3.06 in^2

Use 7 No. 6 bars

Column C6

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	9931 lbs
upper column weight	27000 lbs
P_{2nd}	287581 lbs

Shear loads

Proof	94650 lbs
P6th	117934 lbs
P5th	281915 lbs
P4th	284665 lbs
P3rd	284665 lbs
P2nd	287581 lbs
Pu	1351409 lbs

$P'u = \dfrac{Pu}{0.7} = 1930585$ lbs

Moment

interior girder moment = 40104000 lbs-in

$M'u = \dfrac{Mu}{0.7} = 57291429$ lbs-in

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$e = \dfrac{M}{P} = 29.67$ in

$\dfrac{e}{t} = .824$

$\dfrac{d}{t} = .945$

$K = \dfrac{P'u}{(b)(t)(f'c)} = .248$

$K\dfrac{e}{t} = .205$

$p_t m = 0.3$

Steel Reinforcement

$As = 0.85(p_t m)(b)(t)\dfrac{f'c}{fy} = 33.05\ in^2$

Use 16 No. 14 bars

Column C7

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	9931 lbs
upper column weight	24083 lbs
P_{3rd}	284665 lbs

Shear loads

Proof	94650 lbs
P6th	117934 lbs
P5th	281915 lbs
P4th	284665 lbs
P3rd	284665 lbs
Pu	1063828 lbs

$$P'u = \frac{Pu}{0.7} = 1519754 \text{ lbs}$$

Moment

interior girder moment = 40104000 lbs-in

$$M'u = \frac{Mu}{0.7} = 57291429 \text{ lbs-in}$$

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$$e = \frac{M}{P} = 37.70 \text{ in}$$

$$\frac{e}{t} = 1.05$$

$$\frac{d}{t} = .945$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .195$$

$$K\frac{e}{t} = .205$$

$p_t m = 0.325$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 35.8 \text{ in}^2$$

Use 16 No. 14 bars

Column C8

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	9931 lbs
upper column weight	24083 lbs
P_{4th}	284665 lbs

Shear loads

Proof	94650 lbs
P6th	117934 lbs
P5th	281915 lbs
P4th	284665 lbs
Pu	779164 lbs

$P'u = \dfrac{Pu}{0.7} = 1113091$ lbs

Moment

interior girder moment = 40104000 lbs-in

$M'u = \dfrac{Mu}{0.7} = 57291429$ lbs-in

Try

b = 34 in
t = 34 in
d = t-2 = 32 in

$e = \dfrac{M}{P} = 51.47$ in

$\dfrac{e}{t} = 1.51$

$\dfrac{d}{t} = .945$

$K = \dfrac{P'u}{(b)(t)(f'c)} = .16$

$K\dfrac{e}{t} = .242$

$p_t m = 0.45$

Steel Reinforcement

$As = 0.85(p_t m)(b)(t)\dfrac{f'c}{fy} = 44.2\ in^2$

Use 11 No. 18 bars

Column C9

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	9931 lbs
upper column weight	21333 lbs
P_{5th}	281915 lbs

Shear loads

Proof	94650 lbs
P6th	117934 lbs
P5th	281915 lbs
Pu	494499 lbs

$P'u = \dfrac{Pu}{0.7} = 706427$ lbs

Moment

interior girder moment = 40104000 lbs-in

$M'u = \dfrac{Mu}{0.7} = 57291429$ lbs-in

Try

b = 34 in
t = 34 in
d = t-2 = 32 in

$e = \dfrac{M}{P} = 81.10$ in

$\dfrac{e}{t} = 2.385$

$\dfrac{d}{t} = .941$

$K = \dfrac{P'u}{(b)(t)(f'c)} = .102$

$K\dfrac{e}{t} = .243$

$p_t m = 0.5$

Steel Reinforcement

$As = 0.85(p_t m)(b)(t)\dfrac{f'c}{fy} = 49.13\ in^2$

Use 12 No. 18 bars

Column C10

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	94650 lbs
edge beam load x 2	7284 lbs
upper column weight	16000 lbs
P_{6th}	117934 lbs

Shear loads

Proof	94650 lbs
P6th	117934 lbs
Pu	212584 lbs

$$P'u = \frac{Pu}{0.7} = 303692 \text{ lbs}$$

Moment

interior girder moment = 15144000 lbs-in

$$M'u = \frac{Mu}{0.7} = 21634286 \text{ lbs-in}$$

Try

b = 32 in
t = 32 in
d = t-2 = 30 in

$$e = \frac{M}{P} = 71.24 \text{ in}$$

$$\frac{e}{t} = 2.23$$

$$\frac{d}{t} = .938$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .049$$

$$K\frac{e}{t} = .110$$

$p_t m = 0.25$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 21.76 \text{ in}^2$$

Use 14 No. 11 bars

Column C11

Column height = 15 ft
Unsupported length = 11 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	94650 lbs
P_{roof}	94650 lbs

Shear loads

Proof	94650 lbs
Pu	94650 lbs

$$P'u = \frac{Pu}{0.7} = 135214 \text{ lbs}$$

Moment

interior girder moment = 15144000 lbs-in

$$M'u = \frac{Mu}{0.7} = 21634286 \text{ lbs-in}$$

Try

b = 32 in
t = 32 in
d = t-2 = 30 in

$$e = \frac{M}{P} = 160 \text{ in}$$

$$\frac{e}{t} = 5.0$$

$$\frac{d}{t} = .938$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .022$$

$$K\frac{e}{t} = .110$$

$p_t m = 0.25$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 21.76 \text{ in}^2$$

Use 14 No. 11 bars

Column C12

Column height = 20 ft
Unsupported length = 20 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	13013 lbs
P_{roof}	13013 lbs

Shear loads

Proof	13013 lbs
Pu	13013 lbs

$$P'u = \frac{Pu}{0.7} = 18589 \text{ lbs}$$

Moment

edge girder moment = 520500 lbs-in
edge beam moment = 198625 lbs-in

Mu = 719125 lbs-in

$$M'u = \frac{Mu}{0.7} = 1027321 \text{ lbs-in}$$

Try

b = 12 in
t = 12 in
d = t-2 = 10 in

$$e = \frac{M}{P} = 55.26\text{in}$$

$$\frac{e}{t} = 4.60$$

$$\frac{d}{t} = .833$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .021$$

$$K\frac{e}{t} = .10$$

$p_t m = 0.25$

Steel Reinforcement

As = .008(b)(t) = 3.06 in^2

Use 7 No. 6 bars

Column C13

Column height = 20 ft
Unsupported length = 20 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	94650 lbs
P_{roof}	94650 lbs

Shear loads

Proof	94650 lbs
Pu	94650 lbs

$$P'u = \frac{Pu}{0.7} = 135214 \text{ lbs}$$

Moment

interior girder moment = 15144000 lbs-in

$$M'u = \frac{Mu}{0.7} = 21634286 \text{ lbs-in}$$

Try

b = 32 in
t = 32 in
d = t-2 = 30 in

$$e = \frac{M}{P} = 160 \text{ in}$$

$$\frac{e}{t} = 5.0$$

$$\frac{d}{t} = .938$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .022$$

$$K\frac{e}{t} = .110$$

$p_t m = 0.25$

Steel Reinforcement

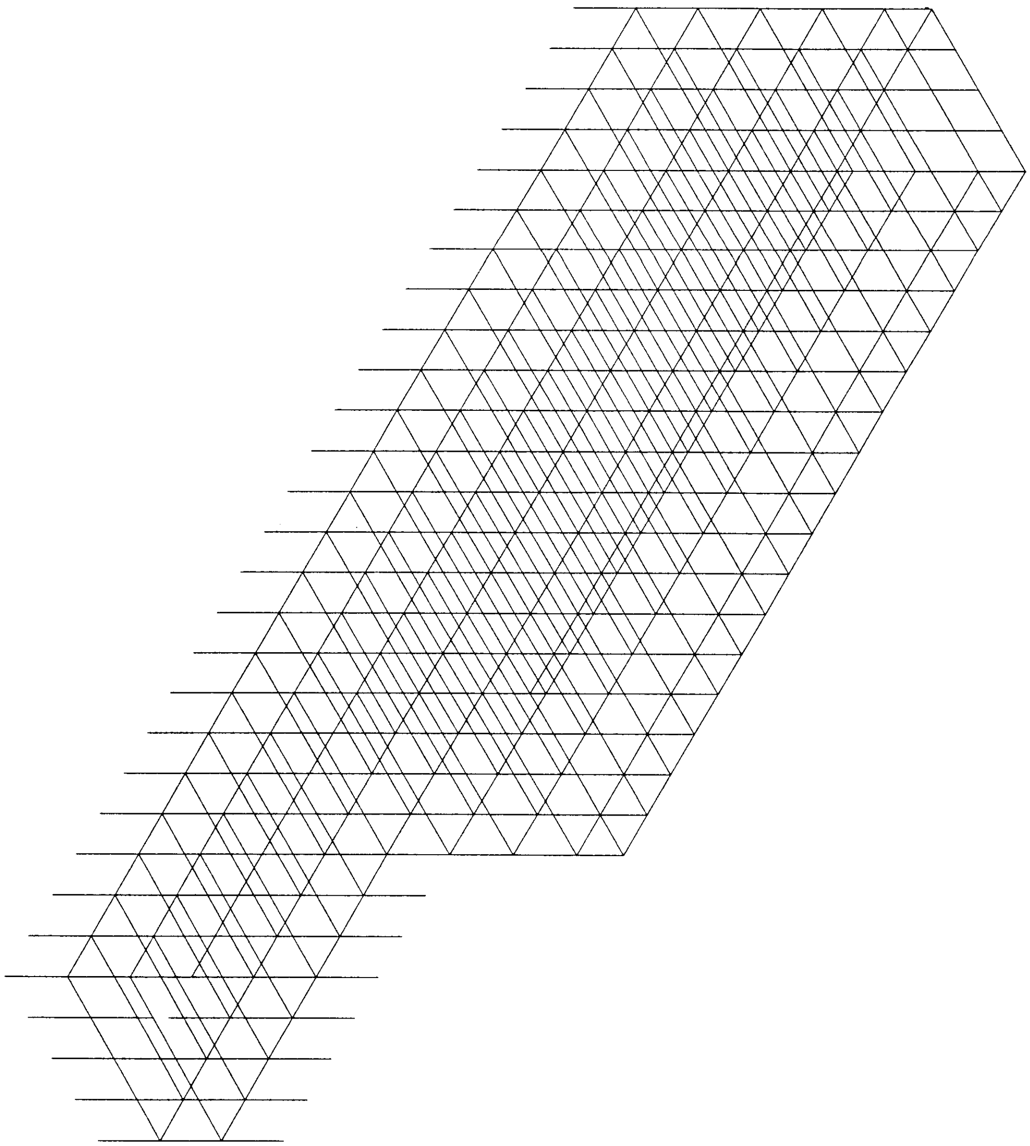
$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 21.76 \text{ in}^2$$

Use 14 No. 11 bars

APPENDIX C:

Structural Drawings and Concrete Schedule

Uniform Sizing Scheme



Isometric View

Uniform-Sizing Scheme

Paper Size: 11" x 17"

CONCRETE SCHEDULE
Uniform Sizing Scheme

Members

Member	b(in.)	d(in.)	l(ft)	Neg.As(in^2)	Neg.Reinf.	Pos.As(in^2)	Pos.Reinf.
B1	10	16	15	2.16	5No.6	0.54	3No.4
G1	24	48	60	19.01	5No.18	9.94	10No.9
G2	10	16	15	2.16	5No.6	1.26	6No.3
G3	24	48	60	19.01	5No.18	14.9	12No.10
G4	10	16	15	2.16	5No.6	1.8	3No.7

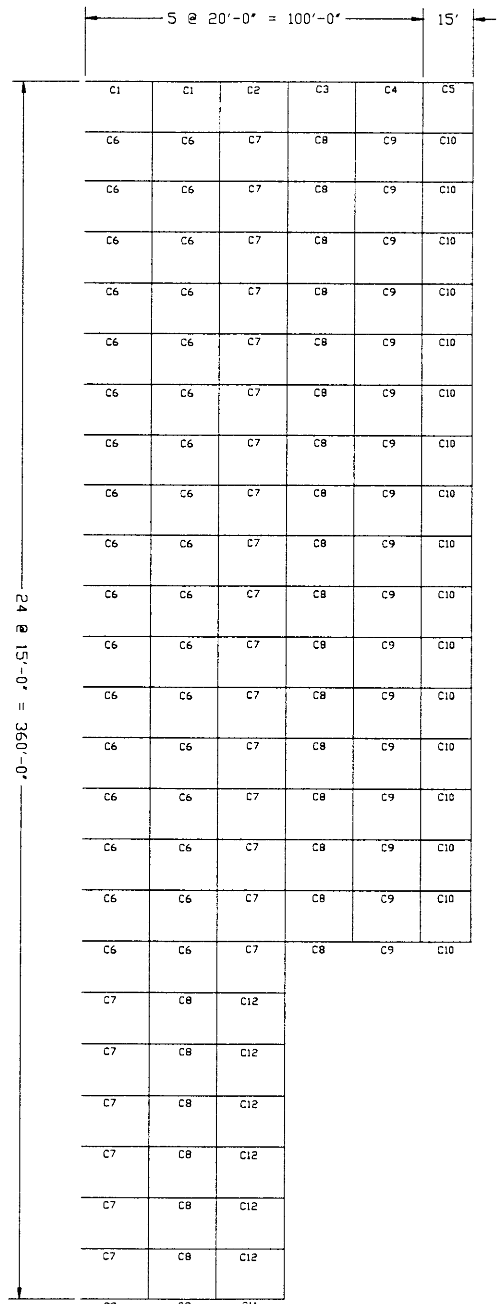
Columns

Column	b(in.)	d(in.)	h(ft)	As(in^2)	Reinf.
C1	16	16	20	1.79	7No.5
C2	16	16	20	2.72	9No.5
C3	16	16	20	3.26	8No.6
C4	16	16	20	2.72	9No.5
C5	16	16	15	2.18	5No.6
C6	36	36	20	33.05	16No.14
C7	36	36	20	38.56	10No.18
C8	36	36	20	44.06	11No.18
C9	36	36	20	49.57	10No.14
C10	36	36	15	22.03	10No.14
C11	16	16	20	2.18	5No.6
C12	36	36	20	22.03	10No.14

Slabs

Slab	t(in.)	w(ft)	l(ft)	Neg.As(in^2)	Neg.Reinf.)	Pos.As(in^2)	Pos.Reinf.	Transv.As(in^2)	Transv.Reinf.
S1	6	180	60	0.17	No.3@8in.	0.11	No.3@10in.	0.13	No.3@10in.
S2	6	180	60	0.53	No.4@4.5in.	0.34	No.4@7in.	0.13	No.3@10in.

Education Version Only
Not For Commercial Use

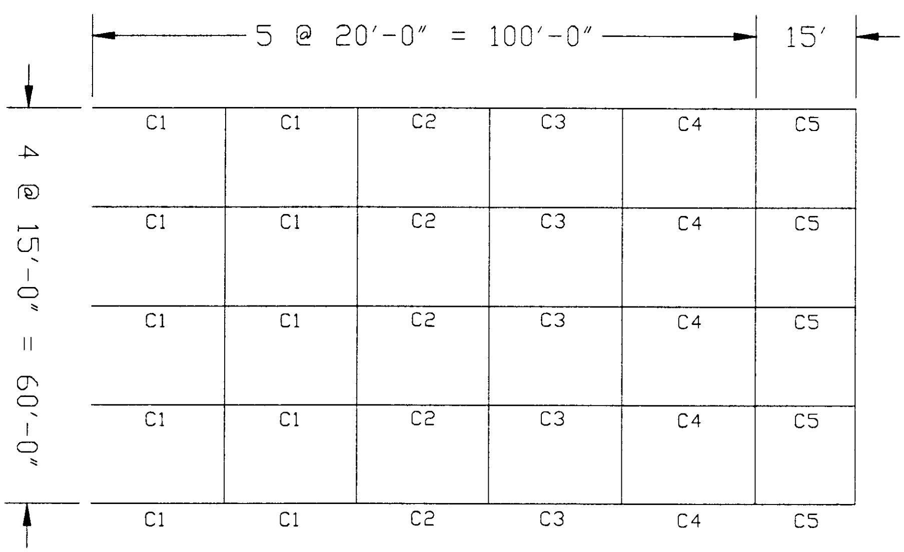


South Elevation

Scale: 1/32"=1'

Uniform-Sizing Scheme

Paper Size: 11" x 17"

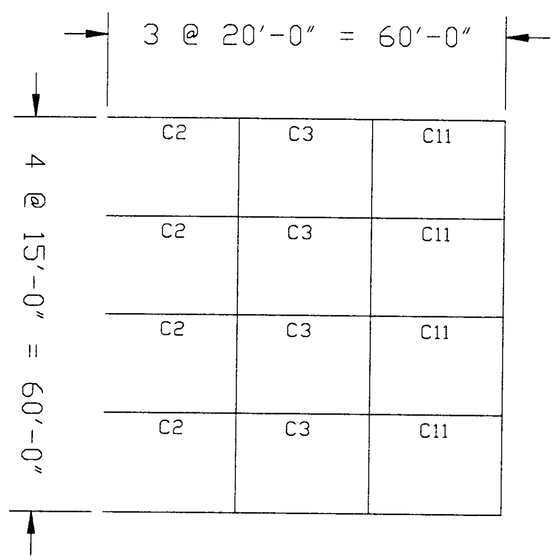


West Elevation

Scale: 1/16"=1'

Uniform-Sizing Scheme

Paper Size: 11" x 17"

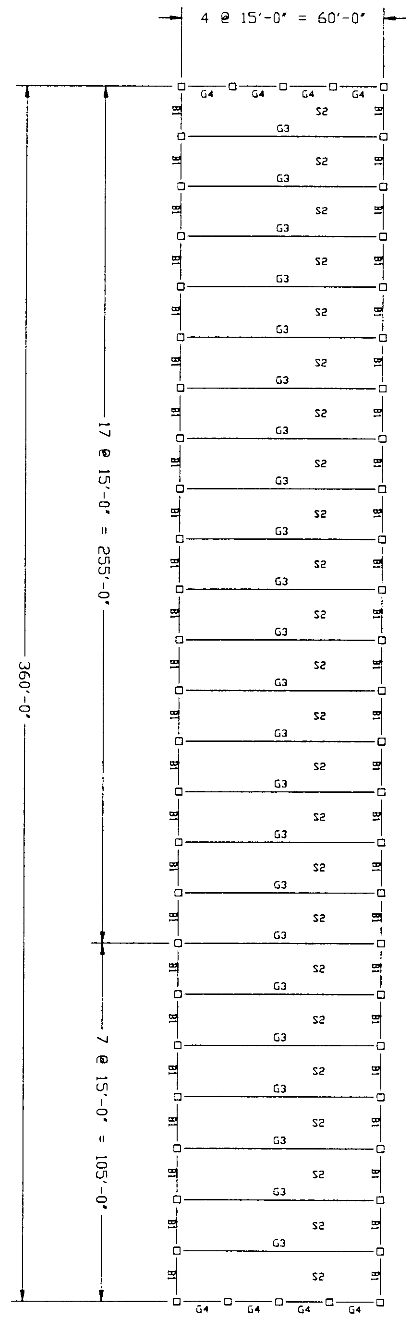


East Elevation

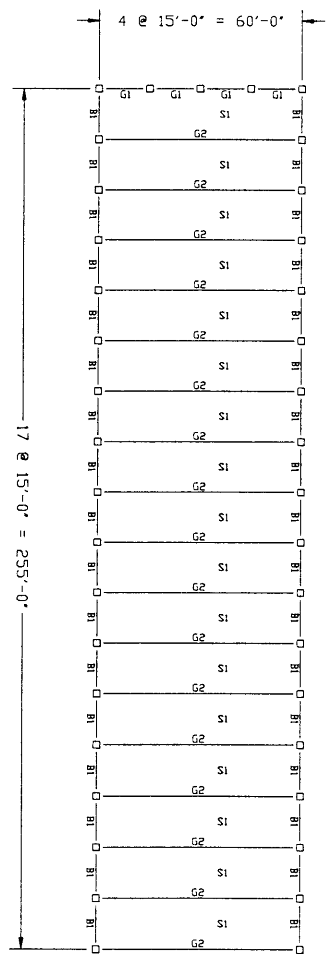
Scale: 1/16"=1'

Uniform-Sizing Scheme

Paper Size: 11" x 17"



Floor Plan	Scale: 1/32"=1'
Uniform-Sizing Scheme	Paper Size: 11" x 17"
Page: 81	



Roof Plan

Scale: 1/32"=1'

Uniform-Sizing Scheme

Paper Size: 11" x 17"

APPENDIX D:

Structural Calculations

Uniform Sizing Scheme

Preliminary Concrete One-Way Slab Design
Uniform Sizing Scheme

Slab S1

Clear span, l = 15ft
unit width = 1 ft
fy = 60 ksi
f'c = 6 ksi
γc = 150 pcf
steel ratio, p = 0.0018

Thickness
deflection control = 30

$$h = \frac{lx12}{30} = 6 \text{ in}$$

trial h = 6 in

Effective Depth
clear cover = 0.75 in
bar radius = 0.25 in
d = h x 0.75 x 0.25 = 5 in

Loads

$$\text{slab} = \frac{h}{12} \text{ x } \gamma c = 75 \text{ psf}$$

live = 10 psf
snow = 30 psf
D.L. = 1.4 x 75 = 105 psf
L.L. = 1.7 x 10 = 17 psf
S.L. = 1.7 x 30 = 51 psf

$$\text{Wu} = \frac{DL + LL + SL}{1000} = 0.173 \text{ ksf}$$

Shear

$$\text{Vu} = \frac{Wu(l)}{2} = 1.3 \text{ k/ft}$$

$$\text{vu} = \frac{Vu}{(1x12)(d)} = 0.0216 \text{ ksi}$$

$$\text{vc} = \frac{2(0.85)\sqrt{(f'c)(1000)}}{1000} = 0.13 > \text{vu ok}$$

Moment Coefficients

$$\text{positive(midspan)} = \frac{1}{14}$$

$$\text{negative(support)} = \frac{1}{9}$$

Moments

$$\text{M positive} = \frac{1}{14}(\text{Wu})(\text{l}^2)(1\text{ft}) = 2.78 \text{ ft-k}$$

$$\text{M negative} = \frac{1}{9}(\text{Wu})(\text{l}^2)(1\text{ft}) = 4.33 \text{ ft-k}$$

Negative Steel Reinf.
trial a = 1in
d-a/2 = 4.5 in

$$\text{As trial1} = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.22 \text{ in}^2$$

$$a = \frac{As(fy)}{0.85(f'c)(1x12)} = 0.21 \text{ in}$$

d-a/2 = 4.90 in
As trial2 = 0.196 in^2
a = 0.1925 in
d-a/2 = 4.90 in
As trial3 = 0.196 in^2
a = 0.19 in
use bar No.3, 7"spacing

Positive Steel Reinf.

a = 0.19 in
d-a/2 = 4.90 in

$$\text{As} = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.126 \text{ in}^2$$

use bar No.3, 10"spacing

Transverse Steel Reinf.

Ag = h x (1x12) = 72 in^2
As = p x h = 0.1296 in^2
use bar No.3, 10"spacing

Slab S2

Clear span, l = 15ft
unit width = 1 ft
fy = 60 ksi
f'c = 6 ksi
γc = 150 pcf
steel ratio, p = 0.0018

Thickness
deflection control = 30

$$h = \frac{lx12}{30} = 6 \text{ in}$$

trial h = 6 in

Loads

$$\text{slab} = \frac{h}{12} \text{ x } \gamma c = 75 \text{ psf}$$

live = 200 psf
D.L. = 1.4 x 75 = 105 psf
L.L. = 1.7 x 200 = 340 psf

$$\text{Wu} = \frac{DL + LL}{1000} = 0.445 \text{ ksf}$$

Moment Coefficients

$$\text{positive(midspan)} = \frac{1}{14}$$

$$\text{negative(support)} = \frac{1}{9}$$

Negative Steel Reinf.
trial a = 1in
d-a/2 = 4.5 in

$$\text{As trial1} = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.55 \text{ in}^2$$

$$a = \frac{As(fy)}{0.85(f'c)(1x12)} = 0.54 \text{ in}$$

d-a/2 = 4.73 in
As trial2 = 0.52 in^2
a = 0.51 in
d-a/2 = 4.74 in
As trial3 = 0.52 in^2
a = 0.51 in
use bar No.4, 4.5"spacing

Effective Depth
clear cover = 0.75 in
bar radius = 0.25 in
d = h x 0.75 x 0.25 = 5 in

Shear

$$\text{Vu} = \frac{Wu(l)}{2} \text{ k/ft}$$

$$\text{vu} = \frac{Vu}{(1x12)(d)} = 0.056 \text{ ksi}$$

$$\text{vc} = \frac{2(0.85)\sqrt{(f'c)(1000)}}{1000} = 0.13 > \text{vu ok}$$

Moments

$$\text{M positive} = \frac{1}{14}(\text{Wu})(\text{l}^2)(1\text{ft}) = 7.15 \text{ ft-k}$$

$$\text{M negative} = \frac{1}{9}(\text{Wu})(\text{l}^2)(1\text{ft}) = 11.13 \text{ ft-k}$$

Positive Steel Reinf.

a = 0.51 in
d-a/2 = 4.74 in

$$\text{As} = \frac{M(12)}{0.9(fy)(d - a/2)} = 0.335 \text{ in}^2$$

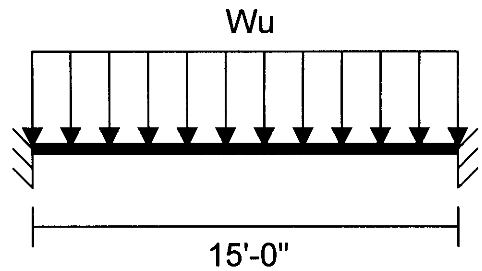
use bar No.4, 7"spacing

Transverse Steel Reinf.

Ag = h x (1x12) = 72 in^2
As = p x h = 0.1296 in^2
use bar No.3, 10"spacing

Preliminary Concrete Beam Design
Uniform Sizing Scheme

Beam B1



Beam length, l = 15ft
floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy} = .018$

Loads
wall load = 20 psf
W_{wall} = (20) x h = 400 ppf

$W_{beam} = \dfrac{bd}{144} \times \gamma c = 167$ ppf

$Wu = 1.4 \times (W_{wall} + W_{beam}) = 793$ ppf

$Pu = \dfrac{Wu(l)}{2} = 5950$ lbs

Negative Moment Reinforcement
Neg.Moment

$Mu = \dfrac{Wu(l)(l)}{9} \times 12 = 238000$ lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c} = 274$

Try
b = 10 in
h = 16 in
d = h-4 = 12 in

$bd^2 = 1440$

Neg.Steel Reinforcement

As = p x b x d = 2.16 in^2
Use bar No.6
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement
Pos.Moment

$Mu = \dfrac{Wu(l)(l)}{14} \times 12 = 153000$ lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c} = 176$

Try
b = 5 in
h = 10 in
d = h-4 = 6 in

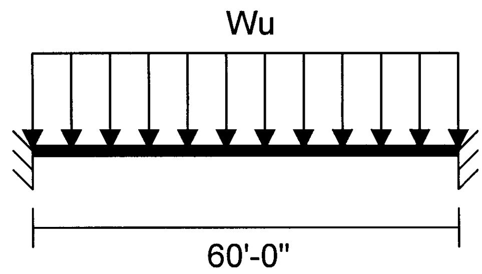
$bd^2 = 180$

Pos.Steel Reinforcement

As = p x b x d = 0.54 in^2
Use 3 No.4 bars

Preliminary Concrete Girder Design
Uniform Sizing Scheme

Girder G1



Girder length, l = 60ft
Trib width, s = 15ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy}$ = .018

Loads
slab design = 173 psf
W_{slab} = (173) x s = 6675 ppf

$W_{beam} = \dfrac{bd}{144} \times \gamma c$ = 1100 ppf

$Wu = W_{slab} + (1.4 \times W_{beam})$ = 2595 ppf

$Pu = \dfrac{Wu(l)}{2}$ = 124050 lbs

Negative Moment Reinforcement
Neg.Moment

$Mu = \dfrac{Wu(l)(l)}{9} \times 12$ = 19848000 lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c}$ = 22814

Try
b = 24 in
h = 48 in
d = h-4 = 44 in

bd^2 = 46464

Neg.Steel Reinforcement

As = p x b x d = 19.0 in^2
Use bar No.18
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement
Pos.Moment

$Mu = \dfrac{Wu(l)(l)}{14} \times 12$ = 12759429 lb-in.

$bd^2 = \dfrac{Mu}{0.145 f'c}$ = 14666

Try
b = 23 in
h = 28 in
d = h-4 = 24 in

bd^2 = 13248

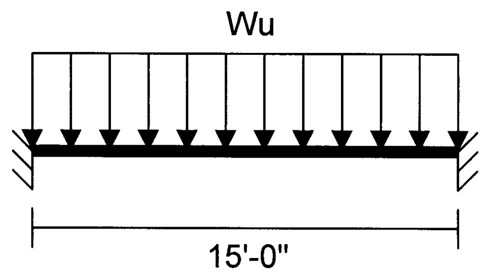
Pos.Steel Reinforcement

As = p x b x d = 9.94 in^2
Use 10 No.9 bars

Girder G2

Girder length, l = 15ft
Trib width, s = 7.5ft
Floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy} = .018$



Loads
slab design = 173 psf
W_{slab} = (173) x s = 1298 ppf
wall load = 20 psf
W_{wall} = (20) x h = 400 ppf

$$W_{beam} = \frac{bd}{144} \times \gamma c = 125 \text{ ppf}$$

$$Wu = W_{slab} + 1.4x(W_{wall}+W_{beam}) = 2033 \text{ ppf}$$

$$Pu = \frac{Wu(l)}{2} = 15244 \text{ lbs}$$

Negative Moment Reinforcement

Neg.Moment

$$Mu = \frac{Wu(l)(l)}{9} \times 12 = 609750 \text{ lb-in.}$$

$$bd^2 = \frac{Mu}{0.145 f'c} = 701$$

Try
b = 10 in
h = 16 in
d = h-4 = 12 in

bd^2 = 1440

Neg.Steel Reinforcement

As = p x b x d = 2.16 in^2
Use bar No.6
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$$Mu = \frac{Wu(l)(l)}{14} \times 12 = 391982 \text{ lb-in.}$$

$$bd^2 = \frac{Mu}{0.145 f'c} = 451$$

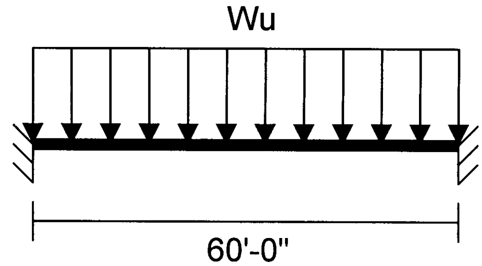
Try
b = 10 in
h = 11 in
d = h-4 = 7 in

bd^2 = 490

Pos.Steel Reinforcement

As = p x b x d = 1.26 in^2
Use 6 No.3 bars

Girder G3



Girder length, l = 60ft
Trib width, s = 15ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \frac{f'c}{fy}$ = .018

Loads
slab design = 445 psf
W_{slab} = (445) x s = 6675 ppf

$W_{beam} = \frac{bd}{144} \times \gamma c$ = 1200 ppf

$Wu = W_{slab} + (1.4 \times W_{beam})$ = 8355 ppf

$Pu = \frac{Wu(l)}{2}$ = 250650 lbs

Negative Moment Reinforcement
Neg.Moment

$Mu = \frac{Wu(l)(l)}{9} \times 12$ = 40104000 lb-in.

$bd^2 = \frac{Mu}{0.145 f'c}$ = 46097

Try
b = 24 in
h = 48 in
d = h-4 = 44 in

bd^2 = 46464

Neg.Steel Reinforcement

As = p x b x d = 19.0 in^2
Use bar No.11
13 bars near supports
4 bars in mid span

Design for Positive Reinforcement
Pos.Moment

$Mu = \frac{Wu(l)(l)}{14} \times 12$ = 25781143 lb-in.

$bd^2 = \frac{Mu}{0.145 f'c}$ = 29634

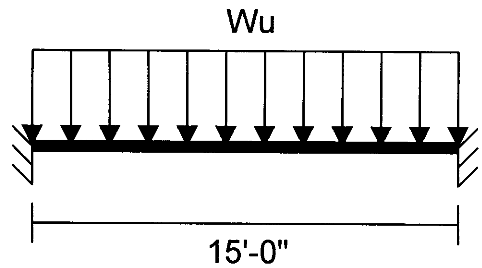
Try
b = 23 in
h = 40 in
d = h-4 = 36 in

bd^2 = 29808

Pos.Steel Reinforcement

As = p x b x d = 14.9 in^2
Use 12 No.10 bars

Girder G4



Girder length, l = 15ft
Trib width, s = 7.5ft
Floor height, h = 20ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

steel ratio, p = $0.18 \times \dfrac{f'c}{fy} = .018$

Loads
slab design = 445 psf
W_{slab} = (445) x s = 3338 ppf
wall load = 20 psf
W_{wall} = (20) x h = 400 ppf

$$W_{beam} = \frac{bd}{144} \times \gamma c = 167 \text{ ppf}$$

$$Wu = W_{slab} + 1.4x(W_{wall}+W_{beam}) = 4131 \text{ ppf}$$

$$Pu = \frac{Wu(l)}{2} = 30981 \text{ lbs}$$

Negative Moment Reinforcement

Neg.Moment

$$Mu = \frac{Wu(l)(l)}{9} \times 12 = 1239250 \text{ lb-in.}$$

$$bd^2 = \frac{Mu}{0.145 f'c} = 1424$$

Try
b = 10 in
h = 16 in
d = h-4 = 12 in

$bd^2 = 1440$

Neg.Steel Reinforcement

As = p x b x d = 2.16 in^2
Use bar No.6
5 bars near supports
2 bars in mid span

Design for Positive Reinforcement

Pos.Moment

$$Mu = \frac{Wu(l)(l)}{14} \times 12 = 796661 \text{ lb-in.}$$

$$bd^2 = \frac{Mu}{0.145 f'c} = 916$$

Try
b = 10 in
h = 14 in
d = h-4 = 10 in

$bd^2 = 1000$

Pos.Steel Reinforcement

As = p x b x d = 1.8 in^2
Use 3 No.7 bars

Preliminary Concrete Column Design
Uniform Sizing Scheme

Column C1

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	5950 lbs
upper column weight	5333 lbs
P_{2nd}	42264 lbs

Shear loads

Proof	15244 lbs
P6th	23094 lbs
P5th	42264 lbs
P4th	42264 lbs
P3rd	42264 lbs
P2nd	42264 lbs
Pu	207394 lbs

$$P'u = \frac{Pu}{0.7} = 296280 \text{ lbs}$$

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 238000 lbs-in

Mu = 1477250 lbs-in

$$M'u = \frac{Mu}{0.7} = 2110357 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 7.12 \text{ in}$$

$$\frac{e}{t} = .445$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .19$$

$$K\frac{e}{t} = .0859$$

$p_t m = 0.05$

Steel Reinforcement

As = .008(b)(t) = 1.792 in^2

Use 7 No. 5 bars

Column C2

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	950 lbs
upper column weight	5333 lbs
P_{4th}	42264 lbs

Shear loads

Proof	15244 lbs
P6th	23094 lbs
P5th	42264 lbs
P4th	42264 lbs
Pu	122866 lbs

$$P'u = \frac{Pu}{0.7} = 175524 \text{ lbs}$$

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 238000 lbs-in

Mu = 1477250 lbs-in

$$M'u = \frac{Mu}{0.7} = 2110357 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 12.02 \text{ in}$$

$$\frac{e}{t} = .751$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .114$$

$$K\frac{e}{t} = .0859$$

$p_t m = 0.125$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 2.72 \text{ in}^2$$

Use 9 No. 5 bars

Column C3

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	30981 lbs
edge beam load	5950 lbs
upper column weight	5333 lbs
P_{5th}	42264 lbs

Shear loads

Proof	15244 lbs
P6th	23094 lbs
P5th	42264 lbs
Pu	80602 lbs

$$P'u = \frac{Pu}{0.7} = 115146 \text{ lbs}$$

Moment

edge girder moment = 1239250 lbs-in
edge beam moment = 238000 lbs-in

Mu = 1477250 lbs-in

$$M'u = \frac{Mu}{0.7} = 2110357 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 18.32 \text{ in}$$

$$\frac{e}{t} = 1.15$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .075$$

$$K\frac{e}{t} = .0859$$

$p_t m = 0.15$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 3.264 \text{ in}^2$$

Use 8 No. 6 bars

Column C4

Column height = 20 ft
Unsupported length = 19 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	15244 lbs
edge beam load	3850 lbs
upper column weight	4000 lbs
P_{6th}	23094 lbs

Shear loads

Proof	15244 lbs
P6th	23094 lbs
Pu	38338 lbs

$$P'u = \frac{Pu}{0.7} = 54768 \text{ lbs}$$

Moment

edge girder moment = 609750 lbs-in
edge beam moment = 238000 lbs-in

Mu = 847750 lbs-in

$$M'u = \frac{Mu}{0.7} = 1211071 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 22.11 \text{ in}$$

$$\frac{e}{t} = 1.38$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .0356$$

$$K\frac{e}{t} = .0493$$

$p_t m = 0.125$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 2.72 \text{ in}^2$$

Use 5 No. 9 bars

Column C5

Column height = 15 ft
Unsupported length = 14 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	15244 lbs
P_{roof}	15244 lbs

Shear loads

Proof	15244 lbs
Pu	15244 lbs

$$P'u = \frac{Pu}{0.7} = 21777 \text{ lbs}$$

Moment

edge girder moment = 609750 lbs-in
edge beam moment = 238000 lbs-in

Mu = 847750 lbs-in

$$M'u = \frac{Mu}{0.7} = 1211071 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 55.6 \text{ in}$$

$$\frac{e}{t} = 3.48$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .0142$$

$$K\frac{e}{t} = .0493$$

$p_t m = 0.1$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 2.176 \text{ in}^2$$

Use 5 No. 6 bars

Column C6

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	11900 lbs
upper column weight	27000 lbs
P_{2nd}	289550 lbs

Shear loads

Proof	124050 lbs
P6th	152000 lbs
P5th	289550 lbs
P4th	289550 lbs
P3rd	289550 lbs
P2nd	289550 lbs
Pu	1434250 lbs

P'u = $\dfrac{Pu}{0.7}$ = 2048929 lbs

Moment

interior girder moment = 40104000 lbs-in

M'u = $\dfrac{Mu}{0.7}$ = 57291429 lbs-in

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

e = $\dfrac{M}{P}$ = 27.96 in

$\dfrac{e}{t}$ = .777

$\dfrac{d}{t}$ = .945

K = $\dfrac{P'u}{(b)(t)(f'c)}$ = .263

K$\dfrac{e}{t}$ = .205

$p_t m$ = 0.3

Steel Reinforcement

As = 0.85(p_tm)(b)(t)$\dfrac{f'c}{fy}$ = 33.05 in^2

Use 16 No. 14 bars

Column C7

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	11900 lbs
upper column weight	27000 lbs
P_{2nd}	289550 lbs

Shear loads

Proof	124050 lbs
P6th	152000 lbs
P5th	289550 lbs
P4th	289550 lbs
Pu	855150 lbs

$$P'u = \frac{Pu}{0.7} = 1221643 \text{ lbs}$$

Moment

interior girder moment = 40104000 lbs-in

$$M'u = \frac{Mu}{0.7} = 57291429 \text{ lbs-in}$$

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$$e = \frac{M}{P} = 46.90 \text{ in}$$

$$\frac{e}{t} = 1.303$$

$$\frac{d}{t} = .945$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .157$$

$$K\frac{e}{t} = .205$$

$p_t m = 0.35$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 38.56 \text{ in}^2$$

Use 10 No. 18 bars

Column C8

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	250650 lbs
edge beam load x 2	11900 lbs
upper column weight	27000 lbs
P_{5th}	289550 lbs

Shear loads

Proof	124050 lbs
P6th	152000 lbs
P5th	289550 lbs
Pu	565600 lbs

$$P'u = \frac{Pu}{0.7} = 808000 \text{ lbs}$$

Moment

interior girder moment = 40104000 lbs-in

$$M'u = \frac{Mu}{0.7} = 57291429 \text{ lbs-in}$$

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$$e = \frac{M}{P} = 70.9 \text{ in}$$

$$\frac{e}{t} = 1.97$$

$$\frac{d}{t} = .945$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .104$$

$$K\frac{e}{t} = .205$$

$p_t m = 0.4$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 44.06 \text{ in}^2$$

Use 11 No. 18 bars

Column C9

Column height = 20 ft
Unsupported length = 16 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	124050 lbs
edge beam load x 2	7700 lbs
upper column weight	20250 lbs
P_{6th}	152000 lbs

Shear loads

Proof	124050 lbs
P6th	152000 lbs
Pu	276050 lbs

$P'u = \dfrac{Pu}{0.7} = 394357$ lbs

Moment

interior girder moment = 19848000 lbs-in

$M'u = \dfrac{Mu}{0.7} = 28354286$ lbs-in

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$e = \dfrac{M}{P} = 71.9$ in

$\dfrac{e}{t} = 2.0$

$\dfrac{d}{t} = .945$

$K = \dfrac{P'u}{(b)(t)(f'c)} = .0507$

$K\dfrac{e}{t} = .1013$

$p_t m = 0.2$

Steel Reinforcement

$As = 0.85(p_t m)(b)(t)\dfrac{f'c}{fy} = 22.03\ in^2$

Use 10 No. 14 bars

Column C10

Column height = 15 ft
Unsupported length = 11 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	124050 lbs
P_{roof}	124050 lbs

Shear loads

Proof	124050 lbs
Pu	124050 lbs

$$P'u = \frac{Pu}{0.7} = 177214 \text{ lbs}$$

Moment

interior girder moment = 19848000 lbs-in

$$M'u = \frac{Mu}{0.7} = 28354286 \text{ lbs-in}$$

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$$e = \frac{M}{P} = 160 \text{ in}$$

$$\frac{e}{t} = 4.44$$

$$\frac{d}{t} = .945$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .023$$

$$K\frac{e}{t} = .1013$$

$p_t m = 0.2$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 22.032 \text{ in}^2$$

Use 10 No. 14 bars

Column C11

Column height = 20 ft
Unsupported length = 20 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

edge girder load	15244 lbs
P_{roof}	15244 lbs

Shear loads

Proof	15244 lbs
Pu	15244 lbs

$$P'u = \frac{Pu}{0.7} = 21777 \text{ lbs}$$

Moment

edge girder moment = 609750 lbs-in
edge beam moment = 238000 lbs-in

Mu = 847750 lbs-in

$$M'u = \frac{Mu}{0.7} = 1211071 \text{ lbs-in}$$

Try

b = 16 in
t = 16 in
d = t-2 = 14 in

$$e = \frac{M}{P} = 55.6 \text{ in}$$

$$\frac{e}{t} = 3.48$$

$$\frac{d}{t} = .875$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .0142$$

$$K\frac{e}{t} = .0493$$

$p_tm = 0.1$

Steel Reinforcement

$$As = 0.85(p_tm)(b)(t)\frac{f'c}{fy} = 2.18 \text{ in}^2$$

Use 5 No. 6 bars

Column C12

Column height = 20 ft
Unsupported length = 20 ft
fy = 60000 psi
f'c = 6000 psi
γc = 150 pcf

Floor loads

interior girder load	124050 lbs
P_{roof}	124050 lbs

Shear loads

Proof	124050 lbs
Pu	124050 lbs

$$P'u = \frac{Pu}{0.7} = 177214 \text{ lbs}$$

Moment

interior girder moment = 19848000 lbs-in

$$M'u = \frac{Mu}{0.7} = 28354286 \text{ lbs-in}$$

Try

b = 36 in
t = 36 in
d = t-2 = 34 in

$$e = \frac{M}{P} = 160 \text{ in}$$

$$\frac{e}{t} = 4.44$$

$$\frac{d}{t} = .945$$

$$K = \frac{P'u}{(b)(t)(f'c)} = .023$$

$$K\frac{e}{t} = .1013$$

$p_t m = 0.2$

Steel Reinforcement

$$As = 0.85(p_t m)(b)(t)\frac{f'c}{fy} = 22.0 \text{ in}^2$$

Use 10 No. 14 bars

APPENDIX E:

Quantities and Cost Estimations

Minimum Sizing Scheme

MATERIAL QUANTITIES
Minimum Sizing Scheme

Members

Member	reinf.(lbs/memb.)	# of members	total reinf.(lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
B1	40	34	1360	136	0.75	5.90
B2	57	164	9348	935	5.14	44.29
G1	3740	34	127160	12716	69.94	402.96
G2	100	8	800	80	0.44	2.60
G3	5818	80	465440	46544	255.99	1422.22
G4	167	24	4008	401	2.20	14.81

Columns

Column	reinf.(lbs/memb.)	# of members	total reinf.(lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
C1	146	5	730	73	0.40	6.58
C2	167	5	835	84	0.46	6.58
C3	286	20	5720	572	3.15	20.16
C4	245	5	1225	123	0.67	3.70
C5	158	5	790	79	0.43	2.78
C6	2448	34	83232	8323	45.78	226.67
C7	2448	34	83232	8323	45.78	226.67
C8	2992	46	137632	13763	75.70	273.54
C9	3264	46	150144	15014	82.58	273.54
C10	1488	34	50592	5059	27.83	179.09
C11	1116	34	37944	3794	20.87	134.32
C12	158	5	790	79	0.43	3.70
C13	1116	12	13392	1339	7.37	63.21

Slabs

Slab	reinf.(lbs/memb.)	# of members	total reinf.(lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
S1	2011	68	67864	6786	37.33	1133.34
S2	2239	82	183598	18360	100.98	1366.67

TOTAL STEEL(TONS)	TOTAL CONC.(CY)
784.21	5813.33

Contact Areas (SFCA)
Square Footage (SF)
Minimum Sizing Scheme

Members	b (in)	d (in)	l (ft)	CA/memb.	#of members	CA total
B1	5	9	15	29	34	978
G2	6	14	15	43	8	340
B2	7	10	15	34	164	5,535
G4	10	16	15	53	24	1,260
G1	24	32	60	440	34	14,960
G3	24	48	60	600	80	48,000

Columns	b (in)	d (in)	l (ft)	CA/memb.	#of members	CA total
C4,C5,C12	12	12	20	80	15	1,200
C3	14	14	20	93	20	1,867
C1,C2	16	16	20	107	10	1,067
C10,C11,C13	32	32	20	213	80	17,067
C8,C9	34	34	20	227	92	20,853
C6,C7	36	36	20	240	68	16,320

Slabs	t (in)	w (ft)	l (ft)	SF/memb.	#of members	SF total
S1,S2	180	6	60	900	150	135,000

CODE	ITEM	UNIT	BARE COSTS $				TOTAL
			MAT.	LABOR	EQUIP.	TOTAL	INCL O&P
031	**CONCRETE FORMWORK**						
031 100	**Struct CIP Formwork**						
031 138	**Forms In Place, Beams and Girders**						
031 138 0500	Exterior spandrel, 5"wide, 1 use	SFCA	3.19	10.32	0.31	13.82	20.28
031 138 0550	Exterior spandrel, 6"wide, 2 use	SFCA	2.66	8.60	0.26	11.52	16.90
031 138 0550	Exterior spandrel, 7"wide, 2 use	SFCA	2.28	7.37	0.22	9.87	14.49
031 138 0550	Exterior spandrel, 10"wide, 2 use	SFCA	1.60	5.16	0.16	6.91	10.14
031 138 2550	Interior beam, 24"wide, 2 use	SFCA	1.16	3.24	0.10	4.50	6.55
031 138 2550	Interior beam, 24"wide, 2 use	SFCA	1.16	3.24	0.10	4.50	6.55
031 142	**Forms In Place, Columns**						
031 142 5550	12"x12" plywood, 2 use	SFCA	1.27	3.63	0.13	5.03	7.35
031 142 6050	14"x14" plywood, 2 use	SFCA	1.23	3.59	0.13	4.95	7.23
031 142 6050	16"x16" plywood, 2 use	SFCA	1.18	3.55	0.13	4.86	7.10
031 142 7050	32"x32" plywood, 2 use	SFCA	1.52	3.74	0.14	5.39	7.76
031 142 7050	34"x34" plywood, 2 use	SFCA	1.43	3.52	0.13	5.07	7.31
031 142 7050	36"x36" plywood, 2 use	SFCA	1.35	3.32	0.12	4.79	6.90
031 150	**Forms In Place, Elev.Slab**						
031 150 1150	Flat plate, 4 use	S.F.	0.76	2.11	0.07	2.94	4.28
032	**CONCRETE REINFORCEMENT**						
032 100	**Reinforcing Steel**						
032 107	**Reinforcing In Place A615 Grade 60**						
032 107 0100	Beams and Girders, #3 to #7	Ton	520.00	555.00		1,075.00	1,550.00
032 107 0150	Beams and Girders, #8 to #18	Ton	510.00	330.00		840.00	1,150.00
032 107 0200	Columns, #3 to #7	Ton	520.00	590.00		1,110.00	1,625.00
032 107 0250	Columns, #8 to #18	Ton	510.00	385.00		895.00	1,250.00
032 107 0400	Elevated slabs, #4 to #7	Ton	550.00	305.00		855.00	1,150.00
032 107 2210	Crane Cost, Average	Ton		16.45	5.60	22.05	34.00
033	**CAST-IN-PLACE CONCRETE**						
033 100	**Structural Concrete**						
033 126	**Concrete, Ready Mix**						
033 126 0411	6000 psi	C.Y.	65.00			65.00	71.50
033 134	**Curing**						
033 134 0300	with sprayed membrane curing compound	C.S.F.	2.05	3.33		5.38	7.55
033 172	**Placing Concrete** and vibrating, including labor and equipment						
033 172 0200	large beams, w/crane and bucket	C.Y.		21.00	13.20	34.20	47.50
033 172 0450	columns, 12", w/crane and bucket	C.Y.		34.00	21.50	55.50	77.50
033 172 0450	columns, 14", w/crane and bucket	C.Y.		34.00	21.50	55.50	77.50
033 172 0650	columns, 16", w/crane and bucket	C.Y.		25.00	15.60	40.60	56.00
033 172 1050	columns, 32", w/crane and bucket	C.Y.		13.65	8.60	22.25	31.00
033 172 1050	columns, 34", w/crane and bucket	C.Y.		13.65	8.60	22.25	31.00
033 172 1050	columns, 36", w/crane and bucket	C.Y.		13.65	8.60	22.25	31.00
033 172 1450	elevated slab, 6", w/crane and bucket	C.Y.		14.40	9.05	23.45	32.50
033 450	**Concrete Finishing**						
033 454	**Finishing Floors**						
033 454 0010	Monolithic, screed finish	S.F.		0.22		0.22	0.33

COST ESTIMATE
Minimum Sizing Scheme

CODE	ITEM	QUANTITY	UNIT	BARE COSTS $				TOTAL INCL O&P
				MAT.	LABOR	EQUIP.	TOTAL	
031	**CONCRETE FORMWORK**							
031 100	**Struct CIP Formwork**							
031 138	**Forms In Place, Beams and Girders**							
031 138 0500	Exterior spandrel, 5"wide, 1 use	978.00	SFCA	3,119.82	10,092.96	303.18	13,515.96	19,833.84
031 138 0550	Exterior spandrel, 6"wide, 2 use	170.00	SFCA	452.20	1,462.00	44.20	1,958.40	2,873.00
031 138 0550	Exterior spandrel, 7"wide, 2 use	2,768.00	SFCA	6,311.04	20,400.16	608.96	27,320.16	40,108.32
031 138 0550	Exterior spandrel, 10"wide, 2 use	630.00	SFCA	1,008.00	3,250.80	100.80	4,353.30	6,388.20
031 138 2550	Interior beam, 24"wide, 2 use	7,480.00	SFCA	8,676.80	24,235.20	748.00	33,660.00	48,994.00
031 138 2550	Interior beam, 24"wide, 2 use	24,000.00	SFCA	27,840.00	77,760.00	2,400.00	108,000.00	157,200.00
031 142	**Forms In Place, Columns**							
031 142 5550	12"x12" plywood, 2 use	600.00	SFCA	762.00	2,178.00	78.00	3,018.00	4,410.00
031 142 6050	14"x14" plywood, 2 use	933.00	SFCA	1,147.59	3,349.47	121.29	4,618.35	6,745.59
031 142 6050	16"x16" plywood, 2 use	533.00	SFCA	628.94	1,892.15	69.29	2,590.38	3,784.30
031 142 7050	32"x32" plywood, 2 use	8,533.00	SFCA	12,970.16	31,913.42	1,194.62	45,992.87	66,216.08
031 142 7050	34"x34" plywood, 2 use	10,427.00	SFCA	14,910.61	36,703.04	1,355.51	52,864.89	76,221.37
031 142 7050	36"x36" plywood, 2 use	8,160.00	SFCA	11,016.00	27,091.20	979.20	39,086.40	56,304.00
031 150	**Forms In Place, Elev.Slab**							
031 150 1150	Flat plate, 4 use	45,000.00	S.F.	34,200.00	94,950.00	3,150.00	132,300.00	192,600.00
			Total =	123,043.16	335,278.40	11,153.05	469,278.71	681,678.70
032	**CONCRETE REINFORCEMENT**							
032 100	**Reinforcing Steel**							
032 107	**Reinforcing In Place A615 Grade 60**							
032 107 0100	Beams and Girders, #3 to #7	8.53	Ton	4,437.68	4,736.37		9,174.05	13,227.70
032 107 0150	Beams and Girders, #8 to #18	325.93	Ton	166,224.30	107,556.90		273,781.20	374,819.50
032 107 0200	Columns, #3 to #7	5.55	Ton	2,886.00	3,274.50		6,160.50	9,018.75
032 107 0250	Columns, #8 to #18	305.89	Ton	156,004.92	117,768.42		273,773.34	382,365.00
032 107 0400	Elevated slabs, #4 to #7	138.30	Ton	76,067.20	42,182.72		118,249.92	159,049.60
032 107 2210	Crane Cost, Average	784.21	Ton		12,900.25	4,391.58	17,291.83	26,663.14
			Total =	405,620.10	288,419.16	4,391.58	698,430.84	965,143.69
033	**CAST-IN-PLACE CONCRETE**							
033 100	**Structural Concrete**							
033 126	**Concrete, Ready Mix**							
033 126 0411	6000 psi	5,813.00	C.Y.	377,845.00			377,845.00	415,629.50
033 134	**Curing**							
033 134 0300	with sprayed membrane curing compound	2,801.33	C.S.F.	5,742.73	9,328.43		15,071.16	21,150.04
033 172	**Placing Concrete and vibrating, including labor and equipment**							
033 172 0200	large beams, w/crane and bucket	1,893.00	C.Y.		39,753.00	24,987.60	64,740.60	89,917.50
033 172 0650	columns, 12", w/crane and bucket	10.18	C.Y.		346.12	218.87	564.99	788.95
033 172 0650	columns, 14", w/crane and bucket	20.16	C.Y.		685.44	433.44	1,118.88	1,562.40
033 172 0650	columns, 16", w/crane and bucket	13.16	C.Y.		329.00	205.30	534.30	736.96
033 172 1050	columns, 32", w/crane and bucket	376.62	C.Y.		5,140.86	3,238.93	8,379.80	11,675.22
033 172 1050	columns, 34", w/crane and bucket	547.08	C.Y.		7,467.64	4,704.89	12,172.53	16,959.48
033 172 1050	columns, 36", w/crane and bucket	453.34	C.Y.		6,188.09	3,898.72	10,086.82	14,053.54
033 172 1450	elevated slab, 6", w/crane and bucket	2,500.00	C.Y.		36,000.00	22,625.00	58,625.00	81,250.00
033 450	**Concrete Finishing**							
033 454	**Finishing Floors**							
033 454 0010	Monolithic, screed finish	135,000.00	S.F.		29,700.00		29,700.00	44,550.00
			Total =	383,587.73	134,938.58	60,312.75	578,839.06	698,273.59
			TOTAL =	912,250.99	758,636.15	75,857.38	1,746,548.61	2,345,095.98

APPENDIX F:

Quantities and Cost Estimations

Uniform Sizing Scheme

MATERIAL QUANTITIES
Uniform Sizing Scheme

Members

Member	reinf. (lbs/memb.)	# of members	total reinf. (lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
B1	105	198	20790	2079	11.43	122.22
G1	4760	34	161840	16184	89.01	604.44
G2	109	8	872	87.2	0.48	4.94
G3	5818	80	465440	46544	255.99	1422.22
G4	167	24	4008	400.8	2.20	14.81

Columns

Column	reinf. (lbs/memb.)	# of members	total reinf. (lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
C1	146	10	1460	146	0.80	13.16
C2	188	10	1880	188	1.03	13.16
C3	240	10	2400	240	1.32	13.16
C4	188	5	940	94	0.52	6.58
C5	113	5	565	57	0.31	4.94
C6	2448	68	166464	16646	91.56	453.34
C7	2720	46	125120	12512	68.82	306.67
C8	2992	46	137632	13763	75.70	306.67
C9	1530	34	52020	5202	28.61	226.67
C10	1530	34	52020	5202	28.61	170.00
C11	150	5	750	75	0.41	6.58
C12	1530	12	18360	1836	10.10	80.00

Slabs

Slab	reinf. (lbs/memb.)	# of members	total reinf. (lbs)	ties (lbs)	total steel (tons)	total conc (c.y.)
S1	1996	68	135728	13573	74.65	1133.34
S2	2239	82	183598	18360	100.98	1366.67

TOTAL STEEL(TONS)	TOTAL CONC.(CY)
842.54	6269.57

Contact Area (SFCA)
Square Footage (SF)
Uniform Sizing Scheme

Member	b (in)	d (in)	l (ft)	CA/memb.	#of members	CA total
B1,G2,G4	10	16	15	53	230	12075
G1,G3	24	48	60	600	114	68400

Columns	b (in)	d (in)	l (ft)	CA/memb.	#of members	CA total
C1-C5,C11	16	16	20	107	45	4800
C6-C10,C12	36	36	20	240	240	57600

Slabs	t (in)	w (ft)	l (ft)	SF/memb.	#of members	SF total
S1,S2	180	6	60	900	150	135000

UNIT COSTS
Uniform Sizing Scheme

CODE	ITEM	UNIT	BARE COSTS $				TOTAL
			MAT.	LABOR	EQUIP.	TOTAL	INCL O&P
031	**CONCRETE FORMWORK**						
031 100	**Struct CIP Formwork**						
031 138	**Forms In Place, Beams and Girders**						
031 138 0650	Exterior spandrel, 10"wide, 4 use	SFCA	1.00	3.81	0.12	4.93	7.35
031 138 2650	Interior beam, 24"wide, 4 use	SFCA	0.86	2.99	0.09	3.94	5.80
031 142	**Forms In Place, Columns**						
031 142 6150	16"x16" plywood, 4 use	SFCA	0.80	3.25	0.12	4.17	6.20
031 142 7150	36"x36" plywood, 4 use	SFCA	0.96	3.05	0.11	4.12	6.05
031 150	**Forms In Place, Elev.Slab**						
031 150 1150	Flat plate, 4 use	S.F.	0.76	2.11	0.07	2.94	4.28
032	**CONCRETE REINFORCEMENT**						
032 100	**Reinforcing Steel**						
032 107	**Reinforcing In Place A615 Grade 60**						
032 107 0100	Beams and Girders, #3 to #7	Ton	520.00	555.00		1,075.00	1,550.00
032 107 0150	Beams and Girders, #8 to #18	Ton	510.00	330.00		840.00	1,150.00
032 107 0200	Columns, #3 to #7	Ton	520.00	590.00		1,110.00	1,625.00
032 107 0250	Columns, #8 to #18	Ton	510.00	385.00		895.00	1,250.00
032 107 0400	Elevated slabs, #4 to #7	Ton	550.00	305.00		855.00	1,150.00
032 107 2210	Crane Cost, Average	Ton		16.45	5.60	22.05	34.00
033	**CAST-IN-PLACE CONCRETE**						
033 100	**Structural Concrete**						
033 126	**Concrete, Ready Mix**						
033 126 0411	6000 psi	C.Y.	65.00			65.00	71.50
033 134	**Curing**						
033 134 0300	with sprayed membrane curing compound	C.S.F.	2.05	3.33		5.38	7.55
033 172	**Placing Concrete** and vibrating, including labor and equipment						
033 172 0200	large beams, w/crane and bucket	C.Y.		21.00	13.20	34.20	47.50
033 172 0650	columns, 16", w/crane and bucket	C.Y.		25.00	15.60	40.60	56.00
033 172 1050	columns, 36", w/crane and bucket	C.Y.		13.65	8.60	22.25	31.00
033 172 1450	elevated slab, 6", w/crane and bucket	C.Y.		14.40	9.05	23.45	32.50
033 450	**Concrete Finishing**						
033 454	**Finishing Floors**						
033 454 0010	Monolithic, screed finish	S.F.		0.22		0.22	0.33

COST ESTIMATE
Uniform Sizing Scheme

CODE	ITEM	QUANTITY	UNIT	BARE COSTS $				TOTAL INCL O&P
				MAT.	LABOR	EQUIP.	TOTAL	
031	**CONCRETE FORMWORK**							
031 100	**Struct CIP Formwork**							
031 138	**Forms In Place, Beams and Girders**							
031 138 0650	Exterior spandrel, 10"wide, 4 use	4,025.00	SFCA	4,025.00	15,335.25	483.00	19,843.25	29,583.75
031 138 2650	Interior beam, 24"wide, 4 use	22,800.00	SFCA	19,608.00	68,172.00	2,052.00	89,832.00	132,240.00
031 142	**Forms In Place, Columns**							
031 142 6150	16"x16" plywood, 4 use	1,600.00	SFCA	1,280.00	5,200.00	192.00	6,672.00	9,920.00
031 142 7150	36"x36" plywood, 4 use	19,200.00	SFCA	18,432.00	58,560.00	2,112.00	79,104.00	116,160.00
031 150	**Forms In Place, Elev.Slab**							
031 150 1150	Flat plate, 4 use	45,000.00	S.F.	34,200.00	94,950.00	3,150.00	132,300.00	192,600.00
			Total =	77,545.00	242,217.25	7,989.00	327,751.25	480,503.75
032	**CONCRETE REINFORCEMENT**							
032 100	**Reinforcing Steel**							
032 107	**Reinforcing In Place A615 Grade 60**							
032 107 0100	Beams and Girders, #3 to #7	14.12	Ton	7,341.62	7,835.77		15,177.39	21,883.68
032 107 0150	Beams and Girders, #8 to #18	345.00	Ton	175,952.04	113,851.32		289,803.36	396,754.60
032 107 0200	Columns, #3 to #7	4.40	Ton	2,286.70	2,594.53		4,881.23	7,145.94
032 107 0250	Columns, #8 to #18	303.39	Ton	154,728.14	116,804.57		271,532.71	379,235.63
032 107 0400	Elevated slabs, #4 to #7	138.30	Ton	76,067.20	42,182.72		118,249.92	159,049.60
032 107 2210	Crane Cost, Average	805.21	Ton		13,245.75	4,509.19	17,754.94	27,377.23
			Total =	416,375.70	296,514.65	4,509.19	717,399.54	991,446.66
033	**CAST-IN-PLACE CONCRETE**							
033 100	**Structural Concrete**							
033 126	**Concrete, Ready Mix**							
033 126 0411	6000 psi	6,270.00	C.Y.	407,550.00			407,550.00	448,305.00
033 134	**Curing**							
033 134 0300	with sprayed membrane curing compound	2,944.30	C.S.F.	6,035.82	9,804.52		15,840.33	22,229.47
033 172	**Placing Concrete** and vibrating, including labor and equipment							
033 172 0200	large beams, w/crane and bucket	2,168.63	C.Y.		45,541.23	28,625.92	74,167.15	103,009.93
033 172 0650	columns, 16", w/crane and bucket	57.58	C.Y.		1,439.50	898.25	2,337.75	3,224.48
033 172 1050	columns, 36", w/crane and bucket	1,543.35	C.Y.		21,066.73	13,272.81	34,339.54	47,843.85
033 172 1450	elevated slab, 6", w/crane and bucket	2,500.00	C.Y.		36,000.00	22,625.00	58,625.00	81,250.00
033 450	**Concrete Finishing**							
033 454	**Finishing Floors**							
033 454 0010	Monolithic, screed finish	135,000.00	S.F.		29,700.00		29,700.00	44,550.00
			Total =	413,585.82	143,551.98	65,421.97	622,559.77	750,412.72
			TOTAL =	907,506.51	682,283.88	77,920.16	1,667,710.55	2,222,363.13

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