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Correctional Facility Analysis and Design

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3. cost analysis

Authorship

The written works in this project were a wholly collaborative effort with the exception of the foundation design, exterior wall design, fire considerations and site design chapters. The preceding chapters were authored as such:

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Abstract

This project investigated the design and analysis of various structural systems for a correctional facility, including an administrative office and detention center. Structural design using parametric analysis of alternative steel and reinforced concrete structural systems were conducted for gravity and lateral load resisting systems using IBC 2003. Performance analyses were generated based upon economy, feasibility, constructability, and serviceability. Additional study topics included foundation design, exterior wall design, fire considerations, and site design.

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1 Introduction

Throughout the course of human history, there have always been those who, whether by inner compulsion or extenuating circumstances, break the rules of society. When these infractions are severe or occur numerous times, it becomes necessary to separate those who cannot or will not function within the bounds of society from those who can. Those people who exhibit deviant behavior are contained within what are called correctional facilities, or more commonly prisons or jails.

A correctional facility is a compound designed with five major purposes in mind.[1] The first is punishment, with the prison itself intended to "'cure' the former (prisoner) of, and frighten the latter (general public) from, criminal behavior."[1] The second purpose is protecting the general public from those who would wish to do harm. Third is the aspect of reform of the prisoner. This means that the building or buildings that make up the complex have space allocated for education, solitary confinement, social training and the like. The fourth purpose is to attempt to cure the prisoner in terms of mental afflictions that may drive his or her deviant behavior through therapy or group activities. The final purpose a correctional facility can serve is to provide "a public statement of moral, political, and social virtue."[1] Thus, institutional characteristics play a critical role in creating an environment of incarceration and rehabilitation without causing a sense of oppression and hopelessness.

These five purposes of incarceration can be met by a variety of different layouts depending upon the type of prisoner who is being held, and where the facility is located.

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[1] Spens, p. 16

There are five different security levels for prisons.[2] The first is the minimum-security prison, which is also known at the Federal Prison Camp (FPC). This type of prison features dormitory style housing and tends to be located close to higher security prisons and can provide the labor needed to run those facilities. The second type of prison is the low security prison. These types of facilities also feature dormitory style housing or cubicle housing and have strong work related components. Medium security prisons have cell type housing, increased security measures and greater internal controls. High security prisons have single and multiple cell housing and extremely controlled internal environments. The fifth type of correctional facility melds the various levels of classification in a single complex. Prisons can be located in any type of setting, whether it be urban or rural. The main difference between the two localities is the design of the structure. Due to spatial constraints, urban correctional facilities tend to have more of a high-rise design whereas rural correctional facilities can be more sprawling.

Over the past 20 years, the population of prisoners housed in correctional facilities has increased dramatically. Due to this population increase, a multitude of prisons on the county, state and federal level are facing overcrowding issues. In fact, between the years of 1995 and 2002, state prison populations increased 23.6 percent and federal prison populations by 71 percent.[3] Additionally, it has been estimated that between 1 to 16 percent of state prisons are over capacity and 33 percent of federal prisons are in the same condition.[3]

As a result of overcrowding, correctional facilities across the country are feeling increased strain on their infrastructure. The pressures of overcrowding affect each

[2] Federal Bureau of Prisons – General Information
[3] Gibeaut, p. 53

facility's ability to operate effectively. With each additional inmate beyond the planned capacity, resources such as food services, education, and rehabilitation, as well as cleanliness and health concerns are compromised. Because of overcrowding and its effects, the goals of the correctional facility to rehabilitate inmates are less likely to be met.

Research has stated that:

> The prison environment is characterized by factors which can have adverse effects on individual inmates. In the prison setting crowded conditions are chronic, people prone to anti-social behavior are gathered, there is an absence of personal control and idleness and boredom can be prevalent. Research has indicated that overcrowding has three types of effects on the daily prison environment. First, there is less of everything to go around, so the same space and resources are made to stretch even further. The opportunities for academic, employment and vocational training are curtailed. The lack of work or work opportunities lead to idleness, often reinforcing the maxim that idleness breeds discontent and disruptive behavior (Cox, Paulus, & McCain, 1984, p.1149). In addition, lack of resources can apply to anything an inmate might need to use, such as washroom availability, library books, television lounge seating and recreational materials. The unavailability of resources can have two-fold consequences. One is the frustration of unpleasantness of being limited or denied a resource and the other is the fact that competition or conflict over limited resources often lead to aggression and violence (Johnson, 1991, p. 19).[4]

With these considerations in mind, the project presented herein sought to develop the design of a hypothetical correctional facility located in the city of Greenfield, Massachusetts (site selection to be detailed in Chapter 8 Site Design). The major focus on this particular project was the structural design of the facility using two different construction materials, reinforced concrete and structural steel and evaluating each option in terms of cost and constructability. The correctional facility as a whole featured a

[4] John Howard Society.

major cellblock component, a structure connecting the cellblocks, an administrative tower, and three out-structures to house inmate services. The design portion of this project focused on the structural design of the cellblocks, connecting core and administrative tower. Specific layouts were defined for each component, and these layouts were designed for gravity considerations, evaluated in terms of cost, constructability and usage factors, then the best options for each scenario was further designed to account for lateral loads.

The following document details all of the steps taken to complete the structural design, layout analysis, and some special considerations that were addressed such as fire resistance, wall construction, foundation design and site design. The background section was written to provide the reader with a better understanding of the evolution of correctional facilities throughout the ages and considerations that must be taken into account in regards to current correctional facility design. In addition, the background presents the issues that must be addressed by the facility in terms of those incarcerated and those living in the area near the correctional facility.

The methodology sections presents an in depth description on means used to select the layouts for the correctional facility and the methods used to complete both gravity and combined gravity and lateral design of the structures in both reinforced concrete and structural steel. The techniques used to evaluate each of the design options are presented in this section as well. The results of the processes described in the methodology are presented in the next section. These results are presented along with the rationale that led to their implementation.

The sections following the results provide an overview of some areas of consideration that were deemed ancillary to the project as a whole. These areas include exterior cladding systems, foundations, fire design and site design. The exterior cladding section provides information on typical materials and design considerations used in correctional facility construction. The foundation system subdivision details information on typical foundation designs and their uses. The fire design section of the special conditions deals with the code requirements set forth by the *2003 International Building Code* with regards to correctional facilities. The final segment will provide general information on typical site design practices for correctional facility. Additionally, this section will illustrate some of the considerations that need to be accounted for in the placement of this particular structure on the parcel of land selected by the group.

The conclusion of this project will tie together the findings produced by the design discussed in the methodology and the information presented in the special conditions. This section will also present other areas of consideration that could be expanded upon in future development of the correctional facility models selected.

2 Background

Over the course of human history, the purposes and methods of incarceration have evolved a great deal. The following sections will provide the reader with a better understanding of the evolution of correctional facilities and the multitude of factors involved with current correctional facility construction and operation. Topics to be presented include the history of prisons and how they have metamorphosed from being simply holding cells located in convenient locations for overlords to the buildings that they are today. Other aspects of correctional facilities that will be discussed are the current prison system in the Commonwealth of Massachusetts, the effects of overcrowding in prisons, and the difference between public and privatized correctional facilities. After these topics have been presented, the design considerations for a modern correctional facility are discussed. These considerations include the layout, services, the location of the correctional facility and the security measures used to keep the inmates, guards and surrounding community safe.

2.1 A History of Incarceration

The reasons for incarceration are many and over the course of human history have taken many different forms. Incarceration is just one of many options available to punish someone for exhibiting socially deviant or illegal behavior. Other options include fines, torture, mandatory labor, public humiliation, banishment and death.
"Prior to modern times, prison was an interlude between court appearance and ultimate punishment, usually torture or death."[5] Today the motivation behind imprisoning

[5] Johnston, Norman B., p.1

individuals is multifaceted. The current goals associated with incarceration are punishment, custody and safekeeping of inmates and defense against outside force, supervision of both prisoners and their keepers, preventing prisoners from corrupting one another through association, maintenance of prisoner's health, and reform of the inmate.[6]

2.1.1 Ancient Societies and Imprisonment

The Chinese are the first culture to be known to utilize some form of incarceration as a means of punishment. The earliest Chinese prison dates back to 2000 BC.[7] The Greeks were the next recorded society to utilize imprisonment beginning around 1100 BC. Greek prisons were used solely for those persons convicted of high treason or debt to the government. The Roman Empire did not recognize imprisonment as a form of punishment, but still practiced confining persons charged with a crime prior to sentencing.

Ancient prison structures were crude at best, and very few examples remain standing today. The Mamertine prison of Rome is one example of an ancient prison.[8] It was constructed at some point during the third and first centuries BC and is located near the Forum. The holding cells are located on two levels. The lower level is believed to have housed those sentenced for life and can only be accessed through a trap door through the upper chamber, which held less serious offenders. Medieval Britain also had prison structures, but these were often located in unused spaces in structures created for other purposes.[9]

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[6] Johnston, Norman B., p.1
[7] Johnston, Norman B., p.5
[8] Johnston, Norman B., p.6
[9] Johnston, Norman B., p.7

2.1.2 Imprisonment during the Middle Ages and Renaissance

During the Middle Ages and Renaissance, imprisonment was used for the sole purpose of detaining individuals as punishment and was seldom legally part of their eventual punishment.[10] Conditions in these prisons were rarely sanitary or humane, and little attention was paid to the health and well-being of the prisoners. Imprisonment during this period was not intended for reform. The concept of reformation through incarceration was introduced later by the church with the rise of Christianity, which is further discussed in section 2.1.3 Religious Confinement.

2.1.2.1 Prisons in Castles and Gatehouses

As time passed and societies developed, holding cells and prison areas became more numerous. In fact, by the twelfth century portions of castles were being constructed for the sole purpose of confining prisoners.[11] The holding cells were typically located in the lower levels of the castle towers, which would otherwise be used for storage. Cells were also located in the gatehouses to the castle proper and the adjoining town. Another option for imprisonment during this time was wooden and iron cages.[12]

The treatment and classification of prisoners varied greatly with social station. High church officials and nobles were often allowed servants and clerks while the peasantry was often thrown in deep dark cells and ignored. The holding areas had a certain hierarchy to them in that the lower the level in the tower a prisoner was held, the lower the social status and usually the more serious the crime.[13]

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[10] Johnston, Norman B., p.16
[11] Johnston, Norman B., p.9
[12] Johnston, Norman B., p.8
[13] Johnston, Norman B., p.9

Famous or infamous depending upon one's point of view, castles with prison cells from this time include Mont Saint Michel, Pierrefonds, Aigues Mortes, the castle at Steinberg in Germany, and Conway Castle in Wales.[14] Mont Saint Michel (Figure 2.1.1) was well known for its iron cages and the suspension of such cages from the ceiling of the compound. Originally, Mont Saint Michel served as an abbey. Through the course of its lifespan, Mont Saint Michel became more and more notorious for its prison conditions; from the time of the French Revolution to the early nineteenth century it was used as a prison of the state.[15] Another castle famous for its prison conditions was Pierrefonds. This notoriety is due mostly to the fact that the prison towers contained what are known as *oubliettes*. An oubliette is an "open circular shaft in the floor of a lower room, into which prisoners were thrown to their deaths."[16]

Figure 2.1.1 Mont Saint Michel[17]

As the role of castles evolved into centers of government and commerce, the areas where prisoners were confined moved from the within inner castle to the outermost walls of the castle or to the surrounding town.[18] Locating prisoners in the gatehouse of towns and castles allowed the gatekeeper to serve the double purpose of supervising both the

[14] Johnston, Norman B., p.8
[15] Mont Saint Michel: Wonder of the Western World
[16] Johnston, Norman B., p.9
[17] Mont Saint Michel
[18] Johnston, Norman B., p.12

gate and the prisoners contained in the gatehouse. Historic examples of gatehouses containing holding cells include London's Newgate and Porta Nuova in Verona, Italy.

2.1.2.2 Prisons of the State and Fortresses

As governments began to become more developed and centralized, structures began to be constructed for the sole purpose of housing prisoners. The Tower of London is a prime example of this trend.[19] Originally, a royal residence and center of government, many of the towers constructed later in the complex's life were designed for detaining convicts. Towers that were converted to prisons include the White, Bell, Wells, Cradle and Salt Towers. Beauchamp Tower was one of the main prisons in the complex. As typical of prisons during the medieval period, the most dangerous and lowest class prisoners were kept in the lowest parts of the tower, and the higher ranking prisoners were held in the chambers above. The Tower of London was also used to detain prisoners of war during the First and Second World Wars.[20]

There are many more examples throughout Europe and Asia of the usage of prisons of the state. The most prominent examples of these structures include the Chateau of Vincennes and the Bastille in France, Petropavlovsk in St. Petersburg Russia and the Kremlin in Moscow.[21] The people of India housed their prisoners in fortresses at Amber and Gwalior. Additionally, smaller locations scattered throughout the world were used as holding cells for the incarcerated during the Middle Ages. These areas included church steeples, abandoned houses, tollhouses, and bridge and town gates.[22]

[19] Johnston, Norman B., p.13
[20] Tower of London
[21] Johnston, Norman B., p.15
[22] Johnston, Norman B., p.16

2.1.3 Religious Confinement

As previously stated, the church was the first entity to utilize imprisonment for reformation. The Catholic church was the first to employ modes of imprisonment as a penalty for sin.[23] It was believed that the misery and solitude of confinement would lead to meditation and contrition on the part of the prisoner. One of the major reasons for the creation of ecclesiastical prisons was the establishment of the practice of sanctuary by William the Conqueror in England.[24] Claiming sanctuary allowed criminals to be granted asylum in the church if they confessed their crimes. The criminal was then required to swear an oath of obedience to the abbot and to wear special clothing for a short period before being deported.

During early Christian times, virtue and holiness were associated with a separation from society. Volunteers would spend long periods of time isolated in the Egyptian desert and in Greece.[25] Another group of people known as the anchorites would intentionally lock themselves into small cells in cathedrals and churches and rarely be let out.[26] The cells that these people dwelled in are known as anchorite cells, anchorages or ankerholds in Great Britain and as *reclusoires* in France. In Great Britain, the cells were built into the walls of churches, providing the encased person with a view of the altar and little else. Anchorites were literally entombed in these cells, given food, water, and little else. Anchorites were regarded as oracles and gossips, were relied upon for their charms and potions and were consulted by kings and used as confessors.[27] An example of an anchorite cell can be found in Figure 2.1.2 Possible Anchorite Cell.

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[23] Johnston, Norman B., p.17
[24] Johnston, Norman B., p.17
[25] Johnston, Norman B., p.18
[26] Johnston, Norman B., p.18
[27] Johnston, Norman B., p.18

Figure 2.1.2 Possible Anchorite Cell[28]

Much more spacious accommodations for repentance were offered via monasteries. Christian monasteries first appeared in the third century and became increasingly popular through the fifth century.[29] Initially, when a monk would commit an offense he was ostracized or if the offense was major he was shunned. The philosophy behind punishment at a monastery was not to punish the offender but to encourage repentance. In fact, punishment at a monastery for the monks was often voluntary. The Benedictine monks would lock themselves in their own sparsely furnished cells when they had committed a wrong.[30] Another form of punishment that monks would subject themselves to was encasement in a penitential chamber.[31] These chambers were much like anchorite cells, and once inside the monks were seldom released. Examples of these cells can be found in Temple Church in London, Canterbury Cathedral, Gloucester Cathedral, and the Collegiate Church of St. Mary in Warwick.[32] Records of prison cells in a monastery did not appear until the sixth century. Essentially those monks who had committed a serious offense were placed in these cells and not freed until some evidence of divine pardon was provided.[33] These monks were forced to live on bread, herbs and

[28] http://www.duston.org.uk/anchorit.htm
[29] Kreis, Steven
[30] Johnston, Norman B., p.19
[31] Johnston, Norman B., p.20
[32] Johnston, Norman B., p.20-21
[33] Johnston, Norman B., p.21

roots. The Benedictines developed the practice of imprisoning the wayward brother with a better behaved brother in the hopes of providing the deviant brother with a good role model.[34] The causes of imprisonment for a monk were often for breaking the rules of the monastery or disrupting the tranquility of the compound. There are no records of prison cells in convents or nunneries.

Monks were not the only people detained within the confines of monasteries. Through the more troubled times of the Dark Ages, more and more disturbed and dissatisfied people began seeking asylum in monasteries.[35] Some records detail people entering the monastery as fugitives and exiting rehabilitated by the monks.

During the Middle Ages, religious courts began appearing across Europe. In addition, if a defendant claimed "benefit of clergy" in civil court, he or she could be tried in an ecclestical court. The sole stipulation of the defendant claiming benefit of clergy was that he or she had to be "of the church".[36] The depository for convicts in ecclestical courts was often the nearest monastery. Due to this practice, monastery prisons know as carceres or decaneta were created.[37] The incarceration for those sentenced to monastery prisons was not the most trying. The detainees were confined to a prison room with an adjoining room that contained a latrine and a food hatch. This room would often serve as the detaining cell for the monks as well, but not always. Monastic prisons were used in Russia and Austria until the 20th century for political prisoners.[38] More typical however was the decline of the monastery in the early fifteen hundreds.[39]

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[34] Johnston, Norman B., p.21
[35] Johnston, Norman B., p.19
[36] Johnston, Norman B., p.22
[37] Johnston, Norman B., p.22
[38] Johnston, Norman B., p.23
[39] Kreis, Steven.

Holding centers for prisoners of the church could also be found in churches and cathedrals. Some examples of churches with prison cells include the collegiate church of Nesle (Somme) and Notre Dame de Boulonge; St. Etienne at Caen, France; Tewksbury and St. Alban's in England, and Ewenny in Glamorganshire.[40] The rationale and churches as prisons was that they were well fortified.

Figure 2.1.3 Tewksbury Abbey[41]

During the Middle Ages, the church and state were virtually a single entity. At the beginning of the thirteenth century, the Christian church was ending the holy wars in the Middle East and soon had to find somewhere to direct the zeal of its most fervent followers. The outlet that they directed their followers to was the Inquisition.[42] A wide range of punishments were used against those deemed heretics, but as the strength of the Inquisition grew, so did the popularity of imprisonment.[43] There were three forms of incarceration to which those accused of religious crimes were subjected.[44] The first was the most mild form and known as *marus largus*. Under this form of imprisonment, the prisoners were granted a relatively large degree of freedom as they were allowed to move freely throughout the halls of the prison. The second form of imprisonment was known

[40] Johnston, Norman B., p.23
[41] Somerville, Johann P.
[42] Jones, Robert
[43] Johnston, Norman B., p.26
[44] Johnston, Norman B., p.26

as *marus strictus* and was more restrictive than *marus largus*. Prisoners detained under *marus strictus* were chained to the walls of their cells and subsisted on bread and a combination of water and beer. The most secure level of imprisonment during the Inquisition was *marus strictissimus* under which prisoners were bound hand and foot in their cells and seldom fed.

The Inquisition soon produced more prisoners than monasteries and cathedral prison cells could hold, thus special Inquisition prisons were constructed.[45] These prisons came at a great expense to the communities in which they were located. The conditions in these prisons were extremely harsh and the death rate high. Reliable descriptions of Inquisition prisons are difficult to determine due to the secrecy involved with the entire affair and the lack of survivor accounts. Prisons constructed solely for the purposes of the Inquisition include Carcassone, Beziers, and Toulouse in France and Evora, Lisbon, and Coimbra in Portugal.[46]

2.1.4 European Prisons of the 16th, 17th and 18th Centuries

The main purpose of prisons during this period of time was to keep people locked in a secure and unpleasant place and that goal was well met.[47] Up until the 18th century, prisoners of all kinds were locked together in communal cells.[48] Women and children were mixed with murderers, rapists and pedophiles. In addition, prisons seldom had water, sewers or bed-stands. Diseases such as typhoid often struck the prison populous, and detainees were often lucky to escape alive and healthy.

[45] Johnston, Norman B., p.26
[46] Johnston, Norman B., p.26
[47] Johnston, Norman B., p.28
[48] Johnston, Norman B., p.28

Prisons of this age were still being placed in pre-existing structures, such as fortresses, which were no longer used for military purposes.[49] This practice was carried out across Europe including Spain, France, Russia, the Scandinavian countries and various small countries in Eastern Europe. Structures that were often converted to prisons were old monasteries and convents.[50] The Spanish utilized this practice not only in Spain but in its colonies in North Africa, Palma de Mallora and Cuba as well. In fact, Morro Castle in Havana Cuba was used as a prison since construction on it was completed.[51] The French often placed departmental prisons in former convents and ecclestical buildings while more long term prisoners were placed into fortified chateaux.[52] This practice continued in poorer European nations until the late twentieth century.

While these structures that were converted to prisons had poor conditions for the inmates, the purposely built prisons of the time were little better. An example of this can be found in the Warwick England County Jail.[53] Prisoners were kept in dank, dark conditions underground where they were chained to center posts and arranged about the room like the spokes of a wheel.

Prison reform during this period was often discussed by the church. The Christian Knowledge Society, founded in 1699 in England, attempted to better the lives of prisoners by providing them religious reading materials.[54] In addition, they proposed the idea of putting prisoners in separate cells. The Christian Knowledge Society had committees on Newgate and Marshalsea prisons in England. Similar societies existed in

[49] Johnston, Norman B., p.28
[50] Johnston, Norman B., p.28
[51] Morro Castle
[52] Johnston, Norman B., p.28
[53] Johnston, Norman B., p.28
[54] Johnston, Norman B., p.29

Florence but none of these entities had much influence on the treatment of prisoners or the design of the prisons.

2.1.4.1 Early Prison Architecture

Yet this period did begin to show the beginnings of what would prove to be a large reformation of prison philosophy in terms of architecture and prisoner treatment. Structures were beginning to be designed with the intent of holding prisoners, and the practices of labor and isolation were being considered for prisoners.[55] Italian, Spanish and German architects generated many ideas in terms of how to house prisoners securely and efficiently. Many of these ideas were borrowed from practices used for hospitals and lazarettos.[56]

Leon Battista Alberti, an Italian architect, called for the reform of underground, tomb-like prisons.[57] His designs included amenities such as latrines and fireplaces. He also introduced the idea of classifying prisoners by level of crime. Another Italian, Antonio Averlino (Filerate), suggested many of the same features and equated better conditions with better behaved prisoners. Cerdan de Tallada of Spain wrote of separating male and female prisoners and providing separation by social class. He also suggested simple architecture, which was against the trends of the time, and greater security for the most dangerous criminals. A final architect to contribute new ideas to prison construction was Joseph Furttenbach of Germany. He suggested isolated cells and classifying prisoners by type of crime. Yet it was J.F. Blondel who truly articulated the new philosophy of prison architecture in a way that resounded with the masses.[58] He

[55] Johnston, Norman B., p.29
[56] Johnston, Norman B., p.29
[57] Johnston, Norman B., p.29
[58] Johnston, Norman B., p.31

proposed the architecture of a prison be menacing, conveying a sense of dread and horror to encourage reform and discourage anyone else from committing crimes.

The evolution of prison architecture is well documented in Venice in the case of Doge's Palace and the New Prisons.[59] During the 9th century, Doge's Palace was the Venetian seat of government and because of the administrative functions it housed; it had a small prison in the lower levels. By the 16th century, the prison had expanded to the point where it occupied the entire ground floor. A new structure was built across the canal from the palace during the 1500's to accommodate the growing number of prisoners. The conditions in this structure were quite unfavorable. The prison, called New Prison, was overrun by hosts of rats and swarms of bugs. In addition, the temperature in the cells had a tendency to fluctuate dramatically.[60] Another prison, of the same name, was constructed to try to alleviate some of these problems yet was unsuccessful. The important aspect of this structure was that it was built for the sole purpose of detaining convicts. It featured three floors of cells facing an inner courtyard along with special cells for new prisoners, torture rooms, and a dormitory for guards.[61] A recent picture of New Prison is displayed in Figure 2.1.4.

[59] Johnston, Norman B., p.32
[60] Johnston, Norman B., p.32
[61] Johnston, Norman B., p.32

Figure 2.1.4 New Prison in Venice[62]

Another of the first purposely built prisons was constructed in Florence. Known as Le Stinche, this prison opened in 1304 and was used in lieu of corporeal punishment or death for convicted persons.[63] The design was such that five large detaining cells surrounded a central courtroom. By the 19th century, cell buildings were added to the complex.

The 16th century saw a great increase in the amount of petty crime occurring across Europe.[64] The root cause of the increase in minor legal infractions is attributed to various social conditions throughout the Low Countries of Europe and the British Isles. Vagrants, prostitutes and thieves of all kinds abounded. To combat this trend, governments began employing workhouses as a corrective alternative to physical punishment or death.[65] In England, work houses were designed after the London's Bridewell and appeared in Oxford, Salisbury, Norwich, Gloucester, Ipswich and Chester. Amsterdam also set up a house of corrections in an old convent and it was upon this model that many other workhouses across the continent were based.[66] In Amsterdam, labor was used to reform the inmates as well as to enforce good habits that they could use

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[62] Venice Page Two
[63] Johnston, Norman B., p. 32
[64] Johnston, Norman B., p. 33
[65] Johnston, Norman B., p. 33
[66] Johnston, Norman B., p. 33

once they were released. Following the Amsterdam model, workhouses were constructed in Leiden, Holland and Lubeck, Bremen, and Hamburg Germany in the early 1600's and in multiple other cities in Germany, Belgium, and the Scandinavian countries throughout the rest of the century.

Workhouses were the first actual attempts at reform through incarceration.[67] The majority were simply large rectangular buildings with no specific architectural elements adding definition to them. Reform in these work houses was via religious instruction and daily work such as carpentry and cobbling.

It was also during the 16th century that architectural commissions for the construction of prisons began occurring.[68] The architecture used on the facades was in styles keeping with the time and the location of the structure, while the interior was kept extremely simplistic with few provisions for sanitation or surveillance. Architects were commissioned to design both the Carcere Nuovo and London's Newgate.[69] The Carcere Nuovo was commissioned by the Pope Innocent X and was designed by Antonio Del Grande to hold prisoners before they were sent out on slave ships. Newgate was designed by George Dancer the Younger. The exterior of this facility was extremely imposing, and the interior featured three different cellblocks, which allowed prisoners to be classified by sex and offense. Yet this design provided few provisions for supervision, and there were only five large cells per cellblock.

German prisons displayed a more rationalized and functional design layout.[70] The most prominent example of this can be found in the prison at Kassel Germany. This

[67] Johnston, Norman B., p.33
[68] Johnston, Norman B., p.33
[69] Johnston, Norman B., p.34
[70] Johnston, Norman B., p.35

prison had cells that housed four inmates and allocated room for them to perform labor in the cell. Women detained in this prison were housed on a separate floor from the men.

Individual cell incarceration also began to appear during the 16th century. The best designs from this period can be witnessed in the *Malefizhaus*, which was constructed in Bamberg and the hospice of San Filippo Neri.[71] *Malefizhaus* was originally constructed to punish and contain witches, sorcerers, and sinners. The second floor of the structure contained individual cells attached to a wide hallway. The hospice of San Filippo Neri was created to house delinquent, homeless and deserted boys and was placed in a former palace in Florence. This hospice featured small cells with day and night isolation and high levels of supervision.

Prison designs utilized in the 18th century seldom had any affect on designs utilized in other parts of the world with three major exceptions. These influential prisons were the House of Corrections of San Michele in Rome, the House of Corrections Milan, and the Ghent Maison de Force.[72] San Michele was one of the first prisons to use cells for confinement. The Milan House of Corrections was designed later and was an improvement on San Michele.[73] This structure featured T shaped cell sections that were constructed between buildings holding workrooms. There were 120 cells on the three levels of the prison. The final influential prison was the Maison de Force in Ghent.[74] Design was commissioned by Count Jean Philippe Vilain, and it borrowed many influences from the workhouses of Amsterdam. The Maison de Force is considered the first "large scale adult penal institution in which a serious attempt was made to bring architecture to the aid of the penological philosophy of treatment in a sophisticated and

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[71] Johnston, Norman B., p.35
[72] Johnston, Norman B., p.35
[73] Johnston, Norman B., p.37
[74] Johnston, Norman B., p.39

skillful manner."[75] The layout of the compound was essentially a giant octagon formed by eight trapezoidal, self contained cell units. The capacity of the entire facility was 2600, and this particular prison was used up until 1935; it was later destroyed during World War II.

The prisons of the 16th, 17th, and 18th centuries were still crude by modern standards, but steps were being taken to provide more humane conditions for prisoners. In addition, the idea of using incarceration for reform instead of merely punishment was garnering more and more consideration in the operation of correctional facilities. In the next major period for prisons, reform of the current system will become a major issue as well as the architectural styles utilized in the construction of prisons.

2.1.5 The Major Period of European Reform: 1780 to 1835

Starting in the year 1780, the idea of prison reform really began to emerge in some European countries. The catalyst for these changes was the presence of new idealistic reformers. Ideals centering upon imprisonment were now focusing on the reform of the prisoner, more humane treatment, surveillance, and separating the prisoners by the offenses they had committed.[76] The country that led the way in reform was England, and the leading reformer in Great Britain was John Howard.

John Howard published multiple volumes detailing the conditions of prisons across Europe.[77] Howard's interest in prisons stemmed from his election as a high county sheriff and from the fact that he "was taken prisoner by prisoner by pirates from a ship bound for Portugal, where he intended to aid the victims of a Lisbon earthquake."[78]

[75] Johnston, Norman B., p.39
[76] Johnston, Norman B., p.42
[77] Johnston, Norman B., p.42
[78] Johnston, Norman B., p.42

Howard believed that prisoners could be reformed through physical labor, confinement and religion. Other major prison reform activists in England during Howard's time were George O. Paul, Elizabeth Fry, James Neild and Jeremy Bentham.

Jails during this time were often very corrupt. Guards would charge prisoners for necessities such as food, water, clothing and candles. Prisoners taught each other tricks of the trade, and a great deal of contraband was exchanged. Once England had lost the American colonies as a dumping ground for criminals, they began to look more in depth at reforming their prison system.[79] Many of the reforms that were enacted by the British government were those suggested by Howard. In 1779, the Penitentiary Act, Hard Labor Bill was passed. This bill called for separating men and women, night isolation and daytime supervision, labor the prison could profit from, a fixed daily routine, cleanliness, mandatory attendance at religious services and no luxuries or amusements.[80] The Gaol Act of 1803 elaborated further on the classification system to be used by the British government.[81]

The reformers in England formed a society known as the London Society for the Improvement Prison Discipline (SIPD) in 1813.[82] This society wanted prisons to meet the following criteria:

1) Punishment
2) Security from escape and defense from outside force
3) Systematic supervision of both prisoners and guards
4) Good health of the occupants
5) Prevention of corruption arising from prisoners mutual contact
6) Reformation by means of labor, religion and possibly education[83]

[79] Johnston, Norman B., p.43
[80] Johnston, Norman B., p.43
[81] Johnston, Norman B., p.45
[82] Johnston, Norman B., p.44
[83] Johnston, Norman B., p.44

The society placed a major emphasis on increasing the amount of supervision in the prison systems.[84] It also held the belief that the majority of deviant behavior could be cured by solitary confinement.

Due to financial difficulties these reforms were hard to enact. The number of guards required to adequately supervise the prisoners in the jails at this time could not be funded by the government. In addition, housing prisoners in separate cells increased both the construction and operation costs of the prison. Progress could be seen however in the jails at Horsham and Pentworth, which began utilizing solitary cells. Solitary cells were also used in the prisons at Wymondham, Norfolk and Manchester.[85]

2.1.5.1 Architecture Attempts to Aid Reform

Three major forms of prison design were created to help enact the reform policies generated by the British Parliament and the SIPD. These forms were created by architects, magistrates, master builders and stone masons.[86] These three forms were rectangular (non-radial), circular and radial plans.

Non-radial prison design was used up until fifty years after John Howard's books were published and presented no real advancement in design.[87] These prisons featured cells arranged in rectangles, "U" shapes and hollow rectangles with inner courts. The layouts were very simple but did manage to provide adequate light, ventilation and sanitation for the prisoners. William Blackburn, a disciple of Howard, designed over sixteen non-radial prisons in England and Ireland. The major criticisms for this type of prison design were that it was impossible to classify prisoners in this type of architectural

[84] Johnston, Norman B., p.45
[85] Johnston, Norman B., p.44
[86] Johnston, Norman B., p.47
[87] Johnston, Norman B., p.47

format and that surveillance was not possible in day rooms, cells or in the prison yards without a guard being physically present.[88]

The second design was the circular or polygonal prisons. Geometric patterns were very common in 18th century architecture thus their usage in prison design could almost be expected.[89] The earliest designs for a polygonal prison were drawn by Pierre Gabriel Bugniet but never constructed. Perhaps the most important design for a circular prison was created by Jeremy Bentham. Bentham's design was borrowed from his brother's design of a circular textile mill.[90] Referred to as a Panopticon, the general design mandated that a circular building with a diameter of 100 to 180 feet be constructed with cells being placed along the outside walls back to back. The design varied from two stories to six stories with an observing station for guards located at the center of the circle with peepholes to observe the prisoners in an unseen manner. [91] Supervision of cells along the outside exterior was impossible for the guards stationed in the observing decks, and at one point Aldous Huxley referred to Bentham's design as a "totalitarian housing project."[92]

Bentham tried to get the British government to construct his Panopticon for twenty-five years with no success, while providing his plans to the Spanish, French and Scots as well with the hopes that they would build his prison. Though his design greatly inspired those at the French Academy of Architecture, he never lived to see one of his prisons built.[93] Semicircular versions were eventually constructed in England, Scotland

[88] Johnston, Norman B., p.48
[89] Johnston, Norman B., p.48
[90] Johnston, Norman B., p.49
[91] Johnston, Norman B., p.50
[92] Johnston, Norman B., p.50
[93] Johnston, Norman B., p.50

and Wales, the most notable being Edinburgh Bridewell and Gloucester House of Corrections in North Leach.

Figure 2.1.5 Edinburgh Bridewell During Construction[94]

The third, and most popular, from of prison was the radial prison. In this design, wings of cellblocks converge upon a center hub.[95] Variations on this design include T-shapes, crosses and spoke type formations. The man behind the usage of radial designs for prison construction was William Blackburn.[96] The intent of Blackburn's designs was to provide an easy means for observing prisoners and guards alike. The hub in the center was often used as a chapel and administrative quarters.

The first radial prison to be constructed in England was the Suffolk County Jail at Ipswich.[97] This prison was in the shape of a Greek cross, with four wings radiating from a center hub, which was octagonal. From the hub, guards could observe activity in the corridors from a central vantage point. Berkshire House of Corrections in Abingdon followed a similar design.[98]

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[94] Modern Athens
[95] Johnston, Norman B., p.55
[96] Johnston, Norman B., p.56
[97] Johnston, Norman B., p.56
[98] Johnston, Norman B., p.56

The most prevalent design pattern in England and Ireland was the T shaped radial design.[99] Often times the cell wings would be separated from the main hub by a gap of ten to thirty feet. This design can be seen in Glamorganshire Prison at Cardiff, the county jail at Cambridge and the Suffolk House of Corrections at Bury St. Edmonds. Other radial prisons constructed in England and Ireland that were not T shaped often had from three to seven cell wings at various angles radiating from the hub.[100] Examples of this kind of design are Meath County Jail (Trim, Ireland), Liverpool Borough Jail, Dartmoor Convict Prison and the prisons at Tothill Fields and Coldbath Fields.

Perhaps the most well known radial prison in England is Millbank. Millbank was constructed on marshland near the Thames where the land was unsuitable for normal foundation construction.[101] The project cycled through four architects prior to completion. The design was that of six pentagons converging to form a hexagonal center area with each pentagon containing six 36 cell wards for various classes of prisoners. The overall configuration of the structure is depicted in Figure 2.1.6. The exterior of the prison was surrounded by an octagonal wall and a moat. The prison itself contained three miles of corridors and was meant to hold one thousand prisoners.[102] This plan was never duplicated and had some major drawbacks. These drawbacks included the high cost of maintenance, the architecture limited the sight lines, it could never be filled to capacity, and the outside wall had a tendency to fall down due to poor construction and soil conditions.[103]

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[99] Johnston, Norman B., p.57
[100] Johnston, Norman B., p.58
[101] Johnston, Norman B., p.62
[102] Millbank
[103] Johnston, Norman B., p.63

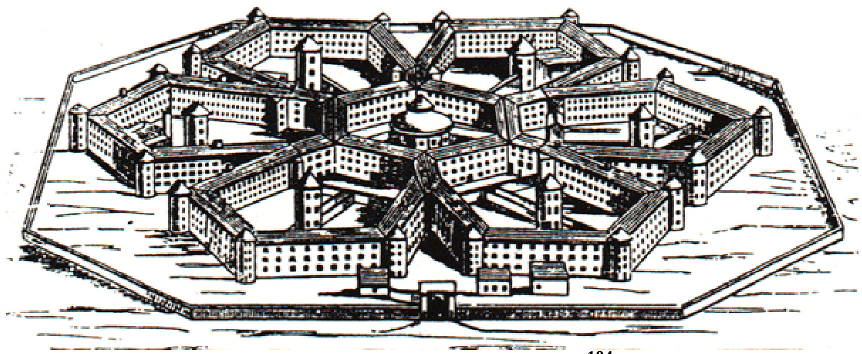

Figure 2.1.6 Millbank Prison[104]

Prisons outside the British Isles were also constructed using the radial plan. When these prisons were constructed, their designs were based on those prisons constructed in England and Ireland.[105] Some prisons applying the radial design include Moscow District Prison, Petit Roquette Prison in Paris, and the House of Correction for Rio de Janeiro Brazil.

2.1.5.2 Overall Impact of English Reform

The goals set by the English reformers were seldom met by the designs created by the architects. Essentially all of the focus in design was placed on observing inmates' outdoor activities while few provisions were made to monitor the interior of the buildings. Separation of inmates by classification was confusing because there was no universal system, and many local forms of classifying prisoners got extremely complex to the point of rendering them unusable. One reform goal that was met however was the creation of menacing facades in the hopes of deterring future criminals. The ideals held by these reformers would have a significant impact on the designs utilized in the United States where they would be improved upon.

[104] Millbank
[105] Johnston, Norman B., p.58

2.1.6 The Early American Prison System

Prior to 1830, the vast majority of prison reform was occurring in Europe, yet after this point, the focus for penitentiary design and operation shifted to North America.[106] Two major types of prison operations are described in this section, the Pennsylvania system and the Auburn system. These two systems served as model for countries in Europe, South America and Asia until the 19th century.

The first system to be developed was the Pennsylvania system, whose roots can be traced to reforms enacted by William Penn.[107] Penn's penal code dictated that the reform of prisoners was more important than their punishment. He also eliminated the death penalty for most crimes in the state of Pennsylvania. Yet, when Penn died his reforms and ideas were overhauled to match those of the rest of the colonies. After the Revolutionary War, Quakers and other leaders in Philadelphia formed the Philadelphia Society for Alleviating the Miseries of Public Prisons (later known as the Pennsylvania Prison Society).[108] This group was formed to help ensure that prisoners received adequate food. The Society also made themselves familiar with the prison reform trends of Europe. They were well versed with the theories of John Howard and established lines of communication with major British and French reformers and Russian philanthropists.

One of the first prisons in Pennsylvania built specifically to hold the incarcerated was the Walnut Street Jail (WSJ), located in Philadelphia. The jail was constructed in 1776 and built in the traditional, non-radial "U" shape prevalent at the time.[109] The cellblocks in the WSJ were the first examples of solitary confinement used as a form of

[106] Johnston, Norman B., p.67
[107] Johnston, Norman B., p.67
[108] Johnston, Norman B., p.67
[109] Johnston, Norman B., p.67

punishment and rehabilitation.[110] The first cellblock was built in 1790, was three stories tall, and measured 40 feet by twenty-five feet.[111] The set up of this cellblock was such that the upper two stories contained four solitary cells facing a large corridor. The accounts of William Crawford, who was sent to the United States from Britain to investigate the state of U.S. prisons, describe the conditions at the Walnut Street Jail as significantly poorer than the ideals set up by the societies of the time.[112] Crawford described the WSJ as poorly ventilated and noted that there were few provisions for labor in cells. In fact, he established that the cells were used solely for the punishment of prisoners who broke the rules of the prison instead of housing those who had been sentenced to solitary confinement.

During the same period, New York State was facing similar issues with the state of its prison system. The structures used as prisons were primitive, there was a great deal of overcrowding, and unhealthy conditions abounded.[113] New York City had a prison much in the style of the Walnut Street Jail and it was named Newgate for the London prison. Newgate was two stories tall and contained fifty-four rooms.[114] Seven solitary cells were located at the end of each cellblock. These cells, much like those at Walnut Street were used solely as punishment as opposed for rehabilitative practices. Massachusetts, New Hampshire, Vermont and Maryland had similar types of prison structures.

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[110] Prison Reform
[111] Johnston, Norman B., p.68
[112] Johnston, Norman B., p.68
[113] Johnston, Norman B., p.68
[114] Johnston, Norman B., p.68

One interesting example of an early American prison is the prison at Thomaston Maine.[115] This prison had 48 cells or pits as they were known. These pits were located underground and could only be accessed through a top hatch. During the day, prisoners were released from their cells to perform various labors then placed back in their cells at night.

2.1.7 Reform of the Early American Prison System

As reformation trends swept through Europe, the prison societies across North America began to take notice. The ideals set forth by the English would be the catalyst for two major prison operating systems that would revolutionize the treatment of prisoners in the United States and eventually across the globe.

With the overcrowding of the Walnut Street Jail, solitary confinement was abandoned and soon the Pennsylvania Prison Society noticed that inmates were corrupting each other.[116] To combat the issue of prisoners from being unhealthy influences on each other the Pennsylvania Prison Society began to make a strong push for the inclusion of solitary confinement as a more common prison practice. Their rationale for utilizing the practice of solitary confinement was that the prisoners would be unable to communicate with each other, it would give the prisoners more time for reflection on the crimes they had committed, and it would also serve to prevent mass uprisings and conspiracies.[117]

One major debate formed by this group centered upon whether or not the prisoners should be allowed to work.[118] One side of the argument believed that prisoners

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[115] Johnston, Norman B., p.68-69
[116] Johnston, Norman B., p.69
[117] Johnston, Norman B., p.69
[118] Johnston, Norman B., p.69

should be kept idle so that they were forced to reflect upon their wrongs. The other side of the argument believed that labor could be used as a tool to reform the prisoners and that it managed to preserve health and sanity while aiding in funding the prison. Eventually, those who believed in labor in the prisons won the argument, and in 1829 an act was passed in Pennsylvania making labor a mandatory part of solitary confinement.[119]

Cherry Hill Prison, also known as the Eastern State Penitentiary, was the first prison to operate under the Pennsylvania system. John Haviland and William Strickland competed over the design of the prison. John Haviland's design was selected, and he eventually fell into the role of overseeing the project as well. Cherry Hill was designed such that seven cell wings radiated from a central hub as can be seen in Figure 2.1.7.[120] The first three wings were constructed at a height of one story, but due to overcrowding in other prisons in Pennsylvania at the time, the final four cell wings were converted to two story structures. The final prison had 450 cells. There was only one entrance to the compound for security reasons. The prison also featured an 80 foot tower at the front of the central hub, and there were additional guard towers at the end of the perimeter walls. The major criticisms of this design were that it cost a great deal of money, and the architecture was perceived to be far too elaborate for a structure containing convicted criminals.[121] Another major criticism of the prison operation itself was the extreme level of isolation to which the prisoners were subjected. Prisoners were allowed out of cells for baths and medical emergencies, and they were not allowed visitors of any kind.[122] Despite the criticisms for the design and practices utilized at Cherry Hill, Europeans began to mimic the design and operational procedures.

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[119] Johnston, Norman B., p.69
[120] Johnston, Norman B., p.70
[121] Johnston, Norman B., p.73
[122] Johnston, Norman B., p.73

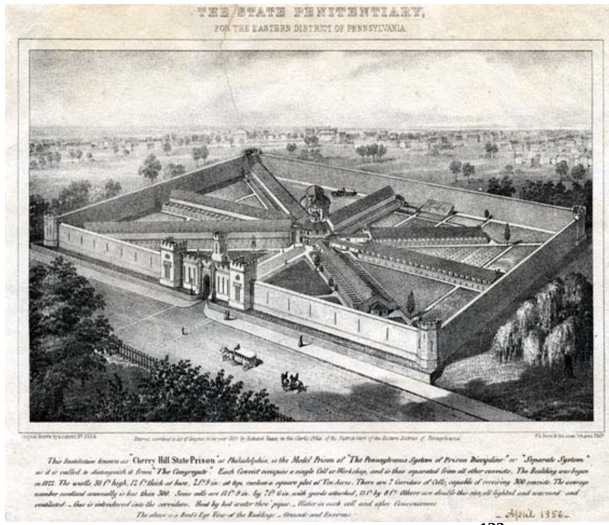

Figure 2.1.7 Cherry Hill Prison[123]

Haviland also designed Trenton State Prison, which he deemed a more refined version of Cherry Hill. The design for this prison consisted of five wings radiating in a semicircle from a main hub.[124] The original plan for the prison called for 300 solitary cells. In the beginning of operation, prisoners never left their cells. This led to physical and mental health problems among the inmate population. By 1838, some inmates were allowed to leave their cells to work. Eventually overcrowding ended the practice of solitary confinement.[125]

As previously mentioned, the Europeans favored the Pennsylvania system of prison design and operation. The trend throughout North America, however, was focused upon a different system known as the Auburn system. The name of this system comes from the prison where it originated. Auburn Prison in New York was initially constructed in the Newgate style, with large rooms that housed from eight to twelve prisoners, with work starting in 1816.[126] This design resulted in a great deal of disorder and a number of insurrections occurred. The design for the second wing of the prison,

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[123] Library Company of Philadelphia Wainwright Lithograph Collection
[124] Johnston, Norman B., p.74
[125] Johnston, Norman B., p.75
[126] Johnston, Norman B., p.75

which was constructed using inmate labor in 1825, was much different from the initial wing. This North Wing was designed by John Cray and was to have 550 small sleeping cells back to back on five tiers in the building. Though this design presented issues with ventilation, heating and lighting, the design was duplicated in the old wing once the original had been demolished.[127]

In 1821, New York set up a classification system for its inmates that stipulated that the most hardened criminals be contained in solitary confinement for the entirety of their sentence.[128] The lesser criminals were to split time between being in groups and night time isolation. By 1824, it became evident that subjecting prisoners to solitary confinement for extended periods of time had a very adverse effect on their mental and physical health. From these findings, the Auburn system was developed. The Auburn system allowed the convicts to work in silence as a group during the day and to be isolated at night.[129] This system is also referred to as the silent system. Eventually, all of the prisons in the United States followed this system, with the exception of those in Pennsylvania.

Sing Sing Prison, constructed in 1825, was the next major prison to use the Auburn system.[130] Constructed using prisoner labor, the final cellblock contained 1,000 cells and formed the east side of a large square that comprised the rest of the complex. The other buildings in the complex were used as shop and administrative buildings. This design quickly became the prototype for prisons in the United States. The design of this prison cites no European influence but it has been theorized that it is at least partially

[127] Johnston, Norman B., p.75
[128] Johnston, Norman B., p.76
[129] Johnston, Norman B., p.76
[130] Johnston, Norman B., p.77

influenced by the Maison de Force in Ghent.[131] This lack of architectural influences attributed is to the fact that the design of the prison emerged from conversations between jail keepers and master builders who had previously constructed jails. Their approach to prison design was based more on functionality than the ideology of the time.[132]

Prisons using the Auburn system were constructed in many states including Massachusetts, Connecticut, Vermont, New Hampshire, Georgia, and Maryland.[133] The typical prison constructed using the Auburn design had 150 to 250 cells per tier in the cellblocks. The Canadians also followed the Auburn system, which was implemented in Kingston Penitentiary.[134] Kingston was designed in the shape of a Greek cross, with three wings of cells. There were five levels of cells per cell wing, and these cells were perhaps the smallest in North America. The cells were two and a half feet wide, six and a half feet deep and eight feet in height.

Figure 2.1.8 Kingston Penitentiary in 1950[135]

American designers also attempted to implement the Panopticon design of Jeremy Bentham. Two large prisons were constructed utilizing this design. The first was the

[131] Johnston, Norman B., p.78
[132] Johnston, Norman B., p.78
[133] Johnston, Norman B., p.79
[134] Johnston, Norman B., p.79-81
[135] Justice Behind the Walls

Virginia Penitentiary in Richmond.[136] The structure was authorized for construction in 1796 and was constructed under the supervision of Benjamin Letrobe. Letrobe combined influences from Europe and Philadelphia in his design. The result was a three-story, horseshoe-shaped structure that consisted of two outer branches of cellblocks meeting in a central keepers quarters. The bottom floors were used as work rooms and the upper floors as housing units. The major drawbacks of this design were that the prisoners could not be directly observed from a central location; there was no heating, ventilation, or sanitation provisions; and the mortality rate was high (especially in the early days of operations).[137]

The second round prison to be designed was the Western State Penitentiary in Pittsburg.[138] The prison was designed by William Strickland and was intended to focus on solitary confinement. It should be noted that the Panopticon style was not intentionally imitated by this design. The prison itself was enclosed by an octagonal wall and featured a building with two rows of cells placed back to back and aligned on the outer radius of a 320 foot diameter circle. The major issues arising from this design were the guard could not observe the prisoners on the exterior of the circle, there was little heating or ventilation, and there were no provisions for sanitation or bathing.[139] Overall, this design was deemed a failure and was demolished by prison labor. John Haviland was later commissioned to build a new prison on the same site.

Many of the American prisons and systems created in the early 19th century had a significant impact on the way that countries in Europe designed and operated their prisons. Observers came from England, France, Prussia and Peru to view the new trends

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[136] Johnston, Norman B., p.82
[137] Johnston, Norman B., p.83
[138] Johnston, Norman B., p.83
[139] Johnston, Norman B., p.84

in incarceration put into place by the Americans.[140] These countries tended to favor the Pennsylvania system of operation while the majority of the United States tended to favor the Auburn or Silent system. Over the next hundred years, the American systems continued to develop and create new trends.

2.1.8 The Evolution of the United States Correctional System from 1835 to World War II

Due to labor shortages across the United States, the Auburn system of prison operation was heavily favored over the Pennsylvania system.[141] There were essentially six different prison layouts utilized during the roughly 100 years, spanning from pre-Civil War to World War II. These layouts included "cellblocks flanking a central administration building; self enclosed; panopticon; radial; telephone-pole; and campus."[142]

In the United States, the states operated their own prisons, which received prisoners from the state courts.[143] The federal government placed their prisoners into state prisons until 1906 when Leavenworth Federal Prison opened. Today Leavenworth is still in operation and is the largest maximum security prison in the United States.[144] The design was done by William Eames and Thomas Young and featured a domed center structure with four radiating cell wings and a fifth wing containing classrooms and offices.[145] Figure 2.1.9 provides an aerial display of the layout used for Leavenworth. The few radial prisons that were constructed were done in the model of the Trenton State prison designed by John Haviland. Another radial prison constructed in the United States

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[140] Johnston, Norman B., p.87
[141] Johnston, Norman B., p.138
[142] Johnston, Norman B., p.139
[143] Johnston, Norman B., p.139
[144] Leavenworth Kansas: Area Prison Information
[145] Johnston, Norman B., p.139

during this time period was the Philadelphia County Prison at Holmesburg.[146] This prison consisted of ten wings radiating out from a central hub and was intended for solitary confinement.

Figure 2.1.9 Postcard of Leavenworth Federal Prison[147]

The most prominent design option for prisons in the nineteenth century throughout the United States was that of the center administration building flanked by multi-tiered cellblocks.[148] Prisons fitting this description include Roosevelt Island, New York; Green Bay, Wisconsin; Columbus, Ohio; and Jackson, Michigan. This design was not very versatile in terms of accommodating various classifications of prisoners and thus gave way to a new, more flexible layout configuration.

This layout was known as the telephone-pole layout, which was first developed by DuCane at Wormwood Scrubs in England and Poussin at Fresnes.[149] The telephone-pole design consists primarily of a central corridor connecting cellblocks and other service buildings in linear fashion. The design first gained acceptance in the U.S. Federal Prison system and then in the State of California. The first prison in the United States to

[146] Johnston, Norman B., p.139
[147] Prison Postcards
[148] Johnston, Norman B., p.139
[149] Johnston, Norman B., p.139

use this design was the Minnesota State Prison at Stillwater.[150] Stillwater had cellblocks connected by a corridor to a central guard room, chapel, laundry, dining room and kitchen.

Figure 2.1.10 Stillwater Prison in 1912[151]

Kilby Prison in Montgomery, Alabama mimicked the Stillwater design. The telephone-pole method of design was also used at the new Eastern State Penitentiary constructed between 1927 and 1933. The federal government used the telephone-pole plan a great deal in its prison designs from the 1930's on.[152]

Two well-known prisons designed during this time were Attica in New York and Alcatraz in San Francisco. Attica had cellblocks forming a square with a large walled enclosure where industrial shops, mess halls, and other services were located.[153] These services were connected to each other via a central corridor. Attica is most well known for a 1971 uprising where the mostly African-American inmates took over the prison for a period of four days before state troops were sent in to retake control.[154] The troops ended up killing 29 of the hostages the inmates had taken and 10 guards. In the wake of

[150] Johnston, Norman B., p.140
[151] History of the Minnesota Department of Corrections
[152] Johnston, Norman B., p.141
[153] Johnston, Norman B., p.143
[154] In Depth- Attica

the incident, a cover up followed along with retaliation against the inmates through torture.

Alcatraz is not notorious for any major incidents; its notoriety stemmed from the inmates themselves and the mystique of the structure. The worst of the worst were sent to Alcatraz, and these inmates included Al “Scarface” Capone, George “Machine Gun” Kelly, and Robert “Birdman” Stroud.[155] Inmates were housed one man to a cell. The design was rather haphazard and did not reflect any of the architectural influences of the time period.[156] Four ranges of interior cells were housed on three levels within a single building. The inconvenient location (an island in the middle of San Francisco Bay) and high operating costs contributed to its closure in 1963.[157]

2.1.9 Conclusions

After World War II, emphasis on rehabilitation of prisoners continued to be of utmost importance in the operation of prisons. The designs of prison structures themselves have not evolved significantly apart from the evolution of hybrid combinations of the six forms utilized from 1835 to World War II. In addition, the location of urban jail structures has spurred the need for the design of high-rise prisons.[158] These prisons are designed with essentially the same philosophy as the other models, but instead of being sprawling in the horizontal direction they reach skyward.

The following sections will present the issues involved with prison construction today in terms of the various structural, political and service requirements that a facility today needs to meet. It should be remembered that prisons today are the most recent

[155] Museum Management Program: Alcatraz Island
[156] Johnston, Norman B., p.144
[157] Johnston, Norman B., p.144
[158] Johnston, Norman B., p,152

evolutions of the ones that were constructed during medieval times and though the conditions and operating philosophies are much improved, there is always room for improvement in both design and operation.

2.2 Effects of Prison Overcrowding

There are many effects of prison overcrowding, both on the inmates and on the correctional system. This section will briefly cover two causes of overcrowding: mandatory sentencing and recidivism. Focus will also be directed towards the effects of overcrowding on the available resources, inmate behavior, and on the correctional system.

2.2.1 Overcrowding Definition

An overcrowded prison population is one in which the number of inmates exceeds the amount of available space, resources, and intended housing. Prison overcrowding can be described as two types: spatial density overcrowding and social density overcrowding. The John Howard Society is an organization devoted to understand the problems of crime and the criminal justice system. From the words of the John Howard Society, spatial density overcrowding is defined as the amount of space available for each person. Social density overcrowding is defined as the amount of people sharing one unit, resource, or privilege.[159] Based on the two definitions, one can see that issues concerning social density overcrowding can have disastrous effects on the rehabilitation of the inmates.

2.2.2 Causes of Overcrowding

There are many causes for the increased population in prisons. On a whole, the incarceration rate across the nation has had a continuous and steady growth starting in the

[159] John Howard Society.

1980s. Figure 2.2.1 shows the trend in the nation's imprisonment. The increase in the prison population can be attributed to many factors, including mandatory sentencing, stricter law enforcement, the "War on Drugs", and the failure of the correctional system to rehabilitate those already incarcerated. Focusing on the state of Massachusetts, the increase of the prison population and the resulting prison overcrowding will be examined using two sources: mandatory minimum sentencing and recidivism.

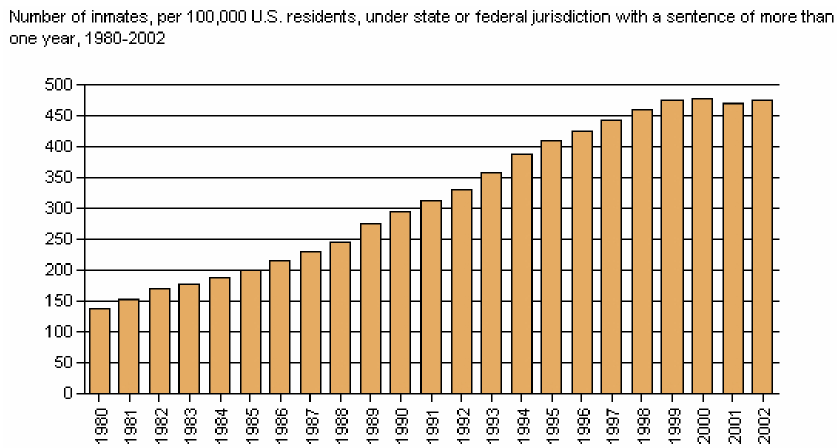

Figure 2.2.1 Nation's Incarceration Rate[160]

Mandatory minimum sentencing is part of a 'tough on crime' approach used to persuade the population from partaking in illegal activities. Mandatory sentencing results in the use of pre-established sentences. These pre-established sentences are to be yielded to by the judge, thereby cramping the judge's ability to fulfill their obligation to charge the offender with his judgment. A report by the Massachusetts review board has determined that mandatory sentencing for drug offenders has been the main cause of prison overcrowding for the state. "Drug offenders have been taking space needed for the incarceration of violent offenders, who are then released into communities. Because of the lack of funds, recently-released violent criminals have little supervision".[161] Figure

[160] http://publicagenda.com
[161] www.ndsn.org

2.2.2 and Figure 2.2.3 illustrate a drop in the nation's violent and property crimes rates. Therefore, the increase in the incarceration rate seen in Figure 2.2.1 may be explained by mandatory minimum sentencing; its effects on sentence length and drug related incarceration.

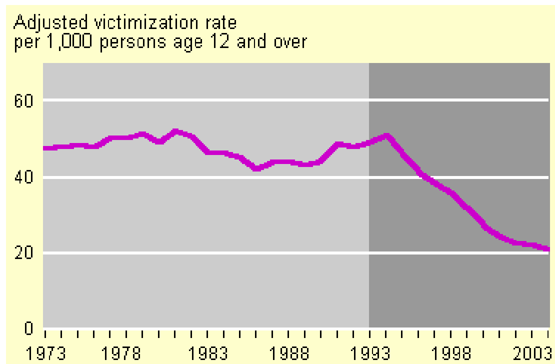

Figure 2.2.2 Violent Crime Rate, United States[162]

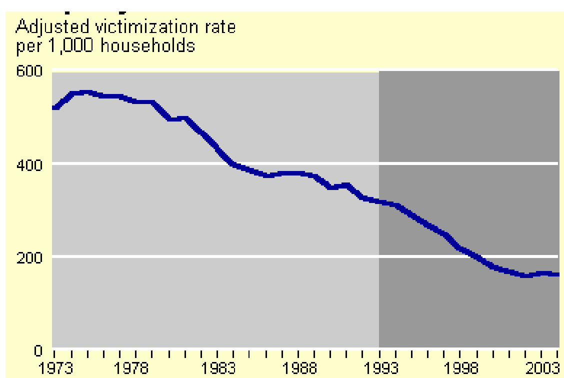

Figure 2.2.3 Property Crime Rate, United States[163]

Recidivism is defined by the *Merriam-Webster Dictionary* as a tendency to relapse into a previous condition or mode of behavior. Recidivism is both a cause and a result of prison overcrowding. There are many reasons for recidivism; these reasons can range from the inability of the inmate to conform to society and the inability of the correctional system to properly reform the inmate. It is the responsibility of a citizen to possess and portray the behavioral and ethical standards of society. If they cannot uphold

[162] www.ojp.usdoj.gov
[163] www.ojp.usdoj.gov

such standards, they must be confined from society until they can accept the standards. Therefore, it is up to society to rehabilitate the aforementioned citizen. This is the role of the correctional system. If a citizen is released from the custody of a correctional system without attaining the accepted societal behavior and ethics, then the system has failed.

In Massachusetts, the recidivism rate is catalyst of the overcrowded population in the state's prisons. A study conducted by the state's Department of Correction's (DOC) Research and Planning Division monitored some 2,961 prisoners who were released from the state's correctional system in 1997. Within three years of their release, 41 percent were imprisoned again within a state facility for a new crime.[164]

"Worse yet, under current sentencing practices, nearly half of all state inmates, who have historically been categorized as the most serious and violent offenders, have no post-release supervision at the end of their sentence and simply leave the prison setting with neither support nor supervision to assist in their reentry".[165] In 2002, 67 percent of prisoners were released on a good conduct discharge; the remaining 33 percent were released on parole. The state's failure to place released inmates on supervision is also a leading cause of recidivism.[166] Angela Antoniewicz, of the Criminal Justice Policy Coalition in Massachusetts, found that in 1990, five percent of those released from a maximum-security facility were released without parole. Adversely, in 2002 there was a 240 percent increase in the number of inmates released without being assigned to parole, a total of 12 percent. Coincidentally, 58 percent of the prisoners released from a maximum-security facility in Massachusetts were convicted of a new offense within three years. This is substantially high when compared to the figures for the state of Texas,

[164] Brooks, Lisa E. p.23
[165] Brooks, Lisa E.
[166] Brooks, Lisa E. p.24

which has the second largest correctional system. In 2002, only 5.8 percent of the maximum-security inmates were released without parole. The recidivism rate for the maximum-security population was 28.3 percent, less than half of that in Massachusetts.[167]

2.2.3 Results of Overcrowding

There are three main results of overcrowding in correctional facilities. First, there are fewer resources available for the prisoners to use. These resources include various educational programs, entertainment, and physical activities and equipment. Second are the behavioral and mental effects of overcrowding on the inmates. Inmates can be classified as those citizens who are socially disobedient. Their disruptive and destructive behavior can be escalated when closely placed with others of the same mindset. The final effect is the inadequacy of the correctional facilities to house the inmates, resulting in misclassification. Misclassification of inmates can result in high costs to the inmate and to the correctional facility in which they are housed. Each of these three results of prison overcrowding will be examined in further detail in subsequent sections.

2.2.3.1 Effects of Overcrowding on Available Resources

Inmates are in a correctional facility for being socially disobedient, displaying deviant behavior, and/or partaking in socially unacceptable acts. Therefore, it is the responsibility of the correctional facility to detain such a person until they have become rehabilitated and are ready to be socially responsible. To accomplish such a task, correctional facilities provide educational, rehabilitation, vocational, and training programs.

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[167] Antoniewicz, Angela.

Overcrowded facilities result in fewer opportunities for inmates to be involved in these programs. Both resources and space are challenged resulting in less availability for all. The effects of this on the prisoner's reintegration back into society are detrimental, both presently and upon their release. The purpose of the programs is to alleviate the barriers that inmates face upon their reintegration into society. These barriers include overcoming substance abuse, mental and health issues, lack of skills in terms of education and labor, and employment problems.

One of the most critical resources needed is educational services. Based on a study of Massachusetts inmates who participated in an education program, their recidivism rate decreased 25 to 50 percent.[168] In the Massachusetts state correctional facilities, 47 percent of the inmates detained in 2002 were without their high school diploma or an equivalent general educational development (GED) certificate. Fourteen percent of those inmates had not made it past the eighth grade during their adolescent school years.

In 2002, 4,000 inmates needed a GED but only 321 were able to enroll into a GED program.[169] Only 17 percent of the inmate population was able to participate in any educational program. A compelling reason for such low numbers is the fact that only three percent of the DOC budget is allocated for inmate programs. Within that, the budget for inmate education and training programs has decreased by 43 percent from 2001 to 2004 while 36 full-time teachers were laid off in 2001 due to the cut-backs in prison education. As a result, the educational programming, which has shown to have a

[168] Antoniewicz, Angela.
[169] Antoniewicz, Angela.

profound effect on inmate behavior, has seen major monetary cuts even in the midst of the correctional system being overcrowded.

2.2.3.2 Effects of Overcrowding on Inmates

The effects of prison overcrowding on inmates can be profound. Problems, both mentally and physically, arise when socially-disobedient people are closely confined for extended periods of time. Overcrowding a prison results in confining many deviant and destructive citizens together who would otherwise keep to themselves. It has been found that overcrowding results in a lack of personal control due to peer pressure (often disastrous to an inmate's personal goals of rehabilitation) as well as idleness and boredom because of the lack of resources to occupy their time.[170] Many inmates may become depressed and social withdrawn because of the inability to escape the omnipresent noises of talking/yelling, television, radio, and foot traffic.

Aggression is another result produced by overcrowded prisons on inmates. This aggressive behavior stems from the competition for space, resources, and self control. "…that the need is not for more room for inmates, but rather for small or moderate amounts of room with some degree of privacy".[171]

2.2.3.3 Effects of Overcrowding on Housing

"To a certain degree, overcrowding has resulted in offenders being classified on the basis of the space available rather than on the security level and programs most suitable for the offender".[172] The effect of incorrectly classifying inmates is that it slows their progression through the corrections system. If an inmate requiring medium-security

[170] John Howard Society.
[171] John Howard Society.
[172] John Howard Society.

is placed in a maximum-security correctional facility, the regimen, programs, and rehabilitation programs provided may not coincide with the needs of the prisoner. This causes a slowing of the inmate's progression through the system. By being placed in a higher security facility, there is a lesser likelihood that the inmate will be released on good behavior. Oftentimes the inmate is labeled as a "failure to adjust", thus does not meet the requirements of the correctional system. This is in opposition to the correctional system meeting the requirements of the inmate for rehabilitation.[173]

There are high costs, both monetarily and in terms of rehabilitating the inmate, with incorrectly classifying inmates in a correctional facility. According to Antoniewicz, in Massachusetts correctional facilities, it costs on average $43,000 annually to house a minimum-security to medium-security inmate, while it costs $48,000 annually to house a maximum-security inmate .[174] The rehabilitative costs of housing inmates in a facility whose security level exceeds that required for the inmate are that possible extraneous rehabilitation, training, and treatment resources are being spent on the inmate.

This trend is prevalent in the Massachusetts correctional system. Between June 2002 and June 2004, five of ten minimum-security facilities were closed. Six hundred and thirty-two beds were lost due to the closings.[175] With the loss of these beds, prisoners who would have once been housed there as minimum-security inmates now are housed in higher security facilities. Antoniewicz's facts show that as a result, there was a 209 percent decrease in the number of inmates housed in minimum-security facilities

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[173] John Howard Society.
[174] Antoniewicz, Angela.
[175] Antoniewicz, Angela.

(while there was a 211 percent increase in the inmate population in maximum-security facilities).[176]

2.2.4 Costs of Overcrowding

In 2003, the Massachusetts budget allocated $816 million for higher education expenditures while the state corrections system received $830 million (DOC was budgeted $438.8 million).[177] Findings of the Massachusetts Taxpayers Foundation has shown that while the budget for the corrections system has seen substantial growth in recent years, the facilities are still operating over capacity. In fact, in the year 2003 the state correctional system was operating at 138 percent of capacity.[178]

In most jurisdictions, the conditions under which the inmates were kept were neglected during the rapid expansion of the prison population during the 1980s (see Figure 2.2.1 for the nation's inmate population growth trends). "Thus rather than improving living conditions and investing in prison programs and meaningful activities in which prisoners could participate, many systems have committed to harsh policies and procedures designed primarily to maintain order and control and little else."[179] In Massachusetts, the investment was in staffing. According to Antoniewicz, Massachusetts employees the second largest corrections staffing-to-inmate ratio; employing a staff-to-inmate ratio equivalent to 1:2 (as compared to the federal ratio of 1:4.3).[180] As a result, Massachusetts spends 73 percent of their DOC budget on labor costs while only three percent is budgeted for inmate programming.

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[176] Antoniewicz, Angela.
[177] Silberman, Ellen J.
[178] www.masstaxpayers.org
[179] Haney, Craig.
[180] Antoniewicz, Angela.

2.2.5 Conclusions

Prisons are essentially the holding ground of those who are socially disobedient and destructive. Therefore, it is the role of the correctional system to rehabilitate and train inmates before their release back into society. The overcrowding of prisons has many disastrous effects on inmates and on the correctional system itself.

In crowded facilities, inmates are often deprived of the essential resources and programs needed to rehabilitate them and to occupy their time. Aggression and tension arises from competition for such resources, essentially undermining the principals of the correctional system to instill good behavior and judgment into those who reside there. By not providing sufficient resources and programming, inmates receive inadequate, if any, rehabilitation and training upon their release into society.

Inmate behavior is greatly affected by the overcrowded situation in many of the nation's prisons. Many turn to social isolation and withdrawal as a means of 'escaping' the noises and interactions that surround them. This is a further hindrance to a successful integration into society.

The effects of overcrowding are also detrimental to the correctional facilities themselves. Besides being unable to sufficiently provide the required resources and programs, concerns for housing and finances come forth. Inmates may be placed in the correctional system not by their needs for safety and security but rather by the availability of open space. Furthermore, the costs of housing inmates annually are rather large and with the growth in the prison population, funding cuts in subsequent programs have to be made. Often, programs regarding inmate programs, which are already being stretched to meet the growing population, are being cut as seen in the Massachusetts inmate educational programs.

The effects of prison overcrowding are severe. The results are a cycle of disastrous consequences for the inmates: incarceration → release → reincarceration. Overcrowded facilities do not allow the correctional system to adequately perform its duties. The result is poor rehabilitation of inmates, increased expenditures, and the release of disobedient and destructive citizens back into society.

2.3 Privatization vs. Public Ownership of Correctional Facilities

In the past, correctional facilities were solely operated and run by either the state or federal government. However, a significant change took place in the mid 1980's when the privatization of prisons became more common. The uncontrollable overcrowding of prisons is believed to have sparked the interest in privatized prisons. In the early 1990's, it cost the government roughly six billion dollars per year to keep up with the constant growth in the number of prisoners.[181] In a privatized prison, the responsibility of keeping up with the growth rate of inmates is shifted to the private owner of the prison rather than the government. This will result in the private owner paying for any necessary changes needed to keep up with the growth rate of the inmate population.

There are many arguments for and against the privatization of prisons; however the economic considerations between the two are the most significant. The arguments for and against the privatization of prisons are listed in Table 2.3.1.

[181] http://mediafilter.org/caq/Prison.html

Table 2.3.1 Arguments For and Against Privatization of Prisons[182]

Arguments in favor of Privatization	Arguments against Privatization
• It costs less than a regular prison	• There is no gaurantee standards will be upheld
• It motivates the employee work force	• No one will maintain security if employees go on strike
• It creates a safer environment	• The public will have regular access to the facility
• It enables imates to make a profit and pay into restitution funds for victimes	• There would be different inmate disciplinary procedures
	• The company would be able to refuse certain inmates
• It raises more taxes for the state	• The company could go bankrupt
	• The company could increase their fees to the state

Many studies have been conducted to analyze the economic benefits of privatized prisons. These studies have all resulted in a consistent trend demonstrating that privatized prisons are more cost effective than conventional, publicly owned, prisons. Some researches even believe that private prisons not only save money, "…but they also put external pressure on the corrections system, further constraining the escalation of costs."[183] It has been estimated that privatized prisons can decrease the daily cost of an inmate by about 40 percent. This trend between private and public prisons can be seen in Figure 2.3.1 below.

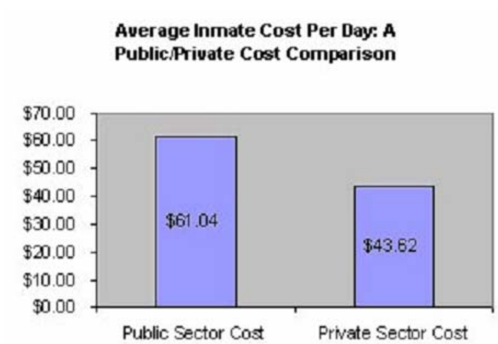

Figure 2.3.1 Cost Comparison Between the Public and Private Sectors[184]

2.4 Correctional Facility Design Considerations

The design and layout of a correctional facility is very involved and complex. Safety and security is of the utmost importance during the design of a prison, however

182 http://faculty.ncwc.edu/toconnor/417/417lect14.htm
183 http://www.rppi.org/competitionincorrections.html
184 http://www.reason.org/commentaries/segal_20030617.shtml

operational costs over the life-span of the facility also plays a major role in the layout of the building. "Each design decision made during preliminary planning and design phases can result in operational, staffing and maintenance cost savings over the course of the building life cycle."[185] The prison design must also be very versatile in order to accommodate various inmate needs.

2.4.1 Security and Visibility

Security guards employed to monitor the safety of a prison can be very costly over the life-span of the facility, which is why prisons are designed with this in mind. It is estimated that for every security guard added to the staff of a correctional facility, an additional one million dollars is added to the operating costs of the prison amortized over the facility's lifespan.[186] More security guards are needed to patrol areas of low visibility; therefore if the prison can be designed with increased visibility, then fewer security guards will be needed.

There are three levels of security in prisons, including maximum security, medium security and low security. Most prisons are strictly limited to one level of security in order to keep operation and design of the facility simple. Maximum security prisons limit the movement of inmates strictly, only allowing them out of the cell for about an hour to shower and/or workout. Medium security prisons are moderately secure, often times providing inmates with dormitory style living. Inmates in most medium security prisons are let out of their cells in groups and rotated in order to keep the number of inmates out of their cells limited. Finally, minimum security prisons are non-secure

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[185] De Chiara, Joseph and Crosbie, Michael J., p.833
[186] http://www.bls.gov/oco/ocos156.htm

dormitories that typically only have one perimeter fence that is patrolled on a regular basis.

Prisons are often designed with cellblocks that create a separation within the prison. Cellblocks can be used to separate women from men, inmates of different security levels and even sex offenders from non-sex offenders. Cellblocks often include what is referred to as a dayroom. The dayroom is a multi-purpose room that can be used as a cafeteria, a classroom, a library, a church and even a recreational area. The advantage of the dayroom is that it limits the movement of the inmates, which provides greater security throughout the facility. However, the disadvantage of the dayroom is that the services must now be brought to each cellblock, which requires more staff. The circulation of people and materials throughout the facility must be well planned and thought out in order to maintain security at all times.

2.4.2 Inmate Service Implications for Layout

Inmates by law deserve proper hygiene care and medical treatment even though they are in prison. Inmate services such as dental care, medical treatment, hair care, and others are often too costly to incorporate into each cellblock. Instead, a separate location is typically used outside the limits of each individual cellblock and the inmates are transported to this location to receive the treatment or service. The circulation of inmates from each cellblock to these services must be clear and unobstructed to maintain security.

2.5 Inmate Services

In order to effectively and responsibly operate a correctional facility there are certain functions that operators must be able to provide. These functions allow the correctional facility to support a certain level of living conditions among inmates. While no one would expect five star hotel accommodations in a prison, basic provisions must be established to maintain minimum standards of living so that the incarcerated are not subjected to cruel or unusual punishments. Some of these provisions include sufficient living space, showers, medical, food and laundry services, recreation and activity areas, etc.[187]

Many correctional facilities have detention areas with cells that are either single or double occupancy. Typically there is minimum floor area for each cell designated by the occupancy of the cell. The minimum floor area of 70 to 80 square feet for single occupancy is generally an acceptable size to allow inmates sufficient space to inhabit the cell. For double occupancy, the allowable floor area is approximately 100 square feet.[188] This minimum cell area is established to ensure that the cell sizes used in correctional facilities are humane and within acceptable standards of living. With the growing number of correctional facility inmates overcrowding can often become an issue, thus limiting cell sizes allows for the maximization of the facility's capacity.

In addition to having an acceptably sized living space, inmates must be allowed to maintain a minimum level of hygiene. As a result, showers with some privacy, with a sufficient shower to inmate ratio must be provided. For every eight inmates, there must be one shower to allow for proper safety and hygiene.[189] While sufficient number of showers may be available to the inmates, they must also be within a certain distance of the cells that they occupy.

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[187] Farberstein, Jay. p.238
[188] Congressional Budget Office
[189] De Chiara, Joseph and Crosbie, Michael J., p.840

Food service must also be provided to inmates since they are otherwise unable to provide for themselves while incarcerated. A certain amount of convenience must accompany food services, such that inmate need not travel too great of a distance to eat. As a result, in high security situations food will be brought to dayrooms of cellblocks or even individual cells. Bringing food to each cell increases the number of operating staff, but ultimately isolates inmates, thus decreasing the potential for inter-inmate incidents. The food provided must meet basic standards consistent with safe food service practices, including cold and dry storage of food, cleanliness, and vermin control.[190]

Considering the duration of the stay of most inmates in state and federal corrections systems is greater than one year, many of these individuals are likely to require some form of medical care. While outsourcing medical care to an outside medical provider could be a consideration for some small correctional facilities, the cost, safety and frequency of transportation for most prisons' medical needs justifies the placement of medical services in the facility itself. Because of the danger to personal health many inmates can pose in any range of security levels, a full range of medical services and appropriate staff should be available twenty-four hours a day.

One of the most important services provided to inmates of correctional facilities is rehabilitation programming. While punishment for one's crimes is an important factor of incarceration, rehabilitation of one's character is often more critical for most prison inmates. With the understanding that many individuals will be released back into society at the end of their sentence, the prospect to reintroduce a safe and productive member of society greatly outweighs the punishment of such persons. Rehabilitation often requires sufficient resources for counseling, classes, library, vocation, and recreation.

[190] Farberstein, Jay. p.238

Many persons being detained for crimes often times require counseling to help their rehabilitation for integration back into society. Consequently, professional counseling staff and appropriate facility provisions must be provided. These provisions may include isolated rooms to conduct counseling sessions. In addition to counseling, many inmates wish to remain in contact with individuals outside of prison walls. For this reason, areas are set aside to allow for contact, noncontact, and lawyer visitation.

While counseling may necessitate the allocation of rooms specifically designated for the purpose of counseling therapy, many activities can take place in multifunction recreational areas. These indoor recreation areas can be used for both physical activities, and educational and religious purposes. Flexible, multifunction areas can be easily converted from basketball courts to classrooms or a place of worship.

Inmate education is one of the most commonly utilized programs in corrections. This education can include earning a GED or learning a craft in vocational courses. Often times a library is included to support educational programming. The library also serves as potential sources of entertainment through reading. Because of the space required to store books, libraries are typically located in a relatively permanent location in the facility.

With an appropriate allocation of inmate services, many inmates can humanely be rehabilitated through the duration of their incarceration.

2.6 Correctional Facility Location Considerations

There are many concerns and effects associated when considering the location of a correctional facility. This section will cover the effects of placing a correctional facility in urban and rural settings. Additionally, attention will be given to the concerns that situating such a facility has on the general population.

2.6.1 Site Location Considerations

The proposed placement of a correctional facility is a politically and socially sensitive subject. The community at large has many questions and concerns about the location, which include reflecting their interest in preserving their neighborhood, safety, and investments. Also, the setting of the correctional facility, whether it be rural or urban, is important to consider. There are pros and cons associated with each type of location, both of which will be discussed. Table 2.6.1 provides basic considerations that should be given when choosing a suitable site to place a correctional facility.

Table 2.6.1 Basic Site Location Considerations[191]

• Access to community services • Access to and from courts • Access to local law enforcement • Access to relatives and friends for those in custody • Access for emergency vehicles and fire services	• Availability of infrastructure • Location to major roads and/or transportation networks • Access for facility staffing • Availability of usable land • Adequate soil conditions • Community acceptance

Access to competent staff to provide needed services is important. Correctional facilities have the responsibility to detain those who are socially disobedient and rehabilitate them. This can be achieved using many resources including education, rehabilitation, therapy, and training. The location of a correctional facility can make it tough to attract qualified personal for such resources.[192]

The location of a correctional facility with regards to local law enforcement agencies and courts is also critical. If a correctional facility is located at some distance away, considerations of transporting and handling inmates between the correctional

[191] Krasnow, Peter Charles. p.12
[192] www.nicic.org/pubs/

facility and the courts must be given.[193] This can result in added costs, which are accrued because of additional personnel and vehicles required in addition to the need for holding cells and visiting rooms in the vicinity of the court.

Access for the inmates' friends and family is also a consideration that must be made. If the facility was located in a remote location, the ability for inmates to receive visitors could be hampered. This could lead to withdrawal and depression for the inmates, undermining the responsibilities of a correctional facility to 'heal' those it houses.

The access to the correctional facility for emergency vehicles and fire services is essential for inmate safety and health. Also, if a correctional facility is located in a remote place, the number of people willing to serve as staff is decreased. Therefore, a correctional facility should be located as to attract people to work there.

The availability of local infrastructure and the proximity of major roadways are essential. Choosing a location that does not have the required infrastructure can result in additional costs and time delays in construction, services, and staffing. Furthermore, considerations regarding local roadways must be given because of the influx in traffic because of inmate transportation, inmate services, and visitation.

The availability of usable land with adequate soil conditions must be checked first and foremost. Considerations of site selection should reflect preserving natural formations such as wetland and floodplains.[194] Constructing a correctional facility in a site having wetlands and floodplains could have disastrous effects on the local ecology and natural drainage system if not properly designed for. The amount of land available has a tremendous effect on the building layout and site design. Also, considerations for

[193] Ricci, Ken. p.10
[194] Krasnow, Peter Charles. p.12

soils with adequate load bearing capabilities should be made to minimize foundation requirements.

Finally, community acceptance is a one of the most critical elements in successfully placing a correctional facility. Protesting citizens have the ability to hold up the progression of design and construction with petitions and lawsuits. It has been shown that a disgruntled community can hold up the development of the correctional facility as many as several years.[195]

2.6.2 Pros and Cons of Site Selection in Urban Locals

An urban area is defined as an area that has a high density of people and structures as compared to its surrounding areas.[196] There are many pros and cons associated with locating a correctional facility in an urban setting. This section will examine some of the benefits and drawbacks of locating such a facility in a city.

2.6.2.1 Pros of Urban Sites

Locating a correctional facility in an urban setting has many benefits for the facility and its inmates. Inmate demographics show that a leading majority of the prison population are minorities.[197] Figure 2.6.1 shows the national inmate demographic breakdown for 2002.[198] Many of those in prison are black men coming from low income families with little education and job experience.[199] Often, this population resides in the nation's cities. Locating a facility in an urban setting would give them a stronger sense

195 www.nicic.org/pubs/, p.10-11
196 http://en.wikipedia.org
197 Wilkinson, Reginald A.
198 www.hrw.org
199 www.lib.umich.edu

of home and family, making rehabilitation and integration into society more successful.[200]

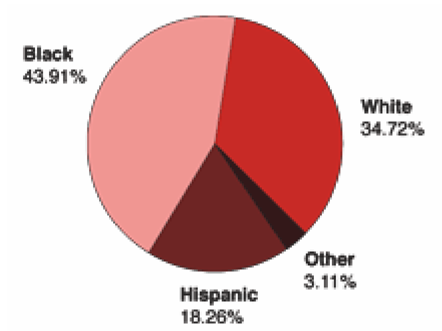

Figure 2.6.1 Inmate Population Demographics, 2002

A correctional facility in an urban environment allows the facility to take full advantage of its local resources.[201] These resources can range from rehabilitative sources and therapy, religious programs, educational sources, as well as employment opportunities. Advice, tutoring, and training are often available to facility staff from outside sources. The city's transit resources also become invaluable to the friends and family of inmates in terms of visitation. Staffing of correctional facilities is made easier when located in an urban setting. Cities have an abundance of colleges, big and small, many of which have some form of law enforcement/civil degree. Job placement of prospective students is heightened with the integration of a new correctional facility.

2.6.2.2 Cons of Urban Sites

Despite the benefits of locating a correctional facility in an urban area, there are also some major problematic issues. Often space is limited for correctional facilities in city locations. Therefore, sprawl within the facility is not capable. Instead of going out, everything must go up. Correctional facilities become taller, high-rise buildings; parking

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[200] Petersilia, Joan. p.4
[201] Wilkinson, Reginald A.

becomes more restrictive at times, requiring the construction of a parking garage to handle staff parking requirements at shift changes, and outdoor recreation yards are harder to place. Figure 2.6.2 shows a typical city correctional facility.[202] Additionally, concerns for future expansion must be considered for the design.

Figure 2.6.2 Hamilton County Justice Center, Cincinnati, OH

Another drawback of placing a correctional facility in an urban setting is the competition for employees.[203] The demands for specialized skills within a correctional facility are crucial to make it as self-sufficient as possible. Correctional facilities operate on a budget set by the state and/or federal government. Therefore, many facilities have a limit on how much they can offer perspective employees, losing the competitive battle to other institutions.

Regardless of the site location, community relations are a concern and often a problem for correctional facilities. This is of particular concern for city dwellers since the presence of a correctional facility is so integrated into everyday life. An institution needs to vigorously maintain a good image, which has the potential to be tarnished instantaneously should a security breach or event of a similar nature occur.

[202] www.charchitects.com
[203] Wilkinson, Reginald A.

2.6.3 Pros and Cons of Site Selection in Rural Locals

A rural area can be defined as of low density, away from the influence of large cities, typically characterized as an agricultural, logging, or tourist economy.[204] Locating a correctional facility in a rural locale has many perceived pros and cons, and these will be discussed in this section.

2.6.3.1 Pros of Rural Sites

The major benefit one hopes to achieve from locating a correctional facility in a rural setting is economic prosperity. Most of the United States' correctional facilities are located in rural areas. "States are increasingly making the need for economic development a major consideration in the prison siting process".[205] These areas have fought the hardest to attract such facilities in a vain effort to achieve economic renewal and job creation. Economic fallout of rural areas resulting from overseas manufacturing and farming along with the construction boom of correctional facilities during the late 1970's to 1980's has turned housing the socially disobedient and deviant into an industry for rural areas.[206] These areas that at one point were economically sound because of farming, logging, and manufacturing are now resurrected from their economical downfall associated with the change to large-scale farming and overseas manufacturing.[207]

Many communities see local prison construction as a positive rather than a negative in hopes of economic prosperity because of their ever-growing demand, increase in prison population, and their recession free nature.[208] In the 1990's, 245 prisons were

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[204] http://en.wikipedia.org
[205] Turner, Robert C. & David Thayer. p.2.
[206] Street, Paul.
[207] Turner, Robert C. & David Thayer. p.3
[208] Turner, Robert C. & David Thayer. p.3

built in 212 (of 2,290) rural counties.[209] These facilities provided a sense of quick relief and opportunity. This economic boon is evidenced in Fremont County, Colorado. Fremont County had 9.2 percent unemployment rate in 1987 prior to any correctional facility construction. The county paid $100,000 for 600 acres, which they donated to the Federal Bureau of Prisons for use as a prison site. As a result of the facility construction, the unemployment rate dropped to 5.3 percent. The county also allotted $40,000 to attract new businesses to the area, resulting in a 34 percent increase in revenue.[210]

2.6.3.2 Cons of Rural Sites

Despite the potential benefits of economic renewal associated with prison construction, there are many adverse consequences associated with correctional facility construction in rural locals. Often, the required infrastructure is not available or is insufficient.[211] The basic utilities of sewer, water, electricity, telephone, cable, and road access must be provided, resulting in increased costs and construction time.

Locating a correctional facility in a city would better serve law enforcement because more criminal activities occur in urban areas. The heightened operational costs associated with transporting inmates to and from the rural correctional facility to their court appearance is another downfall of having a facility in a rural area.[212] Because of the need to transport inmates to the courthouse, holding facilities and other amenities associated with court appearances must be provided at the courthouse.[213] These items would not have otherwise been required had the correctional facility been located next to or in close proximity to the courthouse, as they typically are in a downtown, city location.

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209 Street, Paul.
210 Nadel, Barbara A.
211 Ricci, Ken. p.11
212 Nadel, Barbara A.
213 Ricci, Ken. p.10

The largest problem associated with placing a correctional facility in a rural setting is the disproportionate makeup of the prison population. Referring to Figure 2.6.1, African-Americans constitute a leading majority (nearly 50 percent) of the nation's prison population.[214] In Massachusetts, only 5.4 percent of the total population is black, yet 26.3 percent of its prison population is black. "Reflecting the strong correlation between blackness and urban residence",[215] locating a correctional facility in rural counties results in taking many of the prisoners away from their home, family, and friends. In New York, three-fourths of the prisoners come from the state's urban region but 91 percent of them are housed in upstate New York facilities with 80 percent of the inmate population being African-American.[216]

Locating a facility in an out-of-town location can have disastrous consequences on the inmates imprisoned, while they are incarcerated and upon release. Visitation can be limited because of longer travel time as well as the relative inadequacy, or unavailability, of public transportation to and from rural locals. Visitations are critical for inmates during their incarceration period. Studies have shown a correlation between improved inmate behavior and lower recidivism rates with family and friend visitations.[217] Upon release, if a correctional facility is located rurally successful reintegration into society is hindered because finding a job and a place of residence is often much more difficult when a prisoner is away from their urban home.[218]

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[214] www.hrw.org
[215] Street, Paul.
[216] Street, Paul.
[217] Kupers, Terry A.
[218] Lawrence, Sarah & Travis, Jeremy. p.33

2.6.4 "Not in My Backyard"

"It is argued that when society adopts a policy that will entail certain costs, these costs should be distributed fairly among all members of society."[219] Placing a correctional facility imposes numerous costs. Many feel that a local facility will result in lowering property value, increased traffic resulting from inmate visitation, and even a change in the demographics of the local population.[220] Generally, the location of a parcel of land to a popular attraction increases its value, while its proximity to an unpleasant attraction lowers it. Public safety is a concern for many, with many local residents fearing the threat of inmate escapees and the intrusion of the "criminal element" into their community.[221]

As a result, prisons are often referred to as NIMBYs, not in my backyard, or LULUs, locally unwanted land use.[222] The off-the-cuff reaction of many local officials and citizens is to place a correctional facility on the cheapest, most remote piece of land available.[223] This is because of the greater public's barbaric image of prisons from years past. Many imagine a correctional facility as a dark and cold fortress, surrounded by massive towers, walls, and barbed wire. Today's modern facilities can easily be designed to blend into the local settings. Figure 2.6.3 shows such an example, the Lexington/Fayette County Detention Facility in Lexington, KY.[224] This facility located along a scenic root amongst some of Kentucky's most prestigious agrarian enterprises was designed to resemble a horse farm.

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[219] Andre, Claire & Velasquez, Manuel
[220] Ricci, Ken. p.8
[221] www.nicic.org/pubs/, p.9
[222] Rephann, Terance J. p.1
[223] Ricci, Ken. p.5
[224] www.dmjmhn.aecom.com

Figure 2.6.3 Lexington/Fayette County Detention Facility, Lexington, KY

Surprisingly, prison construction does not provoke as many NIMBY activists as one would expect.[225] This is because many rural areas have a lagging economy, muting their resistance to NIMBYs and LULUs. Many feel that by allowing for such "distasteful" facilities to be constructed in their locale, job creation and economic prosperity will prevail (see section 2.6.3.1 Pros of Rural Sites).

A study by the Florida Atlantic University, the Florida International University Government Center for Environmental and Urban Problems examined the effects of a correctional facility on local property values, public safety, the economy, the quality of life, and local law enforcement.[226] The study found that while correctional facilities did not have a negative effect on any of these issues, it also did not have a positive effect either. Crime rates remained virtually the same, with little deviation from the norm. Housing costs also remained stagnant. The economic prosperity that many felt they would achieve was not fostered. Most if not all the jobs created by the correctional facility are received by those living outside of the community. This is because of the need for trained and skilled staff possessing a certain level of experience. Furthermore, many employees would rather commute than live in the town in which the correctional

[225] Nadel, Barbara A.
[226] www.nicic.org/pubs/, p.2-8

facility is located.[227] Often, many construction materials are brought in from outside sources because of the bulk quantities needed and the "cost-cutting economies" of those supplies,[228] thereby hindering the sales of local merchants. Often, business is brought into the region, but typically in the form of chain stores and fast food restaurants. Correctional facilities can also be a deterrent for stimulating new economic opportunities.[229] New economic opportunities such as big business often turn away from towns having a correctional facility because of their image of being "undesirable neighbors". Although it has been shown that a correctional facility may not hurt the safety, housing values, and economy of a region, it also does not help it either having the potential of turning away any prospective economics opportunities.

2.6.5 Conclusions

There are many considerations that must be made when specifying the location of a correctional facility. Operational concerns are a major issue. The adequacy and availability of local infrastructure such as water works, sewer, electricity, and roadways must be verified to be sufficient for the correctional facility. If they are not, then improvements must be made, adding both time and cost to the project. Access and availability for staffing and other resources is also a concern. For ease of operation, a facility should be located to attract employees as well as to make it accessible for outside sources to be brought.

Public acceptance is also critical for the success of siting a correctional facility. Opponents of the prison construction have the ability to extend the development, design,

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227 Turner, Robert C. & David Thayer. p.7
228 Street, Paul.
229 Turner, Robert C. & David Thayer. p.8

and construction time by picketing and lawsuits. Concern for personal land value, safety, and retaining community identity is often the leading issues of NIMBY. To elevate these concerns and to lessen the haste between choosing a site and opening the facility to inmates, management of public opposition is crucial.

Choosing the location, whether it be urban or rural, has many consequences. Often, public acceptance, accessibility, and resources can be defined by the location of the correctional facility. In addition, the location of the facility has an effect on the demographics of the area since a majority of inmates come from urban locals. Site selection can ultimately have an effect on inmate rehabilitation and release.

2.7 Security Considerations

Correctional facilities use multiple methods and practices when approaching the security of the compound. These elements can take the forms of perimeters, electronic devices, the building itself and the practices utilized to run the facility.[230] All of the aforementioned elements need to be considered and particular methods for applying them need to be determined during the design phase of construction. For example, if the facility is to be located in an urban area, then the perimeter considerations would be much different than those of a facility located in a rural area. The site location and the type of prisoners play a major role in the security considerations that need to be accounted for in the construction. Additionally, the architecture of the building can play a role in the security.[231] Unblocked sight-lines for an entire cellblock both minimize the number of guards needed to secure the area and allow the staff to gain a better view of the actions of the inmates while remaining in a more secure area.

[230] Krasnow, Peter, p.3
[231] Krasnow, Peter, p.3

2.7.1 Security Breakdown

Peter Krasnow, author of *Correctional Facility Design and Detailing*, notes that a correctional facility can be broken down into five different security zones.[232] Each of these zones has its own special purpose and security level. It is essential that all movement within the facility as well as entry and exit be monitored by the staff. The first security zone centers on the monitoring of the building perimeter. These areas include all public spaces as well as administrative areas and employee services. The monitoring of this zone can be done via direct staff observation and/or with electronic support. The second zone is the security perimeter. This area consists of the main building or the perimeter fencing. Essentially all areas where the inmates would be located are monitored in this zone. Access through this zone is usually restricted by sally port and overseen by centralized security personnel. The third zone deals with local or unit control. The purpose of this zone is to monitor movement in and out of housing units (jails) and/or buildings (prisons). The unit control monitors and manages movement of detainees, staff, and other authorized personnel. The fourth zone is the housing unit or pod control. The duties of security in this zone fall to a control officer who oversees inmates partaking in activities in the unit. The fifth and final zone in the correctional facility security zone hierarchy is the one that deals with the inmate cells. Security for this area is provided by the unit control officer who oversees access to the cells. Cell access is determined by the classification of the offender and the operating policy of the facility.

[232] Krasnow, Peter, p.4

In regards to security operations as a whole, correctional facilities often apply a hierarchical approach to the distribution of responsibilities. The most centralized form of security monitoring is referred to as master. Master security considerations are summarized in the following table.[233]

Table 2.7.1 Master Security Considerations

Master Security Considerations	
Monitor facility access	Control perimeter access
Monitor overall facility security	Control major area movement within facility
Coordinate facility's internal communications	Coordinate emergency situations
Monitor smoke/fire detection systems	Control emergency exits, doors, gates, etc.
Monitor outside facility security	

The second tier of correctional facility security is that of local unit supervision. Under this category fall a variety of responsibilities of entry and egress from the housing units of the correctional facility. These duties are summarized in the following table.[234]

Table 2.7.2 Local/Unit Security Considerations

Local/Unit Security Considerations	
Monitor and control inmate and staff movement into and out of housing units	Coordinate visitation with pod officers and direct visitors to same area (jails only)
Coordinate inmate and staff movement to services provided in the housing unit	Monitor smoke/fire detection systems (coordinating with Master
Coordinate inmate housing counts between housing pods with Master	Override security and locking systems/devices in housing units

The third tier of security for a correctional facility relates to the housing of the inmates and the supervision required to maintain a safe atmosphere for both the detainees and the staff. All of the considerations that fall under this category deal directly with the small unit in which the incarcerated spend the majority of their time. The table below summarizes the activities that are involved with this level of security.[235]

Table 2.7.3 Housing Security Considerations

Housing (Pod) Security Considerations	
Supervise all inmate activity within the pod	Control all cell doors in pod
Respond to inmate requests within the pod	Monitor smoke/fire detection systems
Coordinate and provide backup to unit control for emergency situations	Control of staff and inmate movement in and out of living unit (coordinate with

[233] Krasnow, Peter, p.4
[234] Krasnow, Peter, p.4-5
[235] Krasnow, Peter, p.5

The final level of security that is taken into consideration deals with the intake and release of the inmates. For obvious reasons, controlling this aspect of the facility is of extreme importance as allowing inmates to escape raises serious concerns within the correctional facility compound as well as in the communities neighboring the facility. The aspects dealt with in this level of security are summarized in the following table.[236]

Table 2.7.4 Intake/Release Security Considerations

Intake/Release Security Considerations	
Monitor and control vehicle access in and out of vehicular sally port (could also fall under master control	Control access from admission/release areas into facilities main circulation corridor
Provide backup for emergency situations in this area (Coordinate with Master)	Monitor smoke/fire detection systems in this area (Coordinate with Master)

2.7.2 Methods of Supervision

The methods of supervision utilized by the correctional facility currently take two different forms: direct and indirect supervision. Direct supervision relates to the utilization of a guard to monitor inmate day-to-day activities.[237] This method is used mostly within the housing units where the guard controls access to the cells and activities in the day room. There are generally no physical barriers between the guard and the inmate population. This allows for a sense of normalcy for the inmates and helps diffuse tension and build a rapport between the inmates and the staff. Three key components of direct supervision are that the inmates are aware they are being watched; the inmates know that if they cause problems, those problems will be quickly eliminated by placing them in higher security cells; and finally that the inmates are aware the officer in the unit is backed up by electronic monitoring systems.[238]

[236] Krasnow, Peter, p.5
[237] Krasnow, Peter, p.xviii
[238] Beck, Allen R, http://www.justiceconcepts.com/design.htm

The second form of supervision is indirect supervision.[239] It is generally accomplished by utilizing electronic supervision and central observation areas. Indirect supervision typically occurs in areas that are separated from the correctional facility populace by hard barriers such as walls or glazed panels. The officer stationed in the supervision area controls access to rooms and housing units and monitors inmate activities. Communication with inmates occurs via intercom. In the event of major incidents additional backup can be easily called for from this post.

In terms of successful usage, direct supervision has been found to reduce "violence and vandalism and to create a more normalized living environment."[240] In many cases, indirect supervision is used to back up direct supervision.[241] This provides the officers working in the housing units of the prison an extra layer of protection. The usage of indirect supervision cuts down on labor costs due in part to the fact that multiple smaller living units can be monitored by a single individual rather than having multiple guards watching over those smaller units.

2.7.3 Major Design Issues

There are many aspects to be considered when determining a layout for a correctional facility. Krasnow writes that "[a] facility that remains flexible and continues to function as planned is often a good test for a successful design."[242] Oftentimes, layouts tend to be simple and provide good sight clear sight lines. Allen Beck (Ph.D in Criminal Justice) states the following as a major guiding principle of jail design: "Jail

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[239] Krasnow, Peter, p.xviii
[240] Krasnow, Peter, p.xviii
[241] Krasnow, Peter, p.xviii
[242] Krasnow, Peter, p.118

design should be based on direct or indirect supervision of inmates. Linear design should be absolutely avoided."[243]

The benefits of direct and indirect supervision are discussed in the previous section. Linear design is essentially the alignment of cells along a long corridor with a patrolling officer monitoring the inmates in the cells.[244] The major issue with linear design in comparison to direct and indirect supervision from a security stand point is that it becomes difficult for inmate problems to be detected and prevented because of the difficulties for the guard to supervise the entire hallway constantly.[245]

Apart from layout, there are other major interior design issues that impact security. The location of inmates' services, activities, and programs should be centrally located or ideally located in the same space.[246] This is because monitoring and controlling inmate activities is done much more easily when the inmates are not being escorted long distances or need to be moved frequently from activity to activity. Along this vein, the exercise locations for each housing unit and the visitation rooms should be located adjacent or in close proximity to the housing area for the same reasons. Visitation rooms are typically restricted to no contact visitation.[247] This means that the inmate and the visitor are separated by a hard barrier. This type of visitation deters the exchange of contraband. Should contact visitation be allowed, an intensive search and screening process is required.[248]

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[243] Beck, Allen R.
[244] Beck, Allen R.
[245] Beck, Allen R.
[246] Krasnow, Peter, p.120
[247] Krasnow, Peter, p.122
[248] Krasnow, Peter, p.122

For security reasons, the ceiling and corridor heights must be kept to a minimum height of ten feet, with eleven feet recommended.[249] This is done to prevent inmates from scaling the walls and accessing the interstitial space where they could potentially escape or hide contraband and weapons. Additionally, acoustical ceiling tiles or their equivalent should be set in place to minimize noise.

Security for service and loading docks is a major consideration for correctional facility design. These areas require intense supervision, observation and control of activity.[250] This is because the nature of this area makes it extremely vulnerable to contraband transfer and escape. The number of vehicles in loading areas should be limited and the loading docks should be located close to the respective service areas that they serve.

In addition to the interior security challenges presented by interior design of the correctional facility, there are a number of exterior issues to be taken into account as well. These include perimeter fencing, patrol vehicles, and the distance between fencing and the buildings amongst others.

In regards to the perimeter security, continuous fence lines or patrol roads provide the best option.[251] The lines of the fencing or roads should be made so that sight lines are as straight as possible to minimize the number of guards required to supervise these areas. The perimeter guards should be located such that they can see the most amount of area from their vantage point.[252] In addition to having guards directly monitor the exterior

[249] Krasnow, Peter, p.122
[250] Krasnow, Peter, p.122
[251] Krasnow, Peter, p.118
[252] Krasnow, Peter, p.118

perimeter of the facility, close circuit television and other monitoring devices can be used.[253]

Guard towers are often utilized in the perimeter security of larger correctional facility complexes.[254] The major responsibility of those located in the guard tower is to observe, control, and prevent entry into the perimeter of the correctional facility. Guard towers can be employed such that there are many towers covering all angles of entry around the perimeter of the facility. Another option is to use a main guard tower and patrol vehicles to secure the perimeter. Patrol vehicles allow for faster response to incidents than guard towers alone and can be more cost effective over the lifespan of the facility.[255]

The distance between the main buildings and the perimeter fencing should be as large as possible.[256] Typical fence distances are two hundred to three hundred feet from the exterior of the buildings in suburban or rural areas. The minimum distance from the correctional facility exterior should be kept to twenty to twenty-five feet. The goal of the perimeter fencing in the minimum case is to keep people away from the exterior wall of the facility. Should the exterior of the facility constitute the perimeter, windows should be located at the greatest height possible.[257] The goal of the exterior wall in this case should be to eliminate or minimize contact between the outside world and the inmates.

Perimeter access to the facility should be kept at a minimum. More points of entry on the perimeter translate to more potential security breaches. Ideally, there would be one point of pedestrian access and one point of vehicle access to the facility and one

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[253] Krasnow, Peter, p.118
[254] Krasnow, Peter, p.118-119
[255] Krasnow, Peter, p.119
[256] Krasnow, Peter, p.119
[257] Krasnow, Peter, p.119

point of access through the security perimeter.[258] Any time the access road or sidewalk branches there should be a security checkpoint to ensure visitors, employees, deliveries and new inmates are being brought to the appropriate location.

2.7.4 Material Testing and Criteria

Due to the conditions to which the structure and furnishings of a correctional facility are subjected, a higher level of quality and durability is required than in typical residential or commercial construction. The architect must specify the appropriate materials and furnishings to create a safe environment for the inmates and staff.[259] Typically a defense equipment contractor (DEC) is employed as an expert to ensure that all codes are satisfied.[260] The DEC coordinates the security hollow metal with all security hardware and electronics. In addition the DEC makes sure that the aforementioned aspects interface properly with the rest of the correctional facility components such as electrical, mechanical and structural systems. The DEC makes sure that security walls, hollow metal, glass, ceilings, hardware and perimeter fencing are all on par with the requirements needed to provide the desired level of security for the correctional facility.

2.7.4.1 Security Walls

There are many types of security walls that can be used in correctional facility construction. All of these types of walls must be constructed of impenetrable materials.[261] Typical systems are pre-cast or cast-in-place concrete, concrete masonry

[258] Krasnow, Peter, p.119
[259] Krasnow, Peter, p.182
[260] Krasnow, Peter, p.182
[261] Krasnow, Peter, p.183

units (CMU), and steel panels. Pre-cast and cast-in-place concrete walls must have a concrete compressive strength of at least 4000 psi and a minimum thickness of four inches.[262]

CMU walls need to meet different criteria depending upon the type of inmate the correctional facility is housing. Maximum security walls need to be constructed such that the voids of the CMU are filled with grout and number four reinforcing bars are placed at eight inches on center both horizontally and vertically.[263] Medium security CMU walls need to have all voids filled with grout and number four reinforcing bars placed at sixteen inches on center vertically. Minimum security walls need only have typical grout conditions and number four reinforcing bars placed at sixteen inches on center. The mortar used in this construction should be 2,500 psi type M masonry mortar and the rebar should be anchored at the floor and ceiling.[264]

Steel bar security walls are often used as cell fronts and security checkpoints in corridors.[265] These walls should be constructed of tool resistant steel and welded to metal plates embedded in the walls for anchorage. Steel panels are also used in wall construction. This type of wall should be a minimum of three-sixteenths inches thick and can be fabricated of hollow metal steel.[266]

Other wall options include woven wire mesh, security gypsum board, and glass blocks. Woven wire mesh can be used for cell fronts in areas that are less than maximum security.[267] It is also used for holding cells, tool crib storage, and partitions in large activity areas. Security gypsum board is essentially at least two layers of gypsum board

[262] Krasnow, Peter, p.183
[263] Krasnow, Peter, p.183
[264] Krasnow, Peter, p.183
[265] Krasnow, Peter, p.184
[266] Krasnow, Peter, p.184
[267] Krasnow, Peter, p.184

with heavy expanded mesh set between the layers.[268] Security gypsum board is only used in areas where inmates will vacate after short periods. Glass blocks can be used to provide natural light to facility spaces and come in two different variations.[269] The first is the solid type of glass block, which is usually put in window frames and can be used in secure areas. The second type of glass blocking is a hollow unit that can only be used in non-secure areas.

2.7.4.2 Security Hollow Metal

The hollow metal used in a correctional facility must be durable and able to withstand extreme conditions due to inmate abuse. There are multiple tests to which the hollow metal is subjected, and these tests are summarized in the table below.[270]

Table 2.7.5 Security Hollow Metal Tests

Security Hollow Metal (Door and Frame) Tests	
Test Type	***Description***
Static Load	Tests maximum deflection and door strength
Rack	Tests maximum allowable deflection and how far an inmate can pry the door open
Impact Load	Tests door behavior under riot conditions, ramming device hits door
Removable Glazing Stop	Ramming device hits glass, makes sure glass stop is as strong as rest of door
Bullet Resistant UL-742	Tests door for bullet penetration or breakage that could cause injury

The gauge of the material used to construct the face sheet of doors in the facility is 14 gauge.[271] Doors should be constructed to resist the types of attacks and abuse that occur in a correctional facility. In addition, inmates should not be able to hide contraband or weapons in doors, and the doors should have a flush closure channel that has been welded and dressed smooth.[272]

[268] Krasnow, Peter, p.184
[269] Krasnow, Peter, p.184
[270] Krasnow, Peter, p.184
[271] Krasnow, Peter, p.185
[272] Krasnow, Peter, p.185

Much like the doors, hollow metal frames for the construction facility should be made of at least 14-gauge steel and be fastened securely to the wall.[273] This is important because the frame must be able to withstand the same abuse as the door it is holding.

2.7.4.3 Security Glass

The goal in providing a specific level of resistance to attack of security glass is not to prevent escape or breach entirely but merely to elongate the amount of time escape or breach takes so that the situation can be put under control.[274] Security glass is used to prevent the exchange of contraband and is commonly used in non-contact visiting booths and as windows in the building. The architect is responsible for determining the size, location and degree of security for the glazing used in the facility.[275]

Much like security hollow metal, security glass is subject to a number of tests. Typical test are presented and described in the following table.[276]

Table 2.7.6 Security Glass Tests

Security Glass Tests	
Test Type	***Description***
UL-742 Bullet Resistant	Test resistance to projectiles, their fragments and if complete penetration occurs
UL-972 Burglary Resistant	Tests resistance to a 5 lb. ball dropped at 10' and 40'
HP White	Tests resistance to blows from random objects (chisels, fire extinguishers, etc.)
WMFL	Similar to HP White but with a time limit to attacks (30, 60 and 60 minutes w/ bullet resistance)
ASTM 1233	Tests ballistic impact, blunt tool impact, sharp tool impact, thermal stresses and chemical decay

There are many types of security glass used in correctional facilities today. One type is polycarbonate plastics.[277] Polycarbonate plastics provide a high degree of security and low cost. These plastics can be applied in many layers and can be constructed with a two hour fire rating. All conditions dealing with the fire retardant nature of various

[273] Krasnow, Peter, p.185
[274] Krasnow, Peter, p.186
[275] Krasnow, Peter, p.186
[276] Krasnow, Peter, p.186-187
[277] Krasnow, Peter, p.186

materials will be discussed in section 7 Fire Considerations. The major problem with this type of security glazing is that it is easily scratched and can be burnt.

Glass laminates are also used in correctional facilities. This mode of security glazing consists of multiple layers of glass bonded together with an inner layer of wire mesh or fiberglass material.[278] Glass laminates provide a moderate degree of security and under heavy attack, penetration with sharp metal objects is possible.

Two other forms of glazing material common in correctional facility construction are replacement glass systems and glass clad polycarbonates. Replacement glass systems consist of polycarbonate, air space and a piece of sacrificial heat treated glass.[279] This system is held together with a bonding strip of foam and is not hermitically sealed thus is unadvisable for exterior use. Glass clad polycarbonates are polycarbonate and glass bonded together with a urethane interlayer and are very widely used in correctional facility construction.[280]

2.7.4.4 Security Windows

Windows present an easy mode of escape for inmates if they are not designed correctly. Knowing this, certain considerations are taken into account when specifying windows for a correctional facility. The first major consideration is that the windows be anchored so that they cannot be removed from the window opening.[281] The anchors holding the window in place must be able to withstand attempts at escape from inmates.

Two of the most popular window frames used include split window frames and casting the frame into a pre-cast concrete panel. Split window frames are secured to the

[278] Krasnow, Peter, p.186
[279] Krasnow, Peter, p.186
[280] Krasnow, Peter, p.186
[281] Krasnow, Peter, p.187

structure via weld plates to which clip angles installed by the manufacturer.[282] The other option allows for the window frame itself to become part of the curtain wall structure, thus it is more difficult to remove. In all cases, the typical steel used is either 12 or 14 gauge.[283]

2.7.4.5 Security Ceilings

Older correctional facilities relied on concrete, security plaster and steel plates to cover the interstitial space. More modern trends have lent themselves to lighter, thus less expensive alternatives. The general purpose of the security ceiling in the correction facility is to keep inmates from exploiting the interstitial space for escape or contraband and weapons storage.[284] Considerations governing what type of ceiling should be set in place include the supervision within the space, clear height from floor to ceiling, the accessibility of the ceiling to inmates, and the type of inmates that will be in the particular area.[285]

The types of ceilings utilized in correctional facilities include metal security ceilings, security gypsum board, metal panel ceilings, and hollow metal ceilings along with more traditional ACT. Metal security ceilings are typically composed of 14, 16 or 18 gauge steel that is twelve inches wide and interlocking or mechanically fastened together.[286] Minimum specified requirements for the system include being able to withstand a 40 psf uniform live load, a concentrated load of 420 pounds and have a maximum deflection of L/240. A second type of ceiling material is security gypsum board. This system is similar to that used for security walls in that two layers of gypsum

[282] Krasnow, Peter, p.187
[283] Krasnow, Peter, p.187
[284] Krasnow, Peter, p.187
[285] Krasnow, Peter, p.188
[286] Krasnow, Peter, p.188

board encase a layer of expanded wire mesh.[287] Access panels for this option and all other options should be composed of security grade access panels with heavy duty hinges and security locks. Metal panel ceilings should be comprised of 18 to 20 gauge steel or aluminum, be two feet by four feet or two feet square, and install like their commercial counterparts. Hollow metal ceilings are constructed in much the same manner as hollow metal doors for construction facilities.[288] Additionally, truss or high hat reinforcing can be incorporated into the ceiling design.

Security gypsum board tends to be used in lower security areas, where as the metal panel and plank options can be used in more secure areas.[289] Hollow metal ceilings are typically used in maximum security applications.

2.7.4.6 Security Hardware

There are many important considerations that need to be taken into account when specifying the type of hardware to be used in a correctional facility. Life safety and fire codes come into play when regarding this matter as egress from the building must be accommodated for in case of an emergency. A list of the most important considerations for security hardware selection is provided below.[290]

- Determine which doors require security hardware
- Determine which doors require electrical or mechanical operation
- Determine points of control for electronically operated doors (primary and secondary locations)
- Determine the level of security required
- Determine the type of key required: paracentric or mogul
- Determine which of the doors are interlocked
- Establish emergency exit plans

[287] Krasnow, Peter, p.188
[288] Krasnow, Peter, p.188
[289] Krasnow, Peter, p.188
[290] Krasnow, Peter, p.189

There are many types of security locks that can be used in a correctional facility. One type is the mechanical type locks. These locks are used where officers are manually opening the doors for inmates.[291] Where double doors are concerned, a Cremone bolt is used.[292] This type of bolt is similar to a head/foot bolt but provides greater security.

A second type of lock utilized in a correctional facility is the electrical type locks. These are used where remote locking and unlocking is required.[293] Doors are wired so that they can be opened individually or in groups. This type of lock is good for providing emergency release for egress. The third type of locking mechanism is the sliding door type lock. This type of lock is used for sally-ports and allows the door to slide open without the need to push or pull it.[294] This type of lock can be wired for electronic use or can be operated manually. The final type of lock used in correctional facilities is a pneumatic lock. This type of lock functions by sending air through nylon hose to activate the lock.[295]

The type of locks chosen should reflect the level of security desired for a given area. Other factors that come into play when selecting locks are where the inmate will end up if the lock is breached, the amount of abuse the lock will get and the characteristics of that lock. Areas where inmates need to be contained en masse need to have more rugged locks than other areas because the potential amount of abuse is much higher.

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[291] Krasnow, Peter, p.189
[292] Krasnow, Peter, p.189
[293] Krasnow, Peter, p.190
[294] Krasnow, Peter, p.190
[295] Krasnow, Peter, p.190

Hardware accessories used in the correctional facility should have the same strength characteristics as the doors and frames to which they are attached.[296] There are a plethora of hardware accessories that need to be considered in correctional facility design. These include security door pulls, indication switches (to monitor the position of the bolt), door position switches (to monitor if the door is open or shut), door closers, and should wooden doors be used, door strike plates to prevent the inmates from breaking through the wooded doors.[297]

2.7.4.7 Perimeter Security Materials

The security perimeter is put into place to keep the inmates from escaping and to keep the general public from entering the site. Fencing should be designed to provide a high level of security and provide the physical height and materials to prevent breach.[298] The main goal of the fence is to prevent anyone from climbing over it, digging under it, or cutting through it or the supports. In addition, special care must be taken when regarding locations where the fence line is broken by buildings or other structures.[299]

The typical fencing system for a correctional facility is set up as follows.[300] The main body of the perimeter security system is composed of the fence, which has two tiers, and interior and exterior. Both lines of fencing are usually capped with barbed tape coils of one type or another at the top. A concrete grade beam is placed under the inner fence line of the perimeter to prevent inmates from digging under the fence. An alternative to this is running the fence below grade. The depth of the grade beam or fencing depends upon soil conditions. The area between the interior fence and the exterior fence is kept

[296] Krasnow, Peter, p.191
[297] Krasnow, Peter, p.191
[298] Krasnow, Peter, p.193
[299] Krasnow, Peter, p.193
[300] Krasnow, Peter, p.193

free of vegetation and there are usually rolls of barbed tape or razor wire run along the length of these spaces. The outer fencing tends to be higher than the inner fencing and will have multiple barbed tape rolls at the top and bottom.

Typical perimeter fencing materials have basic specifications that need to be followed. Wire mesh must be nine-gauge, with maximum security possibly needing a higher gauge, and have gaps no larger than two inches wide.[301] Barbed tape comes in three different configurations and is typically used as a visual and physical deterrent.[302] The three different types of barbed tape are spiral coil (single helical loop), single coil concertia (single helical loop with clips to make loop spacing smaller), and double coil concertia (one coil 30 inches in diameter and a second inside at that is 24 inches in diameter).[303]

[301] Krasnow, Peter, p.194
[302] Krasnow, Peter, p.194
[303] Krasnow, Peter, p.194

3 Methodology

3.1 Office Layout

The strategy for deciding the column spacing options for the administrative tower structure was based on the desire to make the space as versatile as possible and to keep the structure feasible in terms of cost of construction. This meant balancing the need to minimize the number of columns placed in the building with trying to keep the column spacing from getting too large to avoid large and expensive members. The rationale behind this was that the larger the member length, the greater load it would have to support and the more substantial it would be in terms of size and cost.

The administrative tower was designed to serve a multitude of support functions for the adjoining detention center. The first floor of the building was planned to serve as a lobby and reception area and a security checkpoint for persons not incarcerated to gain entry to the detention center. Essentially any visitors and persons who are not employed by the correctional facility will enter the detention center via this entry and this entry only. The second, third and fourth floors were planned to house the main administrative functions such as accounting and operational supervisors necessary to manage a fully functional correctional facility. Thus this area was planned to be a versatile office space with movable partitions should reorganization of the floors be desired. The uppermost floors were designed with the needs of the correctional facility staff in mind. The area was designed to serve the needs of the guards in a variety of different ways. It was intended to provide them with training space, and to include a library and workout area in addition to a place to sleep should the need for keeping off-duty guards on site arise. These floors were designed as office floors as well in case of the event of organizing of the building should occur. This allows for reorganization of the building to occur not just on a single floor, but throughout the entire building.

In order to meet the desire to provide a versatile space, large open spaces were provided by minimizing the number of columns on the short side to four for the entire sixty-five foot length. Along the long side of the building, two different column spacing options were compared. In order to effectively compare these two layouts, means of egress (i.e. stairwells and elevator banks) were kept in the same locations and the rest of the space was designed around them. Visual depictions of each layout can be found in the following paragraphs along with dimensionless members. Accompanying these visual representations is a brief written summary of each layout.

The first scenario developed with the exterior bays being ten feet wide and the interior bays being sixteen feet wide (10'-16'-16'-16'-16'-16'-10'). The short side column spacing was set up such the spacing as 20'-24'-20'. This layout provided central areas of 250 square feet on the ends and four hundred square feet in the interior. The exterior areas were 200 square feet and 320 square feet for the exterior and interior regions respectively. Figure 3.1.1 is a plan view of option one for the office.

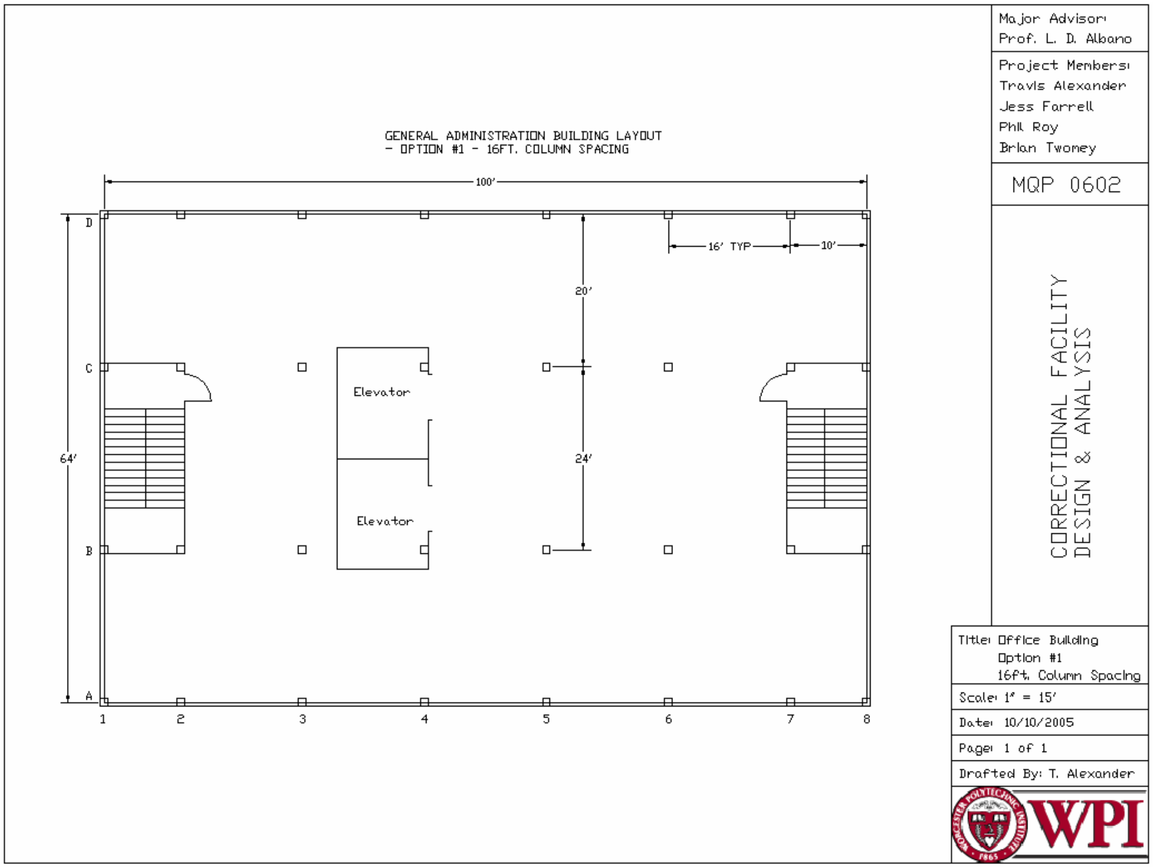

Figure 3.1.1 Office Layout, Option 1

Option two was a simple repetitive twenty foot column placement (20'-20'-20'-20'-20'), where on the ends the bay length ran from the end of the exterior column to the center of the first interior column and the rest of the bays were twenty feet on center thereafter. This layout provided central areas in the building with open spaces of approximately 500 square feet (not including column size) and exterior open spaces of 400 square feet (not including column size). Figure 3.1.2 is a plan view of option two.

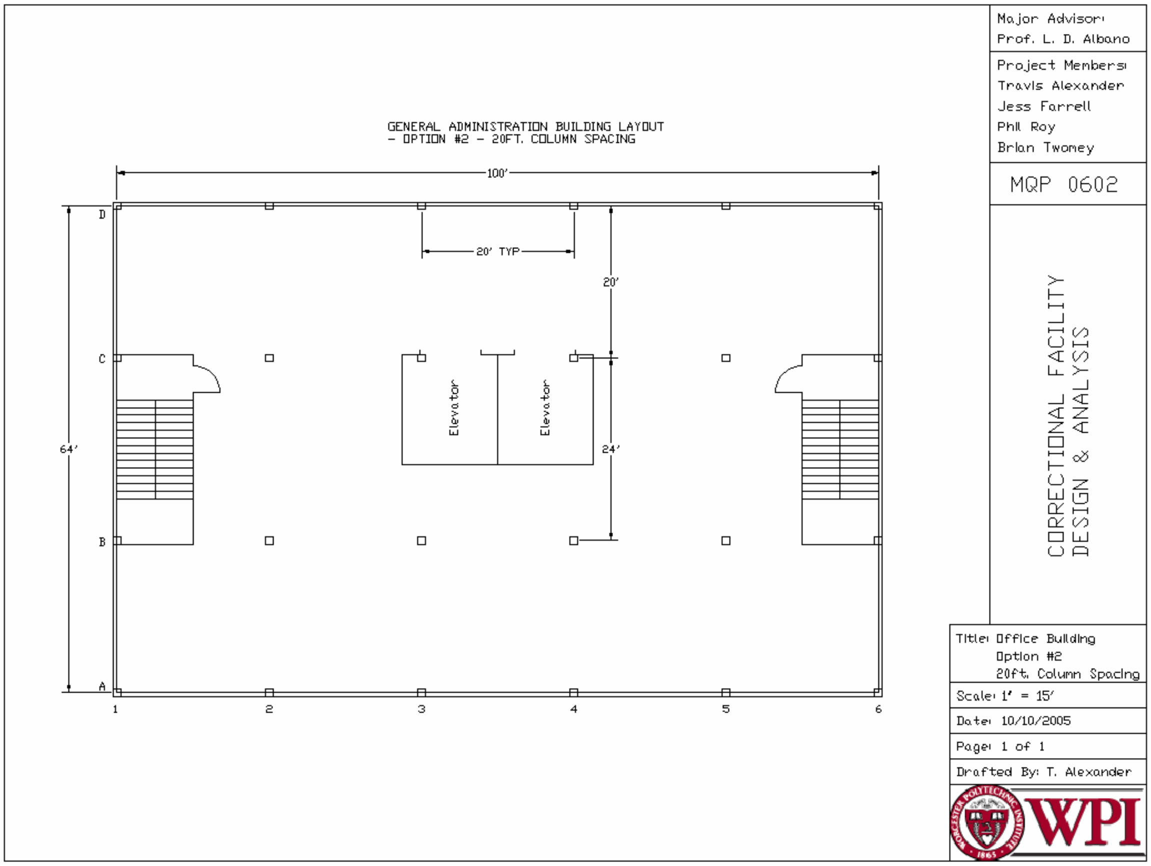

Figure 3.1.2 Office Layout, Option 2

The following table is a comparison of the number of members contained in each of the two different layout scenarios. Please note that this element count does not include any columns that would be designed for a basement or sublevel in the building.

Table 3.1.1 Office Member Count

Office Member Count		
Element	***Option 1 (10'/16' long side)***	***Option 2 (20' long side)***
Columns	32	24
Beams	26 - 20 foot 13 - 25 foot	22 - 20 foot 11 - 25 foot
Girders	8 - 10 foot 20 - 16 foot	20 - 20 foot
Total	99	77

3.2 Correctional Facility Layout

There are many considerations that need to be evaluated when designing the layout of a correctional facility. Visibility, functionality and egress are three of the major concerns that were given the most consideration throughout the design and layout of the correctional facility. The prison was designed with four similar but separate cellblocks that are arrayed around a centralized core. Each cellblock is restricted to a level of security with two cellblocks devoted to medium security and the others respectively limited to minimum and maximum security (refer to Figure 3.2.1 for prison layout).

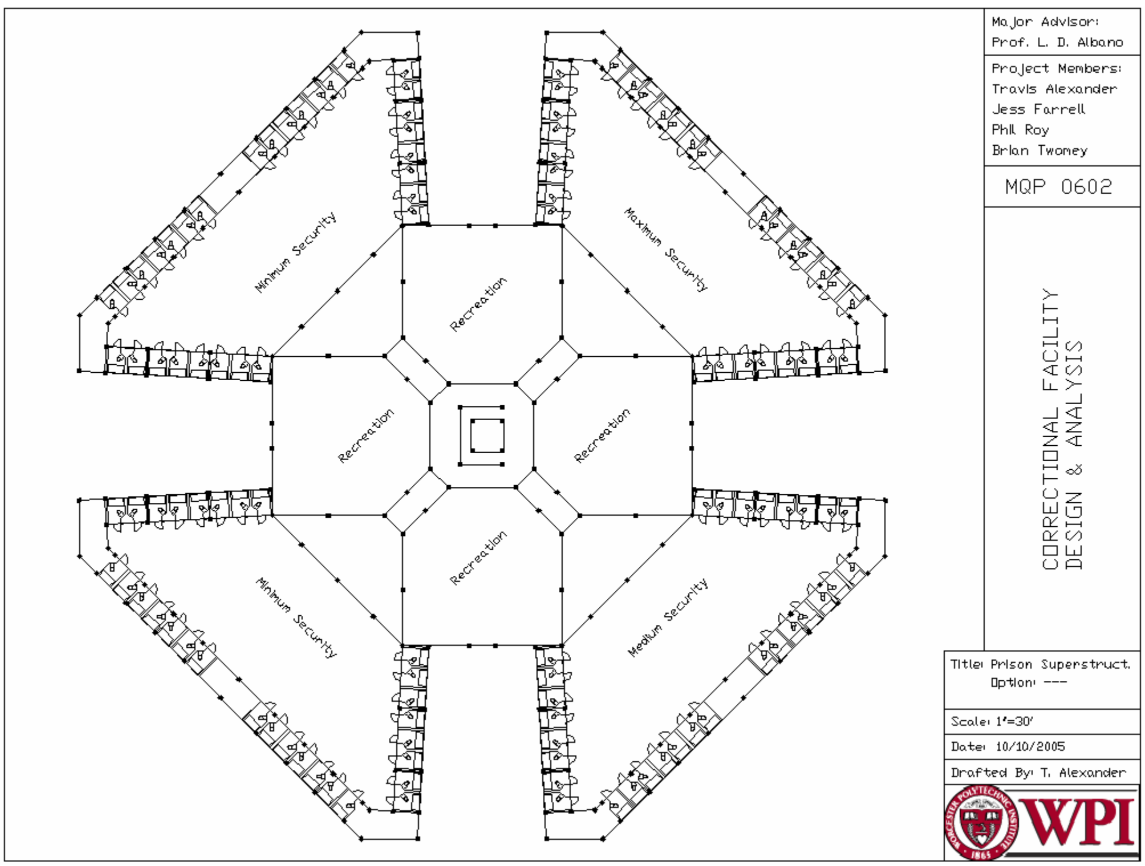

Figure 3.2.1 Prison Structure

3.2.1 Cellblock Layout

Direct supervision and visibility of prisoners is very important when designing a cellblock. This used to be achieved by security guards patrolling long narrow hallways with cells lining the corridor. However, direct supervision is currently much more efficiently achieved with security guards monitoring large open rooms that enhance the security guard's vision, which ultimately requires less security personnel and reduces operational costs. It is estimated that for every security guard added to the staff, an additional one million dollars is added to the operating costs of the prison amortized over the lifespan of the facility.[304] This added overhead is the major reason why prison designs with large open rooms are favored over those with long narrow hallways. The open room layout was utilized in the design of the cellblocks in order to take advantage of the efficiency of security guard requirements.

Each cellblock features two floors with each floor containing two levels of cells as can be seen in a cross section in Figure 3.2.2. Each floor is 24 feet tall, which is composed of two 12 foot tall cell units stacked on each other. A cantilevered walkway provides access to the cells on the second level of each floor. Additionally, there are two stair cases in each cellblock, which are located in the corners of the triangular shaped cellblock. These stair cases provide access to the cantilevered walkway on the first floor all the way up to the second level of cells on the second floor. The cells run along the perimeter of the triangular shaped cellblock, which creates a dayroom in the center of each cellblock. The dayroom can be used for prisoner activities such as education, recreation, training and dining. Additionally, security guard posts are positioned on a

[304] http://www.bls.gov/oco/ocos156.htm

raised platform in a central location of the dayroom, providing an effective location to monitor both levels of cells (refer to Figure 3.2.2 and Figure 3.2.3 for the typical cellblock layout). Security guards in each cellblock are responsible for 30 single occupancy cells per level and a total of 60 single occupancy cells per floor. The total occupancy for each cellblock is 120 inmates. All four cellblocks combined contain 480 single occupancy cells.

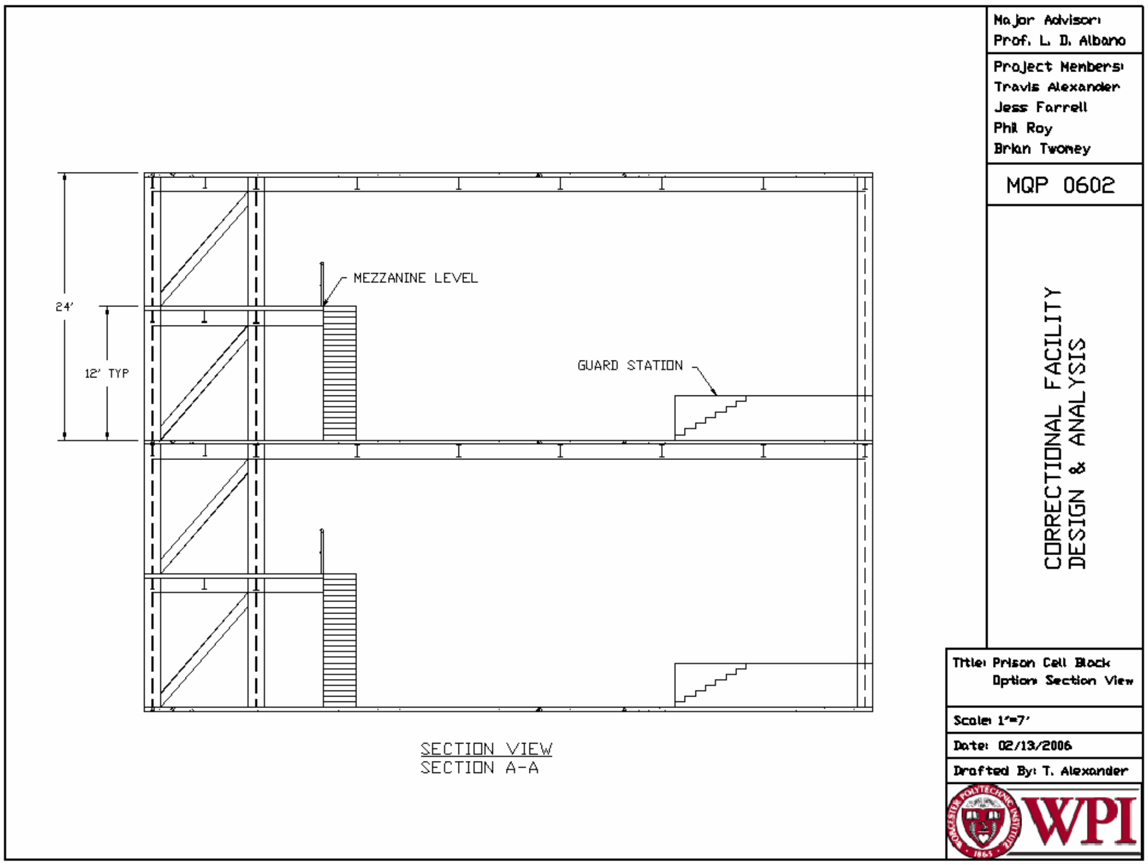

Figure 3.2.2 Section View

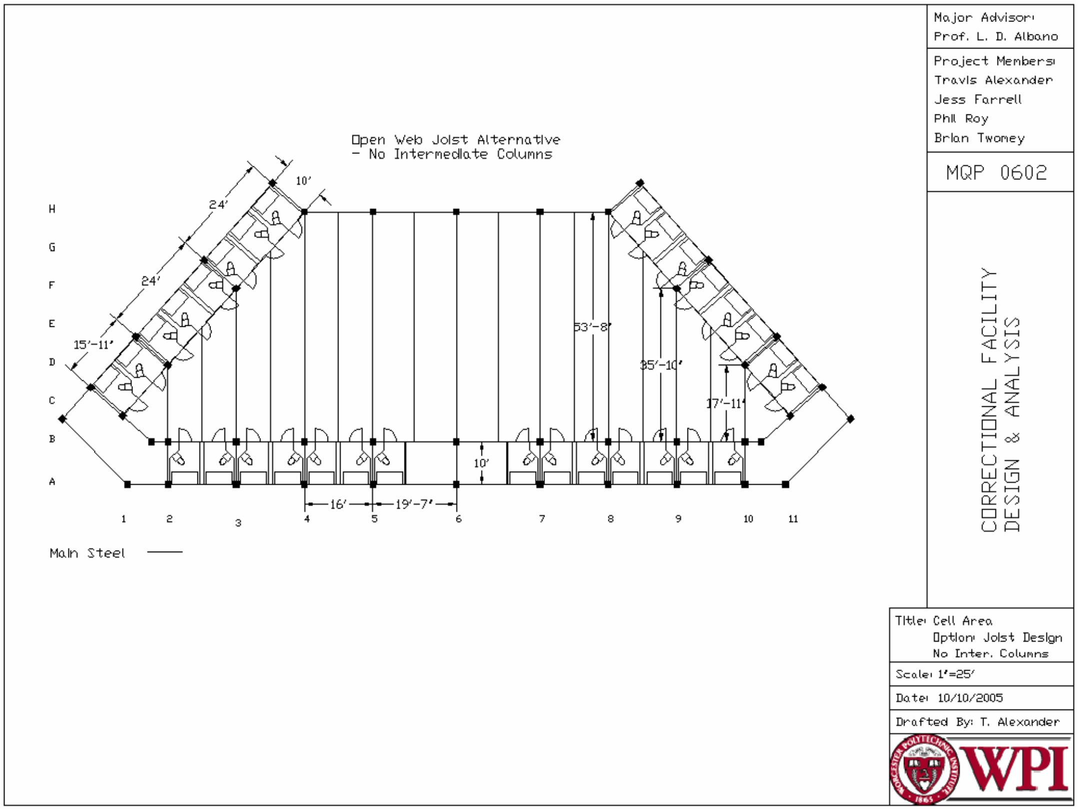

Figure 3.2.3 Prison Cellblock, Open-web Joist Option

The living arrangements provided to inmates are very minimal and only offer basic necessities. To comply with the minimum design standards, cells were designed to be ten feet deep by seven feet wide with 35 square feet of uninhibited open space.[305] Additionally, windows were provided in each cell to allow essential natural light to enter. Other typical furniture that was included in each cell was a bed, a chair and a table. This furniture was designed to be mounted to the floor or wall to prevent the fixtures from being used in a harmful manner. The cells were also designed to be "wet cells" which includes a toilet and a sink.[306] Central showers are located in the center of each level of cells along the main row of cells.

[305] Congressional Budget Office
[306] Chiara, Joseph De and Crosbie, Michael J., p.834

When designing the structural frame of the cellblocks, visibility and security played a major role in defining column placement. Three different layouts were researched in order to find the most economical structural frame while trying to maximize the visibility for security guards. The first option included no intermediate columns within the dayroom to allow for maximum visibility (see Figure 3.2.4 for Option 1 layout). The second option introduced one intermediate row of columns, and the third option included two intermediate rows of columns within the dayroom (refer to Figure 3.2.5 and Figure 3.2.6 respectively for Option #2 and Option #3 layouts). Table 3.2.1 provides a summary for the three cellblock layout options.

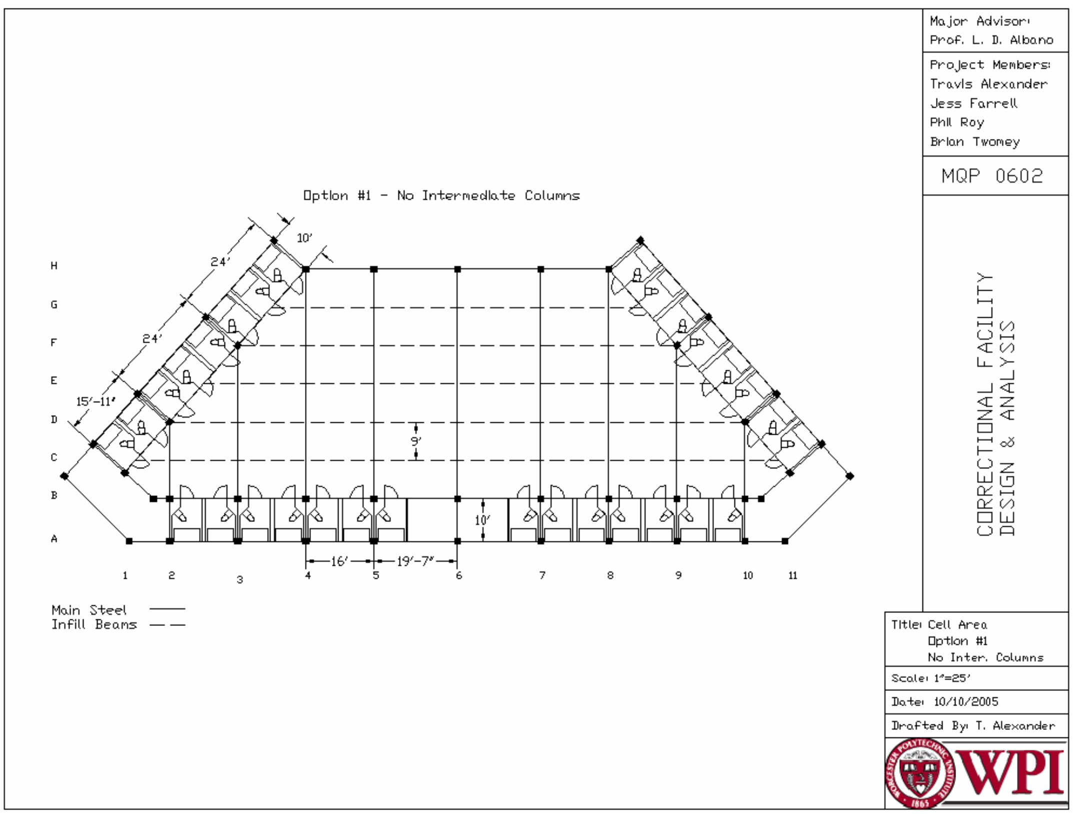

Figure 3.2.4 Prison Cellblock, Option 1

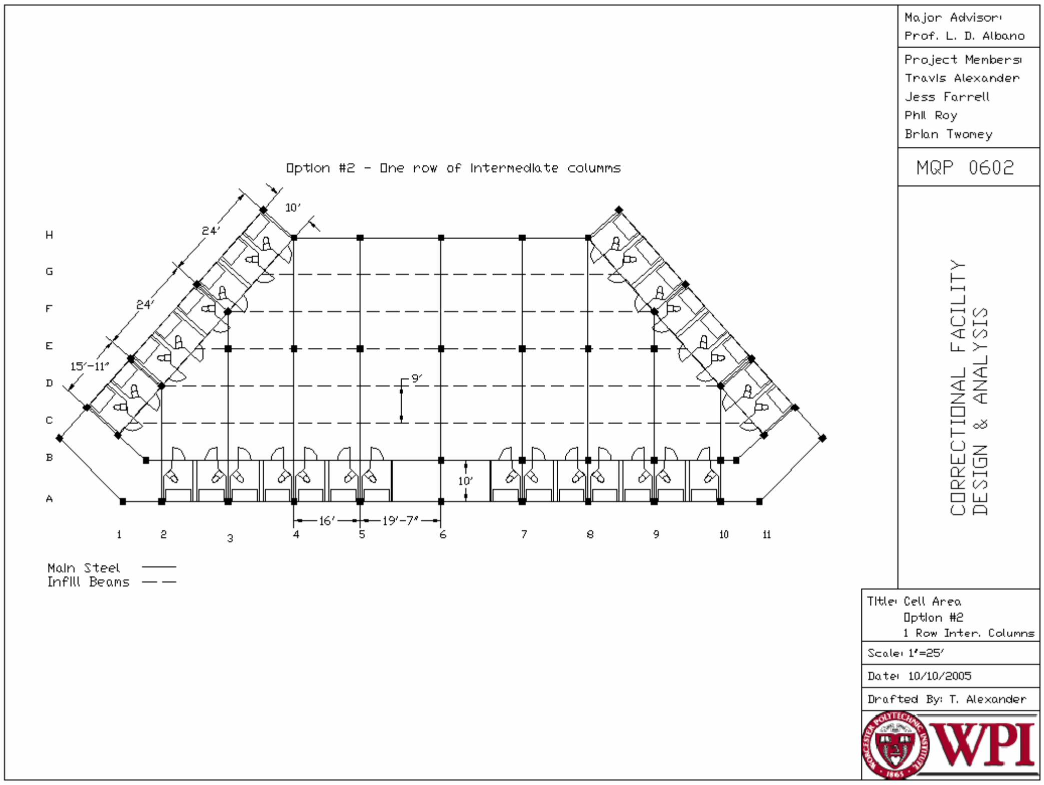

Figure 3.2.5 Prison Cellblock, Option 2

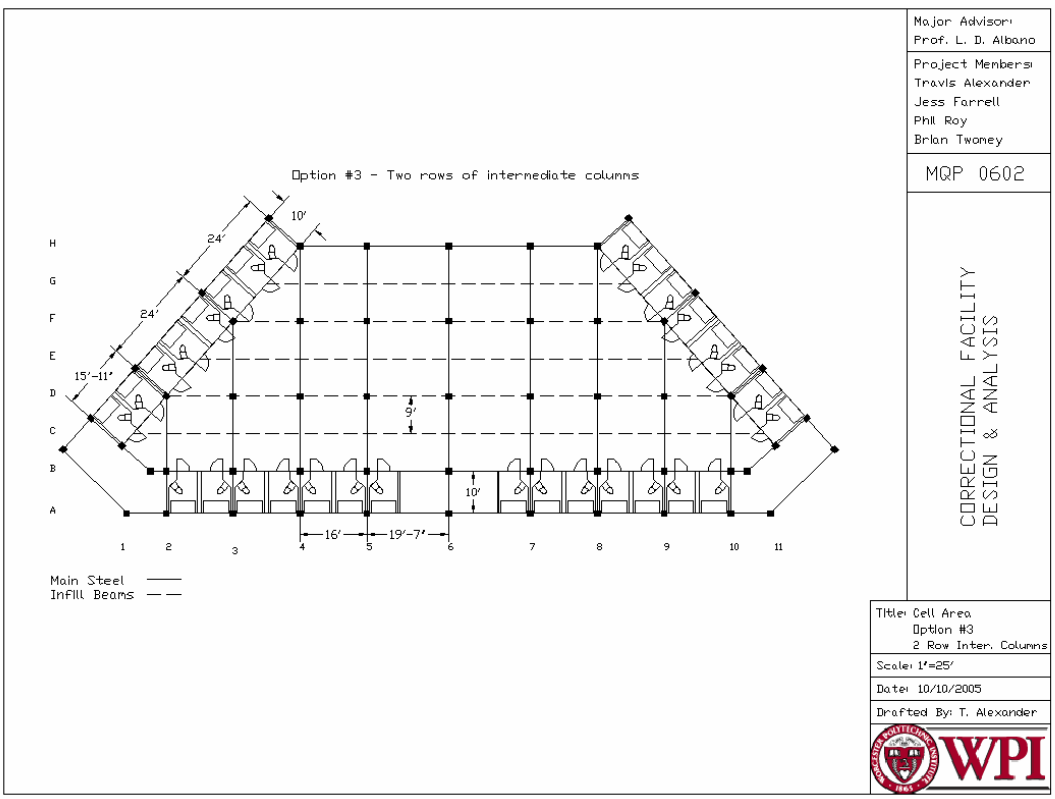

Figure 3.2.6 Prison Cellblock, Option 3

Table 3.2.1 Cellblock Column Layout Options

	Column Layout	Implications
Option 1	No intermediate columns in dayroom	Long girder spans Optimum visibility
Option 2	One row of intermediate columns within the dayroom	Compromise between long girder spans and visibility
Option 3	Two rows of intermediate columns within the dayroom	Short girder spans Minimum visibility

3.2.2 Core Layout

Each cellblock is connected to the core of the correctional facility through a sallyport regulated by the central security station. The core of the correctional facility includes a central security station and four half-court basketball courts (refer to Figure 3.2.7 for the core layout). Featured within the central security station is an 8,000 pound freight elevator that is wrapped by a centralized staircase. The floor height within the core is 24 feet, which is continuous with each cellblock.

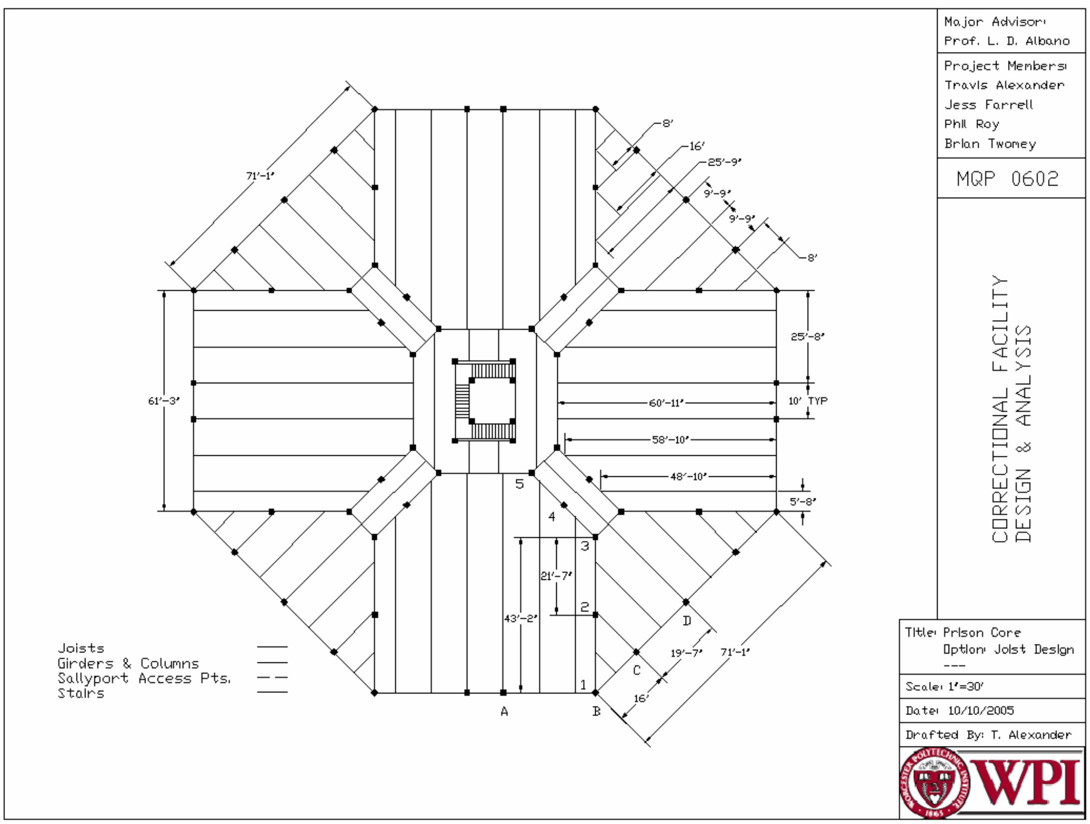

Figure 3.2.7 Prison Core

3.3 *Material Considerations*

3.3.1 Material Selection Considerations

The materials in consideration for construction should adhere to the fire safety requirements as dictated by the *2003 International Building Code* and any special considerations spelled out by the American Correctional Association. Each structure has a different allowable construction type, depending on the usage of the building and its fire suppressant system. This construction type leads to a required fire resistance rating which can vary between elements of the building such as floors, interior walls and exterior walls.

To determine the fire rating required for a correctional facility, the *2003 International Building Code* was referenced. The code prescribes a construction type of 1A or 1B construction due to the constraints of the layout. This led to the selection of non-combustible material such as concrete and steel. Refer to section 7 Fire Considerations for further discussion.

3.3.2 Composite Action

Composite beam-and-slab systems were explored as an alternative for both the office building and prison structure. Composite systems are described as having the concrete floor slab and the supporting steel beam and girders working together to resist the applied loads. In a non-composite system, the concrete slab is said to span one-way, from infill member to infill member. As with this system, the slab transfers the loads perpendicular into the member. Alternatively, composite systems have the added benefits of transferring the loads perpendicular between members but also parallel to the member, thereby increasing the carrying capacity.

Composite action can be developed by embedding the top (compressive) flange of the supporting member in the concrete slab. If the full depth of the flange is within the slab, then full composite action is developed. Full composite action can also be developed when the plastic neutral axis falls within the web of the beam. In this case the properties of both the steel and concrete resist shear forces. When the plastic neutral axis falls solely within the slab, the concrete properties determine the shear resisting capacity.

Composite action can also be developed with the use of shear studs welded to the compressive flange. The shear studs must be designed to resist the full shear generated within the compressive flange to be considered as full composite. By embedding the top flange of the member, or by attaching the shear studs, a 'no slip' condition is created between the concrete slab and the structural steel beams. As a result, both elements act in unison to resist the applied loads.

The benefit of developing composite action is that heavier loads can be carried by the floor system without a concomitant increase in beam and girder sizes. The development of full composite action can lead to a 33 to 50% increase in the carrying capacity of the steel members than if the steel gravity system acted alone.[307] The increase in carrying capacity can be attributed to the increase in the moment of inertia of the cross-section. The increased moment of inertia of the composite section also contributes to an increase in overall stiffness, which allows for greater deflection control. Figure 3.3.1 shows a comparison between the deflections of composite and non-composite systems. Therefore, smaller members would be required to carry the applied loads if a composite system was detailed because of the increase in carrying capacity and deflection control.

[307] Salmon, Charles G. & Johnson, John E.

There would be a cost savings associated with using smaller members, only offset by the cost and installation of the shear studs.

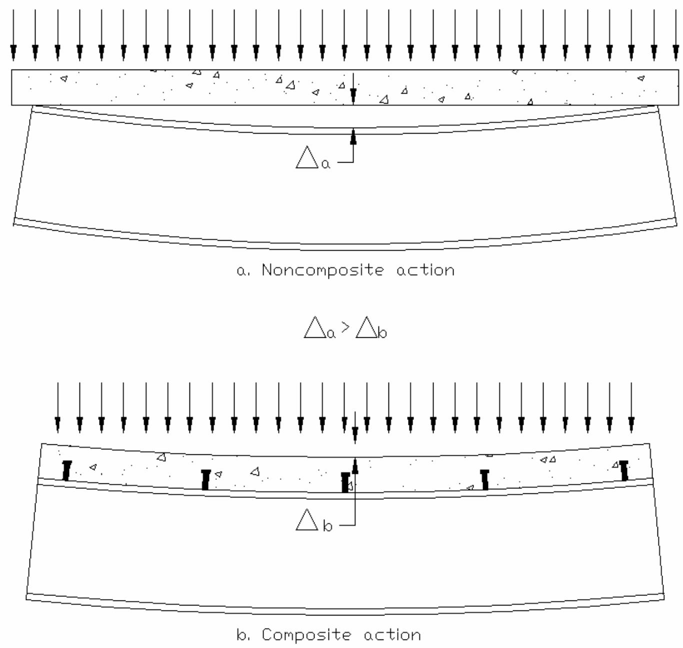

Figure 3.3.1 Non-composite vs. Composite Action

3.3.3 Wide-Flange Sections

For the design of both the administrative building and the prison structure, considerations for wide-flange sections (WF-shape) were given because of their use in the construction industry. The basis for using steel in the structural designs is discussed here on. Consideration was given to the material's overall performance and relative cost.

Steel members have the ability to utilize the properties of compression and tension to provide support. In fact, steel is one of the few structural materials that can adequately support both tension and compression forces. However, it should be noted that steel is by far superior to reinforced concrete and timber in terms of its tensile strength. Added considerations must be given for buckling of steel members when designed for compressive stresses.

An added benefit of steel is its uniformity in material properties. The behavior of steel upon applying loads is very similar to assumptions made and values used during the design process. Material properties and member cross-sectional properties can be accurately calculated. The cross-sectional properties of steel do not significantly change overtime due to temperature fluctuations, load applications, or old age. Such changes occur with wood and concrete materials. The dimensions of wood change with the temperature; humidity in the summer months cause the members to swell and the members shrink during the winter due to the dry air. Concrete members tend to "creep" over time because of the effect of the applied loads to redistribute absorbed water particles. These fluctuations in cross-sectional dimensions can raise concerns about carrying capacity and deflection control.

Steel members have the ability to experience large amounts of elongation and sectional area reduction due to excessively high applied loads before failure. The reason for steel to be able to withstand large loads as well as shock loads is because the material is not hard and brittle; it has a high amount of ductility. "Ductility is more commonly defined as the ability of a material to deform easily upon the application of a tensile force, or as the ability of a material to withstand plastic deformation without rupture."[308] Often failure comes in the form of fracture. Steel will not break as a result of sudden shock. Therefore, steel is ideal for construction in locations of high seismic activity. As a result of plastic deformation, the original shape of the member before bending, buckling, warping, or twisting is not recoverable.

Another benefit of steel's ductility is its ability to resist large load concentrations. Often, these high levels of stresses develop because of general use of the structure. Steel

[308] Engineers Edge.

members have the ability to yield locally to prevent premature failures. When a member locally yields, it means that point of interest has reached its carrying capacity. As a result, the point "bottoms out" and any additional loads are redistributed through the member.

In terms of steel design for the gravity and lateral system, wide-flange members were used. The reason for this is the overall acceptance of wide-flange members to be used in framing systems. Furthermore, wide-flange sections are exceptional in resisting bending. This is due in part to their relatively high moment of inertia about their strong axis. Moment of inertia is defined as the ability to resist bending. The value is dependent on an object's shape and mass (weight) distribution in the cross-section. The farther away the mass is from the centroid of the cross-section, the larger the moment of inertia will be. Wide-flange members have a large moment of inertia because a majority of the mass is located in the flanges, than the webs. Figure 3.3.2 shows a typical wide-flange cross-section.

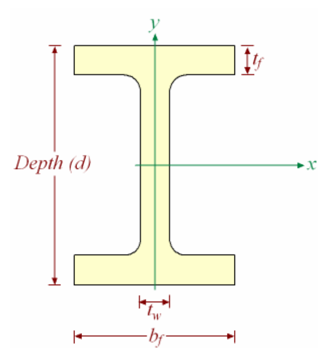

Figure 3.3.2 Typical Wide-Flange Cross-section

3.3.4 Open-web Joists

Open-web joists were another structural steel member that were considered for the various building layouts. In particular, open-web joists provided by the Vulcraft Group, a division of the Nucor Corporation, were considered and used in the design process. Open-web joists have the advantage of being extremely light while maintaining a high strength-to-weight ratio. The lightweight nature of such members results in lighter members required to support them. These include girders and columns that are designed using wide-flange members. This consideration further increases the economy of the structure.

Open-web joists have the ability to span such long distances because of their structural geometry. Open-web joists are comprised of angle or rod members oriented at an angle to one another in the plane of the joist in a manner similar to that of a truss. Top and bottom chords running the length of the joist tie the assembly together. The triangular geometry of open-web joists provides a great amount of stability for the member over large distances. Furthermore, the geometry of such members allows them to transfer large applied loads while maintaining a relatively light weight (hence their height strength-to-weight ratio). Figure 3.3.3 shows a typical Vulcraft open-web joist.

Figure 3.3.3 Vulcraft Open-Web Joist[309]

The reason for considering open-web joists was because of their ability to span large distances without any support. This consideration was relative to the design of the dayroom in the prison cellblock as well as the design of the prison core, in particular the recreation area. Both designs benefited from the lack of column placements within the intermediate area. Additional column placements in the dayroom would decrease the

[309] Vulcraft

amount of space that guards could observe at one time. This in turn would result in added costs associated with the necessity to have more security guards to observe activities occurring in the blind spots created by the extra columns. In addition, columns intruding into the recreation area would result in an unusable space when such a location was specified for sporting activities.

3.3.5 Reinforced Concrete

The usage of concrete as a construction material dates back the Egyptians of 2500 BC and the Romans of 300 BC.[310] In the 1800's the usage of reinforcement in concrete became a common practice, and since then concrete and reinforced concrete have become widely used as a building material options all over the globe.[311] This is due to the ease of mixing concrete, the availability of reinforcing bars, the lack of skilled labor needed to place concrete, and the relatively inexpensive nature of the substance.[312]

In terms of the purposes that the materials in a reinforced member serve, first regard concrete. Concrete has a high compressive strength in comparison to its tensile strength, thus the concrete portion of a reinforced concrete member resists any compressive forces that may be applied. The tensile forces in the member are handled by the reinforcing steel.[313]

There are many advantages to utilizing reinforced concrete to form a structure. The first is the economy of construction.[314] As previously mentioned, less skilled labor is needed to construct a reinforced concrete structure than one made of steel. Additionally, the materials are widely available, and there is no need for special fabrication of members,

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[310] Lambert, Paul
[311] Lambert, Paul
[312] Macgregor, James G. p.1
[313] MacGregor, p.1
[314] MacGregor, p.6

which can drive up the cost of construction. Another advantage to constructing a building of reinforced concrete is that it is malleable and can be finished.[315] This means that concrete can be readily formed to almost any shape indicated by architect. Concrete can also be finished so that the surface of the material itself is the final product instead of having to cover it with drywall or another form of cladding. A third advantage to using reinforced concrete is the natural fire resistance it possesses.[316] Reinforced concrete has a fire rating of between one and three hours without including any special fire proofing. The rigidity of a reinforced structure is another positive trait in its construction. Reinforced concrete structures tend to have very few vibrations from wind loading and other such occurrences. Final advantages include the fact that reinforced concrete structures are easy to maintain and as previously mentioned the materials used in construction are relatively simple to obtain.[317]

Reinforced concrete as a building material also has a few disadvantages. The first is that concrete itself is very weak in tension.[318] The tensile strength of concrete is typically only one tenth of the compressive strength. This is dealt with by utilizing steel reinforcing bars in the concrete member itself to deal with any tensile forces that may be present. Another drawback to utilizing reinforced concrete as a building structure is the need for forms and shoring.[319] Forms are needed to contain the concrete while it is being placed and gaining strength while hydrating. In addition, the formwork needs to be removed once the concrete is of adequate strength. This is both labor and time intensive

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[315] MacGregor, p.6
[316] MacGregor, p.6-7
[317] MacGregor, p.7
[318] MacGregor, p.7
[319] MacGregor, p.7

and can impact cost. Concrete members must also be shored until they possess enough strength to support themselves.

A third major issue in using reinforced concrete as a building material lie in the material properties themselves.[320] Concrete is generally more heavy than steel and usually requires a greater volume than an equivalent structure composed of steel. Additionally, because of the weight of concrete and volume needed to handle forces, it is typically not an advisable material to use for long spans. A final detraction from the usage of concrete is the fact that over time, it will shrink.[321] This is due in part to the fact that as concrete dries it loses volume. Deflections will also increase due to creep as a result of sustained loading on the structure.

3.4 Gravity Systems

Once the layouts and structural materials have been selected, the designs of the gravity system can commence. The design for gravity loads is critical, as it is the basis for the rest of the structure. Invariably gravity, or service loads, will be applied to the structure. Furthermore, the design of the gravity system is important because the lateral system, which resists applied lateral loads resulting from wind and seismic activity, is incorporated into the gravity system later in the design process.

The gravity loads are defined by the materials of construction, usage, occupancy, and geographic location. The values of these loads are a necessity to effectively perform the design of the gravity system. To determine the applicable loads, the appropriate building code provisions were referenced. For the office and prison structure, the *2003 International Building Code* was used to define the dead and live loads for design.

[320] MacGregor, p.7
[321] MacGregor, p.7

Reference was also made to the *ASCE 07-02 Minimum Design Loads for Buildings and Other Structures* for the applicable snow load calculations.

Dead loads are defined as the weight of materials incorporated into the structure during construction as well as the materials that help define and finish the structure aesthetically. These loads include member self weights, the weights of finishes such as windows, walls, exterior cladding, flooring, ceilings, and interior partitions. An important factor regarding interior partition is that they can act as a 20 psf live load when designing for maximum versatility of a structure. This load is added to the other live loads that the space needs to conform to and is done to account for varying locations of the partitions throughout the life of the structure. Infrastructure requirements such as plumbing, electrical, and mechanical considerations including heating, ventilating, and air-conditioning ductwork also add to the dead load exerted onto the structure.

Live loads are a function of the structure's intended use. These loads are defined by the intended occupancy and usage of the structure, not including dead loads. The weight of furniture and movable partitions are included in live loads since these items are not affixed to structural elements of the building. Often in design, a live load reduction based on the tributary area of the member being designed may be implemented because of the unlikely probability that the entire area will be fully loaded. The live load values used for the structures comprising this project were taken from Table 1607.1 in the IBC.

Snow loads are defined by the geographic implications of the design site. The equivalent snow load is a function of the roof's slope, the structure's exposure factor, the thermal factor as defined by the governing building code, the importance factor of the structure, and the ground snow load. It should be noted that snow loads are only considered to be applied to the structure's roof.

Table 3.4.1 summarizes the loadings applied to the various structures that make up the correctional facility compound.

Table 3.4.1 Gravity Load Values

Dead Load		Live Load		Snow Load	
Material	(varies)	Occupancy (office)		Snow Load	35 psf
Mechanical	11 psf	- 1^{st} & 5^{th} floors	100 psf		
Ceilings	5 psf	- 2^{nd} thru 4^{th} floors	80 psf		
Flooring	4 psf	- 6^{th} floor	40 psf		
		Prison Cellblock	40 psf		
		Prison Recreation	100 psf		

3.4.1 Gravity System Components

All structures are exposed to gravity loads. Therefore, a gravity system must be designed to resist the applied loads. The gravity system must be capable of being self supporting. During the construction phase, each structural member must be able to support its self weight, applied construction loads, and the weights of other members that tie into it. The design considerations during the construction phase are critical. For example, in steel design with composite action, the WF-shape beam and concrete slab are designed to carry the applied loads in unison. However, during construction the WF-shape beam is solely responsible for supporting its self weight, the concrete's self weight, and the applied construction loads while being within a certain deflection limit. Once the structure has been fully built, the gravity system must be able to resist the service loads while only allowing a minimal amount of deflection. The check for deflection is critical not only for the structural integrity, but also for the occupants' comfort.

The structural member design order is based on the load path. The load path is defined as the path in which the loads travel from member to member along the stiffest path. For the gravity system, the load path for both the concrete and steel design is the same. The uniformly distributed service loads and point loads are applied to the slab or decking which are supported by the beams. The end reactions of the beams transfer the loads onto the girders. End reactions of the girder result in load transfer to the supporting columns, which in turn carry the load down to the foundations.

For a typical steel building, the gravity system is composed of six structural elements. These include the decking, beams, girders, columns, footings, and connections. The cross-sectional properties of each member are examined to verify their adequacy to carry the applied loads. These analyses focus on bending moments for beams and girders, and axial loads in columns for braced frames. The composition of a reinforced concrete structure is slightly less complex as it is composed of reinforced slabs, beams, girders, columns and footings. Note that the design of the reinforced structure does not include separate design considerations for connections. This is due to the fact that the reinforcing bars of the girders are tied into the columns forming a more continuous design that that of steel members whose connections are composed of discrete elements. Essentially, the connections for reinforced concrete members are achieved through the construction process by tying together the reinforcing steel of the different elements and by placing concrete into continuous forms.

The following Table 3.4.2 summarizes the design assumptions and code implications considered in the analysis of different options for the development of the structures.

Table 3.4.2 Gravity Design Assumptions and Code Implications

System Type	Elements	Design Assumptions	Code Evaluation
WF-Shape w/ Composite	concrete slab	Simply supported beams and girders, pin ended columns, slab connected by shear studs.	N/A
	shear studs		Shear capacity
	beams		Moment capacity, local buckling, deflection
	girders		Moment capacity, local buckling, lateral/torsional buckling and deflection.
	columns		Axial load capacity
	formwork		N/A
WF-Shape w/ Reinforced Slab	reinforced slab	Simply supported beams and girders with pin ended columns.	Moment capacity, tributary width capacity
	beams		Moment capacity, local buckling, deflection
	girders		Moment capacity, local buckling, lateral/torsional buckling and deflection.
	columns		Axial load capacity
	formwork		N/A
Open Web Joist w/ Composite	concrete slab	Simply supported joists and girders, pin ended columns, slab is placed upon metal decking with shear studs.	Tributary width capacity
	shear studs		Shear capacity
	open web joists		Deflection checks
	girders		Moment capacity, local buckling, lateral/torsional buckling and deflection.
	columns		Axial load capacity
	decking		Tributary width capacity
Reinforced Concrete	reinforced slab	The connections between the slab, beams, girders and columns are all moment connections	Moment capacity, tributary width capacity
	beams		Moment capacity, shear capacity
	girders		Moment capacity
	columns		Axial load capacity, moment capacity

3.4.2 Design Procedure

When designing a structure, it is critical to consider the uncertainties that lie within the material properties and applied loads. Material uncertainties reflect a variance in material properties, residual stresses as a result of fabrication, member eccentricities, and transportation and construction damages. Load uncertainties consider additional, unexpected loads during construction and the life-cycle of the structure.

The Allowable Stress Design procedure takes all of the uncertainties and addresses them with one factor of safety value. The ASD procedure stipulates that the material elastic stress must not exceed an allowable stress as a result of the applied loads.[322] Adversely, the Load Resistance and Factor Design separates material uncertainties and load uncertainties. In the LRFD procedure, each uncertainty is addressed separately. Regarding reinforced concrete in the LRFD procedure, the ultimate strength of the member's cross-section is examined. For steel design with the LRFD procedure, the elastic design method is used to examine the maximum carrying capacity of steel members. Furthermore, load combination factors are used to address the issue of load uncertainties in accordance with the applied loads. For the design of the office and prison structures, the LRFD procedure was used for both concrete and steel design.

The design of a gravity system must follow the load path of the gravity loads. One cannot design the columns if the girders have not been designed, and the girders cannot be designed until the beams sizes have been determined. In terms of using steel members, composite action or some other form of decking must first be determined to adequately follow the design process. The design process cannot commence until the appropriate design approach, either Allowable Stress Design (ASD) or Load Resistance and Factor Design (LRFD), has been chosen along with the applicable loads with reference made to the governing building codes. Both approaches are valid for design; it must simply be one approach or the other. The most common approach is LRFD, and it must be remembered that combining ASD and LRFD methods for design results in invalid practices.

3.4.3 Gravity System Design in Steel

The gravity system design for both the office and the prison structures was performed using structural steel framing with a concrete slab. For both structures,

[322] McCormac, Jack C. & Nelson Jr., James K., p.48

various combinations of steel framing and concrete decking were used. These combinations include composite design with a concrete slab and WF-shape beams, composite design with a concrete slab, metal decking, and Vulcraft® open-web joists, and a reinforced slab and WF-shape beams. All combinations included WF-shape girder and column members. Member properties for WF-shapes were taken from the AISC *Manual of Steel Construction: Load and Resistance Design* 3rd Edition, and Vulcraft's *Composite and Noncomposite Floor Joists* manual was used for open-web joist properties. For further design procedure details and sample calculations, the reader can reference McCormac and Nelson's *Structural Steel Design: LRFD Method, 3rd Ed.* as well as Salmon and Johnson's *Steel Structures: Design and Behavior, 4th Ed.*

Assumed properties for the steel members included yield strength of 50 ksi and a modulus of elasticity equivalent to 29,000 ksi. When composite systems were designed with WF-shapes, a unit weight of concrete of 150 pcf and a compressive strength of 3,000 psi were used. Light-weight concrete (110 pcf) was used in composite systems with open-web joists. The yield strength of the shear connectors was taken as 65 ksi for ¾ inch diameter studs. Considerations for reinforced concrete slabs followed those given in section 3.4.4 Gravity Design for Reinforced Concrete Structure.

Once the slab type was defined, either composite or reinforced (non-composite), the design of the infill beams commenced. Consideration will first be given here to composite action. Composite design with WF-shape members was based on plastic analysis. Plastic analysis limits the use of various limit states such as instability, fatigue, or brittle fracture. When designing with plastic analysis, the members must be able to achieve a strength equivalent to the plastic moment strength in which the entire cross section has reached the yield stress of the material. Considerations that were used in the design of beams in composite action are summarized in Table 3.4.3. Corresponding considerations for composite action with open-web joists, based on the design procedure laid forth by the Vulcraft Group are given in Table 3.4.4.

Table 3.4.3 Composite WF-shape Design Checks

Design Checks	Limits
Moment Capacity	$Z_x \geq Z_{required}$
Local Buckling	$b_f/2t_f \leq 9.15$ $h/t_w \leq 90.55$
Location of PNA	$a \leq t_s$
Deflection	$\Delta_{service} \leq L/360$ 75% $\Delta_{constr.} \leq L/360$

Table 3.4.4 Composite Vulcraft Open-web Joist Design Checks

Design Checks	Limits
Moment of Inertia	$I_{provided} \geq I_{required}$
Deflection	$\Delta_{service} \leq L/360$ $\Delta_{constr.} \leq L/360$

The moment capacity design check for the composite WF-shape members insured that the moment (carrying) capacity of the member was adequate to sustain the applied loads. The local buckling concerns examined the cross-sectional properties of the member to see that both the flange and web thicknesses were sufficient for the applied loads; that not one element would prematurely buckle before the plastification of the cross-section. The design check for the location of the plastic neutral axis (axis through the cross-section, which divides the regions of compression and tension) verified that the PNA was within the slab to achieve full composite action. The final checks were for deflection during the construction phase and while in service. The deflection during construction is critical if no decking or shoring material is used to help support the weight of the concrete before it has sufficiently cured, as was the case with the design of composite WF-shape in the office and prison structures. During this time, no help is provided by the concrete to resist loads. Therefore, the structural steel members have to be sufficiently strong to sustain the material self weights and construction loads while limiting the amount of deflection to the prescribed limits.

Regarding the design checks for the composite open-web joists, the moment of inertia of the joist was verified to be adequate to resist bending (deflection). Furthermore, deflection checks were performed regarding construction and service loads.

The considerations for non-composite beam and girder designs are presented in Table 3.4.5. As with the composite WF-shape beam design, non-composite WF-shape beam design as well as girder design was based on plastic analysis. Therefore, the members were designed to be able to achieve the plastic moment (carrying) capacity because the unbraced length was not an issue. Lateral torsional buckling is often a crucial check for wide flange members not in composite action. The lateral torsional buckling limits required that the factored design moment be less than the critical moment capacity defined by the unbraced length of the member. This is a critical check because the affects of lateral torsional buckling often reduce the moment capacity of the member. The effects of lateral torsional buckling can often be neglected because the decking and/or slab can be assumed to provide lateral bracing along the entire length of the member.

Table 3.4.5 Non-composite Beam and Girder WF-shape Design Checks

Design Checks	Limits
Moment Capacity	$Z_x \geq Z_{required}$
Local Buckling	$b_f/2t_f \leq 9.15$ $h/t_w \leq 90.55$
Lateral Torsional Buckling	$M_u \leq \Phi M_{cr}$
Deflection	$\Delta_{service} \leq L/360$

The design check regarding column design was based on the axial capacity of the column. Since all members were designed with simple pin-ended connections, moments between members could not be transferred. Therefore, girder reactions in the form of axial loads were the only consideration needed to design the columns. The compression strength of the columns was confirmed to be greater than the factored compressive load on the column.

3.4.3.1 Use of Design Spreadsheets

To aid in the design of the composite beam-and-slab sections, an Excel spreadsheet was created to perform repetitive calculations. Once material properties and applied loads have been defined and a trial WF-shape section is chosen, the spreadsheet checks the adequacy of the configuration based on the member section properties referenced to the *Manual of Steel Construction, Load Resistance and Factor Design, 3rd Edition* provided by the American of Steel Construction. Section property checks, load capacity checks, and deflection checks are performed and compared to the allowable values. See Appendix C.1 Composite WF-Shape Beam Design for further spreadsheet explanations and user instructions.

With regards to the design of composite open-web joists, an Excel spreadsheet was created. The spreadsheet followed the design procedure presented in the *Vulcraft® Composite and Noncomposite Floor Joists* Catalog. The applicable gravity loads and decking properties need to be entered. Once they are entered and a trial size chosen, the spreadsheet conducts a series of calculations based on deflection checks, comparing the results to an allowable deflection. See Appendix C.2 Vulcraft Composite Open-web Joist Design for further spreadsheet explanations and user instructions.

A third Excel spreadsheet was created to facilitate the calculations associated with non-composite WF-shape beam and girder designs. As with the composite beam spreadsheet, material property information and load information, including both point loads (applicable to girder design) and gravity loads, must be entered and a trial size chosen. Member properties are referenced to the *Manual of Steel Construction, Load Resistance and Factor Design, 3rd Edition.* Once a trial size has been determined, various checks are generated, examining the effects of lateral torsional buckling and deflection due to the loads. See Appendix C.3 WF-Shape Beam and Girder Design for further spreadsheet explanations and user instructions.

A fourth Excel spreadsheet was generated which examined column design. This spreadsheet was designed to be used in conjunction with columns that only have axial loads and no end moments. As with the other spreadsheets, the material properties and loads need to be entered. A WF-shape trial size may be chosen with reference to the *Manual of Steel Construction, Load Resistance and Factor Design, 3rd Edition* for member section properties. Once the necessary information has been entered including consideration for member end fixity (K-value) and unbraced length, calculations are performed regarding the column's capacity. See appendix C.4 WF-Shape Column Design (Axial Loads Only) for further spreadsheet explanations and user instructions.

3.4.4 Gravity Design for Reinforced Concrete Structure

When designing a reinforced concrete structure there are four major elements that need to be considered. These elements are the floor or roof slab, the beams, the girders and the columns. In the following section the design process for this project is described. This procedure is rather straight forward, and a complete detailed calculation of such a design is provided in Appendix D Sample Calculations. The calculations are based on the presentation in MacGregor's text, *Reinforced Concrete: Mechanics and Design*.

Conditions that were assumed in doing this design were that the concrete would have a unit weight of 150 pcf and a compressive strength of 3,000 psi. Another assumption that was made was that the reinforcing steel bars would have a yield strength of 40,000 psi.

In embarking upon designing a reinforced concrete structure it is typical to begin with the design of the floor slab. In summary, the first step taken was to establish a trial thickness. This minimum slab thickness is based upon the slab's span length and deflection concerns. Essentially it is estimated that the depth of a slab is $L_1/24$ or $L_2/28$,[323] where L_1 is the length of the end bay in inches, and L_2 is the length of a typical interior bay in inches. The use of these ratios avoids the need for a detailed deflection analysis. Once this minimum slab thickness was established, the dead weight per square foot of slab was determined by multiplying the weight per cubic foot of concrete by the thickness, in feet, of the slab. From this the dead load acting on the entire surface was

[323] ACI Table 9.5(a)

determined. This involved taking into account all of the dead loads, such as flooring and mechanical equipment, that would be present in the building. The trial weight of the slab was added to this dead load consideration.

Once the design value for the dead load was calculated, combined loading on the floor slab was determined. This involved adding the dead and live loads together after applying the appropriate load factor to each. More detailed information about the derivation of loadings for the structures can be found in section 3.4 Gravity Systems. After this procedure, the thickness required for moment and shear due to the loading was discovered. In doing these calculations, the reinforced ratio rho was assumed to be 0.01 for all calculations. Rho is seldom higher than 0.01 in slabs and essentially corresponds to the tension controlled limit for the neutral axis depth, which is a function of the area of steel, depth and base length of the slab.[324]

At this point, the thicknesses required for both moment and shear were compared to the original estimated thickness, and the thickness of the slab was reduced to the smallest size greater than or equal to four inches. The reasoning behind keeping four inches as the minimum depth stemmed from considerations accounting for reinforcing steel needing adequate clear cover and the size of aggregate that could be present in the concrete mixture. In addition, the floor must meet a minimum fire resistance and the logic applied to using the thickest slab option was that the thicker the slab, the greater the fire resistant value. After this the reinforcement for the strength of the slab was computed in addition to reinforcement for temperature and shrinkage of the concrete. In order to expedite this procedure an Excel spreadsheet was developed so that numerous calculations could be completed rapidly to check values when loading or span lengths

[324] MacGregor, James G., p.389

differed. An example of the reinforced slab spreadsheet along with instructions on how to use it can be found in Appendix C.5 Reinforced Concrete Slab Design.

The second step of the gravity system design was the design of the beams. Two variations on beam design were utilized during the evaluation of the correctional complex as a whole. In the office, T-beam design was utilized. The T-beams were designed to span the sixty-five foot length of the building. In the cellblocks rectangular beams were utilized. The beam spans that were considered for the cellblocks were sixteen feet long, twenty feet long and twenty-four feet long. The steps in completing the design of each of these types of beams are quite similar, thus the following paragraph will discuss a generalized version of the T-beam design process as it applies to the office and not cover a more in depth description of rectangular beam design.

The first step in the office T-beam design process was determining the factored dead and live loads placed upon the beam from the slab. In doing so, three separate influence areas and loading configurations were considered. The first influence area, titled influence area A, consisted of the area in the first bay of the short side of the building as the width and the depth of the largest long side bay (20 feet for the 20'x (5) column spacing and 16 for the 10'-16' x (5)-10' column spacing). The second influence area, influence area B, was a combination of the first and second short side bays as a width and the same depth as the first influence area. The final influence area, influence area C, was half the dimensions of the second interior bay on the short side and the largest long side bay as the depth. The beam stem design was based upon the negative moment, thus the loading from Influence Area B was chosen as the live loading to apply during design. Figure 3.4.1 illustrates the influence areas for the 20 foot column spacing option. Appendix D.7 illustrates in greater detail all of the factors taken into account for influence areas in both designs.

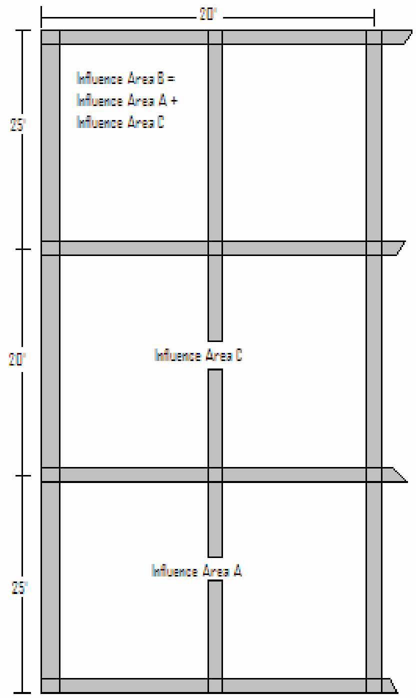

Figure 3.4.1 Influence Areas for T-Beam Design

Once the factored loads were determined, the trial size of the beam was estimated. The estimated trial size was then used to determine the possible actual sizes for the T-beam. This was done by computing the maximum moment and shear that would be present in the beam and determining beam sizes that would accommodate such forces. The equations for this calculation were found in ACI Table 9.5(a) (Table A-14) with an assumed reinforced ratio rho of 0.01.

From the sizes determined to be suitable for the beam a base, height and depth were chosen and the area of steel needed to account for moment loads was computed. Each moment area in the beam, both positive and negative, was taken into consideration, and a suitable amount of reinforcement was selected for each. After the moment steel had been designed, the reinforcing steel needed to provide adequate shear resistance was developed by manner of determining stirrup placement. The final step involving the design of the T-beam was determining the cut off points for the moment reinforcing steel. Two different types of T-beams sections were developed to meet the differing conditions in the building. One T-beam was developed for the interior spans of the building, and the other was designed for the exterior. These two separate designs were necessary due to the smaller loading applied to the exterior beam. Design of this exterior beam did however take into account a loading that would be placed upon it by a twelve-inch thick exterior wall, composed of concrete masonry. This process was a lengthy one and was quickened by the development of an Excel spreadsheet. The T-beam design spreadsheet can be found in Appendix C.7 Reinforced Concrete T-Beam Design, along with a description of its use.

The next step in the design of the gravity system was to develop sizes for girders. In determining the loadings to be placed on the girders, the dead loads of the slab, T-beams or beams, and the self weight of the girder were taken into account along with the design live load and the dead load from other material and mechanical considerations. These loads were converted into point loads that affected the girders where the T-beams intersected the columns and at the mid-span of each bay where the T-beams met with the girder.

In the office, it was decided that the girder would be continuous along the long side of the building. This made the girder 100 feet in length. This was not a clear span however as the girder intersected the columns of the building giving it added support. In the cellblock area, the girder spans were varied consistent with the different column spacing options. The girder spans for option 1 were 54 feet, 36 feet, 18 feet and ten feet. The girder spans for option 2 were 27 feet, 18 feet and ten feet and 18 feet and ten feet for option 3. Refer to Appendix B.9 for the details of these layouts.

The next calculation involved determining the moments caused by the concentrated loads acting along the length of the girder. Once these moments were computed, the maximum moment was used to determine options for the base thickness and height of the girder. This procedure was based upon the requirements called for in ACI Table 9.5 with an assumed reinforced ratio rho of 0.01. After a selection had been made from the base and height options, the reinforcing steel needed in the girder for both positive and negative moments was calculated. Again, an Excel spreadsheet was created to facilitate the multiple design iterations. This spreadsheet and instructions on its proper use are contained in Appendix C.8 Reinforced Concrete Girder Design.

Once girder design was completed, the appropriate columns for the building were devised. The forces developed in the girder design also gave the reaction moment and axial forces that were applicable to the column, thus these forces were used to determine column sizes. The forces in the columns were recomputed after the first floor, then after the third floor, fifth floor, and another set of calculations was done to determine the most effective columns for the sixth floor (the space between the uppermost floor and the roof). Special consideration was given to the first floor as the story height was designed to be 15 feet as opposed to the 12 foot tall stories in the rest of the building. In the cellblocks, the columns were designed after the sublevel floor (a fifteen foot length), then in each level of the cellblock structure (24 foot lengths spanning two floors).

After the forces were identified, the trial size of the columns was determined along with the e/h ratio, with e being the moment acting through the column divided by the axial load. This ratio was used to determine where reinforcing should be placed in the column based on Fig. 11-23 in MacGregor. Considerations were also made to ensure that the column was not too slender. Had the column been too slender, there would have been a risk of buckling which would have been extremely dangerous from a design standpoint as multiple stories could collapse with the failure of a single column. The computation of gamma occurred next, which led to the referencing of Figure A-6 and A-7 in MacGregor to determine an applicable ρ_g for the column. Once ρ_g was computed, options for the area of reinforcing steel were determined. This led to a preliminary column design, which was evaluated to see if it had adequate moment capacity. If the column was acceptable, the lap splices and stirrups were designed along with the spacing for each. A final Excel spreadsheet was developed for these elements. An example of this spreadsheet and a description of how to use it can be found in Appendix C.9 Reinforced Concrete Column Design.

3.5 Evaluating Options

Evaluating options is an integral part of the design process. Unlike applying mathematics and mechanics in the design of the gravity and lateral system, evaluating design options involves more creative thinking and intuitive insight. In this section, knowledge will be given on the basis of evaluating options. Information will be presented regarding the topics covered when evaluating the various layout and material options.

3.5.1 Value Engineering

"Value engineering is an evaluation of a building and its systems and components. It does not mean reduction in cost. It means getting the most for the money."[325] Value engineering is a combination of examining the performance of the structure and its materials as well as achieving cost optimization. Value is defined as performance divided by cost.[326] The purpose of value engineering is to not only cut unnecessary costs, but to increase the benefits of the design leading to the most efficient design.[327] The best design can be described as having the lowest cost or one that performs the best. When evaluating various options, it is critical that the option's impact on the entire design is examined as opposed to sub-optimization of its impact on a particular part. Project performance criteria and cost parameters need to be determined prior to the design stage to effectively use value engineering when determining which option to pursue further.

Performance is a criterion, and its measurement that should be integrated throughout the design. There are many aspects of performance that need to be considered. For this project three categories were evaluated: cost, constructability, and serviceability. The chosen categories and their subsequent measurements need to be reliable and applicable to all options under consideration in order to be effective.

Both qualitative and quantitative measurements are acceptable for evaluating performance. Examples of qualitative measurements include versatility, visibility, mechanical placement, and the capacity for future modifications and renovations to name a few; these are criteria that are not definable by set values. Quantitative measurements

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[325] The Construction Specifications Institute.
[326] Hunter, George & Stewart, Robert B., p.1
[327] Hunter, p.1

on the other hand can be described by established values. Such measurements include the number of members involved, expected construction time, and overall cost.

Performance can be based on the materials of construction. To achieve the best performance, materials should be chosen with a service life that corresponds to the life-cycle of the structure, or one may compare the trade-off between one material's low replacement costs against a more durable material's high initial cost. The added performance gained by using materials that have a longer life-expectancy than that of the structure are negligible when compared to the additional costs accrued.

The materials' added performance gains are a function of its life-cycle costs. Life-cycle costs are described as the time-value of money in the present and the future as well as the service periods of the material. Therefore, performance criteria that are based on the materials of construction represent considerations for cost.

Evaluating different design options is made possible once the performance criteria have been defined. When evaluating an option, consideration must be given to the fact that two or more criteria may have an effect on one another. As a result, it is important to determine which criteria are more important and assign a weighting factor in the design process.

3.5.2 Evaluation in the Design Process

During the design of the gravity system, various options concerning member placement and construction materials were examined while the overall building footprint was maintained. The purpose of the options was to determine the combination of structural layout and materials that would lead to the best overall performing design. For layout concerns, considerations such as column placement and spacing, infill beam tributary widths, and the type of floor systems were explored in the quest for the best solution.

The consideration for column placement was to determine sizes that perform the best in resisting the applied loads while meeting the overall functionality requirements of the structure once complete. A member's performance in resisting loads can be defined as size versus capacity. Typically, the smallest member that can handle the applied loads is said to perform the best. The purpose for varying infill beam spacing was to establish member sizes that were capable of adequately transferring the service loads while controlling member depths to maintain an established floor-to-ceiling height and an overall building height limit. Floor systems considerations were affected by the material used in the structural framing system. Concerning steel construction, two floor systems were considered: composite and non-composite. Reinforced concrete construction focused solely on one-way reinforced slabs. For further descriptions of material considerations used throughout the design, reference section 3.3 Material Considerations.

Having discussed the alternatives that were evaluated, it is important to understand the premise of evaluating the options and its occurrence in the design process. The evaluative stage of the design process occurs between the gravity system design and the lateral system design. The gravity system is based on service loads, which are defined by the materials of construction and the building's intended usage. Furthermore, the gravity system is the first stage in developing a structural system. The lateral system and building foundation are integrated into the gravity system and are defined by the layout established within the gravity system. As a result, it is imperative that the gravity system be well defined before further design is progressed. When regarding the gravity system from perspective of cost, it typically accounts for upwards of 80 percent of the cost of the framing in low-rise structures. This varies in taller buildings due to the large amount of wind pressure to which the faces of the building are exposed.

By achieving a gravity system that functions well, material optimization and exploitation may be utilized with the lateral system design. Because of the complexities involved in the lateral system design with considerations given to applied end moments as well as combined bending and axial loads for columns in rigid frames (and even the added design of braces and their resulting connections in braced frames) it is essential to have an established gravity system. There are many unknowns involved in the design of the lateral system. The lateral loads resulting from wind and seismic action are a function of the building layout and mass. If the evaluative process occurred after the design of the lateral system, iterations of lateral load calculations would need to be performed.

3.5.3 Evaluation Considerations

As discussed above, it is critical for the evaluative process to follow the gravity system design since further design proceedings require a well-defined layout and established materials. The evaluative process for the office building and prison layouts examined three criteria. These included overall cost, constructability, and serviceability.

Cost is a major factor on any project. Cost estimates are imperative to make certain that the finances available fit well within the means of the project. If the overall cost of the project exceeds the funds, then the project will soon be in a standstill with high cost overruns. By providing cost estimates for the design, one can determine where the majority of the costs lie and where the money needs to be allocated. Defining the areas that need large sums of money can lead to their redesign in an effort to lower the cost and enabling the best use of the design effort.

The cost is a factor of the life-cycle costs of the materials used in construction. If the selected materials are inexpensive and offer a shorter service life, their life-cycle costs may be higher because of more required maintenance, repair and replacement. Adversely, materials that have higher initial costs may have lower life-cycle costs because they are more resilient to the adverse effects of weather. Cost estimates were made possible by referencing *RSMeans 2005 Heavy Construction Cost Data* and *RSMeans 2005 Building Construction Data*. The cost estimates considered the cost of the material, labor, assembly, and equipment. Because the unit cost data included allowances for labor, assembly, and equipment, separate analysis was not given to these measurements.

Constructability concerns deal with the effects of the design on the ease of construction. The use of typical and atypical sections, the chosen material of construction, and the use of added components such as shear studs, decking, formwork, and rebar all have an effect on constructability. In addition, consideration for any limitations regarding the transportation of structural members from the fabricating shop to the work site, the use of prefabricated or site fabricated assemblies, and the requirement for specialized labor must be considered when evaluating constructability.

The final evaluation criterion is serviceability. Serviceability explores the intended usage of the structure once complete and how well the design allows for the intended use. A design that does not allow the owner to use the structure as intended or negatively impacts the usage can be considered a poor design. Consideration for open space and versatility with minimal inconvenience into the habitable space must be made.

Table 3.5.1 summarizes the specific performance attributes that were used to evaluate the cost, constructability, and serviceability of the office building and prison structure.

Table 3.5.1 Evaluation Criteria Breakdown

Evaluation Criteria	Performance Attributes
Cost	• Component cost breakdown • Total cost
Constructability	• Total number of members • Types of connections required • Added components (shear studs, decking, reinforcement, etc.)
Serviceability	• Floor depth • System Integration • Visibility • Versatility

3.5.3.1 Evaluating Cost

The evaluation of cost was broken into two performance attributes: component cost and total cost. Component costs were made for beams (including shear studs if composite action was designed for), girders, columns, and slab costs including all their requirements (framework, rebar, finishes, etc.) where reinforced concrete was considered. The component costs were examined to see where the costs lie within each option. By performing this evaluation, consideration could be made for which option had the best infill beam spacing layout, which had the best column spacing, and which has the best slab design (either composite or non-composite). Graphical comparisons were created to show the relative component costs within each option as well as comparing them amongst the other options.

The total cost was examined to make a comparison between each layout/material option in terms of the bigger picture. The purpose was to determine which combination offered the lowest total cost. Using the total costs for each option, graphic comparisons were made. The purpose of these graphical comparisons was to see the difference in cost between the various material considerations for each option.

3.5.3.2 Evaluating Constructability

Constructability was based on evaluating many performance attributes. One such attribute was the total number of members. An option may have many members when compared to another option for the same footprint. The added benefit of having more members is that their overall size is less, making them easier to handle. On the other hand, with more members conflict arises with scheduling, availability of members, the transportation of the members, and the number of connections needed to be made.

The types of connections were evaluated. In steel, mechanical connections in the form of welding and/or bolting must be made. However, with reinforced concrete, the advantage is that no separate connections devices are used. The connections between members occur with the connections between rebar and the concrete placement and subsequent curing.

Constructability was also evaluated based on added components required. These included shear studs, decking, and rebar. These components were evaluated based on constructability because they are dependent on the type of material and design assumptions made, and are independent of one another. Each results in more construction time and the possibility of specialized labor, primarily with the shear stud and rebar installation.

3.5.3.3 Evaluating Serviceability

Four performance attributes were considered in the evaluation of serviceability: floor depth, system integration, visibility and versatility. Floor depth is a critical consideration because it has an affect on the floor-to-ceiling height. If the floor-to-ceiling or clear story height is a critical dimension, often the overall building height will increase to accommodate the specified clear story height plus depth of construction for the floor system. Large floor depths can result in added costs associated with exterior wall finishes and the length of vertical mechanical components such as piping, conduit, and cabling needed to cover the added building height. Therefore, maintaining a minimum floor depth is critical.

System integration is also an important consideration. The placement and location of MEP systems such as plumbing, electrical, and HVAC is evaluated for ease of installation, maintenance, and repair. The mechanical positioning becomes a function of floor depth because it is often placed within the ceiling plenum space. Therefore, consideration is given to those structural elements that allow mechanical components to pass through them with no added costs or design concern.

The final serviceability performance attribute examined was a combination of visibility and versatility. For the prison structure, visibility was only considered since versatility is not a main priority because the facility only has one function. Visibility was considered for security reasons. A layout option that placed columns within the dayroom was considered undesirable. These columns would impede the guards' ability to view the prisoners and their activities. Furthermore, for every guard added, an additional $1 million would be added to the life-cycle costs of the institution.[328] Therefore, visibility is critical in minimizing extraneous costs.

Versatility was a concern for the office building. This attribute was considered to allow for realignment, renovation, and expansion of the office. Options that require load bearing walls, close column spacing, or the intrusion of braces in undesirable locations are therefore less desirable.

3.5.4 Evaluating Strategies

Three evaluative schemes were derived to evaluate the efficiency of each option in terms of the criteria previously described. These schemes include cost comparison, a relative comparison, and a grading comparison. Table 3.5.2 illustrates the considerations of each scheme as well as the performance criteria that each evaluated.

[328] http://www,bls.gov/oco/ocos156.htm

Table 3.5.2 Evaluative Scheme Considerations

Evaluative Schemes	Considerations	Evaluated Performance Attributes
Cost Comparison	• Determine component cost • Determine overall cost • Number of components	Cost • Component cost breakdown • Total cost • Cost trends
Grading Comparison	• Rankings a function of the best option • Values assigned for qualitative measures • Rankings based on 0 (worst) to (1) best • Relative weight assigned to criteria to not over penalize	Constructability • Number of members • Type of connections • Shear Studs • Decking • Rebar Serviceability • Floor depth • Mechanical placement • Visibility / Versatility
Relative Comparison	• Options ranked from 1 (best) to "N" (worst) • No relative weight between each criteria was assigned	Cost • Overall cost Constructability • Number of members • Type of connections Serviceability • Floor depth • Mechanical placement • Visibility / Versatility

3.5.4.1 Grading Comparison Evaluation

The process of evaluation by grading comparison assigned rankings based on the user's judgment. Values ranged from 0.0 to 1.0, with 1.0 representing the best. With the exception of comparing the number of members and floor depth, values chosen for ranking the other criteria were qualitative, reflecting the perceived importance of each criterion to the overall design. The values for grading the number of members and floor depth were based on a relative comparison to the option that had the least number of members and the smallest floor depth respectively. When ranking for some items, such as shear studs, decking, and reinforcement, the options that required the most number of elements were given the lowest value. A relative weight was considered when assigning the values for each criterion. For instance, the installation of decking is much easier than the installation of shear studs, which is quicker and less cumbersome than the installation of rebar. Therefore, options with decking were penalized but not as severely as those with shear studs or rebar. Furthermore, since system integration is a function of floor depth, the rankings assigned for this attribute did not penalize the option as much as the floor depth rankings did. The rankings for each option with respect to the chosen criteria were multiplied together and divided into the overall cost. The option with the lowest cost/ranking was determined to offer the best performance.

3.5.4.2 Relative Comparison Evaluation

The relative comparison was a more refined approach, which was based on ranking each option against the others on various performance criteria. The ranking used reflected the number of options considerations, one through "N" where N is the total number of options under consideration. The option that was ranked one meant that it was the "best" in that particular evaluation criterion. The option that was given a value of ranked N meant that the option performed the "worst".

The scope of constructability concerns was limited because of the different methods of construction used for each material and the difference between their most effective uses in ideal systems. Many performance attributes considered in the relative comparison were neglected in the grading comparison. These included decking, shear studs for composite action, and rebar for reinforced concrete. Each of these items was included in the cost estimates. Therefore, it was not reasonable to consider these items for ranking because a penalty had already been implemented on their behalf.

3.5.4.3 Results

Although three evaluative schemes were used, only one was chosen as a determinant for the best performing option. The purpose of the cost comparisons was to see how much each option cost and the cost breakdown for each. Comparisons were made between the results generated from the grading comparison and the relative comparison. The purpose was to see if there were similar trends resulting from both analyses. The relative comparison was chosen to be the most beneficial evaluative scheme because it offered the concise analysis with minimal user implications. Results from the evaluation are further discussed in section 4.1 Gravity System Design Results.

3.6 Lateral Design

3.6.1 Lateral Considerations

Similar to gravity loads; structural systems are exposed to lateral loads. These loads can be attributed to events such as wind or seismic activities. The appropriate determination of these loads and their affects of the structure are critical in the design of the lateral framing system. Procedures for calculating the lateral loads are to be followed as designated in the local governing building code. The lateral load determination for the office and prison building follow the *2003 International Building Code (IBC 2003)* and the associated *ASCE 07-02* provisions.

The procedures for calculating wind loads in the *IBC 2003* take into consideration basic wind speed, topography, building use, structure dimensions, wall openings, etc. Basic wind speed and topography are greatly dependent on the specific site location of the project. All of these considerations ultimately modify the design wind pressure for the structure. Design wind speeds are provided in the *IBC 2003* for most locations in the

Seismic United States. Based on the chosen site, the assumed values for

calculating wind loads are shown in Table 3.6.1. Procedures for calculating topographical factors due to hills and escarpments are also provided in the *IBC 2003* and *ASCE 07-02*. See Appendixes F.1, F.2, and F.3 for additional information on the calculating the wind pressures for the office and prison structures.

Table 3.6.1 Wind and Seismic Design Assumptions

Wind	Basic Wind Speed Exposure Factor Importance Factor	100 mph Exposure C 1.15
Seismic	Ground Acceleration Seismic Response Factor Soil Classification Weight of Building Seismic Use Group Importance Factor	.25g / 0.09g Varies Class C Varies II 1.25

loads acting on a building develop from ground accelerations. Depending on soil conditions, weight of the structure and structural framing category, seismic loads vary greatly. Similar to topographic factors, soil conditions are site specific, thus a particular location or soil type must be established. Since seismic loads are caused by a differential acceleration caused by the soil-structure interaction, the mass (weight), and stiffness of the structure and the magnitude of the local ground acceleration directly affect the lateral forces acting on the structural frame of the building. See Appendixes F.4 and F.5 for sample calculations seismic load determination.

3.6.2 Lateral Systems

For gravity loads, each structure has a gravity system to resist these loads. Correspondingly, each structure has a lateral system to oppose seismic or wind loads. Pressure from wind loads act on exterior wall elements and transfers loads into spandrel beams. For spandrel beams that do not maintain the capacity to transfer wind loads to their ends and into girders, the load is collected into a shear diaphragm. The function of the shear diaphragm is to gather the lateral loads at each story level and transfer them to the lateral resisting frames. Shear diaphragms also assist in transferring seismic loads to lateral resisting frames. Since seismic loads are inertia forces and a function of structure weight, with large amounts of structure weight attributable to floor system dead loads, lateral seismic loads act on the interior as well as the exterior of the structure. Consequently, interior portions of the structure must be capable of transferring seismic lateral loads from heavy elements such as floor systems.

Concrete slabs are commonly used for floor systems in non-residential structures. Concrete floors include reinforced concrete and composite systems. These slabs perform closely to an ideal case of a rigid diaphragm,[329] making them an effective and important component of the lateral load system. A rigid diaphragm acts as a short beam element, with little deflection in the plane of the load (this can be modeled by holding a textbook at the top and bottom edges with the palm of your hands and applying a load to the spine). The short beam analysis simplifies the determination of the shear stresses in the rigid diaphragm. For a short beam, most of the element stresses act in shear, not bending, allowing the diaphragm to effectively transfer loads to its transverse edges (edges parallel to the load), and into a lateral resisting frame.[330]

Three common lateral framing systems include braced frames, moment frames, and shear walls. Because lateral systems are most frequently integrated with the gravity system, exploiting material abilities and properties of members used in the gravity system contributes to the efficiency of the structural design. Thus, members of the gravity

[329] Luttrell, Larry D.
[330] Army Corps of Engineers, Section 9-1

system ultimately see increased axial and bending stresses because of combined gravity and lateral loads. For this reason braced frames and moment frames were compared to evaluate the frames for steel and concrete scenarios.

Braced frames are typically composed of columns, girders, and braces with pin-ended connections. Pin-ended connections simplify the analysis of the frame because member forces and stresses can be calculated using truss analysis. The members in a braced frame act as axial members when lateral loads are applied to the joints of the frame. This means that the braces act as axial members and can be designed the same way the columns were designed for gravity loads.

Moment frames, on the other hand, consist of only girders and columns with rigid connections. This means that the members resist load both axially and in bending, which complicates the analysis because of the combined stresses and secondary moment effects. Also, there are interactions between the members based on relative stiffness and rigidity, which increases the difficulty of analyzing the load paths and resulting member forces. In order to calculate the stresses in moment frame members, some form of indeterminate structural analysis, such as moment distribution calculations, must be conducted. These analyses can be done by hand calculations or by computer-based structural analysis software. For both moment frames and braced frames, RISA2D Demo was used to analyze the response of the structures.

In the case of the office structure, the frame reaches a height of ninety feet. At this height, one may begin to be concerned with the allowable lateral deflection (sway) of the frame due to wind forces. A braced frame typically has more sway control than a moment frame because it is restrained by the axial stiffness of its members, thus the sway is solely a function of axial compression and tension deflection. In comparison, the members of a moment frame undergo both bending and axial deflection, generally resulting in more sway than for a braced frame. Refer to Figure 3.6.1 and Figure 3.6.2 for a graphical interpretation of sway tendencies in braced and moment frames.

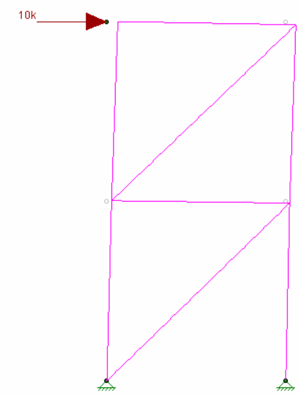

Figure 3.6.1 Braced Frame Deflection

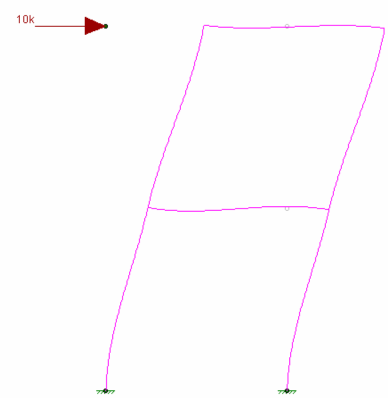

Figure 3.6.2 Moment Frame Deflection

One of the difficulties with using braced frames in structures requiring large amounts of open space is that brace members must lie in the plane of the bay. This means that any brace elements in the interior portions of the building are likely to obstruct open space. Consequently, braced frames are often located in permanent walls such as those surrounding elevator shafts, stairwells, and exterior walls. In the case of moment frames, no braces are needed, which means that the moment frame is no more likely to intrude on open space than ordinary post-and-beam construction. As a result, moment frames are often used where lateral support is needed, but the intrusion of braces is unacceptable.

Depending on the frame material, certain frames are typically not used. For example, concrete systems use moment frames and shear walls, while steel systems utilize braced frames and moment frames. In the case of this project, when concrete framing scenarios are designed, moment frames are used, and when steel frames scenarios are designed, braced frames are applied. Because of the complexity of moment connections in steel and the sway control benefits of braced frames, braced frame systems are generally a preferential and economic choice in steel lateral resisting frames. In the case of concrete, moment connections for girders are the standard connection used for gravity frame design, thus insurance of proper rebar detailing must be made to transfer the load. Table 3.6.2 shows the lateral resisting frame considerations for the office and prison structures.

Table 3.6.2 Lateral Resisting Frame Considerations

Material	Frame Type	Connection Considerations	Sway Considerations	Open Space Obstruction
Steel	Braced	Shear resisting	Limited sway	Potential space obstruction from braces
	Moment	Shear resisting Moment resisting	Potential for sway control issues	No addition space obstruction
Concrete	Moment	Integrated shear and moment resisting	Potential for sway control issues	No addition space obstruction

Braced frames within the scope of this project consist of WF-shape columns and girders, but a number of different brace sections, as well as brace geometry were explored. The use of both K, cross, and diagonal braces were all investigated for use in the braced frames. Braces orientation should be a balance of unbraced length and brace angle. Ideally, a brace should be oriented at near horizontal to minimize axial loading due to lateral load. In order to transfer the load from the brace into the girder-column connection, braces oriented as in the aforementioned manner typically required a near 45 degree brace angle. Refer to Figure 3.6.3 for illustrations of the three mentioned brace orientations.

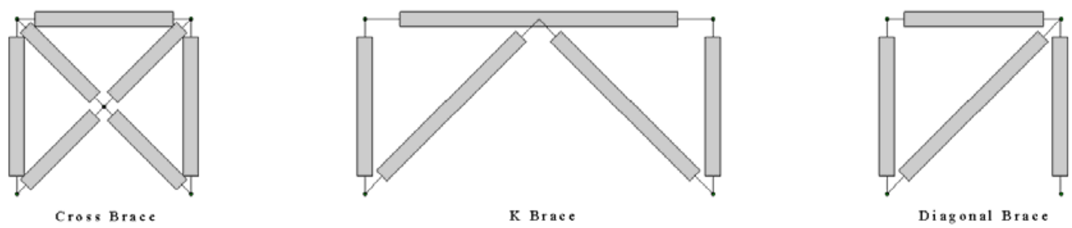

Figure 3.6.3 Cross, K, and Diagonal Braced Frame

In some cases, story height and bay width do not allow for the 45 degree angle, allowing the brace to tie into the frame at the girder-column connection. As the angle of a brace diverges from horizontal, the geometry causes that member to work harder, meaning more force to resist. Aside from the geometry, the number of connections for a particular brace pattern is considered when investigating the constructability of a frame. Figure 3.6.3 shows the five connections in the cross, three in the K, and two in the diagonal brace.

Depending on the brace pattern, the unbraced length can be affected. In the case of the cross brace, the weak axis of one brace can potentially be braced at its mid-span by the other crossing member, leading to an increase axial capacity. Both the K and diagonal brace remain unbraced along the length of the member, which can lead to concerns of compression failure.

Aside from being able to choose from a number of different brace patterns, different member sections can be utilized. WF-shape, HSS, and double angles were examined for bracing in both the office and prison structure. Table 3.6.3 shows considerations for the use of the different sections in the braced frames.

Table 3.6.3 Brace Section Considerations

Section	Considerations
Wide Flange	Wide flanges are used throughout the steel schemes, maintaining continuity Simple connection with gusset plate
HSS	Can be symmetrical about both axes
Double Angles	Can be oriented long leg back to back, short leg back to back, and equal leg back to back

For the reinforced concrete lateral frame design, moment frames were considered to be placed throughout the structure. The use of shear walls as part of the lateral frame was not considered because the intrusion into open space. The moment frames are integrated into the reinforced concrete gravity frame system. Moment frames would likely require modifications to member sizes but without the design of additional members associated with braced frames.

4 Results

Topics presented here reflect the outcome of the gravity design and lateral considerations. A breakdown is given for the gravity design with reference to the outcomes of the scenario evaluation. The results of the analysis yielded which type of gravity system to further develop a proposed solution. Upon choosing the system, the lateral system was designed. The resulting data is provided to give a sense of the consequences of lateral loads on member sizing, the resulting costs, and the type of lateral systems utilized.

4.1 Gravity System Design Results

Following the procedures delineated in the methodology, schematic designs of the office building and prison cellblocks were completed. Numerous layouts were explored with a combination of various material considerations. Each of the schematics for both building types were evaluated and compared using the procedures outlined in section 3.5. The following sections present the results from the evaluations of the gravity systems.

4.1.1 Office Gravity System Design

Table 4.1.1 summarizes the various frame and floor system combinations considered for the two layout options. The results from the six various designs are presented in Figure 4.1.1 and Table 4.1.2. Figure 4.1.1 illustrates the cost distribution within each scheme. Cost is given for each member type as well as a summation. Table 4.1.2 provides information regarding the number of discrete beams, columns and girders required for each option. Elaboration on the cost breakdown for each gravity system design is presented in Appendix B-1.

Table 4.1.1 Office Scheme Options

		Floor System		
		WF-Shape Composite	**Reinforced Slab**	**Composite Open-Web Joist**
Frame	**WF-Shape**	Option 1 / Option 2	Option 2	Option 2
	Reinforced		Option 1 / Option 2	

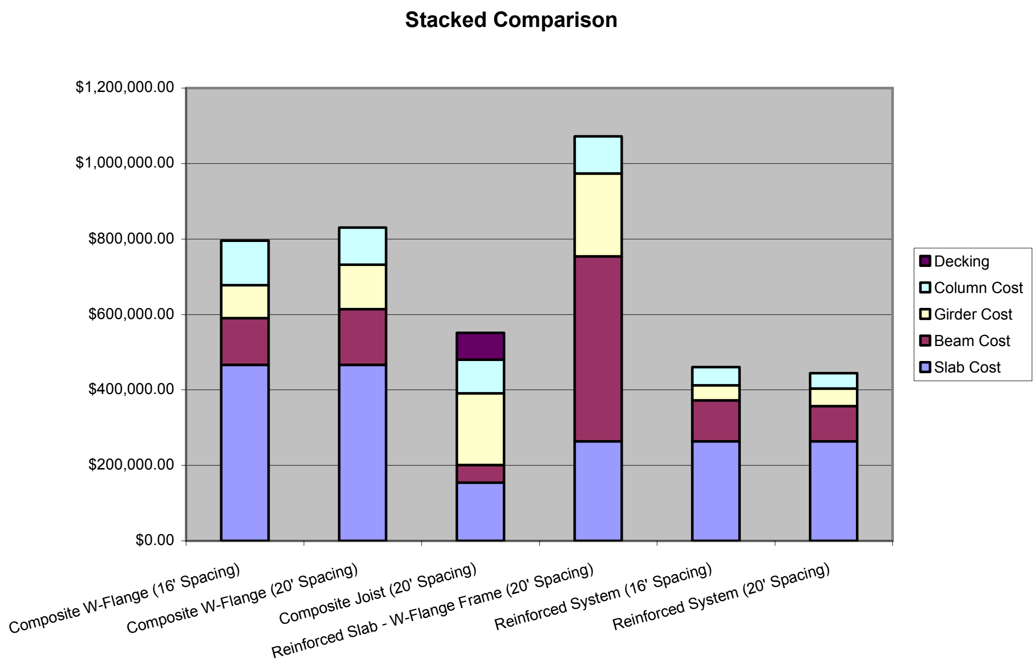

Figure 4.1.1 Office Price Breakdown

Table 4.1.2 Office Member Count

Scheme	Option	# Beams	# Girders	# Columns	Total #
Composite System	16' Spacing	189	196	128	513
w/ W-Flange Sections	20' Spacing	231	140	96	467
Composite System	16' Spacing	---	---	---	0
w/ Joist Sections	20' Spacing	105	266	96	467
Reinforced Slab - Steel Frame	16' Spacing	---	---	---	0
	20' Spacing	231	140	96	467
Reinforced System	16' Spacing	91	28	192	311
	20' Spacing	77	28	120	225

Using the information given in Figure 4.1.1 and Table 4.1.2, an evaluation amongst all options was conducted. Focus was given to cost as well as overall performance. Table 4.1.3 presents the results from the grading analysis. The results of Table 4.1.3 show that the reinforced concrete options are the least expensive. When functionality considerations are taken into effect, the reinforced concrete schemes become less desirable because of greater floor depth, more difficult mechanical placement, and other factors based on a combination of functionality and cost performance. It was determined that the composite open-web joist system would offer the best design option of those presented. For the reasons of design exercise and competitive evaluation, both composite open-web joist and the reinforced concrete system (option 2 – 20ft spacing) were further developed in design in terms of lateral loads to create a finalized design.

Table 4.1.3 Office Relative Evaluation

		# Members	Connections	Floor Depth	Mechanical	Versatility	Cost	Total	Overall Ranking
Composite System	*16' Spacing*	6	2	1	3	2	4	18	5
w/ W-Flange Sections	*20' Spacing*	3	2	3	3	1	5	17	4
Composite w/ Joist	*16' Spacing*	---	---	---	---	---	---	---	---
w/ Joist Sections	*20' Spacing*	3	2	1	1	1	3	11	1
Reinforced Slab	*16' Spacing*	---	---	---	---	---	---	---	---
w/ Steel Frame	*20' Spacing*	3	2	3	3	1	6	18	5
Reinforced System	*16' Spacing*	2	1	5	3	2	2	15	3
	20' Spacing	1	1	6	3	1	1	13	2

4.1.2 Prison Cellblock Gravity System Design

Various combinations of layouts and materials considerations were utilized in the design of the prison cell. Table 4.1.4 provides a checklist of schemes investigated. The results from the various designs are given in Figure 4.1.2 and Table 4.1.5. Figure 4.1.2 shows the cost allocation for each scheme; Table 4.1.5 presents the number of individual members and types of members required.

Table 4.1.4 Prison Cellblock Scheme Options

		Floor System		
		WF-Shape Composite	**Reinforced Slab**	**Composite Open-Web Joist**
Frame	**WF-Shape**	Option 1 / Option 2 / Option 3	Option 1 / Option 2 / Option 3	Option 1
	Reinforced		Option 1 / Option 2 / Option 3	

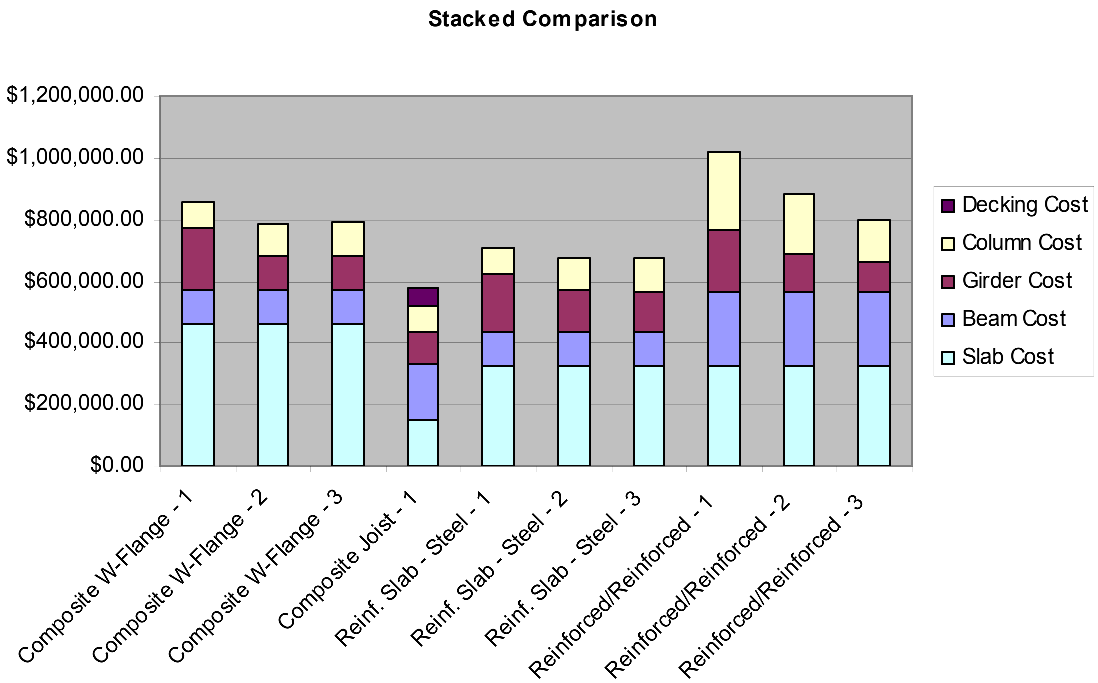

Figure 4.1.2 Prison Cellblock Price Breakdown

Table 4.1.5 Prison Cellblock Member Count

Scheme	Option	# Beams	# Girders	# Columns	Total #
Composite System	1	250	218	129	597
w/ W-Flange Sections	2	250	251	150	651
	3	250	272	165	687
Composite System	1	121	237	129	487
w/ Joist Sections	2	---	---	---	---
	3	---	---	---	---
Reinforced Slab - Steel Frame	1	156	222	129	507
	2	156	237	150	543
	3	156	258	165	579
Reinforced System	1	216	51	129	396
	2	216	96	150	462
	3	216	108	165	489

Using the information presented in the above figure and table, two evaluations were conducted. Table 4.1.6 shows the relative comparison, while Appendix B.5.2 is the grading comparison. Comparing the results between both evaluations, the top three rankings are the same for both. After the top three rankings, the similarities diverge. Because both evaluations yielded similar results for the top rankings, further consideration and use was only given to the relative evaluation for the reason that the grading evaluation imposed user bias. Based on the overall ranking, the composite open-web joist open was considered for further design development via the application of lateral loads.

Table 4.1.6 Prison Cellblock Relative Evaluation

		# Members	Connections	Floor Depth	Mechanical	Visibility	Cost	Total	Overall Ranking
Composite w/ W-Flange	Option #1	8	2	4	3	1	8	26	7
	Option #2	9	2	1	3	5	5	25	5
	Option #3	10	2	1	3	10	6	32	10
Composite w/ Joist	Option #1	3	2	7	1	1	1	15	1
		---	---	---	---	---	---	---	---
		---	---	---	---	---	---	---	---
Reinforced Slab w/ Steel Frame	Option #1	5	2	4	3	1	4	19	2
	Option #2	6	2	1	3	5	3	20	3
	Option #3	7	2	1	3	10	2	25	5
Reinforced	Option #1	1	1	10	3	1	10	26	7
	Option #2	2	1	3	3	5	9	23	4
	Option #3	4	1	2	3	10	7	27	9

4.1.3 Gravity System Design Results Conclusions

Based on the evaluative results for the office schemes, composite open-web joist system and reinforced concrete option 2 were chosen for further design development. The composite open-web joist system offered the best overall performance, but because of further considerations for lateral design, the reinforced scheme was also selected to be elaborated upon. The need for braced frames in the open-web joist system would result in additional cost, while these additional members are not required for the moment frames of the reinforced concrete. Following the lateral design, a final relative evaluation and cost comparison was conducted.

As seen in the graphical representation of the cellblock price breakdown (Figure 4.1.2), the composite WF-shape options cost varied as the column placement changed. This variation can be roughly approximated as a parabolic curve. When looking at the composite WF-shape portion of the graph (left three bars), a large portion of the cost is likely attributed to large girder sizes in option 1. On the other hand, the cost increase of the third option is linked to the increased number of columns, even though the member sizes are reduced. Option 2 had a balance of both girder and column sizes, which ultimately lead to a lower cost than the other options. Similar to the composite WF-shape schemes, the reinforced concrete slab on steel frame options displayed a comparable trend.

The reinforced concrete slab and frame schemes showed what appeared to be the negative slope of a parabolic curve. This would lead one to believe that the layout with the lowest cost had not yet been reached. An increase in the number of intermediate columns would control the required girder sizes. At some point, a limit would be reached where the cost of increasing the number of columns would exceed the cost savings of reducing girder sizes. A fourth possible layout could have been considered for reinforced concrete, but the feasibility and function of the layout would result in a further reduction in operational cost and performance.

Based on the evaluative results for the prison cellblock, composite open-web joist system was chosen for further design. This can be attributed to the excessive spans of the dayroom. Because open-web joists are intended for long spans, the dayroom span took full advantage of the section properties. Similar spans can be found in the recreation area of the prison core; as a result, open-web joists were chosen to be utilized in the prison core design.

4.2 Lateral System Design Results

4.2.1 Concrete Option - Office

As discussed in the methodology, concrete frames use either moment-resisting (rigid) frames or shear walls to resist lateral loads. In consideration for the office building, rigid frames were designed for the lateral systems because of their ability to allow uninterrupted open space. Shear walls require permanent, continuous walls in interior portions. The office layout was not conducive for the use of shear walls because only the walls around the elevator core would allow for these walls. Only three of the sides would have continuous walls, providing for shear resistance.

Analysis for the rigid frames was conducted using the calculated wind loads discussed in section 3.6.1 Lateral Considerations. Moment resisting frames were used throughout the building in all bays. Girder and columns sizes were modified to resist the additional axial and bending stresses resulting from the wind loading. The cost spreadsheets were adjusted to reflect the modified member sizes. Following the wind analysis and associated design modifications, the loads from seismic considerations were compared to those of wind. The magnitude of the seismic loads was less than those of wind for this scheme. As a result, the wind loads were the controlling loads and the initial modifications for wind proved sufficient for lateral design. All applicable information regarding these findings can be found in Appendices F.1 and F.5.

Reinforced concrete structures act in a manner that necessitates the redesign of all members for lateral loading if one particular bay is under-designed. With this in mind, the majority of girder and column sizes were increased to accommodate lateral loading. The increase in cost after the lateral considerations had been taken into account for this structure was a total of \$54,000 to a cost of \$490,000. The percentage of cost of the building attributable to the gravity system of the building is 88 percent.

4.2.2 Steel Option – Office and Prison

Regarding steel design, braced frames were chosen to control frame drift. With the office and prison structures approaching heights of 90 feet and 63 feet, respectively, drifts resulting from lateral loadings had the potential to exceed the allowable limits.

While additional members needed to be added as braces, the primary frame members (columns and girders) remained virtually unchanged as a result of lateral loads. Some lower level columns required an increase in member section because of the added axial loads. Had a moment frame been designed, both the columns and the girders would have required larger member sizes to adequately transfer the combined axial and bending loads resulting from the wind or seismic action.

Considerations for braced frame placement were given to locations of permanent fixtures and exterior walls. Placing a braced frame around a permanent fixture is an ideal location so the braced members do not impede on open, usable space. For these buildings, braced frames were placed around stairways, elevator shafts, permanent partition walls, and cell units. Both K- and diagonal-braces were considered in the design process, based on maintaining satisficing brace geometry depending on various frame locations.

Occurring in many structures is a situation described as cantilever lateral condition. This occurs when a portion of the structure extends beyond the outermost lateral frames. This leaves the laterally unsupported portion of the structure to resemble the overhanging end of a cantilever beam. The cantilever lateral condition can occur in both symmetrical and nonsymmetrical buildings. Consideration to elevate this in combination with optimum geometry drove the placement of the braced frames.

Office Building Lateral Design

In the case of the office building, K-braces were used in conjunction with diagonal-braces to resist the lateral loads. Braces were first placed orthogonally to each other at the corners of the structure. This eliminated the cantilever lateral condition. The bay size dimensions at the corners allowed for near perfect geometry for the K-braces. Consideration was also given to locations along the elevator shaft and the stairways. See Figure 4.2.1 for frame locations within the office structure.

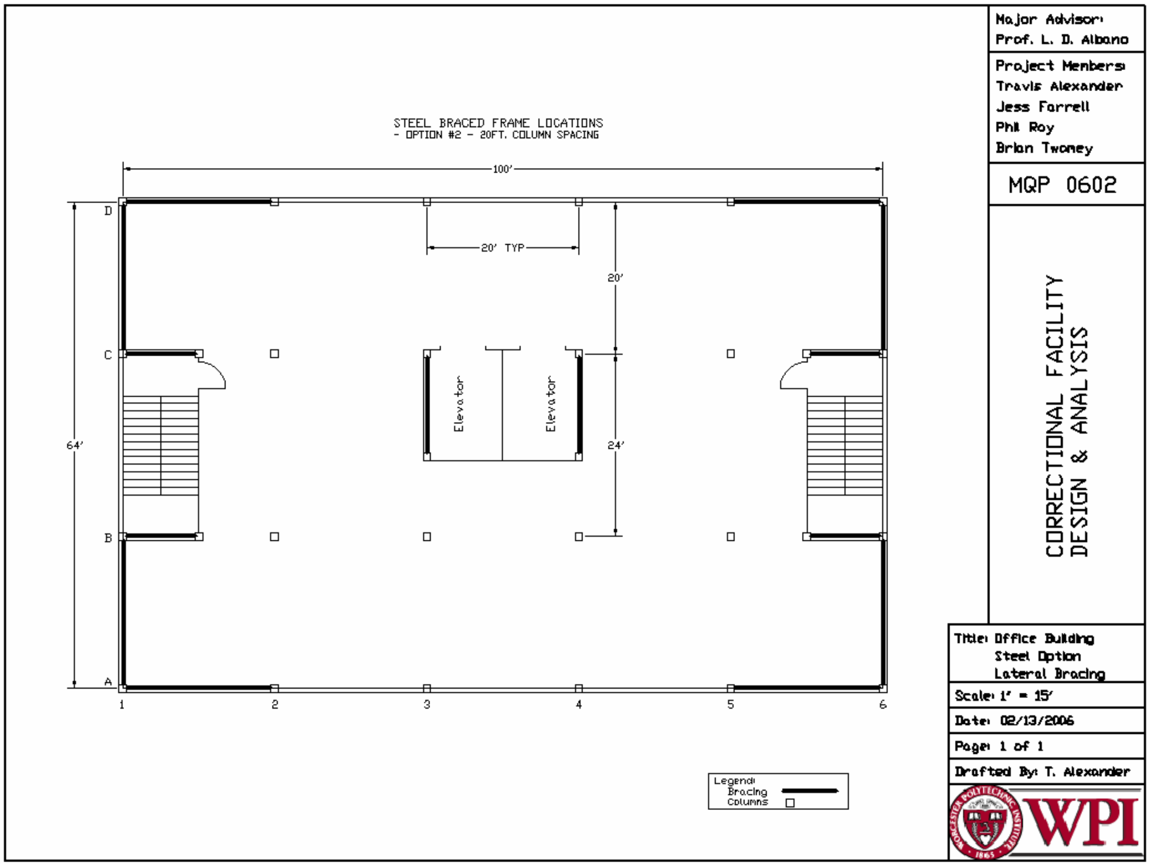

Figure 4.2.1 Braced Frame Locations - Office Steel Design

Concerning the braces located around the stairway, the geometry yielded the use of diagonal-braces. These members were designed primarily for tensile stresses resulting from wind loads. Because seismic activity acts in all directions throughout the structure, the adequacy of the members were examined to be satisfactory for compressive forces resulting from either wind or seismic loads.

With considerations for wind loads, brace sections were designed for both X- and K-orientations. For each orientation, a number of different sections were chosen to establish the lightest brace option. Refer to Table 4.2.1 for the comparison of orientation and section. The bolded values show the total weight for a typical brace in the office. From this comparison, hollow structural steel (HSS) sections were chosen to be the lightest. For continuity, HSS sections were selected to be used for all steel braces. As a result of lateral design, the cost of the structural steel system for the office increased by $68,000, from $552,000 to $620,000.

Table 4.2.1 Office Brace Orientation and Section Comparison

OFFICE X-Bracing Comparison
Middle Brace - Short Direction (BC3)

		W-Flange		HSS		Double Angle					
Location	**Length (ft)**	**Section**	**Weight**	**Rectangular**	**Weight**	**Equal Leg**	**Weight**	**LLBB**	**Weight**	**SLBB**	**Weight**
Basement	20.52	W10x33	0.7	HSS8x8x1/2	1.0	2L6x6x3/4	1.2	2L8x6x3/4	1.4	2L8x4x7/8	1.4
1st Floor	20.52	W10x30	0.6	HSS8x6x3/8	0.7	2L5x5x3/4	1.0	2L8x6x3/4	1.4	2L6x4x5/8	0.8
2nd Floor	18.44	W10x22	0.4	HSS8x4x3/8	0.5	2L5x5x1/2	0.6	2L6x4x1/2	0.6	2L6x3-1/2x1/2	0.6
3rd Floor	18.44	W10x22	0.4	HSS8x4x3/8	0.5	2L5x5x1/2	0.6	2L6x4x1/2	0.6	2L5x3-1/2x1/2	0.5
4th Floor	18.44	W10x19	0.4	HSS6x4x5/16	0.4	2L4x4x1/2	0.5	2L6x4x1/2	0.6	2L5x3x1/2	0.5
5th Floor	18.44	W10x12	0.2	HSS4x4x3/8	0.3	2L3-1/2x3-1/2x3/8	0.3	2L5x3x7/16	0.4	2L3-1/2x3x3/8	0.3
6th Floor	18.44	W10x12	0.2	HSS3-1/2x3-1/2x3/16	0.1	2L2-1/2x2-1/2x3/8	0.3	2L4x3x3/8	0.3	2L3x2-1/2x1/4	0.2
		∑sum (tons) =	**2.9**		**3.5**		**4.5**		**5.3**		**4.2**

OFFICE K-Bracing Comparison
Middle Brace - Short Direction (BC3)

		W-Flange		HSS		Double Angle					
Location	**Length (ft)**	**Section**	**Weight**	**Rectangular**	**Weight**	**Equal Leg**	**Weight**	**LLBB**	**Weight**	**SLBB**	**Weight**
Basement	19.21	W10x33	0.6	HSS6x6x5/16	0.4	2L6x6x1/2	0.8	2L8x4x3/4	1.1	2L8x6x3/4	1.3
1st Floor	19.21	W10x33	0.6	HSS6x6x5/16	0.4	2L6x6x1/2	0.8	2L6x4x5/8	0.8	2L8x6x3/4	1.3
2nd Floor	16.97	W10x26	0.4	HSS6x4x5/16	0.3	2L5x5x1/2	0.6	2L5x3-1/2x1/2	0.5	2L6x4x9/16	0.6
3rd Floor	16.97	W10x22	0.4	HSS6x4x5/16	0.3	2L4x4x5/8	0.5	2L5x3-1/2x1/2	0.5	2L6x4x9/16	0.6
4th Floor	16.97	W10x22	0.4	HSS4x4x1/4	0.2	2L3-1/2x3-1/2x7/16	0.3	2L5x3x7/16	0.4	2L4x3-1/2x1/2	0.4
5th Floor	16.97	W10x22	0.4	HSS4x4x1/4	0.2	2L3-1/2x3-1/2x7/16	0.3	2L3-1/2x3x5/16	0.2	2L4x3-1/2x1/2	0.4
6th Floor	16.97	W10x12	0.2	HSS3x3x1/8	0.1	2L3-1/2x3-1/2x7/16	0.3	2L3-1/2x3x5/16	0.2	2L4x3-1/2x1/2	0.4
		∑sum (tons) =	**3.0**		**2.0**		**3.6**		**3.6**		**5.1**

Prison Cellblock Lateral Design

The lateral design in the prison cellblocks was more complex because of the nonsymmetrical layout. The placement of braced frames was limited to be within the partition walls of the cellblocks. Based on geometry, diagonal-braces were chosen to be the most efficient. One cellblock was examined as a unit as opposed to examining the prison superstructure as a whole. The lateral design was based on the premise that the wind would only act on the exposed face of each cellblock. This means that diagonal braces could be oriented such that they act primarily in tension. With all four cellblocks oriented around the core, there would be resistance to wind in all four direction of the superstructure.

When considerations were given for seismic loads, the assumption could not be made that the load would be applied only to the exposed face. Instead, loads could act in any direction applied to interior and exterior portions of the cellblock. See Figure 4.2.2 for the potential seismic load directions. As seen in seismic load directions 2 and 3, braced frames were required along the diagonally orientated cell walls. Through analysis, it was determined that each frame has approximately 7.77K of base shear resistance based on the weight distribution of the cellblock. Refer to Figure 4.2.3 and Figure 4.2.4 for the proposed braced frame locations within the prison cellblocks.

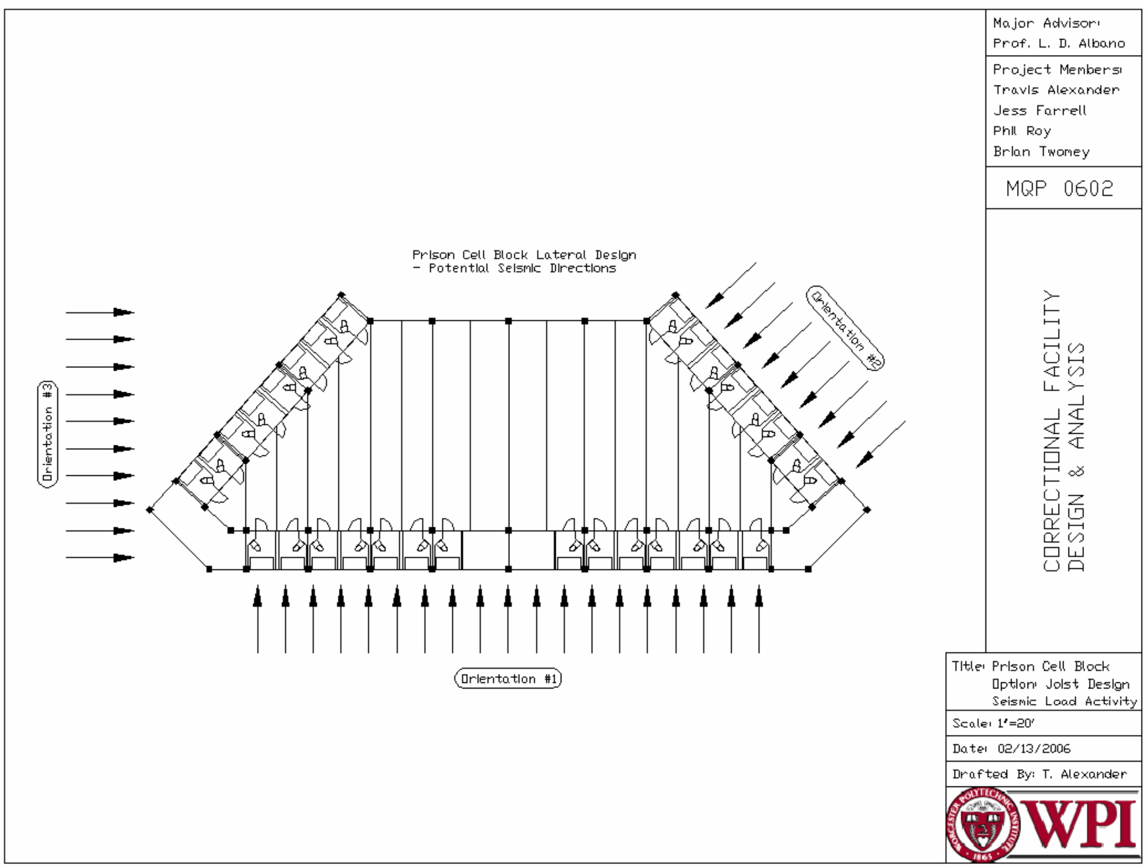

Figure 4.2.2 Prison Cellblock Potential Seismic Load Directions

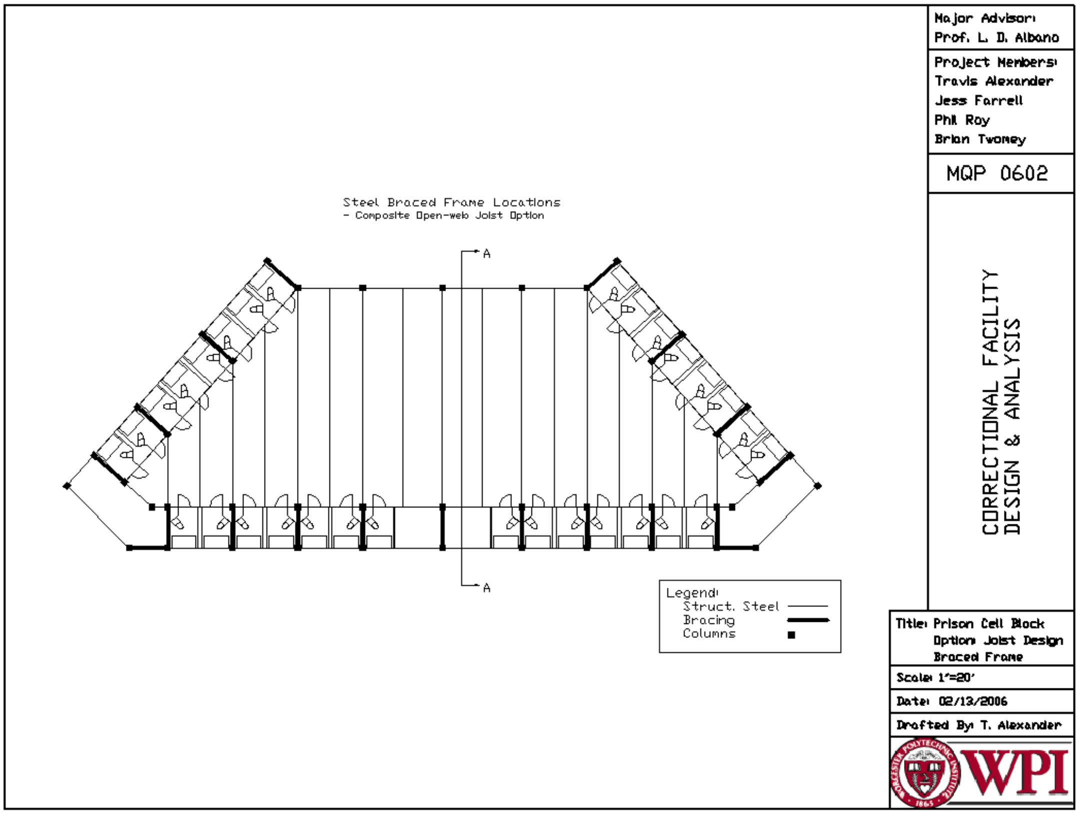

Figure 4.2.3 Prison Cellblock Braced Frame Location Considerations

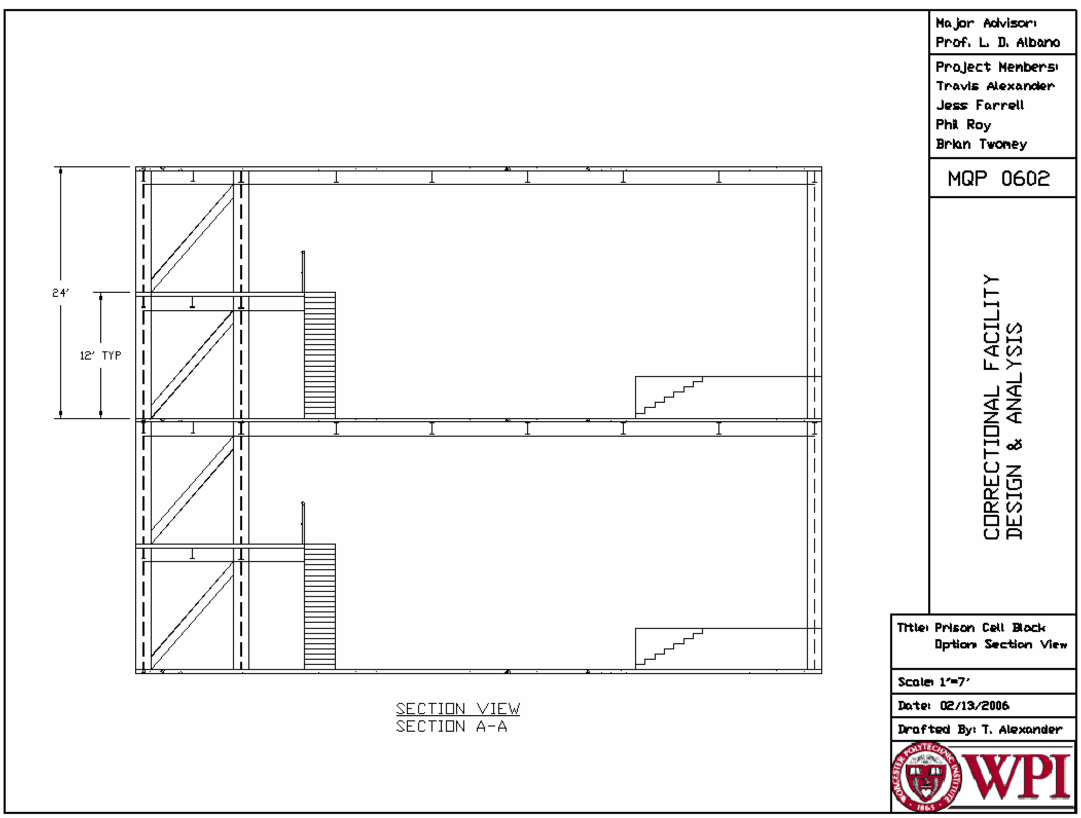

Figure 4.2.4 Prison Cellblock Section View

In any of the three seismic load directions, a component of the load is being directed into a secondary frame. When seismic activity is applied perpendicular to the long face, the equivalent of 13 frames is available. This equated to 101K (7.77K/frame * 13 frames) of seismic base shear, which was greater than the required base shear of 59.5K. These 13 frames are a combination of the eight frames parallel to the direction of wind in seismic load direction 1 plus a sine function of the eight frames oriented diagonally (see Figure 4.2.3).

When the seismic load is applied perpendicular to the diagonal face (see Figure 4.2.2), nine frames are available to resist the seismic load; four frames parallel to the direction of the load plus a sine function of eight frames along the long face (see Figure 4.2.3). This equates to 71K of seismic base shear resistance. The final seismic load consideration was parallel to the long face (see Figure 4.2.2). In this orientation, the component of eight frames was available resulting in 41.6K of base shear resistance. This capacity was insufficient for the total seismic base shear of 59.5K. To increase the base shear resistance, braced frames were placed at the ends of the long face (see Figure 4.2.3), parallel to seismic load direction 3 as located in Figure 4.2.2. By placing braced frames at these locations, the effects of cantilever action were alleviated.

In the prison cellblock structure, shear studs were required along the girders on the floor and roof levels. These shear studs were used to create a rigid diaphragm to connect the numerous braced frames. Shear studs were spaced along the girders based on the maximum allowable spacing. Without this rigid diaphragm, there would be a disconnect between the braced frames surrounding the cellblocks. The rigid diaphragm would transfer components of the seismic loads to secondary braced frames within the structure. As a result of lateral design, the cost of the structural steel system for each of the prison cellblocks increased by $35,000, from $424,000 to $459,000.

Prison Core

While the prison core is exposed to relatively little wind loads because of the placement of the cellblocks, it is potentially susceptible to substantial seismic loads. Based on wind analysis, rigid diaphragms were chosen to transfer lateral loads to lateral resisting frames. Because of the open space required for the recreational area, no impeding columns or braces could be located in this area. For this reason the rigid diaphragm was utilized to transfer the lateral loads to the appropriate frames. See Figure 4.2.5 for brace locations based on wind analysis.

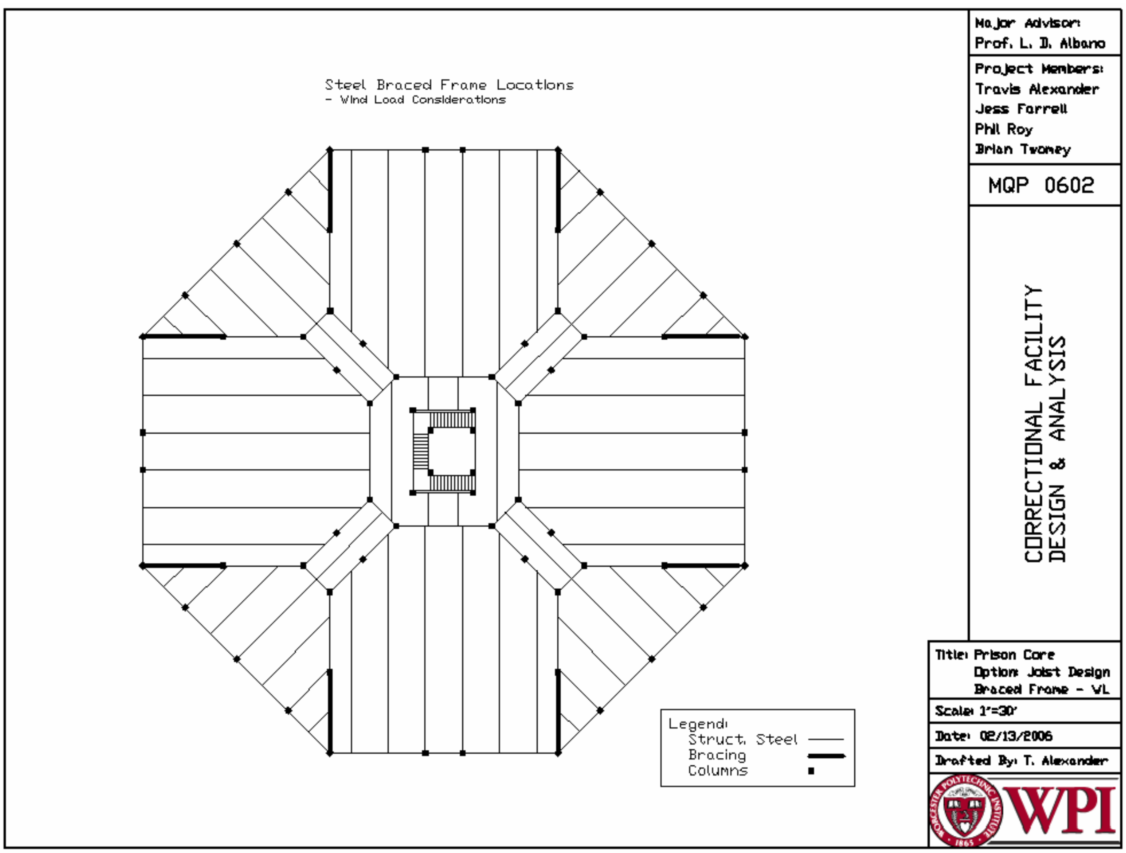

Figure 4.2.5 Prison Core Braced Frame Locations - Wind Considerations

Because of considerations discussed in section 3.2.2 Core Layout, floor to ceiling height of 24 feet is desirable for a typical basketball court. With this consideration, a twenty-four foot floor height was chosen to be synonymous with the day room height. Because of the considerations for brace geometry, girders were placed at mid-height to allow approximately a forty-five degree orientation for the brace. As a result of this additional girder, the unbraced length of the brace was reduced, minimizing the effects of lateral torsional buckling.

Since seismic loads are a function of the total building weight, the effects can occur in any direction throughout the structure. To adequately resist seismic loads, additional frames were placed. Figure 4.2.6 illustrates the location of these additional braces. The additional costs for these lateral frames are tabulated in Appendix B.8.1 as part of the total cost breakdown. As a result of lateral design, the cost of the structural steel system for the prison core increased by $112,000, from $1,145,000 to $1,257,000.

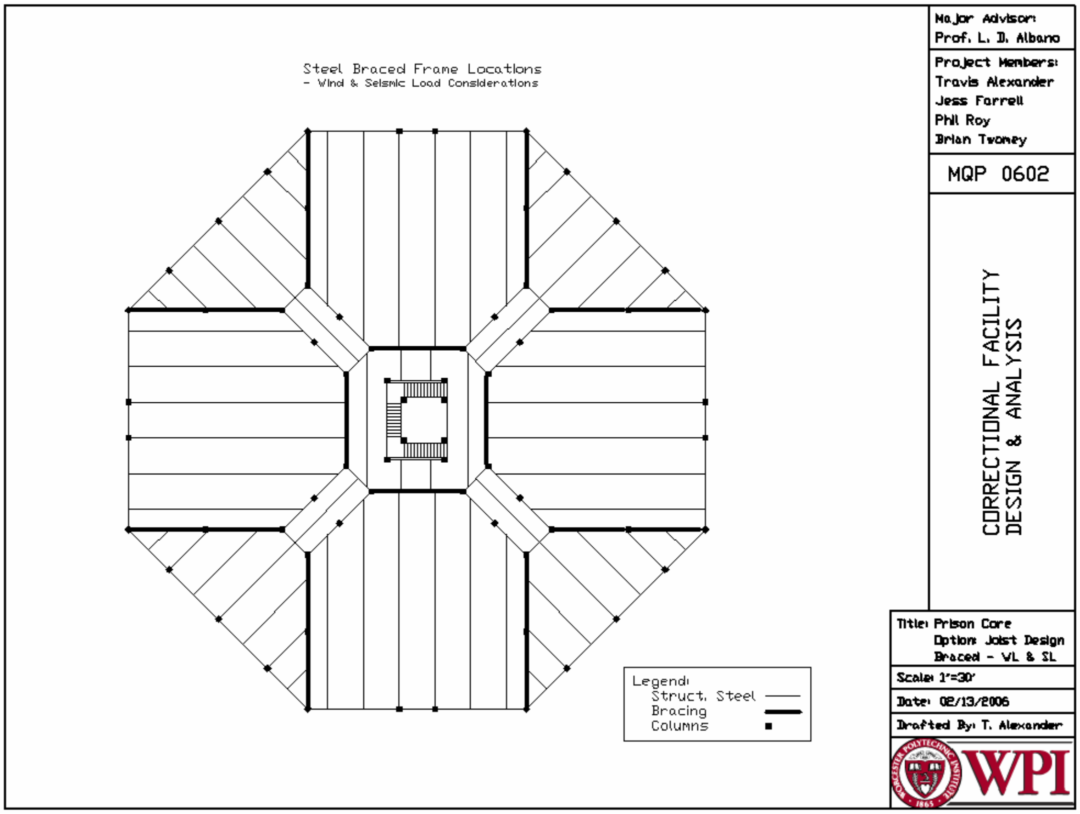

Figure 4.2.6 Prison Core Braced Frame Locations - Wind & Seismic Considerations

5 Foundation Design

Connections are one of the most important aspects of building construction. It has been said that "A structure is no stronger than its connections."[331] Foundations can be viewed as a connection between the ground and the structure, therefore increasing the importance of the design of foundations. Trial-and-error design of foundations was primarily used before the nineteenth century at which point foundation engineering was finally developed.[332] The trial-and-error design was based on previous successes and failures, which were implemented by empirical formulas. However, when new soil conditions were encountered only an educated guess could be made according to the strength and performance of the foundation. Foundation engineers must be familiar with the structural loads applied to the foundation, the geotechnical interaction between the foundation and the soil, and finally the construction of the foundation. The most difficult of these areas is the geotechnical aspect of the design because the strength of the soil is variable and is not clearly defined.

Soil tests are conducted in order to understand and determine the bearing capacity of the soil. The soil is tested using borings and test pits. A standard boring rig "…consists of a tripod or frame with a pulley and a small winch. A hammer is raised by the winch and allowed to fall free, driving a pipe casing into the ground." [333] The number of blows required to drive the pipe one foot into the ground is used as a standard measurement of soil compactness. Test pits are holes dug in the ground to evaluate the upper layers of soil by inspection. Test pits may be used for more immediate results but

[331] Coduto, Donald P., p.3
[332] Coduto, Donald P., p.4
[333] Watson, Donald, p.A1.1-2

are limited in depth to about ten feet. Borings are typically done fifteen to twenty feet below foundation level, and they contribute to much more reliable soil test results.

There are two major categories of foundations that are commonly used: shallow and deep. Shallow foundations are often referred to as footings and are "… those that transmit structural loads to the near-surface soils."[334] Shallow footings require solid soil conditions for adequate load transfer from the structure to the ground with limited settling over time. Shallow foundations typically support much smaller loads than deep foundations. Deep foundations are "… those that transmits some or all of the applied load to soils well below the ground surface."[335] Typically, deep foundations extend about 50 feet below the ground surface and can extend up to 150 feet.[336]

5.1 Shallow Foundations

Spread footings are the most common type of shallow foundation, primarily due to their constructability and low cost. Most footings are made of reinforced concrete and are often constructed for each column and bearing wall. These footings distribute the load of the columns or bearing walls into the soil, working in unison with the bearing capacity of the soil to support the structure. As a general rule, if spread footings "…cover more than fifty percent of the building footprint area, a mat or some type of deep foundation will usually be more economical."[337] If this general rule does not apply, then shallow foundations will most likely be the most cost effective foundation for the given structure.

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[334] Coduto, Donald P., p.145
[335] Coduto, Donald P., p.374
[336] Watson, Donald, p.A1.1-2
[337] Coduto, Donald P., p.153

There are several different types of shallow foundations commonly used. These shallow foundations include square spread footings, rectangular spread footings, circular spread footings, continuous spread footings, combined footings, and ring spread footings. The uses of each of these footings are outlined in Table 5.1.1.

Table 5.1.1 Types of Footings and their Uses

Footing Type	Typical Uses
Square Spread Footing	Centrally located columns
Rectangular Spread Footing	Obstructions present & Large moments present
Circular Spread Footing	Light standards, Flag poles & Power lines
Continuous Spread Footing	Bearing walls
Combined Footing	Support for more than one column (many columns in one area)
Ring Spread Footing	Similar to continuous footing but wrapped in a circle & Circular storage tanks

Additionally, a drawing of each footing in Table 5.1.1 is shown below. The drawings were taken from the second edition of Foundation Design: Principles and Practices written by Donald P. Coduto. [338]

Abbreviated Terms:
T – Thickness
D – Depth (from bottom of footing to ground surface)
B – Base width
L – Base Length

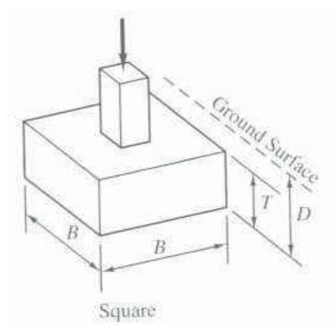

Figure 5.1.1 Square Spread Footing[339]

[338] Coduto, Donald P., p.146
[339] Coduto, Donald P., p.146

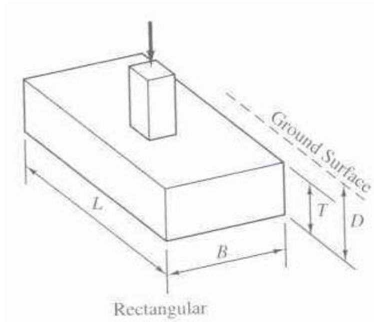

Figure 5.1.2 Rectangular Spread Footing[340]

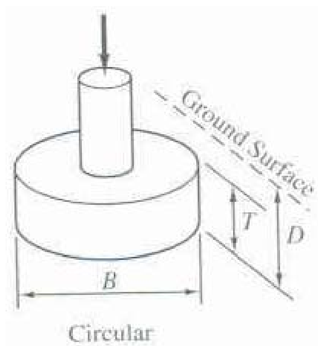

Figure 5.1.3 Circular Spread Footing[341]

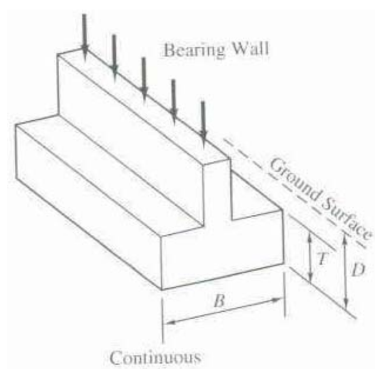

Figure 5.1.4 Continuous Spread Footing[342]

[340] Coduto, Donald P., p.146
[341] Coduto, Donald P., p.146
[342] Coduto, Donald P., p.146

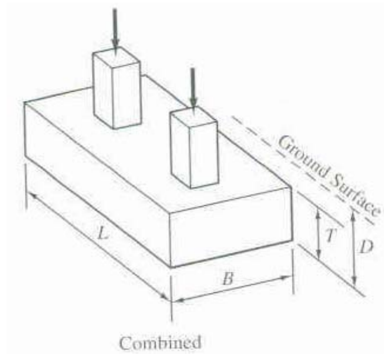

Figure 5.1.5 Combined Footing[343]

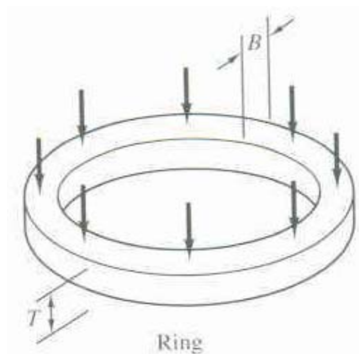

Figure 5.1.6 Ring Spread Footing[344]

Each of the footing designs featured in Figure 5.1.1 through Figure 5.1.6 gather applied loads from columns or bearing walls and distribute the load into the soil over a larger area. The reason for the different shapes and configurations is to allow for flexibility to adapt to the given design and layout. For example, a square footing may not be used because an existing building nearby will obstruct the footing. As an alternative option, a rectangular footing may be used to reduce the width of the footing but still allow for adequate support.

5.2 Deep Foundations

Deep foundations are designed to support loads when unstable or inadequate soil conditions exist.[345] Essentially, deep foundations are constructed to penetrate through the

[343] Coduto, Donald P., p.146
[344] Coduto, Donald P., p.146

softer compressible soils, anchoring into the solid soil below the unstable soil. Figure 5.2.1 shows how a typical deep foundation extends through the soft compressible soil and anchors into the hard incompressible soil.

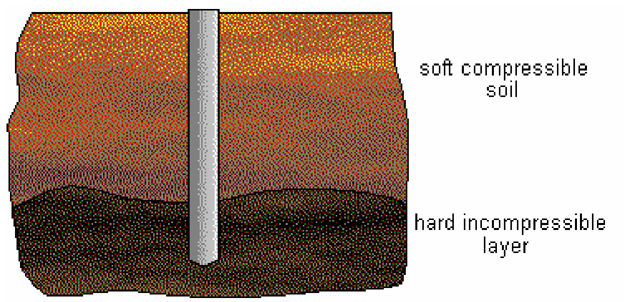

Figure 5.2.1 Deep Foundation Extending into Hard Incompressible Soils[346]

Spread footings are preferred over deep foundations because they are simple and inexpensive. However, there are many situations where spread footings are not adequate and deep foundations or a mat foundation must be utilized. Some of these situations outlined in the second edition of Foundation Design: Principles and Practices by Donald P. Coduto include[347]:

- The upper soils are so weak and/or the structural loads so high that spread footings would be too large. A good rule-of-thumb for buildings is that spread footings cease to be economical when the total plan area of the footings exceeds about one-third of the building footprint area.
- The upper soils are subject to scour or undermining. This would be especially important with foundations for bridges.
- The foundation must penetrate through water, such as those for a pier.
- A large uplift capacity is required.
- There will be a future excavation adjacent to the foundation, and this excavation would undermine shallow foundations.

There are many different types of deep foundations**.** Unfortunately, different names are commonly used to define similar designs of these foundations. This creates confusion as to what foundation design is actually desired and often leads to

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345 Coduto, Donald P., p.374
346 http://fbe.uwe.ac.uk/public/geocal/foundations/Fountype.htm#PILES
347 Coduto, Donald P., p.374

misunderstandings during construction. The names used below to describe deep foundations were taken from Donald P. Coduto's text.[348] Types of deep foundations include:

- **Piles**: constructed by prefabricating slender prefabricated members and driving or otherwise forcing them into the ground
- **Drilled Shafts**: are constructed by drilling a slender cylindrical hole into the ground, inserting reinforcing steel, and filling it with concrete.
- **Caissons**: prefabricated boxes or cylinders that are sunk into the ground to some desired depth and then filled with concrete. Some engineers use the term "caisson" to describe drilled shafts, so this is one of the more confusing terms in foundation engineering.
- **Mandrel-driven thin shells filled with concrete**: thin corrugated steel shells that are driven into the ground using a mandrel, then filled with concrete.
- **Auger-cast piles**: constructed by drilling a slender cylindrical hole into the ground using a hollow-stem auger, then pumping grout through the auger while it is slowly retracted.
- **Pressure-injected footings**: cast in place concrete that is rammed into the soil using a drop hammer.
- **Anchors**: include several different kinds of deep foundations that are specifically designed to resist uplift loads.

Piles are among the most commonly used deep foundations. They can be made from wood, steel and concrete, or a composite of any of these materials. There are approximately nine different varieties of piles that are used for deep foundations. These types of piles can be seen in Figure 5.2.2.

[348] Coduto, Donald P., p.374-375

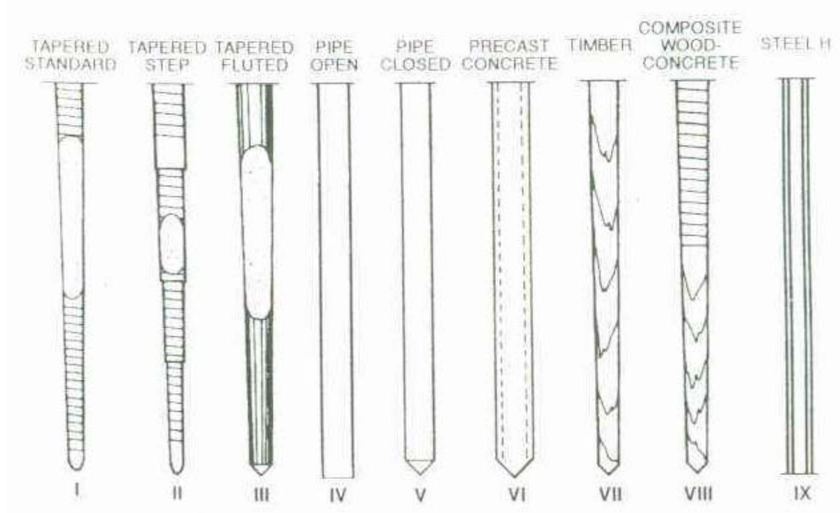

Figure 5.2.2 Types of Piles[349]

A description of the piles featured in Figure 5.2.2 can be found in Table 5.2.1.

Table 5.2.1 Description of Types of Piles[350]

Type I	Cast-in-place concrete pile. A light-gage steel shell, driven on a mandrel, which is then withdrawn, is inspected and filled with concrete. Care must be taken to avoid collapsing of the shell when an adjacent pile is driven.
Type II	Cast-in-place concrete pile. A light-gage steel shell, driven on a mandrel, which is then withdrawn, is inspected and filled with concrete. Care must be taken to avoid collapsing of the shell when an adjacent pile is driven.
Type III	Similar to Type I & II except that the shell gage is heavier and no mandrel is required
Type IV	Type IV is an open-end steel pipe. It is excavated, often by air jet, as it is advanced, and then filled with concrete after refusal has been reached. In lieu of reaching refusal, driving may stop while a concrete plug is
Type V	Closed end Pile. After driving, it is filled with concrete. Often it is used inside buildings with low headroom. Shorter lengths are simply spliced with steel collars.
Type VI	Type VI is a pre-cast concrete pile. It is good in marine structures but requires heavy handling equipment and accurate estimation of tip elevation as it is difficult to cut off in the field.
Type VII	Type VII is a wood pile which is the least expensive pile. Wood piles are typically used in marine situations where the pile is partially exposed permanently above water level. The timber must be treated with a wood
Type VIII	Composite wood and concrete piles are seldom used. The timber is kept below ground water and a greater over-all length is achieved. A closed-end pipe pile may be used in place of the timber section.
Type IX	Rolled steel H section. It is the cheapest of the higher capacity piles. Protection must be provided when driving through cinder fill or other rust-producing material.

Deep foundations require much more effort to construct than spread footings. They require some sort of drilling rig or driving rig to dig or set the foundation. These drilling and driving rigs are expensive and are often in need of repair, which can cause delays during construction.

[349] Watson, Donald, p.A1.1-3
[350] Watson, Donald, p.A1.1-3

5.3 Design Application

Foundation design is very difficult and complex because the bearing capacity of the soil is highly variable. This section outlines the basic steps needed to design a shallow spread footing for a typical column in the correctional facility of this MQP. The first step to designing a foundation is to understand the geotechnical conditions and ultimately the bearing capacity of the soil. Boring tests could not be conducted for this project, therefore results from these tests that define the soil composition of the chosen site do not exist. An assumed bearing capacity based upon an educated guess has been taken from the *2003 International Building Code* (IBC). The IBC states that for sandy gravel and/or gravel, the soil design bearing capacity is approximately 3000 pounds per square foot.[351]

One of the larger reactions from of an interior column within the correctional facility is approximately 600 kips. The minimum depth of a rectangular spread footing is a function the applied axial load, because the footing must be capable of resisting the shear forces applied by the column it supports. The minimum depth of a footing with an applied load of 600 kips is three feet.[352] Applying this load of 600 kips to the soil which was defined to have a bearing pressure of one and a half tons per square foot will result in a footing area of 200 square feet (14.15ft x 14.15ft). The actual footing constructed for this scenario would most likely be rounded up to 15ft x 15ft for optimum constructability. Additionally, in order to resist bending, reinforcing bars must be added. This scenario requires ten number seven bars, spaced 18 inches apart, in both directions of the footing to resist bending. The total cost for materials and labor to place this particular spread

[351] *2003 International Building Code*, p. 363
[352] Coduto, Donald P., p.324

footing is about $6,175. Refer to Appendix H Foundation Design for the hand calculations accompanied with the design of this spread footing.

The IBC states that for foundations exposed to frost conditions the foundation must be placed below the frost line of the locality. The frost line in Western Massachusetts is about four feet below grade.[353] Rather than designing a solid concrete block 200 square feet with a four foot depth, a pedestal can be used to reduce the amount of concrete needed, ultimately reducing the cost. Figure 5.3.1 shows an elevation view of a spread footing with a pedestal to reduce the concrete needed to transfer the load from the column to the footing.

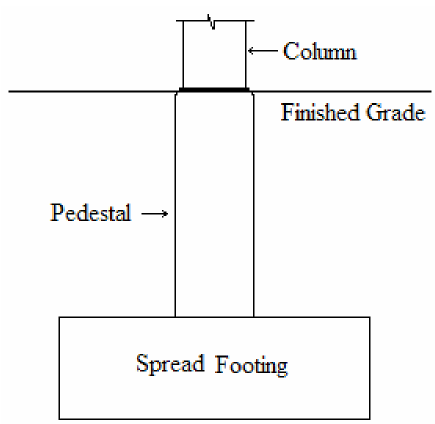

Figure 5.3.1 Elevation of Typical Footing with Pedestal

A 200 square foot footing is quite large and may constitute the use of a deep foundation. As stated previously, if the total area of the footings is about half of the total area of the building then deep foundations or a mat foundation will be more cost effective. In order to determine if deep foundations will be necessary a full design of the foundation system would have to be conducted.

[353] *2003 International Building Code*, p.364

6 Exterior Wall Design

Walls are an integral part of any building structure. Proper design and detailing of assemblies are essential for wall structural stability, occupant safety, and performance. Although walls are may or may not be part of the primary structural system, applicable code sections, load capacities, seismic performance, and design life must all be met to satisfy design requirements. This chapter will examine the functions of a wall assembly, the components necessary to satisfy the performance and functional requirements, and design considerations and implications of wall assemblies on the structural framework.

6.1 Basic Wall Functions

A building's envelope is a critical component of any structure, constituting the primary defenses against environmental infiltration and other sources of discomfort into the building's interior, while enhancing its visual appeal. "The envelope must balance requirements for ventilation and daylight while providing thermal and moisture protection appropriate to the climate conditions of the site."[354] The envelope is the building's skin, comprised of its roof, walls, windows, doors, and other materials that offer a barrier against intrusion. Exterior walls are an essential component of a building's envelope, providing protection and aesthetics to the vertical faces. According to Michael J. Crosbie, an architect and author of *Time-Saver Standards for Architectural Design Data*, wall performance criteria can be summarized into three categories: aesthetics, structural and safety, and environmental control.[355] Design considerations for exterior walls to meet the performance categories include stopping exterior environmental ingress,

[354] U.S. Department of Energy, www.eere.energy.gov/
[355] Crosbie, Michael J.

minimizing unwanted heat transfer between the interior and exterior portions, remaining stable in the presence of gravity loads while accommodating lateral movement, and enhancing the structure's aesthetics.[356]

6.1.3 Aesthetics

Walls help to define the visual appeal of a structure from the outside. Therefore, choosing the appropriate materials, textures, and colors are critical. Proper material selection is also important for life-cycle costs, maintenance, and repair. Walls must be able to withstand applicable loads and deflection without cracking. While cracks may not have an implication on the structural stability of the assembly, their occurrence can be assumed as a failure because of the visual degradation.[357]

Aesthetics provide a sense of form, scale, proportion, and function. A building that is aesthetically pleasing can provide a sense of place, belonging, and ownership, both to the building and to the community at large. A facility such as a correctional institution, which is often faced by sharp criticism and protest from local citizens, can be well accepted by its visual appeal. The reader is referred to the Lexington/Fayette County Detention Facility in Lexington, KY shown in Figure 2.6.3. Here, the materials, colors, textures, and design of the wall assembly were chosen to replicate a horse farm, thereby blending into the countryside in which the facility is located.

6.1.4 Structural and Safety

Wall assemblies can be designed as structural or non-structural components. Walls considered as structural components, or load bearing, transfer service loads

[356] Brock, Linda, p.14
[357] KPFF Consulting Engineers, www.masonconf.com/miw/struct_veneer

resulting from reactions of the roof and floor systems to the footings. While non-structural wall assemblies are not designed to support service loads, they are responsible for protecting the structural members that do. Walls must provide protection from harsh environmental conditions; the intrusion of airborne pollutants, chemicals, and acid rain can react with the member material, degrading their strength.

Occupant safety is also credited to exterior walls. Life safety concerns impose the need to limit the spread of a fire within a structure. Should a fire ignite in a structure, the spread of the fire within the building and to adjacent facilities can result in loss of life and property damage. Therefore, local, national, and model building codes prescribe minimum fire ratings for exterior wall assemblies. The basis of the fire ratings is to ensure adequate structural stability and protection, providing time for occupant egress, fire fighter ingress, and to prevent structural collapse. Factors such as the proximity to nearby buildings, intended use, and the overall building size affect the required fire resistance ratings.

As part of a building's passive fire protection system, exterior walls play a critical role in managing the spread of fire. Connections between walls and structural framework must be properly detailed to prevent the updraft of fire.[358] If a fire migrates into a wall cavity, the tight airspace acts like a chimney, pulling the flame and smoke upwards. Therefore, proper consideration and detailing is required for floor to exterior wall connections. Figure 6.1.1 shows a typical connection used to prevent the updraft of fire within a wall assembly. A fire stopping material is required to fill the void between slab and the exterior walls. Typical fire stopping material includes mineral and rock fiber seals, noncombustible foams, and intumescent coatings that expand when exposed to heat.

[358] Nashed, Fred, p.37

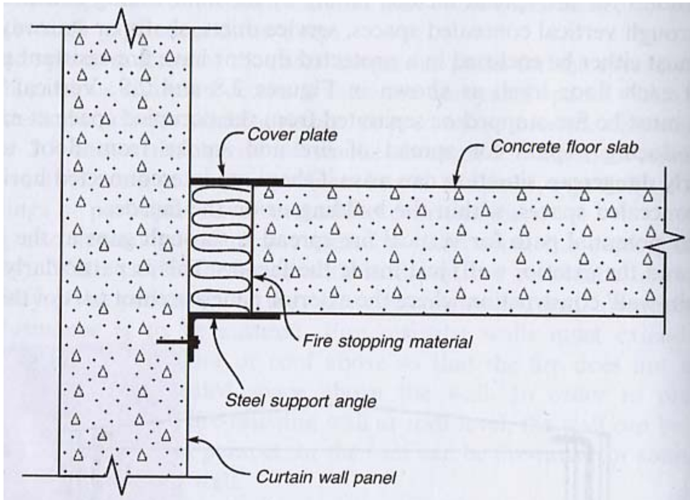

Figure 6.1.1 Typical Wall-slab Fire Resistant Connection[359]

Improper detailing of the slab-to-wall connection can result in the failure to provide proper fire protection. The consequence is that a fire spread vertically through a structure, leading to excessive property damage and life safety hazards. Figure 6.1.2 (a) shows the effects of improper connection detailing.

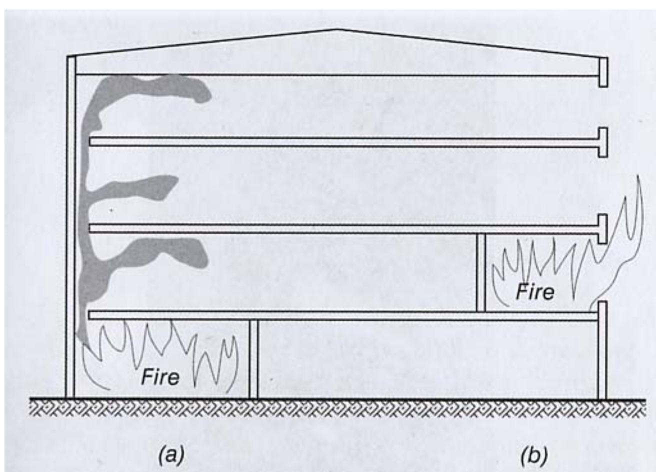

Figure 6.1.2 Fire Spread[360]

Fire spread from the outside is also a concern in wall design. Effective wall barriers should be constructed of noncombustible materials. Materials such as concrete

[359] Buchanan, Andrew, p.24
[360] Buchanan, Andrew, p.25

and masonry do not burn or ignite like wood or plastics do, thereby providing longer resistance to fire spread.

6.1.5 Environmental Control

The principal function of exterior wall assemblies is to contribute to the environmental control within the facility. Exterior walls are the primary defense against minimizing environmental infiltration into the controlled environment of a building's interior. They provide a barrier against weather conditions such as sleet, snow, rain, and wind, sound intrusion, component corrosion, and thermal efficiency. Table 6.1.1 provides a summary for the various environmental conditions that exterior walls must protect against and their consequences.

Table 6.1.1 Environmental Conditions Imposed on Exterior Walls

Environmental Condition	Consequences
Weather Conditions • Moisture (sleet, snow, and rain) • Wind	 Freeze-thaw damage, staining, damage to finishes Air infiltration and exfiltration, water infiltration
Sound Intrusion	Occupant discomfort
Electrolysis	Component corrosion and failure
Thermal Efficiency	Higher operating costs to maintain desired environment

Moisture that migrates into a wall assembly can cause damage to interior finishes as well as freeze-thaw damage within the structure.[361] Figure 6.1.3 illustrates various ways for water infiltration to occur. Proper design and detailing is required to resist all types of infiltration. This includes proper orientation and channeling of joints, proper installation and location of barriers, and proper material selection and/or coatings for porous materials.

[361] Nashed, Fred, p.15

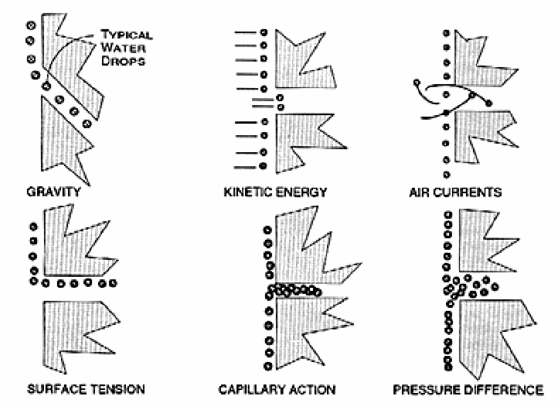

Figure 6.1.3 Water Infiltration Action[362]

Wind can force water to migrate through any cracks or discontinuities in a wall assembly. The effects of wind can also cause a pressure differential between the exterior and interior portions of a wall, thereby "pulling" the water inward. Water can infiltrate into a wall assembly through the forces of gravity as well as the inherent surface tension of water, allowing water to defy gravitational forces and cling to surfaces. The surface tension and a water molecule's attraction for free oxygen molecules along a surface allow capillary action to occur in which the water can "climb" through an opening, thus impeding on the interior portions.

Air infiltration and exfiltration of the interior portions is also of concern. The drifting of outside, unconditioned air into the controlled environment can cause occupant discomfort. Air infiltration does not only result in occupant discomfort, it is also a catalyst for water infiltration. The forces of wind exerted on a building's face have the ability to force moisture migration into a wall assembly (see Figure 6.1.3, *Air Currents*). Air seeks inward access at points of discontinuity of the facade. Therefore, proper detailing around openings, joints, and sealants is required to minimize the adverse affects of air infiltration. Also, if not properly detailed, the conditioned air within the controlled

[362] KPFF Consulting Engineers. www.masonryinstitute.com/veneer/

environment may escape through exterior wall assemblies, resulting in elevated operating costs to maintain the desired interior environment.

Sound absorption can also be a concern for minimizing interior discomforts. Sound entering as ambient noise and through structural vibration can contribute to occupant discomfort.[363] The prevention of electrolysis is also a function of the environmental control of wall assemblies. Electrolysis is the corrosion of a metal in the presence of an acidic solution or other materials.[364] If connectors and anchors or certain metals come in contact with airborne pollutants, it can lead to corrosion, degradation, and eventual failure.

The type and amount of protection provided is dependent on the wall material. Material characteristics and component affect the environmental control. For example, concrete facades are more porous than those of brick or metal. Therefore, concrete facades are susceptible to greater moisture intrusion into the wall assembly and possibly decreased durability. A material's R-value, which is its resistance to heat flow, affects the amount of heat transfer into and out of the interior. A material with a higher R-value would provide greater resistance to unwanted heat differentials. Wall assembly components are essential to minimize air and water infiltration. The components required are mandated by the type of materials used. For example, weep holes are required for masonry construction to allow drainage of infiltrative water but are not required for metal cladding or concrete panels. Further discussion on the types of components and their performance are discussed in section 6.2 Wall Components.

[363] Nashed, Fred. p.36
[364] Nashed, Fred, p.36

The type of wall construction also has a role in the environmental protection provided. Single-wythe walls provide one barrier against environmental infiltration. Double-wythe assemblies provide more protection. In the event that air or water penetrates the first wythe, a second line of defense is provided with the second wythe.[365]

Considerations for load bearing and non-load bearing walls can also affect environmental protection. Because of the loads that they are required to resist, load bearing walls are often of considerable thickness as compared to non-load bearing walls. The thickness and resulting mass have also proven adequate in resisting environmental infiltration.[366] Thus, the effects of temperature gradients between the daytime and nighttime are often negligible in the interior. Water and air infiltration are also hampered with load bearing walls. Because of the thickness, migrated moisture is often released back into the environment before it seeps into the interior.

6.2 Wall Components

There are many components within a wall system that must be properly detailed to maximize a wall's performance. Although not all of the components offer structural support, they are nonetheless important to maintain a continuous barrier against infiltration. Such components are sealants, sealers, dampproofing, waterproofing, insulation, and vapor barriers. Table 6.2.1 provides a list of typical components in a wall assembly as well as their intended function. Figure 6.2.1 shows the locations of typical components in a metal stud backer wall assembly.

[365] Nashed, Fred, p.81
[366] Nashed, Fred, p.80

Table 6.2.1 Wall Components and Function[367]

Component	Function
Air Barrier	Sheating used to resist air passing through a wall
Anchor	Typically metal connectors used to connect wall assemblies to the structural framework
Dampproofing	House wrap is used to minimize moisture infiltration
Flashing	Impermeable material placed around openings to redirect any infiltrative water out from inner surfaces
Gypsum	Interior finishes which can act as an air barrier as well as a barrier against fire spread
Insulation	Material placed within a wall assembly to minimize heat transfer into and out of interior portions
Joints	Pliable material placed between panels allowing for deflection and thermal expansion
Lintel	Structural member placed above openings to transfer the wall weight to solid portions
Panel	Wall panel which spans between supports; may or may not carry any applied loads
Reinforcement	Used in concrete and masonry construction to provide tensile strength to the assembly
Sealant	Seal gaps between panels; highly cohesive and adhesive, allowing for deflection
Sealer	Applied to the exterior portions of some walls to decrease permeability
Shelf Angle	Provide support to the wall assembly at intermediate floor levels
Soft Seal	Placed on the underside of shelf angles to provide a seal as well as thermal expansion for the assembly
Vapor Barrier	Membrane used to stop the intrusion of condensation within a wall assembly to interior surfaces
Weep Hole	Used in cavity walls; voids in mortar to allow the collected water to pass

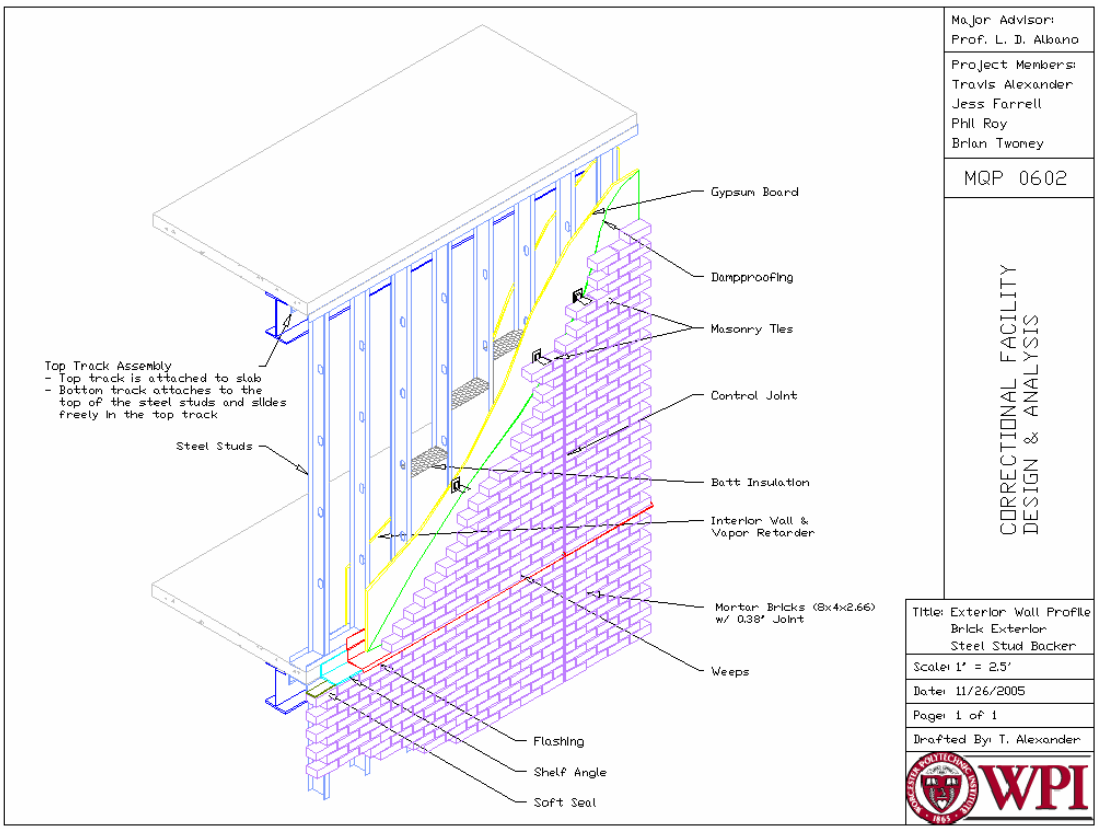

Figure 6.2.1 Brick Veneer with Metal Stud Wall Assembly

[367] Nashed, Fred

6.3 Structural Loads and Wall Types

The type of wall, load bearing or non-load bearing, defines the structural loads imposed on the wall. Load bearing walls carry portions of the service, or gravity loads. These walls are typically made of concrete or reinforced masonry, requiring considerable thickness.

The benefit of load bearing walls is that the loads are transferred to the exterior walls; therefore, fewer structural members are required. Load bearing walls not only provide a barrier against infiltration but also respond as columns, allowing the infill beams and floor slab to tie into them, and provide lateral stability. Also as previously stated, load bearing walls perform exceptionally well at minimizing environmental infiltration, sound infiltration, and fire-related safety because of their shear size. The limitations in the application of load bearing walls reside in the number, size, and placement of openings for windows and doors. Considerations for openings revolve around the necessity to provide portions of solid, uninterrupted wall areas to transfer the loads to the foundation or footings.

The use of load bearing walls in large scale buildings has decreased. This is because of concerns for constructability, economics, and size.[368] Desires to maximize floor space are a main reason for shifting away from load bearing walls. For taller structures, lower portions of load bearing walls would have to be of considerable thickness to transfer the loads while maintaining structural stability, thereby lessening the valuable net floor space for occupancy. Furthermore, the design for more natural light in

[368] Nashed, Fred, p.80

buildings has led to an increase in the use of non-load bearing walls as opposed to load bearing.[369]

Non-load bearing walls allow for many openings to be 'punched' into their surface. They are often classified as 'nonstructural components,'[370] meaning they are not responsible for transferring any gravity loads beside their own self-weight, resulting in lighter wall assemblies. The use of this type of wall system is made possible by the use of the frame building.[371]

Most non-load bearing walls are composed of an exterior veneer backed by a metal stud or concrete masonry unit backer wall. The material variations available for non-load bearing wall construction are abundant. Applications ranging from precast and tilt-up concrete panels, brick and stone veneer, glass panels, and metal panels are often used on the facade.

There is also a variety of non-load bearing wall construction. These include curtain walls, infill walls, and cavity walls. Figure 6.3.1 shows a section view of all three. Curtain walls span the outside of the structural frame work, while being attached to the floor and roof slabs. Infill walls span between girders or slabs and are directly supported by the structural elements. Cavity walls are comprised of an outer wythe, typically veneer that is anchored to a backer wall, which is directly supported by the structural elements.

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[369] Weber, Blaine J., http://www.djc.com/news/ae/11151058.html
[370] Memari, Ali M., www.mmtmagazine.org/page/?id=181
[371] Brocks, Linda, p.89

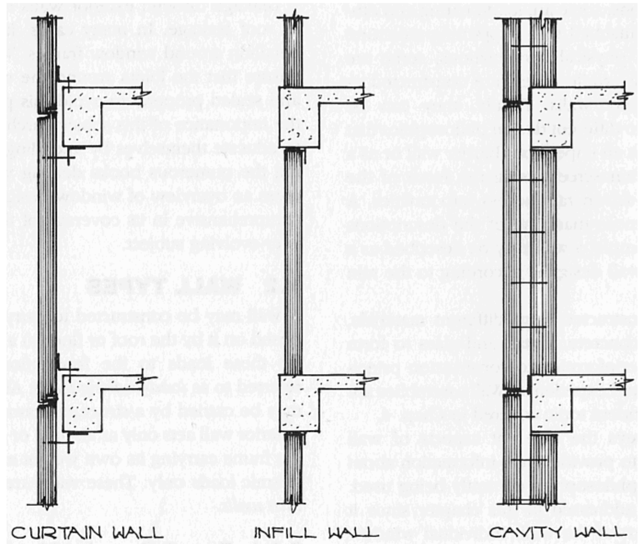

Figure 6.3.1 Non-load Bearing Wall Types[372]

The benefit of non-load bearing walls is the savings in weight. The use of lighter materials eases construction, manpower, and the requirement for additional equipment. Compared to exterior wall systems, non-load bearing walls have an economic advantage in terms of the net space available for a given building footprint. In some instances, this advantage may be trade-off with the increased complexity and components required in design and construction. Also, larger structural members may be required to support such a wall system.

Regardless of the type of wall construction, both load bearing and non-load bearing assemblies must be designed to sustain wind and seismic loads. Wall assemblies act as a vertical diaphragm, transferring the horizontally applied wind pressures to its horizontal and vertical supports. Wall components normal to the wind direction (windward face) must be designed to withstand the positive forces exerted onto it, while the leeward wall face must sufficiently endure the resulting negative (suction) forces.[373]

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[372] Nashed, Fred, p.80
[373] IBC 2003

The effects of the lateral loads on a structure are deflection cause by wind and lateral drift caused by seismic activity. Concerns for wall assemblies revolve around minimizing cracks, material spall, and maintaining stability in the presence of these lateral loads. The formation of cracks or material expulsion can have dire consequences for the wall's ability to provide protection from the environment.

Consideration must be given to out-of-plane bending resulting from wind loads. Wall assemblies can adequately transfer lateral loads to adjacent structural members through bending.[374] When a wall bends, it is absorbing the energy from the horizontal loads. This puts one face in compression and the opposite face in tension. Most materials, such as metals, can adequately handle both compressive and tensile forces. When concrete or masonry walls are designed, reinforcement is required because concrete and masonry have exceptional compressive strength, but are weak in tension (tensile strength is only about ten percent of the compressive strength).

In-plane bending is the result of seismic loads. Unlike wind loads, which are only applied to the face normal to the wind direction, seismic loads are applied to the entire structure. Therefore, each floor level in a multistory building moves horizontally in relation to one another. The relative stiffness of wall assemblies allows them to withstand the deformations exerted onto them since they span between floors.[375] As a result, excessive shear stresses develop between wall panels. Proper detailing of joints is required to ensure that both the sealants are not susceptible to being overstressed and that the joints are properly sized to accommodate any panel displacement without damage.[376]

[374] KPFF Consulting Engineers, www.masonconf.com/miw/struct_veneer
[375] Ting, Raymond, www.tingwall.com/articles/article_05.html
[376] Ting, Raymond, www.tingwall.com/articles/article_05.html

6.4 Wall Design Considerations for Correctional Facilities

Correctional facilities are a special type of infrastructure onto themselves. Such facilities require additional security measures to provide safety to the neighboring community and to the inmates contained inside. Security requirements for wall assemblies are to provide full security both into and out of the facility.[377] Walls must be detailed accordingly to not allow a breech of the exterior wythe by intruders or escaping inmates.

The American Society of Testing and Materials (ASTM) International has developed standards for walls in correctional facilities. "It is the intent of these test methods to help ensure that detention security walls perform at or above minimum acceptable levels to control passage of unauthorized or secure areas, to confine inmates, to delay and frustrate escape attempts, and to resist vandalism."[378] The tests were developed to help facility administrators develop standardized security levels for security walls to withstand attacks and damage resulting from manpower forces and common handguns. Standard tests include resistance to bullet penetration and impact testing simulating a battering ram.[379]

Precast concrete, concrete masonry units, and steel panels have a proven track record for providing structural stability and safety requirements.[380] For the design of the correctional facility in question, detail considerations will be given to precast concrete, or tilt-up, and concrete masonry units. With regards to precast concrete, typical design

[377] Krasnow, Peter, p.183
[378] ASTM, p.1435
[379] ASTM, p.1436
[380] Krasnow, Peter, p.183

values include 4,000psi compressive strength concrete with reinforcement and a minimum wall thickness of four inches.[381]

Consideration must be given for the ability of the precast concrete panels to withstand substantial amounts of deflection.[382] The use of large panels is more advantageous as they are inherently more flexible and allow for larger deflections. Smaller panel sizes have an increased stiffness, thereby reducing the amount of deflection that they can endure.

Precast concrete panels have the ability to enhance the blast-resistance of a building if properly designed and constructed. The use of pilasters as part as the backer and additional reinforcement allow the walls to adequately transfer additional loads and sudden shock.[383] Moreover, the overall weight and stiffness of precast walls allow the assemblies to absorb energy released from an explosion or projectile impact.

Finally, design of connections must allow for the energy absorbed by the walls to be transferred to the floor system. These high forces can result in elevated stresses in local columns if transferred directly from the walls.[384] By transferring the load from the walls to the floor, the floor can act as a rigid diaphragm, dispersing the load throughout the structure.

Material design values and material considerations for concrete masonry unit (CMU) construction vary depending on the extent of security offered by the correctional facility. Because the proposed facility design will include minimum, medium, and maximum-security cellblocks, design values for maximum-security facilities will be used. This convention will provide unison throughout the structure, and the flexibility to

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[381] Krasnow, Peter, p.183-184
[382] Shutt, Craig A., www.pci.org/markets/markets.cfm?path=justice&id=blast.cfm
[383] Shutt, Craig A., www.pci.org/markets/markets.cfm?path=justice&id=blast.cfm.
[384] Shutt, Craig A., www.pci.org/markets/markets.cfm?path=justice&id=blast.cfm

rearrange occupant layout and any further expansion. Therefore, for maximum-security facilities, all voids in CMU blocks should be fully grouted with a minimum of #4 rebar placed in each cavity.[385] It is further recommended that type M mortar should be used to bond the blocks. For most construction, type N or type S mortar is used. Type M mortars are typically used for walls resisting high compressive strengths. Type S mortars are used for masonry at grade, providing high compressive and tensile strengths. Type N mortars are used for exterior finishes above grade requiring moderate compressive strengths.[386]

As with precast concrete wall assemblies, the overall weight and stiffness of the masonry walls contribute to their inherent structural stability and carrying capacity in the effect of a sudden force. Studies conducted by the Canadian Masonry Research Institute have shown that masonry wall assemblies have a high bullet resistance.[387] Tests have shown that although a direct hit from a bullet of common artillery will chip or puncture holes into the surface of the wall, the bullets are not able to penetrate through the entire wall system.

Concrete masonry walls that are constructed as fully grouted with adequate reinforcement also have exceptional battering resistance. Tested in compliance with ASTM methods previously described, CMU walls have the ability to control attempts of inmate escape through interior vandalism.

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[385] Krasnow, Peter, p.184
[386] Ericson, David, www.taunton.com/finehomebuilding/pages/h00032.asp
[387] Masonry Institute of British Columbia

6.5 Wall Design Comparisons

When choosing a wall assembly, consideration needs to be given to the desired performance, design implications for the assembly, consequences on the structural framework, and other construction requirements needed. Table 6.5.1 provides a comparison of related design and construction implications associated with the three wall types typically used in prison construction. Figure 6.5.1 presents a qualitative comparison between the three wall systems, focusing on issues related to cost, security, durability, and scheduling. These four issues are important for any project to ensure completion without any delays and a long service life. Wall security is of particular importance for a correctional facility for reasons of public safety as well as inmate confinement.

Table 6.5.1 Wall Assembly Design and Construction Considerations[388, 389]

Assembly	Cost ($ per s.f.)	Structural Implications	Construction Requirements
Precast Concrete	15.41	• Panels can span from column to column and floor to floor • Framework must support wall weight ≈ 150pcf	• Hoist for panel placement • Onsite casting beds • Transportation • Approximately 10 crewmen
Tilt-up Concrete	8.29	• Optimum panel size limited to 300 to 500 s.f. • Lateral support required at a minimum of 30 times the wall thickness • Framework must support wall weight ≈ 150pcf	• Hoist for panel placement • Strongbacks for support • Clear space on floor slabs • Approximately 19 crewmen
Reinforced Masonry	8.57	• Multiple components required • Multiple tradesmen required • Framework must support wall weight ≈ 55psf • Overall wall thickness	• Time of construction • Approximately 5 crewmen

388 RSMeans, Building Construction Cost Data, 64th Annual Ed., p.109, 532, 535, 632, & 634
389 RSMeans, Assemblies Cost Data, 31st Annual Ed., p.132 & 137

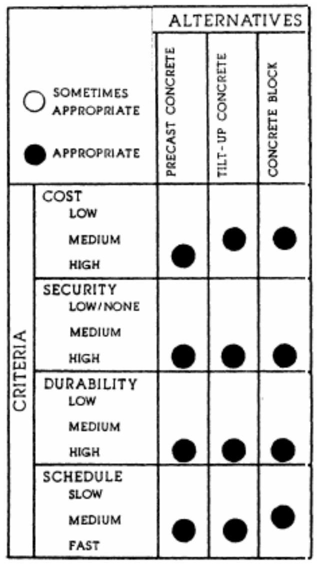

Figure 6.5.1 Qualitative Comparison between Wall Assemblies[390]

6.5.1 Precast Concrete Panels

Precast concrete panels are a desirable wall assembly used in correctional facilities because of the security and durability they provide. This wall type offers variations in exterior finishes including textures, colors, and patterns. Thin veneers of brick or stone may be cast into the panels, thereby enhancing the aesthetic appeal of the building. Wall and door assemblies may also be cast in place at the factory, lessening the labor and construction time required at the jobsite. Wall panel connectors and tieback anchors can be embedded into the panel when the concrete is placed. With these components embedded in the panel, less mechanical components are exposed, thereby increasing the inherent strength and security of the assembly.

Precast concrete panels gain their competitive edge versus tilt-up concrete and reinforced masonry because they allow for fast-track construction. Typical precast concrete panels are cast offsite. The benefit of this is that the panels are housed within a controlled environment while curing. This can ensure consistent quality as well as

[390] http://www.bdcorr.ca.gov/cppd/more%20for%20less/4-.pdf

adequate strength. Manufacture of the panels may begin at the start of the construction project. This is crucial since concrete typically has to cure for up to 28 days to gain adequate strength. If the panels are cast after the structural framework is erected, then this could lead to a delay in construction. Mechanical components and interior finishes cannot be placed until the exterior walls have been placed.

There are no prescribed limitations on precast concrete panel sizes. Because of steel reinforcement, precast panels have the ability to span substantial distances both vertically and horizontally, while maintaining the deflection limits prescribed by the building codes.

Design limitations on panel size would reflect a desire to minimize panel thickness. Panels with a larger surface area require a greater thickness to maintain its intrinsic stiffness. Concerns for wall panel thickness revolve around minimizing the overall building footprint and intrusion into the interior space.

The transportation of the panels to the jobsite also provides restrictions. Limitations for the panel's dimensional properties and gross weight must be within the truck's hauling capacity and those values permitted to safely travel along the roadways.[391] If either the panel size or weight exceeds the allowable values for roadway transit then special permitting is required, which could lead to delays in transportation and construction. Since panel size restrictions are stipulated on factors other than structural, there are no implications on the placement of columns within the structural framework.

Precast concrete panels are typically designed as load bearing wall units. The use of load bearing walls would result in the need for fewer or smaller structural members.

[391] Nashed, Fred, p.170

For the design of the correctional facility, the structural framework was detailed to transfer all the gravity loads. Therefore, if precast concrete panels were used for an exterior finish, the design of the aforementioned structural members should be reviewed and revised accordingly.

6.5.1.1 Tilt-up Concrete Panels

Tilt-up concrete panels are a form of precast concrete panels. Tilt-up concrete panels utilize onsite casting. In terms of economics, tilt-up construction typically has a lower cost than precast concrete. This can be attributed to not having to transport the panels to the jobsite. Another cost savings with tilt-up wall construction is that lower strength concrete can be utilized. In precast construction, concrete strength of 5,000psi is typically used because of the stresses associated with transporting the panels to the jobsite. Because tilt-up panels are constructed on site, 3,000psi concrete can be utilized.

Typical wall panel sizes range from 20 to 30 feet and length and not more than 50 feet in height.[392] An optimum panel size is 300 to 500 square feet.[393] The limiting factor for the panel size is governed by the square footage of the floor slab. This is because tilt-up panels are often cast on the floor slab because of its smooth surface and homogenous material.[394] This results in the major structural implication of tilt-up concrete wall construction.

The concrete floor slabs must be designed and detailed to adequately transfer the loads produced during the casting stage of the wall panel construction. Considerations to prevent cracks as a result of the loading must be made. The formation of cracks would

[392] http://www.bdcorr.ca.gov/cppd/more%20for%20less/4-d.pdf
[393] RSMeans, Building Construction Cost Data, 64^{th} Annual Ed., p.632
[394] http://tilt-up.org/

not only affect the performance of the floor slab but also the appearance of the panels. Slab design must also accommodate the weight of any hoists that may be needed to lift the concrete panels into place.

Although tilt-up concrete panels resemble a slab in appearance, their design and analysis resembles that of a column.[395] This is because of the relative slenderness of the cross-section as well as concerns for buckling under combined axial and lateral loading. As columns, tilt-up panels are only connected at their top and bottom edges. These connections are thought to be pinned with the structural steel, slabs, or footings.[396] As a result, additional load considerations must be given to these components in their design. Because tilt-up panels are only connected at their top and bottom portions, panel size has little implications on the placement of columns. Wall panels are not connected vertically to one another, allowing for expansion and contraction resulting from temperature changes as well as creep.[397]

Design consideration for the panels must reflect the stresses associated with placing the panels into position. In the casting process, inserts are placed in the wall assembly. These inserts are used to hoist the panels into position, which involves rotating the panels from their horizontal casting position to the vertical. As the wall is tilted, the outermost portion of the panel is cantilevered past the outer inserts. As a result, tensile forces develop along the surface of the wall. Design checks must be made to ensure that the resulting tensile forces do not exceed the tensile capacities of the concrete and reinforcement.

[395] Kripanarayanan, K. M., p.24
[396] Tilt-up Concrete Association
[397] http://tilt-up.org/

6.5.2 Reinforced Masonry Walls

Reinforced masonry wall assemblies were the third wall assembly consideration for the correctional facility. Reinforced masonry offers a wide variety of shapes, colors, and textures to enhance the visual appeal of the building. Masonry stacking patterns and orientation can add interest to the facade.

There is a plethora of brick shapes, colors, and textures upon which a design can be based. The colors and textures can have a profound effect on the strength characteristics because of admixtures required to alter the color and fabrication techniques needed to give the texture. The nominal size of the units affects the design requirements. Considerations for the placement of reinforcement and grout can all be affected. Furthermore, block size also can affect the layout of the masonry. Sizes should be chosen to minimize the number of specialty cuts required during the construction around openings and wall ends.

Unlike precast and tilt-up concrete wall assemblies, which are comprised of large panels, reinforced masonry wall assemblies are comprised of small blocks bonded together. Therefore, proper design and detailing is required to maintain continuity and structural integrity. The strength of the wall assembly is not based on one homogenous material. Instead, the strength of a reinforced masonry wall is due to a combination of the masonry units, mortar, and grout, and not the weakest component of the system.[398]

Criteria for in-plane deflection and out-of-plane deformation can be more stringent for reinforced masonry assemblies because of the increased chance of forming cracks or material spall. Therefore, the vertical spacing between floors and the horizontal

[398] KPFF Consulting Engineers, http://www.masonryinstitute.com/hollow/

spacing of columns is of greater concern for reinforced masonry wall construction than for the wall assemblies previously described. This is because the permissible unsupported wall length and height are a function of the wall thickness.[399] The farther that a wall spans, the thicker it needs to be to maintain its stiffness to resist deflections and deformations.

6.6 Results of Prison Superstructure Exterior Wall Design

For the prison superstructure, reinforced masonry construction was chosen to develop the wall design. In addition to the value it contributed to the project, choosing reinforced masonry offered an opportunity to pursue design requirements for a building material that was not addressed in previous coursework. Reference was made to publications offered by KPFF Consulting Engineers including *Design Guide for Structural Brick Veneer* and *Notes on the Selection, Design, and Construction of Reinforced Hollow Clay Masonry* for basic requirements and considerations associated with reinforced masonry design. Design procedures were adopted from KPFF Consulting Engineers' *Note on the Selection, Design, and Construction of Reinforced Hollow Clay Masonry* as well as Robert R. Schneider's and Walter L. Dickey's *Reinforced Masonry Design, 2nd Ed*. Design values for reinforcement placement and distribution as well as allowable stresses were taken from the *2003 International Building Code* and *The Massachusetts State Building Code (780 CMR), 2nd Ed*.

Table 6.6.1 provides the initial block sizes and design assumptions used for the reinforced masonry wall design. These include the type of concrete block and strength design values, mortar type and strength, as well as steel reinforcement grade and strength.

[399] The Massachusetts State Building Code (780 CMR)., 2108.1

The wall was assumed to span 19.58 feet horizontally between columns and 12 feet between floors. The wall was also treated as non-load bearing.

Table 6.6.1 Assumed Reinforced Masonry Wall Design Values

Design Hollow Concrete Block Information	
Type of Block	• 8" hollow concrete block • Nom. Dim. = 16"L x 8"W x 8"T • Fully grouted • f_m = 4,000psi • f'_m = 2,300psi • f_c = 460psi
Design Mortar Information	
Type of Mortar	• Type M Mortar • f_c = 3,000psi
Design Mortar Information	
Type of Rebar	• Grade 60 • f_s = 60,000psi

Because the reinforced masonry wall was considered as non-load bearing, no service loads were imposed on the wall except for its self weight. Overturning moments were calculated with respect to the self-weight of the wall and the applied lateral load due to wind. The wind pressure was used because it was shown to govern the lateral design of the structural frame as compared to the forces associated with seismic action.

In calculating the nominal moment applied to the wall, the assembly was treated as a simply supported beam that spanned between the adjacent floors. Therefore, the equations used to calculate the moment resulting from the wall self-weight as well as from the lateral loads resembled those used to calculate the applied moments for a beam. The moment resulting from the wind load was equated to be greater than that resulting from the wall self-weight; hence, it was used to further the design.

Based on the moment resulting from the wind force, the area of steel per foot of wall required to resist the tensile forces was determined. The area of steel needed equaled 0.0262 square inches per foot of wall. With that, an initial rebar size and spacing was chosen. Number four rebar spaced at 48 inches on center was used, providing an area of steel equal to 0.049 square inches per foot of wall. The provided area of steel far exceeded the calculated required amount, almost doubling it. A lesser amount of steel could not be provided because Section 2104.3.2 of *The Massachusetts State Building Code (780 CMR), 6th Ed.* mandates that the vertical reinforcement should consist of at least number four rebar spaced no greater than 48 inches on center.[400]

Having determined the nominal moment imposed on the wall as well as the required amount of reinforcing steel to resist the tensile forces, the adequacy of the concrete blocks was determined. A stress block was drawn with reference to the location of the reinforcing steel in tension and the compression region within the concrete block. Based on geometry of the block and the tension and compression regions, the maximum allowable moment was calculated using the allowable working stress for the concrete block. The maximum allowable moment was determined to be greater than the applied moment from the forces, thereby verifying that bending did not govern the design of the wall assembly.

The distribution of reinforcing steel was the last design check performed. Referencing section 2104.4.3 of *The Massachusetts State Building Code (780 CMR), 6th Ed.*, the minimum vertical reinforcing steel required is equivalent to 0.0007bt, where *b* is the width of wall and *t* is the thickness.[401] Based on this equation, number four rebar spaced at 36 inches on center was required. The horizontal reinforcement is the difference between the minimum total amount of steel required (0.002bt) and the provided vertical reinforcement. As a result, number four rebar spaced at 18 inches was needed.

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[400] The Massachusetts State Building Code (780 CMR)., 2104.4.3
[401] The Massachusetts State Building Code (780 CMR)., 2104.3.2

The reinforcing steel required to satisfy the *Massachusetts State Building Code* was less than that recommended for correctional facilities. As previously stated with respect to reinforced masonry walls, the intended security level of the cellblock determines the minimal reinforcement.[402] For the correctional facility in question, the suggested minimal requirements for maximum security facilities would be advisable since all levels of security will be provided, and it would allow for ease of further expansion and rearrangement. Therefore, number four rebar should be placed eight inches on center both horizontally and vertically in conjunction with the eight inch hollow concrete block, fully grouted. Detailed design calculations of the reinforced masonry wall assembly can be found in Appendix I.2 Prison Cellblock Exterior Wall Design.

Having completed the design, there were a few structural implications that became apparent. The *Massachusetts State Building Code* mandates that the allowable maximum ratio of wall height to wall thickness and wall length to wall thickness cannot exceed 18 for a non-load bearing exterior wall.[403] With a wall thickness of 7.625 inches (nominal thickness equal to eight inches), the ratio of wall height to wall thickness equals 18.9 for a 12 foot height wall. Redesigning the wall assembly with a ten inch wide block would resolve the problem. However, when examining the wall length to wall thickness, the ratio with the initial design equals 30.8, and the use of a ten inch wide concrete block would still not adequately solve the requirements. Therefore, it is advisable to alter column spacing as well as wall thickness to satisfy the requirements. A balance between

[402] Krasnow, Peter, p.184
[403] The Massachusetts State Building Code (780 CMR)., 2108.1

wall thickness and column spacing should be met in order to facilitate the design requirements and intended use.

6.7 Results of Administration Building Exterior Wall Design

The exterior wall design chosen for the administration building was chosen to compliment the visual aesthetics of the prison superstructure. An exterior wall assembly consisting of a brick veneer anchored to steel studs was designed. This system would provide a continuity of materials, colors, and textures with those provided by the prison. Although the exterior wall assembly could have been designed using reinforced masonry, such as that detailed for the prison, the brick veneer and steel stud backer assembly provided a less expensive and lighter means of capturing the same visual appeal. Steel stud backer wall assemblies provide the stiffness for the wall assembly, thereby reducing the amount of reinforcement needed, if any, for the brick veneer. This results in a considerable decrease in the self weight of the wall, up to 75 percent less than the weight of a reinforced masonry wall assembly.[404]

In the design process, reference for design considerations, procedures, and values was made to KPFF Consulting Engineers' *Design Guide For Anchored Brick Veneer Over Steel Studs* and *The Massachusetts Building Code (780 CMR), 6th Ed*. Table 6.7.1 provides initial design considerations and assumptions. The design proceeded with the requirements for the first floor of the administration building, presenting the worst case because of increased wall heights. Based on the structural layout, column spacing of 20 feet and a floor-to-floor height of 15 feet for the first floor was used in the design.

[404] Nashed, Fred.

Table 6.7.1 Assumed Brick Veneer Over Steel Studs Design Values

Design Brick Veneer Information	
Type of Brick:	• Normal brick • Nom. Dim. = 12"L x 4"W x 2 2/3"T • f'_c = 2,500psi • E = 1,875ksi
Design Steel Stud Information	
Type of Stud:	• Dietrich supplied • Cold-rolled steel • F_y = 33ksi • E = 29,000ksi • Stud spacing = 16" on center

Brick veneer over steel stud assemblies are considered as non-load bearing wall assemblies. Therefore, steel stud assemblies are responsible for transferring only lateral loads. The steel studs must be designed to reduce deflections to maintain a relatively high stiffness and to minimize cracks within the brick veneer. Therefore, a maximum allowable lateral deflection of L/600 was used in the design, and this limit determined the required moment of inertia. The required moment of inertia was calculated to be $6.44 in^4$. Through the use of the *Dietrich Curtain Wall / Light-Gauge Structural Framework Products* catalog, a CSW 10 gauge, eight inch stud was chosen, providing a moment of inertia equal to $6.577 in^4$. Once the initial steel stud size was chosen, its adequacy to transfer the applied moment was checked. An elastic analysis of bending demonstrated that the CSW 10 gauge, eight inch stud was sufficient to transfer the applied loads.

For the wall design of the administrative building, a brick veneer over steel stud wall assembly was designed. It is recommended to use CSW 10 gauge, eight inch studs supplied by Dietrich. Anchors are recommended to be placed 16 inches on center in both the horizontal and vertical direction and should be designed to transfer 140 pounds of force each. For detailed calculations, refer to Appendix I.1 Office Exterior Wall Design.

7 Fire Considerations

Over the lifespan of many buildings, fire poses a major threat from the early stages of construction until the last of their use. Fire poses a threat because of its rapid ability to destroy the functionality and safety of building. Because fire can directly impact the health of individuals inside in a building during a fire event, protecting the occupants from the effects of the fire becomes a primary concern for the design of passive and active fire protection systems.

While buildings are not typically designed to allow occupants to remain inside through the duration of a fire event, they should be designed to allow sufficient time for all occupants to exit in a safe manner. With the safety of the occupants in mind, controlling the spread of fire can often be the difference between saving and losing lives.

In addition to the threat to human lives, structural fires can cause considerable financial distress for the interests involved in the building. In the case where a number of businesses occupy one commercial building, for example, a fire could disrupt the function of any of the businesses involved and create a financial burden for the owners of the commercial space. The major factor for protecting against a fire is time. The longer a building can safely withstand the effects of heat, the more time occupants have to leave the building and first responders have to bring the situation under control and possibly save the building from costly damages.

While materials such as steel and concrete are noncombustible, structures composed of these materials are never really fireproof. Every structure has a certain amount of fuel depending on what is contained inside. A library, for instance, contains a large quantity of fuel in the form of books and furniture. It is items such as these that feed flames over the course of a fire event. Even though a steel or concrete beam, girder, and column is not likely not to catch fire, the effect of heat on these elements caused by combustion of flammable sources can be critical, contributing to the degradation of strength and stiffness and introducing additional forces and moments.

7.1 Occupancy Use Group and Construction Type

Depending on the occupancy use and size of a building, building codes prescribe the different construction types that are considered to be acceptable. Correctional facilities fall under the occupancy group of Institutional 3 (I-3) in the *2003 International Building Code*.[405] The I-3 group takes into consideration that inmates housed in correctional facilities are often restricted in their movement throughout the building. This restricted movement can lead to an increased susceptibility to fire exposure due to interruption in egress paths. Within the I-3 use group there are five sub-classifications that further describe the detail of the occupancy based on the restriction of inmate movement.[406]

Table 503 of the *2003 IBC* indicates the allowable building construction types based on occupancy use group, floor area, and building height.[407] Depending on these factors, there are five construction types, each with “Type A” or “Type B,” with the exception Type IV (Heavy Timber) with only one type. Table 7.1.1 presents the various construction types and whether combustible or noncombustible material is permitted. The materials allowed in these construction types range from wood construction for Type V to noncombustible concrete and steel for Type I. Because the floor area of the prison is

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[405] *2003 International Building Code*, Section 308.4
[406] *2003 International Building Code*, Section 308.4 develops the five sub-classifications based restricted movement of inmates in the correctional facility.
[407] *2003 International Building Code*, Table 503

over 40,000 sq. ft. per dayroom level, the construction type was limited to Type I for occupancy use group I-3. Construction Type IB allows the floor area, but is limited to four stories in height. While the basement of the prison could potentially used for the same purposes as the other floors, the *IBC* states that the number of stories should be counted for those above grade. Since the prison is only four stories above grade, construction type IB is permissible.

Table 7.1.1 Construction Types[408]

Construction Type	I		II		III		IV	V	
Subtype	IA	IB	IIA	IIB	IIIA	IIIB	Heavy Timber	VA	VB
Construction Allowed	Non Combustible		Non Combustible		Combustible		Combustible	Combustible	

While the office structure has a much smaller floor area than the prison, it extends to a height of six stories above grade. Since this is greater than the maximum height allowed for Type IB, it must conform to Type IA.[409] Table 7.1.2 graphically shows the constraints of the construction types considered for the correctional facility.

Table 7.1.2 Permissible Height and Floor Area for Occupancy Use Group I-3

Occupancy Use Group	Construction Type	Allowable Area per Floor	Allowable Height	Application to Project
I-3	Type IA	Unlimited	Unlimited	Office or Prison
	Type IB	Unlimited	4 Stories	Prison

7.2 Fire Resistance Ratings Requirements

For a given construction type, particular elements of the structure must comply with certain fire resistance ratings. These fire ratings are based on the times that these elements should be expected to perform effectively when exposed to fire. The table below shows data from the *2003 IBC* for fire resistance rating times for construction types IA and IB for various typical structural elements.

[408] *2003 International Building Code*, Chapter 6
[409] *2003 International Building Code*, Section 504.2 states that the use of automatic sprinkler systems allow for a 20' increase in height, or 1 story, for a given construction type. In the case of construction Type IB, increasing the allowable height to 5 stories does not help in the case of the office, thus still necessitating Type IA.

Table 7.2.1 Fire Resistance Rating Requirements (hrs)

		Construction Type	
		Type IA	Type IB
Building Element	Structural Frame	3	2
	Bearing Wall	3	2
	Non-Bearing Wall		
	- Interior	0	0
	- Exterior (601)	0	0
	Floor Construction	2	2
	Roof Construction	1.5	1

While basic construction materials may not provide enough time to satisfy the ratings above, certain insulating materials can improve fire resistance. The concept of insulating material will be further discussed in the following section on passive fire protection.

7.3 Fire Protection

When designing a building to withstand a fire event, there are two basic stances from which the structure can be protected. These two provisions are described as passive and active fire protection systems. Though passive and active systems provide some protection against a fire event, the two are not necessarily mutually exclusive of each other. Often times, passive and active fire protection systems work in tandem to increase the overall safety of a building.

7.3.1 Passive Fire Protection

Passive fire protection systems typically consist of strategic placement of partitions and fire resistive insulation to increase the time that structural elements can function effectively during fire conditions. This form of fire protection does not eliminate the threat of fire. Partitions function as barriers to the spread of fire throughout the building, and both partitions and fire resistive insulation slow the heat transfer between fire and structural elements. For most structural materials, increasing the temperature of a material is detrimental to its strength and stiffness.

In cases where steel is exposed to elevated temperatures, its yield strength will decrease. This means that as steel temperature rises, a members' ability to carry load decreases. Figure 7.3.1 shows the change in yield strength as a function of time of an unprotected W18x40 WF-shape beam, supporting a floor slab, subjected to elevated temperatures. The time-temperature curve follows that established by the ASTM E-119 time-temperature curve. Yield strength is assumed 50 ksi at ambient temperature.

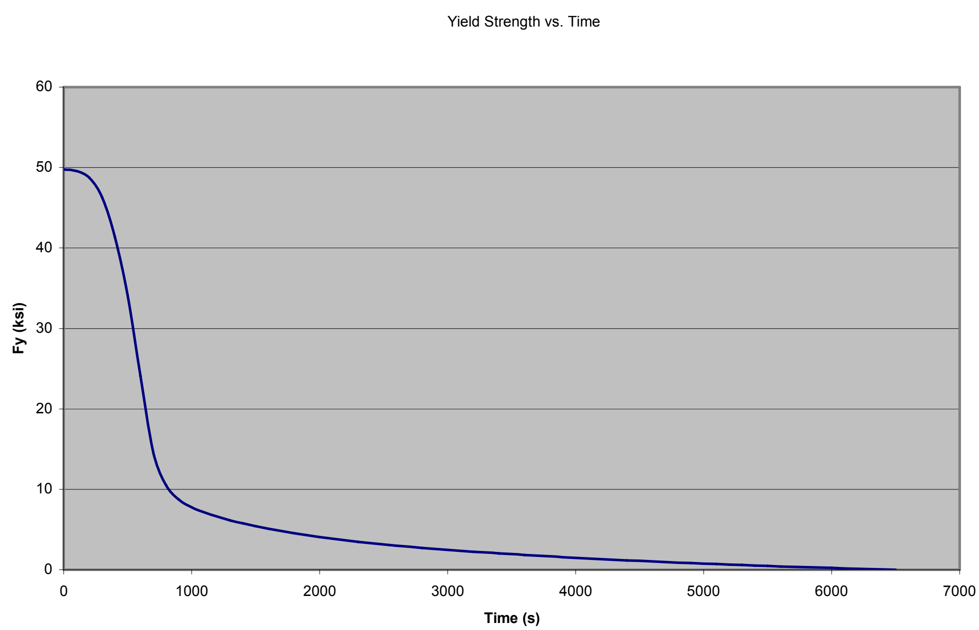

Figure 7.3.1 Unprotected W18x40 Yield Strength vs. Time

As a result of this decrease in yield strength, the ability for structural members to resist load decreases. Figure 7.3.2 illustrates the live load capacity of the same WF-shape beam exposed to ASTM E-119 time-temperature curve. Notice that in less than 15 minutes, the member is no longer capable of carrying any live load. Since the live load acting on a beam includes the weight of furnishing and other non-permanent fixtures, some level of live load capacity is needed to maintain structural integrity during fire conditions. Thus, the member will likely fail before the capacity ratio reaches zero.

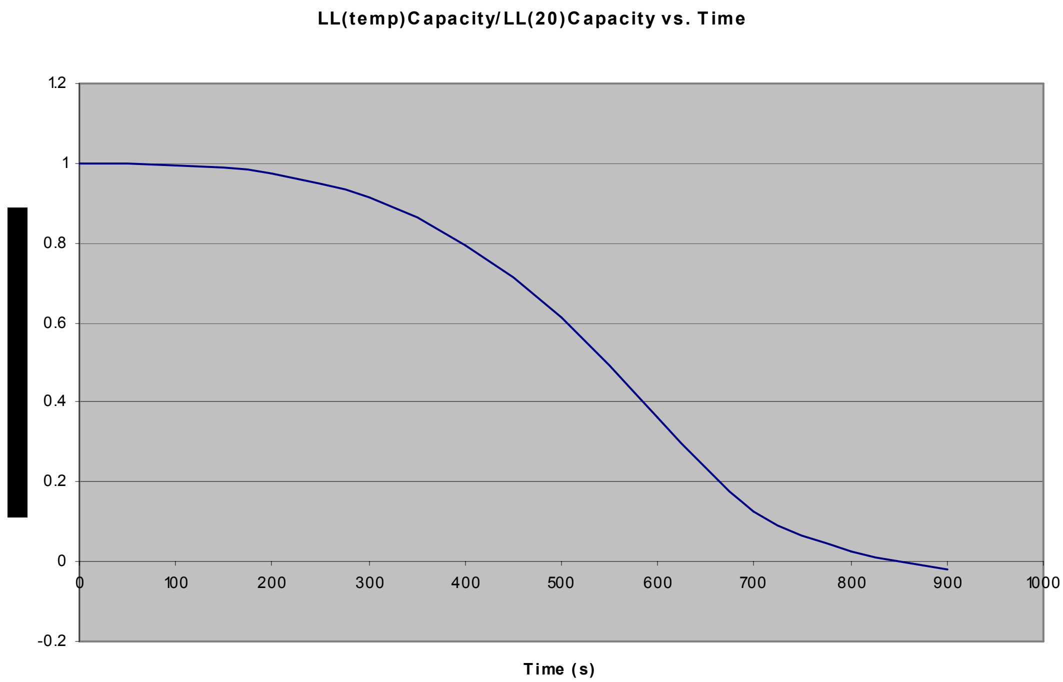

Figure 7.3.2 Unprotected W18x40 Live Load Capacity vs. Time

In addition to decreasing yield strength with increasing temperatures, the modulus of elasticity also decreases. Consequently, deflections of members will increase as a function of the temperature. Figure 7.3.3 demonstrates the change in modulus of elasticity of the unprotected W18x40 as a function of exposure to the ASTM E-119 time-temperature curve.

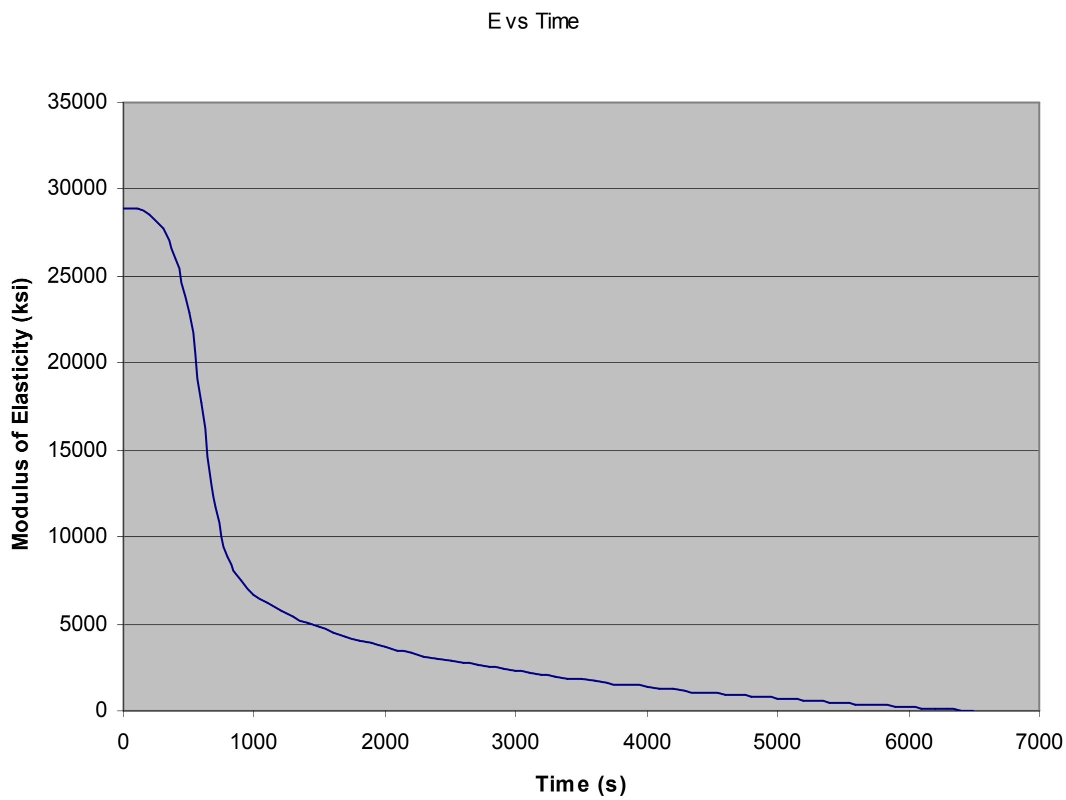

Figure 7.3.3 Unprotected W18x40 Modulus of Elasticity vs. Time

Based on yield strength, the beam was only capable of carrying live loads for fifteen minutes, which would be grossly insufficient in most structures. In addition, the W18x40 beam investigated was overdesigned to approximately five times the necessary live load capacity at normal, ambient temperatures. Even with this substantial overdesign, the exposed beam rapidly lost its ability to carry load. Furthermore, most beams designed for this situation would have even shorter safe functioning times when exposed to fire because they would most likely be designed to carry normal ambient temperature live loads.

Since over designing members to five times there live load capacity is a significant waste of material and money, a more economical solution is required. The more economical solution is where fire resistant insulation comes into play. There are many different types of insulation available including gypsum board, cementitious spray, mineral fiber spray, perlite or vermiculite spray, and even intumescent paint.[410] Depending on the material, various installation methods are available. One common technique involves spraying insulation to difficult to reach locations, allowing the laborer to apply insulation at a high rate from a distance.

Cementitious insulation is a heavy solution to the problem of protecting structural members. At a weight approximately 100 pounds per cubic foot, use of cementitious insulation may increase the structural member sizes because of the increased dead load. Fibrous insulation is commonly composed of mineral, perlite, or vermiculite fibers embedded in the insulation mixture. While fibrous alternatives are lighter in weight than cementitious insulations, they are more fragile and are more susceptible to damage.

Intumescent paint is a special paint that expands "into a thick charry mass when it is heated."[411] This expanded mass improves the protective barrier surrounding the beam. Since the paint only expands when heated, it allows for a more aesthetic approach to fire protection than other spray alternatives. Because intumescent paint may require a particular thickness depending on the desired fire resistance rating, it is often applied in several coats.[412] Because of the high cost of intumescent paint, other spray-on insulations are often desired.[413]

So the question becomes, "How much does fire protective insulation really help the steel beam?" To answer that question, Figure 7.3.4 compares the yield strength of an unprotected beam to that of the same beam with perlite spray insulation. The beam being investigated is the same W18x40 as seen before. As before, it is assumed that the yield

[410] Buchanan, Andrew H., p.182
[411] Buchanan, Andrew H., p.190
[412] Buchanan, Andrew H., p.191
[413] Buchanan, Andrew H., p.190

strength of the steel is 50 ksi at ambient temperature. Insulation thickness of ½, 1, and 1-½ inches are considered.

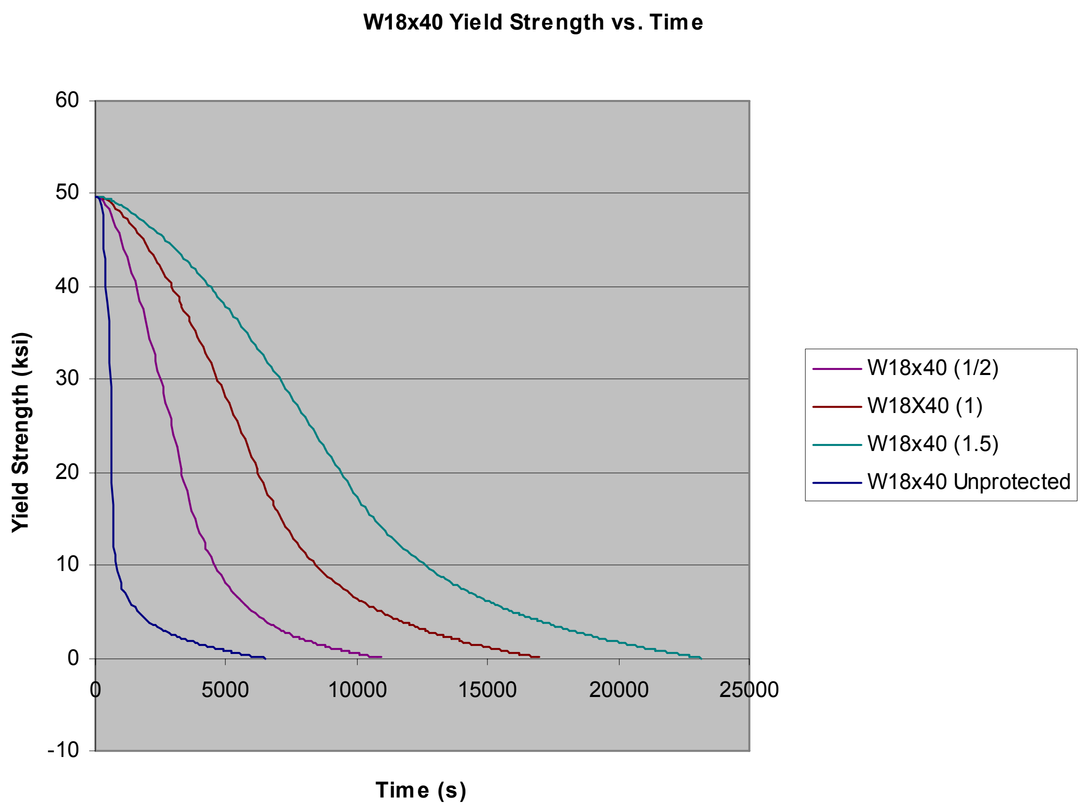

Figure 7.3.4 Comparison of Insulated and Unprotected W18x40

As Figure 7.3.4 clearly shows, the addition of fire resistant insulation to structural members can increase the time duration for which a steel member functions adequately when exposed to elevated temperatures. While the effects of elevated temperatures on steel are detrimental to material strength, concrete is not impervious to the effects of fire.

It is true that various forms of concrete-based insulations may be used to insulate steel members. In spite of this fact, reinforced concrete members are still susceptible to exposure to elevated temperatures. Because of the composition of concrete, it generally performs well in fire conditions.[414] Not only is the reinforcing steel within reinforced concrete member in danger of losing strength due to heat, concrete itself can undergo changes. A clear cover of concrete typically protects reinforcing steel in a reinforced concrete member.

In certain situations, a phenomenon known as spalling can occur. While the definite cause of spalling is unknown, it is believed that a major cause is due to expansion of water vapor within the concrete.[415] This expansion due to the formation of water vapor causes tensile stresses in the concrete. Since concrete performs relatively poorly in tension, the concrete has the potential to crack and fall off the concrete members. With less concrete surrounding the reinforcing steel, the steel becomes more susceptible to the effects of fire. Though the likelihood of serious spalling is low, provisions should be taken in "critical structures or those containing high-strength concrete."[416] One possible defense against spalling is to add steel or polypropylene fibers to the concrete mix.[417]

7.3.2 Active Fire Protection

While passive fire protection systems alone may allow sufficient time for occupants to evacuate and fire services to extinguish the fire, active systems may reduce the threat of the fire from the start. Active fire protection systems often include automatic sprinkler systems. Automatic sprinkler systems serve as a fire suppression apparatus. By suppressing the fire, active systems reduce the fire exposure time for both occupants and structural elements.

[414] Buchanan, Andrew H., p.225
[415] Buchanan, Andrew H., p.226
[416] Buchanan, Andrew H., p.229
[417] Buchanan, Andrew H., p.229

Similar to any mechanical system, automatic sprinkler systems are susceptible to malfunction. Without proper maintenance, sprinkler systems may not operate to the best of their ability. Maintenance visits may disrupt the operations of a building, such as a correctional facility, and ultimately increase operating costs. Even with regular maintenance, automatic sprinkler systems are not failsafe, thus there is some probability that they may not function correctly during a fire event.

In cases where automatic sprinkler systems are unable to stop the growth of a fire, extreme cases of structural deformation may cause additional detriment to sprinkler system function. Since these systems rely on water to suppress fire, structural deformation can disrupt or damage the plumbing that supplies the sprinkler systems. For this reason, the use of automatic sprinkler systems does not eliminate the need for passive fire protection systems.

7.4 Implications of Structural Design for Fire Conditions

For the case of the correctional facility designed in the scope of this project, there are two buildings: the office and the prison. Since the layouts and functions of these buildings are distinctly different, the allowable construction types were investigated separately. Since the operation of the correctional facility is dependent on both structures, the controlling construction type was selected for both buildings. From Table 7.1.2 construction type IA was the only type appropriate for the office, and consequently the prison.

From the determination of the allowable construction type, the fire resistance ratings under type IA from Table 7.2.1 were selected for both buildings. In the case of the reinforced concrete scheme for the office, minimum clear cover to protect from fire effects followed provisions set forth in the *2003 IBC*.[418] In general, the clear cover distance required for typical reinforced concrete construction provided adequate cover (greater than 1-3/4 of an inch) to provide fire resistance ratings from Table 7.2.1. Floor slabs, on the other hand, require a minimum thickness of five inches for a two hour rating of normal weight concrete.[419] To provide additional protection for the floors in the reinforced concrete system, vermiculite gypsum suspended ceilings were prescribed.[420]

For the steel scheme in the office, lightweight, carbonate concrete insulation was recommended to provide the fire protection for columns, beams and girders. Table 7.4.1 shows the required thickness of the concrete insulation to provide appropriate fire resistance ratings. Similar to the reinforced concrete system of the office, the steel system required vermiculite gypsum suspended ceilings to protect the floor system.

Table 7.4.1 Office Steel Frame Insulation

	Element	Insulation Thickness	Resistance Rating
Carbonate Lightweight Concrete	Column	1 Inch	3 Hours
	Beam/Girder	1.5 Inch	3 Hours

The steel frame in the prison structure has a higher likelihood of being exposed to damages. For example, activities in the recreation area of the prison could potentially harm fire resistant insulation. Objects such as basketballs could easily break insulation off of the open web joists above. Because of the fragility of the spray fiber insulation, and the weight of the cementitious insulation, a more durable, lightweight material was considered. For this reason, intumescent paint was selected as the primary joist, girder, and column insulation. Although intumescent paint is initially more expensive than other alternatives, repairs of damaged fiber insulations would ultimately disrupt the function of the facility and increase life-cycle costs. If the effective lifespan of fire resistant

[418] *2003 International Building Code*, Table 720.1
[419] *2003 International Building Code*, Table 720.1
[420] *2003 International Building Code*, Table 720.1

insulation could be established, a more accurate cost comparison could be conducted for the life-cycle cost of fire insulation in the correctional facility. With some knowledge of the durability of different fire insulations, intumescent paint was selected to provide fire resistance in the recreation area of the prison.

The cellblocks in the prison are less likely to see the damaging activities of the recreation area. Since the floor-to-ceiling height of the dayrooms is approximately twenty-two feet, the likelihood of heavily projectiles, such as chairs, reaching the ceiling is limited. As a result, 5/8 in Type X wallboard suspended from the open web joists shall provide sufficient fire resistance ratings and durability for the open web joists and floors of the dayroom. If it were decided that wallboard's durability was insufficient, intumescent paint could be applied in its place. For the individual cells, similar wallboard could be used. If durability of the wallboard were of concern, an alternative concrete ceiling could be used.

In any situation where significant number of people may be put at risk because of a fire, automatic sprinkler systems are a good idea. Because inmates in the prison have additional risk of being trapped inside due to locking devices, the implementation of an automatic sprinkler system has the potential to save lives. Though adding the sprinklers does not affect the structural design of this facility, the lifesaving capability of the system justifies its use. Therefore, automatic sprinkler systems were chosen to be used for both the office and prison. Table 7.4.2 organizes the fire protection provisions for the correctional facility.

Table 7.4.2 Fire Resistance Rating and Insulation Summary

Office Structure		Element	Fire Resistance Rating (hrs)	Insulation/Protection
	Steel	Columns	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Girders	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Beams/Joists	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Floor	2	Vermiculite Gypsum Suspended Ceiling
	Concrete	Columns	3	Intrinsic Clear Cover of Beam
		Girders	3	Intrinsic Clear Cover of Beam
		Beams	3	Intrinsic Clear Cover of Beam
		Floor	2	Vermiculite Gypsum Suspended Ceiling
Prison Structure				
- Recreation Area				
	Steel	Columns	3	Intumescent Paint
		Girders	3	Intumescent Paint
		Joists	3	Intumescent Paint
		Floor	2	Intumescent Paint
- Dayroom				
	Steel	Columns	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Girders	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Beams/Joists	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Floor	2	Lightweight Carbonate Concrete/Fibrous Insulation
- Cell				
	Steel	Columns	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Girders	3	Lightweight Carbonate Concrete/Fibrous Insulation
		Beams/Joists	3	5/8 in Type X Wallboard Ceiling
		Floor	2	5/8 in Type X Wallboard Ceiling

Calculating the total cost of fire protection systems would require the tabulation of total length of every different type of structural member used in the office and prison design. Because of the depth of calculation required for that evaluation, it was not considered in the scope of this chapter. Not to totally dismiss the issue of insulation cost, Table 7.4.3 provides a sense of relative cost of fire insulation by comparing insulation costs of a typical sixteen in deep WF-shape steel beam.

Table 7.4.3 Fire Resistance Insulation Cost Summary[421]

Insulation Type	Fire Resistance Rating	Thickness	Weight (plf)	**Cost ($/l.f.)**
Concrete	2	-	165	**38.05**
	3	-	195	**42.15**
5/8" Gypsum Board	2	5/8"	20	**28.19**
	3	5/8"	31	**28.95**
Gypsum Plaster	3	-	25	**26.5**
Perlite Plaster	2	-	21	**26.95**
	3	-	26	**28.25**
Spray Fiber	2	-	32	**6.27**
	3	-	36	**6.97**
Intumescent Paint	2	7/16"	-	**~31.20**
	3	1-1/16"	-	**~54.65**

[421] RSMeans: Assemblies Cost Data, p. 105, p. 254

8 Site Design

In this section, the major considerations for correctional facility site planning and landscape architecture are presented. The initial portion of this chapter addresses the desired traits for locating a correctional facility. Following this overview, a hypothetical location for the correctional facility is introduced, and the major elements and considerations for site design of this specific location are discussed. Also presented in this section is a drawing of the prison on the selected lot with the infrastructure and perimeter for the facility outlined. The final section of the chapter introduces several perimeter security options that could be utilized in the design of the site and details specifically what they are and to which applications these devices are utilized. The goal of this section is to introduce the reader to the many elements of the correctional facility site that differ from typical construction projects.

8.1 Correctional Facility Site Selection

Site design plays a large role in the overall security of a correctional facility complex. The design of the perimeter security systems, access and landscaping are integral components in the monitoring of the inmates and ensuring that they remain on site and separated from one another when necessary. Another lesser known role that site design can play in correctional facility operation occurs during construction.

In regards to the construction of the facility, "site design can bring with it significant opportunities to reduce a project's budget, speed up its schedule, and increase operational efficiency."[422] These advantages can be realized by using site design to ease

[422] Flannery, Michael R.

the delivery of materials to the site and allowing for construction that is more efficient by providing a better base with which to work. Site design also allows the designer to be creative in a project that is otherwise very well defined in terms of cost and structural design, offering opportunities for value engineering. One example of value engineering through site design would be soil testing to place buildings in areas of the site where foundations can be constructed in a more cost effective manner than on top of soils that would not have supported the loads of the buildings elsewhere on site.[423]

Correctional facilities have a great many considerations that are unique to them that can cause cost overruns if not accounted for properly. Some important features of correctional facility site design are "sight line requirements, lighting demands, perimeter security fencing, patrol roads, storm sewer size restrictions, gas and electric demands as well as water flow, pressure and storage requirements…"[424] Flannery suggests three guidelines when working on the site design for a correctional facility. The first is to consider the site design a major part of the project scope and to allow yourself to be innovative with it. The second goal is to incorporate an experienced correctional facility site designer or civil engineer in the development of the project more so than on a typical project. It is important that he or she be familiar or quickly become familiar with the unique nature of this type of project. The final guideline is to give the designer or civil engineer the opportunity to review the plans and make improvements if possible.

In order to streamline site design, a site must first be selected. Peter Krasnow, author of *Correctional Facility Design and Detailing*, lists several criteria that should be taken into account when selecting the location of a correctional facility. The first major

[423] Flannery, Michael R.
[424] Flannery, Michael R.

topic of concern is access to courts, local law enforcement agencies, community services, and emergency medical and fire services.[425] These are all important factors because of the level of care that the inmates require. As discussed in Chapter 2, current practices center on rehabilitating prisoners through education, labor and therapy. If the facility is located in such a way that providing these services becomes an inconvenience for the staff, then the operating cost could rise because of the need to increase wages to draw qualified personnel to work in the facility. Access also becomes important when considering garbage and snow removal, laundry services, and utilities such as power, water and telephone.[426] The correctional facility, for security reasons, must act as its own miniature community. Thus, all the requirements for a small town must be met by the design because essentially that is what the correctional facility is.

Other factors that need to be considered in selecting a location for a correctional facility are the location of nearby flood plains, wetlands, and fault lines.[427] Soil conditions are also important when selecting a site. If soil conditions do not have the bearing capacity to withstand the weight of the facility, the resulting foundation design options can become very expensive.

Krasnow also has a list of characteristics the ideal site should possess.[428] This list is summarized below:

- Sufficient size to support the initial facility and future expansion.
- Utility services to the facility developed for initial use and further expansion.
- Topography needed for positive drainage of surface water, logical grading of facility buildings, and direct vehicular and pedestrian access to all major entrances.
- Outdoor recreational needs of the current facility and room to expand in the future.

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[425] Krasnow, Peter, p.12
[426] Krasnow, Peter, p.12
[427] Krasnow, Peter, p.12
[428] Krasnow, Peter, p.12

- Adequate parking for staff, visitors, and law enforcement personnel. Suggested that staff and public parking be separate.
- Infrastructure that allows for the adequate circulation of public, staff, emergency personnel and law enforcement officials.
- Minimum number of entrances into the facility for staff, visitors, inmates (intake/transfer) and services.
- Screening of security entrances from public view.
- Screening of inmate outdoor activities and windows from public view.
- Landscaping based on ease of maintenance and compatibility with climate. Want to screen entire facility from public view.
- Building blends in with neighborhood architecture.
- Exterior lighting provides security but does not illuminate the entire neighborhood.[429]

8.2 Hypothetical Site Details

The site chosen for this correctional facility lies primarily in Greenfield, Massachusetts with a small portion jutting less than a quarter mile into Bernadston, Massachusetts. The size of the site is roughly sixty-one and one half acres. The selected site appears in Figure 8.2.1 below. The following figure, Figure 8.2.2, highlights the exact proposed location of the correctional facility on the site.

Figure 8.2.1 Proposed Site Location[430]

[429] Krasnow, Peter, p.12
[430] Mass GIS 2001

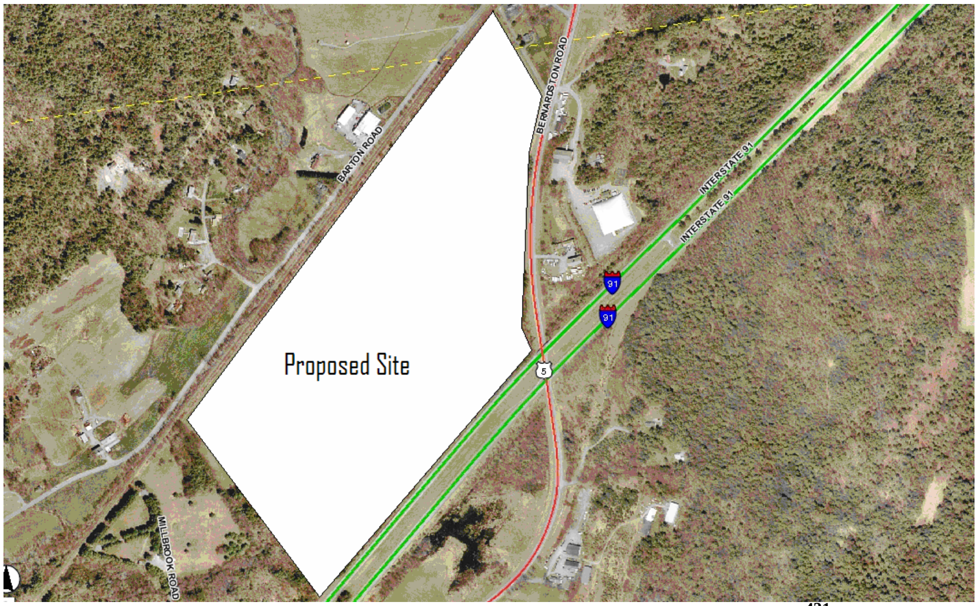

Figure 8.2.2 Proposed Site Location with Highlighted Lot[431]

This site is bounded on two sides by a major arterial roadway and a minor arterial roadway. The southeast side is determined by Interstate 91 and the northeast side by route 5 (locally known as Bernadston Road). The north-northeast face of the lot is bounded by a small local road that may actually be a driveway for the single house located on the road. It is not a through road, nor is it paved. The northwestern boarder of the proposed site is bounded by a railroad line. The amount of usage that this particular section of track incurs is unknown, but upon investigation of the site, there was no plant growth in or around the track bed, which suggests that this branch of railway is still active. An additional point of interest is that along the west side of the track bed (not in the proposed location for the prison) there is a small brook. The southwest edge of the lot is demarcated by a line of trees, presumably separating the proposed parcel of land from the adjoining one, which appears to be a series of pumping stations after consulting the topographic maps.

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[431] Mass GIS 2001

The proposed site provides relatively easy access for all staff and administrative personnel. Greenfield lies at the intersection of Route 2 and Interstate 91. Other minor arterials that run through the town are Routes 5, 10 and 2A. The major neighboring facilities include an indoor athletic facility, an automotive customization business, and another non-privately owned structure on the northwest side of the lot. There is also a small youth baseball field located approximately a quarter mile northwest of the proposed site. There are a few private residences located in this area. The closest cluster is located along the northwest face of the lot. The area becomes more densely populated to the south, along Route 5. Factors other than access that come into play for this site are discussed in the following section. Solely in terms of access to major roadways and amenities, this site will meet the needs required for a correctional facility, which was the major reason it was selected. The following figures are photos of the site taken in the middle of November 2005.

Figure 8.2.3 Site Photo 1 – Northeast side of lot along Rt. 5 (facing east southeast)

Figure 8.2.4 Site Photo 2 – Center area of lot (facing west southwest)

Figure 8.2.5 Site Photo 3 – Northwest edge of the Lot (facing west southwest)

8.2.3 Physical and Environmental Considerations

Figure 8.2.6 provides a two dimensional representation of the topography of the site. Were this site to be developed, a surveyor would be commissioned to provide a more accurate depiction of the topography of the land as well as lot dimensions.

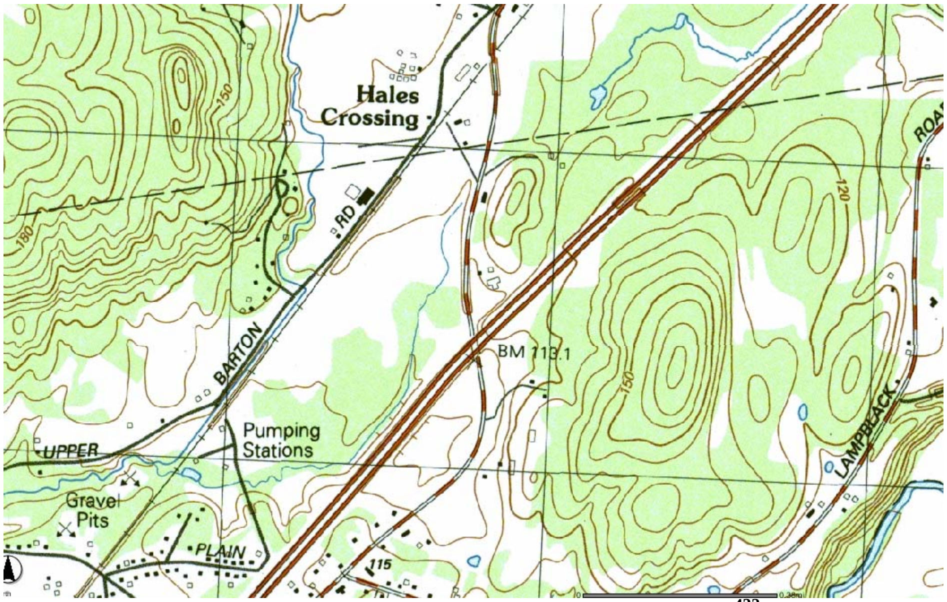

Figure 8.2.6 Topographic Map of Proposed Site[432]

Note that there is apparently another stream running through the proposed building area. This stream could possibly be seasonal, which is why it would not have been detected on the date of the site visit, but it still may present some issues when regarding the overall design of the site. The variation in elevation encountered over the course of the site is not at all significant. The maximum elevation encountered at this site appears to be close to 114ft and the minimum elevation approximately 96ft.

With this in mind, it appears that drainage for the site can be accomplished three different ways. The first would be by laying the needed submerged piping throughout the site so that all of the water flows to the stream running through it. Conditions involving the flow capacity of the stream and whether this option is legally viable would need to be investigated. The second would involve adjusting the topography to ensure that all surface runoff is directed to the stream. The same concerns for stream drainage exist in this scenario. In addition, adjusting the topography for drainage purposes would require a basic knowledge of the current runoff characteristics of the site and those desired by

[432] USGS 25k Topographic Maps

altering the topography. In order to calculate the runoff of a particular area, the area, average intensity of rainfall and the storm water runoff from the area must be determined.[433] The final option would be to create a catch basin or detention pond for the runoff to be collected. This option would allow for gravitational settling of dense particulate matter and accommodate some remediation to take place in terms of pollution contained in the runoff should pollution considerations become an issue.

Subsurface drainage provisions would be dependent upon the soil characteristics, the water table in the area, the stability of the subgrade and its susceptibility to frost, as well as removing surface runoff.[434] Subsurface drainage is typically accomplished by creating a horizontal system that feeds into a main outlet line. There are four typical types of systems.[435] The first is the natural system, which is used in areas that do not require complete drainage. The second is the Herringbone system, which is utilized in areas where the concave surface of the land slopes in different directions. Angles in the Herringbone system should be limited to 45 degrees. The third system is the Gridiron system where drainage is set up such that lateral pipes intersect a main line at a 90-degree angle. The final system is the Interceptor, which is used near the upper edge of wet areas to drain them.

There are some special considerations that need to be taken into account when designing for this type of drainage however. Pipes running through a correctional facility have a limit on the size of their diameter specified by the American Correctional Association. Diameter of drainage pipes is specified to hinder escape and the exchange

[433] Rubenstein, Harvey M., p.180
[434] Rubenstein, Harvey M., p.188
[435] Rubenstein, Harvey M., p.189

of contraband through the lines. It would also be wise to consider submerging the stream to ensure prisoners do not attempt to harm each other or escape in it.

Wetland areas must also be accounted for in the design of any site. The proposed site is illustrated in below with symbols marking any areas that are considered wetlands by the Massachusetts Department of Environmental Protection. In order to construct a building upon an environmentally protected area, permits will need to be issued from the local conservation commission and possibly the state Department of Environmental Protection.

Figure 8.2.7 denotes that there are essentially swampy conditions all along the southeastern side of the proposed parcel of land. This finding could mean major problems in terms of soil conditions for the foundations of the builds, should they be placed on that section of the lot. Additionally, this area may be protected by the DEP and local conservation commission from construction. Knowing this, it would be wise to consult the DEP and conservation commission to determine if any special considerations need to be made in developing this site. The drainage of the area, as discussed before, becomes even more crucial knowing this bit of information.

Figure 8.2.7 Wetlands Overlay on Proposed Site[436]

Keeping the swampy area in mind, this geographical feature could be used as additional perimeter security. Perimeter security practices will be elaborated upon further in a later section, but for the purposes of this discussion, the swamp could be used as an additional buffer zone if it is cleared of brush to the point where guards can easily monitor activities that could occur in that area. The amount of vegetation removed from the area should be determined in conjunction with the DEP and conservation commission so that the swamp is preserved as a viable habitat. Fencing could be placed on the interior perimeter such that the swamp is not enclosed and on the exterior such that the swamp is enclosed. This area could serve as an added deterrent for anyone trying to escape or enter through this approach. The landscaping of this area would be crucial to its effectiveness. Vegetation should be minimized in terms of high standing brush and any kind of trees in the area.

[436] DEP Wetlands

The next major consideration for any site would be to determine the zoning restrictions of the particular location in which one wishes to build. Some towns will have zoning requirements and other towns will not. It is best to consult the local zoning board for information regarding this matter. The Commonwealth of Massachusetts does provide a website detailing some of this information. Figure 8.2.8 shows the results of the findings from the Massachusetts website in terms of zoning.

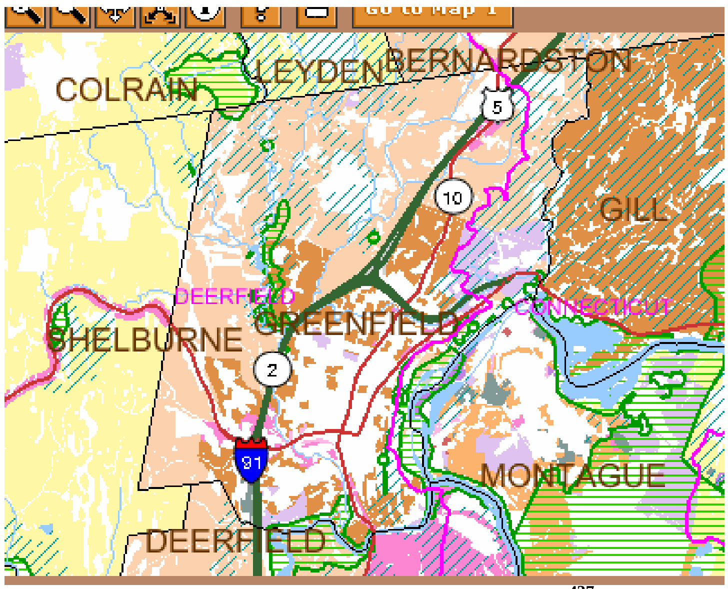

Figure 8.2.8 Zoning for Proposed Site[437]

According to the legend, the area in which the proposed site is located is zoned for light residential and a section near to the area is zoned for general business. Typically, zoning will not be a major issue should the state desire to acquire the land.

It is also important to determine whether or not the land can be developed according to the zoning restrictions. Some parcels of land are protected from development under Open Space Conservation Acts and Commissions. The following figure, Figure 8.2.9, shows the land usage requirements for the City of Greenfield.

[437] Mass. Zoning and Preservation Maps

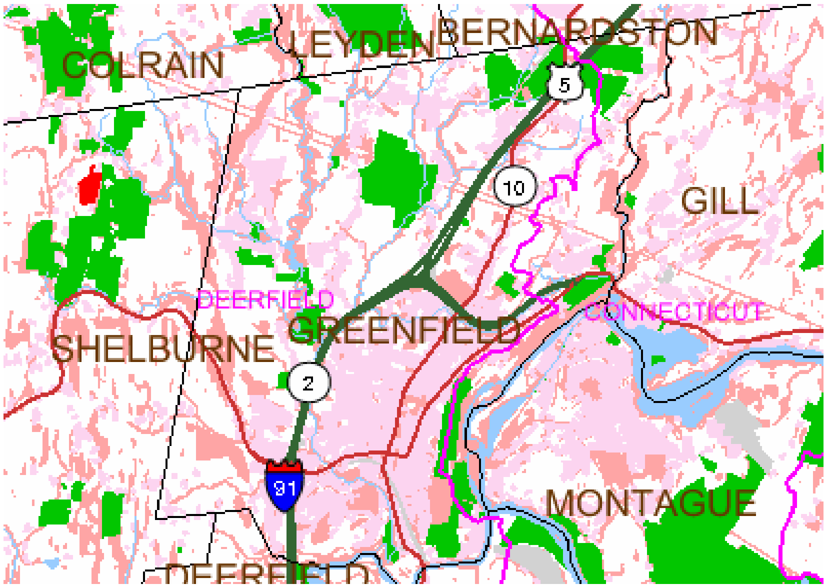

Figure 8.2.9 Open Space Map for Greenfield, Ma and Surrounding Towns[438]

It should be noted that the area for the proposed site of the prison is marked protected as open space. To determine if this land could possibly be developed it would be important to consult the local Open Space Committee. Since this location is hypothetical, the remainder of this section will proceed as if site design of this parcel of land could actually be accommodated by the governing body in charge of open space. The loss of open space in this area could be made up for the governing body in charge of the correctional facility purchasing another parcel of land and dedicating in to open space.

In regards to infrastructure serving the site, it is desirable to control access and the number of entrances and exits. It is also important to provide means to observe all those who are entering and exiting the compound and to control their movement once inside the perimeter. With this in mind, sallyports should be placed to restrict access to prisoner drop off and removal locations. Pedestrian access should also be limited to prevent contact with inmates who are outdoors in exercise yards.

[438] Mass. Zoning and Preservation Maps

Roadways entering the facility should be capable of accommodating two lanes of traffic, one entering and one exiting. According to Kevin Lynch, author of *Site Planning*, traffic lanes should be nine to ten feet wide for minor roads, with twenty feet being acceptable for a two-way minor road.[439] Due to the need to account for large tractor trailer trucks making deliveries, this road width will be widened to twenty six feet to accommodate for an increased size of the typical vehicle. Sidewalks on site should have a minimum width of four feet, and due to the location of the facility, they will only be located within the perimeter as necessary to accommodate pedestrians entering the administrative building.[440] In regards to onsite parking, each parking space should measure about nine feet by twenty feet with aisle widths for the lot measuring 12 feet for unidirectional traffic flow and 22 feet for two-way traffic flow.[441]

The utilities serving the building must also be incorporated into the design of the site. For this facility, all utilities should be located underground to avoid site conflicts with guards observing the site. Electricity, communications, sewer, and water are all major concerns. For safety and ease of access, electrical raceways and communication lines should be located in the ground under or directly adjacent to the access road to the facility. The facility should have provisions for back up generators in case of power outages. Lighting for the site should be provided in such a way that limits the obstruction to the view afforded to the guards, but maximizes the amount of illumination provided to the site. The façade of the building should be kept illuminated at all times to prevent any opportunity for unnoticed breaches in security under cover of darkness.

[439] Lynch, Kevin., p.138
[440] Lynch, Kevin., p.139
[441] Lynch, Kevin., p.144

With all of these factors in mind, it now becomes important to address the options available in terms of perimeter security. This topic is presented in the following section. The following figures depict the position of the correctional facility on the site selected as well as perimeter security considerations and access.

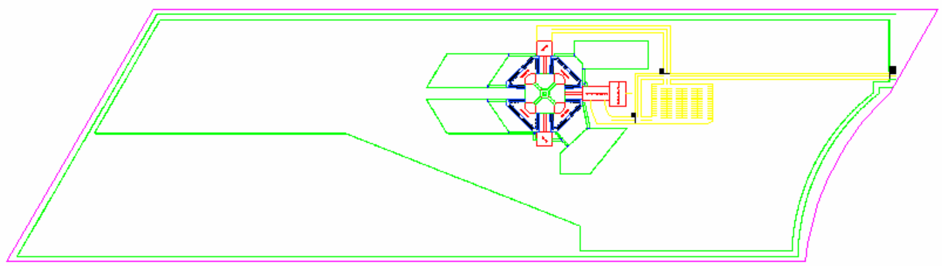

Figure 8.2.10 Overall Site Layout

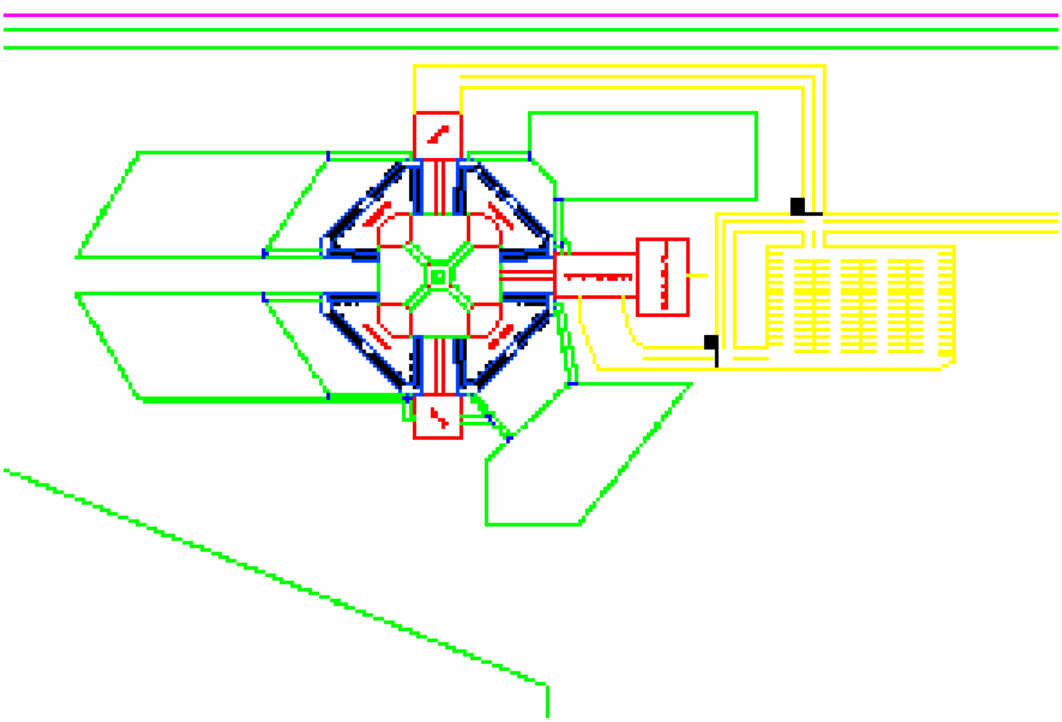

Figure 8.2.11 Close Up of Correctional Facility Site Design

8.3 Perimeter Security Considerations

The typical considerations for fencing materials are presented in the section 2.7 Security Considerations. This section provides a detailed description of other perimeter security options that can be used in conjunction with fencing to prevent unwanted admittance to the facility and to keep the inmates securely enclosed in the compound.

A perimeter detection system must balance the need for sensitivity and the goal of eliminating false alarms. The issue with false alarms is that the guards will become less attentive to the alarms if they are being set off frequently as the result of non-security related events. The greater the sensitivity the more likely detection will occur, but in turn so increases the probability of false alarm.[442] Typical perimeter security system design will incorporate a number of different elements to achieve the desired level of expectations when implemented. Keys to successful electronic detections are summarized below.[443]

- Selection of an appropriate technology to support the site constraints and facility objectives.
- Assurance that the system is installed properly and in conformance with the system manufacture's recommendations.
- Thorough system testing to ensure system performance supports the design objectives.
- Routine testing of the system to verify system performance.
- A maintenance program to ensure that the system is maintained in order to perform according to initial acceptance standards.

The first type of perimeter security device to be presented is the taut wire system. This system consists of a series of wires under tension that are connected to sensors.[444] The wires are placed such that an intruder or an escapee must move them in order to

[442] Krasnow, Peter, p.322
[443] Krasnow, Peter, p.322
[444] Krasnow, Peter, p.322

escape. When the wires are moved this triggers the sensors, which in turn alerts the guards. The wires can be used as a stand-alone security option or woven into more typical wire mesh fencing. The taut wire system tends to conform well to site topography and can be installed anywhere a fence can be installed. Height of the sensors is dictated by the number of wires and the spacing. This system has an extremely low nuisance and false alarm rate. One drawback is that the detection zones must be linear. In addition, the cost of maintenance is relatively high and can be influenced by adverse weather conditions. The cost of this type of system is typically $150 to $180 per foot of length.

The next type of system presented is the microwave system. It provides detection by changes in radio frequency across a prescribed field.[445] The topography of the area in which this system is used must be stringently controlled to ensure detection reliability, which can be thrown off by shadow effects. The horizontal beam width of the signal must be controlled to limit the width of the detection zone. Vertical detection is controlled by antenna patterns in the transmitter and receiver. These systems are subject to a high rate of false alarms and nuisance from blowing debris, deep snow, high-level electromagnetic interference, small animals, water flowing across the detection zone, and uncontrolled vegetation.[446] Installation is relatively simple as long as a stable mounting post is provided. The cost of this system is typically $5,000 per detection zone.

Seismic geophones are also used in perimeter security applications for correctional facilities.[447] These systems process signals generated by movement of sensors to which the geophone is attached. The major advantage to this type of system is that the geophone will only translate vibrations in the axis of its sense coil. This allows

[445] Krasnow, Peter, p.323
[446] Krasnow, Peter, p.234
[447] Krasnow, Peter, p.324

for a reduction of nuisances and false alarms. Geophones can be buried in the ground or mounted on fencing. The fence to which the geophone is mounted must be sufficiently rigid to decrease the number of false alarms from natural fence movement. The detectors are also less susceptible to weather-related false alarms than the previously discussed systems. Geophones must conform to either the site topography (if buried) or the fence line. It is also important to take into account the frost line and type of soil in which a geophone is buried. The major type of nuisance or alarm occurring from this system is the result of casual fence movement, high winds pushing the fence and other impacts. Installation is relatively easy in the case of fence-mounted geophones. Buried geophones are more difficult to install as the soil the in which the geophone is located must be one in which it can function properly. The cost of the geophone perimeter security option is approximately $45 per foot.

A fourth type of perimeter security system is a cable based detection system. There are two major types of systems, the first of which is the electret cable system. This system uses microphonic cables to detect noise generated along the fence.[448] This system consists of a sensitized cable attached to a fence using cable ties. The fence must be taut in order for this system to work most effectively. The detection zone follows the fence line and is independent of topography and straight-line sight zones. These systems are susceptible to nuisance and false alarms from heavy rain, hail, sleet, and fence movement. The sensitivity of the processors for the system also determines the frequency of false alarms and the nuisance factor. The installation of this type of detection system is much simpler than that of a buried system or one that requires beam alignment.

[448] Krasnow, Peter, p.326

Electromagnetic cables are the second type of cable system for perimeter security for correctional facilities. These systems operate as strain sensors in the cable configuration that act upon a linear transducer.[449] It operates in a manner consistent with that of the discrete geophone. The cable system detects the movement of the fence and sends signals that trigger alarms. As with the electret cable system, the fence must be taut for proper operation. The detection zone is not constrained by topography or straight-line zone segments, which is also similar to the way that the electret cable system operates. In addition, the same climatic conditions have an adverse affect on electromagnetic cable detection system. Installation and cost of the two systems is also similar at roughly thirty-five dollar per foot of cable.

Post-mounted electromagnetic lines detect disturbances by sensing imbalances in electromagnetic fields.[450] The fields for this type of system are created by using a transmit line and sensing field lines. This type of system can be mounted on isolated posts or incorporated in fencing. This system follows the topography of the site, and zones can deviate from a straight line. False alarms and nuisances are typically the result of high winds, electromagnetic interference, threshold settings and blowing debris. Installation of this type of system is relatively easy and revolves around providing stable mounting posts and keeping lines taut. The cost of this type of perimeter detection system is typically $35 per foot of length.

Infrared sensors have also found a place in the world of perimeter security systems. These systems operate by detecting infrared signals and measuring their

[449] Krasnow, Peter, p.326
[450] Krasnow, Peter, p.327

variation from an established norm.[451] The detection is measured from a transmitter to a receiver, both of which are often post mounted. The topography of the area between the transmitter and receiver must be controlled. Nuisance and false alarm signals are usually the result of blowing debris, small animals, heavy fog snow and direct sunlight on the lenses. The cost per foot of this system is typically on the order of $60 per square foot.

Video monitoring systems for correctional facilities are quite complex, but provide a good measure of security. These systems operate by using signals to determine if there is an intruder.[452] If the alarm is tripped, video of that area is displayed. Video detection zones must overlap in order to be effective. The area in which video detection systems are employed must be illuminated either constantly or by sensors reacting to the video detection system activating. The sensitivity of the detection system should be such that birds and small animals do not cause false alarms. Mounting poles should accommodate sight lines needed by guards. The cost of this system depends on the number of cameras and their proximity to signal processors and the central monitoring bank. The cost of this type of system is approximately $75 per square foot.

The final major type of perimeter security system is the electric fence. Electric fences are typically composed of electrically charged wires running from one post to the next.[453] These types of fences are placed between the interior and exterior fences. The voltage traveling through the wires can be set from levels that are lethal to those that will simply stun. This system is dictated by the topography of the land and can be installed anywhere a fence can be installed. The height of the fencing can be variable and nuisances can occur in deep snow conditions, small animals, and blowing debris. Care

[451] Krasnow, Peter, p.328
[452] Krasnow, Peter, p.329
[453] Krasnow, Peter, p.240

must be taken to make sure that small animals and bird are not electrocuted. In additions, considerations must be made in regards to addressing the death of an inmate should the fence be set to a lethal level.

Table 8.3.1 summarizes the various perimeter security options, base costs and costs for this particular location and design. It should be noted that options are often combined to optimize security and economics.

Table 8.3.1 Perimeter Security Option and Cost Summary

Security System	Cost	Total Cost	
Taut Wire	$150 to $180/linear foot	$1,146,000.00	$1,375,200.00
Microwave System	$5000 per zone	$722,000.00	
Geophonic System	$45 per linear foot	$343,800.00	
Cable Based Systems	$35 per linear foot	$267,400.00	
Infrared Systems	$65 per square foot	$144,403,460.00	
Video monitoring	$75 per square foot	$10,830,000.00	

8.4 Site Design Conclusions

Site design for a correctional facility involves a host of considerations that need to be taken into account. These factors range from ecological to technological to accessibility. Care should be taken so that the facility is designed in such a way that it fits the local environment and provides adequate security to the surrounding community. When a site is designed to accommodate a correctional facility, the perimeter security is of primary concern. It is also extremely important to know the conditions presented by the site in terms of soil, topography, and water table characteristics. In the end, the site should provide secure access for staff, administrators and visitors while keeping the prisoners detained and accommodating environmental conditions.

9 Conclusions

Due to the current state of overcrowding in correctional facilities across the United States, the need for more prisons to adequately house the nation's criminals becomes essential. Overcrowding can lead to the early release and poor rehabilitation of inmates, impacting the safety and wellbeing of the community. An economical solution to the overcrowding problem is driven by the construction of future correctional facilities. This project uses parametric analysis to address the issues of economic design by investigating various layouts and materials for structural design.

In addition to providing a challenge to produce an economical design solution for a correctional facility, this project allowed for creativity in design. Breaking from the realm of typical office or commercial construction, the design of a prison required considerations for components specific to a correctional facility. These components include:

- Security
- Housing
- Inmate services
- Administrative functions

The development of the facility layouts required integration of these factors. With security in mind, consideration for guard placement and access points can greatly affect the operating cost of the facility. Concerns for housing depend on providing adequate space for each inmate, while allowing potential for future expansion. Inmate services must be centrally located, giving access to all cellblocks, while maintaining a high level of security. Administrative functions are critical for a facility to operate effectively, thus adequate space must be provided so that executive tasks can be performed.

With considerations for security and inmate housing, a triangular shaped prison cellblock was developed. Placing the cells along the perimeter of the cellblock allowed for:

- Centralized security post

- High security visibility
- Large central dayroom
- Daylight within the cell

The triangular shape accommodated cellblocks to be arrayed around a central prison core. The four cellblocks provided the opportunity to isolate security levels within an integrated structure.

The prison core permitted the use of a concentric access point for all of the cellblocks. Designing in this manner enabled the creation of a high level of security by minimizing the number of entrances to the cellblocks. Effective placement of the cellblocks around the prison core provided open space, allowing for recreational areas.

In the case of the administrative building, the structure was designed to be separate from the prison superstructure. By separating the administrative operations, it can function independently of the status of the prison. For example, if a breach of security were to occur within the prison, the executive oversight of the facility could continue without interference. In determining the layout, space was allotted for the following uses:

- Administrative operations
- Guard training
- On-call guard housing

Materials of construction were based on the necessity for fire resistance and the ability to span long distances. Therefore, steel and reinforced concrete were used in the design. A comparison was conducted between the two materials based on their performance according to gravity design.

The gravity systems were developed based on loads prescribed by the *2003 International Building Code*. Multiple layouts were proposed for each material type for both the office and prison structures. The types of systems investigated for steel construction included:

- Non-composite WF-floor deck
- Composite WF-shape floor deck
- Composite Open-web Joist floor deck

All steel scenarios were designed with WF-shape girders and columns. Reinforced concrete options included:

- One-way slab design
- T-beam design
- Rectangular beam and girder design
- Square column design

Evaluations of the gravity systems were conducted based on:

- Cost
- Constructability
- Serviceability

Cost was considered because of the issue of economical design discussed earlier. Constructability has major implications on the timeliness and success of a project. Types of connections, number of members, and materials used are all constructability considerations that were examined in the evaluation. Serviceability encompasses the functionality of the structure and the quality at which it operates. Criteria such as visibility and versatility ultimately affect the cost and safety at which the correctional facility functions.

These evaluations were conducted to determine a satisficing design, which is a solution alternative that best satisfies a number of objectives. For academic purposes, although one design scenario was selected to be sufficient for the office, both reinforced concrete and steel systems were further developed. In terms of steel design, composite open-web joists with WF-shape girders and columns proved to be the highest ranked option investigated.

As a result of the long spans required in the prison cellblock, steel proved to be superior because of its lightweight construction relative to concrete. Of the two steel floors systems, composite open-web joists provided the highest ranking evaluation because of their ability to carry large loads over long spans. Since the open-web joists were sufficient for the prison cellblocks the framing system was also applied to the core of the prison.

For the next stage of design, lateral loads were applied to the structural systems chosen in the evaluations. Reinforced concrete systems transferred lateral loads through moment frames, while steel systems utilized braced frames to resist lateral loads and minimize sway. Both concrete and steel member sizes were adjusted accordingly to accommodate additional stresses due to lateral loads.

Following the design of lateral systems a revised cost evaluation was conducted for each scenario. The cost estimates were based on the application of unit cost data obtained from the RSMeans series of cost estimation books to the structural elements of the prison and administrative buildings.

These elements included:

- Floor systems
- Beams/joists
- Girders
- Columns
- Braces

Table 9.1 provides a summary of the cost per square foot for each building designed.

Table 9.1 Summary of Structural Cost

		Material	# Substructures	Cost	Total Cost	Square Footage	$/Sq.ft.
Office		Steel	1	$619,000	**$619,000**	44,800	**$13.82**
		Concrete	1	$488,000	**$488,000**	44,800	**$10.89**
Prison	Cellblock	Steel	4	$459,000	$1,836,000	179,200	$10.25
	Core	Steel	1	$1,257,000	$1,257,000	63,405	$19.82
				Total:	**$3,093,000**	242,605	**$12.75**

While the primary goal of this project was to produce a structural design for a correctional facility, other components of structural considerations were researched. By investigating these additional areas, the scope of the project was broadened beyond the analysis and proportioning of structural members. These supplementary areas of study included:

- Foundation considerations
- Exterior wall design
- Fire considerations
- Site design

Foundations were researched in order to develop an understanding of the connections between the structural frame and the earth. There are two categories of foundations, which include deep and shallow. Shallow foundations are the most cost effective solution, however deep foundations may be necessary in some highly loaded regions within the facility.

Exterior wall design was considered because of the importance of security and confinement within a correctional facility. Various wall systems were examined and a functional design was developed for both the office and the prison superstructure. For the office a brick veneer over a steel stud backer wall was designed, while a fully grouted reinforced concrete masonry unit (CMU) was developed for the prison.

Additionally, the effects of fire conditions on a structure can play a critical role in the safety and performance of a building. Because of the detrimental effects of fire on materials and structural members, some form of protection is generally required. According to the provisions of the 2003 IBC correctional facilities necessitate the use of noncombustible materials that include steel and concrete. While these materials are fire resistant they are still susceptible to fire damage. Therefore, structural components of the correctional facility require the additional installation of fire resistant insulation and automatic sprinkler systems.

Finally site design was considered. Considerations for correctional facility site design were elaborated upon including perimeter security options, drainage, and environmental and zoning considerations. Site specific considerations were developed for a parcel of land in Greenfield, MA, and a preliminary site plan was created detailing access, perimeter fencing, and the layout of the site as a whole. The driving factor behind the design was to provide the maximum amount of security for the correctional facility compound.

In the process of completing this project, it was learned that there are many considerations that need to be taken into account in the development of a correctional facility. Many of these considerations coincided with required capstone criteria of a Major Qualifying Project.

The capstone criteria met in this project include:

- Social
- Ethical
- Health and Safety
- Manufacturability (Constructability)
- Economics

One of these considerations is the social impact the design and placement of a correctional facility has upon the local community. Correctional facilities carry a negative stigma, and many people feel that having one in the community will somehow reduce the volume and quality of life in the area. Others feel that locating a prison within a community will boost the economy and provide jobs to locals. In the process of completing this project it was learned that the impact of a correctional facility upon the community reached neither extreme. Communities where prisons were placed had neither an increased crime rate nor a boost in its economy.

In regards to ethics, the quality of conditions and the humane treatment of prisoners are essential in the design of the prison. Space must be allocated to activities such as counseling, rehabilitation, education, vocational training, recreation and entertainment. With layout considerations for these elements, the quality of living of the prisoners may be raised to a level where humane treatment of the prisoners is attainable.

Through the development of the various designs, health and safety were accounted for via security and allocation of medical services. In addition, appropriate layouts, design procedures, and fire considerations dictated the manner in which the project evolved. Appropriate layouts were governed by the need to provide adequate egress and the proper amount of space for the usage of the structures. The design procedures utilized contained safety factors to ensure that the buildings would be capable of withstanding the loadings to which they would be subjected without failure. Fire considerations overlapped slightly with the design considerations for layouts in terms of egress and provided added measures of safety should a fire event occur.

The evaluations in determining the most appropriate options for gravity design relied upon assessment of constructability in conjunction with economic factors. In regards to the constructability, the number of members needing to be assembled, the amount of formwork, and the size of the elements were taken into consideration. These metrics enabled us to quantify the relative benefits and drawbacks of different schemes and compare different design options.

The major focus of this project was to create an economically feasible design for the construction of the correctional facility. The overall cost of construction often dictates whether or not a facility can be built and what type of structure is erected. The layouts and materials used for the construction of different facilities can provide options for value engineering and the reduction of overall cost by design of various options. Economics was a key element in the selection of the final design options.

Further development of this project can continue along many different veins. One of the major elements that can be developed is the design of the service buildings and the inmate intake and release building. Subsurface corridors could also be designed in such a manner that the service buildings can be accessed from the prison core. In addition, more variance in the basic designs and layouts of the prison and administrative tower could be explored to determine if value engineering is possible. The use of other building materials such as pre-stressed concrete and pre-cast cell modules could also be explored for construction of the prison. Finally, the special considerations (i.e., foundations, exterior enclosures, fire safety, and site design) could be further developed into their own project or as subsets of a larger project centering on these considerations.

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Appendix

A Proposal

Correctional Facility Analysis and Design

A Major Qualifying Project Report Proposal:

Submitted to the Faculty

of the

Worcester Polytechnic Institute

In partial fulfillment of the requirements for the

Bachelor of Science Degree

By

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October 31, 2005

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Introduction

Throughout the course of human history, there have always been those who, whether by inner compulsion or extenuating circumstances, break the rules of society. When these infractions are severe or occur numerous times, it becomes necessary to separate those who cannot or will not function within the bounds of society from those who can. These people who exhibit deviant behavior are contained within what are called correctional facilities, or more commonly prisons or jails.

A correctional facility is a compound designed with five major purposes in mind.[454] The first is punishment, with the prison itself intended to "'cure' the former (prisoner) of, and frighten the latter (general public) from, criminal behavior."[1] The second purpose is protecting the general public from those who would wish to do harm. Third is the aspect of reform of the prisoner. This means that the building or buildings that make up the complex have space allocated for education, solitary confinement, social training and the like. The fourth purpose is to attempt to cure the prisoner in terms of mental afflictions that may drive his or her deviant behavior through therapy or group activities. The final purpose a correctional facility can serve is to provide "a public statement of moral, political, and social virtue."[1] Thus, institutional characteristics play a critical role in creating an environment of incarceration and rehabilitation without causing a sense of oppression and hopelessness.

These five purposes of incarceration can be met by a variety of different layouts depending upon the type of prisoner who is being held and where the facility is located. There are five different security levels for prisons.[455] The first is the minimum security prison, which is also known at the Federal Prison Camp (FPC). This type of prison features dormitory style housing and tends to be located close to higher security prisons and can provide the labor needed to run those facilities. The second type of prison is the low security prison. These types of facilities also feature dormitory style housing or cubicle housing and have strong work related components. Medium security prisons have cell type housing, increased security measures and greater internal controls. High security prisons have single and multiple cell housing and extremely controlled internal

[454] Spens, p. 16
[455] Federal Bureau of Prisons – General Information

environments. The fifth type of correctional facility melds the various levels of classification in a single complex. Prisons can be located in any type of setting, whether it be urban or rural. The main difference between the two localities is the design of the structure. Due to spatial constraints, urban correctional facilities tend to have more of a high rise design whereas rural correctional facilities can be more sprawling.

Over the past 20 years, the population of prisoners housed in correctional facilities has increased dramatically. Due to this population increase, a multitude of prisons on the county, state and federal level are facing overcrowding issues. In fact, between the years of 1995 and 2002, state prison populations increased 23.6 percent and federal prison populations by 71 percent.[456] Additionally, it has been estimated that between 1 to 16 percent of state prisons are over capacity and 33 percent of federal prisons are in the same condition.[3]

As a result of overcrowding, correctional facilities across the country are feeling increased strain on their infrastructure. The pressures of overcrowding affect each facility's ability to operate effectively. With each additional inmate beyond the planned capacity, resources such as food services, education, and rehabilitation, as well as cleanliness and health concerns are compromised. Because of overcrowding and its effects, the goals of the correctional facility to rehabilitate inmates are less likely to be met.

Research has stated that:

> The prison environment is characterized by factors which can have adverse effects on individual inmates. In the prison setting crowded conditions are chronic, people prone to anti-social behavior are gathered, there is an absence of personal control and idleness and boredom can be prevalent. Research has indicated that overcrowding has three types of effects on the daily prison environment. First, there is less of everything to go around, so the same space and resources are made to stretch even further. The opportunities for academic, employment and vocational training are curtailed. The lack of work or work opportunities lead to idleness, often reinforcing the maxim that idleness breeds discontent and disruptive behavior (Cox, Paulus, & McCain, 1984, p.1149). In addition, lack of resources can apply to anything an inmate might need to use, such as washroom availability, library books, television lounge seating and recreational materials. The

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[456] Gibeaut, p. 53

> unavailability of resources can have two-fold consequences. One is the frustration of unpleasantness of being limited or denied a resource and the other is the fact that competition or conflict over limited resources often lead to aggression and violence (Johnson, 1991, p. 19).[457]

Scope

With these facts in mind, the following MQP proposes to complete the structural design of a correctional facility. In conjunction with designing the structures that comprise the complex, the role of building codes, design alternatives, constructability and cost estimates of the buildings will be completed based on various design layouts and construction materials. The general thought processes for considerations of design of this facility are displayed graphically at the end of this proposal in Figures 1-6.

The complex that is being considered for design is made up of five buildings: an office tower, the main detention structure composed of four cellblocks and a central core, then three buildings connected by subterranean corridor to the main detention structure. The design of the main detention structure was gleaned from an example schematic in Correctional Facility Design and Detailing. The proposed layout was chosen because it defies the traditional concept of how a correctional facility appears. Typical prisons use separated buildings to house prisoners of different security levels which in turn leads to the need for an increased security detail which affects operating costs. In addition, housing prisoners in separate buildings increases operating costs related to food transport, medical treatment, and education services. Our proposed layout places all levels of prisoner security around one central core, thereby reducing the number of security guards and other personal needed to address the welfare of the inmates. This proposed layout was chosen primarily because it effectively utilizes security placement, which ultimately can reduced operating costs over the life span of the facility.

The buildings that will be connected via subterranean corridors will primarily house prisoner intake and release, medical areas, and food services in a more secure manner than they would were they incorporated into the main detention structure. The primary type of correctional facility will be a combination medium – high security prison.

Correctional facilities require many services to fulfill typical correctional facility

[457] Prison Overcrowding

standards which are governed by the American Correctional Association (ACA). These services include education, food, health, recreation, rehabilitation, and training. Research will be conducted to identify what these services are and the corresponding facilities needed to support them. Additionally, research will be done to determine what facilities are needed for the administration building to adequately service the inmate intake/release, probation, and visitation process. Building codes will have a profound affect on the overall design. Research on these codes will help in providing a background on what the minimum requirements are for construction of a safe facility that can be occupied legally. Topics such as fire safety codes and handicap accessibility will have an affect on the layout of the complex and thus must be given careful attention in the design process especially when accounting for means of egress.

The chosen location for the correctional facility is in Massachusetts, and it will be designed to house a single occupancy of 512 inmates or a possibility of 1,024 inmates if double occupancy is considered. Based on research, the optimum prison population is between 400 and 500 inmates to allow for the most economical operational and management costs.[458]

Materials that are being considered for the construction of this facility are structural steel and reinforced concrete. In regards to steel construction, floor framing using rolled steel beams and open web joists will both be considered with sample connection detailed for each designed. In terms of decking for the steel structures, composite decking and one-way reinforced concrete slabs will be considered with the possibility of exploring the usage of metal decking. Reinforced concrete frame design will investigate the usage of rectangular beams versus T beams, with the possibility of two-way slabs placed directly on columns (for the office and inmate service buildings) and concrete joist framing being investigated. In addition, construction using pre-stressed concrete could be explored utilizing the PCI Handbook.

These different materials will first be investigated using the dead and live loads that they are to support in to determine which materials make the most logical sense to use in construction. This determination will be based in regards to how the materials compare dimensionally, cost wise, and if they are feasible to construct. The cost analysis

[458] Federal Prison Construction, Alternative Approaches

will take into account the materials and labor involved for each scenario. Once a material or materials has been selected for each layout, the structural members will be investigated further to determine if they will be able to withstand the lateral loading encountered in Massachusetts from wind and seismic forces. After the lateral loads have been accounted for in the design, a second round of cost and constructability analysis will take place. This will lead to recommendations of which materials, layouts and member designs are most desirable in terms of constructability and cost effectiveness in terms of general material cost and cost of construction.

In terms of construction methods, the placement of materials and the methods of securing the members together will be investigated. In terms of the steel design, different types of connections will be explored in terms of strength and ease of assembly and fabrication. In terms of reinforced concrete, formwork, method of concrete placement and rebar fabrication and layout will be investigated. In addition to these considerations, the construction methods investigated will also consider the ease of constructability of different layouts for each material being considered.

Beyond the design of the primary structure, analysis will be performed to determine exterior wall systems function best for this facility and how they interface with the structural frame. This will include materials such as concrete masonry units and structural studded steel, both incorporated with a brick façade. Other façade options include the investigation of the properties of concrete tilt up panels and glass curtain walls (for the office only). This portion of the design process will also investigate which system is most appropriate for the type of building it is being applied to in terms of security, constructability and cost efficiency.

Special areas of study on this project will most likely involve an in-depth analysis of fire protection for a building of the size of the main detention facility and the outlying buildings. In addition, special attention will be given to the design of the foundations of these structures and the tunnels used to connect them. A final topic of special investigation will be site work for the complex including drainage and perimeter fencing options.

Capstone Criteria

In completing the scope of work surrounding the construction of correctional facilities, it is our hope to meet several of the capstone criteria. These criteria are namely design, economics, constructability, health and safety concerns and social implications.

The purpose of design on this project is to expose the participants to a more global method of regarding the scope surrounding the design of a facility. This project will provide awareness of the design process from conceptual layouts to finalized schematic designs. In so doing, the participants will become more proficient at not only purely designing structural members, but making educated decisions on which type of member is most appropriate for different scenarios in terms of material, loading, constructability and cost effectiveness.

The second criterion of economics will be satisfied by the cost analysis of the various scenarios and materials used to construct them. Economics play a lead role in the construction of a correctional facility due to the fact that correctional facilities are funded by governing bodies. Taxpayers like to see their money being spent in an efficient manner, thus the need for an economically feasible design that still meets code requirements and the needs of the incarcerated. The investigation of different column spacing and beam sizing will play a large role in the cost analysis of the structures designed in this project.

The consideration of the constructability of our various designs will be done to satisfy the criterion of manufacturability. Constructability can be regarded in terms of material properties and how they apply to the design of different members. In addition, the constructability of a member is concerned with the cross-sectional dimensions and how the member interfaces with other members and the rest of the elements of the building. Inherent qualities in certain materials lend them to different applications. Steel is a lightweight design option that provides an option for spanning greater lengths, while reinforced concrete is more efficient and less labor intensive at shorter spans. Steel is more beneficial for the long spans seen throughout this design, but the fabrication and construction considerations for steel must be considered when regarding cost of construction. In addition, the ease of formwork set up for reinforced concrete plays a role in the construction cost. Pre-stressed concrete is also a factor that could play a role in constructability of this facility with its material properties being able to span longer distances than reinforced concrete but its fabrication needs being more involved.

The topic of ethics plays a significant role in the design of a prison in terms of cruel and unusual punishment, location in respect to other parts of the surrounding community, and services provided to prisoners. In terms of cruel and unusual punishment, cell sizing must be considered along with the provision for natural light sources. The services provided to the prisoners also play a role in determining the layout of the correctional facility and the number of buildings that it constitutes.

Health and safety concerns will be addressed by designing the buildings in the complex such that they meet International Building Code provisions and take into account various life safety codes, such as fire codes. In terms of social aspects to be addressed by the scope of this work, the rehabilitation of prisoners as opposed to merely confining them in cells has an effect on what happens when those incarcerated are released back into the general public. Additionally, placement of the correctional facility is a significant concern of the general public because most citizens do not want a prison located in their back yard. These social concerns become intertwined with issues facing a correctional facility from the political forum. Political issues that will affect our scope of work include the type of prison we are designing (whether it be county, state, or federal) and where the funding for each comes from and whether correctional facilities are readily willing to transfer prisoners from one type of facility to the other to alleviate overcrowding. In addition to these considerations, the method of land acquisition and location of correctional facility in regards to the rest of the community could be a major source of political turmoil in respect to this type of project. There are also the feelings of the constituency on the issues of overcrowding and general prisoner treatment that dictates what happens politically.

Deliverables

The end result of this investigation will take several forms. One major portion of the deliverable will consist of schematic drawings of feasible layouts and engineering sketches of all calculated members and connections. This will provide pictorial representation of the written report, which will document the design and analysis process as well as the logic used to determine appropriate options for each scenario. Example calculations will further the depth of detail by presenting numerical examples and sketches, describing the process behind the explanations in the written report. Example spreadsheets will also be provided to illustrate application of some of the tools developed by the project group. There will also be a section describing how to utilize the spreadsheets should an outside party wish to use them for his or her own purposes. Finally, there will be a section detailing the design of the walls and how calculations were made to determine which systems would be most appropriate for our buildings.

Schedule

In terms of a schedule of work for this project the group has chosen to pursue the following path:

Week 1 – Determine a topic to pursue

Week 2 – Investigate the feasibility of the topic and being researching

Week 3 – Begin designing layouts for the detention facility and the administrative tower. Continue researching codes and other issues associated with the construction of a correctional facility.

Weeks 4 – Structural design for gravity loadings of the structures created in the layout design phase (spreadsheets and hand calculations). Continue researching topics surrounding correctional facilities.

Week 5 – Outline proposal topics. Continue structural design for gravity loadings. Continue researching correctional facility topics. Preliminary cost analysis of various options to identify options that are simply not feasible based on constructability and to assess the difference in materials in terms of construction cost.

Week 6 – Begin writing proposal. Continue gravity loading design, cost analysis, and research.

Week 7 – Finalize proposal and initial reference list. Continue gravity loading design, cost analysis, and research.

Week 8 – Initial investigation of exterior wall design and materials that could possibly be considered for said design. Continue gravity loading design, cost analysis and research. Begin writing introduction to final MQP.

Week 9 – Begin design with lateral loading considerations. Begin to finalize gravity load designs and assess if modifications will need to be made to withstand lateral loadings. Continue writing introduction of MQP and begin rough draft of background. Continue researching and wall design.

Week 10 – Continue lateral loading analysis. Continue writing, research, and exterior wall design. Begin connection design.

Week 11 – Begin investigating life safety system issues and codes. Continue lateral loading analysis, writing, research and wall design. Begin investigating more complex options such as two way reinforced concrete slabs.

Week 12 – Begin preliminary methodology write up. Continue life safety system design and evaluation, lateral loading analysis, writing introduction, background, wall design and complex options.

Week 13 – Begin final cost analysis of completed designs. Begin initial foundation design for the various footprints. Continue life safety system design and evaluation, writing methodology, wall design, and complex options. Start to finalize the introduction and background of the report.

Week 14 – Continue foundation design and begin to finalize wall design and life safety systems. Also continue writing methodology and begin consideration of results of structural analysis in terms of cost and constructability. Begin working on special studies and individual interests.

Week 15 – Begin in depth cost analysis of finalized structural designs to determine which are the most feasible and where the major differences lie in cost in relation to design and material. Continue complex option analysis and begin to finalize said analysis. Continue foundation design and writing final report. Continue working with special studies and individual interests.

Week 16 – Continue cost analysis and finalize complex designs. Continue foundation design and writing final report. Finish working with special studies and interests.

Week 17 – Begin organizing finalized parts of the deliverables: namely spreadsheets and example calculations. Begin drafting finalized options. Continue writing final report focusing on the cost analysis discussion and feasibility of different designs. Begin to finalize foundation design.

Week 18 – Begin analyzing the cost of the foundations for each building. Continue drafting final layouts, members and connections. Continue writing final report and revising where necessary. If necessary, begin creating an exterior wall primer.

Week 19 – Revisions of written report and drafting. Begin to tie up any loose ends and evaluate where the project stands in terms of completion.

Week 20 - More writing and revisions. Identify any issues left in the project and address them as promptly as possible.

Week 21 – Final draft. All revisions made and all deliverable finalized and corrected. MQP due.

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Figure 1 – Overall Scope Considerations for Correctional Facility Design

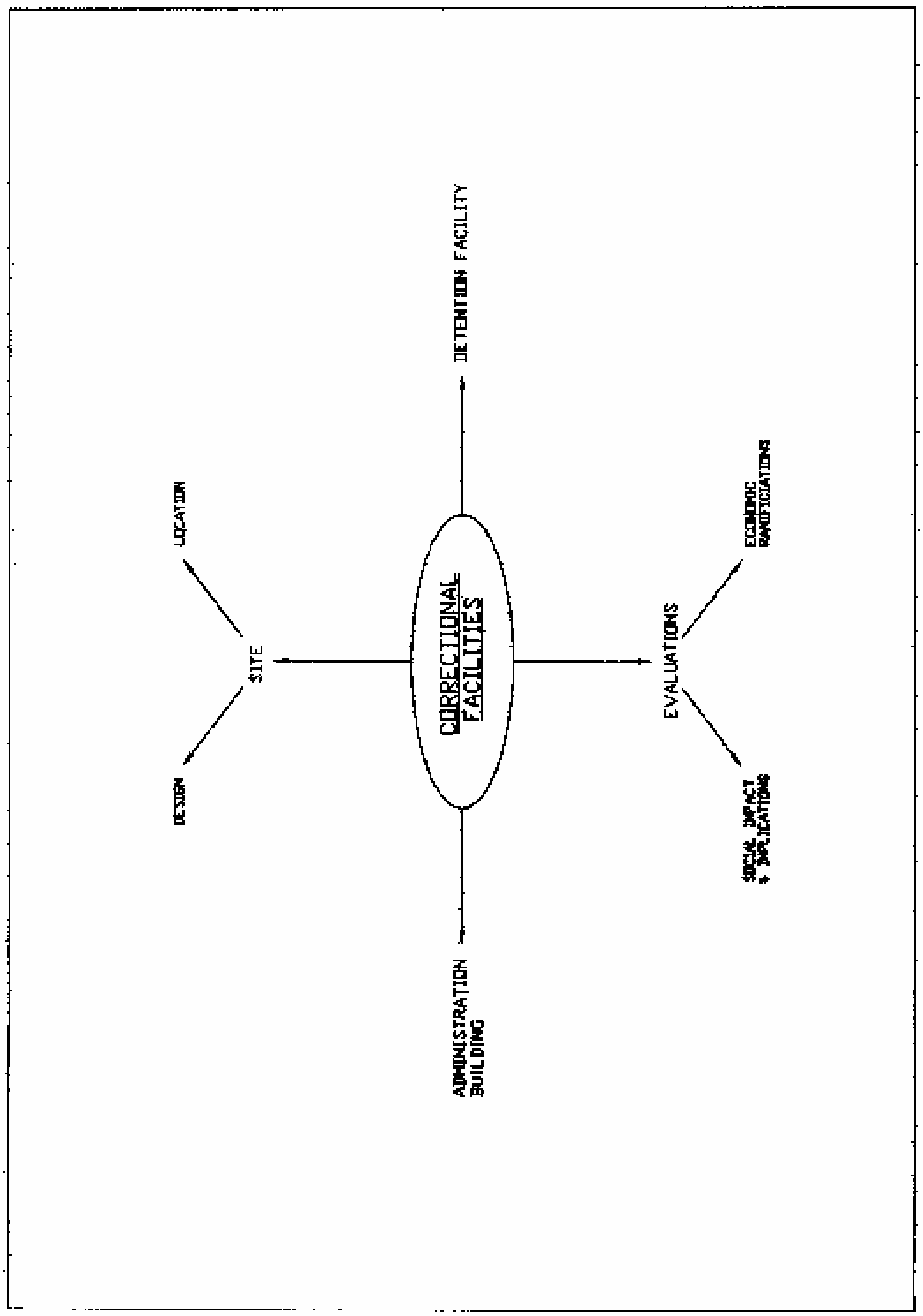


Figure 2 – Administrative Tower and Inmate Services Considerations

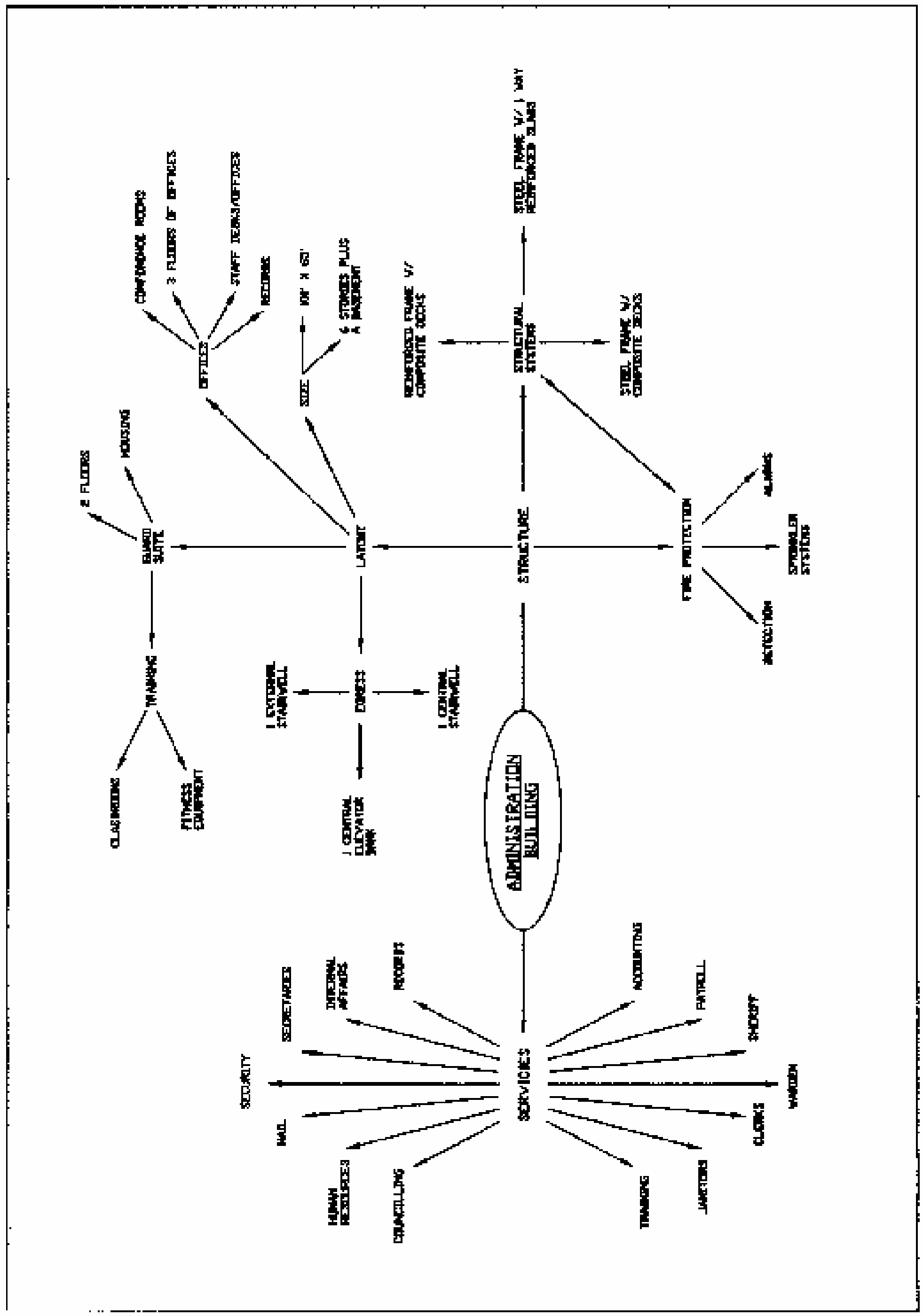


Figure 3 – Detention Facility Considerations

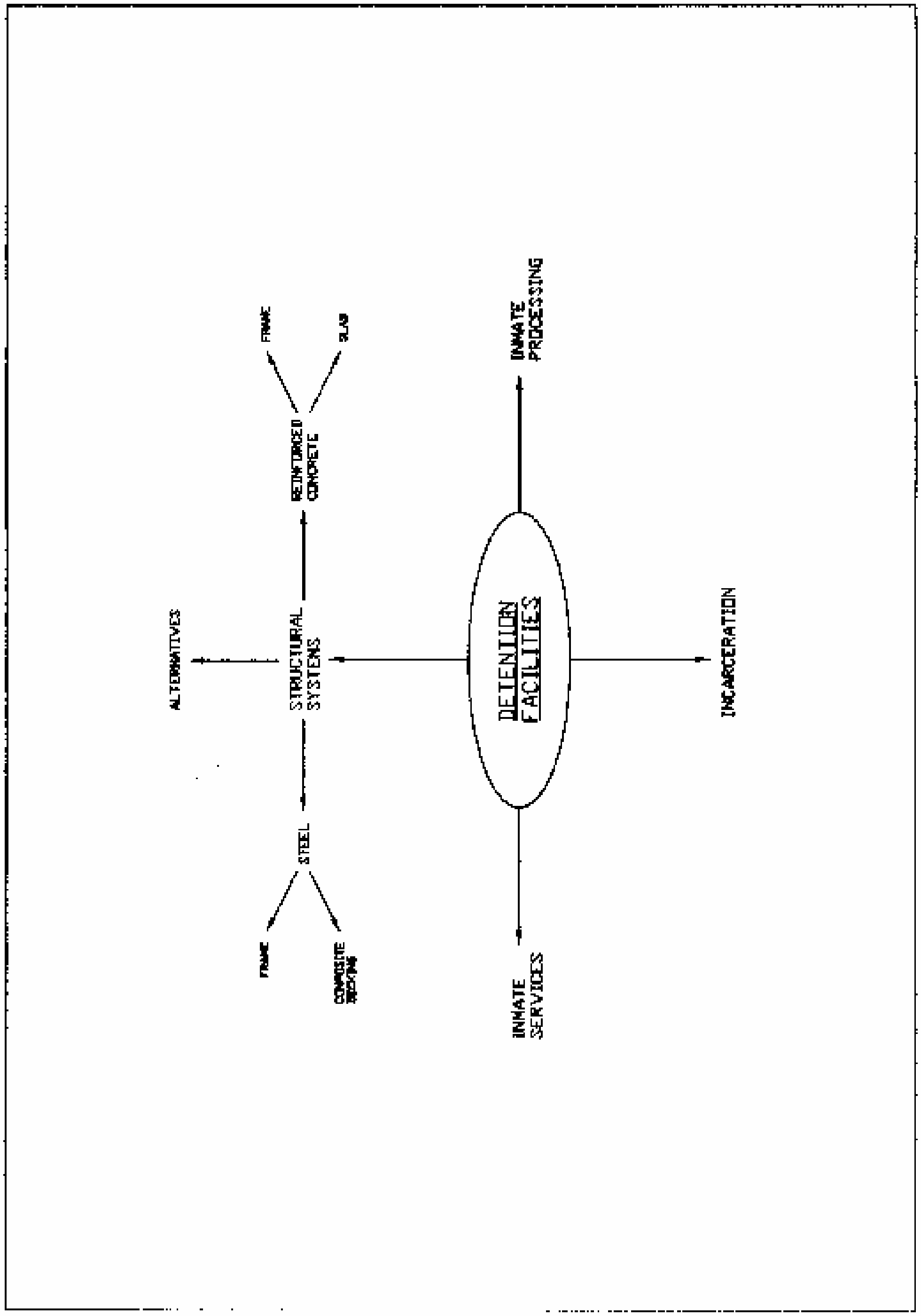


Figure 4 – Inmate Processing Considerations

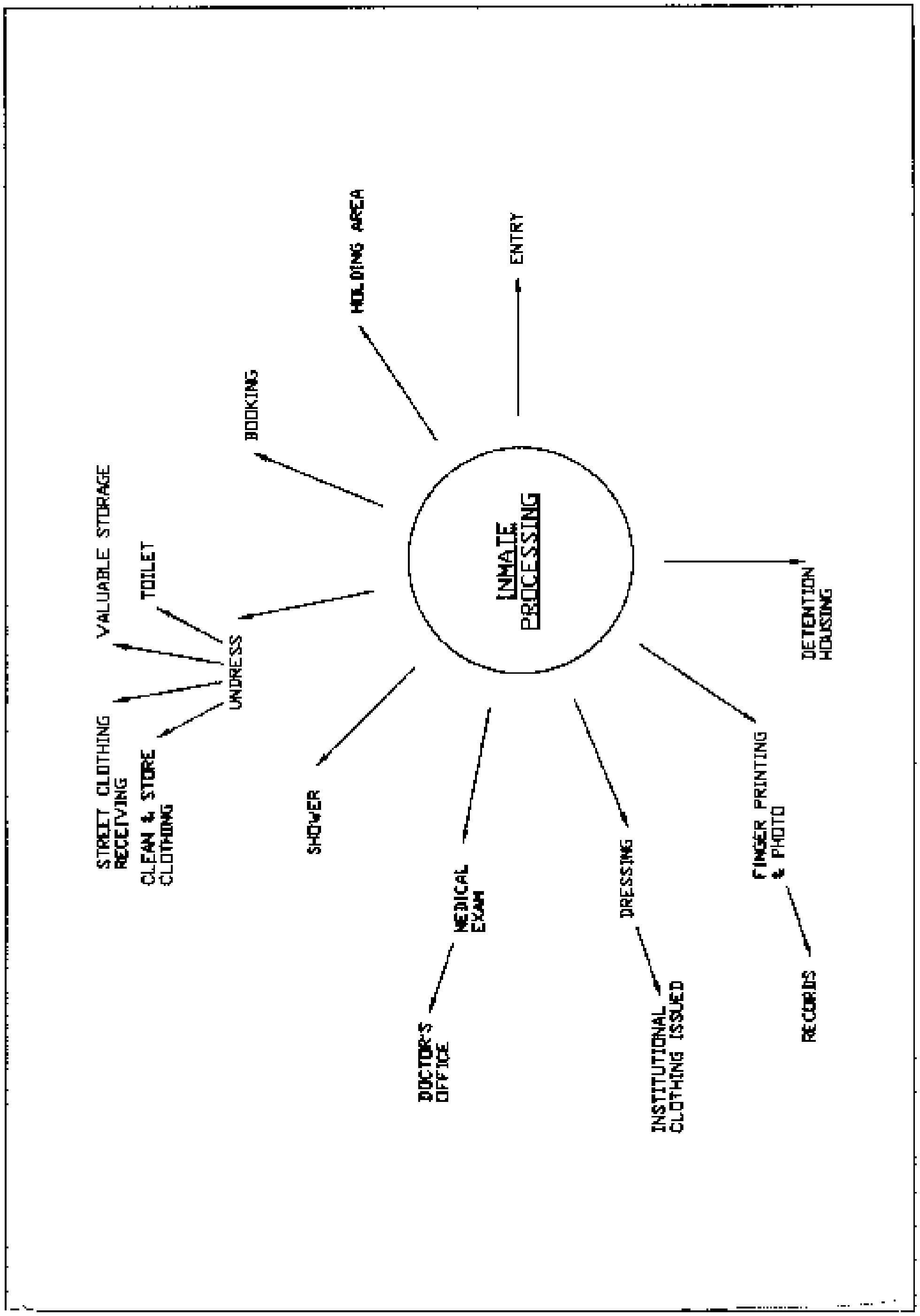


Figure 5 – Inmate Service Considerations by Building

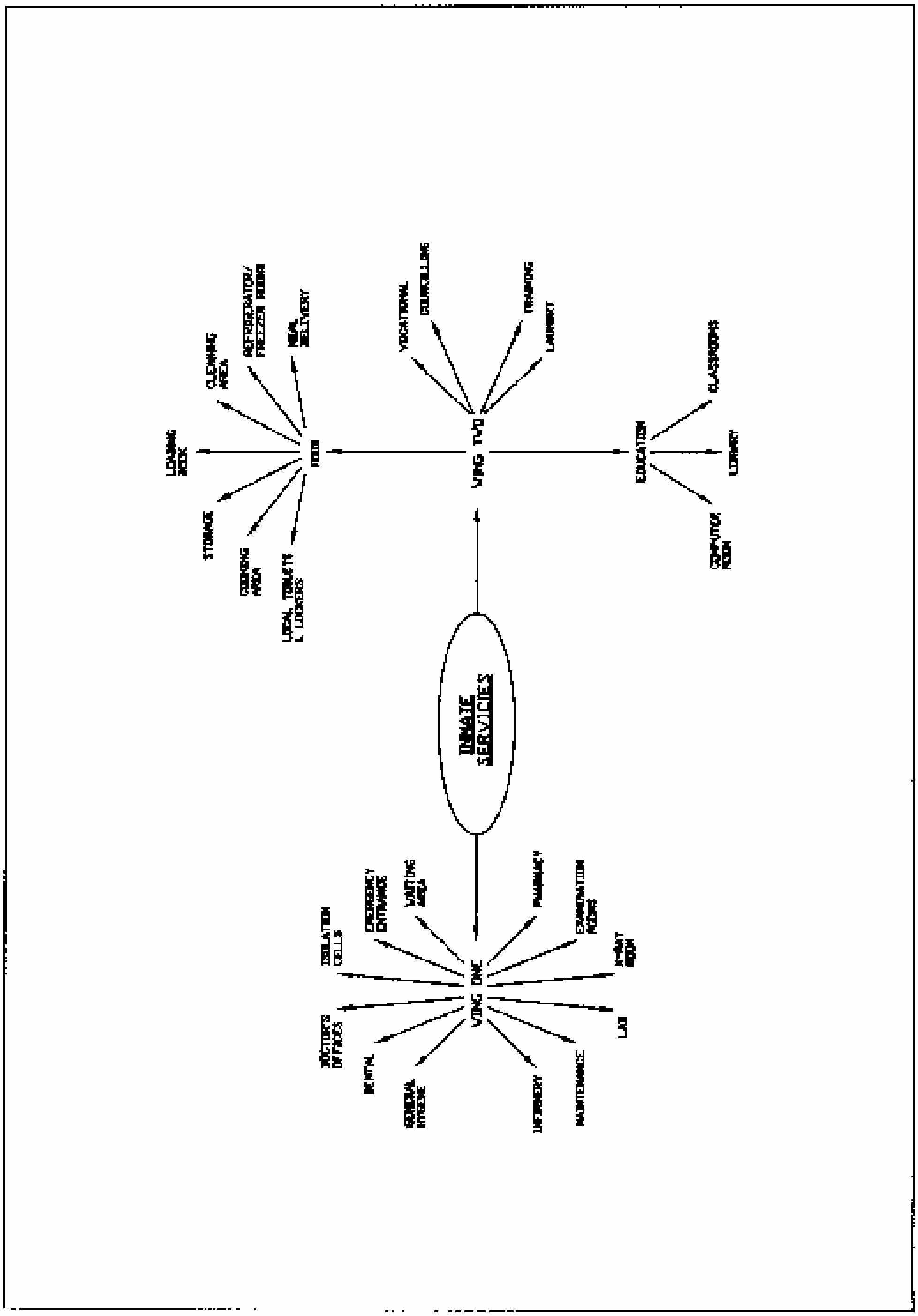
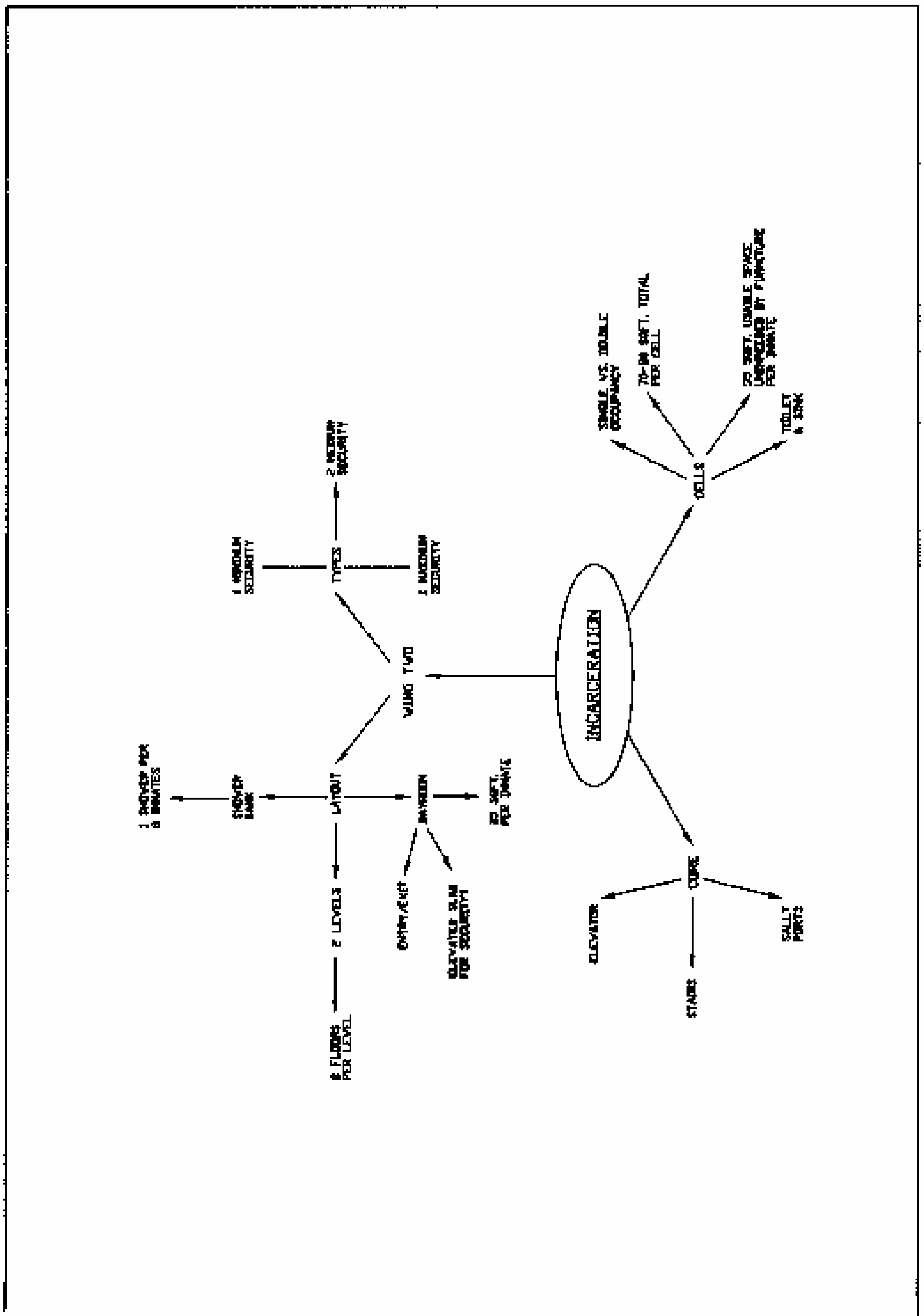


Figure 6 – Incarceration Unit Considerations



B Deliverables

B.1 Office Layout – Gravity Loads

B.1.1 Steel Cost

- OPTION #1 Composite Beam Design – 10ft. & 16ft. Column Spacing

Cost of Slabs	= $467,000
Cost of Beams	= $124,000
Cost of Girders	= $88,000
Cost of Columns	= $118,000
Total Cost of Option 1	**= $797,000**

Number of different beam member sizes	= 5
Number of different girder member sizes	= 5
Number of different column member sizes	= 7

- OPTION #2A Composite Beam Design – 20ft. Column Spacing

Cost of Slabs	= $467,000
Cost of Beams	= $148,000
Cost of Girders	= $118,000
Cost of Columns	= $99,000
Total Cost of Option 2A	**= $832,000**

Number of different beam member sizes	= 5
Number of different girder member sizes	= 5
Number of different column member sizes	= 11

- OPTION #2B Composite Open Web Joist Design – 20ft. Column Spacing

Cost of Slabs	= $226,000
Cost of Open Web Joists	= $48,000
Cost of Girders	= $190,000
Cost of Columns	= $89,000
Total Cost of Option 2B	**= $553,000**

Number of different open web joist sizes	= 5
Number of different girder member sizes	= 8
Number of different column member sizes	= 8

- OPTION #2C Non-composite Open Beam Design – 20ft. Column Spacing

Cost of Slabs	= $330,000
Cost of Open Web Joists	= $490,000
Cost of Girders	= $221,000
Cost of Columns	= $89,000
Total Cost of Option 2C	**= $1,130,000**

Number of different open web joist sizes	= 6
Number of different girder member sizes	= 7
Number of different column member sizes	= 10

Due to cost and total number of members, option number 2b is the most feasible option. With this method, less members are required resulting in a smaller number of connections.

B.1.1.1 Steel Cost Breakdown – Option 1 (Composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
BC45	1	W14x30	1	30	42	12	12	24	288	4.32	504	32.77	9437.76	1.29	650.16
CD45	1	W14x22	1	22	32	15	15	20	300	3.3	480	24.75	7425.00	1.29	619.2
BC45	2,3,4	W12x30	3	30	42	12	36	24	864	12.96	1512	32.75	28296.00	1.29	1950.48
CD45	2,3,4	W12x22	3	22	32	15	45	20	900	9.9	1440	24.86	22374.00	1.29	1857.6
BC45	5	W12x35	1	35	48	12	12	24	288	5.04	576	37.70	10857.60	1.29	743.04
CD45	5	W12x26	1	26	36	15	15	20	300	3.9	540	28.86	8658.00	1.29	696.6
BC45	6	W12x26	1	26	36	12	12	24	288	3.744	432	28.86	8311.68	1.29	557.28
CD45	6	W12x16	1	16	22	15	15	20	300	2.4	330	18.75	5625.00	1.29	425.7
BC45	Roof	W12x26	1	26	36	12	12	24	288	3.744	432	28.86	8311.68	1.29	557.28
CD45	Roof	W12x16	1	16	22	15	15	20	300	2.4	330	18.75	5625.00	1.29	425.7
					Total =		189		Total =	51.708	6576		**114921.72**		**8483.04**

Girder

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
C45	1	W14x38	1	38	NA	10	10	16	160	3.04	NA	40.75	6520.00
C45	1	W14x38	1	38	NA	4	4	10	40	0.76	NA	40.75	1630.00
D45	1	W14x26	1	26	NA	10	10	16	160	2.08	NA	28.43	4548.80
D45	1	W14x26	1	26	NA	4	4	10	40	0.52	NA	28.43	1137.20
C45	2,3,4	W14x34	3	34	NA	10	30	16	480	8.16	NA	36.70	17616.00
C45	2,3,4	W14x34	3	34	NA	4	12	10	120	2.04	NA	36.70	4404.00
D45	2,3,4	W14x22	3	22	NA	10	30	16	480	5.28	NA	24.75	11880.00
D45	2,3,4	W14x22	3	22	NA	4	12	10	120	1.32	NA	24.75	2970.00
C45	5	W14x38	1	38	NA	10	10	16	160	3.04	NA	40.75	6520.00
C45	5	W14x38	1	38	NA	4	4	10	40	0.76	NA	40.75	1630.00
D45	5	W14x26	1	26	NA	10	10	16	160	2.08	NA	28.75	4600.00
D45	5	W14x26	1	26	NA	4	4	10	40	0.52	NA	28.75	1150.00
C45	6	W14x30	1	30	NA	10	10	16	160	2.4	NA	32.77	5243.20
C45	6	W14x30	1	30	NA	4	4	10	40	0.6	NA	32.77	1310.80
D45	6	W14x22	1	22	NA	10	10	16	160	1.76	NA	24.75	3960.00
D45	6	W14x22	1	22	NA	4	4	10	40	0.44	NA	24.75	990.00
C45	Roof	W14x30	1	30	NA	10	10	16	160	2.4	NA	32.77	5243.20
C45	Roof	W14x30	1	30	NA	4	4	10	40	0.6	NA	32.77	1310.80
D45	Roof	W14x22	1	22	NA	10	10	16	160	1.76	NA	24.75	3960.00
D45	Roof	W14x22	1	22	NA	4	4	10	40	0.44	NA	24.75	990.00
					Total =		196		Total =	40	0		**87614.00**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A1	Basement	W14x30	1	30	NA	4	4	15	60	0.90	NA	32.77	1966.20
A1	1, 2	W14x30	1	30	NA	4	4	27	108	1.62	NA	32.77	3539.16
A1	3,4	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A2	Basement	W14x43	1	43	NA	4	4	15	60	1.29	NA	45.70	2742.00
A2	1,2	W14x43	1	43	NA	4	4	27	108	2.32	NA	45.75	4941.00
A2	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.77	3145.92
A2	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A3	Basement	W14x43	1	43	NA	8	8	15	120	2.58	NA	45.70	5484.00
A3	1,2	W14x43	1	43	NA	8	8	27	216	4.64	NA	45.70	9871.20
A3	3,4	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
A3	5,6	W14x22	1	22	NA	8	8	24	192	2.11	NA	24.75	4752.00
B1	Basement	W14x43	1	43	NA	4	4	15	60	1.29	NA	45.70	2742.00
B1	1, 2	W14x38	1	38	NA	4	4	27	108	2.05	NA	40.75	4401.00
B1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.77	3145.92
B1	5,6	W14x26	1	26	NA	4	4	24	96	1.25	NA	28.43	2729.28
B2	Basement	W14x61	1	61	NA	4	4	15	60	1.83	NA	63.75	3825.00
B2	1,2	W14x61	1	61	NA	4	4	27	108	3.29	NA	63.75	6885.00
B2	3,4	W14x38	1	38	NA	4	4	24	96	1.82	NA	40.75	3912.00
B2	5,6	W14x26	1	26	NA	4	4	24	96	1.25	NA	28.43	2729.28
B3	Basement	W14x68	1	68	NA	8	8	15	120	4.08	NA	70.75	8490.00
B3	1,2	W14x61	1	61	NA	8	8	27	216	6.59	NA	63.75	13770.00
B3	3,4	W14x43	1	43	NA	8	8	24	192	4.13	NA	45.70	8774.40
B3	5,6	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
					Total =		128		Total =	54.86	0		**117557.04**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($5.90/S.F.)	Finishing ($0.29/S.F.)	Total Cost
7	5536	4.5	150	2179.8	1614.67	160578.60	228636.80	11238.08	400453.48
1	6400	4.5	150	360	266.67	26520.00	37760.00	1856.00	66136.00
									466589.48

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Office Building Cost (16ft Column Spacing) = $795,165.28

B.1.1.2 Steel Cost Breakdown – Option 2A (Composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
BC45	1	W14x34	1	34	48	11	11	24	264	4.488	528	36.7	9688.8	1.29	681.12
CD45	1	W14x26	1	26	36	22	22	20	440	5.72	792	28.43	12509.2	1.29	1021.68
BC45	2,3,4	W14x30	3	30	42	11	33	24	792	35.64	1386	32.77	25953.84	1.29	1787.94
CD45	2,3,4	W14x22	3	22	32	22	66	20	1320	43.56	2112	24.75	32670	1.29	2724.48
BC45	5	W14x34	1	34	48	11	11	24	264	4.488	528	36.7	9688.8	1.29	681.12
CD45	5	W14x26	1	26	36	22	22	20	440	5.72	792	28.43	12509.2	1.29	1021.68
BC45	6	W12x26	1	26	36	11	11	24	264	3.432	396	28.43	7505.52	1.29	510.84
CD45	6	W12x19	1	19	26	22	22	20	440	4.18	572	21.75	9570	1.29	737.88
BC45	Roof	W12x26	1	26	36	11	11	24	264	3.432	396	28.86	7619.04	1.29	510.84
CD45	Roof	W12x19	1	19	26	22	22	20	440	4.18	572	21.75	9570	1.29	737.88
					Total =		231		Total =	114.84	8074		**137284.4**		**10415.46**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
C45	1	W16x57	1	57	NA	10	10	20	200	5.7	NA	60.00	12000.00
D45	1	W16x31	1	31	NA	10	10	20	200	3.1	NA	33.77	6754.00
C45	2,3,4	W16x50	3	50	NA	10	30	20	600	45	NA	52.25	31350.00
D45	2,3,4	W16x31	3	31	NA	10	30	20	600	27.9	NA	33.77	20262.00
C45	5	W16x57	1	57	NA	10	10	20	200	5.7	NA	60.00	12000.00
D45	5	W16x31	1	31	NA	10	10	20	200	3.1	NA	33.77	6754.00
C45	6	W16x40	1	40	NA	10	10	20	200	4	NA	42.75	8550.00
D45	6	W16x26	1	26	NA	10	10	20	200	2.6	NA	28.40	5680.00
C45	Roof	W16x40	1	40	NA	10	10	20	200	4	NA	42.75	8550.00
D45	Roof	W16x26	1	26	NA	10	10	20	200	2.6	NA	28.40	5680.00
					Total =		140		Total =	103.7	0		**117580.00**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
A1	Basement	W14x38	1	38	NA	4	4	15	108	2.05	NA	40.75	4401.00
A1	1,2	W14x34	1	34	NA	4	4	27	96	1.63	NA	36.70	3523.20
A1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.77	3145.92
A1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A2	Basement	W14x53	1	53	NA	8	8	15	120	3.18	NA	55.73	6687.60
A2	1,2	W14x43	1	43	NA	8	8	27	216	4.64	NA	45.75	9882.00
A2	3,4	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
A2	5,6	W14x22	1	22	NA	8	8	24	192	2.11	NA	24.75	4752.00
B1	Basement	W14x61	1	61	NA	4	4	15	60	1.83	NA	63.75	3825.00
B1	1,2	W14x48	1	48	NA	4	4	27	108	2.59	NA	50.75	5481.00
B1	3,4	W14x34	1	34	NA	4	4	24	96	1.63	NA	36.70	3523.20
B1	5,6	W14x26	1	26	NA	4	4	24	96	1.25	NA	28.43	2729.28
B2	Basement	W14x82	1	82	NA	8	8	15	120	4.92	NA	84.75	10170.00
B2	1,2	W14x68	1	68	NA	8	8	27	216	7.34	NA	70.75	15282.00
B2	3,4	W14x48	1	48	NA	8	8	24	192	4.61	NA	50.75	9744.00
B2	5,6	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
					Total =		96		Total =	46.05	0		**98105.88**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($5.90/S.F.)	Finishing ($0.29/S.F.)	Total Cost
7	5536	4.5	150	2179.8	1614.67	160578.60	228636.80	11238.08	400453.48
1	6400	4.5	150	360	266.67	26520.00	37760.00	1856.00	66136.00
									466589.48

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Office Building Cost (20ft Column Spacing) = $829,975.22

B.1.1.3 Steel Cost Breakdown – Option 2B (Composite Open-Web Joist)

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
AB-2.5	1	14VC 1800	1	17	34	10	10	20	200	1.7	340	1464.50	2489.65	1.32	448.8
BC-2.5	1	14VC 1800	1	19	24	5	5	24	120	1.14	120	1464.50	1669.53	1.32	158.4
AB-2.5	2,3,4	12VC 1600	3	13	26	10	30	20	600	11.7	780	1464.50	17134.65	1.32	1029.6
BC-2.5	2,3,4	14VC 1500	3	16	32	5	15	24	360	8.64	480	1464.50	12653.28	1.32	633.6
AB-2.5	5	14VC 1800	1	17	34	10	10	20	200	1.7	340	1464.50	2489.65	1.32	448.8
BC-2.5	5	14VC 1800	1	19	24	5	5	24	120	1.14	120	1464.50	1669.53	1.32	158.4
AB-2.5	6	14VC 1100	1	10	18	10	10	20	200	1	180	1464.50	1464.5	1.32	237.6
BC-2.5	6	14VC 1300	1	14	28	5	5	24	120	0.84	140	1464.50	1230.18	1.32	184.8
AB-2.5	Roof	14VC 1100	1	10	18	10	10	20	200	1	180	1464.50	1464.5	1.32	237.6
BC-2.5	Roof	14VC 1300	1	14	28	5	5	24	120	0.84	140	1464.50	1230.18	1.32	184.8
					Total =		105		Total =	29.7	2820		**43495.65**		**3722.4**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
AB3	1	W14x30	1	30	42	12	12	20	240	3.6	504	32.77	7864.80		
BC3	1	W14x30	1	30	42	6	6	24	144	2.16	252	32.77	4718.88	1.32	332.64
AB3	2,3,4	W12x30	3	30	42	12	36	20	720	32.4	1512	32.75	23580.00	1.32	1995.84
BC3	2,3,4	W12x30	3	30	42	6	18	24	432	19.44	756	32.75	14148.00	1.32	997.92
AB3	5	W14x30	1	30	42	12	12	20	240	3.6	504	32.77	7864.80	1.32	665.28
BC3	5	W14x30	1	30	42	6	6	24	144	2.16	252	32.77	4718.88	1.32	332.64
AB3	6, Roof	W12x22	2	22	32	12	24	20	480	10.56	768	24.86	11932.80	1.32	1013.76
BC3	6, Roof	W12x22	2	22	32	6	12	24	288	6.336	384	24.86	7159.68	1.32	506.88
C45	1	W16x50	1	50	NA	10	10	20	200	5	NA	52.25	10450.00	NA	NA
D45	1	W14x30	1	30	NA	10	10	20	200	3	NA	32.77	6554.00	NA	NA
C45	2,3,4	W16x40	3	40	NA	10	30	20	600	36	NA	42.75	25650.00	NA	NA
D45	2,3,4	W16x26	3	26	NA	10	30	20	600	23.4	NA	28.40	17040.00	NA	NA
C45	5	W16x50	1	50	NA	10	10	20	200	5	NA	52.25	10450.00	NA	NA
D45	5	W14x30	1	30	NA	10	10	20	200	3	NA	32.44	6488.00	NA	NA
C45	6	W16x36	1	36	NA	10	10	20	200	3.6	NA	38.75	7750.00	NA	NA
D45	6	W14x22	1	22	NA	10	10	20	200	2.2	NA	24.75	4950.00	NA	NA
C45	Roof	W16x36	1	36	NA	10	10	20	200	3.6	NA	38.75	7750.00	NA	NA
D45	Roof	W14x22	1	22	NA	10	10	20	200	2.2	NA	24.75	4950.00	NA	NA
					Total =		266		Total =	167.256	4932		**184019.84**		**5844.96**

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Roof	1.5VL22	6400	1	6400	1.76	11264
Floors 1-6	1.5VL22	5680	6	34080	1.76	59980.8
			Total =	40480	Total =	**71244.80**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
A1	Basement	W14x34	1	34	NA	4	4	15	60	1.02	NA	36.70	2202.00
A1	1,2	W14x30	1	30	NA	4	4	27	108	1.62	NA	32.44	3503.52
A1	3,4	W14x26	1	26	NA	4	4	24	96	1.25	NA	28.43	2729.28
A1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A2	Basement	W14x48	1	48	NA	8	8	15	120	2.88	NA	50.75	6090.00
A2	1,2	W14x43	1	43	NA	8	8	27	216	4.64	NA	45.70	9871.20
A2	3,4	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
A2	5,6	W14x22	1	22	NA	8	8	24	192	2.11	NA	24.75	4752.00
B1	Basement	W14x48	1	48	NA	4	4	15	60	1.44	NA	50.75	3045.00
B1	1,2	W14x43	1	43	NA	4	4	27	108	2.32	NA	45.70	4935.60
B1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.77	3145.92
B1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
B2	Basement	W14x68	1	68	NA	8	8	15	120	4.08	NA	70.75	8490.00
B2	1,2	W14x61	1	61	NA	8	8	27	216	6.59	NA	63.75	13770.00
B2	3,4	W14x43	1	43	NA	8	8	24	192	4.13	NA	45.70	8774.40
B2	5,6	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
					Total =		96		Total =	41.39	0		**88644.60**

Concrete

Number of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Cost / cyd. (total)	Total Cost
7	5680	2.75	110	1002.283333	1012.41	119.70	132715.57
1	6400	2.75	110	161.3333333	162.96	119.70	21362.67
				GRAND TOTAL =	**1175.37**		**154078.23**

Note: Concrete, Ready Mix: $101.25/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f - screen finish

Grand Total for Office Building Cost (20ft Column Spacing) - Joist Alternative = $551,050.48

B.1.1.4 Steel Cost Breakdown – Option 2C (Non-composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
BC4.5	1	W16x67	1	67	NA	11	11	24	264	8.84	NA	69.75	18414.00
CD4.5	1	W14x43	1	43	NA	22	22	20	440	9.46	NA	45.75	20130.00
BC4.5	2,3,4	W16x67	3	67	NA	11	33	24	2376	79.60	NA	69.75	165726.00
CD4.5	2,3,4	W14x43	3	43	NA	22	66	20	3960	85.14	NA	45.75	181170.00
BC4.5	5	W14x67	1	67	NA	11	11	24	264	8.84	NA	69.75	18414.00
CD4.5	5	W14x43	1	43	NA	22	22	20	440	9.46	NA	45.75	20130.00
BC4.5	6	W14x61	1	61	NA	11	11	24	264	8.05	NA	63.75	16830.00
CD4.5	6	W14x38	1	38	NA	22	22	20	440	8.36	NA	40.75	17930.00
BC4.5	Roof	W14x48	1	48	NA	11	11	24	264	6.34	NA	50.75	13398.00
CD4.5	Roof	W14x38	1	38	NA	22	22	20	440	8.36	NA	40.75	17930.00
						Total =	231		Total =	232.452	0		**490072.00**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
B45	1	W16x57	1	57	NA	10	10	20	200	5.70	NA	59.75	11950.00
D45	1	W16x31	1	31	NA	10	10	20	200	3.10	NA	33.75	6750.00
B45	2,3,4	W16x50	3	50	NA	10	30	20	1800	45.00	NA	52.75	94950.00
D45	2,3,4	W14x30	3	30	NA	10	30	20	1800	27.00	NA	32.75	58950.00
B45	5	W16x57	1	57	NA	10	10	20	200	5.70	NA	59.75	11950.00
D45	5	W16x31	1	31	NA	10	10	20	200	3.10	NA	33.75	6750.00
B45	6	W16x45	1	45	NA	10	10	20	200	4.50	NA	47.75	9550.00
D45	6	W14x26	1	26	NA	10	10	20	200	2.60	NA	28.75	5750.00
B45	Roof	W16x36	1	36	NA	10	10	20	200	3.60	NA	38.75	7750.00
D45	Roof	W14x26	1	26	NA	10	10	20	200	2.60	NA	28.75	5750.00
						Total =	140		Total =	102.9	0		**220100.00**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
A1	Basement	W14x38	1	38	NA	4	4	15	60	1.14	NA	40.75	2445.00
A1	1,2	W14x34	1	34	NA	4	4	27	108	1.84	NA	36.75	3969.00
A1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	33.00	3168.00
A1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	25.00	2400.00
A2	Basement	W14x53	1	53	NA	8	8	15	120	3.18	NA	55.75	6690.00
A2	1,2	W14x48	1	48	NA	8	8	27	216	5.18	NA	50.75	10962.00
A2	3,4	W14x34	1	34	NA	8	8	24	192	3.26	NA	36.75	7056.00
A2	5,6	W14x26	1	26	NA	8	8	24	192	2.50	NA	28.75	5520.00
B1	Basement	W14x53	1	53	NA	4	4	15	60	1.59	NA	55.75	3345.00
B1	1,2	W14x48	1	48	NA	4	4	27	108	2.59	NA	50.75	5481.00
B1	3,4	W14x34	1	34	NA	4	4	24	96	1.63	NA	36.75	3528.00
B1	5,6	W14x26	1	26	NA	4	4	24	96	1.25	NA	28.75	2760.00
B2	Basement	W14x82	1	82	NA	8	8	15	120	4.92	NA	84.75	10170.00
B2	1,2	W14x68	1	68	NA	8	8	27	216	7.34	NA	70.75	15282.00
B2	3,4	W14x43	1	43	NA	8	8	24	192	4.13	NA	45.75	8784.00
B2	5,6	W14x30	1	30	NA	8	8	24	192	2.88	NA	33.00	6336.00
						Total =	96		Total =	45.93	0		**97896.00**

Cost Analysis of Reinforced Slab Options

Floors 1-6: Assume Broom Finish and 4 use formwork

Option #	slab length	slab width	Number of Slabs	Slab Thickness	Surface Area	Gap Area	Total Area	Minimum Rebar Spacing	Size of Bar	# of bars per foot	concrete cost	rebar cost	Forms	Finishes	Total Cost per floor	C. Cost
Option 2	10	20	16	0.333	200	612.5	5888	12	#4	1.000	$6,563.47	$1,632.00	$34,736.25	$1,884.00	$46,498.72	**$278,992.33**
	10	25	8	0.333	250			12		1.000		$1,020.00				
	10	20	4	0.333	200			12		1.000		$408.00				
	10	25	2	0.333	250			12		1.000		$255.00				

Cost Analysis of Reinforced Slab Options

Roof : Assume Broom Finish and 4 use formwork

Option #	slab length	slab width	Number of Slabs	Slab Thickness	Surface Area	Gap Area	Total Area	Minimum Rebar Spacing	Size of Bar	# of bars per foot	concrete cost	rebar cost	Forms	Finishes	Total Cost of Roof
Option 2	10	20	16	0.333	200	0	6500	12	#4	1.000	$7,246.30	$1,632.00	$38,350.00	$2,080.00	**$50,991.30**
	10	25	8	0.333	250			12		1.000		$1,020.00			
	10	20	4	0.333	200			12		1.000		$408.00			
	10	25	2	0.333	250			12		1.000		$255.00			

GRAND TOTAL FOR OPTION #1 (REINFORCED SLABS, STEEL FRAME) = $1,138,051.63

B.1.2 Concrete Cost

Option #1 - 10' – 16' Column Spacing Options

Option 1:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $111,784.07
Cost of Girders	= $40,404.67
Cost of Columns	= $48,330.63
Total Cost of Option 1	**= $464,066.55**

Option 2:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $112,513.88
Cost of Girders	= $40,200.43
Cost of Columns	= $48,330.63
Total Cost of Option 2	**= $464,592.11**

Option 3:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $113,319.23
Cost of Girders	= $40,200.43
Cost of Columns	= $48,330.63
Total Cost of Option 3	**= $465,397.46**

Option 4:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $108,416.85
Cost of Girders	= $40,404.67
Cost of Columns	= $48,330.63
Total Cost of Option 4	**= $460,699.34**

Option 5:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $108,416.85
Cost of Girders	= $40,200.43
Cost of Columns	= $48,330.63
Total Cost of Option 5	**= $461,224.90**

Option 6:	
Cost of Slabs	= $263,547.18
Cost of T-Beams	= $109,952.02
Cost of Girders	= $40,200.43
Cost of Columns	= $48,330.63
Total Cost of Option 6	**= $462,030.25**

Option 7:

Cost of Slabs	= $263,547.18
Cost of T-Beams	= $119,851.56
Cost of Girders	= $40,781.44
Cost of Columns	= $48,330.63
Total Cost of Option 7	**= $472,510.81**

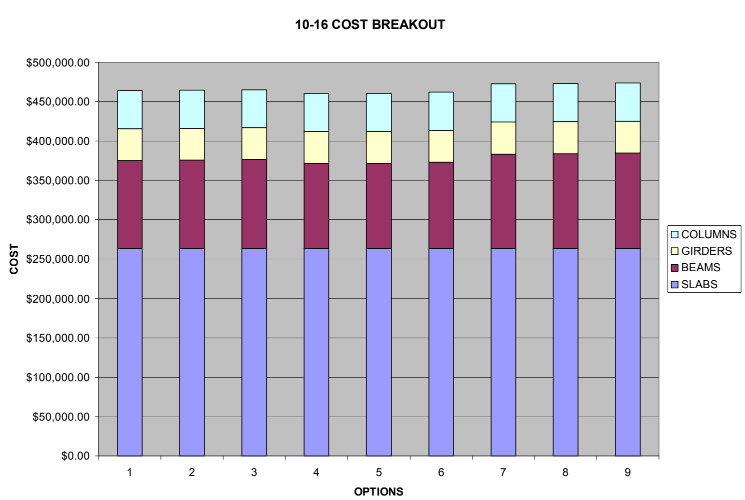
Option 8:

Cost of Slabs	= $263,547.18
Cost of T-Beams	= $120,581.36
Cost of Girders	= $40,577.20
Cost of Columns	= $48,330.63
Total Cost of Option 8	**= $473,036.37**

Option 9:

Cost of Slabs	= $263,547.18
Cost of T-Beams	= $121,386.72
Cost of Girders	= $40,577.20
Cost of Columns	= $48,330.63
Total Cost of Option 9	**= $473,841.72**

The number of T-beams, columns and girders remain constant for every option in this scheme. There are four girders, thirteen T-beams, and 32 columns per floor in this layout. The graph below provides a visual representation of the cost of each option and amount each element contributes to the total cost of each option.



Option #2 - 20' Column Spacing Options

Option 1:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $80,240.86
Cost of Girders	= $46,368.69
Cost of Columns	= $40,338.65
Total Cost for Option 1	**= $430,495.37**

Option 2:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $80,973.02
Cost of Girders	= $46,257.21
Cost of Columns	= $40,338.65
Total Cost for Option 2	**= $431,116.05**

Option 3:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $81,537.21
Cost of Girders	= $46,257.21
Cost of Columns	= $40,338.65
Total Cost for Option 3	**= $431,680.25**

Option 4:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $79,819.58
Cost of Girders	= $47,019.98
Cost of Columns	= $40,338.65
Total Cost for Option 4	**= $430,725.38**

Option 5:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $80,551.74
Cost of Girders	= $46,908.50
Cost of Columns	= $40,338.65
Total Cost for Option 5	**= $431,346.06**

Option 6:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $81,115.93
Cost of Girders	= $46,908.50
Cost of Columns	= $40,338.65
Total Cost for Option 6	**= $431,910.26**

Option 7:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $86,133.72
Cost of Girders	= $44,801.13
Cost of Columns	= $40,338.65
Total Cost for Option 7	**= $434,820.68**

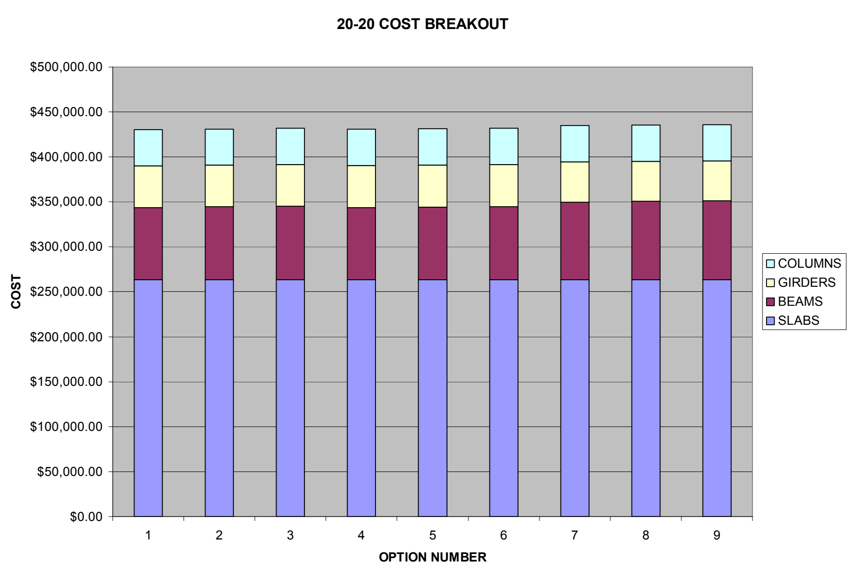
Option 8:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $86,865.88
Cost of Girders	= $44,689.65
Cost of Columns	= $40,338.65
Total Cost for Option 8	**= $435,441.36**

Option 9:

Cost of slabs	= $263,547.18
Cost of T-Beams	= $87,430.07
Cost of Girders	= $44,689.65
Cost of Columns	= $40,338.65
Total Cost for Option 9	**= $436,005.55**

The number of T-beams, girders and columns for this scheme remain constant no matter the option number. There were always 11 T-beams per floor, four girders per floor, and 24 columns. A visual aid is provided by the figure below to better illustrate the similar cost of the options and the amount of the total cost each member type contributed.



B.1.2.1 Concrete Cost Breakdown – Option 1

Total Cost Summary

Costs

	Slabs	
Floor	$225,897.58	4 " slab
Roof	$37,649.60	4 " slab

	T Beams	
Floor	Option 1	$97,047.35
	Option 2	$93,680.13
	Option 3	$105,114.84
Roof	Option 1	$14,736.72
	Option 2	$15,466.53
	Option 3	$16,271.88

Floor Girders

Girders for Floor T Beam Option 1		Girders for Floor T Beam Option 2		Girders for T Beam Option 3	
FGA1-1	$17,838.75	FGB1-1	$17,838.75	FGC1-1	$18,000.96
FGA2-1	$16,827.90	FGB2-1	$16,827.90	FGC2-1	$17,042.46
FGA2-2	$17,537.02	FGB2-2	$17,537.02	FGC2-2	$18,192.98

Roof Girders

Girders for Floor T Beam Option 1		Girders for Floor T Beam Option 2		Girders for T Beam Option 3	
RGA1-1	$2,962.29	RGB1-1	$2,850.81	RGC1-1	$2,850.81
RGA2-1	$2,775.74	RGB2-1	$2,775.74	RGC2-1	$2,775.74
RGA2-2	$2,854.64	RGB2-2	$2,682.97	RGC2-2	$2,682.97
		RGB2-3	$2,812.90	RGC2-3	$2,812.90

	Column Cost
all Columns	$48,330.63

	Combinations	**TOTAL**	SLAB	BEAMS	GIRDERS	COLUMNS
1	FT1, RT1, FGA1-1, FGA2-1, RGA1-1, RGA2-1	$464,066.55	$263,547.18	$111,784.07	$40,404.67	$48,330.63
2	FT1, RT2, FGA1-1, FGA2-1, RGB1-1, RGB2-2	$464,592.11	$263,547.18	$112,513.88	$40,200.43	$48,330.63
3	FT1, RT3, FGA1-1, FGA2-1, RGC1-1, RGC2-2	$465,397.46	$263,547.18	$113,319.23	$40,200.43	$48,330.63
4	FT2, RT1, FGB1-1, FGB2-1, RGA1-1, RGA2-1	$460,699.34	$263,547.18	$108,416.85	$40,404.67	$48,330.63
5	FT2, RT2, FGB1-1, FGB2-1, RGB1-1, RGB2-2	$461,224.90	$263,547.18	$108,416.85	$40,200.43	$48,330.63
6	FT2, RT3, FGB1-1, FGB2-1, RGC1-1, RGC2-2	$462,030.25	$263,547.18	$109,952.02	$40,200.43	$48,330.63
7	FT3, RT1, FGC1-1, FGC2-1, RGA1-1, RGA2-1	$472,510.81	$263,547.18	$119,851.56	$40,781.44	$48,330.63
8	FT3, RT2, FGC1-1, FGC2-1, RGB1-1, RGB2-2	$473,036.37	$263,547.18	$120,581.36	$40,577.20	$48,330.63
9	FT3, RT3, FGC1-1, FGC2-1, RGC1-1, RGC2-2	$473,841.72	$263,547.18	$121,386.72	$40,577.20	$48,330.63

Slab Cost Breakdown

10x16 Column Spacing - Floors 1-6

4 Use Formwork Assembled on Site and Broom Finish applied

option	width	length	thickness	# of slabs	surface area	Gap area	Total area	Rebar Spacing	Bar Size	# of bars/ft	concrete cost	rebar cost	form cost	Finishes	Total for 1 floor	Total for Floors 1-6
option1	8.33	20	0.333	4	666.4	612.5	5020.4	12	#4	1.000	$5,596.82	$84.97	$29,620.36	$1,606.53	$37,175.53	$223,053.17
	7	20	0.333	20	2800.0			12		1.000		$71.40				
	8.33	25	0.333	2	416.5			12		1.000		$106.21				
	7	25	0.333	10	1750.0			12		1.000		$89.25				
option 2	8.33	20	0.292	4	666.4	612.5	5020.4	9	#4	1.333	$4,897.21	$113.29	$29,620.36	$1,606.53	$36,593.20	$219,559.20
	7	20	0.292	20	2800.0			9		1.333		$95.20				
	8.33	25	0.292	2	416.5			9		1.333		$141.61				
	7	25	0.292	10	1750.0			9		1.333		$119.00				
option 3	8.33	20	0.250	4	666.4	612.5	5020.4	9	#4	1.333	$4,197.61	$113.29	$29,620.36	$1,606.53	$35,893.60	$215,361.59
	7	20	0.250	20	2800.0			9		1.333		$95.20				
	8.33	25	0.250	2	416.5			9		1.333		$141.61				
	7	25	0.250	10	1750.0			9		1.333		$119.00				

10x16 Column Spacing - Roof

4 Use Formwork Assembled on Site and Broom Finish applied

option	width	length	thickness	# of slabs	surface area	Gap area	Total area	Rebar Spacing	Bar Size	# of bars/ft	concrete cost	rebar cost	form cost	Finishes	Total for 1 floor
option1	8.33	20	0.333	4	666.4	612.5	5020.4	12	#4	1.000	$5,596.82	$84.97	$29,620.36	$1,606.53	$37,175.53
	7	20	0.333	20	2800.0			12		1.000		$71.40			
	8.33	25	0.333	2	416.5			12		1.000		$106.21			
	7	25	0.333	10	1750.0			12		1.000		$89.25			
option 2	8.33	20	0.250	4	666.4	612.5	5020.4	9	#4	1.333	$4,197.61	$113.29	$29,620.36	$1,606.53	$35,893.60
	7	20	0.250	20	2800.0			9		1.333		$95.20			
	8.33	25	0.250	2	416.5			9		1.333		$141.61			
	7	25	0.250	10	1750.0			9		1.333		$119.00			
option 3	8.33	20	0.229	4	666.4	612.5	5020.4	8	#4	1.500	$3,847.81	$127.45	$29,620.36	$1,606.53	$35,602.43
	7	20	0.229	20	2800.0			8		1.500		$107.10			
	8.33	25	0.229	2	416.5			8		1.500		$159.31			
	7	25	0.229	10	1750.0			8		1.500		$133.88			

T-Beam Cost Breakdown

10-16 Floors 1-6 T Beams

Beam B3-B4-B3

					As + 1			***As - 1***			***As + 2***			***As - 2***												
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor	Total per option per floor	6 floors	
Option 1	12	18	65	97.50	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	103	$326.08	$238.25	$516.75	$164.80	$1,245.89	11	$13,704.76	$16,174.56	$97,047.35	Option 1
Option 2	10	18	65	81.25	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	103	$271.74	$238.25	$516.75	$164.80	$1,191.54	11	$13,106.94	$15,613.36	$93,680.13	Option 2
Option 3	14	16	65	101.11	#7	3	20	#7	2	6.67	#7	4	25	#8	4	8.33	109	$338.16	$321.52	$516.75	$174.40	$1,350.83	11	$14,859.10	$17,519.14	$105,114.84	Option 3

Beam B1-B2-B1

					As + 1			***As - 1***			***As + 2***			***As - 2***										
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
Option 1	10	16	65	72.22	#7	2	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$241.54	$232.05	$604.50	$156.80	$1,234.90	2	$2,469.79
Option 2	12	16	65	86.67	#7	2	20	#6	2	6.67	#5	4	25	#7	4	8.33	98	$289.85	$202.05	$604.50	$156.80	$1,253.21	2	$2,506.41
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.67	#7	3	25	#7/#6	2/2	8.33	96	$338.16	$233.76	$604.50	$153.60	$1,330.02	2	$2,660.03

10-16 Roof T Beams

Beam B3-B4-B3

					As + 1			***As - 1***			***As + 2***			***As - 2***												
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor	Total per option per floor	
Option 1	10	18	65	81.25	#7	2	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$271.74	$192.48	$516.75	$144.00	$1,124.96	11	$12,374.58	$14,736.72	Option 1
Option 2	12	16	65	86.67	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	94	$289.85	$225.26	$516.75	$150.40	$1,182.26	11	$13,004.87	$15,466.53	Option 2
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.67	#7	3	25	#7	4	8.33	94	$338.16	$240.25	$516.75	$150.40	$1,245.56	11	$13,701.21	$16,271.88	Option 3

Beam B1-B2-B1

					As + 1			***As - 1***			***As + 2***			***As - 2***										
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
Option 1	10	16	65	72.22	#5	3	20	#5	3	6.67	#6/#7	2/1	25	#6	4	8.33	90	$241.54	$191.03	$604.50	$144.00	$1,181.07	2	$2,362.14
Option 2	12	16	65	86.67	#7	2	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$289.85	$192.48	$604.50	$144.00	$1,230.83	2	$2,461.66
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$338.16	$198.68	$604.50	$144.00	$1,285.34	2	$2,570.67

Combinations

Floor		Combo's	
Option 1	$97,047.35	FO1-RO1	$111,784.07
Option 2	$93,680.13	FO1-RO2	$112,513.88
Option 3	$105,114.84	FO1-RO2	$113,319.23
Roof			
Option 1	$14,736.72	FO2-RO1	$108,416.85
Option 2	$15,466.53	FO2-RO2	$109,146.66
Option 3	$16,271.88	FO2-RO3	$109,952.02
		FO3-RO1	$119,851.56
		FO3-RO2	$120,581.36
		FO3-RO3	$121,386.72

Girder Cost Breakdown

Girder Summary 10-16 Frame

Floors 1-6

T Beam Option 1

					As + 1			*As -1*			*As + 2*			*As - 2*									
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	18	100	150.00	#5	2	7.50	#5	2	3.33	#6	2	12.00	#6	2	5.33	$501.67	$54.90	$930.00	$1,486.56	2	$2,973.12	$17,838.75
G2-1	12	18	100	150.00	#7	2	7.50	#5	3	3.33	#6	4	12.00	#6	4	5.33	$501.67	$105.66	$795.00	$1,402.32	2	$2,804.65	$16,827.90
G2-2	12	20	100	166.67	#7	2	7.50	#7	2	3.33	#7	3	12.00	#7	3	5.33	$557.41	$109.01	$795.00	$1,461.42	2	$2,922.84	$17,537.02

T Beam Option 2

					As + 1			*As -1*			*As + 2*			*As - 2*									
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	18	100	150.00	#5	2	7.50	#5	2	3.33	#6	2	12.00	#6	2	5.33	$501.67	$54.90	$930.00	$1,486.56	2	$2,973.12	$17,838.75
G2-1	12	18	100	150.00	#7	2	7.50	#5	3	3.33	#6	4	12.00	#6	4	5.33	$501.67	$105.66	$795.00	$1,402.32	2	$2,804.65	$16,827.90
G2-2	12	20	100	166.67	#7	2	7.50	#7	2	3.33	#7	3	12.00	#7	3	5.33	$557.41	$109.01	$795.00	$1,461.42	2	$2,922.84	$17,537.02

T Beam Option 3

girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	18	100	150.00	#5	2	7.50	#5	2	3.33	#7	2	12.00	#7	2	5.33	$501.67	$68.41	$930.00	$1,500.08	2	$3,000.16	$18,000.96
G2-1	12	18	100	150.00	#7	2	7.50	#5	3	3.33	#8	3	12.00	#6	4	5.33	$501.67	$123.54	$795.00	$1,420.20	2	$2,840.41	$17,042.46
G2-2	16	16	100	177.78	#7	2	7.50	#7	2	3.33	#6	5	12.00	#6	5	5.33	$594.57	$126.51	$795.00	$1,516.08	2	$3,032.16	$18,192.98

Roof

T Beam Option 1

					As + 1			*As -1*			*As + 2*			*As - 2*								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	18	100	150.00	#4	2	7.50	#4	2	3.33	#6	2	12.00	#6	2	5.33	$501.67	$49.48	$930.00	$1,481.15	2	$2,962.29
G2-1	12	18	100	150.00	#6	2	7.50	#6	2	3.33	#8	2	12.00	#8	2	5.33	$501.67	$91.20	$795.00	$1,387.87	2	$2,775.74
G2-2	12	20	100	166.67	#6	2	7.50	#6	2	3.33	#7	2	12.00	#7	2	5.33	$557.41	$74.91	$795.00	$1,427.32	2	$2,854.64

T Beam Option 2

					As + 1			*As -1*			*As + 2*			*As - 2*								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	16	100	133.33	#4	2	7.50	#4	2	3.33	#6	2	12.00	#6	2	5.33	$445.93	$49.48	$930.00	$1,425.40	2	$2,850.81
G2-1	12	18	100	150.00	#6	2	7.50	#6	2	3.33	#8	2	12.00	#8	2	5.33	$501.67	$91.20	$795.00	$1,387.87	2	$2,775.74
G2-2	12	16	100	133.33	#6	2	7.50	#6	2	3.33	#7	3	12.00	#7	3	5.33	$445.93	$100.56	$795.00	$1,341.49	2	$2,682.97
G2-3	14	16	100	155.56	#6	2	7.50	#6	2	3.33	#8	2	12.00	#8	2	5.33	$520.25	$91.20	$795.00	$1,406.45	2	$2,812.90

T Beam Option 3

					As + 1			*As -1*			*As + 2*			*As - 2*								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	16	100	133.33	#4	2	7.50	#4	2	3.33	#6	2	12.00	#6	2	5.33	$445.93	$49.48	$930.00	$1,425.40	2	$2,850.81
G2-1	12	18	100	150.00	#6	2	7.50	#6	2	3.33	#8	2	12.00	#8	2	5.33	$501.67	$91.20	$795.00	$1,387.87	2	$2,775.74
G2-2	12	16	100	133.33	#6	2	7.50	#6	2	3.33	#7	3	12.00	#7	3	5.33	$445.93	$100.56	$795.00	$1,341.49	2	$2,682.97
G2-3	14	16	100	155.56	#6	2	7.50	#6	2	3.33	#8	2	12.00	#8	2	5.33	$520.25	$91.20	$795.00	$1,406.45	2	$2,812.90

Column Cost Breakdown

Column Size Summary w/ Cost Data

10-16 First Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	16.00	16.00	15.00	26.67	#8	4	17	$89.19	$117.00	$32.81	$114.00	$353.00	20	$7,059.90
C2	16.00	16.00	15.00	26.67	#8	4	17							
C3	16.00	16.00	15.00	26.67	#8	4	17							
C4	16.00	16.00	15.00	26.67	#8	4	17							
C5	24.00	24.00	15.00	60.00	#8	8	17	$200.67	$234.00	$51.00	$115.50	$601.17	12	$7,214.00
C6	24.00	24.00	15.00	60.00	#8	8	17							

10-16 Floors 2 and 3 Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	20	$4,249.07
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	12.00	12.00	12.00	12.00	#6	4	18							
C5	16.00	16.00	12.00	21.33	#8	4	14	$71.35	$93.60	$27.02	$23.16	$215.13	12	$2,581.54
C6	16.00	16.00	12.00	21.33	#8	4	14							

10-16 Floors 4 and 5 Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	32	$6,798.51
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	12.00	12.00	12.00	12.00	#6	4	18							
C5	12.00	12.00	12.00	12.00	#6	4	18							
C6	12.00	12.00	12.00	12.00	#6	4	18							

10-16 Top Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	32	$6,798.51
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	12.00	12.00	12.00	12.00	#6	4	18							
C5	12.00	12.00	12.00	12.00	#6	4	18							
C6	12.00	12.00	12.00	12.00	#6	4	18							

Total Cost for Building $48,330.63

B.1.2.2 Concrete Cost Breakdown – Option 2

Total Cost Summary

Costs

	Slabs			
Floor	$225,897.58	4 " slab	total slab	$263,547.18
Roof	$37,649.60	4 " slab		

	T Beams	
Floor	Option 1	$67,754.06
	Option 2	$67,332.78
	Option 3	$73,646.92
Roof	Option 1	$12,486.80
	Option 2	$13,218.96
	Option 3	$13,783.15

Floor Girders

Girders for Floor T Beam Option 1		Girders for Floor T Beam Option 2		Girders for T Beam Option 3	
FGA1-1	$18,880.40	FGB1-1	$19,531.69	FGC1-1	$18,539.14
FGA2-1	$21,191.27	FGB2-1	$21,191.27	FGC1-2	$18,880.40
FGA2-2	$21,859.32	FGB2-2	$21,859.32	FGC2-1	$19,964.98
FGA2-3	$22,331.60			FGC2-2	$21,859.32
				FGC2-3	$22,331.60

Roof Girders

Girders for Floor T Beam Option 1		Girders for Floor T Beam Option 2		Girders for T Beam Option 3	
RGA1-1	$3,119.87	RGB1-1	$3,008.39	RGC1-1	$3,008.39
RGA2-1	$3,177.15	RGB2-1	$3,177.15	RGC2-1	$3,177.15
RGA2-2	$3,232.89	RGB2-2	$3,232.89	RGC2-2	$3,232.89
	Column Cost	RGB2-3	$3,465.91	RGC2-3	$3,465.91
all Columns	$40,338.65				

		Combinations	SLAB	BEAMS	GIRDERS	COLUMNS	Total
1	$430,495.37	FT1, RT1, FGA1-1, FGA2-1, RGA1-1, RGA2-1	$263,547.18	$80,240.86	$46,368.69	$40,338.65	$430,495.37
2	$431,116.05	FT1, RT2, FGA1-1, FGA2-1, RGB1-1. RGB2-1	$263,547.18	$80,973.02	$46,257.21	$40,338.65	$431,116.05
3	$431,680.25	FT1, RT3, FGA1-1, FGA2-1, RGC1-1, RGC2-1	$263,547.18	$81,537.21	$46,257.21	$40,338.65	$431,680.25
4	$430,725.38	FT2, RT1, FGB1-1, FGB2-1, RGA1-1, RGA2-1	$263,547.18	$79,819.58	$47,019.98	$40,338.65	$430,725.38
5	$431,346.06	FT2, RT2, FGB1-1, FGB2-1, RGB1-1, RGB2-1	$263,547.18	$80,551.74	$46,908.50	$40,338.65	$431,346.06
6	$431,910.26	FT2, RT3, FGB1-1, FGB2-1, RGC1-1, RGC2-1	$263,547.18	$81,115.93	$46,908.50	$40,338.65	$431,910.26
7	$434,820.68	FT3, RT1, FGC1-1, FGC2-1, RGA1-1, RGA2-1	$263,547.18	$86,133.72	$44,801.13	$40,338.65	$434,820.68
8	$435,441.36	FT3, RT2, FGC1-1, FGC2-1, RGB1-1, RGB2-1	$263,547.18	$86,865.88	$44,689.65	$40,338.65	$435,441.36
9	$436,005.55	FT3, RT3, FGC1-1, FGC2-1, RGC1-1, RGC2-1	$263,547.18	$87,430.07	$44,689.65	$40,338.65	$436,005.55

Slab Cost Breakdown

Reinforced Frame - Slab Cost Estimates

20x20 Column Spacing - Floors 1-6

4 Use Formwork Assembled on Site and Broom Finish applied

option	width	length	thickness	# of slabs	surface area	Gap area	Total area	Rebar Spacing	Bar Size	# of bars/ft	concrete cost	rebar cost	form cost	Finishes	Total for 1 floor	Total for Floors 1-6
option1	8.46	20	0.333	4	676.8	612.5	5078.9	12	#4	1.000	$5,662.03	$86.29	$29,965.51	$1,625.25	$37,649.60	$225,897.58
	8.83	20	0.333	16	2825.6			12		1.000		$90.07				
	8.46	25	0.333	2	423.0			12		1.000		$107.87				
	8.83	25	0.333	8	1766.0			12		1.000		$112.58				
option 2	8.46	20	0.292	4	676.8	612.5	5078.9	9	#4	1.333	$4,954.28	$115.06	$29,965.51	$1,625.25	$37,074.11	$222,444.67
	8.83	20	0.292	16	2825.6			9		1.333		$120.09				
	8.46	25	0.292	2	423.0			9		1.333		$143.82				
	8.83	25	0.292	8	1766.0			9		1.333		$150.11				
option 3	8.46	20	0.250	4	676.8	612.5	5078.9	9	#4	1.333	$4,246.52	$115.06	$29,965.51	$1,625.25	$36,366.36	$218,198.14
	8.83	20	0.250	16	2825.6			9		1.333		$120.09				
	8.46	25	0.250	2	423.0			9		1.333		$143.82				
	8.83	25	0.250	8	1766.0			9		1.333		$150.11				

20x20 Column Spacing - Roof

4 Use Formwork Assembled on Site and Broom Finish applied

option	width	length	thickness	# of slabs	surface area	Gap area	Total area	Rebar Spacing	Bar Size	# of bars/ft	concrete cost	rebar cost	form cost	Finishes	Total for 1 floor
option1	8.46	20	0.333	4	676.8	612.5	5078.9	12	#4	1.000	$5,662.03	$86.29	$29,965.51	$1,625.25	$37,649.60
	8.83	20	0.333	16	2825.6			12		1.000		$90.07			
	8.46	25	0.333	2	423.0			12		1.000		$107.87			
	8.83	25	0.333	8	1766.0			12		1.000		$112.58			
option 2	8.46	20	0.292	4	676.8	612.5	5078.9	9	#4	1.333	$4,954.28	$115.06	$29,965.51	$1,625.25	$37,074.11
	8.83	20	0.292	16	2825.6			9		1.333		$120.09			
	8.46	25	0.292	2	423.0			9		1.333		$143.82			
	8.83	25	0.292	8	1766.0			9		1.333		$150.11			
option 3	8.46	20	0.250	4	676.8	612.5	5078.9	9	#4	1.333	$4,246.52	$115.06	$29,965.51	$1,625.25	$36,366.36
	8.83	20	0.250	16	2825.6			9		1.333		$120.09			
	8.46	25	0.250	2	423.0			9		1.333		$143.82			
	8.83	25	0.250	8	1766.0			9		1.333		$150.11			

T-Beam Cost Breakdown

T Beam Summary Table

20-20 Floors 1-6 T Beams

Beam B3-B4-B3

					As + 1			***As - 1***			***As + 2***			***As - 2***												
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor	Total per option per floor	6 floors
Option 1	12	18	65	97.50	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$326.08	$238.25	$516.75	$156.80	$1,237.89	9	$11,140.99	$13,550.81	$67,754.06
Option 2	10	20	65	90.28	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$301.93	$238.25	$516.75	$156.80	$1,213.73	9	$10,923.60	$13,466.56	$67,332.78
Option 3	14	16	65	101.11	#7	3	20	#7	2	6.77	#7	4	25	#8	4	8.33	104	$338.16	$321.81	$516.75	$166.40	$1,343.12	9	$12,088.11	$14,729.38	$73,646.92

Beam B1-B2-B1

					As + 1			***As - 1***			***As + 2***			***As - 2***										
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
Option 1	10	18	65	81.25	#6/ #7	1/1	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$271.74	$184.68	$604.50	$144.00	$1,204.91	2	$2,409.82
Option 2	12	16	65	86.67	#7	2	20	#5	3	6.67	#6	4	25	#6	4	8.33	98	$289.85	$220.33	$604.50	$156.80	$1,271.48	2	$2,542.96
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.77	#7	3	25	#7/#6	2/2	8.33	90	$338.16	$233.97	$604.50	$144.00	$1,320.64	2	$2,641.27

20-20 Roof T Beams

Beam B3-B4-B3

					As + 1			***As - 1***			***As + 2***			***As - 2***												
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor	Total per option per floor	
Option 1	10	18	65	81.25	#7	2	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$271.74	$192.48	$516.75	$144.00	$1,124.96	9	$10,124.66	$12,486.80	Option 1
Option 2	12	16	65	86.67	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	94	$289.85	$238.25	$516.75	$150.40	$1,195.26	9	$10,757.30	$13,218.96	Option 2
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.77	#7	3	25	#7	4	8.33	94	$338.16	$240.47	$516.75	$150.40	$1,245.78	9	$11,212.04	$13,783.15	Option 3

Beam B1-B2-B1

					As + 1			***As - 1***			***As + 2***			***As - 2***										
Option	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
Option 1	10	16	65	72.22	#5	3	20	#5	3	6.67	#6/#7	2/1	25	#6	4	8.33	90	$241.54	$191.03	$604.50	$144.00	$1,181.07	2	$2,362.14
Option 2	12	16	65	86.67	#7	2	20	#6	2	6.67	#6	3	25	#7	3	8.33	90	$289.85	$192.48	$604.50	$144.00	$1,230.83	2	$2,461.66
Option 3	14	16	65	101.11	#6	3	20	#6	2	6.77	#6	3	25	#7	3	8.33	90	$338.16	$198.89	$604.50	$144.00	$1,285.55	2	$2,571.11

Combinations

	Floor		Combo's	
Option 1	$67,754.06		FO1-RO1	$80,240.86
Option 2	$67,332.78		FO1-RO2	$80,973.02
Option 3	$73,646.92		FO1-RO2	$81,537.21
	Roof			
Option 1	$12,486.80		FO2-RO1	$79,819.58
Option 2	$13,218.96		FO2-RO2	$80,551.74
Option 3	$13,783.15		FO2-RO3	$81,115.93
			FO3-RO1	$86,133.72
			FO3-RO2	$86,865.88
			FO3-RO3	$87,430.07

Girder Cost Breakdown

Girder Summary 20-20 Frame

Floors 1-6

T Beam Option 1

					As + 1			As -1			As + 2			As - 2									
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	18	100	150.00	#6	3	15	#6	3	6.667	#6	3	15	#6	3	6.667	$501.67	$141.70	$930.00	$1,573.37	2	$3,146.73	$18,880.40
G2-1	14	22	100	213.89	#7	4	15	#8	3	6.667	#7	4	15	#8	3	6.667	$715.34	$255.60	$795.00	$1,765.94	2	$3,531.88	$21,191.27
G2-2	16	20	100	222.22	#6	6	15	#6	6	6.667	#6	6	15	#6	6	6.667	$743.21	$283.40	$795.00	$1,821.61	2	$3,643.22	$21,859.32
G2-3	18	18	100	225.00	#7	5	15	#7/#8	2/2	6.667	#7	5	15	#7/#8	2/2	6.667	$752.50	$313.47	$795.00	$1,860.97	2	$3,721.93	$22,331.60

T Beam Option 2

					As + 1			As -1			As + 2			As - 2									
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	20	100	166.67	#6	3	15	#5	4	6.667	#6	3	15	#5	4	6.667	$557.41	$140.23	$930.00	$1,627.64	2	$3,255.28	$19,531.69
G2-1	14	22	100	213.89	#7	4	15	#8	3	6.667	#7	4	15	#8	3	6.667	$715.34	$255.60	$795.00	$1,765.94	2	$3,531.88	$21,191.27
G2-2	16	20	100	222.22	#6	6	15	#6	6	6.667	#6	6	15	#6	6	6.667	$743.21	$283.40	$795.00	$1,821.61	2	$3,643.22	$21,859.32

T Beam Option 3

					As + 1			As -1			As + 2			As - 2									
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor	for 6 floors
G1-1	12	16	100	133.33	#8	2	15	#8	2	6.667	#8	2	15	#8	2	6.667	$445.93	$169.00	$930.00	$1,544.93	2	$3,089.86	$18,539.14
G1-2	12	18	100	150.00	#6	3	15	#6	3	6.667	#6	3	15	#6	3	6.667	$501.67	$141.70	$930.00	$1,573.37	2	$3,146.73	$18,880.40
G2-1	12	22	100	183.33	#7	4	15	#8	3	6.667	#7	4	15	#8	3	6.667	$613.15	$255.60	$795.00	$1,663.75	2	$3,327.50	$19,964.98
G2-2	16	20	100	222.22	#6	6	15	#6	6	6.667	#6	6	15	#6	6	6.667	$743.21	$283.40	$795.00	$1,821.61	2	$3,643.22	$21,859.32
G2-3	18	18	100	225.00	#7	5	15	#7/#8	2/2	6.667	#7	5	15	#7/#8	2/2	6.667	$752.50	$313.47	$795.00	$1,860.97	2	$3,721.93	$22,331.60

Roof

T Beam Option 1

					As + 1			As -1			As + 2			As - 2								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	18	100	150.00	#7	2	15	#7	2	6.667	#7	2	15	#7	2	6.667	$501.67	$128.27	$930.00	$1,559.93	2	$3,119.87
G2-1	12	20	100	166.67	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$557.41	$236.17	$795.00	$1,588.57	2	$3,177.15
G2-2	14	18	100	175.00	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$585.28	$236.17	$795.00	$1,616.44	2	$3,232.89

T Beam Option 2

					As + 1			As -1			As + 2			As - 2								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	16	100	133.33	#7	2	15	#7	2	6.667	#7	2	15	#7	2	6.667	$445.93	$128.27	$930.00	$1,504.19	2	$3,008.39
G2-1	12	20	100	166.67	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$557.41	$236.17	$795.00	$1,588.57	2	$3,177.15
G2-2	14	18	100	175.00	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$585.28	$236.17	$795.00	$1,616.44	2	$3,232.89
G2-3	18	16	100	200.00	#7	4	15	#7/#8	2/2	6.667	#7	4	15	#7/#8	2/2	6.667	$668.89	$269.07	$795.00	$1,732.96	2	$3,465.91

T Beam Option 3

					As + 1			As -1			As + 2			As - 2								
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	Concrete Cost	Rebar Cost	Formwork	Total	# of Girders	Cost/Floor
G1-1	12	16	100	133.33	#7	2	15	#7	2	6.667	#7	2	15	#7	2	6.667	$445.93	$128.27	$930.00	$1,504.19	2	$3,008.39
G2-1	12	20	100	166.67	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$557.41	$236.17	$795.00	$1,588.57	2	$3,177.15
G2-2	14	18	100	175.00	#6	5	15	#6	5	6.667	#6	5	15	#6	5	6.667	$585.28	$236.17	$795.00	$1,616.44	2	$3,232.89
G2-3	18	16	100	200.00	#7	4	15	#7/#8	2/2	6.667	#7	4	15	#7/#8	2/2	6.667	$668.89	$269.07	$795.00	$1,732.96	2	$3,465.91

Column Cost Breakdown

Column Size Summary w/ Cost Data

20-20 First Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	16.00	16.00	15.00	26.67	#8	4	17	$89.19	$117.00	$32.81	$114.00	$353.00	12	$4,235.94
C2	16.00	16.00	15.00	26.67	#8	4	17							
C3	16.00	16.00	15.00	26.67	#8	4	17							
C4	24.00	24.00	15.00	60.00	#8	8	17	$200.67	$234.00	$51.00	$115.50	$601.17	12	$7,214.00
C5	24.00	24.00	15.00	60.00	#8	8	17							
C6	24.00	24.00	15.00	60.00	#8	8	17							

20-20 2nd and 3rd Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	12	$2,549.44
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	16.00	16.00	12.00	21.33	#8	4	14	$71.35	$93.60	$27.02	$91.20	$283.17	12	$3,398.02
C5	16.00	16.00	12.00	21.33	#8	4	14							
C6	16.00	16.00	12.00	21.33	#8	4	14							

20-20 4th and 5th Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	12	$2,549.44
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	16.00	16.00	12.00	21.33	#8	4	14	$71.35	$93.60	$27.02	$91.20	$283.17	12	$3,398.02
C5	16.00	16.00	12.00	21.33	#8	4	14							
C6	16.00	16.00	12.00	21.33	#8	4	14							

20-20 Top Floor Columns

Column type	b(in)	h(in)	L(ft)	Volume (ft^3)	# bar	# of bars	total ties	concrete cost	Rebar cost	Tie cost	formwork	total for 1	# of columns	total for floor
C1	12.00	12.00	12.00	12.00	#6	4	18	$40.13	$52.32	$25.20	$94.80	$212.45	24	$5,098.88
C2	12.00	12.00	12.00	12.00	#6	4	18							
C3	12.00	12.00	12.00	12.00	#6	4	18							
C4	12.00	12.00	12.00	12.00	#6	4	18							
C5	12.00	12.00	12.00	12.00	#6	4	18							
C6	12.00	12.00	12.00	12.00	#6	4	18							

Please note that for all floors there are only two true columns options which correspond to whether or not the column lies with Girder 1 or Girder 2. These are the only sizings that will be explored at this point in time

total Column Price for building $40,338.65

B.2 Office Layout – Evaluating Options

B.2.1 Office Scheme Relative Comparison

OFFICE SCENARIO RANKINGS

		# Members	Connections	Floor Depth	Mechanical	Versatility	Cost	Total	Overall Ranking
Composite System w/ W-Flange Sections	*16' Spacing*	6	2	1	3	2	4	18	5
	20' Spacing	3	2	3	3	1	5	17	4
Composite w/ Joist w/ Joist Sections	*16' Spacing*	---	---	---	---	---	---	---	---
	20' Spacing	3	2	1	1	1	3	11	1
Reinforced Slab - Steel Frame	*16' Spacing*	---	---	---	---	---	---	---	---
	20' Spacing	3	2	3	3	1	6	18	5
Reinforced System	*16' Spacing*	2	1	5	3	2	2	15	3
	20' Spacing	1	1	6	3	1	1	13	2

Notes:

1. Grading for # members is based on ranking each scenario from the one with the least number of members to the one with the most number of members (1-6).
2. Grading for the connections is based on the types of connections. Reinforced concrete received a value of 1 and steel received a value of 2 because steel connections would involve either bolt connections or welded (moment) connections.
3. Grading for floor depth was based on ranking each scenario from the one with the shallowest floor depth (deepest girder depth plus slab thickness) to the one with the greatest floor depth (1-6).
4. Grading for mechanical was based on the ease of passing mechanical considerations such as ducting, plumbing, and electrical through the support members. The open web joist option received a value of 1 because of the available openings. All other options received a value of 3. They received a value of 3 because all though consideration could be given to run mechanical through the webs of wide-flange section and concrete members, design considerations as well as labor would need to be evaluated, making those options slightly less appealing when consideration the placement of mechanical.
5. Grading for visibility was based on the versatility of the option. The option with the 16ft column spacing received a value of 2 because of the relative closeness of the columns and the added number of columns on each floor. The option with the 20ft column spacing received a value of 1 because of the spacing and the smaller number of required columns.
6. Grading for cost was based on ranking each scenario from the one with the lowest costs to the one with the highest cost (1-6).
7. The total is a summation of the grading for each criteria for all the scenarios.
8. The overall ranking is based on ranking the scenario with the lowest total to the highest total (1-6). The scenario with the lowest ranking (lowest total) is determined to be the most effective based on the criteria offered.

B.3 Office Layout – Gravity and Lateral Loads

B.3.1 Steel Cost

B.3.1.1 Steel Cost Breakdown – Option 2 (Open-Web Joist)

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
AB-2.5	1	14VC 1800	1	17	34	10	10	20	200	1.7	340	1464.50	2489.65	1.32	448.8
BC-2.5	1	14VC 1800	1	19	24	5	5	24	120	1.14	120	1464.50	1669.53	1.32	158.4
AB-2.5	2,3,4	12VC 1600	3	13	26	10	30	20	600	3.9	780	1464.50	5711.55	1.32	1029.6
BC-2.5	2,3,4	14VC 1500	3	16	32	5	15	24	360	2.88	480	1464.50	4217.76	1.32	633.6
AB-2.5	5	14VC 1800	1	17	34	10	10	20	200	1.7	340	1464.50	2489.65	1.32	448.8
BC-2.5	5	14VC 1800	1	19	24	5	5	24	120	1.14	120	1464.50	1669.53	1.32	158.4
AB-2.5	6	14VC 1100	1	10	18	10	10	20	200	1	180	1464.50	1464.5	1.32	237.6
BC-2.5	6	14VC 1300	1	14	28	5	5	24	120	0.84	140	1464.50	1230.18	1.32	184.8
AB-2.5	Roof	14VC 1100	1	10	18	10	10	20	200	1	180	1464.50	1464.5	1.32	237.6
BC-2.5	Roof	14VC 1300	1	14	28	5	5	24	120	0.84	140	1464.50	1230.18	1.32	184.8
						Total =	105		Total =	16.14	2820		**23637.03**		**3722.4**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
AB3	1	W14x30	1	30	42	12	12	20	240	3.6	504	32.77	7864.80	1.32	665.28
BC3	1	W14x30	1	30	42	6	6	24	144	2.16	252	32.77	4718.88	1.32	332.64
AB3	2,3,4	W12x30	3	30	42	12	36	20	720	10.8	1512	32.75	23580.00	1.32	1995.84
BC3	2,3,4	W12x30	3	30	42	6	18	24	432	6.48	756	32.75	14148.00	1.32	997.92
AB3	5	W14x30	1	30	42	12	12	20	240	3.6	504	32.77	7864.80	1.32	665.28
BC3	5	W14x30	1	30	42	6	6	24	144	2.16	252	32.77	4718.88	1.32	332.64
AB3	6, Roof	W12x22	2	22	32	12	24	20	480	5.28	768	24.86	11932.80	1.32	1013.76
BC3	6, Roof	W12x22	2	22	32	6	12	24	288	3.168	384	24.86	7159.68	1.32	506.88
C45	1	W16x50	1	50	NA	10	10	20	200	5	NA	52.25	10450.00	NA	NA
D45	1	W14x30	1	30	NA	10	10	20	200	3	NA	32.77	6554.00	NA	NA
C45	2,3,4	W16x40	3	40	NA	10	30	20	600	12	NA	42.75	25650.00	NA	NA
D45	2,3,4	W16x26	3	26	NA	10	30	20	600	7.8	NA	28.40	17040.00	NA	NA
C45	5	W16x50	1	50	NA	10	10	20	200	5	NA	52.25	10450.00	NA	NA
D45	5	W14x30	1	30	NA	10	10	20	200	3	NA	32.44	6488.00	NA	NA
C45	6	W16x36	1	36	NA	10	10	20	200	3.6	NA	38.75	7750.00	NA	NA
D45	6	W14x22	1	22	NA	10	10	20	200	2.2	NA	24.75	4950.00	NA	NA
C45	Roof	W16x36	1	36	NA	10	10	20	200	3.6	NA	38.75	7750.00	NA	NA
D45	Roof	W14x22	1	22	NA	10	10	20	200	2.2	NA	24.75	4950.00	NA	NA
						Total =	266		Total =	84.65	4932		**184019.84**		**6510.24**

Bracing

Bracing Location	(Floor Location)	Bracing Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ton	Total Cost
A1-2	Basement	HSS6x4x1/4	1	15.6	NA	8	8	18.03	144.24	1.13	NA	2500.00	2812.68
A1-2	1,2	HSS6x4x1/4	2	15.6	NA	8	16	16.83	269.28	2.10	NA	2500.00	5250.96
A1-2	3,4	HSS3x3x3/16	2	6.85	NA	8	16	15.62	249.92	0.86	NA	2500.00	2139.94
A1-2	5,6	HSS3x3x3/16	2	6.85	NA	8	16	15.62	249.92	0.86	NA	2500.00	2139.94
AB1	Basement	HSS6x5x5/16	1	21.2	NA	8	8	18.03	144.24	1.53	NA	2500.00	3822.36
AB1	1,2	HSS6x5x5/16	2	21.2	NA	8	16	16.83	269.28	2.85	NA	2500.00	7135.92
AB1	3,4	HSS6x3x5/16	2	16.9	NA	8	16	15.62	249.92	2.11	NA	2500.00	5279.56
AB1	5,6	HSS4x4x3/16	2	9.4	NA	8	16	15.62	249.92	1.17	NA	2500.00	2936.56
B1-1.5	Basement	HSS6x6x1/2	1	35.1	NA	4	4	18.03	72.12	1.27	NA	2500.00	3164.27
B1-1.5	1,2	HSS6x6x1/2	2	35.1	NA	4	8	16.83	134.64	2.36	NA	2500.00	5907.33
B1-1.5	3,4	HSS6x5x1/4	2	17.3	NA	4	8	15.62	124.96	1.08	NA	2500.00	2702.26
B1-1.5	5,6	HSS5x4x3/16	2	10.7	NA	4	8	15.62	124.96	0.67	NA	2500.00	1671.34
BC3	Basement	HSS6x6x5/16	1	23.3	NA	4	4	19.21	76.84	0.90	NA	2500.00	2237.97
BC3	1,2	HSS6x6x5/16	2	23.3	NA	4	8	18.09	144.72	1.69	NA	2500.00	4214.97
BC3	3,4	HSS6x4x5/16	2	19	NA	4	8	16.97	135.76	1.29	NA	2500.00	3224.30
BC3	5,6	HSS4x4x1/4	2	12.2	NA	4	8	16.97	135.76	0.83	NA	2500.00	2070.34
						Total =	168		Total =	22.68	0		**56710.69**

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Roof	1.5VL22	6400	1	6400	1.76	11264
Floors 1-6	1.5VL22	5680	6	34080	1.76	59980.8
			Total =	40480	Total =	**71244.80**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members/Floor	Number of Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit cost / ft	Total Cost
A1	Basement	W14x61	1	61	NA	4	4	15	60	1.83	NA	63.75	3825.00
A1	1,2	W14x43	1	43	NA	4	4	27	108	2.32	NA	45.70	4935.60
A1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.44	3114.24
A1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
A2	Basement	W14x53	1	53	NA	8	8	15	120	3.18	NA	55.75	6690.00
A2	1,2	W14x43	1	43	NA	8	8	27	216	4.64	NA	45.70	9871.20
A2	3,4	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
A2	5,6	W14x22	1	22	NA	8	8	24	192	2.11	NA	24.75	4752.00
B1	Basement	W14x61	1	61	NA	4	4	15	60	1.83	NA	63.75	3825.00
B1	1,2	W14x43	1	43	NA	4	4	27	108	2.32	NA	45.70	4935.60
B1	3,4	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.77	3145.92
B1	5,6	W14x22	1	22	NA	4	4	24	96	1.06	NA	24.75	2376.00
B1.5	Basement	W14x109	1	109	NA	4	4	15	60	3.27	NA	111.75	6705.00
B1.5	1,2	W14x90	1	90	NA	4	4	27	108	4.86	NA	92.75	10017.00
B1.5	3,4	W14x48	1	48	NA	4	4	24	96	2.30	NA	50.75	4872.00
B1.5	5,6	W14x30	1	30	NA	4	4	24	96	1.44	NA	32.44	3114.24
B2	Basement	W14x74	1	74	NA	8	8	15	120	4.44	NA	76.75	9210.00
B2	1,2	W14x61	1	61	NA	8	8	27	216	6.59	NA	63.75	13770.00
B2	3,4	W14x43	1	43	NA	8	8	24	192	4.13	NA	45.70	8774.40
B2	5,6	W14x30	1	30	NA	8	8	24	192	2.88	NA	32.77	6291.84
					Total =		112		Total =	56.02	0		**118892.88**

Concrete

Number of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Cost / cyd. (total)	Total Cost
7	5680	2.75	110	1002.3	1012.41	119.70	132715.57
1	6400	2.75	110	161.3	162.96	119.70	21362.67
				GRAND TOTAL =	**1175.37**		**154078.23**

Note: Concrete, Ready Mix: $101.25/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f - screen finish

Grand Total for Office Building Cost (20ft Column Spacing) - Joist Alternative = $618,816.11

B.3.2 Concrete Cost

The concrete office structure was redesigned for lateral such that one idealized option was generated. The cost of this structure is broken down in the following section and in the spreadsheets included after that detailing the member sizes and costs of each part of the element. Additionally, a comparison is provided to illustrate the weight of the cost given to the gravity load.

Lateral and Gravity Load Costs

Cost of slabs	= $263,547.18
Cost of T-Beams	= $108,694.46
Cost of Girders	= $53,904.28
Cost of Columns	= $61,122.90
Total Cost	**= $487,268.81**

Frame Cost - Lateral			*Frame Cost - Gravity Only*	
Slab	$263,547.18		**Slab**	$263,547.18
T-Beams	$108,694.46		**T-Beams**	$80,240.86
Girders	$53,904.28		**Girders**	$46,368.69
Columns	$61,122.90		**Columns**	$40,338.65
Total	$487,268.81		**Total**	$430,495.37
	difference	$56,773.43	**Percentage of gravity loading (in terms of cost)**	88.35

B.3.2.1 Concrete Cost Breakdown – Option 2

First Floor T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	18	65	81.25	#6/ #7	1/1	20	#7	2	6.67	#6	3	25	#7	3	8.33	90	$271.74	$261.02	$604.50	$216.00	$1,353.26	2.00	$2,706.52
B3-B4-B3	14	18	65	113.75	#6	3	20	#7	2	6.67	#6	4	25	#7	2	8.33	98	$380.43	$388.46	$516.75	$235.20	$1,520.84	9.00	$13,687.53
								#8	2	6.67				#8	2	8.33								

Second Floor T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	18	65	81.25	#6/ #7	1/1	20	#7	2	6.67	#6	3	25	#7	3	8.33	90	$271.74	$261.02	$604.50	$216.00	$1,353.26	2.00	$2,706.52
B3-B4-B3	12	18	65	97.5	#6	3	20	#8	3	6.67	#6	4	25	#7	2	8.33	98	$326.08	$374.98	$516.75	$235.20	$1,453.02	9.00	$13,077.15
														#8	2	8.33								

Third Floor T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	18	65	81.25	#6/ #7	1/1	20	#7	3	6.67	#6	3	25	#7	3	8.33	90	$271.74	$280.76	$604.50	$216.00	$1,373.00	2.00	$2,746.00
B3-B4-B3	12	18	65	97.50	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$326.08	$318.19	$516.75	$235.20	$1,396.23	9.00	$12,566.05

Fourth Floor T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	18	65	81.25	#6/ #7	1/1	20	#7	3	6.67	#6	3	25	#7	3	8.33	90	$271.74	$280.76	$604.50	$216.00	$1,373.00	2.00	$2,746.00
B3-B4-B3	12	18	65	97.50	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$326.08	$318.19	$516.75	$235.20	$1,396.23	9.00	$12,566.05

Fifth Floor T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	18	65	81.25	#6/ #7	1/1	20	#7	3	6.67	#6	3	25	#7	3	8.33	90	$271.74	$280.76	$604.50	$216.00	$1,373.00	2.00	$2,746.00
B3-B4-B3	12	18	65	97.50	#6	3	20	#6	2	6.67	#6	4	25	#7	4	8.33	98	$326.08	$318.19	$516.75	$235.20	$1,396.23	9.00	$12,566.05

Roof T-Beams

Dimensions					As + 1			As - 1			As + 2			As - 2										
beam	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	# of stirrups	Concrete Cost	Rebar Cost	Formwork Cost	Stirrup Cost	Cost of beam	# per floor	Cost per floor
B1-B2-B1	10	16	65	72.22	#5	3	20	#5	3	6.67	#6/#7	2/1	25	#6	4	8.33	90	$241.54	$254.23	$604.50	$216.00	$1,316.28	2.00	$2,632.56
B3-B4-B3	10	18	65	81.25	#7	2	20	#5	4	6.67	#6	3	25	#7	3	8.33	90	$271.74	$279.29	$516.75	$216.00	$1,283.78	9.00	$11,553.98

Girder Cost Summary For Lateral Design

Floors 1-6 G1 & G2 (Same type of girder for all floors)

Sizing Summary					As -1			As + 1			As - 2			As + 2			As -3			As +3			As -4		
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)
G1	12	24	100	200.00	#6	2	6.66666667	#5	3	15	#5	3	6.66667	#5	3	15	#6	4	6.66667	#5	3	15	#7	4	6.6667
G2	20	26	100	361.11	#6	4	6.66666667	#7	4	15	#6	4	6.67	#7	4	15	#7	4	6.67	#6	2	15	#7	6	6.67
																	#6	2	6.67				#5	2	6.67

Roof G1 and G2 sizes

Sizing Summary					As -1			As + 1			As - 2			As + 2			As -3			As +3			As -4		
girder type	b (in)	h (in)	L (ft)	Volume (ft^3)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)	bar #	# of bars	L (ft)
G1	12	24	100.00	200.00	#6	2	6.66666667	#7	2	15	#6	2	6.66666667	#7	2	15	#7	2	6.66666667	#7	2	15	#7	2	6.66666667
																							#6	2	
G2	16	27	100.00	300.00	#7	2	6.66666667	#7	3	15	#7	2	6.66666667	#7	3	15	#6	5	6.66666667	#7	3	15	#7	4	6.66666667

Column Sizing for Lateral Load with Girder Size Considerations

Floor	number	b	h	Rebar	Ties	Cost/column	Total/Floor
1	24	24	24	8 #8	17	$601.17	$14,428.00
2	20	24	24	8 #8	14	$465.95	$9,319.07
	4	16	16	4 #8	14	$283.17	$1,132.67
3	4	12	12	4 #6	14	$212.45	$849.81
	4	16	16	4 #8	14	$283.17	$1,132.67
	16	24	24	8 #8	14	$465.95	$7,455.25
4	4	12	12	4 #6	14	$212.45	$849.81
	4	16	16	4 #8	14	$283.17	$1,132.67
	16	24	24	8 #8	14	$465.95	$7,455.25
5	4	12	12	4 #6	14	$212.45	$849.81
	4	16	16	4 #8	14	$283.17	$1,132.67
	16	24	24	8 #8	14	$465.95	$7,455.25
6	16	12	12	4 #6	14	$212.45	$3,399.25
	16	16	16	4 #8	14	$283.17	$4,530.69
						Total	$61,122.90
						Original	$40,338.65
						Difference	$20,784.25

B.4 Prison Cellblock Layout – Gravity Loads

B.4.1 Steel Cost

- OPTION #1A Composite Beam Design – No Intermediate Columns
  Cost of Slabs = \$460,000
  Cost of Member = \$110,000
  Cost of Girders = \$202,000
  Cost of Columns = \$87,000
  Total Cost of Option 1A = \$859,000

  Number of different beam member sizes = 6
  Number of different girder member sizes = 9
  Number of different column member sizes = 8

- OPTION #1B Composite Open –Web Joist Design – No Intermediate Columns
  Cost of Slabs = \$210,000
  Cost of Open Web Joists = \$180,000
  Cost of Girders = \$107,000
  Cost of Columns = \$83,000
  Total Cost of Option 1B = \$580,000

  Number of different open web joist sizes = 5
  Number of different girder member sizes = 7
  Number of different column member sizes = 7

- OPTION #2 Composite Beam Design – 1 Row of Intermediate Columns
  Cost of Slabs = \$460,000
  Cost of Member = \$110,000
  Cost of Girders = \$112,000
  Cost of Columns = \$104,000
  Total Cost of Option 2 = \$786,000

  Number of different beam member sizes = 6
  Number of different girder member sizes = 8
  Number of different column member sizes = 9

- OPTION #3 Composite Beam Design – 2 Rows of Intermediate Columns
  Cost of Slabs = \$460,000
  Cost of Member = \$110,000
  Cost of Girders = \$114,000
  Cost of Columns = \$111,000
  Total Cost of Option 3 = \$795,000

Number of different beam member sizes	= 6
Number of different girder member sizes	= 6
Number of different column member sizes	= 6

Based on cost and nominal member sizes, Option 1B is the most feasible choice. With respect to serviceability and functionality with regards for security within the structure, option number one increases visibility by eliminating the center columns in the dayroom open space.

B.4.1.1 Steel Cost Breakdown – Option 1 (Composite WF-shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
D56 (typical)	1,2 - Dayroom	W10x15	3	15	22	12	36	19.54	703.50	15.83	792	21.11	14850.91	1.29	1021.68
D45 (typical)	1,2 - Dayroom	W10x15	3	15	22	30	90	16.00	1440.00	32.40	1980	21.11	30398.40	1.29	2554.2
A.5-45 (typical)	Cell Area	W12x14	5	14	20	8	40	16.00	640.00	22.40	800	17.36	11110.40	1.29	1032
D.25F.25-1.75 2.75 (typ	Cell Area	W12x30	5	30	42	4	20	24.00	480.00	36.00	840	33.00	15840.00	1.29	1083.6
A.5-56 (typical)	Shower Area	W12x19	5	19	26	4	20	19.54	390.83	18.56	520	22.00	8598.35	1.29	670.8
D.5F.5-1.5 2.5	Cell Area	W14x26	5	26	NA	4	20	24.00	480.00	31.20	NA	28.43	13646.40	1.29	NA
B45	Cell Area	W12x14	3	14	20	6	18	16.00	288.00	6.05	360	17.36	4999.68	1.29	464.4
B56	Cell Area	W14x19	3	19	26	2	6	19.54	117.24	3.34	156	22.00	2579.28	1.29	201.24
						Total =	250		Total =	165.78	5448		**102023.42**		**7027.92**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Cell Area	W12x14	5	14	NA	8	40	16.00	640.00	22.40	NA	17.36	11110.40
B45	Cell Area	W12x14	2	14	NA	6	12	16.00	192.00	2.69	NA	17.36	3333.12
B56	Cell Area	W12x19	2	19	NA	2	4	16.00	64.00	1.22	NA	17.36	1111.04
AB4	Cell Area	W14x22	5	22	NA	6	30	10.00	300.00	16.50	NA	24.75	7425.00
AB5	Cell Area	W14x22	5	22	NA	2	10	10.00	100.00	5.50	NA	24.75	2475.00
B.5-1 - D2	Cell Area	W14x22	5	22	NA	2	10	16.00	160.00	8.80	NA	24.75	3960.00
D.5-1.5 - D2	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
F.5F-2.5 3	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
D2-F3	Cell Area	W14x34	5	34	NA	8	40	24.00	960.00	81.60	NA	36.75	35280.00
AB6	Shower Area	W14x22	5	22	NA	1	5	10.00	50.00	2.75	NA	24.75	1237.50
BD2	1,2 - Dayroom	W16x26	3	26	NA	2	6	17.92	107.52	4.19	NA	28.40	3053.57
BF3	1,2 - Dayroom	W21x101	3	101	NA	2	6	35.83	214.98	32.57	NA	101.41	21801.12
BH4	1,2 - Dayroom	W27x114	3	114	NA	2	6	53.75	322.50	55.15	NA	113.83	36710.18
BH5	1,2 - Dayroom	W27x129	3	129	NA	2	6	53.75	322.50	62.40	NA	130.00	41925.00
BH6	1,2 - Dayroom	W27x129	3	129	NA	1	3	53.75	161.25	31.20	NA	130.00	20962.50
						Total =	218		Total =	352.97	0		**201756.42**

(Continued on the next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
A2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
A4	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A4	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
A6	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
A6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
B2	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
B2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x43	1	43	NA	4	4	15.00	60.00	1.29	NA	45.70	2742.00
B4	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
B4	Level 2	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
B5	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
B5	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B5	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B6	Basement	W14x48	1	48	NA	1	1	15.00	15.00	0.36	NA	50.75	761.25
B6	Level 1	W14x34	1	34	NA	1	1	24.00	24.00	0.41	NA	36.75	882.00
B6	Level 2	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
C.5-.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
C.5-.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
D2	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
D2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
D.5-1.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
F3	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
F3	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
F.5-2.5	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
F.5-2.5	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x43	1	38	NA	2	2	15.00	30.00	0.57	NA	45.70	1371.00
H4	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
H4	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
H5	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.75	1912.50
H5	Level 1	W14x61	1	61	NA	2	2	24.00	48.00	1.46	NA	63.75	3060.00
H5	Level 2	W14x61	1	61	NA	2	2	24.00	48.00	1.46	NA	63.75	3060.00
H6	Basement	W14x68	1	68	NA	1	1	15.00	15.00	0.51	NA	70.75	1061.25
H6	Level 1	W14x68	1	68	NA	1	1	24.00	24.00	0.82	NA	70.75	1698.00
H6	Level 2	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
						Total =	129		Total =	39.58	0		**86668.77**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($5.90/S.F.)	Finishing ($0.29/S.F.)	Total Cost
4	9513	4.5	150	2140.425	1585.50	157677.98	224506.80	11035.08	393219.86
2	3200	4.5	150	360	266.67	26520.00	37760.00	1856.00	66136.00
									459355.86

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Prison Cell Block Cost (Option #1) = $856,832.39

B.4.1.2 Steel Cost Breakdown – Option 1 (Open-Web Joists)

Total Beam, Girder, Column Steel, and Concrete Weight - Joist Alternative

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
BH6	Dayroom	36VC 900	3	27	28	3	9	53.69	483.21	6.52	252	1724.00	11246.23	1.32	332.64
BH5	Dayroom	36VC 800	3	22	30	2	6	53.69	322.14	3.54	180	1724.00	6109.06	1.32	237.60
BH4.5	Dayroom	33VC 700	3	20	46	6	18	53.69	966.42	9.66	828	1724.00	16661.08	1.32	1092.96
BF3	Dayroom	24VC 800	3	13	26	6	18	35.83	644.94	4.19	468	1464.50	6139.35	1.32	617.76
A.5 - 5 6	Shower Area	14VC 1100	5	10	18	2	10	19.55	195.54	0.98	180	1464.50	1431.85	1.32	237.60
A.5 - 4 5	Cell Area	14VC 1100	5	10	18	8	40	16.00	640.00	3.20	720	1464.50	4686.40	1.32	950.40
D.25F.25 - 1.75-2.75	Cell Area	14VC 1300	5	14	28	4	20	24.00	480.00	3.36	560	1464.50	4920.72	1.32	739.20
						Total =	121		Total =	31.46	3188		**51194.69**		**4208.16**

Girder

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Cell Area	W12x14	5	14	NA	8	40	16.00	640.00	22.40	NA	16.75	10720.00
A56	Cell Area	W12x19	5	19	NA	2	10	19.75	197.50	9.38	NA	21.75	4295.63
AB4	Cell Area	W14x22	5	22	NA	6	30	10.00	300.00	16.50	NA	24.75	7425.00
AB5	Cell Area	W14x22	5	22	NA	2	10	10.00	100.00	5.50	NA	24.75	2475.00
B.5-1 - D2	Cell Area	W14x22	5	22	NA	2	10	16.00	160.00	8.80	NA	24.75	3960.00
C.5-.5 - D.5-1.5	Cell Area	W14x26	5	26	NA	4	20	16.00	320.00	20.80	NA	28.43	9097.60
D.5F.5-1.5 2.5	Cell Area	W12x30	5	30	NA	4	20	24.00	480.00	36.00	NA	32.75	15720.00
F.5F-2.5 3	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
AB6	Shower Area	W14x22	5	22	NA	1	5	10.00	50.00	2.75	NA	24.75	1237.50
B45	Cell Area	W14x26	5	26	NA	6	30	16.00	480.00	31.20	NA	28.43	13646.40
H45	1,2 Dayroom	W14x26	3	26	NA	2	6	16.00	96.00	3.74	NA	28.43	2729.28
B56	Cell Area	W14x34	5	34	NA	2	10	19.75	197.50	16.79	NA	36.70	7248.25
H56	1,2 - Dayroom	W14x34	3	34	NA	2	6	19.75	118.50	6.04	NA	36.70	4348.95
D2-F3	Cell Area	W14x30	5	30	NA	2	10	24.00	240.00	18.00	NA	32.77	7864.80
F3-H4	Cell Area	W14x43	5	43	NA	2	10	24.00	240.00	25.80	NA	45.70	10968.00
						Total =	237		Total =	236.71	0		**107422.41**

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Cell Area	1.5VL22	3200	2	6400	1.76	11264
Day Room (BH6)	1.5VL22	9513	3	28539	1.76	50228.64
			Total =	34939	Total =	**61492.64**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x22	1	22	NA	2	2	15.00	30.00	0.33	NA	24.75	742.50
A2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x22	1	22	NA	12	12	15.00	180.00	1.98	NA	24.75	4455.00
A4	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A4	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x22	1	22	NA	2	2	15.00	30.00	0.33	NA	24.75	742.50
A5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x22	1	22	NA	1	1	15.00	15.00	0.17	NA	24.75	371.25
A6	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
A6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
B2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x43	1	43	NA	4	4	15.00	60.00	1.29	NA	45.70	2742.00
B4	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
B4	Level 2	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
B5	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
B5	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B5	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
B6	Basement	W14x43	1	43	NA	1	1	15.00	15.00	0.32	NA	45.70	685.50
B6	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
B6	Level 2	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
C.5-.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
C.5-.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
D2	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
D2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x22	1	22	NA	2	2	15.00	30.00	0.33	NA	24.75	742.50
D.5-1.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.25	1897.50
F3	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
F3	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
F.5-2.5	Basement	W14x26	1	26	NA	4	4	15.00	60.00	0.78	NA	28.43	1705.80
F.5-2.5	Level 1	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
F.5-2.5	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.25	1897.50
H4	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
H4	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H5	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.75	1912.50
H5	Level 1	W14x61	1	61	NA	2	2	24.00	48.00	1.46	NA	63.75	3060.00
H5	Level 2	W14x63	1	53	NA	2	2	24.00	48.00	1.27	NA	53.25	2556.00
H6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.75	956.25
H6	Level 1	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
H6	Level 2	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
						Total =	129		Total =	37.83	0		**82877.58**

Total Beam, Girder, Column Steel, and Concrete Weight - Joist Alternative

Concrete

Number of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Cost / cyd. (total)	Total Cost
3	9513	2.75	110	719.420625	726.69	119.70	95260.80
2	3200	2.75	110	161.3333333	162.96	119.70	21362.67
				GRAND TOTAL =	889.65		**116623.47**

Note: Concrete, Ready Mix: $101.25/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f - screen finish

Grand Total for Prison Cell Block Cost = $423,818.95

B.4.1.3 Steel Cost Breakdown – Option 1 (Non-composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
C45	Floor	W12x35	2	35	NA	30	60	15.98	958.80	33.56	NA	37.70	36146.76
	Roof	W12x35	1	35	NA	30	30	15.98	479.40	8.39	NA	37.70	18073.38
C56	Floor	W12x40	2	40	NA	12	24	19.55	469.20	18.77	NA	42.75	20058.30
	Roof	W12x40	1	40	NA	12	12	19.55	234.60	4.69	NA	42.75	10029.15
C.5-0.5 - D.5-1.5	Floor	W12x30	4	30	NA	2	8	15.90	127.20	7.63	NA	33.00	4197.60
	Roof	W12x30	1	30	NA	2	2	15.90	31.80	0.48	NA	33.00	1049.40
D.5-1.5 - F.5-2.5	Floor	W12x45	4	45	NA	4	16	24.01	384.16	34.57	NA	47.75	18343.64
	Roof	W12x40	1	40	NA	4	4	24.01	96.04	1.92	NA	42.75	4105.71
						Total =	156		Total =	110.01	0		**112003.94**

Girder

Beam Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Floor	W12x30	4	30	NA	6	24	15.98	383.52	23.01	NA	33.00	12656.16
	Roof	W12x30	1	30	NA	6	6	15.98	95.88	1.44	NA	33.00	3164.04
A56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
B45	Floor	W12x35	4	35	NA	6	24	15.98	383.52	26.85	NA	37.70	14458.70
	Roof	W12x35	1	35	NA	6	6	15.98	95.88	1.68	NA	37.70	3614.68
B56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
AB4	Floor	W14x22	4	22	NA	6	24	10.00	240.00	10.56	NA	24.75	5940.00
	Roof	W14x22	1	22	NA	6	6	10.00	60.00	0.66	NA	24.75	1485.00
AB5	Floor	W14x22	4	22	NA	2	8	10.00	80.00	3.52	NA	24.75	1980.00
	Roof	W14x22	1	22	NA	2	2	10.00	20.00	0.22	NA	24.75	495.00
AB6	Floor	W14x22	4	22	NA	1	4	10.00	40.00	1.76	NA	24.75	990.00
	Roof	W14x22	1	22	NA	1	1	10.00	10.00	0.11	NA	24.75	247.50
BD2	Floor	W14x30	2	30	NA	2	4	17.92	71.68	2.15	NA	32.77	2348.95
	Roof	W14x26	1	26	NA	2	2	17.92	35.84	0.47	NA	28.43	1018.93
BF3	Floor	W24x68	2	68	NA	2	4	35.83	143.32	9.75	NA	69.70	9989.40
	Roof	W24x68	1	68	NA	2	2	35.83	71.66	2.44	NA	69.70	4994.70
BH4	Floor	W27x94	2	94	NA	2	4	53.75	215.00	20.21	NA	94.20	20253.00
	Roof	W27x84	1	84	NA	2	2	53.75	107.50	4.52	NA	84.70	9105.25
BH5	Floor	W27x94	2	94	NA	2	4	53.75	215.00	20.21	NA	94.20	20253.00
	Roof	W27x94	1	94	NA	2	2	53.75	107.50	5.05	NA	94.20	10126.50
BH6	Floor	W27x102	2	102	NA	1	2	53.75	107.50	10.97	NA	102.50	11018.75
	Roof	W27x102	1	102	NA	1	1	53.75	53.75	2.74	NA	102.50	5509.38
B.5-1 - D2	Floor	W14x22	4	22	NA	2	8	15.90	127.20	5.60	NA	24.75	3148.20
	Roof	W14x22	1	22	NA	2	2	15.90	31.80	0.35	NA	24.75	787.05
D2-F3	Floor	W16x31	4	31	NA	4	16	24.01	384.16	23.82	NA	33.77	12973.08
	Roof	W16x31	1	31	NA	4	4	24.01	96.04	1.49	NA	33.77	3243.27
D.5-1.5 - D2	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
F.5-1.5 - F2	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
						Total =	222		Total =	223.82	0		**187887.80**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x43	1	43	NA	4	4	15.00	60.00	1.29	NA	45.70	2742.00
	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
	Level 2	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
B5	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B6	Basement	W14x43	1	43	NA	1	1	15.00	15.00	0.32	NA	45.70	685.50
	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
	Level 2	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
H5	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.75	1912.50
	Level 1	W14x61	1	61	NA	2	2	24.00	48.00	1.46	NA	63.75	3060.00
	Level 2	W14x53	1	53	NA	2	2	24.00	48.00	1.27	NA	55.25	2652.00
H6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.75	956.25
	Level 1	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
	Level 2	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
						Total =	129		Total =	38.97	0		**85251.36**

Slabs

Slab Location	Slab Section (in)	Slab Area (SF)	Number of Floors	Volume of Conc. (CY)	Tons of Reinf. (Steel / SF)	Total Tons of Steel	Cost of Conc. ($81/CY)	Cost of Reinf. Steel (elevated) $1270/ton	Cost of Reinf. Steel (on grade) $1290/ton	Placing Slab on Grade ($19.95/CY)	Placing Elevated Slab ($18.45/CY)	Finish Slab ($0.29 /SF)	Formwork ($5.90/SF)	TOTAL COST
Slab on Grade	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00		$ 4,296.02	$ 2,343.51		$ 2,759.35		
Elevated Slab	4	25412	2	627.4567901	0.00035	8.8942	$ 50,824.00	$ 11,295.63			$ 11,576.58	$ 7,369.48	$ 149,930.80	
Roof	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00	$ 4,229.42			$ 2,167.31	$ 2,759.35	$ 56,138.50	
	Total=	44442	4	862.3950617		Total=	$ 69,854.00	$ 15,525.05	$ 4,296.02	$ 2,343.51	$ 13,743.88	$ 12,888.18	$ 206,069.30	**$324,719.95**

GRAND TOTAL FOR OPTION #1 (REINFORCED SLABS, STEEL FRAME) = $709,863.05

B.4.1.4 Steel Cost Breakdown – Option 2 (Composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
D56 (typical)	1,2 - Dayroom	W10x15	3	15	22	12	36	19.54	703.50	15.83	792	21.11	14850.91	1.29	1021.68
D45 (typical)	1,2 - Dayroom	W10x15	3	15	22	30	90	16.00	1440.00	32.40	1980	21.11	30398.40	1.29	2554.2
A.5-45 (typical)	Cell Area	W12x14	5	14	20	8	40	16.00	640.00	22.40	800	17.36	11110.40	1.29	1032
D.25F.25-1.75 2.75 (typ	Cell Area	W12x30	5	30	42	4	20	24.00	480.00	36.00	840	33.00	15840.00	1.29	1083.6
A.5-56 (typical)	Shower Area	W12x19	5	19	26	4	20	19.54	390.83	18.56	520	22.00	8598.35	1.29	670.8
D.5F.5 - 1.5 2.5	Cell Area	W14x26	5	26	NA	4	20	24.00	480.00	31.20	NA	28.43	13646.40		
B45	Cell Area	W12x14	3	14	20	6	18	16.00	288.00	6.05	360	17.36	4999.68	1.29	464.4
B56	Cell Area	W14x19	3	19	26	2	6	19.54	117.24	3.34	156	22.00	2579.28	1.29	201.24
						Total =	250	Total =		165.78	5448		**102023.42**		**7027.92**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Cell Area	W12x14	5	14	NA	8	40	16.00	640.00	22.40	NA	17.36	11110.40
B45	Cell Area	W12x14	5	14	NA	6	30	16.00	480.00	16.80	NA	17.36	8332.80
B56	Cell Area	W12x19	2	19	NA	2	4	16.00	64.00	1.22	NA	17.36	1111.04
AB4	Cell Area	W14x22	5	22	NA	6	30	10.00	300.00	16.50	NA	24.75	7425.00
AB5	Cell Area	W14x22	5	22	NA	2	10	10.00	100.00	5.50	NA	24.75	2475.00
B.5-1 - D2	Cell Area	W14x22	5	22	NA	2	10	16.00	160.00	8.80	NA	24.75	3960.00
D.5-1.5 - D2	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
D2-F3	Cell Area	W14x34	5	34	NA	4	20	24.00	480.00	40.80	NA	36.75	17640.00
F.5F-2.5 3	Cell Area	W14x26	5	26	NA	8	40	10.00	400.00	26.00	NA	28.43	11372.00
AB6	Shower Area	W14x22	5	22	NA	1	5	10.00	50.00	2.75	NA	24.75	1237.50
BD2	1,2 - Dayroom	W16x26	3	26	NA	2	6	17.92	107.52	4.19	NA	28.40	3053.57
BE4	1,2 - Dayroom	W16x36	3	36	NA	6	18	26.88	483.75	26.12	NA	38.75	18745.31
BE5	1,2 - Dayroom	W16x36	3	36	NA	4	12	26.88	322.50	17.42	NA	38.75	12496.88
BE6	1,2 - Dayroom	W16x40	3	40	NA	2	6	26.88	161.25	9.68	NA	42.75	6893.44
						Total =	251		Total =	211.17	0		**111538.93**

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Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
A2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	4	12	15.00	180.00	2.70	NA	32.77	5898.60
A4	Level 1	W14x22	1	22	NA	4	12	24.00	288.00	3.17	NA	24.75	7128.00
A4	Level 2	W14x22	1	22	NA	4	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
A6	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
A6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
B2	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
B2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x38	1	38	NA	4	4	15.00	60.00	1.14	NA	40.75	2445.00
B4	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
B4	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
B5	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
B5	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B6	Basement	W14x43	1	43	NA	1	1	15.00	15.00	0.32	NA	45.70	685.50
B6	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
B6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
C.5-.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
C.5-.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
D2	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
D2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
D.5-1.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
E3	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
E3	Level 1	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
E3	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
E4	Basement	W14x48	1	48	NA	2	2	15.00	30.00	0.72	NA	50.75	1522.50
E4	Level 1	W14x48	1	48	NA	2	2	24.00	48.00	1.15	NA	50.75	2436.00
E4	Level 2	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
E5	Basement	W14x58	1	58	NA	2	2	15.00	30.00	0.87	NA	60.75	1822.50
E5	Level 1	W14x58	1	58	NA	2	2	24.00	48.00	1.39	NA	60.75	2916.00
E5	Level 2	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
E6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.75	956.25
E6	Level 1	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
E6	Level 2	W14x43	1	43	NA	1	1	24.00	24.00	0.52	NA	45.70	1096.80
F3	Basement	W14x34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.75	1102.50
F3	Level 1	W14x30	1	26	NA	2	2	24.00	48.00	0.62	NA	32.77	1572.96
F3	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
F.5-2.5	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
F.5-2.5	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
H4	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
H4	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H5	Basement	W14x53	1	53	NA	2	2	15.00	30.00	0.80	NA	55.75	1672.50
H5	Level 1	W14x53	1	53	NA	2	2	24.00	48.00	1.27	NA	55.75	2676.00
H5	Level 2	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
H6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.75	956.25
H6	Level 1	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
H6	Level 2	W14x43	1	43	NA	1	1	24.00	24.00	0.52	NA	45.70	1096.80
						Total =	150		Total =	47.24	0		103243.35

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($5.90/S.F.)	Finishing ($0.29/S.F.)	Total Cost
4	9513	4.5	150	2140.425	1585.50	157677.98	224506.80	11035.08	393219.86
2	3200	4.5	150	360	266.67	26520.00	37760.00	1856.00	66136.00
									459355.86

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Prison Cell Block Cost (Option #2) = $783,189.48

B.4.1.5 Steel Cost Breakdown – Option 2 (Non-composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
C45	Floor	W12x35	2	35	NA	30	60	15.98	958.80	33.56	NA	37.70	36146.76
	Roof	W12x35	1	35	NA	30	30	15.98	479.40	8.39	NA	37.70	18073.38
C56	Floor	W12x40	2	40	NA	12	24	19.55	469.20	18.77	NA	42.75	20058.30
	Roof	W12x40	1	40	NA	12	12	19.55	234.60	4.69	NA	42.75	10029.15
C.5-0.5 - D.5-1.5	Floor	W12x30	4	30	NA	2	8	15.90	127.20	7.63	NA	33.00	4197.60
	Roof	W12x30	1	30	NA	2	2	15.90	31.80	0.48	NA	33.00	1049.40
D.5-1.5 - F.5-2.5	Floor	W12x45	4	45	NA	4	16	24.01	384.16	34.57	NA	47.75	18343.64
	Roof	W12x40	1	40	NA	4	4	24.01	96.04	1.92	NA	42.75	4105.71
						Total =	156		Total =	110.01	0		**112003.94**

Girder

Beam Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Floor	W12x30	4	30	NA	6	24	15.98	383.52	23.01	NA	33.00	12656.16
	Roof	W12x30	1	30	NA	6	6	15.98	95.88	1.44	NA	33.00	3164.04
A56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
B45	Floor	W12x35	4	35	NA	6	24	15.98	383.52	26.85	NA	37.70	14458.70
	Roof	W12x35	1	35	NA	6	6	15.98	95.88	1.68	NA	37.70	3614.68
B56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
AB4	Floor	W14x22	4	22	NA	6	24	10	240.00	10.56	NA	24.75	5940.00
	Roof	W14x22	1	22	NA	6	6	10	60.00	0.66	NA	24.75	1485.00
AB5	Floor	W14x22	4	22	NA	2	8	10	80.00	3.52	NA	24.75	1980.00
	Roof	W14x22	1	22	NA	2	2	10	20.00	0.22	NA	24.75	495.00
AB6	Floor	W14x22	4	22	NA	1	4	10	40.00	1.76	NA	24.75	990.00
	Roof	W14x22	1	22	NA	1	1	10	10.00	0.11	NA	24.75	247.50
BD2	Floor	W14x30	2	30	NA	2	4	17.92	71.68	2.15	NA	32.77	2348.95
	Roof	W14x26	1	26	NA	2	2	17.92	35.84	0.47	NA	28.43	1018.93
BE4	Floor	W16x36	2	36	NA	6	12	26.88	322.50	11.61	NA	38.75	12496.88
	Roof	W16x36	1	36	NA	6	6	26.88	161.28	2.90	NA	38.75	6249.60
BE5	Floor	W16x40	2	40	NA	4	8	26.88	215.04	8.60	NA	42.75	9192.96
	Roof	W16x36	1	36	NA	4	4	26.88	107.52	1.94	NA	38.75	4166.40
BE6	Floor	W16x40	2	40	NA	2	4	26.88	107.52	4.30	NA	42.75	4596.48
	Roof	W16x40	1	40	NA	2	2	26.88	53.76	1.08	NA	42.75	2298.24
B.5-1 - D2	Floor	W14x22	4	22	NA	2	8	15.90	127.20	5.60	NA	24.75	3148.20
	Roof	W14x22	1	22	NA	2	2	15.90	31.80	0.35	NA	24.75	787.05
D2-F3	Floor	W16x31	4	31	NA	4	16	24.01	384.16	23.82	NA	33.77	12973.08
	Roof	W16x31	1	31	NA	4	4	24.01	96.04	1.49	NA	33.77	3243.27
D.5-1.5 - D2	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
F.5-1.5 - F2	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
						Total =	237.00		Total =	178.37	0.00		**135638.37**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.75	1102.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x34	1	34	NA	4	4	15.00	60.00	1.02	NA	36.75	2205.00
	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
B5	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B6	Basement	W14x43	1	43	NA	1	1	15.00	15.00	0.32	NA	45.70	685.50
	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
E3	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
E4	Basement	W14x48	1	48	NA	2	2	15.00	30.00	0.72	NA	50.75	1522.50
	Level 1	W14x48	1	48	NA	2	2	24.00	48.00	1.15	NA	50.75	2436.00
	Level 2	W14x38	1	38	NA	2	2	24.00	48.00	0.91	NA	40.75	1956.00
E5	Basement	W14x48	1	48	NA	2	2	15.00	30.00	0.72	NA	50.75	1522.50
	Level 1	W14x48	1	48	NA	2	2	24.00	48.00	1.15	NA	50.75	2436.00
	Level 2	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
E6	Basement	W14x53	1	53	NA	1	1	15.00	15.00	0.40	NA	55.75	836.25
	Level 1	W14x53	1	53	NA	1	1	24.00	24.00	0.64	NA	55.75	1338.00
	Level 2	W14x43	1	43	NA	1	1	24.00	24.00	0.52	NA	45.70	1096.80
F3	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H5	Basement	W14x48	1	48	NA	2	2	15.00	30.00	0.72	NA	50.75	1522.50
	Level 1	W14x48	1	48	NA	2	2	24.00	48.00	1.15	NA	50.75	2436.00
	Level 2	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
H6	Basement	W14x53	1	53	NA	1	1	15.00	15.00	0.40	NA	55.25	828.75
	Level 1	W14x53	1	53	NA	1	1	24.00	24.00	0.64	NA	55.25	1326.00
	Level 2	W14x43	1	43	NA	1	1	24.00	24.00	0.52	NA	45.70	1096.80
						Total =	150		Total =	46.14	0		**100863.57**

Slabs

Slab Location	*Slab Section (in)*	*Slab Area (SF)*	Number of Floors	*Volume of Conc.* (CY)	*Tons of Reinf.* (Steel / SF)	*Total Tons of Steel*	*Cost of Conc.* ($81/CY)	*Cost of Reinf. Steel* (elevated) $1270/ton	*Cost of Reinf. Steel* (on grade) $1290/ton	*Placing Slab on Grade* ($19.95/CY)	*Placing Elevated Slab* ($18.45/CY)	*Finish Slab* ($0.29 /SF)	*Formwork* ($5.90/SF)	TOTAL COST
Slab on Grade	4	9515	1	117.469	0.00035	3.33025	9515		$4,296.02	2343.510		$2,759.35		
Elevated Slab	4	25412	2	627.457	0.00035	8.8942	50824	$11,295.63			$11,576.58	$7,369.48	$149,930.80	
Roof	4	9515	1	117.469	0.00035	3.33025	9515	$4,229.42			$2,167.31	$2,759.35	**$56,138.50**	
	Total=	44442	4	862.395		Total=	69854	$15,525.05	$4,296.02	2343.510	$13,743.88	$12,888.18	$206,069.30	**$324,719.95**

GRAND TOTAL FOR OPTION #2 (REINFORCED SLABS, STEEL FRAME) = $673,225.83

B.4.1.6 Steel Cost Breakdown – Option 3 (Composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost	Installed Shear Studs	Total Cost Shear Studs
D56 (typical)	1,2 - Dayroom	W10x15	3	15	22	12	36	19.54	703.50	15.83	792	21.11	14850.91	1.29	1021.68
D45 (typical)	1,2 - Dayroom	W10x15	3	15	22	30	90	16.00	1440.00	32.40	1980	21.11	30398.40	1.29	2554.2
A.5-45 (typical)	Cell Area	W12x14	5	14	20	8	40	16.00	640.00	22.40	800	17.36	11110.40	1.29	1032
D.25F.25-1.5 2.5 (typ)	Cell Area	W12x30	5	30	42	4	20	24.00	480.00	36.00	840	33.00	15840.00	1.29	1083.6
A.5-56 (typical)	Shower Area	W12x19	5	19	26	4	20	19.54	390.83	18.56	520	22.00	8598.35	1.29	670.8
D.5F.5-1.5 2.5	Cell Area	W14x26	5	26	NA	4	20	24.00	480.00	31.20	NA	28.43	13646.40		
B45	Cell Area	W12x14	3	14	20	6	18	16.00	288.00	6.05	360	17.36	4999.68	1.29	464.4
B56	Cell Area	W14x19	3	19	26	2	6	19.54	117.24	3.34	156	22.00	2579.28	1.29	201.24
						Total =	250	Total =		165.78	5448		**102023.42**		**7027.92**

Girder

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Cell Area	W12x14	5	14	NA	8	40	16.00	640.00	22.40	NA	16.75	10720.00
B45	Cell Area	W12x14	5	14	NA	6	30	16.00	480.00	16.80	NA	17.36	8332.80
B56	Cell Area	W12x19	2	19	NA	2	4	16.00	64.00	1.22	NA	17.36	1111.04
AB4	Cell Area	W14x22	5	22	NA	6	30	10.00	300.00	16.50	NA	24.75	7425.00
AB5	Cell Area	W14x22	5	22	NA	2	10	10.00	100.00	5.50	NA	24.75	2475.00
B.5-1 - D2	Cell Area	W14x22	5	22	NA	2	10	16.00	160.00	8.80	NA	24.75	3960.00
D.5D-1.5 2	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
F.5F-2.5 3	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	13.00	NA	28.43	5686.00
D2-F3	Cell Area	W14x34	5	34	NA	8	40	24.00	960.00	81.60	NA	36.75	35280.00
AB6	Shower Area	W14x22	5	22	NA	1	5	10.00	50.00	2.75	NA	24.75	1237.50
BD2	1,2 - Dayroom	W16x26	3	26	NA	2	6	17.92	107.52	4.19	NA	28.40	3053.57
BD4	1,2 - Dayroom	W16x26	3	26	NA	10	30	17.92	537.60	20.97	NA	28.40	15267.84
BD5	1,2 - Dayroom	W16x26	3	26	NA	6	18	17.92	322.56	12.58	NA	28.40	9160.70
BD6	1,2 - Dayroom	W16x26	3	26	NA	3	9	17.92	161.28	6.29	NA	28.40	4580.35
						Total =	272		Total =	225.60	0		**113975.80**

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Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
A2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
A4	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A4	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
A6	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
A6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
B2	Level 1	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
B2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x34	1	34	NA	4	4	15.00	60.00	1.02	NA	36.75	2205.00
B4	Level 1	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.00	3072.00
B4	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
B5	Basement	W14x34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.70	1101.00
B5	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B6	Basement	W14x38	1	38	NA	1	1	15.00	15.00	0.29	NA	40.75	611.25
B6	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
B6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
C.5-.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
C.5-.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
D3	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
D3	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
D.5-1.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
F3	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F4	Basement	W14x43	1	43	NA	6	6	15.00	90.00	1.94	NA	45.70	4113.00
F4	Level 1	W14x43	1	43	NA	6	6	24.00	144.00	3.10	NA	45.70	6580.80
F4	Level 2	W14x30	1	30	NA	6	6	24.00	144.00	2.16	NA	32.77	4718.88
F5	Basement	W14x43	1	43	NA	6	6	15.00	90.00	1.94	NA	45.70	4113.00
F5	Level 1	W14x43	1	43	NA	6	6	24.00	144.00	3.10	NA	45.70	6580.80
F5	Level 2	W14x34	1	34	NA	6	6	24.00	144.00	2.45	NA	36.70	5284.80
F6	Basement	W14x43	1	43	NA	3	3	24.00	72.00	1.55	NA	45.70	3290.40
F6	Level 1	W14x43	1	43	NA	3	3	24.00	72.00	1.55	NA	45.70	3290.40
F6	Level 2	W14x34	1	34	NA	3	3	24.00	72.00	1.22	NA	36.70	2642.40
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
F.5-2.5	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
F.5-2.5	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
H4	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H4	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
						Total =	165		Total =	50.50	0		**110440.20**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($5.90/S.F.)	Finishing ($0.29/S.F.)	Total Cost
4	9513	4.5	150	2140.425	1585.50	157677.98	224506.80	11035.08	393219.86
2	3200	4.5	150	360	266.67	26520.00	37760.00	1856.00	66136.00
									459355.86

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Prison Cell Block Cost (Option #3) = $792,823.20

B.4.1.7 Steel Cost Breakdown – Option 3 (Non-composite WF-Shape)

Beam

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
C45	Floor	W12x35	2	35	NA	30	60	15.98	958.80	33.56	NA	37.70	36146.76
	Roof	W12x35	1	35	NA	30	30	15.98	479.40	8.39	NA	37.70	18073.38
C56	Floor	W12x40	2	40	NA	12	24	19.55	469.20	18.77	NA	42.75	20058.30
	Roof	W12x40	1	40	NA	12	12	19.55	234.60	4.69	NA	42.75	10029.15
C.5-0.5 - D.5-1.5	Floor	W12x30	4	30	NA	2	8	15.90	127.20	7.63	NA	33.00	4197.60
	Roof	W12x30	1	30	NA	2	2	15.90	31.80	0.48	NA	33.00	1049.40
D.5-1.5 - F.5-2.5	Floor	W12x45	4	45	NA	4	16	24.01	384.16	34.57	NA	47.75	18343.64
	Roof	W12x40	1	40	NA	4	4	24.01	96.04	1.92	NA	42.75	4105.71
						Total =	156		Total =	110.01	0		**112003.94**

Girder

Beam Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Floor	W12x30	4	30	NA	6	24	15.98	383.52	23.01	NA	33.00	12656.16
	Roof	W12x30	1	30	NA	6	6	15.98	95.88	1.44	NA	33.00	3164.04
A56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
B45	Floor	W12x35	4	35	NA	6	24	15.98	383.52	26.85	NA	37.70	14458.70
	Roof	W12x35	1	35	NA	6	6	15.98	95.88	1.68	NA	37.70	3614.68
B56	Floor	W12x40	4	40	NA	2	8	19.55	156.40	12.51	NA	42.75	6686.10
	Roof	W12x40	1	40	NA	2	2	19.55	39.10	0.78	NA	42.75	1671.53
AB4	Floor	W14x22	4	22	NA	6	24	10	240.00	10.56	NA	24.43	5863.20
	Roof	W14x22	1	22	NA	6	6	10	60.00	0.66	NA	24.43	1465.80
AB5	Floor	W14x22	4	22	NA	2	8	10	80.00	3.52	NA	24.43	1954.40
	Roof	W14x22	1	22	NA	2	2	10	20.00	0.22	NA	24.43	488.60
AB6	Floor	W14x22	4	22	NA	1	4	10	40.00	1.76	NA	24.43	977.20
	Roof	W14x22	1	22	NA	1	1	10	10.00	0.11	NA	24.43	244.30
BD4	Floor	W14x26	2	26	NA	12	24	17.92	430.08	11.18	NA	28.43	12227.17
	Roof	W14x26	1	26	NA	12	12	17.92	215.04	2.80	NA	24.43	5253.43
BD5	Floor	W14x30	2	30	NA	6	12	17.92	215.04	6.45	NA	33.00	7096.32
	Roof	W14x30	1	30	NA	6	6	17.92	107.52	1.61	NA	33.00	3548.16
BD6	Floor	W14x34	2	34	NA	3	6	17.92	107.52	3.66	NA	36.75	3951.36
	Roof	W14x30	1	30	NA	3	3	17.92	53.76	0.81	NA	36.75	1975.68
B.5-1 - D2	Floor	W14x22	4	22	NA	2	8	15.90	127.20	5.60	NA	24.75	3148.20
	Roof	W14x22	1	22	NA	2	2	15.90	31.80	0.35	NA	24.75	787.05
D2-F3	Floor	W16x31	4	31	NA	4	16	24.01	384.16	23.82	NA	33.70	12946.19
	Roof	W16x31	1	31	NA	4	4	24.01	96.04	1.49	NA	33.70	3236.55
D.5-1.5 - D2	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
F.5-2.5 - F3	Floor	W14x26	4	26	NA	4	16	10.00	160.00	8.32	NA	28.43	4548.80
	Roof	W14x26	1	26	NA	4	4	10.00	40.00	0.52	NA	28.43	1137.20
						Total =	258.00		Total =	171.83	0.00		**127144.44**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A2	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B2	Basement	W14x34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.75	1102.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B4	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
B5	Basement	W14X34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.75	1102.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B6	Basement	W14x34	1	34	NA	1	1	15.00	15.00	0.26	NA	36.75	551.25
	Level 1	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
C.5-.5	Basement	W14x26	1	26	NA	2	2	15.00	30.00	0.39	NA	28.43	852.90
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x34	1	34	NA	2	2	15.00	30.00	0.51	NA	36.75	1102.50
	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F4	Basement	W14x43	1	43	NA	6	6	15.00	90.00	1.94	NA	45.70	4113.00
	Level 1	W14x43	1	43	NA	6	6	24.00	144.00	3.10	NA	45.70	6580.80
	Level 2	W14x30	1	30	NA	6	6	24.00	144.00	2.16	NA	32.77	4718.88
F5	Basement	W14x43	1	43	NA	4	4	15.00	60.00	1.29	NA	45.70	2742.00
	Level 1	W14x43	1	43	NA	4	4	24.00	96.00	2.06	NA	45.70	4387.20
	Level 2	W14x30	1	30	NA	4	4	24.00	96.00	1.44	NA	32.77	3145.92
F6	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
	Level 2	W14x34	1	34	NA	2	2	24.00	48.00	0.82	NA	36.75	1764.00
F.5-2.5	Basement	W14x30	1	30	NA	4	4	15.00	60.00	0.90	NA	32.77	1966.20
	Level 1	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
	Level 2	W14x22	1	22	NA	4	4	24.00	96.00	1.06	NA	24.75	2376.00
H4	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H5	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.70	1371.00
	Level 1	W14x43	1	43	NA	2	2	24.00	48.00	1.03	NA	45.70	2193.60
	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
H6	Basement	W14x43	1	43	NA	1	1	15.00	15.00	0.32	NA	45.70	685.50
	Level 1	W14x43	1	43	NA	1	1	24.00	24.00	0.52	NA	45.70	1096.80
	Level 2	W14x34	1	34	NA	1	1	24.00	24.00	0.41	NA	36.75	882.00
						Total =	165		Total =	49.39	0		**108211.68**

Slabs

Slab Location	Slab Section (in)	Slab Area (SF)	Number of Floors	Volume of Conc. (CY)	Tons of Reinf. (Steel / SF)	Total Tons of Steel	Cost of Conc. ($81/CY)	Cost of Reinf. Steel (elevated) $1270/ton	Cost of Reinf. Steel (on grade) $1290/ton	Placing Slab on Grade ($19.95/CY)	Placing Elevated Slab ($18.45/CY)	Finish Slab ($0.29 /SF)	Formwork ($5.90/SF)	TOTAL COST
Slab on Grade	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00		$ 4,296.02	$ 2,343.51		$ 2,759.35		
Elevated Slab	4	25412	2	627.4567901	0.00035	8.8942	$ 50,824.00	$ 11,295.63			$ 11,576.58	$ 7,369.48	$ 149,930.80	
Roof	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00	$ 4,229.42			$ 2,167.31	$ 2,759.35	$ 56,138.50	
	Total=	44442	4	862.3950617		Total=	$ 69,854.00	$ 15,525.05	$ 4,296.02	$ 2,343.51	$ 13,743.88	$ 12,888.18	$ 206,069.30	**$324,719.95**

GRAND TOTAL FOR OPTION #3 (REINFORCED SLABS, STEEL FRAME) = $672,080.01

B.4.2 Concrete Cost

Option 1:

Cost of Slabs	= $324,719.95
Cost of Beams	= $238,596.38
Cost of Girders	= $199,609.99
Cost of Columns	= $253,193.48
Total Cost for Option 1	**= $1,016,119.79**

Option 2:

Cost of Slabs	= $324,719.95
Cost of Beams	= $238,596.38
Cost of Girders	= $ 125,007.94
Cost of Columns	= $195,457.19
Total Cost for Option 2	**= $883,781.45**

Option 3:

Cost of Slabs	= $324,719.95
Cost of Beams	= $238,596.38
Cost of Girders	= $100,637.78
Cost of Columns	= $132,555.78
Total Cost for Option 3	**= $796,509.88**

B.4.2.1 Concrete Cost Breakdown – Option 1

Beams

Beam Locatation	(Floor Location)	Height (in)	Width (in)	Tons of Reinf. / Member	Number of Floors	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)
Typ. 1	Dayroom	18	10	0.073	3	12	36	19.54	703.44
Typ. 2	Dayroom	14.5	10	0.048	3	24	72	16	1152
Typ. 3	Cell Area	16.5	10	0.073	3	6	18	19.54	351.72
Typ. 4	Cell Area	13.5	10	0.05	3	24	72	16	1152
Typ. 5	Cell Area	19.5	10	0.09	3	6	18	24	432
						Total =	216		Total =

Girders

Girder Location	(Floor Location)	Height (in)	Width (in)	Tons of Reinf. / Member	Number of Floors	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)
Typ. 1	Dayroom	41.5	18	0.64	3	3	9	54	486
Typ. 2	Dayroom	39	18	0.64	3	2	6	54	324
Typ. 2.1	Dayroom	28.5	14	0.22	3	2	6	36	216
Typ. 2.2	Dayroom	17.5	10	0.05	3	2	6	18	108
Typ. 3	Cell Area	15	10	0.02	3	17	51	10	510
									Total=

Columns

Column Location	Height (ft)	Width	Length	Members / Floor	Total Length (ft)	Volume of Conc. (CY)	Cost of 12" Columns ($1202 /CY)	Cost of 16" Columns ($988 /CY)	Cost of 24" Columns ($771 /CY)
C1 Upper	24	16	16	21	504	33.19		32786.96296	
C1 Mid	24	16	16	21	504	33.19		32786.96296	
C1 Basement	24	12	12	21	504	18.67	$ 39,888.59		
C2 Upper	24	12	12	17	408	15.11	$ 39,888.59		
C2 Mid	24	24	24	17	408	60.44			46602.67
C2 Basement	24	24	24	17	408	60.44			46602.67
C3 Upper	15	16	16	5	75	4.94		4879.012346	
C3 Mid	15	16	16	5	75	4.94		4879.012346	
C3 Basement	15	16	16	5	75	4.94		4879.012346	
			Total=	129		Total =	$ 79,777.19	$ 80,210.96	$ 93,205.33

Slabs

Slab Location	Slab Section (in)	Slab Area (SF)	Number of Floors	Volume of Conc. (CY)	Tons of Reinf. Steel / SF	Total Tons of Steel	Cost of Conc. ($81/CY)	Cost of Reinf. Steel (elevated) $1270/ton	Cost of Reinf. Steel (on grade) $1290/ton
Slab on Grade	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00		$ 4,296.02
Elevated Slab	4	25412	2	627.4567901	0.00035	8.8942	$ 50,824.00	$ 11,295.63	
Roof	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00	$ 4,229.42	
	Total=	44442	4	862.3950617		Total=	$ 69,854.00	$ 15,525.05	$ 4,296.02

(Continued From Above)

Volume of Conc.	Cost of Conc. ($81/CY)	Tons of Reinf. Steel	Cost of Reinf. Steel ($2125/Ton)	Formwork (SF)	Cost of Formwork ($14.85/SF)	Placing Conc. (CY)	Cost to Place Conc. ($40.50/CY)		
32.57	$ 2,637.90	2.628	$ 5,584.50	2696.52	$ 40,043.32	32.57	$ 1,318.95		
42.96	$ 3,480.00	3.456	$ 7,344.00	3744	$ 55,598.40	42.96	$ 1,740.00		
14.93	$ 1,209.04	1.314	$ 2,792.25	1260.33	$ 18,715.90	14.93	$ 604.52		
40.00	$ 3,240.00	3.6	$ 7,650.00	3552	$ 52,747.20	40.00	$ 1,620.00		
21.67	$ 1,755.00	1.62	$ 3,442.50	1764	$ 26,195.40	21.67	$ 877.50		**Total Beam Cost**
152.12	$ 12,321.94	12.62	$ 26,813.25	13016.85	$ 193,300.22	152.12	$ 6,160.97		**$ 238,596.38**
Volume of Conc.	**Cost of Conc. ($81/CY)**	**Tons of Reinf. Steel**	**Cost of Reinf. Steel ($2125/Ton)**	**Formwork (SF)**	**Cost of Formwork ($14.85/SF)**	**Placing Conc. (CY)**	**Cost to Place Conc. ($40.50/CY)**		
93.38	$ 7,563.38	5.76	$ 12,240.00	4090.5	$ 60,743.93	93.38	$ 3,781.69		
58.50	$ 4,738.50	3.84	$ 8,160.00	2592	$ 38,491.20	58.50	$ 2,369.25		
22.17	$ 1,795.50	1.32	$ 2,805.00	1278	$ 18,978.30	22.17	$ 897.75		
4.86	$ 393.75	0.3	$ 637.50	405	$ 6,014.25	4.86	$ 196.88		
19.68	$ 1,593.75	1.02	$ 2,167.50	1700	$ 25,245.00	19.68	$ 796.88		**Total Girder Cost**
198.58	$ 16,084.88	12.24	$ 26,010.00	10065.5	$ 149,472.68	198.58	$ 8,042.44		**$ 199,609.99**
									Total Coumn Cost
									$ 253,193.48
Placing Slab on Grade ($19.95/CY)	**Placing Elevated Slab ($18.45/CY)**	**Finish Slab ($0.29 /SF)**	**Formwork ($5.90/SF)**						
$ 2,343.51		$ 2,759.35							
	$ 11,576.58	$ 7,369.48	$ 149,930.80						
	$ 2,167.31	$ 2,759.35	$ 56,138.50						**Total Cost of Slab**
$ 2,343.51	$ 13,743.88	$ 12,888.18	$ 206,069.30						**$ 324,719.95**
									Total Option Cost
									$ 1,016,119.79

B.4.2.2 Concrete Cost Breakdown – Option 2

Beams									
Beam Locatation	**(Floor Location)**	**Height (in)**	**Width (in)**	**Tons of Reinf. / Member**	**Number of Floors**	**Members / Floor**	**Total Members**	**Member Length (ft)**	**Total Length (ft)**
Typ. 1	Dayroom	18	10	0.073	3	12	36	19.54	703.44
Typ. 2	Dayroom	14.5	10	0.048	3	24	72	16	1152
Typ. 3	Cell Area	16.5	10	0.073	3	6	18	19.54	351.72
Typ. 4	Cell Area	13.5	10	0.05	3	24	72	16	1152
Typ. 5	Cell Area	19.5	10	0.09	3	6	18	24	432
						Total =	216		Total =
Girders									
Girder Location	**(Floor Location)**	**Height (in)**	**Width (in)**	**Tons of Reinf. / Member**	**Number of Floors**	**Members / Floor**	**Total Members**	**Member Length (ft)**	**Total Length (ft)**
Typ. 1	Dayroom	24	12	0.08	3	6	18	27	486
Typ. 2	Dayroom	21.5	12	0.1	3	6	18	27	486
Typ. 2.1	Dayroom	17.5	10	0.05	3	3	9	18	162
Typ. 3	Cell Area	15	10	0.02	3	17	51	10	510
									Total=
Columns									
Column Location	**Height (ft)**	**Width**	**Length**	**Members / Floor**	**Total Length (ft)**	**Volume of Conc. (CY)**	**Cost of 12" Columns ($1202 /CY)**	**Cost of 16" Columns ($988 /CY)**	**Cost of 24" Columns ($771 /CY)**
C1 Upper	24	12	12	21	504	18.67		18442.66667	
C1 Mid	24	12	12	21	504	18.67		18442.66667	
C1 Basement	24	12	12	21	504	18.67	$ 22,437.33		
C2 Upper	24	24	24	17	408	60.44	$ 22,437.33		
C2 Mid	24	24	24	17	408	60.44			46602.67
C2 Basement	24	24	24	17	408	60.44			46602.67
C3 Upper	15	16	16	7	105	6.91		6830.617284	
C3 Mid	15	16	16	7	105	6.91		6830.617284	
C3 Basement	15	16	16	7	105	6.91		6830.617284	
C4 Upper	24	12	12	5	120	4.44			
C4 Mid	24	16	16	5	120	7.90			
C4 Basement	24	24	24	5	120	17.78			
				Total=	150		Total =	$ 44,874.67	$ 57,377.19
Slabs									
Slab Location	**Slab Section (in)**	**Slab Area (SF)**	**Number of Floors**	**Volume of Conc. (CY)**	**Tons of Reinf. Steel / SF**	**Total Tons of Steel**	**Cost of Conc. ($81/CY)**	**Cost of Reinf. Steel (elevated) $1270/ton**	**Cost of Reinf. Steel (on grade) $1290/ton**
Slab on Grade	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00		$ 4,296.02
Elevated Slab	4	25412	2	627.4567901	0.00035	8.8942	$ 50,824.00	$ 11,295.63	
Roof	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00	$ 4,229.42	
	Total=	44442	4	862.3950617		Total=	$ 69,854.00	$ 15,525.05	$ 4,296.02

(Continued From Above)

Volume of Conc.	**Cost of Conc. ($81/CY)**	**Tons of Reinf. Steel**	**Cost of Reinf. Steel ($2125/Ton)**	**Formwork (SF)**	**Cost of Formwork ($14.85/SF)**	**Placing Conc. (CY)**	**Cost to Place Conc. ($40.50/CY)**		
32.57	$ 2,637.90	2.628	$ 5,584.50	2696.52	$ 40,043.32	32.57	$ 1,318.95		
42.96	$ 3,480.00	3.456	$ 7,344.00	3744	$ 55,598.40	42.96	$ 1,740.00		
14.93	$ 1,209.04	1.314	$ 2,792.25	1260.33	$ 18,715.90	14.93	$ 604.52		
40.00	$ 3,240.00	3.6	$ 7,650.00	3552	$ 52,747.20	40.00	$ 1,620.00		
21.67	$ 1,755.00	1.62	$ 3,442.50	1764	$ 26,195.40	21.67	$ 877.50		**Total Beam Cost**
152.12	$ 12,321.94	12.62	$ 26,813.25	13016.85	$ 193,300.22	152.12	$ 6,160.97		**$ 238,596.38**
Volume of Conc.	**Cost of Conc. ($81/CY)**	**Tons of Reinf. Steel**	**Cost of Reinf. Steel ($2125/Ton)**	**Formwork (SF)**	**Cost of Formwork ($14.85/SF)**	**Placing Conc. (CY)**	**Cost to Place Conc. ($40.50/CY)**		
36.00	$ 2,916.00	1.44	$ 3,060.00	2430	$ 36,085.50	36.00	$ 1,458.00		
32.25	$ 2,612.25	1.8	$ 3,825.00	2227.5	$ 33,078.38	32.25	$ 1,306.13		
7.29	$ 590.63	0.45	$ 956.25	607.5	$ 9,021.38	7.29	$ 295.31		
19.68	$ 1,593.75	1.02	$ 2,167.50	1700	$ 25,245.00	19.68	$ 796.88		**Total Girder Cost**
95.22	$ 7,712.63	4.71	$ 10,008.75	6965	$ 103,430.25	95.22	$ 3,856.31		**$ 125,007.94**
									Total Coumn Cost
$ 93,205.33									**$ 195,457.19**
Placing Slab on Grade ($19.95/CY)	**Placing Elevated Slab ($18.45**	**Finish Slab ($0.29 /SF)**	**Formwork ($5.90/SF)**						
$ 2,343.51		$ 2,759.35							
	$ 11,576.58	$ 7,369.48	$ 149,930.80						
	$ 2,167.31	$ 2,759.35	$ 56,138.50						**Total Cost of Slab**
$ 2,343.51	$ 13,743.88	$ 12,888.18	$ 206,069.30						**$ 324,719.95**
									Total Option Cost
									$ 883,781.45

B.4.2.3 Concrete Cost Breakdown – Option 3

Beams									
Beam Locatation	(Floor Location)	Height (in)	Width (in)	Tons of Reinf. / Member	Number of Floors	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)
Typ. 1	Dayroom	18	10	0.073	3	12	36	19.54	703.44
Typ. 2	Dayroom	14.5	10	0.048	3	24	72	16	1152
Typ. 3	Cell Area	16.5	10	0.073	3	6	18	19.54	351.72
Typ. 4	Cell Area	13.5	10	0.05	3	24	72	16	1152
Typ. 5	Cell Area	19.5	10	0.09	3	6	18	24	432
						Total =	216		Total =
Girders									
Girder Location	(Floor Location)	Height (in)	Width (in)	Tons of Reinf. / Member	Number of Floors	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)
Typ. 1	Dayroom	19	10	0.05	3	9	27	18	486
Typ. 2	Dayroom	17.5	10	0.05	3	10	30	18	540
Typ. 3	Cell Area	15	10	0.02	3	17	51	10	510
									Total=
Columns									
Column Location	Height (ft)	Width	Length	Members / Floor	Total Length (ft)	Volume of Conc. (CY)	Cost of 12" Columns ($1202/CY)	Cost of 16" Columns ($988/CY)	Cost of 24" Columns ($771/CY)
C1 Upper	24	12	12	21	504	18.67	$ 22,437.33		
C1 Mid	24	12	12	21	504	18.67	$ 22,437.33		
C1 Basement	24	12	12	21	504	18.67	$ 22,437.33		
C2 Upper	24	12	12	13	312	11.56	$ 13,889.78		
C2 Mid	24	12	12	13	312	11.56	$ 13,889.78		
C2 Basement	24	12	12	13	312	11.56	$ 13,889.78		
C3 Upper	15	16	16	9	135	8.89		$ 8,782.22	
C3 Mid	15	16	16	9	135	8.89		$ 8,782.22	
C3 Basement	15	12	12	9	135	5.00	$ 6,010.00		
C4 Upper	24	12	12	7	168	6.22	$ 7,479.11		
C4 Mid	24	12	12	7	168	6.22	$ 7,479.11		
C4 Basement	24	12	12	7	168	6.22	$ 7,479.11		
C5 Upper	24	12	12	5	120	4.44	$ 5,342.22		
C5 Mid	24	12	12	5	120	4.44	$ 5,342.22		
C5 Basement	24	16	16	5	120	7.90		$ 7,806.42	
				Total=	165		Total =	$ 114,991.33	$ 17,564.44
Slabs									
Slab Location	Slab Section (in)	Slab Area (SF)	Number of Floors	Volume of Conc. (CY)	Tons of Reinf. Steel / SF	Total Tons of Steel	Cost of Conc. ($81/CY)	Cost of Reinf. Steel (elevated) $127(	Cost of Reinf. Steel (on grade) $129
Slab on Grade	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00		$ 4,296.02
Elevated Slab	4	25412	2	627.4567901	0.00035	8.8942	$ 50,824.00	$ 11,295.63	
Roof	4	9515	1	117.4691358	0.00035	3.33025	$ 9,515.00	$ 4,229.42	
	Total=	44442	4	862.3950617		Total=	$ 69,854.00	$ 15,525.05	$ 4,296.02

(Continued From Above)

Volume of Conc.	Cost of Conc. ($81/CY)	Tons of Reinf. Steel	Cost of Reinf. Steel ($2125/Ton)	Formwork (SF)	Cost of Formwork ($14.85/SF)	Placing Conc. (CY)	Cost to Place Conc. ($40.50/CY)		
32.57	$ 2,637.90	2.628	$ 5,584.50	2696.52	$ 40,043.32	32.57	$ 1,318.95		
42.96	$ 3,480.00	3.456	$ 7,344.00	3744	$ 55,598.40	42.96	$ 1,740.00		
14.93	$ 1,209.04	1.314	$ 2,792.25	1260.33	$ 18,715.90	14.93	$ 604.52		
40.00	$ 3,240.00	3.6	$ 7,650.00	3552	$ 52,747.20	40.00	$ 1,620.00		
21.67	$ 1,755.00	1.62	$ 3,442.50	1764	$ 26,195.40	21.67	$ 877.50		**Total Beam Cost**
152.12	$ 12,321.94	12.62	$ 26,813.25	13016.85	$ 193,300.22	152.12	$ 6,160.97		**$ 238,596.38**
Volume of Conc.	Cost of Conc. ($81/CY)	Tons of Reinf. Steel	Cost of Reinf. Steel ($2125/Ton)	Formwork (SF)	Cost of Formwork ($14.85/SF)	Placing Conc. (CY)	Cost to Place Conc. ($40.50/CY)		
23.75	$ 1,923.75	1.35	$ 2,868.75	1944	$ 28,868.40	23.75	$ 961.88		
24.31	$ 1,968.75	1.5	$ 3,187.50	2025	$ 30,071.25	24.31	$ 984.38		
19.68	$ 1,593.75	1.02	$ 2,167.50	1700	$ 25,245.00	19.68	$ 796.88		**Total Girder Cost**
67.73	$ 5,486.25	3.87	$ 8,223.75	5669	$ 84,184.65	67.73	$ 2,743.13		**$ 100,637.78**
									Total Coumn Cost
$ -									**$ 132,555.78**
Placing Slab on Grade ($19.95/CY)	Placing Elevated Slab ($18	Finish Slab ($0.29 /SF)	Formwork ($5.90/SF)						
$ 2,343.51		$ 2,759.35							
	$ 11,576.58	$ 7,369.48	$ 149,930.80						
	$ 2,167.31	$ 2,759.35	$ 56,138.50						**Total Cost of Slab**
$ 2,343.51	$ 13,743.88	$ 12,888.18	$ 206,069.30						**$ 324,719.95**
									Total Option Cost
									$ 796,509.88

B.5 Prison Cellblock Layout – Evaluating Options

B.5.1 Prison Cellblock Scheme Relative Comparison

PRISON CELL BLOCK SCENARIO RANKINGS

		# Members	Connections	Floor Depth	Mechanical	Visibility	Cost	Total	Overall Ranking
Composite w/ W-Flange	Option #1	8	2	4	3	1	8	26	7
	Option #2	9	2	1	3	5	5	25	5
	Option #3	10	2	1	3	10	6	32	10
Composite w/ Joist	Option #1	3	2	7	1	1	1	15	1
		---	---	---	---	---	---	---	---
		---	---	---	---	---	---	---	---
Reinforced Slab w/ Steel Frame	Option #1	5	2	4	3	1	4	19	2
	Option #2	6	2	1	3	5	3	20	3
	Option #3	7	2	1	3	10	2	25	5
Reinforced	Option #1	1	1	10	3	1	10	26	7
	Option #2	2	1	3	3	5	9	23	4
	Option #3	4	1	2	3	10	7	27	9

Notes:
1. Grading for # members is based on ranking each scenario from the one with the least number of members to the one with the most number of members (1-10).
2. Grading for the connections is based on the types of connections. Reinforced concrete received a value of 1 and steel received a value of 2 because steel connections would involve either bolt connections or welded (moment) connections.
3. Grading for floor depth was based on ranking each scenario from the one with the shallowest floor depth (deepest girder depth plus slab thickness) to the one with the greatest floor depth (1-10).
4. Grading for mechanical was based on the ease of passing mechanical considerations such as ducting, plumbing, and electrical through the support members. The open web joist option received a value of 1 because of the available openings. All other options received a value of 3. They received a value of 3 because all though consideration could be given to run mechanical through the webs of wide-flange section and concrete members, design considerations as well as labor would need to be evaluated, making those options slightly less appealing when consideration the placement of mechanical.
5. Grading for visibility was based on the amount of columns disrupting the open space of the dayroom in the cell blocks. Option #1 received a value of 1 because no columns intrude in the dayroom; option #2 received a value of 5 because one row of columns intrude the dayroom; option #3 received a vlaue of 10 because two rows of column intrude the dayroom. These values of 1, 5, and 10 were chosen, although drastic, because of the dramatic effect of having to add another guard to the prison cell block. For every guard that is added, about $1,000,000 is added to the operating costs over the lifespan of the prison/
6. Grading for cost was based on ranking each scenario from the one with the lowest costs to the one with the highest cost (1-10).
7. The total is a summation of the grading for each criteria for all the scenarios.
8. The overall ranking is based on ranking the scenario with the lowest total to the highest total (1-10). The scenario with the lowest ranking (lowest total) is determined to be the most effective based on the criteria offered.

B.5.2 Prison Cellblock Scheme Grading Comparison

PRISON CELL BLOCK SCENARIO RANKINGS

	Option	# Members	# Sizes	Connections	Shear Studs	Decking	Rebar	Floor Depth	Mechanical	Visibility	Total	Cost	Cost/Coeff.	Overall Ranking
Composite w/ W-Flange	1	0.85		0.8	0.7	1	1	0.88	0.90	1	0.37	$856,832.39	$2,287,936.96	4
	2	0.83		0.8	0.7	1	1	0.98	0.90	0.8	0.33	$783,189.48	$2,382,741.90	7
	3	0.81		0.8	0.7	1	1	0.98	0.90	0.6	0.24	$792,823.20	$3,289,530.58	10
Composite w/ Joist	1	0.90		0.8	0.8	0.95	1	0.83	1.00	1	0.46	$580,039.88	$1,274,296.37	1
Reinforced Slab w/ Steel Frame	1	0.89		0.8	1	1	0.8	0.88	0.90	1	0.45	$709,863.05	$1,569,536.39	2
	2	0.86		0.8	1	1	0.8	1.00	0.90	0.8	0.40	$673,225.83	$1,689,711.40	3
	3	0.84		0.8	1	1	0.8	1.00	0.90	0.6	0.29	$672,080.01	$2,309,676.67	5
Reinforced	1	1.00	1	1	1	1	0.6	0.81	0.90	1	0.44	$1,016,119.80	$2,318,527.32	6
	2	0.93	1	1	1	1	0.6	0.92	0.90	0.8	0.37	$883,781.46	$2,386,755.49	8
	3	0.90	1	1	1	1	0.6	0.96	0.90	0.6	0.28	$796,509.89	$2,840,186.78	9

B.6 Prison Cellblock Layout – Gravity and Lateral Loads

B.6.1 Steel Cost

B.6.1.1 Steel Cost Breakdown – Option 1 (Composite Open-Web Joist)

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
BH6	Dayroom	36VC 900	3	27	28	3	9	53.69	483.21	6.52	252	1724.00	11246.23	1.32	332.64
BH5	Dayroom	36VC 800	3	22	30	2	6	53.69	322.14	3.54	180	1724.00	6109.06	1.32	237.60
BH4.5	Dayroom	33VC 700	3	20	46	6	18	53.69	966.42	9.66	828	1724.00	16661.08	1.32	1092.96
BF3	Dayroom	24VC 800	3	13	26	6	18	35.83	644.94	4.19	468	1464.50	6139.35	1.32	617.76
A.5 - 5 6	Shower Area	14VC 1100	5	10	18	2	10	19.55	195.54	0.98	180	1464.50	1431.85	1.32	237.60
A.5 - 4 5	Cell Area	14VC 1100	5	10	18	8	40	16.00	640.00	3.20	720	1464.50	4686.40	1.32	950.40
D.25F.25 - 1.75-2.75	Cell Area	14VC 1300	5	14	28	4	20	24.00	480.00	3.36	560	1464.50	4920.72	1.32	739.20
						Total =	121		Total =	31.46	3188		**51194.69**		**4208.16**

Girder

Beam Location	(Floor Location)	Beam Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A45	Cell Area	W12x14	5	14	NA	8	40	16.00	640.00	4.48	NA	16.75	10720.00
A56	Cell Area	W12x19	5	19	NA	2	10	19.75	197.50	1.88	NA	21.75	4295.63
AB4	Cell Area	W14x22	5	22	NA	6	30	10.00	300.00	3.30	NA	24.75	7425.00
AB5	Cell Area	W14x22	5	22	NA	2	10	10.00	100.00	1.10	NA	24.75	2475.00
B.5-1 - D2	Cell Area	W14x22	5	22	NA	2	10	16.00	160.00	1.76	NA	24.75	3960.00
C.5-.5 - D.5-1.5	Cell Area	W14x26	5	26	NA	4	20	16.00	320.00	4.16	NA	28.43	9097.60
D.5F.5-1.5 2.5	Cell Area	W12x30	5	30	NA	4	20	24.00	480.00	7.20	NA	32.75	15720.00
F.5F-2.5 3	Cell Area	W14x26	5	26	NA	4	20	10.00	200.00	2.60	NA	28.43	5686.00
AB6	Shower Area	W14x22	5	22	NA	1	5	10.00	50.00	0.55	NA	24.75	1237.50
B45	Cell Area	W14x26	5	26	NA	6	30	16.00	480.00	6.24	NA	28.43	13646.40
H45	1,2 Dayroom	W14x26	3	26	NA	2	6	16.00	96.00	1.25	NA	28.43	2729.28
B56	Cell Area	W14x34	5	34	NA	2	10	19.75	197.50	3.36	NA	36.70	7248.25
H56	1,2 - Dayroom	W14x34	3	34	NA	2	6	19.75	118.50	2.01	NA	36.70	4348.95
D2-F3	Cell Area	W14x30	5	30	NA	2	10	24.00	240.00	3.60	NA	32.77	7864.80
F3-H4	Cell Area	W14x43	5	43	NA	2	10	24.00	240.00	5.16	NA	45.70	10968.00
						Total =	237		Total =	48.65	0		**107422.41**

Bracing

Bracing Location	(Floor Location)	Bracing Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ton	Total Cost
A6-B6	Basement	HSS4-1/2x4-1/2x3/8	1	19.7	NA	17	17.00	18.03	306.51	3.02	2500.00	7547.80875
A6-B6	Level 1	HSS4x4x1/4	1	12.2	NA	17	17.00	15.26	259.42	1.58	2500.00	3956.155
A6-B6	Mezzanine 1	HSS3-1/2x3-1/2x1/4	1	8.78	NA	17	17.00	15.26	259.42	1.14	2500.00	2847.1345
A6-B6	Level 2	HSS3-1/2x3-1/2x1/4	1	8.78	NA	17	17.00	15.26	259.42	1.14	2500.00	2847.1345
A6-B6	Mezzanine 2	HSS2-1/2x2-1/2x3/16	1	4.94	NA	17	17.00	15.26	259.42	0.64	2500.00	1601.9185
						Total =	85.00		Total =	7.52		**18800.15**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ft	Total Cost
A1	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A1	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A1	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A2	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A4	Basement	W14x30	1	30	NA	12	12	15.00	180.00	2.70	NA	32.77	5898.60
A4	Level 1	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A4	Level 2	W14x22	1	22	NA	12	12	24.00	288.00	3.17	NA	24.75	7128.00
A5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
A5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
A6	Basement	W14x30	1	30	NA	1	1	15.00	15.00	0.23	NA	32.77	491.55
A6	Level 1	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
A6	Level 2	W14x22	1	22	NA	1	1	24.00	24.00	0.26	NA	24.75	594.00
B.5-0	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
B.5-0	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B.5-0	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
B2	Basement	W14x43	1	43	NA	2	2	15.00	30.00	0.65	NA	45.75	1372.50
B2	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B2	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
B4	Basement	W14x53	1	53	NA	4	4	15.00	60.00	1.59	NA	55.75	3345.00
B4	Level 1	W14x34	1	34	NA	4	4	24.00	96.00	1.63	NA	36.75	3528.00
B4	Level 2	W14x26	1	26	NA	4	4	24.00	96.00	1.25	NA	28.43	2729.28
B5	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.25	1897.50
B5	Level 1	W14x38	1	38	NA	2	2	24.00	48.00	0.91	NA	40.75	1956.00
B5	Level 2	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
B6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.25	948.75
B6	Level 1	W14x38	1	38	NA	1	1	24.00	24.00	0.46	NA	40.75	978.00
B6	Level 2	W14x30	1	30	NA	1	1	24.00	24.00	0.36	NA	32.77	786.48
C.5-.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
C.5-.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
C.5-.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D2	Basement	W14x38	1	38	NA	2	2	15.00	30.00	0.57	NA	40.75	1222.50
D2	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
D2	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
D.5-1.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
D.5-1.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F3	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.25	1897.50
F3	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
F3	Level 2	W14x26	1	26	NA	2	2	24.00	48.00	0.62	NA	28.43	1364.64
F.5-2.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
F.5-2.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
F.5-2.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H.5-3.5	Basement	W14x30	1	30	NA	2	2	15.00	30.00	0.45	NA	32.77	983.10
H.5-3.5	Level 1	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H.5-3.5	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H4	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.25	1897.50
H4	Level 1	W14x30	1	30	NA	2	2	24.00	48.00	0.72	NA	32.77	1572.96
H4	Level 2	W14x22	1	22	NA	2	2	24.00	48.00	0.53	NA	24.75	1188.00
H5	Basement	W14x61	1	61	NA	2	2	15.00	30.00	0.92	NA	63.75	1912.50
H5	Level 1	W14x61	1	61	NA	2	2	24.00	48.00	1.46	NA	63.75	3060.00
H5	Level 2	W14x63	1	53	NA	2	2	24.00	48.00	1.27	NA	53.25	2556.00
H6	Basement	W14x61	1	61	NA	1	1	15.00	15.00	0.46	NA	63.75	956.25
H6	Level 1	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
H6	Level 2	W14x61	1	61	NA	1	1	24.00	24.00	0.73	NA	63.75	1530.00
						Total =	141		Total =	43.93	0		**95780.79**

Total Beam, Girder, Column Steel, and Concrete Weight - Joist Alternative

Concrete

Number of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Cost / cyd. (total)	Total Cost
3	9513	2.75	110	719.42	726.69	119.70	95260.80
2	3200	2.75	110	161.33	162.96	119.70	21362.67
				GRAND TOTAL =	889.65		**116623.47**

Note: Concrete, Ready Mix: $101.25/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f - screen finish

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Cell Area	1.5VL22	3200	2	6400	1.76	11264
Day Room (BH6)	1.5VL22	9513	3	28539	1.76	50228.64
			Total =	34939	Total =	**61492.64**

Diaphragm Continuity Shear Studs

Linear Feet (total)	Max Spacing (in.)	Total # Studs	Installed Shear Studs	Total Cost Shear Studs
3833	22	2091	1.32	**2759.76**

Grand Total for Prison Cell Block Cost = $458,282.07

B.7 Prison Core Layout – Gravity Loads

B.7.1 Steel Cost

B.7.1.1 Steel Cost Breakdown

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
A1-A4 (Access Core)	Floor	22VC 1600/100/30	3	30	30	2	6	37.42	224.52	6.74	180	1464.50	9864.29	1.32	237.60
	Roof	24VC 1400/100/30	1	25	20	2	2	37.42	74.84	1.87	40	1464.50	2740.08	1.32	52.80
A1-A5	Floor	36VC 1400/105/30	3	46	46	8	24	60.92	1462.08	67.26	1104	1464.50	98495.94	1.32	1457.28
	Roof	40VC 1300/35/30	1	37	32	8	8	60.92	487.36	18.03	256	1464.50	26408.33	1.32	337.92
B1-B4.5	Floor	36VC 1400/105/30	3	46	46	8	24	58.83	1411.92	64.95	1104	1464.50	95116.81	1.32	1457.28
	Roof	36VC 1300/35/30	1	40	36	8	8	58.83	470.64	18.83	288	1464.50	27570.09	1.32	380.16
B1-B2 (Access Core)	Floor	20VC 1400/100/30	3	11	14	4	12	8.92	107.04	1.18	168	1374.00	1617.80	1.32	221.76
	Roof	20VC 1400/100/30	1	11	14	4	4	8.92	35.68	0.39	56	1374.00	539.27	1.32	73.92
C1-C3.5	Floor	30VC 1600/105/30	3	40	42	8	24	48.83	1171.92	46.88	1008	1464.50	68651.07	1.32	1330.56
	Roof	33VC 1400/35/30	1	32	28	8	8	48.83	390.64	12.50	224	1464.50	18306.95	1.32	295.68
E1-E1.5	Floor	20VC 1400/45/30	3	11	18	8	24	7.99	191.76	2.11	432	1374.00	2898.26	1.32	570.24
	Roof	20VC 1600/35/30	1	12	16	8	8	7.99	63.92	0.77	128	1374.00	1053.91	1.32	168.96
F1-F2	Floor	20VC 1000/45/30	3	9	14	8	24	15.98	383.52	3.45	336	1374.00	4742.61	1.32	443.52
	Roof	20VC 1100/35/30	1	9	14	8	8	15.98	127.84	1.15	112	1374.00	1580.87	1.32	147.84
G1-G2.5	Floor	20VC 1100/45/30	3	12	22	8	24	25.76	618.24	7.42	528	1374.00	10193.54	1.32	696.96
	Roof	20VC 1300/35/30	1	13	20	8	8	25.76	206.08	2.68	160	1374.00	3681.00	1.32	211.20
H1-H3	Floor	22VC 1100/45/30	3	15	30	4	12	30.53	366.36	5.50	360	1374.00	7550.68	1.32	475.20
	Roof	18VC 1300/35/30	1	22	18	4	4	30.53	122.12	2.69	72	1374.00	3691.44	1.32	95.04
H3-H4 (Access Core)	Floor	14VC 1800/105/30	3	22	30	4	12	25.11	301.32	6.63	360	1374.00	9108.30	1.32	475.20
	Roof	16VC 1300/35/30	1	15	24	4	4	25.11	100.44	1.51	96	1374.00	2070.07	1.32	126.72
						Total =	248		Total =	272.51	7012		**395881.33**		**9255.84**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ft	Total Cost
A1-D1	Floor	W24x76	3	76	NA	8	24	25.64	615.36	23.38	76.97	47364.26
	Roof	W24x62	1	62	NA	8	8	25.64	205.12	6.36	63.47	13018.97
A1-A1	Floor	W24x76	3	76	NA	4	12	10.00	120.00	4.56	76.97	9236.40
	Roof	W24x62	1	62	NA	4	4	10.00	40.00	1.24	63.47	2538.80
A.5 5 - A.5 5	Floor	W24x84	3	84	NA	4	12	25.85	310.20	13.03	84.70	26273.94
	Roof	W24x68	1	68	NA	4	4	25.85	103.40	3.52	69.47	7183.20
A.5 5 - B.5 4	Floor	W14x34	3	34	NA	8	24	12.54	300.96	5.12	36.70	11045.23
	Roof	W14x26	1	26	NA	8	8	12.54	100.32	1.30	28.43	2852.10
D1-F1	Floor	W14x22	3	22	NA	8	24	15.98	383.52	4.22	24.75	9492.12
	Roof	W14x22	1	22	NA	8	8	15.98	127.84	1.41	24.75	3164.04
D2-D3	Floor	W14x30	3	30	NA	16	48	21.59	1036.32	15.54	32.77	33960.21
	Roof	W14x30	1	30	NA	16	16	21.59	345.44	5.18	32.77	11320.07
F1-H1	Floor	W16x45	3	45	NA	8	24	19.55	469.20	10.56	47.75	22404.30
	Roof	W14x26	1	26	NA	8	8	19.55	156.40	2.03	28.43	4446.45
G.5 3 - H.5 3	Floor	W14x26	3	26	NA	8	24	10.00	240.00	3.12	28.43	6823.20
	Roof	W14x22	1	22	NA	8	8	10.00	80.00	0.88	24.75	1980.00
						Total =	256		Total =	101.45		**213103.28**

(Continued on next page)

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ft	Total Cost
A1	Basement	W14x43	1	43	NA	8	8	15.00	120.00	2.58	45.70	5484.00
	1	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
A.5-5	Basement	W14x61	1	61	NA	8	8	15.00	120.00	3.66	63.75	7650.00
	1	W14x38	1	38	NA	8	8	24.00	192.00	3.65	40.75	7824.00
	2	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
B.5-4	Basement	W14x34	1	34	NA	8	8	15.00	120.00	2.04	36.70	4404.00
	1	W14x26	1	26	NA	8	8	24.00	192.00	2.50	28.43	5458.56
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
D1	Basement	W14x61	1	61	NA	8	8	15.00	120.00	3.66	63.75	7650.00
	1	W14x38	1	38	NA	8	8	24.00	192.00	3.65	40.75	7824.00
	2	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
D2	Basement	W14x26	1	26	NA	8	8	15.00	120.00	1.56	28.43	3411.60
	1	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
D3	Basement	W14x43	1	43	NA	8	8	15.00	120.00	2.58	45.70	5484.00
	1	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
F1	Basement	W14x53	1	53	NA	8	8	15.00	120.00	3.18	55.25	6630.00
	1	W14x53	1	53	NA	8	8	24.00	192.00	5.09	55.25	10608.00
	2	W14x43	1	43	NA	8	8	24.00	192.00	4.13	45.70	8774.40
H1	Basement	W14x61	1	61	NA	4	4	15.00	60.00	1.83	63.75	3825.00
	1	W14x61	1	61	NA	4	4	24.00	96.00	2.93	63.75	6120.00
	2	W14x43	1	43	NA	4	4	24.00	96.00	2.06	45.70	4387.20
						Total =	180		Total =	67.17		**144462.12**

Total Beam, Girder, Column Steel, and Concrete Weight - Joist Alternative

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Floor	1.5VL22	20756.63	3	62269.89	1.76	109595.0064
Roof	1.5VL22	21126.51	1	21126.51	1.76	37182.6576
			Total =	83396.4	Total =	**146777.664**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($---/S.F.)	Finishing ($0.29/S.F.)	Total Cost
3	20756.63	2.75	145	2069.176553	1585.58	157685.52	0.00	18058.27	175743.79
1	21126.51	2.75	145	702.0163219	537.94	53498.49	0.00	6126.69	59625.17
									235368.96

Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Grand Total for Prison Cell Block Cost = $1,144,849.20

B.8 Prison Core Layout – Gravity and Lateral Loads

B.8.1 Steel Cost

B.8.1.1 Steel Cost Breakdown

Joist

Joist Location	(Floor Location)	Joist Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Total Shear Studs	Unit Cost / ton	Total Cost	Installed Shear Studs	Total Cost Shear Studs
A1-A4 (Access Core)	Floor	22VC 1600/100/30	3	30	30	2	6	37.42	224.52	6.74	180	1464.50	9864.29	1.32	237.60
	Roof	24VC 1400/100/30	1	25	20	2	2	37.42	74.84	1.87	40	1464.50	2740.08	1.32	52.80
A1-A5	Floor	36VC 1400/105/30	3	46	46	8	24	60.92	1462.08	67.26	1104	1464.50	98495.94	1.32	1457.28
	Roof	40VC 1300/35/30	1	37	32	8	8	60.92	487.36	18.03	256	1464.50	26408.33	1.32	337.92
B1-B4.5	Floor	36VC 1400/105/30	3	46	46	8	24	58.83	1411.92	64.95	1104	1464.50	95116.81	1.32	1457.28
	Roof	36VC 1300/35/30	1	40	36	8	8	58.83	470.64	18.83	288	1464.50	27570.09	1.32	380.16
B1-B2 (Access Core)	Floor	20VC 1400/100/30	3	11	14	4	12	8.92	107.04	1.18	168	1374.00	1617.80	1.32	221.76
	Roof	20VC 1400/100/30	1	11	14	4	4	8.92	35.68	0.39	56	1374.00	539.27	1.32	73.92
C1-C3.5	Floor	30VC 1600/105/30	3	40	42	8	24	48.83	1171.92	46.88	1008	1464.50	68651.07	1.32	1330.56
	Roof	33VC 1400/35/30	1	32	28	8	8	48.83	390.64	12.50	224	1464.50	18306.95	1.32	295.68
E1-E1.5	Floor	20VC 1400/45/30	3	11	18	8	24	7.99	191.76	2.11	432	1374.00	2898.26	1.32	570.24
	Roof	20VC 1600/35/30	1	12	16	8	8	7.99	63.92	0.77	128	1374.00	1053.91	1.32	168.96
F1-F2	Floor	20VC 1000/45/30	3	9	14	8	24	15.98	383.52	3.45	336	1374.00	4742.61	1.32	443.52
	Roof	20VC 1100/35/30	1	9	14	8	8	15.98	127.84	1.15	112	1374.00	1580.87	1.32	147.84
G1-G2.5	Floor	20VC 1100/45/30	3	12	22	8	24	25.76	618.24	7.42	528	1374.00	10193.54	1.32	696.96
	Roof	20VC 1300/35/30	1	13	20	8	8	25.76	206.08	2.68	160	1374.00	3681.00	1.32	211.20
H1-H3	Floor	22VC 1100/45/30	3	15	30	4	12	30.53	366.36	5.50	360	1374.00	7550.68	1.32	475.20
	Roof	18VC 1300/35/30	1	22	18	4	4	30.53	122.12	2.69	72	1374.00	3691.44	1.32	95.04
H3-H4 (Access Core)	Floor	14VC 1800/105/30	3	22	30	4	12	25.11	301.32	6.63	360	1374.00	9108.30	1.32	475.20
	Roof	16VC 1300/35/30	1	15	24	4	4	25.11	100.44	1.51	96	1374.00	2070.07	1.32	126.72
						Total =	248		Total =	272.51	7012		**395881.33**		**9255.84**

Girder

Girder Location	(Floor Location)	Girder Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ft	Total Cost
A1-D1	Floor	W24x76	3	76	NA	16	48	25.64	1230.72	46.77	76.97	94728.52
	Roof	W24x62	1	62	NA	8	8	25.64	205.12	6.36	63.47	13018.97
A1-A1	Floor	W24x76	3	76	NA	8	24	10.00	240.00	9.12	76.97	18472.80
	Roof	W24x62	1	62	NA	4	4	10.00	40.00	1.24	63.47	2538.80
A.5 5 - A.5 5	Floor	W24x84	3	84	NA	4	12	25.85	310.20	13.03	84.70	26273.94
	Roof	W24x68	1	68	NA	4	4	25.85	103.40	3.52	69.47	7183.20
A.5 5 - B.5 4	Floor	W14x34	3	34	NA	8	24	12.54	300.96	5.12	36.70	11045.23
	Roof	W14x26	1	26	NA	8	8	12.54	100.32	1.30	28.43	2852.10
D1-F1	Floor	W14x22	3	22	NA	8	24	15.98	383.52	4.22	24.75	9492.12
	Roof	W14x22	1	22	NA	8	8	15.98	127.84	1.41	24.75	3164.04
D2-D3	Floor	W14x30	3	30	NA	24	72	21.59	1554.48	23.32	32.77	50940.31
	Roof	W14x30	1	30	NA	16	16	21.59	345.44	5.18	32.77	11320.07
F1-H1	Floor	W16x45	3	45	NA	8	24	19.55	469.20	10.56	47.75	22404.30
	Roof	W14x26	1	26	NA	8	8	19.55	156.40	2.03	28.43	4446.45
G.5 3 - H.5 3	Floor	W14x26	3	26	NA	8	24	10.00	240.00	3.12	28.43	6823.20
	Roof	W14x22	1	22	NA	8	8	10.00	80.00	0.88	24.75	1980.00
						Total =	316		Total =	137.16		**286684.04**

(Continue ontnext page)

Bracing

Brace Location	(Floor Location)	Brace Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ton	Total Cost
D1-D1.5	Basement	HSS6x5x1/4	1	17.3	NA	16	16	18.5	296	2.56	2500	6401.00
	Level 1 - lower	HSS5x5x3/16	1	12	NA	16	16	16.1	257.6	1.55	2500	3864.00
	Level 1 - upper	HSS5x4x3/16	1	10.7	NA	16	16	16.1	257.6	1.38	2500	3445.40
	Level 2 - lower	HSS4x4x3/16	1	9.4	NA	16	16	16.1	257.6	1.21	2500	3026.80
	Level 2 - upper	HSS4x4x3/16	1	9.4	NA	16	16	16.1	257.6	1.21	2500	3026.80
A1-B1:1.5	Basement	HSS3x3x3/16	1	6.85	NA	16	16	18.5	296	1.01	2500	2534.50
	Level 1 - lower	HSS3x3x3/16	1	6.85	NA	16	16	16.1	257.6	0.88	2500	2205.70
	Level 1 - upper	HSS3x3x3/16	1	6.85	NA	16	16	16.1	257.6	0.88	2500	2205.70
	Level 2 - lower	HSS3x3x3/16	1	6.85	NA	16	16	16.1	257.6	0.88	2500	2205.70
	Level 2 - upper	HSS3x3x3/16	1	6.85	NA	16	16	16.1	257.6	0.88	2500	2205.70
Elevator Shaft (3 sides)	Basement	HSS3x3x3/16	1	6.85	NA	3	3	18.9	56.7	0.19	2500	485.49
	Level 1 - lower	HSS3x3x3/16	1	6.85	NA	3	3	16.6	49.8	0.17	2500	426.41
	Level 1 - upper	HSS3x3x3/16	1	6.85	NA	3	3	16.6	49.8	0.17	2500	426.41
	Level 2 - lower	HSS3x3x3/16	1	6.85	NA	3	3	16.6	49.8	0.17	2500	426.41
	Level 2 - upper	HSS3x3x3/16	1	6.85	NA	3	3	16.6	49.8	0.17	2500	426.41
						Total =	175		Total =	13.32		**33312.44**

Column

Column Location	(Floor Location)	Column Section	Number of Floors	Unit Weight (plf)	Shear Studs/Member	Members / Floor	Total Members	Member Length (ft)	Total Length (ft)	Total Member Weight (tons)	Unit Cost / ft	Total Cost
A1	Basement	W14x43	1	43	NA	8	8	15.00	120.00	2.58	45.70	5484.00
	1	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
A.5-5	Basement	W14x61	1	61	NA	8	8	15.00	120.00	3.66	63.75	7650.00
	1	W14x38	1	38	NA	8	8	24.00	192.00	3.65	40.75	7824.00
	2	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
B.5-4	Basement	W14x34	1	34	NA	8	8	15.00	120.00	2.04	36.70	4404.00
	1	W14x26	1	26	NA	8	8	24.00	192.00	2.50	28.43	5458.56
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
D1	Basement	W14x61	1	61	NA	8	8	15.00	120.00	3.66	63.75	7650.00
	1	W14x48	1	48	NA	8	8	24.00	192.00	4.61	50.75	9744.00
	2	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
D2	Basement	W14x26	1	26	NA	8	8	15.00	120.00	1.56	28.43	3411.60
	1	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
D3	Basement	W14x43	1	43	NA	8	8	15.00	120.00	2.58	45.70	5484.00
	1	W14x30	1	30	NA	8	8	24.00	192.00	2.88	32.77	6291.84
	2	W14x22	1	22	NA	8	8	24.00	192.00	2.11	24.75	4752.00
F1	Basement	W14x53	1	53	NA	8	8	15.00	120.00	3.18	55.25	6630.00
	1	W14x53	1	53	NA	8	8	24.00	192.00	5.09	55.25	10608.00
	2	W14x43	1	43	NA	8	8	24.00	192.00	4.13	45.70	8774.40
H1	Basement	W14x61	1	61	NA	4	4	15.00	60.00	1.83	63.75	3825.00
	1	W14x61	1	61	NA	4	4	24.00	96.00	2.93	63.75	6120.00
	2	W14x43	1	43	NA	4	4	24.00	96.00	2.06	45.70	4387.20
						Total =	180		Total =	68.13		**146382.12**

Total Beam, Girder, Column Steel, and Concrete Weight - Joist Alternative

Steel Deck

Deck Location	Deck Type	Area/Floor	Num. Floors	Total Deck Area	Cost/sq.ft.	Total Cost
Floor	1.5VL22	20756.63	3	62269.89	1.76	109595.0064
Roof	1.5VL22	21126.51	1	21126.51	1.76	37182.6576
			Total =	83396.4	Total =	**146777.664**

Concrete

# of Floors	Area (ft^2)	Slab Depth (in)	Concrete (pcf)	Weight (k)	Cubic Yards	Concrete Cost ($99.45/CYD)	Form Work ($---/S.F.)	Finishing ($0.29/S.F.)	Total Cost
3	20756.63	2.75	145	2069.18	1585.58	157685.52	0.00	18058.27	175743.79
1	21126.51	2.75	145	702.02	537.94	53498.49	0.00	6126.69	59625.17
									235368.96

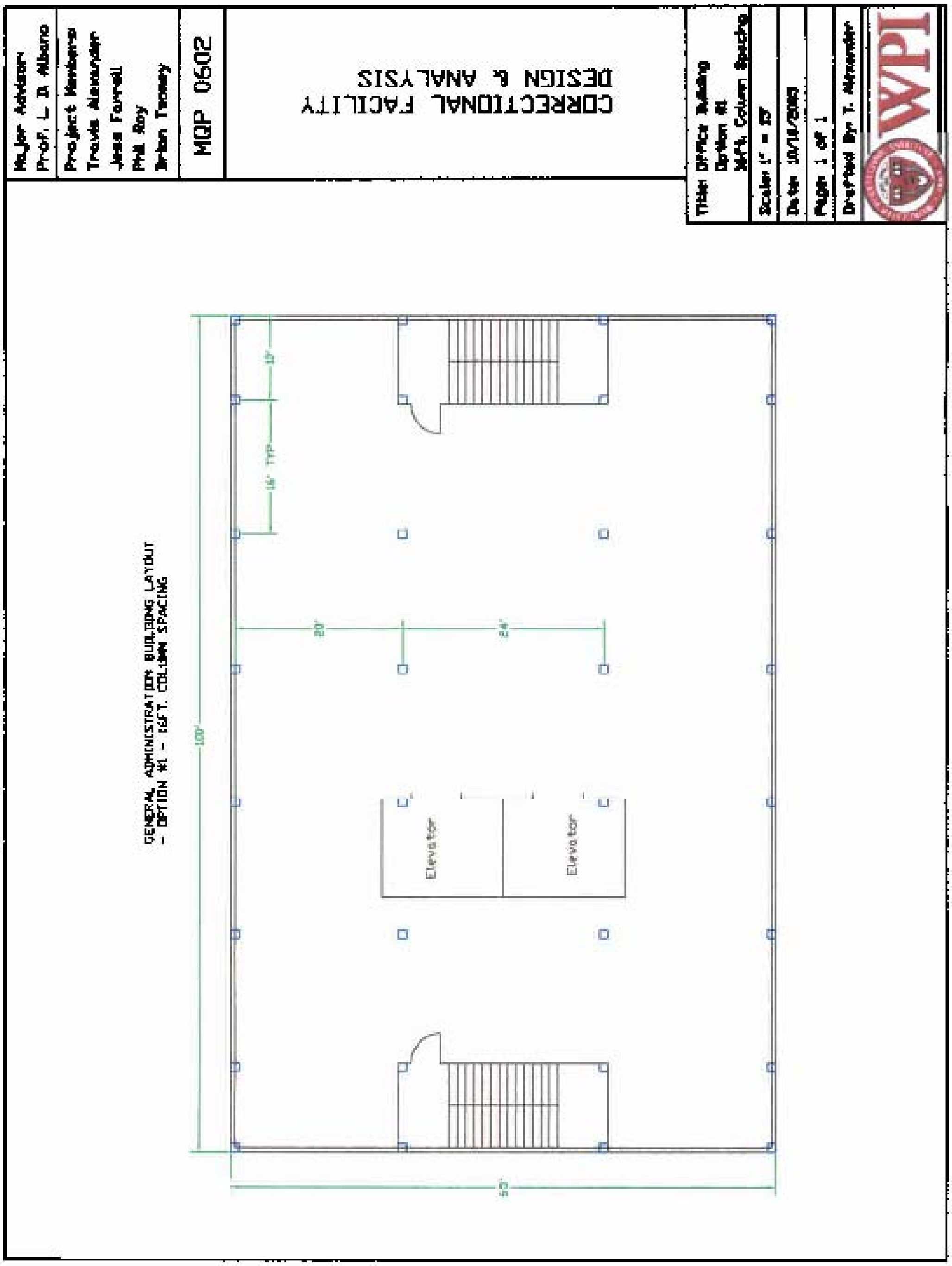
Note: Concrete, Ready Mix: $81.00/cyd
Concrete placing: $18.45/cyd.
Concrete finishing: $0.29/s.f. - screen finish
Concrete formwork: $5.90/s.f. - 4 use

Diaphragm Continuity Shear Studs

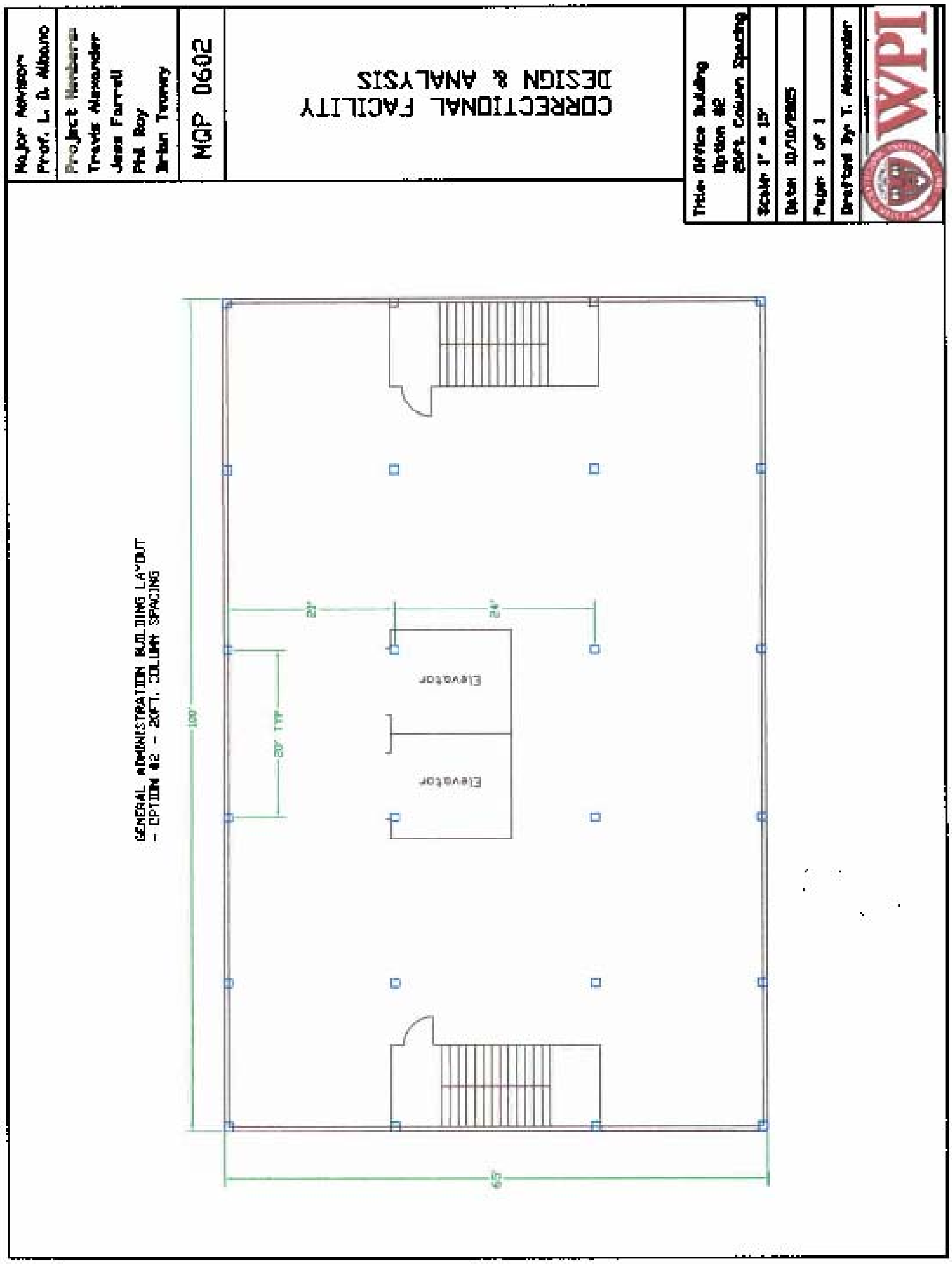
Linear Feet (total)	Max Spacing (in.)	Total # Studs	Installed Shear Studs	Total Cost Shear Studs
3537.81	22	1930	1.32	**2547.22**

Grand Total for Prison Cell Block Cost = $1,256,209.63

B.9 Drawings



Major Advisor:
Prof. L. D. Albano
Project Members:
Travis Alexander
Jess Farrell
Phil Roy
MQP 0602
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Page 1 of 1
Drafted By: T. Alexander
WPI
GENERAL ADMINISTRATION BUILDING LAYOUT
- OPTION #1 - 16FT. COLUMN SPACING
Elevator
Elevator
100'
53'
20'
24'
16' TYP
19'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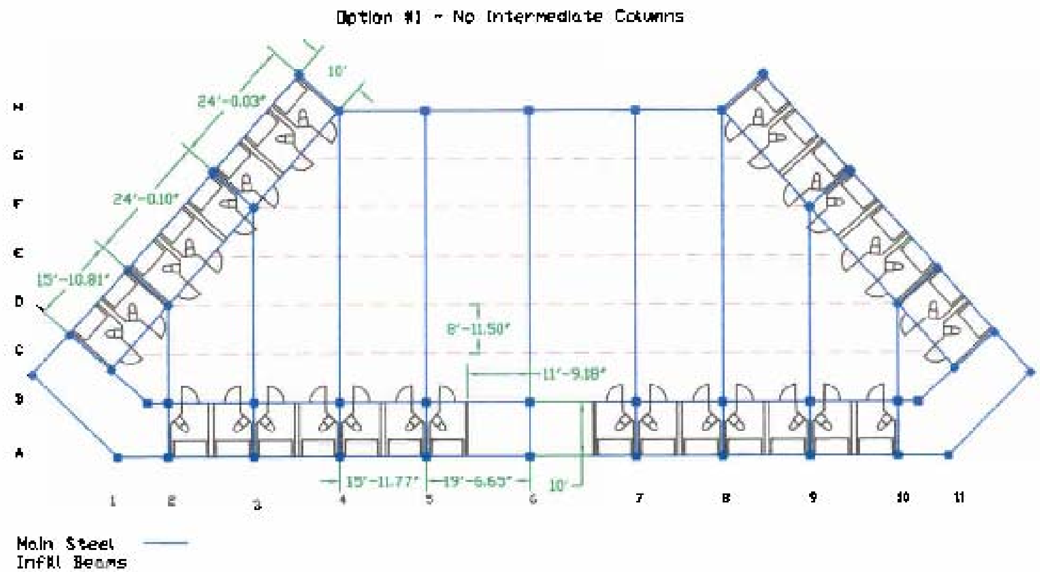
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Title: Office Building
Option #2
20Ft. Column Spacing
Scale: 1" = 15'
Page 1 of 1
Drafted By: T. Alexander
WPI
GENERAL ADMINISTRATION BUILDING LAYOUT
- OPTION #2 - 20FT. COLUMN SPACING
21'
24'
Elevator
Elevator
20' TYP
100'
65'



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Project Members
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Jess Farrell
Phil Roy
Brian Twomey

MQP 0602

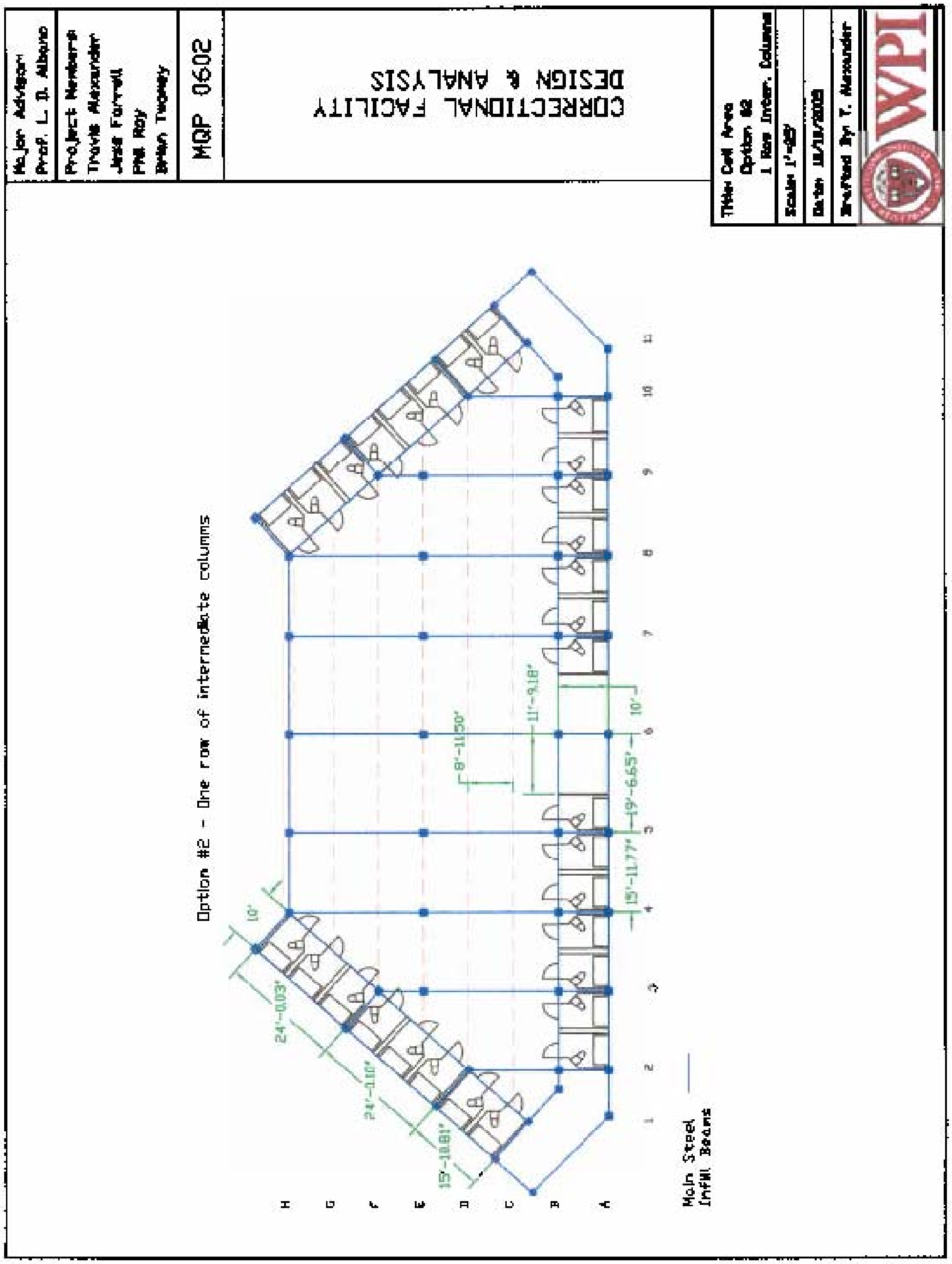
CORRECTIONAL FACILITY
DESIGN & ANALYSIS

Title: Cell Area
Option #1
No Inter. Columns

Scale: 1"=25'

Date: 10/18/2005

Drafted By: T. Alexander


Major Advisor
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Jess Farrell
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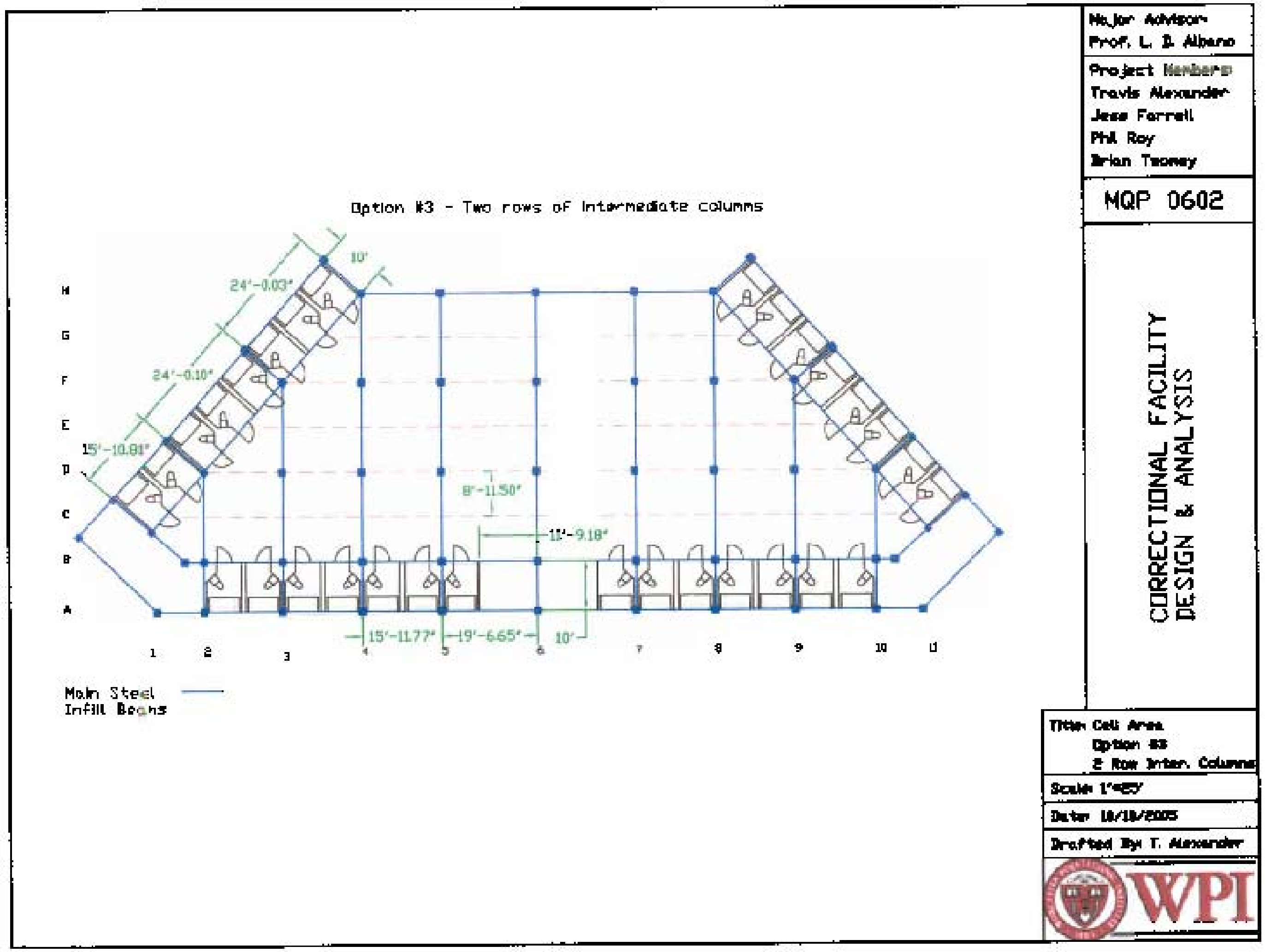
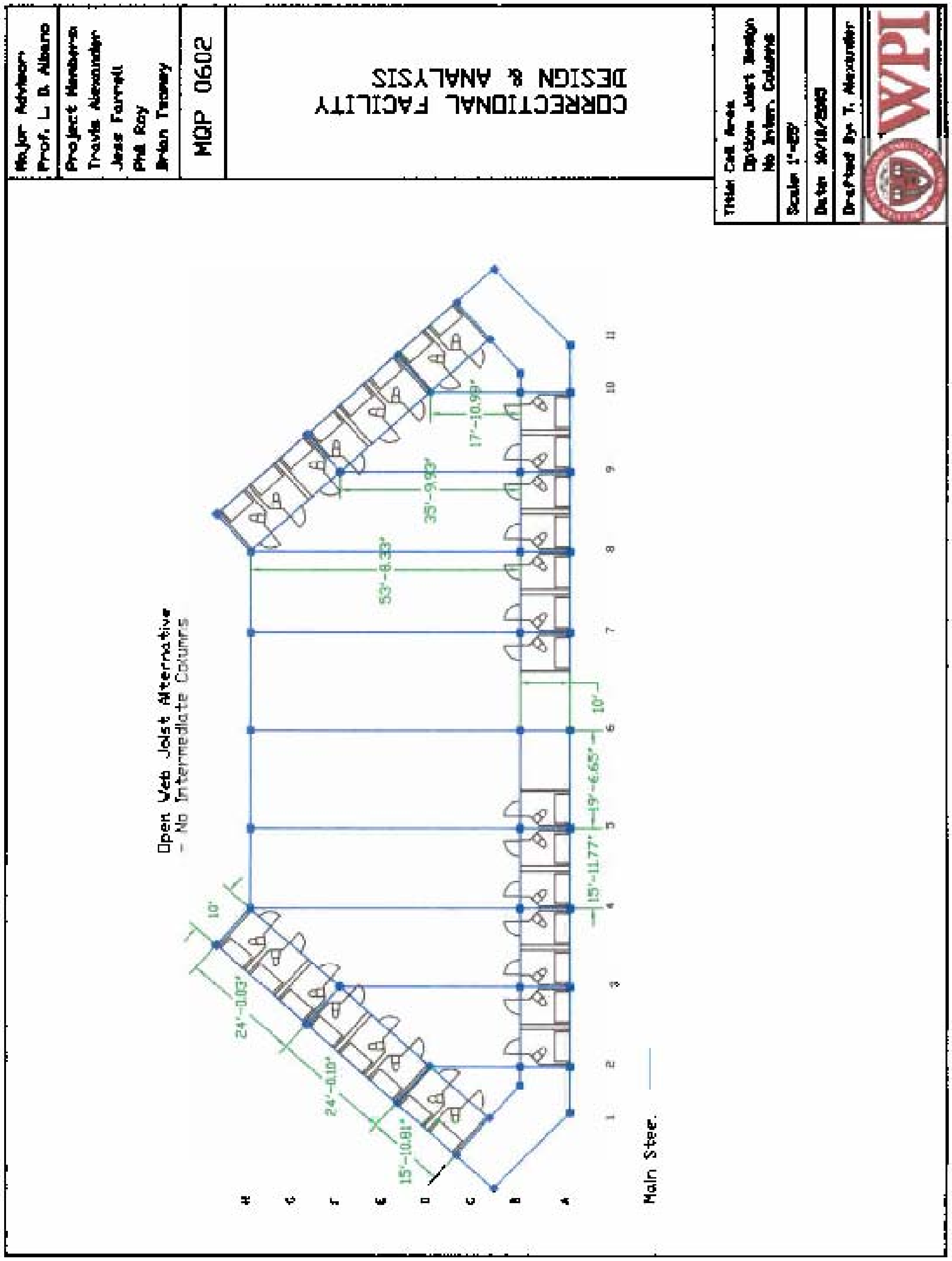
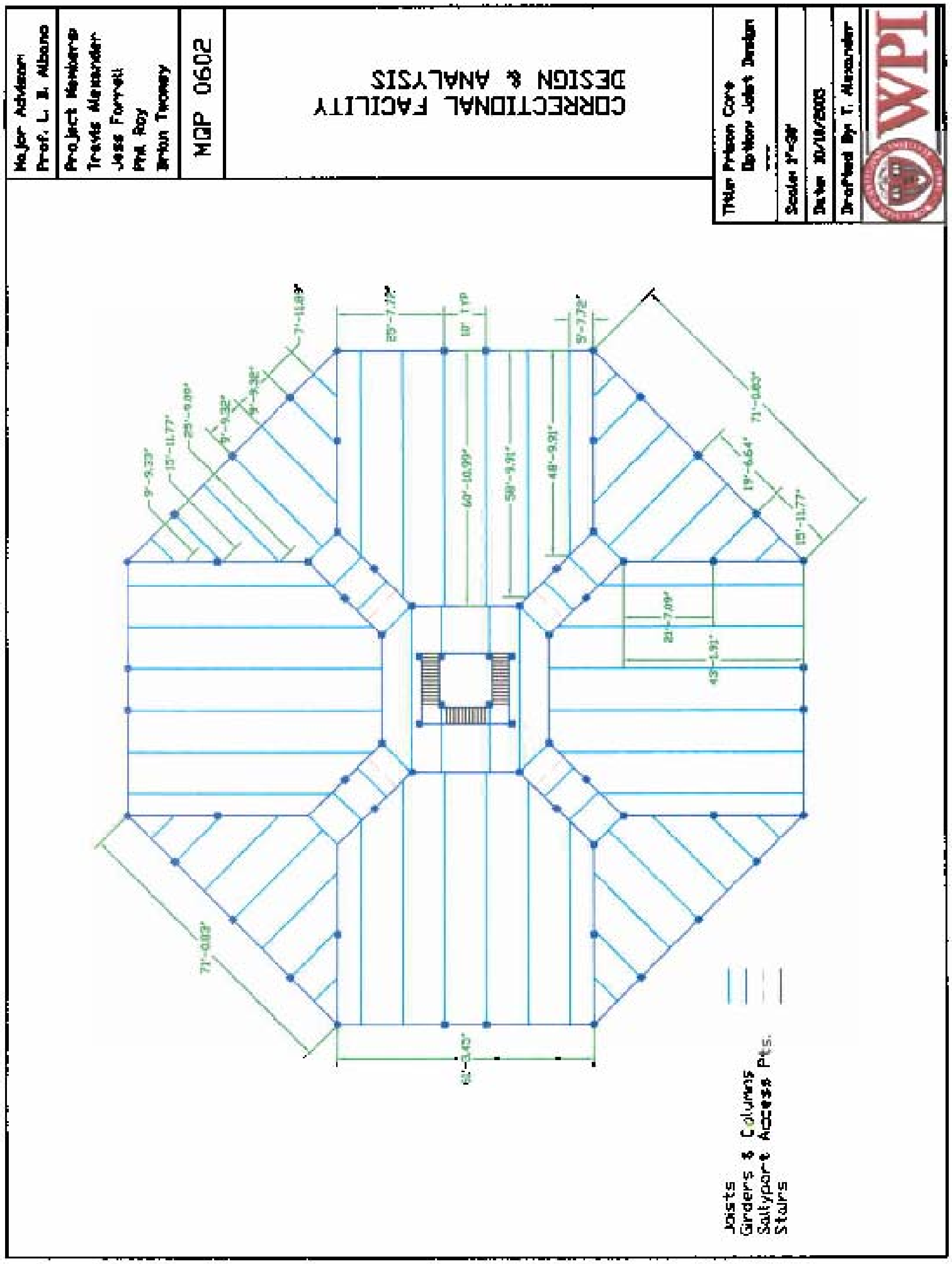
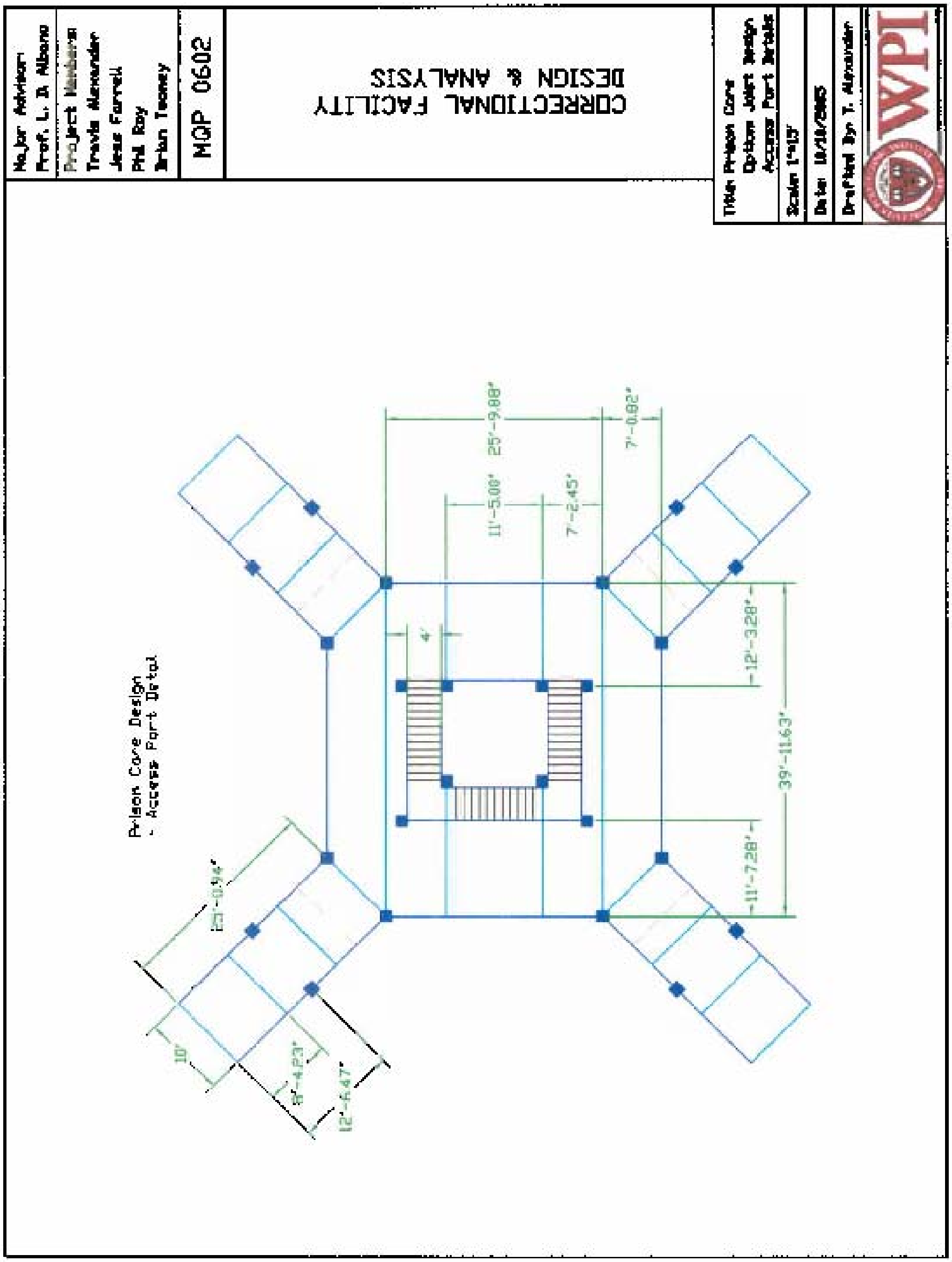
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Title Cell Area
Option #2
1 Row Inter. Columns
Drafted By T. Alexander
WPI
Option #2 - One row of intermediate columns
Main Steel
Infill Beams


Major Advisor:
Prof. L. D. Albano
Project Members:
Travis Alexander
Jess Ferrell
Phil Roy
Brian Tooney
MQP 0602
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Title: Cell Area
Option #3
2 Row Inter. Columns
Drafted By: T. Alexander
Option #3 - Two rows of intermediate columns
10'
24'-0.03"
24'-0.10"
15'-10.81"
8'-11.50"
11'-9.18"
15'-11.77"
19'-6.65"
10'
H
G
F
E
D
C
B
A
1
2
3
4
5
6
7
8
9
10
11
Main Steel
Infill Beams


Major Advisor:
Prof. L. D. Albano
Project Members:
Travis Alexander
Jess Farrell
Phil Roy
Brian Tracey
MQP 0602
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Title: Cell Area
Option: Joist Design
No Interm. Columns
Drafted By: T. Alexander
WPI
Open Web Joist Alternative
- No Intermediate Columns
53'-8.33"
35'-9.93"
17'-10.99"
10'
19'-6.65"
15'-11.77"
24'-0.03"
24'-8.10"
15'-10.81"
Main Stee.


Major Advisor
Prof. L. D. Albano
Project Members
Travis Alexander
Jess Forrest
Phil Roy
Brian Twomey
MQP 0602
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Title: Prison Core
Option: Joist Design
Scale: 1"=30'
Date: 10/18/2005
Drafted By: T. Alexander
WPI
60'-10.99"
58'-9.91"
48'-9.91"
71'-0.83"
71'-0.83"
Joists
Girders & Columns
Sallyport Access Pts.
Stairs



Major Advisor:
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Project Members
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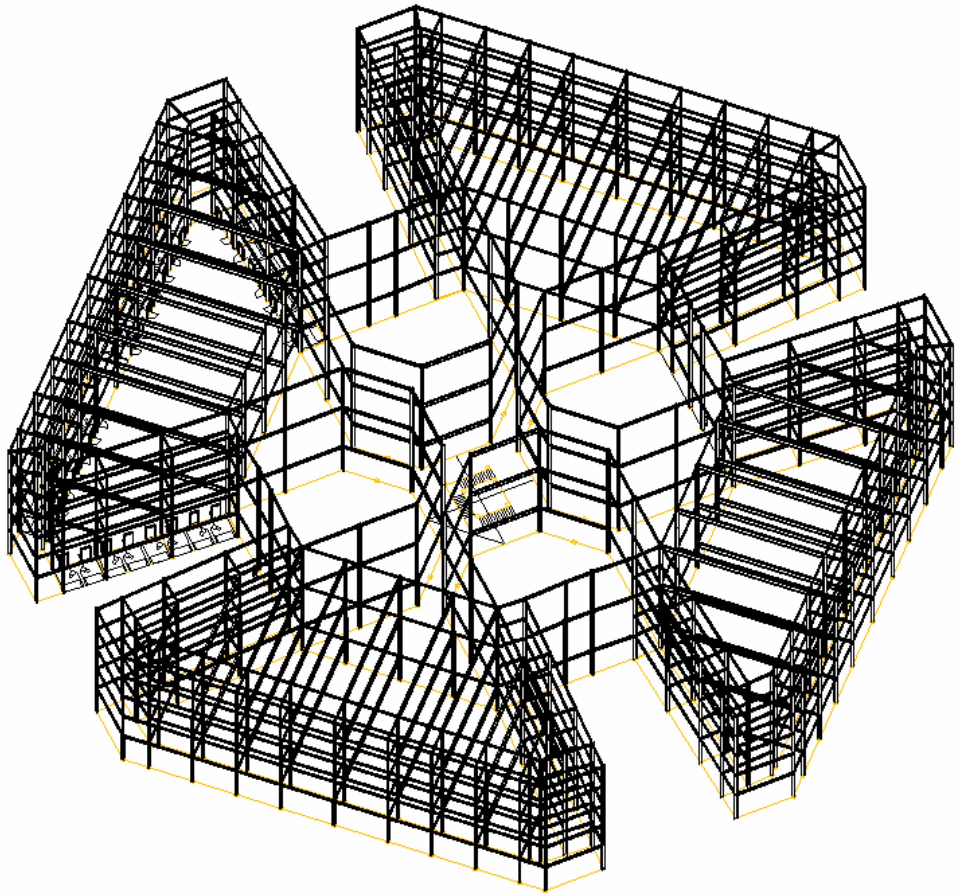
Jess Farrell
Phil Ray
Brian Twomey
MQP 0602
CORRECTIONAL FACILITY
DESIGN & ANALYSIS
Prison Core Design
- Access Port Detail
25'-9.88"
11'-5.08"
7'-2.45"
7'-0.82"
4'
12'-3.28"
39'-11.63"
11'-7.28"
10'
WPI



C Spreadsheet Instructions

C.1 Composite WF-Shape Beam Design

C.1.1 Spreadsheet Description

A spreadsheet was designed to provide an aid in the design of composite steel beam sections. This spreadsheet proved to be helpful when performing the repetitive calculations associated with composite design. Furthermore, by attaching and referencing the W-shape section list provided by the AISC, little time wasted because of having to enter section properties.

To begin using the spreadsheet, the unfactored dead loads and live loads need to be determined. The unfactored dead loads may include but are not limited to the self weight of the concrete, partitions, mechanical, flooring, and ceiling. This value should not include an estimated beam self weight because it is factored in later. The live load values are generally determined by IBC 2003 or other appropriate building codes. These loads need to be in pounds per square foot. The tributary width and the length of the beam as well as the slab depth also need to be inputted into the spreadsheet. The properties of the composite section such as the yield strength of the steel (F_y), modulus of elasticity of the steel (E_s), yield strength of the concrete (f'_c), and the yield strength of the shear studs (F_u) need to be defined.

	A	B	C	D	E	F
1	**Composite Beam Design**					
2						
3						
4	**Given Information**					
5	Dead Load?	71.88	psf			
6	Live Load?	70.00	psf			
7	Tributary Width?	9.12	ft	=	109.40	in
8	Length of Beam?	20.71	ft	=	248.52	in
9	Fy	50.00	ksi			
10	Es	29000.00	ksi			
11	f'c	3.00	ksi			
12	Fu	65.00	ksi			
13	Slab depth (t)	4.50	in			
14						
15	**Load Combinations**					
16	1.4D	917.36	lb/ft			
17	1.2D + 1.6L	1807.38	lb/ft			
18	This Value Governs	1807.38	lb/ft			
19						
20	Mu	96898.97	ft-lb	=	96.90	ft-k
21	Z	25.84	in³			
22						
23	*Trial Beam?*	**W12x22**				
24	As	W12x35 W12x30				
25	Z	W12x26	?			
26	*Uniform load of beam?*	W12x22	ft			
27		W12x19				
28	Recalculated wu	W12x16	ft			
29	Recalculated Mu	W12x14 W10x112	b	=	99.15	ft-k
30	Recalculated Z	26.44	in³			
31	Is the Beam Adequate?	**Beam is Adequate**				

	A	B	C	D	E	F	G
33	**Local Buckling**						
34	Flange:						
35	bf/(2tf)	4.70					
36	Buckling?	No Buckling					
37	Web:						
38	h/(tw)	41.80					
39	Buckling?	No Buckling					
40	*Section is Compact*						
41							
42	**Effective Flange Width (be)**	62.13	in				
43							
44	**Determining ϕbMn**						
45	depth of beam (d)	12.31	in				
46	tw	0.26	in				
47	h/tw	41.80	in				
48	h	10.87					
49	ϕb is	0.85					
50							
51	**Locate PNA**						
52	a is	2.05	in				
53	PNA is in	the SLAB					
54	Mn = Mp =	3120.92	in-k	=	260.08	ft-k	

	A	B	C	D	E	F	G
55	**ϕbMn is**	221.07	ft-k				
56							
57	**Design of Shear Studs**						
58	Stud Diameter?	0.75	in				
59	Shear (Vn) is	324.00	k				
60	Qn is	21.58	k		Ec if w is 150 psf	=	3181.98 ksi
61	# of studs required	16.00	studs				
62	Stud Spacing	7.77	in/stud				
63	Minimum Spacing	4.50	in				
64	Maximum Spacing	36.00	in				

Once the above values are specified, an estimated moment value and corresponding Z-value are generated. With this initial Z-value, a trial size can be chosen. The trial size is chosen using a drop down menu. This drop down menu is made possible by referencing an attached spreadsheet that has all the W-shaped cold-rolled steel cross-sections and their corresponding properties provided by the <u>AISC Manual of Steel Construction Load and Resistance Factor Design</u>. When choosing a W-shape cross-section with the drop down menu, the required properties for the particular shape are implemented without having to manually type them in. The only value that needs to be manually typed is the uniform weight of the beam.

After the trial size W-shape is chosen, the spreadsheet performs a series of checks to validate whether or not the chosen trial size and corresponding composite section is adequate using the Load Resistance Factor Design method. The checks that are performed include checking for local buckling in the flanges and web of the W-shape, determining the effective flange width (b_e), determining the nominal carrying capacity of the composite section (ϕM_n), locating the plastic neutral axis, and designing the shear studs by calculating the shear, determining capacity per each ¾” diameter stud, the required number of shear studs per half length, and the corresponding spacing.

Composite section deflections are also calculated in the spreadsheet. The deflection due to the unfactored dead loads (including the beam self weight) and the live loads are calculated using the centroid and the moment of inertia of the composite section. Serviceability concerns are also checked by examining the construction deflection. This deflection is based on the beam having to sustain all the loads before the concrete has cured to 75 percent of its strength. The unfactored load used in this deflection check is based on the beam self weight, the concrete self weight plus an extra ten percent for ponding, and a construction load of 20 pounds per square foot. Seventy-five percent of the deflection due to construction must be less than or equal to the maximum allowable deflection, which is also calculated within the spreadsheet.

Finally, an unshored construction check is also calculated. The factored loads used in this check include the beam weight as a dead load, and the concrete weight plus ten percent for ponding and a construction load both as live loads. With these loads, a corresponding Z-value is generated and verified to be less than the Z-value of chosen trial size W-shape.

66	**Composite Section Deflection**						
67	As	6.48	in^2				
68	Ys	6.16	in				
69	Ac	6.48	in^2				
70	Yc	15.79	in				
71	Y	10.97	in				
72							
73	**Moment of Inertia**						
74	Is	156	in^4				
75	I	456.62	in^4				
76							
77	**Calculated Deflection**						
78	Δ	0.42	in				
79							
80	**Unshored Construction Check**						
81	Wu = 1.2DL+1.6LL(concrete)	1236.28	lb/ft				
82	Mu	66.28	ft-k				
83	z	17.67	in^3				
84	Is the Beam Adequate?	**Beam is Adequate**					
85							
86	**Serviceability Concerns**						
87	Δmax	0.69	in				
88	Δconstruction	0.71	in	if W (just DL)	=	781.43	lb/ft
89	75% Δconstruction	0.54	in				
90	Is the Beam Adequate?	**Beam is Adequate**					

As the spreadsheet goes through all of the previously described checks, it lets the user know whether the composite section is adequate or inadequate. If the trial size W-shape fails one or more of the checks, the user is able to see which check failed and the user is then able to decide on a different wide flange section appropriate for the loading condition.

C.1.2 Spreadsheet User Guide

Load and Resistance Factor Design procedure is used for calculating composite beams in this spreadsheet. Plastic analysis is performed to determine the member sizes. Note: all text highlighted in blue in column A is information that must be entered manually.

1. Enter loading conditions (dead load and live load) without any load factors into cells B5 and B6.

2. Enter the tributary width and length of beam in feet into cells B7 and B8.

3. The uniformly distributed load, w_u, is calculated based on various load combinations. These include 1.4DL and 1.2DL+1.6LL. The governing value is determined by the spreadsheet and presented in cell B18.

4. The resulting moment, M_u, from the applied loads is calculated (cell B20).

$$M_u := \frac{w \cdot L^2}{8}$$

5. The plastic section modulus, Z_x, is determined (cell B21).

$$Z := \frac{M_u}{\Phi_b \cdot F_y}$$

6. Pick a trial size. A drop down menu is offered in cell B23 that references all the W-flange sections provided by the AISC <u>Manual of Steel Construction: Load and Resistance Factor Design</u> 3rd Edition. Applicable section properties are automatically referenced and inputted into the spreadsheet once a trial size is chosen.

7. Enter the uniform beam weight (cell B26)

8. The uniformly distributed load, w_u, is recalculated to include the beam self weight (cell B28).

9. M_u is recalculated using the revised w_u value (cell B29).

$$M_{u\_recalc} := M_u + \frac{1.2 \cdot W_{beam} \cdot L^2}{8}$$

10. The Z_x value is determined based off of the revised M_u (cell B30).

$$Z_{recalc} := \frac{M_{u\_recalc}}{\Phi_b \cdot F_y}$$

11. The trial size adequacy is checked to carry the applied loads (cell B31).

If $Z_x > Z_{recalc}$, then "Beam is Adequate".

12. The flange local buckling check is performed (cell B36).

If $\frac{b_f}{2t_f} < 0.38\sqrt{\frac{E}{F_y}}$, then "No Buckling".

13. The web local buckling check is performed (cell B39).

If $\frac{h}{t_w} < 3.76\sqrt{\frac{E}{F_y}}$, then "No Buckling".

14. The effective flange width of composite slab, b_e, is determined (cell B42).

b_e is the less value of $b_e := \text{Trib\_width}$ and $b_e := \frac{L}{4}$.

15. The location the plastic neutral axis, a, is derived (cell B52).

$$a := \frac{A_s \cdot F_y}{0.85 f_c \cdot b_e}$$

If $a < t_s$, then the PNA is in "the SLAB".

16. The nominal carrying capacity, ΦM_n, of the composite section is determined (cell B55).

$$\Phi M_p := \Phi_b \cdot F_y \cdot Z_x$$

17. Design shear studs is performed next.

    a. Determine resulting shear, V_n (cell B59).

$$V_n := 0.85 f_c \cdot t_s \cdot b_e > F_y \cdot A_s$$

b. Determine the number of shear studs required (cell B61).

$$n := \frac{V_n}{Q_n}$$

c. Determine shear stud spacing (cell B62).

$$6 \cdot \text{stud diameter} < \text{spacing} := \frac{\frac{L}{2}}{n} < 8 \cdot t_s$$

18. The composite section moment of inertia is evaluated.

d. Determine centroid of composite section, Y (cell B71).

$Y := \dfrac{A_s \cdot Y_s + A_c \cdot Y_c}{A_s + A_c}$ where $Y_s := \dfrac{d}{2}$, $A_c := \dfrac{0.85 \cdot f_c \cdot a \cdot b_e}{F_y}$, and $Y_c := d + t_s - \dfrac{a}{2}$

e. Determine moment of inertia, I (cell B75).

$$I := I_s + A_s \cdot (Y - Y_s)^2 + A_c \cdot (Y_c - Y)^2$$

19. Deflection, Δ, is calculated based on the applied loads (cell B78).

$$\Delta_{Wu} := \frac{5 w_u \cdot L^4}{384 \cdot E \cdot I}$$

20. Unshored construction check is considered (cell B84).

If $Z_x > Z_{construction}$ where $Z_{construction} := \dfrac{\dfrac{w_{construction} \cdot L^2}{8}}{\phi F_y}$, then "Beam is Adequate".

21. Checks for serviceability concerns are performed (cell B90).

If $75\%\Delta_{construction} < \Delta_{max}$, then "Beam is Adequate".

$$\Delta_{construction} := \frac{5 \cdot w_{constructionDL} \cdot L^4}{384 \cdot E \cdot Is}$$

$$\Delta_{max} := \frac{L}{360}$$

C.2 Vulcraft Composite Open-web Joist Design

C.2.1 Spreadsheet Description

A spreadsheet was designed to provide an aid in the design of composite open web joist sections. This spreadsheet proved to be helpful when performing the repetitive calculations associated with steel joist design. Unlike the wide flange design spreadsheets, all input values must be entered manually, for there is no reference table for the joist in the spreadsheet. This spreadsheet is specific to be used with the *Vulcraft Composite and Noncomposite Floor Joists* and design process.

To begin the design process, the length and the tributary width of the joist under consideration need to be inputted into the spreadsheet. The appropriate loads need to be entered. There are three load categories to be considered for deflection. These include composite live loads, composite dead load, and non-composite dead load. Composite dead loads are loads that are applied after the concrete has reached an acceptable strength (75% yield strength). These loads include but are not limited to mechanical, ceiling, and flooring. The non-composite dead loads consist of the concrete self weight, an estimated joist self weight, the weight of the decking, and the bridging. The concrete and decking are based off of tabulated values provided in the Vulcraft specifications, which are a function of the composite live load.

Once the loads are determined, the spreadsheet calculates the allowable deflection based L/360. Based off of the allowable deflection, the spreadsheet provided the required moment of inertia (I) which is based on the allowable deflection and the non-composite dead loads. Based off of this I-value, a trial size joist can be chosen. With a specified trial size, information such as the joist depth, load category, weight (pound per linear foot), acceptable live load to give the maximum deflection (W_{360}), and the size and number of shear studs required for the entire length can be determined using the design tables provided by Vulcraft.

	A	B	C	D	E	F
1	**Guard Section - Joist J 1-2**					
2	**Location**					
3	Length	15.98	ft			
4	Trib. Width	8.89	ft			
5						
6	**Composite Live Load**					
7	Occupancy	100.00	psf			
8	Reduciton Factor	1.00				
9	Reduced Occupancy	100.00	psf			
10	Other	5.00	psf			
11	*Total*	105.00	psf			
12						
13	**Composite Dead Load**					
14	Mechanical	15.00	psf			
15	Ceiling	2.00	psf			
16	Flooring	10.00	psf			
17	Other	0.00	psf			
18	*Total*	27.00	psf			
19						
20	**Noncomposite Dead Load**					
21	Concrete	33.00	psf	Thickness =	2	in. (above ribs)
22	Decking	1.78	psf	Weight =	145	pcf
23	Bridging	0.10	psf	Type =	1.5VL22	
24	Joist (approximation)	5.00	psf	Rib Height =	1.5	in.
25	*Total*	39.88	psf			
26						
27	**Total Loads**	1527.73	lb/ft			
28						
29	**Allowable Deflection**					
30	Δ_{allow}	0.53	in			
31	$I_{req'd}$	35.59	in^4			
32						
33	**Trial Size**					
34	Joist	16VC 1600/105/27				
35	Depth	16.00	in			
36	Total Load Category	1600.00	lb/ft			
37	W_{TL}	12.00	lb/ft			
38	W_{360}	2017.00	lb/ft			
39	N-ds	20 - 1/2		Stud Dia.	1/2	in
40				Stud Length	3 1/4	in

After the trial size is chosen, the spreadsheet performs a series of checks to validate whether or not the chosen trial size and corresponding composite section is adequate. The deflections due to the non-composite dead lead, composite dead load, and the composite live load are calculated. These deflections should be allowable because the trial size was chosen based off of calculating the moment of inertia with respect to the maximum allowable deflection. The recommended camber is also given as a function of all three deflections.

A final deflection check based off of construction concerns is also calculated. The loads included in this deflection check consist of the concrete (plus ½" for ponding), decking, bridging, joist, and construction weights equal to five two-hundred pound workers and/or equipment.

41	**Deflection Checks**		
42	$I_{[illegible]}$	149.91	in^4
43	$\Delta_{[illegible], DL}$	0.13	in
44	$\Delta_{[illegible], DL}$	0.06	in
45	$\Delta_{[illegible], DL}$	0.24	in
46	Δ_{TL}	0.21	in
47	Camber Required?	0.30	in
48			
49	**Construction Deflection Check**		
50	Concrete	39.04	psf
51	Decking	1.78	psf
52	Bridging	0.10	psf
53	Joist	1.35	psf
54	Construction	7.04	psf
55	Other	0	psf
56	*Total*	49.31	psf
57	$\Delta_{[illegible]}$	0.16	in
58			
59	Adequate?	Adequate	
60			
61	[1] Reference: Vulcraft Composite and Noncomposite Floor Joists		

C.2.2 Spreadsheet User Guide

The spreadsheet that this user guide outlines follows the design procedure given within the Vulcraft Composite and Noncomposite Floor Joists Manual. Note: all text highlighted in blue in column A is information that must be entered manually.

1. The length and tributary width of the joist under design must be entered in units of feet (cells B5 and B6).

2. The composite live load has to be inputted (cell B9). Composite live loads are those expected once the structure is fully erected (occupancy loads). A reduction factor is calculated based on the tributary area of the member and is applied to reduce the occupancy live load (cells B10 and B11, respectively).

$$R_o = 0.25 + \frac{15}{\sqrt{2\cdot(\text{Trib area})}}$$

$$LL_o = R_o \cdot LL$$

   Other live loads may be entered which would not be affected by the occupancy live load reduction (cell B12). A sum of the composite live loads is tallied (cell B13).

3. Composite dead loads may be entered next. Composite dead loads are those resulting from finishing materials. Locations for mechanical, ceiling, flooring, and *other* dead load values are given (cells B16 through B19). A sum of the composite dead loads is tallied (cell B20).

4. Non-composite dead loads are the last to be entered. These include the weight of the structural members. Locations for concrete, decking, bridging, and approximate joist weights are given (cells B23 through B26). To determine the type of decking and concrete to use, considerations for the clear span and composite live loads must be given and referenced within the Vulcraft Composite and Noncomposite Floor Joists. The concrete thickness above the decking ribs, the weight of the concrete, the type of decking, and the rib height should be specified (cells E23 through E26).

5. Allowable deflection, $\Delta_{allowable}$, is then calculated. (cell B32).

$$\Delta_{allowable} = \frac{L}{360}$$

6. The required moment of inertia, $I_{required}$, is then determined based on the total non-composite dead loads and $\Delta_{allowable}$.

$$I_{required} = 1.15 \cdot \left(\frac{5}{384 \cdot E \cdot \Delta_{allowable}}\right) \cdot \left[DL_{NC} \cdot (\text{Trib width})\right] \cdot [L(in) - 4(in)]^4$$

7. A trial size must next be entered that satisfies $I_{required}$. Section properties must also be entered including the joist size, depth, total weight category (must be greater than the sum of the composite live loads, composite dead loads, and non-composite dead loads), the uniform weight of the joist, and allowable allied uniform weight to satisfy $\Delta_{allowable}$, and the quantity and size of the required shear studs (cells B36 through B41). Such information is attained when referencing the Vulcraft <u>Composite and Noncomposite Floor Joists</u> manual.

8. Deflection checks are performed next.

   a. The moment of inertia, I_{calc}, for the chosen joist section is calculated (cell B44).

   $$I_{calc} = 0.0488 W_{TJ} \cdot D^2$$

   b. Deflection resulting from non-composite dead loads, $\Delta_{noncompDL}$, is determined (cell B45).

   $$\Delta_{noncompDL} = 1.15 \cdot \left(\frac{5}{384 \cdot E \cdot I_{calc}}\right) \cdot \left[DL_{NC} \cdot (\text{Trib width})\right] \cdot [L(in) - 4(in)]^4$$

   c. Composite dead load deflection, Δ_{compDL}, is calculated (cell B46).

   $$\Delta_{compDL} = \frac{DL_{comp} \cdot (\text{Trib} \cdot \text{width})}{W_{360}} \cdot \frac{L(in) - 4(in)}{360}$$

   d. A final deflection check for composite live loads, Δ_{compLL}, is performed (cell B47).

   $$\Delta_{compLL} = \frac{LL_{comp} \cdot (\text{Trib} \cdot \text{width})}{W_{360}} \cdot \frac{L(in) - 4(in)}{360}$$

   e. Total deflection, Δ_{TL}, is summed (cell B48).

   $$\Delta_{Total} = \Delta_{noncompDL} + 0.5 \cdot \Delta_{compDL} + 0.2 \cdot \Delta_{compLL}$$

   f. The camber required is then given (cell B49).

9. Construction deflection checks are then calculated.

a. Construction loads must be entered. These values are referenced from earlier in the spreadsheet.

b. Deflection caused by the construction loads, $\Delta_{construction}$, is calculated.

$$\Delta_{construction} = 1.15 \cdot \left(\frac{5}{384 \cdot E \cdot I_{calc}}\right) \cdot \left[W_{construction\_total} \cdot (\text{Trib width})\right] \cdot [L(in) - 4(in)]^4$$

$\Delta_{construction} < \Delta_{allowable}$.

C.3 WF-Shape Beam and Girder Design

C.3.1 Spreadsheet Description

A spreadsheet was designed to provide an aid in the design of non-composite steel beams and steel girder sections. This spreadsheet proved to be helpful when performing the repetitive calculations associated with steel beam design. Furthermore, by attaching and referencing the W-shape section list provided by the AISC, little time wasted because of having to enter section properties.

To begin using this spreadsheet, steel properties such as modulus of elasticity (E_s), yield stress (F_y), critical buckling factor (C_b), and load resistance factor for bending (Φ_b) need to be defined. Information defining the area to be carried by the structural member needs to be entered. These include the beam length and tributary width.

The loads to be supported by the steel member need to be specified. If point loads are present on the member, both the factored and unfactored loads need to be inputted along with the load location from one end of the member. It is recommended to use the left end of the member as the 0ft mark when defining the point load locations for consistency. If the design of a girder is being performed, the point loads are equal to the end reactions of the infill members that tie into the girder. Furthermore, the uniformly distributed loads need to be entered. These include any dead load, live load, and snow load. Three applicable load combinations are calculated and the one resulting in the highest uniform load is used throughout the calculations. An unfactored uniform load is also calculated.

Once the loads, both point loads and uniformly distributed loads, have been defined, the moments resulting from these loads can be calculated. Based on the law of superposition, the moment caused by the point loads can be calculated separately from the moment caused by the uniformly distributed load. Consider the moment caused by the point loads. If only one point load is present at the center of the steel member, then the spreadsheet will provide the maximum moment equation to use as well as the value of the moment (M_u). If the point load is not located at the center of the steel member or if there are more than one point loads, then the user has to calculate the moment separately and enter the value into the designated cell. Moment equations are provided for up to five evenly spaced point loads as well as the equation for an offset singular point load. Finally, the spreadsheet will also calculate the moment resulting from the largest of the three calculated uniformly distributed loads.

Once the moments are summed up from the point load(s) (automatic or manually calculated) and the uniformly distributed loads, an estimated minimum Z-value is given. With this initial Z-value, a trial size can be chosen. The trial size is chosen using a drop down menu. This drop down menu is made possible by referencing an attached spreadsheet that has all the W-shaped cold-rolled steel cross-sections and their corresponding properties provided by the AISC Manual of Steel Construction Load and Resistance Factor Design. When choosing a W-shape cross-section with the drop down menu, the required properties for the particular shape are implemented without having to manually type them in. The only value that needs to be manually typed is the uniform weight of the beam. The spreadsheet automatically recalculates M_u and the Z-value because of the additional dead load of the previously unknown member size.

	A	B	C	D	E	F	G	H	I	J	K	L	M
1					**BUILDING DESIGN**								
2					**Beam-Girder Noncomposite Design**								
3					**Beam-Girder Location**								
4													
5	**Given Information**												
6	Φ_b	0.85											
7	E	29000	ksi										
8	F_y	50	ksi										
9	C_b (critical buckling)	1											
10													
11	**Beam Information**												
12	Beam Length	16.00	ft										
13	Tributary Width	8.00	ft										
14													
15	**Point Load Details**												
16	Loads (factored)	48.75	k (1)		k (2)		k (3)		k (4)		k (5)		
17	Loads (unfactored)	30.00	k (1)		k (2)		k (3)		k (4)		k (5)		
18	Load Location	8.00	ft (1)		ft (2)		ft (3)		ft (4)		ft (5)		
19													
20	**Uniform Load Details**												
21	Dead Load	10.00	psf										
22	Live Load	10.00	psf		LL Reduction =	1.00							
23	Snow Load	0.00	psf										
24	W_u =1.4DL(TW)	0.11	k/ft										
25	W_u = 1.2DL(TW)+1.6LL(TW)+0.5SL(TW)	0.22	k/ft										
26	W_u = 1.2DL(TW)+0.5LL(TW)+1.6SL(TW)	0.14	k/ft										
27	W_u (unfactored)	0.16	k/ft										
28													
29	**Moment**												
30	Moment	P*L/4		Wu(L^2)/8			n = 1	M_u = PL/4		n = 4	M_u = 3PL/5		Note: P_u is equal at all locations
31	M_u (calculate)	195.01	ft-k	7.17	ft-k		n = 2	M_u = PL/3		n = 5	M_u = 3PL/4		Note: The applied loads are evenly spaced in all equations
32	$M_{u\ Total}$	202.18					n = 3	M_u = PL/2		uneven	M_u = Pab/L		Note: In case n = 1, P_u is not applied in the center
33	Z	57.08	in^3										
34							**Manual Calucated Moment =**			---	ft-k		
35	**Trial Size**	W14x53											
36	A_s	W14x53											
37	d	W14x48 W14x43											
38	Z	W14x38											
39	I_x	W14x34 W14x30											
40	X_1	W14x26 W14x22											
41	X_2	0.00225	$(1/ksi)^2$										
42	S_x	77.8	in^3										
43	r_y	1.92	in.										
44	Beam Weight	53	lb/ft										
45													
46	**Recalculate M_u**	203.87	ft-k										
47	Recalculate Z	57.56	in^3										
48	Is the girder adequate?	Adequate											

After the trial size W-shape is chosen, the spreadsheet performs a series of checks to validate whether or not the chosen trial size and corresponding composite section is adequate using the Load Resistance Factor Design method. The checks that are performed include checking for local buckling in the flanges and web of the W-shape. Reduced moment capacity (ΦM_n) due to lateral torsional buckling of the member is calculated based on member unbraced length. The spreadsheet also calculates the plastic and inelastic inflection points of the ΦM_n vs. unbraced length (L_b), and further determine which mode (region) of bending the member undergoes. Furthermore, a moment capacity check is conducted based on the ΦM_n vs M_u to make certain that the member is not overloaded for its unbraced length.

Member deflections are also calculated in this spreadsheet. The deflection due to the unfactored dead loads (including the beam self weight) and the live loads is calculated.

As with the moment considerations, deflections can also be additive based on the law of superposition. For loading other than a point load at the midspan, the deflection must be calculated using the given deflection equations for up to five evenly spaced and one unevenly space point loads. The spreadsheet automatically calculated the deflection resulting from the uniformly distributed loads. The total deflection is the sum of the deflection due to the point loads and the uniformly distributed loads, which must be less than or equal to the maximum allowable deflection, which is also calculated within the spreadsheet.

50	**Local Buckling**									
51	Flange:									
52	bf/(2tf)	6.10								
53	Buckling?	No FLB								
54	Web:									
55	h/(tw)	30.80								
56	Buckling?	No WLB								
57	Is the girder compact?	Compact								
58										
59	**Lateral Torsional**									
60	*Unbraced Length, L_b*	8	ft							
61	L_p	6.78	ft							
62	L_r	20.07	ft							
63	ΦM_p critical point	308.48	ft-k							
64	ΦM_r critical point	220.43	ft-k							
65	Plastic Region?			ΦM_p	0.00	ft-k				
66	Inelastic Region?	**Inelastic**		ΦM_r	300.41	ft-k				
67	Elastic Region?			ΦM_{cr}	0.00	ft-k				
68										
69	**ΦM_n vs. M_u**									
70	Is the beam adequate?	Adequate								
71										
72	**Deflection**									
73	Due to P_u	(P*L^3)/(48EI)			n = 1	$\Delta = (Pa^2b^2)/(3EIL)$		n = 4	$\Delta = (PL^3)/(19.04EI)$	Note: P_u is equal at all locations
74	Deflection due to P_u	0.282	in		n = 2	$\Delta = (5PL^3)/(648EI)$		n = 5	$\Delta = (PL^3)/(29.716EI)$	Note: The applied loads are evenly spaced in all equations
75	Deflection due to W_u	0.020	in		n = 3	$\Delta = (3PL^3)/(128EI)$		uneven	$\Delta = (Pa^2b^2)/(3EIL)$	Note: In case n = 1, P_u is not applied in the center
76	Total Deflection	0.302	in							Note: In case n = 2, P_u is applied at L/3
77	Δ_{max}	0.533	in		**Manual Calucated Deflect. =**		---	in		
78	Is the girder adequate?	Adequate								

As the spreadsheet goes through all of the previously described checks, it lets the user know whether the girder section is adequate or inadequate. If the trial size W-shape fails one or more of the checks, the user is able to see which check failed and the user is then able to decide on a different wide flange section appropriate for the loading condition.

C.3.2 Spreadsheet User Guide

The procedures of the AISC Manual of Steel Construction: Load and Resistance Factor Design 3rd Edition were followed in the column design spreadsheet. The use of this spreadsheet, and the equations involved are discussed below. Note: all text highlighted in blue in column A is information that must be entered manually.

1. Enter values of F_y, E, Φ_c, and C_b in cells B4, B5, B6, and B7.

2. In cell B9, enter member length.

3. In row 11, enter factored point loads. Row 12 is for unfactored point loads, to be used for deflection checks later.

4. Load location should be entered in cell B13. If load is located at the midpoint of the member (half of value entered in B10), the moment will automatically be calculated in cell B16, and cell B15 will display

$$M_u = \frac{P \cdot L}{4}$$

   If not, cell B15 will display "See Mu eqns"

5. If cell B15 shows "See Mu eqns", refer to moment equations to the right, to manually calculate the moment on the member. Enter this manually calculated moment in cell H19. This manually calculated value entered in H19 will then automatically show up in cell B16.

6. Cell B17 will calculate a minimum plastic section modulus based on the moment shown in cell B16.

$$Z = \frac{M_u}{\Phi_b \cdot F_y}$$

7. Next, pick a trial size based on a Z value displayed in cell B23. A drop down menu is offered in cell B20 that references all of the W-flange sections provided by the AISC Manual of Steel Construction: Load and Resistance Factor Design 3rd Edition. Applicable section properties are automatically referenced and inputted into the spreadsheet once the trial size is chosen.

8. The beam weight must be entered manually into cell B29. (ex. W14x53 ~ 53 plf)

9. Cell B31 recalculates maximum moment M_u following

$$M_{u\_recalc} = M_u + \frac{W_{beam} \cdot L^2}{8}$$

10. Correspondingly, cell B32 recalculates Z

$$Z_{recalc} := \frac{M_{u\_recalc}}{\Phi_b \cdot F_y}$$

11. Cell B33 checks the recalculated plastic section modulus to that of the trial size chosen and determines whether the section is adequate based plastic section modulus only.

12. Cells B38 and B41 determine whether any flange or web local buckling will occur using limits

$$\frac{b_f}{2t_f} < 0.38 \cdot \sqrt{\frac{E}{F_y}}$$

$$\frac{h}{t_w} < 3.76 \sqrt{\frac{E}{F_y}}$$

If the section properties displayed in cells B37 and B40 are less that these limits, cells B38 and B41 will show "No FLB" and "No WLB". Cell B42 will display "Compact" or "NOT Compact" depending on web and flange local buckling.

13. Enter the laterally unbraced length into cell B45.

14. Cell B46 calculates the plastic limit for unbraced length

$$L_p = 1.76 \cdot r_y \cdot \sqrt{\frac{E}{F_y}}$$

15. Cell B47 calculates the inelastic limit for unbraced length

$$L_r = \left(\frac{r_y \cdot X_1}{F_y - F_r}\right) \cdot \sqrt{1 + \sqrt{1 + X_2 \cdot \left(F_y \cdot F_r\right)^2}}$$

16. Cells B48 and B49 calculate the plastic and inelastic moment capacity following the equations

$$\Phi M_p = \Phi_b \cdot F_y \cdot Z$$

$$\Phi M_r = \Phi\left(F_y - F_r\right) S_x$$

17. Depending on whether the section undergoes plastic, inelastic, or elastic bending, Cells B50 through B52 will display the bending mode based on the relationship of L_b, L_p, and L_r.

18. The moment capacity of the girder is displayed in cells D50 through D52 based on

$$\Phi M_n = \Phi M_p$$

$$\Phi M_n = \frac{\Phi M_p - \Phi M_r}{L_r - L_p} \cdot \left(L_r - L_b\right) + \Phi M_r$$

$$\Phi M_n = \left(\frac{\Phi_b \cdot C_b \cdot S_x \cdot X_1 \cdot \sqrt{2}}{L_b \cdot r_y}\right) \cdot \sqrt{\frac{1 + \left(X_1^{\,2}\right) \cdot X_2}{2 \cdot \left(\frac{L_b}{r_y}\right)^2}}$$

19. Cell B55 determines the adequacy of the beam based on whether the moment acting on the beam is less than or greater than the capacity calculated in cells D50 through D52.

20. Cell B58 will show the deflection equation due to P_u if the point load acts at the midpoint of the beam, otherwise it will display "other". If the display shows other, this means that the deflection due to P_u must be calculated manually and entered into cell H62.

21. Cell B59 will calculate the deflection of the beam due to P_u automatically if the point load acts at the midspan, using the equation

$$\Delta P_u = \frac{\left(P_u \cdot L\right)^3}{48 \cdot E \cdot I}$$

22. Cell B60 calculates the deflection due to the uniform dead load of the beam

$$\Delta W_u = \frac{5 w_u \cdot L^4}{384 \cdot E \cdot I}$$

23. Cell B61 adds the deflections found in cells B59 and B60. Cell B63 compares the total deflection calculated in B61 with the maximum allowable deflection calculated in B62 and determines the member's adequacy.

$$\Delta_{total} = \Delta_{Pu} + \Delta_{Wu}$$

$$\Delta allowable = \frac{L}{360}$$

$$\Delta_{total} < \Delta_{allowable}$$

C.4 WF-Shape Column Design (Axial Loads Only)

C.4.1 Spreadsheet Description

A spreadsheet was designed to provide an aid in the design of steel W-shape column sections. This spreadsheet proved to be helpful when performing the repetitive calculations associated with steel column design. Furthermore, by attaching and referencing the W-shape section list provided by the AISC, little time wasted because of having to enter section properties.

To begin using this spreadsheet, the user must understand that a column may support floors that have various dead and live loads. To accommodate this, three floor and three loading considerations are given. This means that the spreadsheet can be use effectively for the design of a column that has up to three different dead and live load combinations on various floors. The number of floors having a particular dead and live load combination must be entered as well as the applicable loads. Also, the tributary area of floor that the column supports as well as the tributary area of the exterior wall (if the column is an exterior column) must be entered.

Next, the unfactored dead and live loads need to be determined. For cases when the column supports multiple floors, the loads need to be entered for each floor load combination. These loads include dead load for the floors and roof, live load for the floors, snow load for the roof, wall dead load if applicable, and wind load if applicable. Once the loads have been inputted, the spreadsheet calculates various load combinations to determine the maximum applied axial load on the column. Multiple live load reductions are also calculated for each floor loading and revised axial load combinations are computed.

Steel properties need to be established for further calculations. These properties include the yield stress (F_y), modulus of elasticity (E), and load resistance for compression (ϕ_c). With these properties defined, a trial size can be determined. The trial size is chosen using a drop down menu. This drop down menu is made possible by referencing an attached spreadsheet that has all the W-shaped cold-rolled steel cross-sections and their corresponding properties provided by the <u>AISC Manual of Steel Construction Load and Resistance Factor Design</u>.

	A	B	C	D	E	F	G	H	
1	**Office Design Joist Alternative**								
2	**Column Design (20ft Spacing)**								
3	**B1 - Basment (w/ K-Brace Design)**								
4	**Gravity & Wind Loads**								
5	***# floors > column***	2	floors	3	floors	1	floors		
6									
7	**Tributary Area**	1st/Gym		Office		Living		Roof	
8	*Column TA*	112.5	ft^2	112.5	ft^2	112.5	ft^2	112.5	ft^2
9	*Wall TA*		ft^2						
10									
11	**Loads**								
12	*Floor Dead Load*	79.24	psf (a)	77.84	psf (b)	77.84	psf (c')		
13	*Floor Live Load*	100	psf (a)	80	psf (b)	40	psf (c')		
14	*Wind Load*	164.93	k (cumulative @ top of column)						
15	*Roof Dead Load*	77.84	psf						
16	*Roof Snow Load*	35	psf						
17	*Wall Dead Load*		psf						
18									
19	**Load Combos**								
20	P_u = (1.4DL)TA	86.26	k						
21	P_u = (1.2DL)TA + (1.6LL)TA + (0.5SL)TA	162.31	k						
22	P_u = (1.2DL)TA + (0.5LL)TA + (1.6SL)TA	107.24	k						
23	P_u = (1.2DL)TA + (1.6SL)TA + (0.8WL)	212.18	k						
24	P_u = (1.2DL)TA + (1.6WL) + (0.5LL)TA + (0.5SL)TA	366.79	k						
25	<u>Critical Loading</u>	366.79	k						
26									
27	**Live Load Reduction**								
28	N = 1-0.0008(TA-A_B)	0.99		0.99		1.11			
29	N = 0.75-0.20(DL/LL)	0.59	(a)	0.56	(b)	0.36	(c')		
30	N = 0.5 or 0.6	0.60							
31	<u>Critical LL Reduction</u>	0.99	(a)	0.99	(b)	1.00	(c')		
32									
33	**Revised Load Combos**								
34	P_u = (1.4DL)TA	86.26	k						
35	P_u = (1.2DL)TA + (1.6LL)TA + (0.5SL)TA	161.51	k						
36	P_u = (1.2DL)TA + (0.5LL)TA + (1.6SL)TA	106.99	k						
37	P_u = (1.2DL)TA + (1.6SL)TA + (0.8WL)	212.18	k						
38	P_u = (1.2DL)TA + (1.6WL) + (0.5LL)TA + (0.5SL)TA	366.54	k						
39	<u>Critical Loading</u>	366.54	k						
40									
41	**Steel Properties**								
42	F_y	50	ksi						
43	E	29000	ksi						
44	Φ	0.85							

Once the trial size W-shape section has been determined and the corresponding cross-sectional properties inputted, the column capacity can be determined. The end condition K-values need to be inputted by the user as well as the unbraced length in both the strong axis and the weak axis of the member. Once these values are defined, the spreadsheet calculates the capacity (λ_c). Based on the calculated capacity, the spreadsheet determines whether the column buckles inelastically or elastically. Depending on the mode of buckling, the spreadsheet uses the corresponding equation to determine the critical force (F_{cr}).

With the critical force calculated, the spreadsheet derives the critical axial load (ϕP_n) that the column can support. With this critical load determined, the spreadsheet is able to verify whether or not the W-shape trial size chosen is adequate by comparing the applied axial load with that of the critical axial load.

46	**Trial Size**		
47	*Trial Size - Table 4-2 AISC*	**W14x61**	
48	*A_s*	17.9	in^2
49	*X_1*	2720	ksi
50	*X_2*	0.00246	$(1/ksi)^2$
51	*Z*	102	in^3
52	*S_x*	92.2	in^3
53			
54	**Column Capacity**		
55	*K-value (end condition) X-X*	1	
56	*K-value (end condition) Y-Y*	1	
57	*Unbraced Column Length X-X*	15	ft
58	*Unbraced Column Length Y-Y*	15	ft
59	*Radius of Gyration X-X*	5.98	in
60	*Radius of Gyration Y-Y*	2.45	in
61	K_xL_x/r_x	30.10	
62	K_yL_y/r_y	73.47	
63	Maximum KL/r	73.47	
64			
65	**Evaluate Capacity**		
66	λ_c	0.97	
67	Inelastic or Elastic Zone?	**Inelastic**	
68	F_{cr}	(0.658^(λc^2))*Fy	
69	F_{cr}	33.70	ksi
70	ΦP_n	512.67	k
71			
72	**Is the beam adequate for axial loads?**		
73	$P_u/\Phi Pn$	0.71	
74	Is the beam adequate?	**Adequate**	

C.4.2 User Guide

The procedures of the AISC Manual of Steel Construction: Load and Resistance Factor Design 3rd Edition were followed in the column design spreadsheet. The use of this spreadsheet, and the equations involved are discussed below.

1. Enter the types of floor uses in cells B5, D5, and F5 (Lobby, Office, Living, etc.)

2. In the corresponding cells in row 6, enter the number of floors of each type that the column will carry.

3. In row 7, enter the tributary area of the column for each floor type.

4. In rows 11 and 12, enter the dead loads and live loads (in psf) corresponding to the above floor uses.

5. If any additional axial load on the column can be attributed due to wind load analysis of frame, this load can be entered in cell B13 with units of kips.

6. Roof dead load and snow load is entered next in cells B14 and B15 in psf.

7. Next the load combinations designated in the AISC LRFD manual are used in rows 19 through 23 to determine the maximum critical load, shown in cell B24.

8. Typically, live load reductions can be taken depending on tributary area of the column. Cells B27 through B29 compute the live load reduction factor for each floor type by calculating the maximum of:

$$N_{LL} = 1 - 8 \times 10^{-4} \cdot \left(\mathrm{TribArea} - A_B\right)$$

$$N_{LL} = 0.75 - 0.2\left(\frac{DL}{LL}\right)$$

$$N_{LL} = 0.6$$

9. Next, the live load reductions are applied to the load combinations to determine a new revised critical load P_u.

10. Values of F_y, E and Φ_c must be entered in cells B41, B42, and B43.

11. Next, pick a trial size. A drop down menu is offered in cell B46 that references all of the W-flange sections provided by the AISC Manual of Steel Construction: Load and Resistance Factor Design 3rd Edition. Applicable section properties are

automatically referenced and inputted into the spreadsheet once the trial size is chosen.

12. K values for both axes must be entered in cells B54 and B55, depending on the end fixities of the column.

13. Unbraced lengths for both axes of bending must then be inputted into cells B56 and B57.

14. Cells B60 and B61 display the KL/r values for both x and y axes, and the greater of the two values is then used to evaluate the column capacity.

15. Cell B65 calculates the λ_c using Euler's buckling equation:

$$\lambda_c = \frac{K\cdot(L)}{r\cdot\Pi}\cdot\sqrt{\frac{F_y}{E}}$$

16. The critical buckling load is then calculated in cell B68 following:

for $\lambda_c < 1.5$ $$F_{cr} = \left(0.658^{\lambda_c^{\,2}}\right)\cdot F_y$$

for $\lambda_c > 1.5$ $$F_{cr} = \left(\frac{0.872}{\lambda_c^{\,2}}\right)F_y$$

17. Cell B69 computes the axial load capacity of the column

$$\Phi P_n = \Phi\cdot F_{cr}\cdot A_s$$

18. Cell B72 computes the critical applied load to the load capacity of the column and cell B73 states the column's adequacy based on whether

C.5 Reinforced Concrete Slab Design

C.5.1 User Guide

Step 1: Use IBC to determine the appropriate amount of live loading to apply to your floor. Enter this value into the cell directly to the right of the "Live Load?" cell. The value entered should be in units of pounds per square foot.

Step 2: Make an assumption as to the width of the interior beams, exterior beams and girders for your design. This will impact the length of the slab so use a good estimation of what you hope your beam width will be.

Step 3: Determine the spacing between columns.

A) Calculate the length in inches between the center of your first interior column and the outside edge of your exterior column. Then subtract the thickness of an exterior beam (in inches) and half of an interior beam to get the length of the slab. This will be the length of your end bay slab.

B) Calculate the length in inches between the center of your first interior column and your second interior column. Subtract the width of an interior beam from this length. This will be the length of your first interior slab.

C) If the column spacing differs after the first interior column continue this process until the effective lengths of each bay are determined.

Step 4: Enter in Interior and Exterior bay widths. Enter these values in the appropriate cells. Make certain to enter these values in feet.

A) In order to calculate these bay widths for the first interior bay, divide the length of the end bay slab determined in Step 3 by 12 to convert it to feet.

B) For the first interior support, the effective length of the slab will be equal to the exterior bay length plus the first interior bay length divided by two and then that value divided by 12 to convert it to feet.

C) Repeat step 4B until the column spacing is equal between spans.

Step 5: Enter in the material properties of the concrete being used in the slab. Enter in the values in units of pounds per square inch.

Given Information

Live Load?	100	psf
Interior Bay Length?	14.83	LF
End Bay Length?	11.46	LF
Fy?	60000	psi
F'c?	3000	psi

Appendix Figure C.5.1 Given Information

Step 6: Review the Maximum Thicknesses for the slab as provided by the spreadsheet. Determine a value that is either equal to the maximum thickness or a value you would like to test to determine if it is applicable for the span and loading you have chosen.

Step 7: Select an effective depth for the slab. Convention is such that most reinforced concrete slabs are designed with ¾ inch clear cover and #4 reinforcing steel bars. Under these conditions, the typical effective depth is one inch less than the thickness. D = H – 1. If you are designing for a slab with different reinforcing steel or a different clear cover, then the formula used to calculate the effective depth of the slab is as follows:

$$D = H - (\text{clear cover} + .5(\text{diameter of reinforcing steel}))$$

Estimate Thickness of Floor

Required Max Thickness		
End Bay: H=	5.73	in
Interior Bay: H=	6.36	in
Trial Thickness		
Trial Slab Thickness?	6.5	in
D (3/4" cover w/ #4 bars)	5.5	in

Appendix Figure C.5.2 Slab Properties

Step 8: Determine dead loads that will be applied to your slab. Note: these dead loads do not include the self- weight of the concrete. The spreadsheet is designed to calculate that once the trial thickness and depth have been determined.

Step 9: Check to see if the slab is acceptable for moment. The method to determining if the slab thickness and depth are acceptable lies in checking the cell titled "Is the thickness adequate". If thickness is adequate the adjoining cell should read "ADEQUATE". If this is not the case the adjoining cell will read "NOT ADEQUATE". If the design is not adequate for the moment design then the thickness of the slab must be changed. This also means that the effective depth will change.

Is the thickness adequate	**ADEQUATE**

Appendix Figure C.5.3 Moment/Shear Analysis Readout

Step 10: If the slab is capable of withstanding the moment that it will experience then the next check is to determine if it will withstand the shear forces that it will be exposed to. In order to determine this, check the cell labeled "Is the thickness adequate". If it is acceptable then the adjoining cell will read "ADEQUATE". If the slab cannot withstand the shear forces put upon it, then the adjoining cell will read "NOT ADEQUATE".

Step 11: When both the shear and moment requirements are satisfied then the reinforcing steel needs to be designed.

A) The first major step will be done by inputting the effective lengths of the different bays of the structure into the section marked Lenghts
B) Next regard the slab summary table. In this table, look for the maximum design moment. It will be located in the row labeled Mu. (Note the yellow box)

	Slab Summary					
ln	8.08	8.08	11.46	14.83	14.83	14.83
Wu*ln^2	20.35	20.35	40.94	68.56	68.56	68.56
M coeff	0.04	0.07	0.10	0.06	0.09	0.06
Mu	0.85	1.45	4.09	4.29	6.23	4.29
As req.	0.036	0.062	0.174	0.182	0.265	0.182
As min	0.140	0.140	0.140	0.140	0.140	0.140
rebar ?	#4 @15"	#4 @15"	#4 @ 12"	#4 @ 12"	#4 @ 9"	#4 @ 12"
area provided ?	0.16	0.16	0.2	0.2	0.267	0.2

Appendix Figure C.5.4 Slab Summary Example

C) Once the maximum design moment has been determined, input that value into the "Mu max (from Slab Summary)" cell. By doing this you will allow the spreadsheet to calculate an area of steel coefficient so that the required area of steel for a particular section of the slab can be determined. These results will automatically be calculated in the Slab Summary table

D) Next you will need to determine an adequate bar spacing to meet the area of steel needs at various points in your beam. The table provided in the spreadsheet is to be used as a reference. The values of both the "rebar?" cells and the "area provided?" cells must be put in the table manually.

Step 12: Once this is done the slab design is complete. Should you wish to attempt a different thickness, simply begin this process over starting at Step 6.

C.6 Reinforced Concrete Beam Design

C.6.1 User Guide

***Note: Screen shots will be provided to help illustrate the overall procedure. This spreadsheet is meant to be used in conjunction with the* Step by Step Procedure for Using Reinforced Slab Design Spreadsheet for a Reinforced Frame System.**

Step 1: Identify the size of the slab to be placed on the beam, the length of the beam and the amount of tributary area that the beam is responsible for. Also identify the yield strength of the reinforcing steel bars and the compressive strength of the concrete. Insert these values into the spreadsheet in the appropriate cells provided. These cells are shown below and are highlighted in red.

			Beam Design			
	10x16 Column Spacing				*Interior 25' Beam*	
			Dimensions			
Slab Thickness	0.333	ft		fy	60000	psi
Beam Length	25	ft		f'c	3000	psi
Tributary length	8.83	ft				

Step 2: The next step involves identifying the dead and live loads acting upon the structure. This determination should reference all applicable codes such as the *IBC* and state building codes.

			Loading	
Dead Loading	20	psf	169.2	lb/ft
Slab Weight			423	lb/ft
	Exterior Considerations			
	Exterior Wall Thickness		0	in
weight	0	lb/ft		
Total DL	592.2			
live loading	100	psf		
	846	lb/ft		

In regards to the dead loading, the spreadsheet calculates the contributing weight of the slab and combines it with the provisions made for non-structural dead loading and exterior wall considerations. The live loading is determined in terms of the conditions set for by the governing code.

Step 3: With these values determined, the spreadsheet will now generate values for an estimated weight of the reinforced concrete beam itself. These values are based on equations from the ACI. A value should be chosen in the range of the four provided in the green boxes.

Estimate Beam Dead Load

weight 1	143.82	lb/ft	***or***	287.64	lb/ft
weight 2	h1	2	ft	b1	1
	h2	2.5	ft	b2	1.25
	wt. 1	300			
	wt. 2	468.75			

Select trial self weight between the 4 options

self weight	325	lb/ft

Step 4: Once a value has been entered into the self weight cell depicted above, the spreadsheet generates values for the distributed load acting along the beam and the moments generated by that load. These values are then used to determine bases, depths and heights for the beam. The base size can be varied to accommodate different sizes with the depths and heights adjusting automatically.

Compute Factored Mu

wu	2.72	k/ft
Mu	213	k-ft

Compute B and D

ρ	0.01	
ω	0.2	
φKn	476.28	
bd^2	5358	in^3

Options	**b**	**d**	**h**
	12	21.13	24.63
	18	17.25	20.75
	22	15.61	19.11
	24	14.94	17.44
Select B,D & H	24	14	18

It is also important at this stage to select a trial base, depth and height for the column. The next steps will determine if this size is adequate to resist the forces acting through the member.

Step 5: The spreadsheet automatically determines whether the self weight chosen for the beam in step three is large enough to account for the actual self weight. Note the figure below:

Check Dead Load and Revise Mu

self weight	450.000	lb/ft		
wu	2.897	k/ft		
Mu	226.350			
Acceptable ?	**NO GOOD**	***if No Good check here***	**OKAY**	
		if **OKAY** carry on w/design		

Occasionally it occurs that the self weight of the beam creates much larger moment forces than the estimated self weight of the beam. When this happens, another check can be done to determine if a new estimated weight should be selected or not. Should the actual moment be less than ten percent greater than the moment generated by the estimated weight value, the beam design is still acceptable. Were this not the case, a new base, depth, and height would need to be selected or the estimated weight of the beam adjusted to more accurately reflect the actual weight of the beam.

Step 6: Once the size of the beam has been determined, the amount of reinforcement needs to be calculated. The spreadsheet is designed such that the area of steel required by the forces acting on the beam and the minimum area of steel requirements are displayed and the greatest value noted.

Compute Reinforcement

jd	12.25		
As	4.106		
As min	0.920	***or***	1.120
As min	1.120		
As req or As min?		4.106	

Step 7: With the amount of steel required for the beam now known, the appropriate size and number of bars need to be selected. Following the output of the amount of reinforcing bars needed is a chart with different cross-sectional properties listed.

Steel Options					
Bar	Number	As			
single type					
#6	9	3.96	in^2		
# 7	7	4.20	in^2		
combinations			total		
#6	8	3.52	4.14	in^2	
#5	2	0.62			
			0	in^2	
Select As	7 #7 Bars	4.2	in^2		
	One Row Of Rebar				

Bar Properties		
Bar #	diameter	in^2
# 3	0.375	0.11
# 4	0.500	0.20
# 5	0.625	0.31
# 6	0.750	0.44
# 7	0.875	0.60
# 8	1.000	0.79
# 9	1.128	1.00
# 10	1.270	1.27
# 11	1.410	1.56
# 14	1.693	2.25
# 18	2.257	4.00

These properties listed in the chart can be used to generate different combinations for reinforcement. The amount of reinforcing provided should be more than that called for by the "As req or As min?" readout.

Step 8: The final phase of design that the spreadsheet accommodates is the evaluation of the actual depth of the beam. This value is dictated by the diameter of the reinforcing bars selected and the number of rows utilized in design.

Recompute D

ϕ of rebar row 1	0.875	in
ϕ of rebar row 2	0.000	
D	15.7	in

Compute a, check if fs = fy and determine if tension controlled

a	4.12	in^2
a/d	0.262	
Ab/d	0.503	
Atcl/Dt	0.319	
Dt	15.7	
a/Dt	0.262	

tension controlled?	YES
	phi = .90
fs = fy ?	YES

Compute Mn and ΦMn

Mn	286	k-ft
ΦMn	258	k-ft

Design Acceptable?	**Yes**

After the depth has been recomputed, the value of "a" is recomputed to ensure that design does not need to be redone. Additionally, the moment capacity of the beam is determined and compared with the value of the moment acting along the beam. If this value is lower than capacity the design is acceptable, if not the area of steel will need to be recomputed or the size of the beam will need to be adjusted.

C.7 Reinforced Concrete T-Beam Design

C.7.1 User Guide

***Note: Screen shots will be provided to help illustrate the overall procedure. This spreadsheet is meant to be used in conjunction with the* Step by Step Procedure for Using Reinforced Slab Design Spreadsheet for a Reinforced Frame System.**

Step 1: The dimensional properties of the bay that one wishes to design a T-Beam for are the first numbers that need to be inserted in the spreadsheet. Use the *IBC* or other appropriate building code to determine the dead and live loads applicable to the structure. In addition, insert the slab thickness, bay dimensions and desired material properties where applicable.

T-Beam Design

Dimensions and Factored Loadings

live load	*100*	*psf*	*ext. length*	*20*	*ft*	*fy*	*60000*
dead load	*20*	*psf*	*int length*	*25*	*ft*	*f'c*	*3000*
slab thickness	*4*	*inches*	*ext. width*	*0*	*ft*		
			int width	*10*	*ft*		

Step 2: The next phase of this spreadsheet is driven by the bay sizes over which the T-Beam will span. Influence Area A corresponds to the first exterior bay, Influence Area B to the exterior bay and the first interior bay and Influence Area C to the first interior bay. Dimensional properties for these areas are also affected by the assumed size of the base of the beam. It is often assumed that the base of the beam is 12 inches, which is the size selected in this particular example. It should also be noted that the resulting loading from Influence Area B governs due to that being the condition where the maximum negative moment occurs. The figure on the following page provides a visual depiction of the input parameters for the influence areas. Note that the spreadsheet determines the loading per square foot for the user.

Influence Area A

length	19	ft	L	134	psf
ext. width	0	ft			
int. width	9	ft			
beam width	1	ft			
Aa	190	ft^2			

Influence Area B

Length 1	19	ft	L	97	psf
Length 2	24	ft			
ext. width	0	ft			
int. width	9	ft			
beam width	1	ft			
Ab	440	ft^2			

Influence Area C

length	24	ft	L	122	psf
ext. width	0	ft			
int. width	9	ft			
beam width	1	ft			
Ac	240	ft^2			

Stem size will be chosen based on influene area B

Step 3: In the next portion of the spreadsheet, various loading combinations are determined with the result being estimates for the weight of the T-Beam. The user is required to select a value within the range of the four estimates given by the spreadsheet to serve as the experimental dead weight of the T-Beam. In order for the spreadsheet to function correctly this value must be entered into the cell adjacent to the “W beam” cell.

Loadings

slab DL	50	psf		
DL	20	psf		
Reduced LL	97	psf		
factored loading	262	psf		
tributary width	5.5	ft		
factored loading per foot of slab			1.44	k/ft

Estimate weight of Beam Stem

W1	0.144	k/ft				
W2	0.288	k/ft				
	Using B, H and PCF					
H1	2	B1	1	W3	0.300	k/ft
H2	2.5	B2	1.25	W4	0.469	k/ft

Select a trial weight in the range given by W1 - W4

Wbeam	0.400	k/ft	trial loading	1.84	k/ft

Step 4: Keeping the minimum height requirements in mind, the user can now select trial base, depth and height dimensions for the T-Beam. The minimum height requirement can be found adjacent to the cell with “Minimum Height” written in

it. The spreadsheet is designed such that entering different base values will result in the automatic calculation of corresponding depths and heights.

Compute Actual Size of Beam

Minimum height	16.22 in			
based on Mu 1st int. support			93.22	k-ft
ρ	0.013			
ω	0.26			
ϕKn	594.3132			
bd^2	1882.222415 in^3			
options	b	d	h	
	10.00	13.72	16.22	
	12.00	12.52	15.02	
	14.00	11.60	14.10	
try				
b	10.00	in		
d	15.50	in		
h	18.00	in		

Step 5: The next step involves accounting for the shear forces acting on the beam. This is done by selecting the appropriate dimensions for the base, depth and height. The spreadsheet will tabulate the acceptable values of shear and compare those to the shear resistance provided by the member. If the cell adjacent to the one with the text "if d > d1 the OK" reads "GOOD" then dimensions are adequate. If it reads "NO GOOD" then new dimensions must be selected. This step also provides a trial summary of the beam dimensions and the distributed load acting along the beam.

Check Shear Capacity

Max Vu		26.47	k		
min Bwd		56.85	in^2		
for b =		10.00	in	d1 =	5.69
if d > d1 the OK	GOOD				

TRIAL SUMMARY

b	10.00	in
d	15.50	in
h	18.00	in
wu	1.84	k/ft

Step 6: The next aspect this spreadsheet addresses is the determination of the flange width. This property is a function of base size, material properties and size of the member. The minimum flange width should be selected.

Calculate Flange Width at + Moment Regions

choose smallest	60	in
	106	in
	70	in
flange width	60	in

Step 7: With the dimensional values of the beam determined, the moment forces acting through the member can be computed. This is done automatically by the spreadsheet as can be seen in the screenshot below.

Compute Moments

L1	20	LL1	1.14	k/ft
L2	20	LL2	0.82	k/ft
L3	22.5	LL3	1.04	k/ft
L4	25	trib area1	5	ft
L5	22.5	DL	1.24	k/ft
L6	20			
L7	20			

Moment Calculation Table

ln	20	20	22.5	25	22.5	20	20
wu	2.38	2.38	2.06	2.28	2.06	2.38	2.38
wu*ln^2	952	952	1044	1424	1044	952	952
Cm	0.04	0.07	0.10	0.06	0.10	0.07	0.04
Mu	-39.67	68.00	-104.43	88.98	-104.43	68.00	-39.67

Step 8: With the moments at various points in the member determined, the area of steel to resist the maximum negative and positive moment must be computed. A chart is provided so that the appropriate reinforcing steel bars can be selected to meet the requirements for the area of steel computed by the spreadsheet. The maximum negative and positive moment need to be inserted in the appropriate cells in order to calculate the area of steel required. The "Calculated As and Select Bars" Table summarizes forces and dictates the requirements for the area of steel needed.

Design Reinforcement

Point of Max - Moment

Neg. Max Moment	104.43	kft
As	1.71	in^2
a	4.03	in
a/d = a/dt	0.260	
ab/d	0.503	fs=fy
atcl/dt	0.319	phi = .90
recompute As	0.0165	MU

Point of Max + Moment

Pos. Max Moment	88.98	kft
As	1.34	in^2
a	0.527	
a/d = a/dt	0.034	
ab/d	0.503	fs=fy
atcl/dt	0.319	phi = .90
recompute As	0.0146	MU

Compute As Min

Asmin	0.52
As>Asreq?	YES

Bar Properties		
Bar #	diameter	in^2
# 3	0.375	0.11
# 4	0.500	0.20
# 5	0.625	0.31
# 6	0.750	0.44
# 7	0.875	0.60
# 8	1.000	0.79
# 9	1.128	1.00
# 10	1.270	1.27
# 11	1.410	1.56
# 14	1.693	2.25
# 18	2.257	4.00

Calculate As and Select Bars

	As Selection													
Mu		-39.67		68.00		-104.43		88.98		-104.43		68.00		-39.67
As Coeff.		0.0165		0.0146		0.0165		0.0146		0.0165		0.0146		0.0165
As required		0.65		0.99		1.72		1.30		1.72		0.99		0.65
As > As min?	YES		YES		YES		YES		YES		YES		YES	
bars	2 #6		1 #6/1 #7		3 #7		3 #6		3 #7		1 #6/1 #7		2 #6	
As provided		0.88		1.04		1.8		1.32		1.8		1.04		0.88
bw Okay?			YES				Yes				YES			

Step 9: The next steps involve the design of ties and the spacing related to them. The smallest areas of steel provided for both the positive and negative moment regions are analyzed to determine the tie spacing.

Reinforcement Distribution

Positive Moment Region

smallest bar #	1 #6/1 #7	# of bars	2
As ?	1.04	in^2	
diameter?	0.875		
dc	2.3125	in	
A	23.125	in^2/bar	
z	136		
Okay?	OKAY		

Negative Moment Distribution

distributive length	24	in	
diameter of bars	0.875		
type of bars	#7	# of bars	3
dc	2.31	in	
Max A	10.22		
s	2.21	in	

Step 10: With these values computed, the shear through the member and the shear resistance provided by the member are compared and tie spacing is computed for

each section of the T-Beam. This spacing is dictated by the amount of shear in the T-Beam, the length of the bay, and the sectional properties of the ties.

Shear Reinforcement

ln		20			25	
wu		2.38			2.28	
wl		1.14			1.04	
Cv	1.00	0.15	1.15	1.00	0.15	1.00
wuln/2		23.80			28.48	
Vu	23.80	3.57	27.37	28.48	4.27	28.48
Vn	28.00	4.20	32.20	33.50	5.03	33.50

Exterior of B3

d		15.5 in	from support	Vc	16.98	kips
Vn		24.93		Vc/2	8.49	kips
stirrups?	YES					
Max Spacing		17.60 or	7.75			
Max Spacing		7.75				
try		6 in				
# 0f stirrups		20				

Interior end of B3

d		15.50 in			
Vn		28.58			
s		6.79 in o/c	when s =		
try		6 in o/c	Vn	#DIV/0!	kips
use			x	#DIV/0!	in
# of stirrups	Spacing				
20	6	in o/c from middle		120	in from middle
		in o/c from end		0	in from end

End Of B4

d		15.50 in			
Vn		30.56 k			
s		6.18 in	when s =	0	
try		6 in o/c	Vn	#DIV/0!	
# of stirrups	Spacing		x	#DIV/0!	
25	6	in o/c from middle		150	in from middle
0	4	in o/c from end		0	in from end

Final Summary

b		10.00	in
d		15.50	in
h		18.00	in
rebar		length	
Exterior neg bar	2 #6	6.67	ft
1st Int Pos. bar	1 #6/1 #7	20.00	ft
1st Int Neg. bar	3 #7	8.33	ft
2nd Int Pos. bar	3 #6	25.00	ft

See Directly Above for Stirrup Spacing

The final portion of the spreadsheet provides an summary for the entire design of the T-Beam.

C.8 Reinforced Concrete Girder Design

C.8.1 User Guide

The procedures and equations used throughout this girder design spreadsheet are consistent with the 1995 ACI Code for reinforced concrete design. Note: all cells highlighted in red are values that must be manually input.

1. Input the length of the first two interior bay lengths into cells D4 and D5. (Refer to Figure 1)

2. Enter the tributary widths in both the long and short directions in cells D6 though D9. (Refer to Figure 1)

3. Input the slab thickness into cell D10 and concrete properties in cells G4 through G6. (Refer to Figure 1)

4. The factored loadings on the girder must be manually calculated and input into cells G7 through G9. (Refer to Figure 1)

General Information					
1st interor bay length	10	ft	concrete wt.	150	pcf
2nd interior bay length	16	ft	fy	60000	psi
Exterior tributary length (long)	10	ft	f'c	3000	psi
Interior tributary length (long)	0	ft	dead load	20	psf
Exterior tributary length (short)	5	ft	live load	100	psf
Interior tributary length (short)	8	ft	snow load	0	psf
slab thickness	4	in			

Figure 1 General Input Data

5. Next, the dimensions of the beams must be input into the spreadsheet. Cells B13 through B18 are reserved for the respective dimensions of the base and cells C13 through C18 for the height dimensions of each beam type. (Refer to Figure 2)

Beam dimensions			
type	b	h	
B1	12	18	in
B2	12	18	in
B3	12	18	in
B4	10	16	in
B5	10	16	in
B6	10	16	in

Figure 2 Dimensions of Beams

6. Rows 19 through 34 automatically calculate the point loads that are applied to the girder. A summary of the combined loads are featured in cells B36 through B38.

7. The maximum of the combined loads summarized in cells B36 and B38 must be entered into cell H36.

8. Next, the self weight of the girder must be calculated. The only information that needs to be entered for this procedure is the length of the beam, which can be input into cell B44.

9. Cells B50 through B53 will automatically update. These cells summarize the final point loads that will be used to design the girder. Figure 3 which is featured in the design spreadsheet illustrates the abbreviations for the applied loads and the placement to which they are applied.

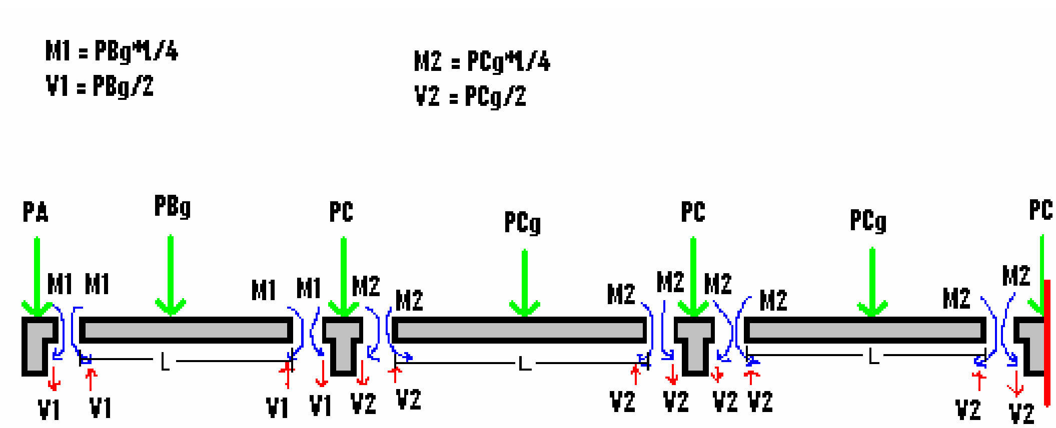


Figure 3 Illustration of Applied and Internal Loading

10. Next the maximum moment featured in cells B69 and G69 must be entered into cell D72.

11. Cell B74 calculates the minimum dimensions of the girder. The dimensions are given as b*d^2. Since the base dimension of reinforced concrete beams and girders are typically within the range of 12-18 inches the depth of the beam can be calculated. Additionally, the height of the girder is a function of the depth of the girder plus the clear cover for the reinforcing bars. Clear cover for one layer of steel rebar is typically 2.5 inches while clear cover for two layers of steel rebar is typically 3.5 inches. Refer to Figure 4 for a sample of how the dimensions of the girder are automatically calculated.

Size options	b	d	h	
	12.00	10.20	13.70	in
	14.00	9.44	12.94	in
	16.00	8.83	12.33	in
	18.00	8.32	10.82	in

Figure 4 Girder Size Options

12. Each of the sizes calculated in the previous step will adequately support the load, however sizes must be manually chosen and entered into cells B82 and B83. The dimensions chosen are influenced by the dimensions of the columns and beams. The girder should be at least as large as the beam and the base should be adequate for attachment to the columns.

13. Next the area of steel needed to support the tensile forces in the girder must be calculated. The required areas of steel for negative moments for the first and second interior spans are calculated in cells B88 and F88. Additionally, the areas of steel to resist the positive bending moments in the first and second interior spans are calculated in cells B95 and F95.

14. One of the tabs within the workbook features a table that summarizes the areas provided by different bars. This table can be seen in Figure 5. Bars must be chosen and their resulting area must be greater than the required area of steel as defined in the previous step. These resulting areas must be entered into cells B94, F94, B101 and F101 respectively.

Bar Properties		
Bar #	diameter	in^2
# 3	0.375	0.11
# 4	0.500	0.20
# 5	0.625	0.31
# 6	0.750	0.44
# 7	0.875	0.60
# 8	1.000	0.79
# 9	1.128	1.00
# 10	1.270	1.27
# 11	1.410	1.56
# 14	1.693	2.25
# 18	2.257	4.00

Figure 5 Summary of Bar Areas

15. Finally, the lengths of the reinforcing bars are automatically calculated. For the interior support the length of the negative reinforcing bars is displayed in cell C103 while the positive reinforcing bars are displayed in cell C104 for the first interior support. Similarly, the lengths of positive and negative reinforcing steel for the second interior bays are respectively displayed in cells G103 and G104

C.9 Reinforced Concrete Column Design

C.9.1 User Guide

The procedures and equations used throughout this girder design spreadsheet are consistent with the 1995 ACI Code for reinforced concrete design. Note: all cells highlighted in red are values that must be manually input.

1. Enter the concrete strength properties into cells B13 and B14. Additionally, define the column height in cell B15, enter the applied axial load and moment in cells E13 and E14, and enter the width of the girder in cell E15.
   Note: For this application a summary of the loads have been incorporated into the workbook. These loads were calculated using RISA2D, a two dimensional structural frame analysis program. The summary is not necessary; however it helps organize the loads applied to the columns.

2. The next step is to calculate the gross area of the column. This gross area is automatically computed and displayed in cell B17. Cell E17 also displays the minimum dimensions of a square column.

3. Next, the dimensions of the column must be entered into cells C20 and C21. These dimensions must meet the minimum gross area requirement that was previously calculated. Additionally, the column cannot be smaller than the width of the girder. Refer to cell E15 to verify that the minimum dimensions of the column are at least equal to the width of the girder.

4. The eccentricity of the column is automatically calculated in rows 24 and 25. As noted in the spreadsheet the eccentricity over the height of the column determines how much reinforcing is needed in the column. Figure 1 shows the equalities that determine the amounts of reinforcing.

for 0.2>e/h>0.1 use reinforcing in all 4 sides, for e/h>0.2 use reinforcing in two sides. For e/h <0.1 use cylindrical column.

Figure 1 Requirements for Reinforcing Bars

5. Based upon the results from step 4, determine the reinforcing needed in the column and make a note in cell C27 that describes the layout of the bars to be used.

6. Next, the slenderness of the column is automatically calculated in checked in cell B31. If cell B31 displays “Okay” then the trial size of the column is ok. However, if cell B31 displays “Bad” then revisions to the trial column size need to be made.

7. Now that the trial size of the column has been calculated cells B33 through B37 are reserved to summarize the column properties. Figure 2 shows an example of the column properties summary.

Trial Design Summary			
b	12	in	
h	12	in	
fy	60000	psi	
f'c	3000	psi	
rebar	2 faces		

Figure 2 Trial Design Summary

8. Next, gamma needs to be calculated. In order to calculate gamma, enter trial rebar sizes into cells B39 and B41. Gamma will automatically be calculated once these values are entered.

9. Refer to tables A-6 and A-7 of MacGregor’s text, *Reinforced Concrete: Mechanics and Design* to determine rho. Rho is the tension controlled limit for the neutral axis depth and is a function of the area of steel, depth, and base length of the slab.

10. The actual required area of steel can now be calculated. Cell B53 displays the actual required area of steel. Choose rebar that will satisfy this required area of steel and input it into cell C57.

11. The spreadsheet will automatically check to see if the capacity of the beam is adequate. If cell B60 displays “Yes” then the spacing of the ties can be designed. If cell B60 displays “No” then the column must be redesigned.

12. Once the column is acceptable the spreadsheet will automatically suggest lengths for the lap splices. Choose an adequate length based of the suggestions given in cells B62 and B63 and input the trial length in cell B64.

13. Once the length of the lap splices has been determined the spacing can be calculated. The spreadsheet automatically calculates spacing using three different methods. Choose the smallest of the options given in cells B66 through B68 and enter it into cell E68.

14. Finally, the spreadsheet will summarize where ties need to be placed within the column. (See figure 3 for an example of the summary of the ties)

ties at	6	in	for 1st	3	ft
ties at	12	in	for middle	6	ft
ties at	6	in	for last	3	ft

Figure 3 Summary of Ties

D Sample Calculations

D.1 Composite Beam – Wide Flange – Sample Calculation

SAMPLE Calculation 1/4

Composite Beam – Wide Flange

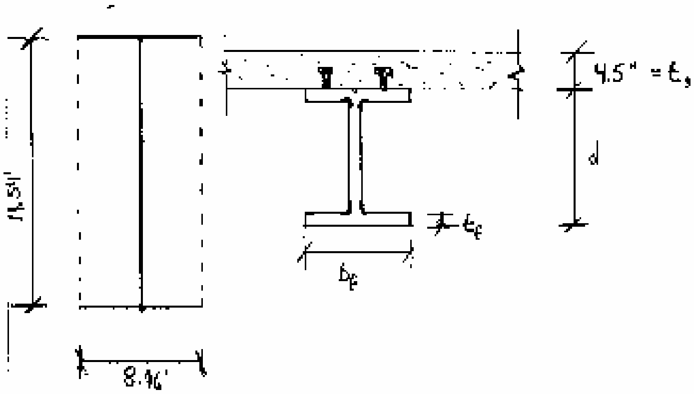
Live Load = 40 psf $\Rightarrow$ $LL = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 40psf\left(0.25 + \frac{15}{\sqrt{2(175)}}\right) \le 40\,psf$

Dead Load =

Concrete = $4.5''\left(\frac{1'}{12''}\right)(150\,pcf)(1.1) = 61.875\,psf$

Mechanical = 10 psf

$DL = 10\,psf + 61.88\,psf = 71.88\,psf$



Length = 19.54' $F_y = 50\,ksi$ $f'_c = 3000\,psi$

Tributary width = 8.96' $E = 29000\,ksi$ $F_u = 65\,ksi$

$t_s = 4.5''$

Load Combinations

1.4DL or 1.2DL + 1.6LL

$1.4\,DL = 1.4(71.88\,psf)(8.96') = 901.67\,plf$

$1.2\,DL + 1.6\,LL = \left[1.2(71.88\,psf) + 1.6(40\,psf)\right]8.96' = \boxed{1346.3\,plf}$

Moment Calculation

$$M_u = \frac{wL^2}{8} = \frac{(1346.3\,plf)(19.54')^2}{8} = 64253.8\,ft\text{-}lb = 64.26\,ft\text{-}k$$

Minimum Z

$$Z = \frac{M_u}{\phi F_y} = \frac{(64.26\,ft\text{-}k)(12''/')}{0.9(50\,ksi)} = 17.14\,in^3$$

SAMPLE Calculation

Composite Beam - Wide Flange Continued

Select Trial Member

W12x16

$Z = 20.1 \text{ in}^3 \quad A_s = 4.71 \text{ in}^2 \quad w_{beam} = 16 \text{ plf}$

Recalculate $M_u = \dfrac{(1346.3 \text{ plf} + 16 \text{ plf})(19.54')^2}{8} = 65.02 \text{ ft-k}$

recalculate $Z_{min} = \dfrac{(65.02 \text{ ft-k})(12 \text{ in/ft})}{0.9(50 \text{ ksi})} = 17.34 \text{ in}^3$ ok

Local Buckling Check

Local Flange Buckling: $\dfrac{b_f}{2t_f} \leq 0.38\sqrt{\dfrac{E}{F_y}} \Rightarrow 7.50 \leq \left[0.38\sqrt{\dfrac{29000}{50}} = 9.15\right]$ ok

Web Local Buckling: $\dfrac{h}{t_w} \leq 3.76\sqrt{\dfrac{E}{F_y}} \Rightarrow 49.4 \leq \left[3.76\sqrt{\dfrac{29000}{50}} = 90.55\right]$ ok

Effective Flange Width (b_e)

lesser of $\dfrac{L}{4}$ or tributary width $\Rightarrow \dfrac{19.54'}{4} = 58.62''$ or $8.96' = 107.52''$

$b_e = 58.62''$

Determine $\phi_b M_n$

$d = 11.99 \text{ in} \quad t_w = 0.220 \text{ in} \quad h/t_w = 49.4 \quad h = 10.87 \quad \phi_b = 0.85$

Locate Plastic Neutral Axis

$a = \dfrac{A_s F_y}{0.85 f'_c b_e} = \dfrac{4.71 \text{ in}^2 (50 \text{ ksi})}{0.85(3.0 \text{ ksi})(58.62'')} = 1.58''$

Plastic Neutral Axis is in the slab since $a < t_s$

SAMPLE Calculation

Composite Beam - Wide Flange Continued

Design of Shear Studs

Diameter = 0.75 in

Shear = $0.85\, f'_c t_s b_e < A_s F_y$

V_n $0.85(3ksi)(4.5in)(58.6in) = 672.6$ kips or $4.71in^2(50ksi) = \boxed{235.5\,k}$

$Q_n = 21.58k = 0.85(\pi)(d^2/4)(f'_c E_c)$

$\#\,studs = \dfrac{V_n}{Q_n} = \dfrac{235.5^k}{21.58^k} = 10.91 \Rightarrow$ 11 studs per midspan

Spacing $= \dfrac{\left(\frac{L}{2}\right)}{\#\,studs} = \dfrac{(19.54/2)(12'')}{11\,studs} = 10.66$ in/stud spacing

Minimum Stud spacing = 6(Diameter) = $6(3/4'') = 4.5$ in

Maximum spacing = $8t_s = 8(4.5in) = 36$ in

$4.5in < 10.66in < 36in$ ok

Composite Section Deflection

$A_s = 4.71\, in^2$ $Y_s = d/2 = 11.99/2 = 6$ in

$A_c = \dfrac{0.85\, f'_c\, a\, b_e}{F_y} = \dfrac{0.85(3ksi)(1.57in)(58.6in)}{50\,ksi} = 4.72\, in^2$

$Y_c = d + t_s - \dfrac{a}{2} = 11.99in + 4.5in - \dfrac{1.58in}{2} = 15.7$ in

$Y = \dfrac{(A_s Y_s + A_c Y_c)}{(A_s + A_c)} = \dfrac{[(4.71in^2)(6in) + (4.72)(15.7in)]}{(4.71in^2 + 4.72in^2)} = 10.85$ in

Moment of Inertia

$I_s = 103\, in^4$

$I = I_s + A_s(Y - Y_s)^2 + A_c(Y_c - Y)^2 = 103in^4 + 4.71in^2(10.85 - 6)^2 + 4.72in^2(15.7 - 10.85)^2$

$I = 324.82\, in^4$

Composite Beam - Wide Flange

Deflection Calculation

$$\Delta = \frac{5wL^4}{384EI} = \frac{5(1018.5\ plf)(19.54')^4(12''/')^3}{384(29000\ ksi)(304.82\ in^4)} = 0.355\ in < L/360 = 0.651\ in$$

Unshored Deflection Check

$$W_u = 1.2\ DL + 1.6\ LL\ (\text{concrete}) = 1.2(16\ plf) + 1.6(1.1(150\ pcf)(\tfrac{4.5''}{12''}) + 20\ psf)(8.96')$$

$$W_u = 1193.96\ plf$$

$$M_u = \frac{wL^2}{8} = \frac{1193.96\ plf\ (19.54')^2}{8} = 56.94\ ft\text{-}k$$

$$Z_{req} = \frac{M_u}{\phi F_y} = \frac{56.94\ ft\text{-}k\ (12''/')}{0.85\ (50\ ksi)} = 16.08\ in^3 < 20.1\ in^3 \quad \underline{ok}$$

Serviceability Concerns

$$\Delta_{max} = 0.651\ in$$

$$\Delta_{construction} = \frac{5w_{constr}L^4}{384EI_s} = \frac{5(748.46\ plf)(19.54')^4(12''/')^3}{384(29000\ ksi)(104\ in^4)} = 0.815\ in$$

$$75\%\ \Delta_{construction} \leq \Delta_{max}$$

$$0.75\ \Delta_{constr} = 0.75(0.815\ in) = 0.612\ in < 0.651\ in\ \Delta_{max}$$

$$\boxed{\text{Beam is Adequate}}$$

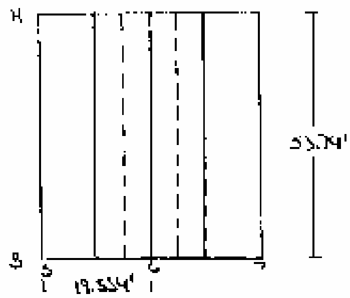
W12x16 A572-50 ksi steel with 22 - 3/4" Shear studs spaced 10.66 in in Pairs

Slab thickness 4.5 in of 3000 psi Concrete

D.2 Composite Beam – Open Web Joist – Sample Calculation

Open Web Joist – Vulcraft Design Procedure

SAMPLE CALCULATION – Prison Cell Block Diagram Joist



Joist Size

- L = 53.74 ft
- Trib. width = 9.78 ft
- Depth ≤ 48 in

Composite LL

Design LL = 40psf

Reduction = $0.25 + 15/\sqrt{2(9.78 \cdot 53.78)}$

= 0.71

Furniture = 5psf

33.51psf

Composite DL

Mechanical = 15psf

Ceiling/Flooring = 3psf

18psf

Noncomposite DL

Joist ≈ 5psf

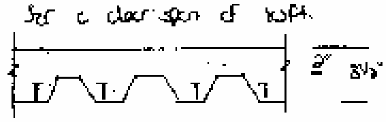
Bridging = 0.1psf

Concrete = 26psf

Decking = 1.78psf

32.88psf

} Use 1.5VL22 decking (1.78psf) w/ 3½" (t = 2") concrete (110pcf, f'c = 3ksi) → carry superimposed LL = 76psf for a clear span of 10ft.



Total Loads

33.51 + 18 + 32.88 = 84.39psf → 825.33 lb/ft

Maximum Allowable Deflection

$\Delta_{max} = \frac{L}{360} = \frac{(53.78\,ft)(12\,in/ft)}{360}$

= 1.74 in

Trial Size

$$I_{req'd} = \frac{1.15(5)(w_{noncomposite})(\text{Truss Length})^4(12^{in}/ft)^3}{384\,E\,\Delta_{max}}$$

$$= \frac{1.15(5)(32.88psf \cdot 9.78ft)(53.78ft - 4/12)^4(12^{in}/ft)^3}{384(29000ksi)(1.79in)(1000^{lb}/k)}$$

$I_{req'd} = 1304.6 in^4$

→ Try 36VC 900/34/18

$V_D = 23^{lb}/ft$

$w_{allow} = 875^{lb}/ft$

$d - d_2 = 36 - 5/8''$ (39/16" long)

900 = total load category (applied $w_{comp} = 825.33^{lb}/ft$)

34 = applied composite LL

18 = applied composite DL

Moment of Inertia – Joist

$\cdot I_{eff} = 0.0466\, V_n d^2$

$= 0.0488(23)(36^2)$

$I_{eff} = 1454.6 in^4 > I_{req'd}$

Deflection Checks

$$\cdot\Delta_{noncomp} = \frac{1.15(5)(w_{nc})(\text{Truss Span})^4(12^{in}/ft)^3}{384\,E\,I}$$

$$= \frac{1.15(5)(32.88psf \cdot 9.78ft)(53.78ft - 4/12)^4(12^{in}/ft)^3}{384(29000ksi)(1000^{lb}/k)(1454.6in^4)}$$

$\Delta_{noncomp} = 1.6 in < \Delta_{max}$

$$\cdot\Delta_{compDL} = \frac{w_{DL}}{w_{all}}\left[\frac{\text{Truss Span}}{360}\right]$$

$$= \frac{(18psf \cdot 9.78ft)}{875^{lb}/ft}\left[\frac{(53.78ft - 4/12)(12^{in}/ft)}{360}\right]$$

$\Delta_{compDL} = 0.36 in < \Delta_{max}$

$$\cdot\Delta_{compLL} = \frac{w_{LL}}{360}\left[\frac{\text{Truss Span}}{360}\right]$$

$$= \frac{(35.5psf \cdot 9.78ft)}{875^{lb}/ft}\left[\frac{(53.78ft - 4/12)(12^{in}/ft)}{360}\right]$$

$\Delta_{compLL} = 0.67 in < \Delta_{max}$

$\cdot\Delta_{DL} = \Delta_{noncomp} + 50\%\,\Delta_{compDL} + 20\%\,\Delta_{compLL}$

$= 1.6in + 0.5(0.36in) + 0.2(0.67in)$

$\Delta_{DL} = 1.92 in$

Camber = 2in

Construction Deflection Check

Construction Loads: Joist wt = 23#/ft = 23psf

Concrete (nom. thick + 1/2") = 32.25psf

Decking = 1.78psf

Bridging = 0.1psf

Construction (3 200lb workers) = 1.41psf

36.34psf → 333.79#/ft

$$\Delta_{construction} = \frac{1.15(5)(w)(\text{Joist span})^4(12''/ft)^3}{384EI}$$

$$= \frac{1.15(5)(333.79\#/ft)(53.75ft - 4in)^4(12''/ft)^3}{384(29000ksi)(1000\#/k)(1454in^4)}$$

$\Delta_{construction} = 1.78$ in $< \Delta_{max}$

For Joist BH6, use:

36VC 900/34 IB

$w_0 = 23$#/ft

$w_{dim} = 850$#/ft

w/ds = 53'-5/16" (3 5/16" long — extra 5/16" for runoff)

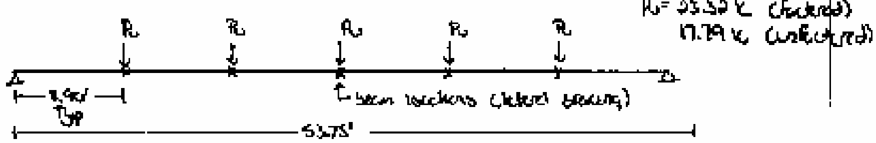
use 1.5VL22 decking w/ 3 1/2" concrete (t = 5") (110pcf, f_c = 3ksi)

D.3 Wide Flange Girder Design – Sample Calculation

Girder Design

Sample Calculation – Prison Cell Block
Option #1 – w/ intermediate columns

Girder Dwg.



- $\phi_b = 0.85$
- $F_y = 50$ ksi
- $E = 29000$ ksi
- $C_b = 1$

M_u

- $M_u = \dfrac{3PL}{4}$ for 5 point loads evenly spaced ($L/6$)

$= \dfrac{3(23.32k)(53.75ft)}{4}$

$M_u = 940.09$ ft-k

- $Z_{req'd} = \dfrac{\phi M_n = M_u}{\phi F_y}$

$= \dfrac{(940.09\text{ ft-k})(12 in/ft)}{0.85(50ksi)}$

$Z_{req'd} = 265.44 in^3$

Trial Size

W27×102	$A_g = 30.0 in^2$	$S_x = 267 in^3$
	$d = 27.09 in$	$r_y = 2.15 in$
	$Z = 305 in^3$	$X_1 = 1840$ ksi
	$I_x = 3620 in^4$	$X_2 = 0.0114 (1/ksi)^2$

Revised M_u and $Z_{req'd}$

- $M_u = \dfrac{3PL}{4} + \dfrac{wL^2}{8}$

$= \dfrac{3(23.32k)(53.75ft)}{4} + \dfrac{(102 lb/ft)(53.75ft)^2}{8 \cdot 1000 lb/k}$

$M_u = 976.93$ ft-k

$Z_{req'd} = \dfrac{\phi M_n = M_u}{\phi F_y}$

$= \dfrac{976.93\text{ ft-k}(12 in/ft)}{0.85(50ksi)}$

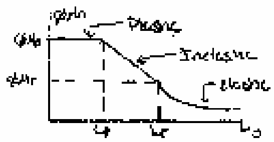
$Z_{req'd} = 275.84 in^3 < Z_{trial size} \rightarrow$ GOOD

Local Buckling

$\frac{b_f}{2t_f} = 6.0 < 9.15$ $\frac{h}{t_w} = 47.0 < 90.53$

∴ Section is compact

Lateral Torsional Buckling...



$L_p = \frac{1.76 r_y}{12 \text{ in/ft}} \sqrt{\frac{E}{F_y}}$

$= \frac{(1.76)(2.15 \text{ in})}{12 \text{ in/ft}} \sqrt{\frac{29000 \text{ ksi}}{50 \text{ ksi}}}$

$L_p = 7.59 \text{ ft}$

$L_r = \frac{r_y X_1}{(F_y - F_r)} \sqrt{1 + \sqrt{1 + X_2 (F_y - F_r)^2}}$

$= \frac{(2.15 \text{ in})(1890 \text{ ksi})}{(50 \text{ ksi} - 10)(12 \text{ in/ft})} \sqrt{1 + \sqrt{1 + \frac{0.014 (50 \text{ ksi} - 10)^2}{\text{ksi}^2}}}$

$L_r = 20.27 \text{ ft}$

$\phi M_p = \phi F_y Z$

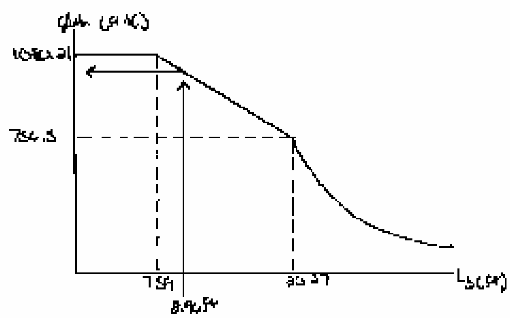
$= \frac{(0.85)(50 \text{ ksi})(305 \text{ in}^3)}{12 \text{ in/ft}}$

$\phi M_p = 1080.21 \text{ ft·k}$

$\phi M_r = \phi F_r S_x$ — residual stresses

$= \frac{0.85 (50 \text{ ksi} - 10)(267 \text{ in}^3)}{12 \text{ in/ft}}$

$\phi M_r = 756.5 \text{ ft·k}$



· 7.59 ft < 8.96 ft < 20.27 ft ⟶ Inelastic Buckling

Reduced Moment, ϕM_n

$\phi M_n = \frac{(\phi M_p - \phi M_r)}{(L_r - L_p)} (L_r - L_b) + \phi F_r S_x$

$= \frac{(1080.21 \text{ ft·k} - 756.5 \text{ ft·k})}{(20.27 \text{ ft} - 7.59 \text{ ft})} (20.27 \text{ ft} - 8.96 \text{ ft}) + \frac{0.85 (50 \text{ ksi} - 10)(267 \text{ in}^3)}{12 \text{ in/ft}}$

$\phi M_n = 1045.24 \text{ ft·k} > M_u \longrightarrow$ Good.

Deflection Checks

- $\Delta_{PL} = \dfrac{PL^3}{28.716EI}$ ← due to 5 evenly space point loads (Pu)

$= \dfrac{(37.94k)(53.75ft)^3(12in/ft)^3}{28.716(29000ksi)(3620in^4)}$

$\Delta_{PL} = 1.56in$

- $\Delta_{wu} = \dfrac{5wL^4}{384EI}$

$= \dfrac{5(102lb/ft)(53.75ft)^4(12in/ft)^3}{384(29000ksi)(3620in^4)(1000lb/k)}$

$\Delta_{wu} = 0.15in$

- $\Delta_{TOTAL} = \Delta_{PL} + \Delta_{wu}$

$= 1.56in + 0.15in$

$\Delta_{TOTAL} = 1.71in$

- $\Delta_{max} = L/360$

$= \dfrac{(53.75ft)(12in/ft)}{360}$

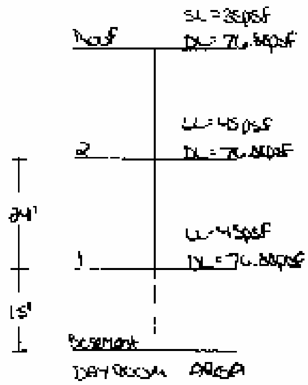
$\Delta_{max} = 1.79in$ $\quad$ $\Delta_{max} > \Delta_{TOTAL} \rightarrow$ GOOD

Use W27×102 A572 50ksi Steel for girder size for the Prison Cell Block, Option #1

D.4 Wide Flange Column Design – Sample Calculation

Column Design 1/

Sample Calculation – Prison Cell Block
Option #4 – two intermediate columns



$\boxed{\text{Column \#6 – Basement}}$

DL = Concrete = 61.88psf
Mechanical = 15psf
76.88psf

LL = occupancy = 40psf
furniture = 5psf
45psf

Tributary Area
(19.34ft × 27.15 × 2) = 1050 ft²
potentially double TA due to identical area on opposite side of cell blocks

Live Load Reductions

- $R_1 = 1 - 0.0008(TA - A_0)$
  $= 1 - 0.0008(1050 - 100)$
  $= 0.24$
- $R_2 = 0.75 - 0.20(DL/LL)$
  $= 0.75 - 0.20(76.88/45)$
  $= 0.41$
- $R_3 = 0.5$ or 0.6
  $= \underline{0.6}$

Load Combinations

- $P_u = 1.4DL(TA)$
  $= 1.4(76.88psf \cdot 3)(1050ft^2)$
  $= 339.04k$
- $P_u = (1.2DL + 1.6LL + 0.5SL)TA$
  $= 1.2(76.88psf \cdot 3)(1050ft^2) + 1.6(45psf \cdot 2)(0.6)(1050ft^2) + 0.5(35psf \cdot 1)(1050ft^2)$
  $= \underline{399.70k}$
- $P_u = (1.2DL + 0.5LL + 1.6SL)TA$
  $= [1.2(76.88psf \cdot 3) + 0.5(45psf \cdot 2)(0.6) + 1.6(35psf \cdot 1)](1050ft^2)$
  $= 377.76k$

Steel Properties

$F_y = 50ksi$
$E = 29000ksi$
$\phi = 0.85$

Trial Size – using Table 4-2, AISC LRFD Manual, Edition 3

Try W14x61

- $A_g = 17.9 in^2$
- $Z = 102 in^3$
- $S_x = 92.2 in^3$

$X_1 = 2170 ksi$

$X_2 = 0.00286 (1/ksi)^2$

Column Capacity

$\frac{KL_x}{r_x} = \frac{(1.0)(15ft)(12in/ft)}{5.98in}$

$\frac{KL_x}{r_x} = 30.10$

$\frac{KL_y}{r_y} = \frac{(1.0)(15ft)(12in/ft)}{2.45in}$

$\frac{KL_y}{r_y} = \underline{73.47}$

Buckling Capacity

$\lambda_c = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}}$

$= \frac{73.47}{\pi}\sqrt{\frac{50ksi}{29000ksi}}$

$\lambda_c = 0.97$

Since $\lambda_c < 1.5$ → column behaves inelastically (short column) → $F_{cr} = 0.658^{\lambda^2} F_y$

$\phi F_{cr} = \phi(0.658^{\lambda^2})F_y$

$= 0.85(0.658^{0.97^2})(50ksi)$

$\phi F_{cr} = 28.67 ksi$

$\phi P_n = \phi F_{cr}(A_g)$

$= (28.67ksi)(17.9in^2)$

$\phi P_n = 513.19k$

$\frac{P_u}{\phi P_n} = \frac{399.72k}{513.19k} = 0.78 < 1.0$ → Good

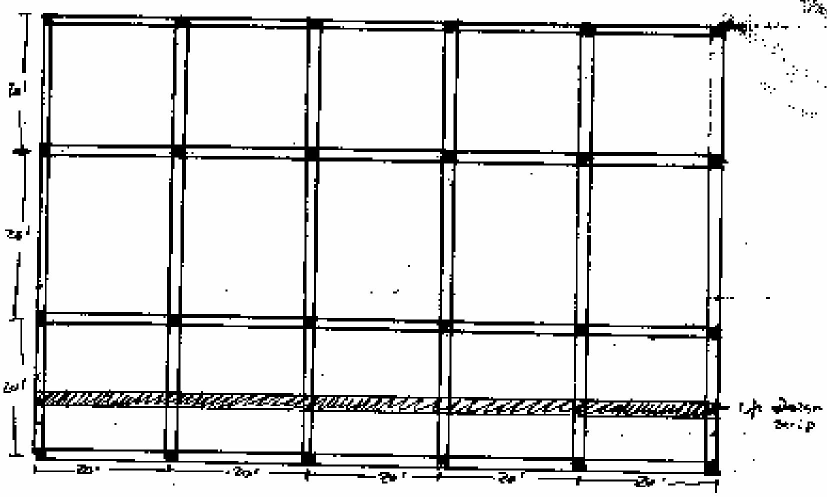
$\boxed{\text{Use A W14x61 A572 50ksi steel column for Column K6-Basement}}$

D.5 Reinforced Concrete Slab Design – Sample Calculation

Slab Design | For Steel Frame | J. Farrell
20'x28'x20' bay by 20' column spacing | Floors 1-6 | MAP

$f_y = 60{,}000$ psi

$f'_c = 3000$ psi



Estimate slab thickness

End bay = $h = \frac{20 \times 12}{24} = 10''$

Interior bay $h = \frac{20 \times 12}{28} = 8.57''$

try 10" slab – 3/4" clear cover, #4 bars $d = 9''$

$w_D = \frac{10 \text{ in}}{12 \text{ in/ft}} \times 150 \text{ pcf} = 125$ psf

Other dead loads

Ceilings = 5 psf
Floor fills = 5 psf
Mechanical = 10 psf

} total = 20 psf {200 - 250 usual construction}

total DL = 145 psf

total LL = 100 psf (IBC 2000 1607-5) — 80 psf + 20 psf (movable partitions) = 100 psf

$w_u = 1.4(145) + 1.7(100) = 373$ psf   load per foot of strip = 373 #/ft

Thickness required for moment

$$\frac{bd^2}{12000} = \frac{M_u}{\phi k_n}$$

$$\phi k_n = \phi f'_c \left[\omega(1 - .59\omega)\right] \quad \text{where} \quad \omega = \frac{\rho f_y}{f'_c}$$

$$\omega = \frac{0.01(60{,}000)}{3000} = .2$$

First interior support

$$M_u = \frac{w_u l_n^2}{10}$$

$$M_u = \frac{373(20)^2}{10} = 14.92 \text{ k/ft}$$

$$\phi k_n = .9(3000)(.2(1 - .59(.2))) = 476.28$$

M_u max = 14.92 k-ft/ft

2nd interior support

$$M_u = \frac{w_u l_n^2}{11} = 13.56 \text{ k-ft/ft}$$

1

Slab Design | For Steel Frame | R. Farrell | Page

20 x 25 x 20 + 7 20' column spacing | MQP | 2/2

M_u = 14.92 k-ft/ft

$$bd^2 = \frac{14.92\,(12000)}{.9(.8)\ldots} = 395.9 \text{ in}^3$$

$$d = \sqrt{\frac{395.9}{12}} = 5.60'' \quad \text{min. thickness} \qquad d = 9'' \text{ so use OK}$$

Check Thickness for Shear

$$V_u = \frac{1.15\, w_u l_n}{2} = \frac{1.15(373)(20)}{2} = 6.29 \text{ kips}$$

$$\phi V_c = .85\left(2\sqrt{f'_c}\, b_w d\right) = .85\left(2\left(\sqrt{3000}\right)(12)(9)\right) = 10.06 \text{ kips}$$

$\phi V_c > V_u$ so slab ok for shear

Reinforcement Design

$$A_s = \frac{M_u \,(12000)}{\phi f_y\, jd} \qquad jd = .925(9) = 8.325$$

$$A_s = \frac{14.92\,(12000)}{.9(60{,}000)(8.325)} = .398 \text{ in}^2/\text{ft}$$

$$a = \frac{A_s f_y}{\phi f'_c\, b_w} = \frac{.398\,(60{,}000)}{.85(3000)(12)} = .780 \qquad jd = d - a/2 \qquad a/2 = \frac{.780}{2} = 8.61$$

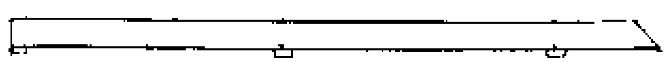
$$jd\,(9) = 8.61 \qquad j = .957$$

$$A_s = \frac{M_u\,(12000)}{.9(60{,}000)(8.61)} = .0258\, M_u$$

$A_{s\,min}$ = .0018bh = .0018(12)(10) = .216

temp + shrinkage

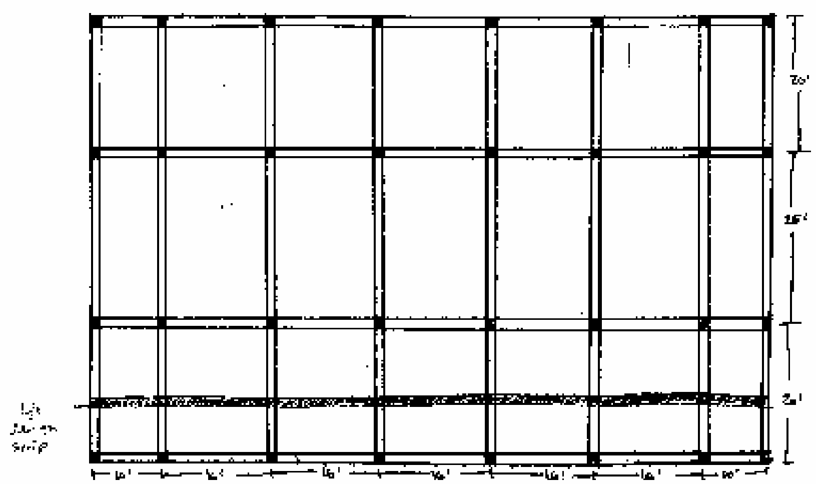
A_s = .0018bh = .216 ∴ use #4 @ 12" o.c.



l_n	20	20	20	20	20	20
$w_u l_n^2$	149.2	149.2	149.2	149.2	149.2	149.2
M coeff.	1/24	1/14	1/10	1/16	1/11	1/16
M_u	6.22	10.66	14.92	9.33	13.56	9.33
A_s req	~~.160~~	.275	.385	.241	.350	.241
$A_{s\,min}$	.216	.216	.216	.216	.216	.216
rebar	#4 @ 9"	#4 @ 8"	#4 @ 6"	#4 @ 9"	#4 @ 6"	#4 @ 9"
A_s provided	.267	.3	.4	.267	.4	.267

This maximum design ... all options are presented in the following spreadsheets

Slab Design | Steel frame | Floors 1-6 | J. Farrell | Page 3 of
20'x25'x20' | 16'x16' column | | MGP |



End bay thickness
$h = \frac{l}{24} = \frac{10 \times 12}{24} = 5''$

Interior bay thickness
$h = \frac{l}{28} = \frac{16 \times 12}{28} = 6.86''$

try a 7" thick slab $\frac{3}{4}''$ clear cover + #5 bars
$\therefore d = 7 - (.75 + \frac{.5}{2}) = 6''$

$w_p = \frac{6\,in}{12\,in/ft} \times 150\,pcf = 75\,psf$

Other dead loads - 2 psf ceiling
9 psf flooring
11 psf Mechanical
} total = 20 psf - from ASD - steel construction

total D_L = 95 psf

total LL = 100 psf - floor 1
80 psf + 20 psf (movable partitions) = 100 psf floors 2-6

$w_u = 1.4(95) + 1.7(100) = 303\,psf \Rightarrow$ load per 1 ft strip = 303 #/ft

Thickness required for Moment
$\frac{bd^2}{12000} = \frac{M_u}{\phi k_n}$

$\phi k_n = \phi f'_c\, \omega(1-.59\omega)$ where $\omega = \frac{\rho f_y}{f'_c} = \frac{.01(60,000)}{3000} = .2$

$\phi k_n = .9(3000)(.2)(1-.59(.2)) = 476.28$

1st interior support
$M_u = \frac{w_u l_n^2}{10} = \frac{303(13)^2}{10} = 5.12\ ^{k\text{-}ft}/_{ft}$ 2nd interior support $M_u = \frac{w_u l_n^2}{11} = \frac{303(16)^2}{11} = 7.05\ ^{k\text{-}ft}/_{ft}$

$M_u\ max = 7.05\ ^{k\text{-}ft}/_{ft}$ $bd^2 = \frac{7.05(12000)}{476.28} = 177.6\ in^3$

$d = \sqrt{\frac{177.6}{12}} = 3.85''$ $d = 3.85''$ to keep precise so. deflection 6" as always.

Check thickness for Shear

7^{th} int $V_u = \frac{1.15\, w_u l_n}{2} = \frac{1.15(303)(16)}{2} = 2765\ lb$

2^{nd} int V_u $\frac{1.15\, w_u l_n}{2} = \frac{1.15(303)(16)}{2} = 2788\ lb$

$\phi V_c = .85\left(2\sqrt{f'_c}\, b_w d\right) = .85(2)(\sqrt{3000})(12)(6) = 6704\ lb$

$\phi V_c > V_u$ so slab is okay for shear

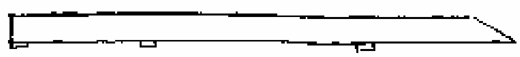
Reinforcement design

$A_s = \frac{M_{max}\,12000}{\phi f_y\,(jd)} = \frac{7.05(12000)}{.9(60000)(.925)(6)} = .282\ in^2$ $jd = .925(6) = 5.55\ in$

$a = \frac{A_s f_y}{.85 f'_c b_w} = \frac{.282(60,000)}{.85(4000)(12)} = .553$ $jd = d - \frac{a}{2} = 6 - \frac{.553}{2} = 5.724$

$j(6) = 5.724$ $j = .954$

$A_s = \frac{M_u(12000)}{\phi f_y jd} = \frac{M_u(12000)}{.9(60,000)(5.724)} = .0388\, M_u$



l_n	10	16	13		16	16	16
$w_u l_n^2$	30.3	30.3	51.2		77.6	77.6	77.6
M_{coeff}	1/24	1/14	1/10	1/16	1/16	1/11	1/16
M_u	1.26	2.16	5.12		4.38	7.05	4.58
$A_{s\,req}$	.049	.084	.199		.188	.274	.188
$A_{s\,min}$	.151	.151	.151		.151	.151	.151
rebar	#4@15"	#4@15"	#4@12"		#4@12"	#4@8"	#4@12"
$A_{s\,provided}$	.16	.16	.2		.2	.3	.2

$A_{s\,min} = .0018\, bh = .0018(12)(7) = .151\ in^2$ max spacing $= 5h = 30'' \rightarrow$ use 18" o/c for code

temperature & shrinkage steel

$A_s = .0018\, bh = .151$ use #4 @ 15" o/c

As a note on how the roof loadings for the reinforced slab were calculated.

$S = 0.7\, I_s\, C_e\, C_t\, C_s\, p_g$

p_g · snow loading = 50 psf   Assuming exposed A

$I_s = 1.1$

$C_e = 1.1$

$C_t = 1.0$

$C_s = 1.0$

$S = 0.7(1.1)(1.1)(1.0)(1.0)(50) = 42.35$ psf

Roof live loading

$L_r = 20\, R_1 R_2$ IBC 2000   1607.11.2

for 20x16 A_t span = 16x25 = 400 ft²

$R_1 = 1.2 - 0.001 A_t = 1.2 - 0.001(400) = .8$

$R_2 = 1$ when $F \leq 4$ F = # of inches of rise per foot. Consider flat roof

$L_r = 20(.8)(1) = 16$ psf

for 20x20 A_t 2025   20x25 = 500 ft²?

$L_r = R_1 R_2\, 20$

$R_1 = 1.2 - .001(500) = .7$

$R_2 = 1$

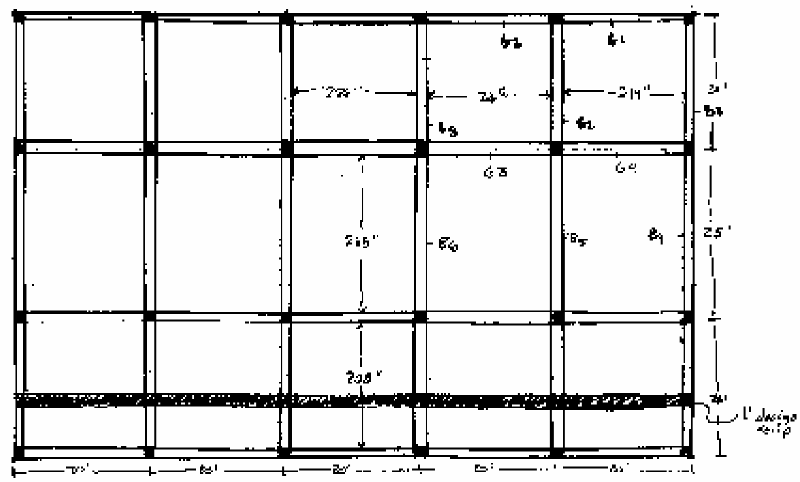
$L_r = 20(.7)(1) = 14$ psf

So to find w_u the calculations were set up as follows

$w_u = 1.4\,(DL_{slab} + DL_{other}) + 1.7\,(L_r + S)$

$f_y = 60{,}000$ psi
$f'_c = 5000$ psi

Assume 16" girders & end beams
Assume 18" interior



1. Estimate floor thickness
end bays: $h = \ell/24 = \frac{215}{24} = 8.95"$
interior bay: $h = \ell/28 = \frac{236}{28} = 8.04"$

Try a 9" thick slab with 3/4" clear cover and #4 bars
$d = 9 - (.75 + \frac{1}{2}) \approx 8"$

2. Compute the total factored loads
slab: DL = (9/12) x 150 pcf = 112.5 psf

other dead loads: floor cover 8 psf, ceiling 5 psf, Mechanical 11 psf → 20 psf (Zero 250 load application)

total DL = 112.5 + 20 = 132.5 psf
total LL = 100 psf for 1st floor + 80 psf + 20 psf = 100 psf for floors 2-6 (IBC 2000 1607.5)

$w_u = 1.4(132.5) + 1.7(100) = 355.5$

Thickness required for flexure
$\frac{bd^2}{12000} = \frac{M_u}{\phi R_n}$

$M_n = \rho f_y [bd(1 - .59\rho\omega)]$ where $\omega = \rho \frac{f_y}{f'_c} = \frac{.01(60000)}{5000} = .12$

$\phi R_n = .9(5000)(.12)(1 - .59(.12)) = 496.25$

1st interior support
$M_u = \frac{w_u \ell_n^2}{10}$

$M_u = \frac{355.5(18.46)^2}{10}$

$M_u = 12.11$ ft-k/ft

$h = \frac{21.9 + 8.26}{2} \times \frac{5}{12} = 16.44$ ft

Slab Design [illegible] A. Farrell

[illegible] MAP

Page 2 of [illegible]

2^{nd} interior support

$$M_u = \frac{w_u l_n^2}{11} = \frac{355(18.83)^2}{11} = [illegible]\ ^{k\text{-}ft}/_{ft}$$

M_u max $= 12.11\ ^{k\text{-}ft}/_{ft}$

$$bd^2 = \frac{M_u(12000)}{\phi R_n} = \frac{12.11(12000)}{[illegible]} = [illegible]\ in^2$$

$d = \left(\frac{[illegible]}{12}\right)^{1/2} = 5.06''$ d min = 5.06" for [illegible] moment, [illegible] $d = 5''$ [illegible]

4. Check if thickness is adequate for shear [illegible] $V_u < \phi V_c$ to neglect shear reinforcement

$V_u = \frac{1.15\,w_u l_n}{2}$ @ 1st int. support $\frac{1.15(355.5)(18.83)}{2} = 3773.5$ lb

2nd interior support $\frac{1.15(355.5)(18.83)}{2} = [illegible]$ lb

$\phi V_c = 0.85(2\sqrt{f'_c}\,b_w d) = 0.85(2)(\sqrt{3000})(12)(8) = 8939$ lb

$\phi V_c > V_u$ so slab is okay for shear

5. A_s

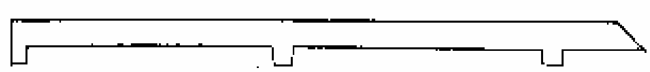
$$A_s = \frac{M_u(12000)}{\phi f_y jd} = \frac{(12.11)(12000)}{.9(60{,}000)(.925(8))} = .364\ in^2/ft$$

$a = \frac{A_s f_y}{\phi f'_c b_w} = \frac{.364(60{,}000)}{.85(3000)(12)} = .714$ $jd = d - a/2 = 8 - .714/2 = 7.643$ $j(8) = 7.643$ $j = .955$

$A_s = \frac{M_u(12000)}{.9(60{,}000)(7.643)} = .029\,M_u$ $A_{s,min} = .0018bh = .0018(12)(8) = .154\ in^2$

[illegible] = .154 @ 12" o/c spacing

Max spacing = (12") or 3t = 24"



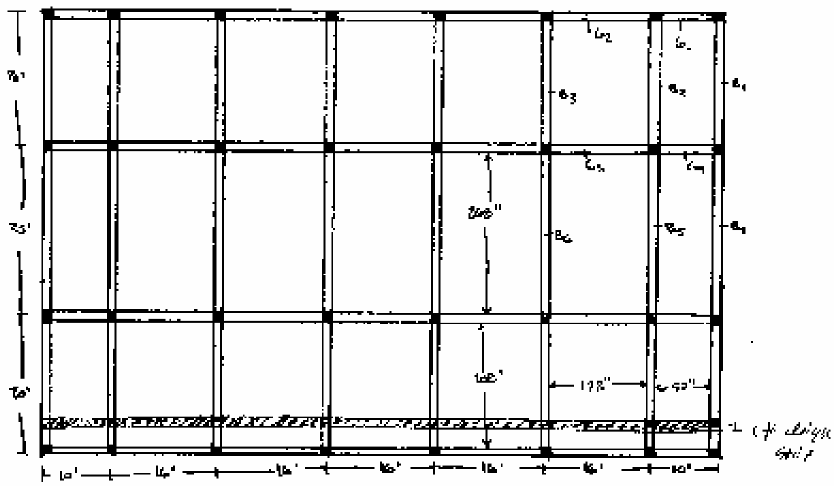
L_n	18.08	19.48	18.46	12.33	18.83	18.83
w_u lb/ft²	116.34	116.91	131.19	126.05	126.05	126.05
Mcoeff	1/24	1/14	1/10 1/11	1/16	1/11	1/16
M_u	9.84	8.30	12.11	7.88	11.46	7.88
$A_{s\,req}$	.146	.241	.351	.229	.332	.229
$A_{s\,min}$	.154	.154	.154	.154	.154	.154
Rebar	#4@12"	#4@9"	#4@6"	#4@9"	#4@6"	#4@9"
A_s provided	.2	.267	.4	.267	.4	.267

Slab Design 20' x 25' x 20'

Reinforced Concrete
floors 1-6

A. Farrell
MAP

Assume 16" girders & end beams $f_y = 60{,}000$
Assume 14" interior beams $f'_c = 3{,}000$



1. Estimate floor thickness

exterior bay $h = l/24 = 97/24 = 4.04"$
interior bay $h = l/28 = 178/28 = 6.36"$

Try a 6.5" thick slab with 3/4" clear cover & #4 bar
$d = 6.5 - (.75 + .5/2) = 5.5"$

2. Compute trial factored load
slab DL = $w_c \cdot \frac{6.5}{12} \cdot 150$ pcf = 81.25 psf

other dead loads: floor cover = 4 psf, ceiling = 5 psf, mechanical = 11 psf } 20 psf (ASCE 7-98 min. design load)

total DL = 81.25 + 20 = 101.25 psf
total LL = 100 psf for 1st floor 80 psf + 20 psf (movable partitions) = 100 psf for floors 2-6

$w_u = 1.4(101.25) + (1.7)(100) = 311.75$ psf ≈ 312 psf

3. Thickness required for moment

$$\frac{bd^2}{12000} = \frac{M_u}{\phi k_n}$$

$\phi k_n = \phi f'_c [\omega(1-.59\omega)]$ $\omega = \rho f_y / f'_c = \frac{.01(60{,}000)}{3000} = .2$
$.9(3000)(.2)(1-.59(.2)) = 476.28$

1st interior support

$l_n = \frac{4.7 + 18.2}{2} = 11.46'$

$M_u = \frac{w_u l_n^2}{10} = \frac{312(11.46)^2}{10} = 4.10 \text{ k-ft/ft}$

2nd interior support

$l_n = \frac{170}{12} = 14.8\bar{3}$

$M_u = \frac{w_u l_n^2}{11} = \frac{312(14.83)^2}{11} = 6.24 \text{ k-ft/ft}$

$M_{u\,max} = 6.24 \text{ k-ft/ft}$ $\qquad bd^2 = \frac{6.24(12000)}{476.88} = 157.2 \text{ in}^3$

$d = \sqrt{\frac{157.2}{12}} = 3.62$

$d_{min} = 3.62 < d = 5.5"$

↳ Slab ok for Moment Allowance

4. Check for Shear

$V_u = \frac{1.15 w_u l_n}{2}$ @ 1st int. support $\quad V_u = \frac{1.15(312)(8.08)}{2} = 1449 \text{ lb/ft}$

@ 2nd int. support $\quad V_u = \frac{1.15(312)(14.83)}{2} = 2660 \text{ lb/ft}$

$\phi V_c = \phi(2\sqrt{f'_c}\, b_w d) = .85(2)(\sqrt{3000})(12)(5.5) = 6145 \text{ lb/ft}$

$\phi V_c > V_{u\,max}$ so slab is okay for shear

5. As $\quad A_s = \frac{M_u(12000)}{\phi f_y j d} = \frac{6.24(12000)}{.9(60000)(.925(5.5))} = .273$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.273(60000)}{.85(3000)(12)} = .536$ $\qquad jd = d - a/2 = 5.5 - \frac{.536}{2} = 5.232$

$j(5.5) = 5.232 \quad j = .951$

$A_s = \frac{M_u(12000)}{.9(60000)(5.232)} = .042 M_u$ $\qquad A_{s\,min} = .0018bh = .0018(12)(6.5) = .140 \text{ in}^2/\text{ft}$

temp + shrinkage $= .0018bh = .140 \text{ in}^2/\text{ft}$

Max spacing $= 18"$ o.c. or $3(6.5") = 19.5"$ o.c.

l_n	3.46	8.28	11.46	14.83	14.83	14.82
$w_u l_n^2$	20.4	20.4	41.0	68.6	68.6	68.6
M coeff	1/24	1/14	1/10 1/11	1/16	1/11	1/16
M_u	.85	1.46	4.10	4.29	6.24	4.29
As req	.0357	.061	.172	.180	.264	.180
As min	.140	.140	.140	.140	.140	.140
Rebar	#4@15"	#4@15"	#4@12"	#4@12"	#4@9"	#4@12"
As provided	.16	.16	.2	.2	.269	.2

As a note on how the roof slab loading was calculated. All other steps are the same as presented before and thus will not be reproduced.

$S = 0.7 I_s C_e C_t C_s P_g$ Assuming exposed

$P_g = 50$ psf
$I_s = 1.1$
$C_e = 1.1$
$C_t = 1.0$
$C_s = 1.0$

$S = 0.7(1.1)(1.1)(1.0)(1.0)(50) = 42.35$ psf

Roof live loading (IBC 2000 1607.11.2)

for 16'x16' $L_r = 20 R_1 R_2$

$R_1 = 1.2 - .001(A_t)$ A_t worst = 16x25 = 400 ft²
$R_1 = 1.2 - .001(400) = .8$
$R_2 = 1$ when $F \le 4$ F = # of inches of rise per foot → consider roof flat

$L_r = 20(.8)(1) = 16$ psf

for 20'x20' $L_r = 20 R_1 R_2$
$R_1 = 1.2 - .001(A_t)$ A_t = 20x25 = 500 ft²
$R_1 = 1.2 - .001(500) = .7$
$R_2 = 1$
$L_r = (20)(.7)(1) = 14$ psf

To find w_u the calculations were set up as follows

$w_u = 1.4(DL_{slab} + DL_{other}) + 1.7(S + L_r)$

D.6 Reinforced Concrete Beam Design – Sample Calculation

Beam Design 20x25x20 | Columns 20x20 | Reinforced Frame Floors 1-6 | A. Farrell MOP | Page 1 of 6

Due to economic and weight considerations, a slab with a thickness of 6" will be used in the beam design. Assume 12" thick concrete block exterior walls. $f_y = 60{,}000$, $f'_c = 3000$.
Beams are simply supported.

Beam 1: Exterior 20' beam

Loading on beam

from slab — tributary length $= \frac{20 \times 12"}{2} = 120"$ — thickness $= 6"$

$$150 \text{ psf} \left(\frac{120" \times 6"}{144}\right) = 750 \text{ lb/ft of beam}$$

from other DL's

floor covering = 5 psf
ceilings = 4 psf
mechanical = 11 psf

$$\left.\right\} 20 \text{ psf} \times \left(\frac{120"}{12}\right) = 200 \text{ lb/ft of beam}$$

exterior wall 12" thickness for concrete block = 90 psf → Design of Wood Structures

$\hookrightarrow = 90 \text{ psf} \times \left(\frac{120}{12}\right) = 900 \text{ lb/ft of beam}$

total dead load (not including self weight) = 750 + 200 + 900 = 1850 lb/ft of beam

live loading

$$100 \text{ psf} \left(\frac{120}{12}\right) = 1000 \text{ lb/ft of beam}$$

1) Estimate Dead load of beam

weight = 10 to 20% of loading = $(1850 + 1000)(.1) = 285$ lb/ft
$(1850 + 1000)(.2) = 570$ lb/ft

or h = 8 to 10% of span and b = .5h

h	b	weight
h = 20(.08)×12 = 19.2"	.8'	192 lb/ft
h = 20(.1)×12 = 24"	1'	300 lb/ft

range 192 to 300 lb/ft, try 300 lb/ft for self weight

2) Compute factored M_u

$$w_u = \frac{1.4(1850 + 300) + 1.7(1000)}{1000} = 4.99 \text{ k/ft}$$

$$M_u = \frac{w_u l_n^2}{8} = \frac{4.99(20)^2}{8} = 250 \text{ k-ft}$$

3) Compute b and d

$$\frac{M_u}{\phi R_n} = \frac{bd^2}{12000} \qquad R_n = f'_c\,\omega\,(1 - .59\omega) \qquad \omega = \frac{\rho f_y}{f'_c} = \frac{.01(60{,}000)}{3000} = .2$$

$$\phi R_n = .9(3000)(.2)(1 - .59(.2))$$

$$\phi R_n = 476.28$$

$$bd^2 = \frac{(250)(12000)}{476.28}$$

$$bd^2 = 6299 \text{ in}^3$$

1

Beam Design Column Reinforced Frame A. Farrell
Beam 24x20 20x20 Floor 1-6 MAP Page 2

$bd^2 = 6299$ if $b = 12"$ $d = 22.89"$ $h = 26.39"$
$b = 14"$ $d = 21.19"$ $h = 24.69"$
$b = 16"$ $d = 19.82"$ $h = 23.32"$
$b = 18"$ $d = 18.69"$ $h = 21.19"$

try using a $b = 18"$ $h = 24"$ $d = 20"$ beam

4.) Check dead load & revise M_u

b x h x γ of concrete =

$1.5 \times 2 \times 150 = 400$ lb/ft of beam

$$w_u = \frac{1.4(1850 + 350) + 1.7(1000)}{1000} = 4.85 \text{ k/ft} \qquad M_u = \frac{w_u l_n^2}{8} = \frac{4.85(20)^2}{8} = 242.5 \text{ ft-k}$$

M_u assumed > M_u actual so sizing is okay

5) Compute reinforcement

$jd = d - a/2 = .875d = .875(20) = 17.5"$

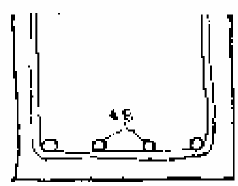
$$A_s = \frac{M_u}{\phi f_y jd} = \frac{242.5 \times 12''/ft}{.9(60)(17.5)} = 3.079 \text{ in}^2$$

$$A_s \min = \frac{3\sqrt{f'_c}}{f_y} b_w d \quad \text{or} \quad \frac{200 b_w d}{f_y} \qquad A_{s\min} = 1.067 \text{ in}^2$$

$$\frac{3\sqrt{3000}}{60000}(16)(20) \quad \text{or} \quad \frac{200(16)(20)}{60000}$$

.876 1.067

$A_{s\,req} > A_{s\min}$ so design for 3.25 in² of steel being needed



Options: 4 #8 bars A_s provided = 3.16 in²
6 #7 bars = 3.60 in²
2 #11 bars = 3.12 in²

6. Compute d assume 1.5" of clear cover & 3/8" stirrup

$$d = 24 - \left(1.5 + 3/8 + \frac{1.00}{2}\right) = 21.6"$$

7. Compute a, check if $f_s = f_y$ & see if tension controlled

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{3.16(60,000)}{.85(3000)(16)} = 4.65 \text{ in} \qquad \frac{a}{d} = \frac{4.65}{21.6} = .215 \qquad \frac{a_b}{d} > .503 \qquad \frac{a(\epsilon)}{d_t} = .34$$

table A-4 Wang & Salmon

$d_t = d = 21.6$ $\frac{a}{d_t} = .215 \leq \frac{a(\epsilon)}{d_t} = .36$ so tension controlled $\phi = .90$

because $\frac{a}{d} < \frac{a_b}{d}$ $f_s = f_y$

8. Compute $M_n = A_s f_y (d - a/2)$

$$M_n = \frac{3.16(60{,}000)(21.5 - \frac{3.86}{2})}{12000} = 305 \text{ ft-k}$$

$$\phi M_n = 337.8(.90) = 874 \text{ ft-k}$$

$\phi M_n > M_u$ therefore design is okay.

Beam 7 - Interior beam 20' long

Loading on beam

from slab tributary length = 20' x 12" = 240" ___ thickness = 6"

$$\left(\frac{240 \times 6}{144}\right)(150 \text{ psf}) = 1500 \text{ lb/ft}$$

other dead loads: floor cover 5 psf, ceiling 4 psf, mechanical 11 psf $\Big\}$ 20 psf

$$20 \text{ psf} \times 20' = 400 \text{ lb/ft}$$

total dead weight = 1500 + 400 = 1900 lb/ft

live loading 100 psf x 20' = 2000 lb/ft

1) Estimate Dead Load of beam

weight = 10 to 20% of loading = .1(2000+1900) = 390 lb/ft
.2(2000+1900) = 780 lb/ft

or h = 8 to 10% of span = .08(20) = 1.6' $b = .5h$ = .8' = 192 lb/ft
.10(20) = 2.0' = 1' = 300 lb/ft

try 600 lb/ft for self weight

2) Compute factored M_u

$$w_u = \frac{1.4(1900+600) + 1.7(2000)}{1000} = 6.90 \text{ k/ft}$$

$$M_u = \frac{w_u l_n^2}{8} = \frac{(6.90)(20)^2}{8} = 345 \text{ k-ft}$$

3) Compute b and d

$$\frac{M_u}{\phi k_n} = \frac{bd^2}{12000}$$

$$k_n = f'_c\,\omega(1-.59\omega) \qquad \omega = \frac{\rho f_y}{f'_c} = \frac{.01(60{,}000)}{3000} = .2$$

$$\phi k_n = .90(3000)(.2)(1-.59(.2)) = 476.28$$

$$bd^2 = \frac{345(12000)}{476.28}$$

$bd^2 = 8692\ in^3$

$b = 12"$	$d = 26.41$	$h = 30.41$
$b = 14"$	$d = 24.42$	$h = 28.42$
$b = 16"$	$d = 23.31$	$h = 26.81$
$b = 18"$	$d = 21.98$	$h = 24.48$

Try an $b = 18"$ $h = 24"$ and $d = 20"$

4) Check dead load and Revise M_u if necessary

b x h x pcf

$$(18/12)(24/12)(150) = 450\ lb/ft$$

$$w_u = \frac{1.4(1800+450)+1.7(2000)}{1000} = 6.64\ k/ft$$

$$M_u = \frac{6.64(20)^2}{8} = 335\ k\text{-}ft$$

M_u assumed > M_u actual so sizing are okay

5) Compute Reinforcement

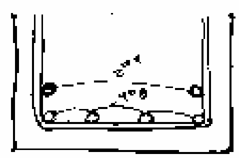
$$jd = d - a/2 \approx .875d = .875(20) = 17.5"$$

$$A_s = \frac{M_u}{\phi f_y jd} = \frac{335(12)}{.9(60)(17.5)} = 4.25\ in^2$$

$$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y}\,b_w d \quad \text{or} \quad \frac{200\,b_w d}{f_y}$$

$$\frac{3\sqrt{3000}}{60{,}000}(18)(20) \quad \text{or} \quad \frac{200(18)(20)}{60000} \qquad A_{s\,min} = 1.20\ in^2$$

$.986$ $\qquad$ 1.2



$A_{s\,req} > A_{s\,min}$ $\quad A_s = 4.25"$

options $\quad$ 6 #8 bars = 4.74 in²

4#8 bars 2#7 = 4.36 in²

6. Compute d

assume Clear cover = 1.5"

$$d = 24 - (1.5 + 3/8 + .1875 + 1.00 + \frac{.750}{2}) = 19.875\ in$$

7) Compare a, check if $f_s = f_y$ & see if tension controlled

$$a = \frac{A_s f_y}{\phi f'_c b_w} = \frac{4.36(60{,}000)}{.85(3000)(18)} = 5.70 \qquad \frac{a}{d} = \frac{5.70}{19.9} = .268$$

$$\frac{a_b}{d} = .503 \qquad \frac{a_{tcl}}{d_t} = .319$$

$$d_t = 24 - (1.5 + (3/8) + (.875/2)) = 21.7$$

$$\frac{a}{d_t} = \frac{5.70}{21.7} = .263$$

$\frac{a}{d} < \frac{a_b}{d}$ so $f_s = f_y$ $\qquad \frac{a}{d_t} < \frac{a_{tcl}}{d_t}$ so tension controlled & $\phi = .90$

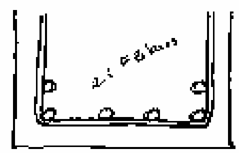
8) $\phi M_n \geq M_u$

$$M_n = A_s f_y (d - a/2) = \frac{4.36(60{,}000)(19.9 - \frac{5.70}{2})}{12000} = 372 \qquad \phi M_n = 335$$

$\phi M_n < M_u \Rightarrow$ redo area of steel

Try 6 #8 bars $A_s = 4.74\ in^2$

6) $d = 24 - (1.5 + (3/8) + (.875/2) + .875 + 1) = 19.8''$



7) Compute a, check if $f_s = f_y$ & see if tension controlled

$$a = \frac{A_s f_y}{\phi f'_c b_w} = \frac{4.74(60{,}000)}{.85(3000)(18)} = 6.20 \qquad \frac{a}{d} = \frac{6.20}{19.8} = .313$$

$$d_t = 24 - (1.5 + (3/8) + (.875/2)) = 21.7 \qquad a/d_t = \frac{6.20}{21.7} = .286$$

$\frac{a}{d} < \frac{a_b}{d}$ so $f_s = f_y$ $\qquad \frac{a}{d_t} < \frac{a_{tcl}}{d_t}$ so tension controlled $\phi = .90$

8) $\phi M_n \geq M_u$

$$M_n = \frac{A_s f_y (d - a/2)}{12000} = \frac{4.74(60{,}000)(19.8 - \frac{6.20}{2})}{12000} = 395\ kft$$

$\phi M_n = 356\ kft$ $\qquad \phi M_n > M_u$ this design works

$b = 18''$ $d = 20''$ $h = 24''$ $A_s = 4.74\ in^2$
6 #8 bars

Due to the similarity of calculations, only the loading determination will be shown for rest of the beam design.

- 20 x 20 column
  Long loading for beams

  - Exterior beam – tributary length = 10'
    10' x 12 = 120"   thickness = 5"

slab DL $\left(\frac{5 \times 120}{144}\right)(150\ \text{pcf}) = 625\ \text{lb/ft}$

other dead loads   20 psf for roofing + mechanical $20\ \text{psf} \times 10 = 200\ \text{lb/ft}$

live loading $S = 42.35$ psf $42.35(10) = 423.5\ \text{lb/ft}$
$L_r = 14$ psf $14(10) = 140\ \text{lb/ft}$

$$w_u = \frac{1.4(625 + 200) + 1.7(423.5 + 140)}{1000} = 2.11\ \text{k/ft}$$ of loading along exterior beams

- Interior beam – tributary length = 20'
  thickness = 5"

slab DL $\left(\frac{5 \times 240}{144}\right)(150) = 1250\ \text{lb/ft}$

other dead loads   20 psf for roofing and mechanical $20 \times 20 = 400\ \text{lb/ft}$

live loading $S = 42.35$ psf $42.35(20) = 847\ \text{lb/ft}$
$L_r = 14$ psf $14(20) = 280\ \text{lb/ft}$

$$w_u = \frac{1.4(1253 + 400) + 1.7(847 + 280)}{1000} = 4.27\ \text{k/ft}$$

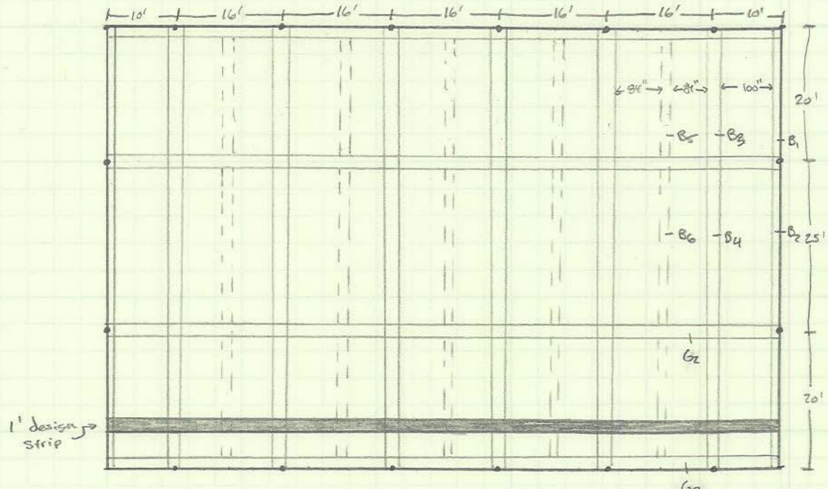
The design is principally the same for the layout with columns spaced at 10' x 16'. The only change is that there needs to be 6 types of columns designed for 3 different tributary areas. The exterior tributary length is 5', the second is 13' and the third is 16'. All of the processes remain the same. Single spreadsheets can be found after the 20x20 column spacing spreadsheets.

Also – please note that beam heights were attempted to be kept uniform over the length of the spans.

D.7 Reinforced Concrete T-Beam Design – Sample Calculations

Reinforced Frame 10x16 Column spacing | Reinforced Slab Example Calculations | Floors 1-6 | Page 3 of

Assume 12" interior beams & 14" girders $\quad f_y = 60000$ psi, $f'_c = 3000$ psi



- Estimate floor thickness

  end bay $= \frac{100}{24} = 4.17"$ $\qquad$ try a 4" thick slab $\qquad h = 4"$

  interior bay $= \frac{84}{24} = 3.5"$ $\qquad$ 3/4" clear cover & #4 bars $\qquad d = 3"$

- Compute factored loads

  $w_D = (4/12")(150 \text{ pcf}) = 50$ psf

  other dead loads: floor cover & fill = 4 psf, ceiling = 5 psf, mechanical = 11 psf $\} = 20$ psf $\qquad$ total dl = 70 psf

  live loading

  1st floor = 100 psf

  2nd - 6th floors = 80 psf + 20 psf (movable partitions) = 100 psf

  $w_u = 1.4(70) + 1.7(100) = 268 \text{ psf} \approx 270 \text{ psf}$

- Thickness required for moment $\quad \frac{bd^2}{12000} = \frac{M_u}{\phi k_n}$

  $\phi k_n = \phi f'_c \omega (1 - .59\omega)$ $\qquad \omega = \frac{\rho f_y}{f'_c}$ $\qquad \omega = \frac{.01(60{,}000)}{3000}$ $\qquad \omega = .2$

  $\phi k_n = .9(3000)(.2)(1-(.59)(.2))$

  $\phi k_n = 476.28$

1st interior support

$M_u = \frac{w_u l_n^2}{10} \qquad l_n = \frac{100" + 84"}{2} = 7.67'$

$M_u = \frac{270(7.67)^2}{10}$

$M_u = 1.59$ k-ft

2nd interior support $\quad M_u = \frac{w_u l_n^2}{11} \quad l_n = 84" = 7' \quad M_u = \frac{270(7)^2}{11} = 1.20$ k-ft

Reinforced Frame	Reinforced Slab	Floors 1-6	Page
16x16 column Spacing	Example Calculations		4 of

M_u max = 1.59 k-ft

$bd^2 = \frac{M_u(12000)}{\phi k_n} = \frac{1.59(12000)}{476.28} = 40.06$ $d = \sqrt{\frac{40.06}{12}} = 1.83$ in

$d = 3" > d_{min} = 1.83"$ so slab is okay for moment

- Check shear

1st interior $V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(7.67)(270)}{2} = 1.19$ k

2nd interior $V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(270)(7)}{2} = 1.09$ k

$\phi V_c > V_u$ so OK for Shear

$\phi V_c = .85\sqrt{f'c}\,(2)(b_w)d = .85(2)(\sqrt{3000})(12)(3) = 3.35^k$

- Compute As

$A_s = \frac{M_u(12000)}{\phi f_y jd}$ $jd = .925d = 2.775$

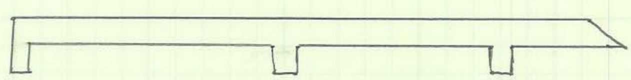
$A_s = \frac{1.59(12,000)}{.9(60,000)(2.775)} = .127$ in²/ft

$a = \frac{A_s f_y}{\phi f'c b_w} = \frac{.127(60,000)}{.85(3000)(12)} = .249$

$A_s = \frac{M_u(12000)}{.9(60,000)(2.88)} = .0772\,M_u$

$jd = d - a/2 = 3 - \frac{.249}{2} = 2.88$ $j = .96$

As min = .0018bh = .0018(12)(4) = .0864 in²/ft
As temp & shrinkage = .0018(12)(4) = .0864 in²/ft
Max spacing = 3h = 12"



l_n	8.33	8.33	7.67	7	7	7
$w_u l_n^2$	18.74	18.74	15.88	13.23	13.23	13.23
M coeff	1/24	1/14	1/10 1/11	1/16	1/11	1/16
M_u =	.781	1.34	1.59	.827	1.20	.827
As req	.06	.103	.123	.0638	.0926	.0638
As min	.0864	.0864	.0864	.0864	.0864	.0864
rebar	#4@12"	#4@12"	#4@12"	#4@12"	#4@12"	#4@12"
As provided	.2	.2	.2	.2	.2	.2

Roof Calculation

As a note on how the roof slabs will be loaded the following details the loading calculation. All of the other steps were the same thus they will not be presented.

$S = 0.7 I_s C_e C_t C_s p_g$ Assuming exposed

$p_g = 50$ psf
$I_s = 1.1$
$C_e = 1.1$
$C_t = 1.0$
$C_s = 1.0$

$S = 0.7(1.1)(1.1)(1.0)(1.0)(50) = 42.35$ psf

Roof live loading

for 10'x16' $L_r = 20 R_1 R_2$

$R_1 = 1.2 - .001(A_t)$ A_t worst $= 10(25) = 250$ ft^2

$R_1 = 1.2 - .001(250) = .95$

$R_2 = 1$ when $F \leq 4$ F = # of inches of slope rise per foot

$L_r = 19$ psf

for 20x20 $L_r = 20 R_1 R_2$

$R_1 = 1.2 - .001(10)(25)$

$R_1 = .95$

$R_2 = 1$

$L_r = 19$ psf

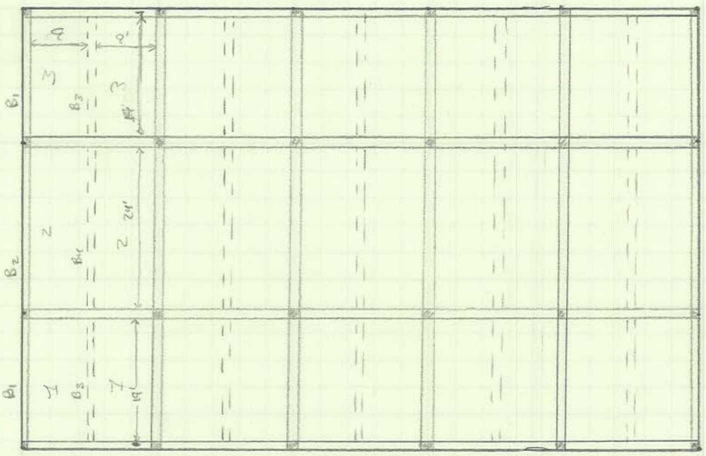
$w_u = 1.4(D_{slab} + D_{other}) + 1.7(L_r + S)$

T beam Design Example

Page 1of

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

concrete = f_y = 60,000 psi   Assume 12" beams & girders   LL = 100 psf
f'_c = 3000 psi   4" slab   DL = 20 psf
stirrups f_y = 40,000 psi



Factored loads on the beam

a) influence area A - Positive moment B3 (1)

A_I: $(15)(9+1+9) = 361 \text{ ft}^2$

live load reduction $L = L_0\left(.25 + \frac{15}{\sqrt{A_I}}\right)$ $L = 100\left(.25 + \frac{15}{\sqrt{361}}\right) = 104$ psf

b) influence Area B - Interior Negative moment (1+2)

$A_B = (15+24+1)(9+1+9) = 836 \text{ ft}^2$

$L = 100\left(.25 + \frac{15}{\sqrt{836}}\right) = 76.88$ psf

c) Influence area C - positive moment B4 (2)

$(24)(9+1+9) = 456 \text{ ft}^2$

$L = 100\left(.25 + \frac{15}{\sqrt{456}}\right) = 95.24$ psf

Stem size will be chosen on basis of negative moment @ 1st interior support

Dead loading → slab = $(4/12)(150)$ = 50 psf
other = 20 psf
} total = 70 psf
reduced live load = 77 psf

Factored load $1.4(70) + 1.7(77) \Rightarrow 228.9 \text{ psf} \approx 230 \text{ psf}$

Tributary width $\left(\frac{9}{2} + 1 + \frac{9}{2}\right) = 10'$

Factored load per foot from slab $10' \times 230\,psf = 2.3\ k/ft$

Estimate beam stem weight

10 to 20% of factored loading on beam

$w_1 = .1(2.30) = .23\ k/ft$

$w_2 = .2(2.30) = .46\ k/ft$

$h = .08$ to $.1\ l_n$ $\quad b_w = .5h$

$h_1 = .08(25) = 2'$ $\quad b_1 = 1'$ $\quad w_3 = .3\ k/ft$

$h_2 = .10(25) = 2.5'$ $\quad b_2 = 1.25'$ $\quad w_4 = .47\ k/ft$

try factored weight of .4 k/ft

trial loading = $2.3 + .4 = 2.7\ k/ft$

- Actual size of beam stem

a) minimum height based on deflections – for longest span = 24

$$\min h = \frac{24}{18.5} = 1.30' = 15.58''$$

b) based on moment @ 1st interior support

$$M_u = \frac{w_u l_n^2}{10} \qquad l_n = \text{average of the spans} = \left(\frac{20+25}{2}\right) = 22.5'$$

$$M_u = \frac{2.7(22.5)^2}{10} = 136.7\ k\text{-}ft \approx 137\ k\text{-}ft$$

try $\rho = 0.013$ no compression control $\qquad \omega = \frac{\rho f_y}{f'_c} = \frac{0.013(60{,}000)}{3000} = .26$

$$\phi k_n = \phi f'_c\, \omega(1 - .59\omega)$$

$$\phi k_n = .9(3000)(.26)(1 - .59(.26)) = 595$$

$$\frac{bd^2}{12000} = \frac{M_u}{\phi k_n}$$

$$bd^2 = \frac{(137)(12000)}{595} = 2764\ in^3$$

Assume 1 layer of steel

b	d	h
10	16.62	19.13
12	15.18	17.68
14	14.05	16.55"

try a 12" x 18" stem $\quad d = 15.5''$

c) Check Shear Capacity

$V_u = \phi(V_c + V_s) \qquad V_c = 2\sqrt{f'_c}\, b_w d \qquad V_s = 8\sqrt{f'_c}\, b_w d$

$V_u = \phi(10)(\sqrt{f'_c}\, b_w d) \qquad \phi = .85$

$$\text{Max } V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(2.7)(24)}{2} = 37.26\ k$$

$$\min b_w d = \frac{V_u(1000)}{8.5\sqrt{3000}} = \frac{37.26(1000)}{8.5\sqrt{3000}} = 80.03\ in^2$$

for $b = 12$ $\quad d = 6.67$ in

Shear does not govern

d) Summary use $b = 12"$
$d = 15.5"$ (1 layer of steel)
$h = 18"$

- Compute dead load of stem & recompute total load per foot

$$\text{weight per foot of slab} = \frac{(12 \times 18)}{144} \times 150\text{ pcf} = 225 \text{ lb/ft} = .225 \text{ k/ft}$$

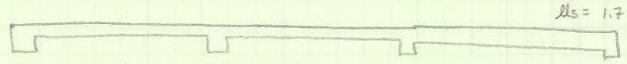
$$\text{total factored load} = 2.3 + 1.4(.225) = 2.62 \text{ k/ft}$$

- Calculate flange width for + moment regions - use smallest option

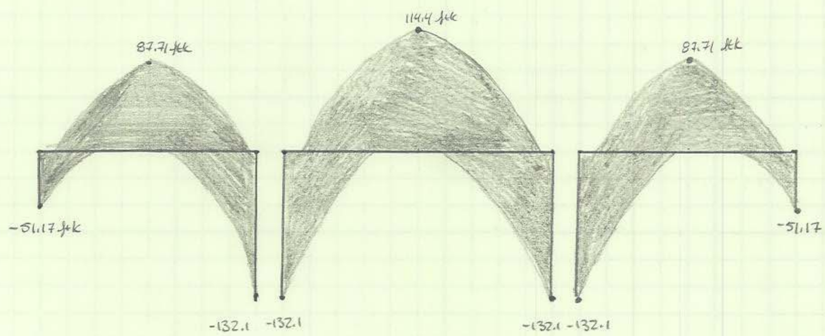
$.25 l_n = .25(20 \times 12) = 60"$
$b_w + 2(8 \times 6) = 12 + 2(48) = 108"$ use flangwidth of 60"
$b_w + \frac{9 \times 12}{2} + \frac{9 \times 12}{2} = 120"$

- Compute Moments dead load $= 1.4\left(\frac{70(10)}{1000} + .225\right) = 1.30$ k/ft

$L_1 = 1.7(104)(10) = 1.77$ k/ft
$L_2 = 1.7(77)(10) = 1.31$ k/ft
$L_3 = 1.7(96)(10) = 1.63$ k/ft



l_n	20	20	22.5	25	22.5	20	20
w_u	3.07	3.07	2.61	2.93	2.61	3.07	3.07
$w_u l_n^2$	1228	1228	1321	1831	1321	1228	1228
C_m	1/24	1/14	1/10	1/16	1/10	1/14	1/24
M_u	-51.17	+ 87.71	- 132.1	+ 114.4	- 132.1	+ 87.71	-51.17



- Design reinforcement
a) Point of Max Negative moment

$$A_s = \frac{M_u(12000)}{\phi f_y j d} \quad \text{assume } j = .875 \quad A_s = \frac{132.1(12000)}{.9(60{,}000)(.875)(15.5)} = 2.16 \text{ in}^2$$

if exactly that much is used ... $a = \frac{2.16(60{,}000)}{.85(3000)(12)} =$

$$a = \frac{A_s f_y}{\phi f'_c b_w} = 4.24 \text{ in}$$

$\frac{a}{d} = \frac{a}{d_t} = \frac{4.24}{15.5} = .283$ since $\frac{a}{d} < \frac{a_b}{d}$ $f_s = f_y$ ultimate

and $\frac{a}{d_t} < \frac{a_{tcl}}{d_t}$ so tension controlled & $\phi = .90$

Recompute A_s $A_s = \frac{M_u(12000)}{.9(60000)(15.5 - \frac{4.24}{2})} = .0166 M_u^-$

b) A_s required for Max positive moment – assume $j = .95$

$A_s = \frac{114.4(12000)}{.9(60{,}000)(.95)(15.5)} = 1.73 \text{ in}^2$ assume $a < h_f$ $b = b_{flange} = 60"$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{1.73(60{,}000)}{.85(3000)(60)} = .678$ $\frac{a}{d} = \frac{a}{d_t} = \frac{.678}{15.5} = .0437$

$\frac{a}{d} < \frac{a_b}{d}$ so $f_s = f_y$ ultimate

$\frac{a_{tcl}}{d_t} > \frac{a}{d_t}$ so tension controlled & $\phi = .90$

Recompute A_s $A_s = \frac{M_u(12000)}{.9(60000)(15.5 - \frac{.678}{2})} = .0146 M_u$

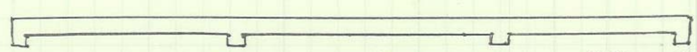
c) Determine $A_{s\,min}$

$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d \geq \frac{200 b_w d}{f_y}$

$= \frac{3\sqrt{3000}}{60{,}000}(12)(15.5) \geq \frac{200(12)(15.5)}{60{,}000}$

$= .509 \geq .62 \text{ in}^2$ $A_s > A_{s\,min}$ in both pos. & neg regions

d) Calculate A_s & select bars



M_u	-51.17	87.71	-132.1	114.4	-132.1	87.71	-51.17
A_{coeff}	.0166	.0146	.0166	.0146	.0166	.0146	.0166
A_s	.849	1.28	2.19	1.67	2.19	1.28	.849
$A_s > A_{s\,min}$	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Bars	2#6	3#6	4#7	4#6	4#7	3#6	2#6
A_s prov.	.88	1.32	2.4	1.76	2.4	1.32	.88
b_w ok?		Yes		Yes		Yes	

- Reinforcement Distribution

a) Positive Moment Region – smallest # of bars = 3#6 bars $A_s = 1.32 \text{ in}^2$

centroid of bars $d_c = 1.5 + .375 + \frac{.750}{2} = 2.25"$

$A = \frac{2 d_c \times b_w}{\# \text{ of bars}} = \frac{2(2.25)(12)}{3} = 18 \text{ in}^2$ per bar

$Z = f_s \sqrt[3]{d_c A}$ $f_s = 0.6 f_y$ $Z = (.6)(60{,}000)(\sqrt[3]{(2.25)(18)})$

$Z = 124$ k/in $Z < 175$ so Okay

b) Negative Moment Region

A_s distributed over smaller of b_w & $\frac{l_n}{10} = \frac{240}{10} = 24''$

@ interior 4 top bars, 2 for stirrup, 2 inside bars

$d_c = 1.5 + .375 + \frac{.875}{2} = 2.31\text{ in}$

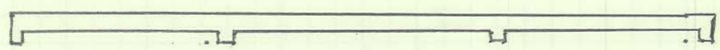
$f_s = 36\text{ ksi}$

$175 = 36\sqrt[3]{2.31A}$

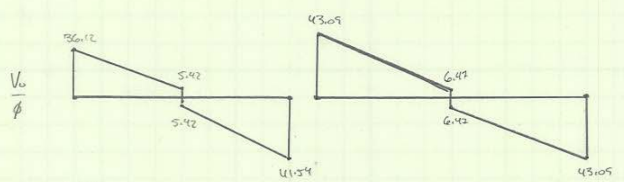
$\max A = 16.23\text{ in}$

$s = \frac{A}{2d_c} = \frac{16.23}{2(2.31)} = 2.21\text{ in}^2$

- Shear Reinforcement



l_n		20'			25'			20'	
w_u		3.07			2.93			2.07	
w_{Lu}		1.77			1.63			1.77	
C_v	1.0	.15	1.15	1.0	.15	1.0	1.15	.15	1.0
$\frac{w_u l_n}{2}$		30.7			36.63			30.7	
V_u	30.7	4.61	35.31	36.63	5.50	36.63	35.31	4.61	30.7
$V_n = \frac{V_u}{\phi}$	36.12	5.42	41.54	43.09	6.47	43.09	41.54	5.42	36.12



a) Exterior of B3

Critical section for shear is @ $d = 15.5''$ from the support

$\frac{V_u}{\phi}$ @ $d = 36.12 - \frac{15.5}{120}(36.12 - 5.42) = 32.15^k$

needs stirrups if $V_n > V_c/2$ $\qquad V_c = 2\sqrt{f'_c}\,b_w d = 2\sqrt{3000}(12)(15.5) = 20.38\text{ k}$

$\frac{V_c}{2} = 10.19$

$\frac{V_u}{\phi} > \frac{V_c}{2}$ so Stirrups needed

#3 Grade 40 @ 90° hook w/ #4 Stirrup support bar

Max stirrup spacing is smaller of $d/2$ & $s = \frac{A_v f_y}{50 b_w}$

$\frac{d}{2} = \frac{15.5}{2} = 7.75''$

$s = \frac{.22(40{,}000)}{50(12)} = 14.67''$

min spacing = 7''

@ D from End

$$S = \frac{.22(40000)(15.5)}{(32.15-10.19)(1000)} = 6.211 \text{ in}$$ use 6" for all spacing through 1st half of beam

$$\text{\# of stirrups} = \frac{10 \times 12" \text{ of length}}{6" \text{ per stirrup}} = 20 \text{ stirrups @ } 6" \text{ o/c}$$

b) at interior end of B3

$$\frac{V_u}{\phi}@d = 41.54 - \frac{15.5}{120}(41.54-5.42) = 36.87$$

$$S = \frac{.22(40,000)(15.5)}{(36.87-10.19)(1000)} = 5.11" \quad \text{go w/4" o/c}$$

when $S = 6"$

$$\frac{V_u}{\phi} = \frac{.22 \times 40 \times 15.5}{6} + \frac{10190}{1000} = 32.92^k$$

$$x = \frac{41.54-32.92}{41.54-5.42} \times 120 \text{ in} = 28.6" \text{ from end}$$

use 24 #3 stirrups @ 4" o/c starting at int. end ; 4 #3 stirrups @ 6" o/c from center

c) Ends of B4

$$\frac{V_u}{\phi} = 43.09 - \frac{15.5}{150}(43.09-6.47) = 39.31$$

$$S = \frac{.22(40,000)(15.5)}{(39.31-10.19)(1000)} = 4.68" \quad \text{go w/4" o/c}$$

$s = 6"$? $$\frac{V_u}{\phi} = \frac{.22 \times 40 \times 15.5}{6} + 10.19 = 32.92$$

$$x = \frac{41.54-32.92}{41.54-6.47} * 150 = 36"$$

for ½ → use on both halves
29 #3 stirrups @ 4" o/c starting @ ends
6 #3 stirrups @ 6" o/c starting @ center

- Design lengths and cutoffs

$d = 15.5$ in

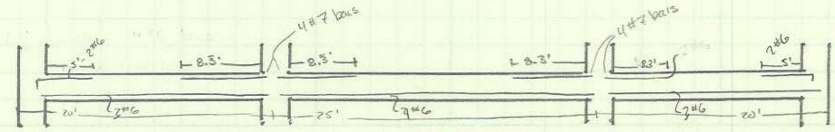
$12 d_b = .875(12) = 10.5$ in for #7 bars

$$\frac{l_n}{16} = \frac{25 \times 12}{16} = 18.75" \text{ for B4}$$

$$\frac{l_n}{16} = \frac{20 \times 12}{16} = 15" \text{ for B3}$$

Using Figure A-5 for detailing

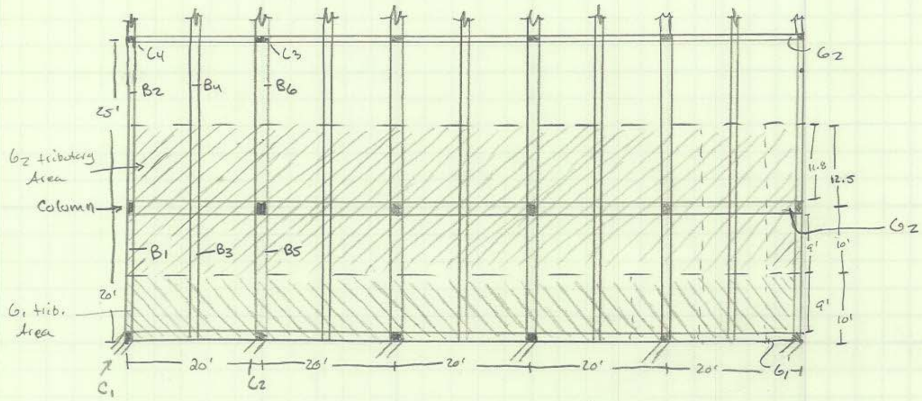
run + moment steel 1st B3 → bottom 2 full wrap



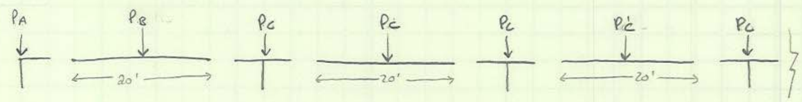
D.8 Reinforced Concrete Girder Design – Sample Calculations

Roof girder Design | Example Calculations | Page 1 of

1) Roof Girder design - using 20x20 option 1 + 4" thick slab



G_1 - design of exterior roof girder



weight of slab per beam = interior = $(4/12)(10')(10')(150\ pcf) = 5000\ lb = P_{si}$

exterior $= (4/12)(10')(5')(150\ pcf) = 2500\ lb = P_{se}$

beam point weight → $B_1 = (12''/12)(18''/12)(10')(150\ pcf) = 2250\ lb = P_1$

$B_3 = (12''/12)(18''/12)(10')(150\ pcf) = 2250\ lb = P_3$

$B_5 = (12''/12)(18/12)(10')(150\ pcf) = 2250\ lb = P_5$

weight from factored loadings w_u roof 20x20 =

$w_{snow} = 42.35$ psf
$w_{dead} = 20$ psf
$w_{live} = 19$ psf
see slab calcs for more details

exterior $= (5')(10')[1.4(20) + 1.7(19 + 42.35)] = 6615\ lb = P_{lde}$

interior $= (10')(10')[1.4(20) + 1.7(19 + 42.35)] = 13230\ lb\ P_{ldi}$

$P_A = P_{se} + P_1 + P_{lde} = 2500 + 2250 + 6615 = 11365\ lb = 11.37^k$

$P_B = P_{si} + P_3 + P_{ldi} = 5000 + 2250 + 13230 = 20480\ lb = 20.48^k$

$P_C = P_{si} + P_5 + P_{ldi} = 5000 + 2250 + 13230 = 20480\ lb = 20.48^k$

self wt. of beam/girder

10 to 20% of wt per span of 20'

1) $.10(20.48) = 2.05^k$
2) $.20(20.48) = 4.10^k$

or h = 8 to 10% length of beam $b = .5h$ wt = ?

note - length = unsupported length = 20'

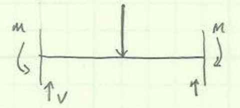
$h = .08(20') = 1.6'$ $b = .8'$ $wt_1 = (1.6)(.8)(20)(150) = 3.84^k$

$h = .10(20') = 2'$ $b = 1'$ $wt_2 = (1)(2)(20)(150) = 6.00^k$

try a girder point load of 4.50^k $P_g = 4.5^k$

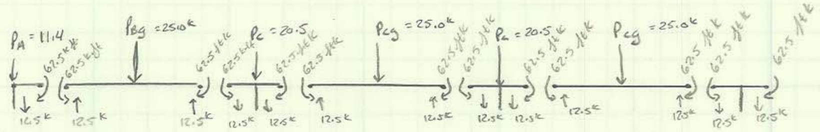
include selfweight of girder in span point loads i.e. $P_{Bg} = P_B + P_g = 20.48 + 4.5 = 25.0^k$

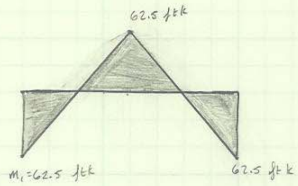
$P_{cg} = P_c + P_g = 20.48 + 4.5 = 25.0^k$



$M = \frac{Pl}{8}$

$V = P/2$





$M_s = \frac{Pl}{4} - \left(\frac{M_1 + M_2}{2}\right) = \frac{25(20)}{4} + \left(\frac{62.5 + 62.5}{2}\right) = 62.5$

M_u max = 62.5 ft k

* find girder dimensions ↓

$f_y = 60{,}000^{psi}$, $f'_c = 3000$ psi

$\frac{M_u}{12000} = \frac{bd^2}{\phi k_n}$

$\phi k_n = \phi f'_c\,\omega(1 - .59\omega)$

$\omega = \frac{\rho f_y}{f'_c} = \frac{0.01(60{,}000)}{3000} = .2$

$\phi k_n = 0.9(3000)(.2)(1 - .59(.2))$

$\phi k_n = 476.28$

$d^+ = 14 - 3.5 =$

$d^- = 14 - 2.5$

$\frac{62.5(12000)}{476.28} = bd^2$

$bd^2 = 1575 \text{ in}^3$

b = 12" $d = 11.45"$ $h = 14.95"$

b = 14" $d = 10.61"$ $h = 14.11"$

$d^+ = 10.5"$

$d^- = 11.5"$

try a 14"x14" girder w/ d = 11.5"

3 #6 bars = 1.32

3 #7 bars = 1.80

2 #8 bars = 1.58

for max $-M_u$ roof G. $A_s = \frac{M_u^-}{\phi f_y jd} = \frac{62.5 \times 12}{.9(60)(.95)(11.5)} = 1.27 \text{ in}^2$

try 3#6 bars

span 7' in on girder from columns

for Max ^+M_u roof G. $A_s = \frac{M_u^+}{\phi f_y jd} = \frac{62.5(12)}{.9(60)(.95)(10.5)} = 1.39 \text{ in}^2$

d top =

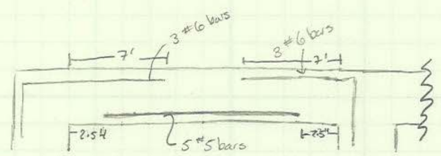
2 #8 bars = 1.58"

5 #5 bars = 1.55"

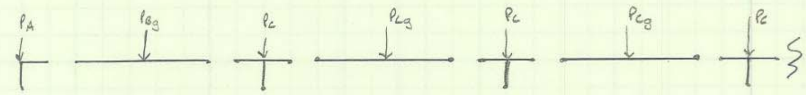
3 #7 bars = 1.80

try 5 #5 bars

span middle 3/4 of the span



G2- design of interior roof girder



weight from factored loadings

exterior $= (10+12.5)(5)[1.4(20) + (1.7)(15+42.35)] = 14.88^k = P_{lde}$

interior $= (10+12.5)(10)[1.4(20) + 1.7(9+42.35)] = 29.77^k = P_{ldi}$

weight from slab

exterior $= (10+12.5)(5)(4/12)(150 \text{ pcf}) = 5625^{lb} = 5.63^k = P_{se}$

interior $= (10+12.5)(10)(4/12)(150 \text{ pcf}) = 11250 \text{ lb} = 11.25^k = P_{si}$

beam point weight

$B_1 = (12/12)(18/12)(10)(150) = 2250^{lb} = 2.25^k = P_1$

$B_2 = (16/12)(18/12)(12.5)(150) = 3750^{lb} = 3.75^k = P_2$

$B_3 = (12/12)(18/12)(10)(150) = 2250^{lb} = 2.25^k = P_3$

$B_4 = (18/12)(18/12)(12.5)(150) = 4219 \text{ lb} = 4.22^k = P_4$

$B_5 = (12/12)(18/12)(10)(150) = 2250 \text{ lb} = 2.25^k = P_5$

$B_6 = (22/12)(18/12)(12.5)(150) = 5156 \text{ lb} = 5.16^k = P_6$

$P_A = P_{lde} + P_{se} + P_1 + P_2 = 14.88 + 5.63 + 2.25 + 3.75 = 26.51^k$

$P_B = P_{ldi} + P_{si} + P_3 + P_4 = 29.77 + 11.25 + 2.25 + 4.22 = 47.49^k$

$P_C = P_{ldi} + P_{si} + P_5 + P_6 = 29.77 + 11.25 + 2.25 + 5.16 = 48.43^k$

Self weight of beam

10 to 20% of wt per 20' span - use max force

$.10(48.43) = 4.84^k$

$.20(48.43) = 9.69^k$

or

$h =$ 8 to 10% of longest span $\qquad b = .5h \qquad$ weight

$h_1 = .08(20') = 1.6' \qquad b_1 = .5(1.6') = .8' \qquad 1.6(.8)(20)(150) = 3.84^k$

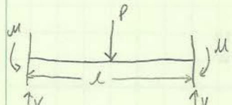
$h_2 = .10(20') = 2' \qquad b_2 = .5(2') = 1' \qquad 2(1)(20)(150) = 6.00^k$

try a 7^k girder self weight

$P_g = 7.00^k$

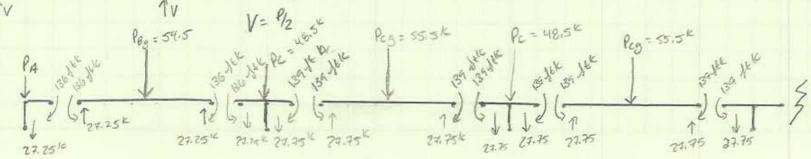
$P_{Bg} = P_B + P_g = 54.49^k \approx 54.5^k$

$P_{cg} = P_c + P_g = 55.42^k \approx 55.5^k$



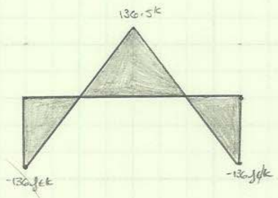
$M = \frac{Pl}{8}$

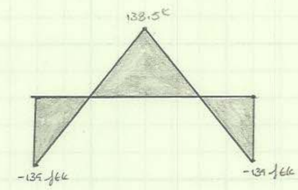
$V = P/2$



1^{st} interior span

2^{nd} interior span





$M_s = \frac{Pl}{4} - \left(\frac{M_1+M_2}{2}\right)$

$M_s = \frac{54.5(20)}{4} - \left(\frac{136+136}{2}\right) = 136.5^{k\text{-}ft}$

$M_s = \frac{Pl}{4} - \left(\frac{M_1+M_2}{2}\right)$

$M_s = \frac{55.5(20)}{4} - \left(\frac{139+139}{2}\right) = 138.5^{k\text{-}ft}$

find girder dimensions M_u max = $139^{k\text{-}ft}$ $f_y = 60{,}000$ psi $f'_c = 3000$ psi

$\frac{M_u}{\phi k_n} = \frac{bd^2}{12000}$

$bd^2 = \frac{(139)(12000)}{476.28} = 3502 \text{ in}^3$

b = "	d "	h "
12	17.08	20.58
14	15.81	19.31
16	14.80	18.3
18	13.95	17.45

try a b = 16" h = 18" $d^+ = 14.5"$ $d^- = 15.5"$

1^{st} interior span

for max M_u^- roof G_2 $A_s = \frac{M_u^-}{\phi f_y j d^-}$

$A_s^- = \frac{(136^{k\text{-}ft})(12")}{.9(60)(.95)(15.5)}$

$A_s^- = 2.05 \text{ in}^2$ try 5 #6 bars

A_s prov = 2.2 in^2

$A_s^+ = \frac{(136.5)(12)}{(.9)(60)(.95)(14.5)}$

$A_s^+ = 2.20 \text{ in}^2$ try 4 #7 bars

A_s prov = 2.40 in^2

2^{nd} interior span

for max M_u^+ = $A_s = \frac{M_u^+}{\phi f_y j d^+}$

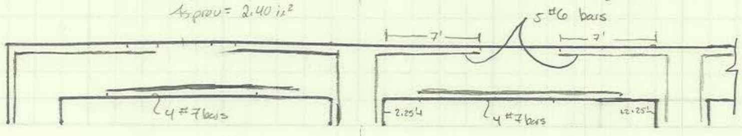
$A_s^- = \frac{(139)(12)}{(.9)(60)(.95)(15.5)}$

$A_s^- = 2.10 \text{ in}^2$ try 5 #6 bars

A_s prov. = 2.2 in^2

$A_s^+ = \frac{(139.5)(12)}{(.9)(60)(.95)(14.5)}$

$A_s^+ = 2.25 \text{ in}^2$ try 4 #7 bars = 2.40 in^2

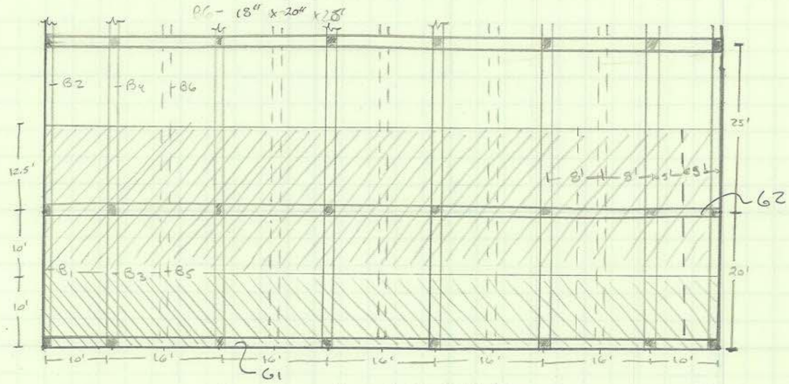


Floor Girder Design	Example Calculations	

1) Floor girder design - using 10-16 option 1 & 4" Slab

Beam sizings - B1 - 12" x 20" x 20'
B2 - 14" x 20" x 25'
B3 - 12" x 20" x 20'
B4 - 18" x 20" x 25'
B5 - 12" x 20" x 20'
B6 - 18" x 20" x 25'

w_{live} = 100 psf
w_{dead} = 20 psf
Slab thickness = 4"



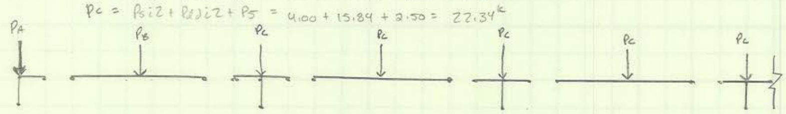
G1 Design

Slab weight
exterior $P_{se} = (5')(10')(4/12)(150\ pcf) = 2500\ lbs = 2.50^k$
interior $P_{si1} = \left(\frac{5+8}{2}\right)(10')(4/12)(150\ pcf) = 3250\ lb = 3.25^k$
$P_{si2} = (8')(10')(4/12)(150\ pcf) = 4000\ lbs = 4.00^k$

factored loadings
$P_{lde} = (5')(10')(1.4(20) + 1.7(100)) = 9900\ lbs = 9.9^k$
$P_{ldi1} = \left(\frac{5+8}{2}\right)(10)(1.4(20) + 1.7(100)) = 12870\ lbs = 12.87^k$
$P_{ldi2} = (8')(10')(1.4(20) + 1.7(100)) = 15840\ lbs = 15.84^k$

Beam loading
$B1 = (12/12)(20/12)(10')(150\ pcf) = 2500\ lbs = 2.50^k = P_1$
$B3 = (12/12)(20/12)(10')(150\ pcf) = 2500\ lbs = 2.50^k = P_3$
$B5 = (12/12)(20/12)(10')(150\ pcf) = 2500\ lbs = 2.50^k = P_5$

$P_A = P_{se} + P_{lde} + P_1 = 2.50 + 9.9 + 2.50 = 14.9^k$
$P_B = P_{si1} + P_{ldi1} + P_3 = 3.25 + 12.87 + 2.50 = 18.62^k$
$P_C = P_{si2} + P_{ldi2} + P_5 = 4.00 + 15.84 + 2.50 = 22.34^k$



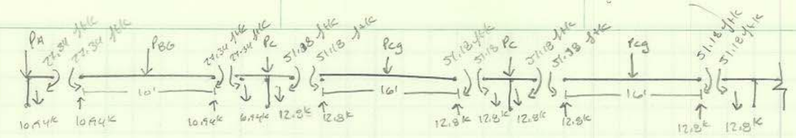
self weight of girder
10 to 20% of wt. per span of 16'
$w_1 = .10(22.34) = 2.23^k$
$w_2 = .20(22.34) = 4.47^k$

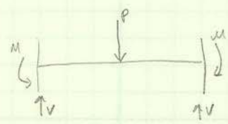
or h = 8 to 10% of l $\qquad b \doteq .5h \qquad$ wt. = ?
$h_1 = .08(16') = 1.28' \qquad b_1 = .64' \qquad w_3 = 1966^{lbs} = 1.97^k$
$h_2 = .10(16') = 1.6' \qquad b_2\ .8' \qquad w_4 = 3072^{lbs} = 3.07^k$

try a self weight of $3.25^k = P_g$ $\qquad P_{Bg} = P_B + P_g$
$P_{Cg} = P_C + P_g$

$P_{Bg} = 21.87$
$P_{Cg} = 25.59$

Floor girder Design | Example Calculations

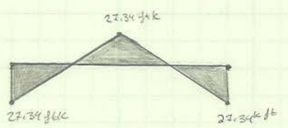




$M = \frac{Pl}{8}$

$V = P/_2$

1st interior bay

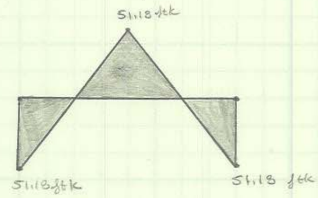


$M_s = \frac{Pl}{4} - \left(\frac{M_1 + M_2}{2}\right)$

$M_s = \frac{(21.87)(10)}{4} - \left(\frac{27.34 + 27.34}{2}\right)$

$M_s = 27.34$ ftk

2nd interior bay



$M_s = \frac{Pl}{4} - \left(\frac{M_1 + M_2}{2}\right)$

$M_s = \frac{25.59(16)}{4} - \left(\frac{51.18 + 51.18}{2}\right)$

$M_s = 51.18$ ftk

find girder dimensions M_u max = 51.18 ftk $f_y = 60{,}000$ psi $f'_c = 3{,}000$ psi

$\frac{M_u}{\phi R_n} = \frac{bd^2}{12000}$

$bd^2 = \frac{(51.18)(12000)}{476.28} = 1289.5 \text{ in}^3$

b	d	h
12"	10.37"	13.78"
14"	9.60"	13.10"
16"	8.98"	12.48"

try a $b = 12"$ $h = 14"$ $d^+ = 10.5"$ $d^- = 11.5"$

1st interior support

$A_s^- = \frac{M_u(12)}{\phi f_y j d^-}$

$A_s^- = \frac{27.34(12)}{.9(60)(.95)(11.5)} = .556 \text{ in}^2$

try 2 #5 bars

$A_{s\,prov} = .62 \text{ in}^2$

$A_s^+ = \frac{M_u(12)}{\phi f_y j d^+}$

$A_s^+ = \frac{27.34(12)}{.9(60)(.95)(10.5)} = .609 \text{ in}^2$

try 2 #5 bars

$A_{s\,provide} = .62 \text{ in}^2$

2nd interior support

$A_s^- = \frac{M_u(12)}{\phi f_y j d^-}$

$A_s^- = \frac{51.18(12)}{.9(60)(.95)(11.5)} = 1.04 \text{ in}^2$

try 4 #5 bars

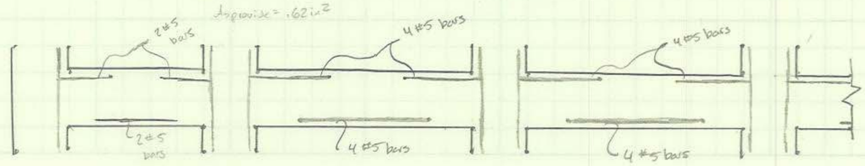
$A_s = 1.24 \text{ in}^2$

$A_s^+ = \frac{M_u(12)}{\phi f_y j d^+}$

$A_s^+ = \frac{51.18(12)}{.9(60)(.95)(10.5)} = 1.14 \text{ in}^2$

try 4 #5 bars

$A_s = 1.24 \text{ in}^2$



Floor Girder Design — Example Calculations

G2 Design

slab weight exterior

$P_{sce} = (5')(10+12.5)(4/12)(150\ pcf) = 5625^{lb} = 5.625^k$

$P_{si1} = (\frac{5+8}{2})(10+12.5)(4/12)(150\ pcf) = 7313^{lb} = 7.313^k$

$P_{si2} = (8)(10+12.5)(4/12)(150\ pcf) = 9000 = 9.00^k$

factored loadings

$P_{le} = (5)(10+12.5)(1.4(20) + 1.7(100)) = 22275^{lbs} = 22.28^k$

$P_{li1} = (\frac{5+8}{2})(10+12.5)(1.4(20) + 1.7(100)) = 28958 = 28.96^k$

$P_{li2} = (8)(10+12.5)(1.4(20) + 1.7(100)) = 35640 = 35.64^k$

$B_1 = (12/12)(20/12)(20)(150) = P_1 = 5000^{lbs} = 5.00^k$

$B_2 = (14/12)(20/12)(25)(150) = P_2 = 7292^{lbs} = 7.29^k$

$B_3 = (12/12)(20/12)(20)(150) = P_3 = 5000^{lbs} = 5.00^k$

$B_4 = (18/12)(20/12)(25)(150) = P_4 = 9375^{lbs} = 9.38^k$

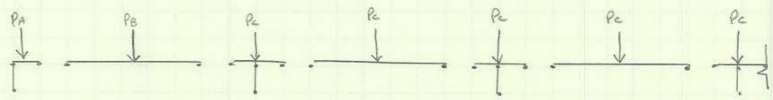
$B_5 = (12/12)(20/12)(20)(150) = P_5 = 5000^{lbs} = 5.00^k$

$B_6 = (18/12)(20/12)(25)(150) = P_6 = 9375^{lbs} = 9.38^k$

$P_A = P_{se} + P_{le} + P_1 + P_2 = 5.625 + 22.28 + 5.00 + 7.29 = 40.2^k$

$P_B = P_{si1} + P_{li1} + P_3 + P_4 = 7.313 + 28.96 + 5.00 + 9.38 = 50.7^k$

$P_C = P_{si2} + P_{li2} + P_5 + P_6 = 9.00 + 35.64 + 5.00 + 9.38 = 59.0$



self weight of girder   10 to 20% of weight per 16' span

$w1 = .10(59.0) = 5.90^k$

$w2 = .20(59.0) = 11.8^k$

or $h = 8$ to 10% of l $b = .5h$ wt

$h_1 = .08(16) = 1.28'$ $b_1 = .64'$ $(1.28)(.64)(16)(150) = 1966^{lbs} = 1.97^k$

$h_2 = .10(16) = 1.6'$ $b_2 = .8'$ $(1.6)(.8)(16)(150) = 3072^{lbs} = 3.07^k$

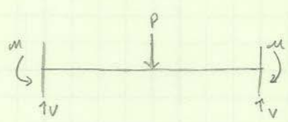
try $6.50^k = P_g$

$P_A = 40.2^k$

$P_{Bg} = 57.2^k$

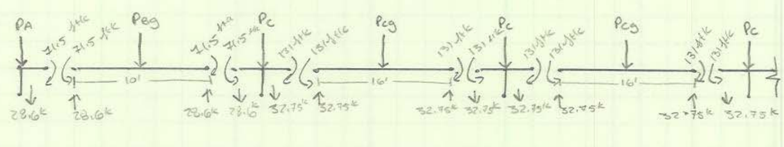
$P_C = 59.0^k$

$P_{Cg} = 65.5^k$

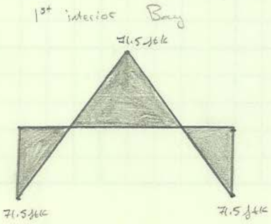


$M = \frac{Pl}{8}$ $M_1 = \frac{57.2(10)}{8} = 71.5\ ft^k$

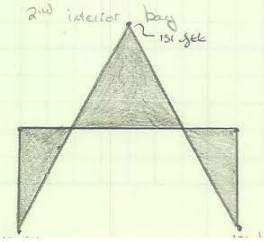
$V = P/2$ $M_2 = \frac{65.5(16)}{8} = 131\ ft^k$



1st interior Bay



2nd interior bay



1st Interior Bay

$$M_s = \frac{Pl}{4} - \left(\frac{M_1 + M_2}{2}\right)$$

$$M_s = \frac{57.2(10)}{4} - \left(\frac{71.5 + 71.5}{2}\right) = 71.5 \text{ ftk}$$

2nd Interior Bay

$$M_s = \frac{Pl}{4} - \left(\frac{M_1 + M_2}{2}\right)$$

$$M_s = \frac{65.5(16)}{4} - \left(\frac{131 + 131}{2}\right) = 131 \text{ ftk}$$

find a girder size M_u max = 131 ftk $f_y = 60{,}000$ psi $f'_c = 3000$ psi

$$\frac{M_u}{\phi k a} = \frac{bd^2}{12000}$$

$$bd^2 = \frac{131(12000)}{476.28} = 3301 \text{ in}^3$$

b	d	h
12"	16.54"	20.04"
14"	15.36"	18.86"
16"	14.36"	17.86"
18"	13.54"	16.04"

try $b = 12"$ $h = 20"$ $d^+ = 16.5"$ $d^- = 17.5"$

1st Interior Support

$$A_s^- = \frac{M_u}{\phi f_y j d^-}$$

$$A_s^- = \frac{71.5(12)}{(.9)(60)(.95)(17.5)} = .956 \text{ in}^2$$

try 2 #7 bars

$A_s = 1.20 \text{ in}^2$

$$A_s^+ = \frac{M_u}{\phi f_y j d^+}$$

$$A_s^+ = \frac{71.5(12)}{.9(60)(.95)(16.5)} = 1.01 \text{ in}^2$$

try 2 #7 bars

$A_s = 1.20 \text{ in}^2$

2nd Interior Support

$$A_s^- = \frac{M_u}{\phi f_y j d^-}$$

$$A_s^- = \frac{131(12)}{.9(60)(.95)(17.5)} = 1.75 \text{ in}^2$$

2 #7 / 1 #8 bars

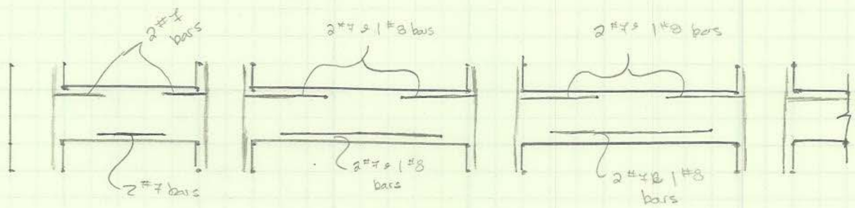
A_s prov = 1.99 in²

$$A_s^+ = \frac{M_u}{\phi f_y j d^+}$$

$$A_s^+ = \frac{(131)(12)}{.9(60)(.95)(16.5)} = 1.86 \text{ in}^2$$

2 #7 / 1 #8 bars

A_s prov = 1.99 in²



D.9 Reinforced Concrete Column Design – Sample Calculations

Roof Column Design
20-20 frame

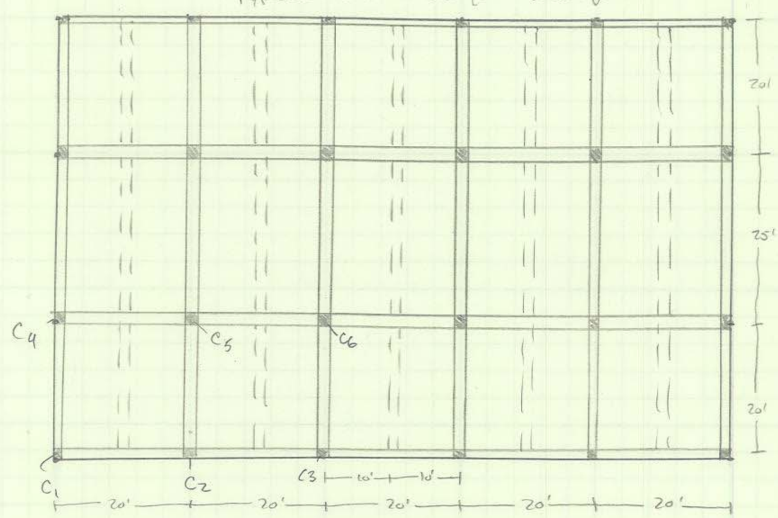
Example Calculation

Page 1 of

Force values taken from Girder Summary Sheet

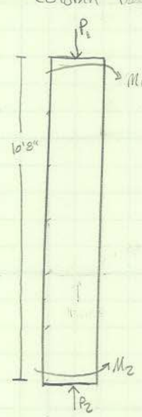
$f_y = 60{,}000$ psi $f'_c = 3000$ psi

Typical Column layout - All floors



using 6.7

Column Design - C_1 - roof



$P_1 = P_{A\,roof} + V_{1\,roof} = 11.6 + 12.7 = 24.3^k$

$P_2 = P_{A\,floor} + V_{1\,floor} = 15.15 + 16.25 = 31.4^k$

$M_1 = 62.5$ ftk

$M_2 = 82.7$ ftk

Select trial properties, trial size & trial reinforcement Ratio

$A_g \geq \dfrac{P_u}{.45(f'_c + f_y\,\rho_t)}$ $\rho_t = 0.015$ $f'_c = 3$ ksi $f_y = 60$ ksi

$A_{g\,trial} \geq \dfrac{31.4}{(.45)(3 + (60 \times .015))} = 17.89\ in^2$ 4.23"x4.23"

try a 12x12 column to be safe

$e = \dfrac{M_u}{P_u} = \dfrac{82.7\ ftk}{31.4\ k} = 2.63\ ft$

try a column w/reinforcing in 2 faces

Slenderness neglected if $\dfrac{k l_u}{r} \leq 34 - 12\left(\dfrac{M_1}{M_2}\right)$

$\dfrac{k l_u}{r} = \dfrac{1.0 \times 128\ in}{.3 \times 12\ in} = 35.6 \leq 34 - (12)\left(\dfrac{24.3}{31.4}\right)$

$35.6 \leq 43.29$ ∴ slenderness can be neglected for trial column → 12"x12" tied w/ bars in 2 faces. $f'c = 3000$ psi, $fy = 60{,}000$ psi

Compute γ = try #8 bars w/ #3 ties @ 1.5" clear cover

$$\gamma = \frac{12 - 2(1.5 + .375 + .5)}{12} = .604$$

$$\frac{\phi P_n}{A_g} = \frac{P_u}{A_g} = \frac{31.4}{12 \times 12} = .218 \text{ ksi}$$

$$\frac{\phi M_n}{A_g h} = \frac{M_u}{A_g h} = \frac{82.7}{12 \times 12 \times 12} = .047 \text{ ksi}$$

McGregor — From A-6 $\rho t = .002$

From A-7 $\rho t = .002$

interpolation for $\gamma = .604$ $\rho t = .002 - 0\left(\frac{.004}{x}\right) = .002$ use $\rho t = 0.01$ ← minimum required by ACI

Select Reinforcement

$A_{st} = \rho t (A_g) = .01(12 \times 12) = 1.44 \text{ in}^2$

Possible Combos
4 #8 bars 3.16 in²
4 #7 bars = 2.40 in²
4 #6 bars = 1.76 in²

try 4 #6 bars - need to redo γ

$$\text{redone } \gamma = \frac{12 - 2(1.5 + .375 + \frac{.750}{2})}{12} = .625$$

$\rho t = .002 - 0\left(\frac{.025}{.15}\right) = 0$ $\rho t = 0.01$ still

Check minimum load capacity

$$\phi P_n = .85\phi\,[.85 f'c (A_g - A_{st}) + f_y (A_{st})]$$

$$\phi P_n = .85(.90)[.85(3000)(144 - 1.76) + 60{,}000(1.76)] = 358.3^k \qquad \phi P_n > P_u$$

Design lap splices

$$l_d = \left(\frac{f_y \alpha \beta \lambda}{20\sqrt{f'c}}\right) d_b$$

assume all bars spliced @ same location
splice length = 1.3 ld

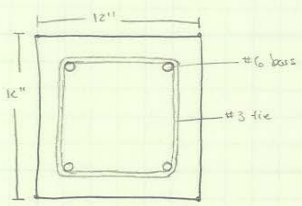
$$l_d = \left(\frac{60{,}000(1)(1)(1)}{20\sqrt{3000}}\right)(.75) = 41.07''$$

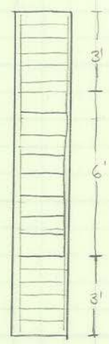
$1.3 l_d = (1.3)(41.07) = 53.4$ in try 54"

Select ties
12 · 3/4 = 12 in – 16 longitudinal bar diameters
48 × 3/8 = 18 in – 48 tie diameters $s = 9''$ if $V_u < \phi .5 V_c$
least dimension - 12 in if $V_u > 0.5\phi V_c$ → bad things
no V_u so no problem
Use #3 ties @ 12" o/c

Use a 12"x12" column with 4 #6 bars, $fy = 60{,}000$ psi, $f'c = 3000$ psi, #3 ties @ 12" o/c, lap splice longitudinal bars @ 54"

See following diagrams





C2 - roof/floor column

$P_1 = P_c$ roof $+ V_1$ roof $+ V_2$ roof $= 20.90 + 12.70 + 12.70$

$P_2 = P_c$ floor $+ V_1$ floor $+ V_2$ floor $= 28.18 + 16.53 + 16.17$

$P_1 = 46.30$

$P_2 = 60.88$

$M_1 = M_1$ roof or M_1 floor (pick larger) $= 82.63$

$M_2 = M_2$ roof or M_2 floor (pick largest) $= 83.56$

Select trial properties, trial size and trial reinforcement ratio

$$A_g \geq \frac{P_u}{.45(f'c + \rho_t f_y)} \qquad \rho_t = 0.015 \quad f'c = 3\text{ ksi} \quad f_y = 60\text{ ksi}$$

$$A_g \geq \frac{60.88}{.45(3 + (.015 \cdot 60))} = 34.67\text{ in}^2 \rightarrow 5.89" \times 5.89"$$

try a 12"x12" column

$$e = \frac{M_u}{P_u} = \frac{83.56}{60.88} = 1.37$$ try a column w/ reinforcing in 2 faces

Slenderness neglected if $\frac{k l_u}{r} \leq 34 - 12\left(\frac{M_1}{M_2}\right)$

$$\frac{k l_u}{r} = \frac{1.0(128)}{.3 \times 12} = 35.6 \leq 34 - 12\left(\frac{46.30}{-60.88}\right)$$

$35.6 \leq 43.13$ so slenderness can be neglected

for trial column try 12"x12" w/ bars in 2 faces, $f_y = 60{,}000$ psi & $f'c = 3000$ psi

try #7 bars & #3 ties @ 1.5" clear cover

$$\gamma = \frac{12 - 2(1.5 + .375 + (\frac{.875}{2}))}{12} = .615$$

$$\frac{\phi P_n}{bA_g} = \frac{P_u}{A_g} = \frac{60.88}{144} = .423 \qquad \frac{\phi M_n}{A_g h} = \frac{M_u}{A_g h} = \frac{83.56}{12 \times 12 \times 12} = .049$$

From A-6 $\rho_g < .01$
From A-7 $\rho_g < .01$ ⟩ McGregor

use $\rho_t = 0.01$

Select reinforcement

$A_{st} = \rho_t (A_g) = 0.01(144) = 1.44 in^2$

try 4 #6 bars

recompute γ $\quad \dfrac{12 - 2(1.5 + .375 + (\frac{.750}{2}))}{12} = .625 \quad \rho_t$ still $= 0.01$

Check Minimum load Capacity

$\phi P_n = .85\phi\,[.85 f'c (A_g - A_{st}) + f_y (A_{st})]$

$\phi P_n = .85(.90)[.85(3000)(144 - 1.76) + 60{,}000(1.76)] = 358.3^k \quad \phi P_n > P_u$

Design lap splices

$l_d = \left(\dfrac{f_y\,\alpha\beta\lambda}{20\sqrt{f'c}}\right) d_b = \left(\dfrac{60{,}000\,(1)(1)(1)}{20\sqrt{3000}}\right)(.750) = 41.07''$

assume all bars lap spliced in the same location splice length $= 1.3 l_d$

$1.3 l_d = (1.3)(41.07'') = 53.4''$ try 54"

Select ties

$s = 12 \times 3/4 = 9 in$
$48 \times 3/8 = 18 in$

$s = 9''$ if $V_u < \phi .5 V_c \quad V_u = 0$

use #3 ties @ 9" o/c

Use a 12"x12" column with 4 #6 bars, $f_y = 60{,}000 psi$, $f'c = 3000 psi$, #3 ties @ 9" o/c, lap splice longitudinal bars @ 54". - See diagram for other column (ironically they were the same).

E Sample Spreadsheet Calculations

E.1 Composite Beam Design Sample Spreadsheet

Prison Beam Design - 5 Infill Beams (19'-6.5" length)							
Given Information							
Dead Load?	71.88	psf		Note: increase concrete wt. by 10% for ponding			
Live Load?	40.00	psf					
Tributary Area?	8.96	ft	=	107.50	in		
Length of Beam?	19.54	ft	=	234.50	in		
Fy	50.00	ksi					
Es	29000.00	ksi					
f'c	3.00	ksi					
Fu	65.00	ksi					
Slab depth (t)	4.50	in					
Load Combinations							
1.4D	901.43	lb/ft					
1.2D + 1.6L	1345.99	lb/ft					
This Value Governs	1345.99	lb/ft					
Mu	64250.26	ft-lb	=	64.25	ft-k		
Z	17.13	in^3					
Trial Beam?	**W12x16**						
As	4.71	in^2					
Z	20.10	in^3					
Uniform load of beam?	16.00	lb/ft					
Recalculated wu	1365.19	lb/ft					
Recalculated Mu	65166.77	ft-lb	=	65.17	ft-k		
Recalculated Z	18.40	in^3					
Is the Beam Adequate?	**Beam is Adequate**						
Local Buckling							
Flange:							
bf/(2tf)	7.50						
Buckling?	No Buckling						
Web:							
h/(tw)	49.40						
Buckling?	No Buckling						
Section is Compact							
Effective Flange Width (be)	58.63	in					
Determining ϕMn							
depth of beam (d)	11.99	in					
tw	0.22	in					
h/tw	49.40	in					
h	10.87						
ϕb is	0.85						
Locate PNA							
a is	1.58	in					
PNA is in	the SLAB						
Mn = Mp =	2286.08	in-k	=	190.51	ft-k		
ϕMn is	161.93	ft-k					
Design of Shear Studs							
Stud Diameter?	0.75	in					
Shear (Vn) is	235.50	k					
Qn is	21.58	k		Ec if w is 150 psf	=	3181.98	ksi
# of studs required	11.00	studs					
Stud Spacing	10.66	in/stud					
Minimum Spacing	4.50	in					
Maximum Spacing	36.00	in					
Composite Section Deflection							
As	4.71	in^2					
Ys	6.00	in					
Ac	4.71	in^2					
Yc	15.70	in					
Y	10.85	in					
Moment of Inertia							
Is	103	in^4					
I	324.92	in^4					
Calculated Deflection							
Δ	0.35	in					
Unshored Construction Check							
Wu = 1.2DL+1.6LL(concrete)	1192.74	lb/ft					
Mu	56.94	ft-k					
z	16.08	in^3					
Is the Beam Adequate?	**Beam is Adequate**						
Serviceability Concerns							
Δmax	0.65	in					
Δconstruction	0.82	in		if W (just DL)	=	749.46	lb/ft
75% Δconstruction	0.62	in					
Is the Beam Adequate?	**Beam is Adequate**						

E.2 Simply Supported Open-web Joist Sample Spreadsheet

	PRISON CORE DESIGN					
	Composite Joist Design					
	Joist A1-5 - Recreation Area					
Location						
Length	60.92	ft				
Trib. Width	10.00	ft				
Composite Live Load						
Occupancy	100.00	psf				
Reduciton Factor	0.68					
Reduced Occupancy	67.97	psf				
Other	5.00	psf				
Total	72.97	psf				
Composite Dead Load						
Mechanical	15.00	psf				
Ceiling	2.00	psf				
Flooring	10.00	psf				
Other	0.00	psf				
Total	27.00	psf				
Noncomposite Dead Load						
Concrete	33.00	psf	Thickness =	2	in. (above ribs)	Note: use (N=9) normal weight concrete (145 pcf)
Decking	1.78	psf	Weight =	145	pcf	with 1.5VL22 metal decking
Bridging	0.10	psf	Type =	1.5VL22		- can carry a superimposed LL of 111psf for
Joist (approximation)	5.00	psf	Rib Height =	1.5	in.	10ft. clear span
Total	39.88	psf				
Total Loads	1398.53	lb/ft				
Allowable Deflection						
Δ_{allow}	2.03	in				
$I_{req'd}$	2361.06	in^4				
Trial Size						
Joist	36VC 1400/105/30		Bridging Rows Required			
Depth	36.00	in		2		
Total Load Category	1400.00	lb/ft				
W_{TJ}	46.00	lb/ft				
W_{360}	1098.00	lb/ft				
N-ds	46.00		Stud Dia.	3/4	in	
			Stud Length	3 3/8	in	
Deflection Checks						
I_{calc}	2909.26	in^4				
$\Delta_{noncomp,DL}$	1.65	in				
$\Delta_{comp,DL}$	0.50	in				
$\Delta_{comp,LL}$	1.34	in				
Δ_{TL}	2.16	in				
Camber Required?	2.20	in				
Construction Deflection Check						
Concrete	39.27	psf				
Decking	1.78	psf				
Bridging	0.10	psf				
Joist	4.60	psf				
Construction	1.64	psf				
Other	0	psf				
Total	47.39	psf				
$\Delta_{construction}$	1.96	in				
Adequate?	Adequate					

E.3 Simply Supported Wide Flange Beam/Girder Design Sample Spreadsheet

PRISON CORE DESIGN												
Beam-Girder Noncomposite Design												
Girder A.5 5 - A.5 5												
Given Information												
Φ_b	0.85											
E	29000	ksi										
F_y	50	ksi										
C_b (critical buckling)	1											
Beam Information												
Beam Length	25.85	ft										
Tributary Width	1.00	ft										
Point Load Details												
Loads (factored)	60.01	k (1)	60.01	k (2)	8.66	k (3)	8.66	k (4)		k (5)		
Loads (unfactored)	42.60	k (1)	42.60	k (2)	6.01	k (3)	6.01	k (4)		k (5)		
Load Location	7.92	ft (1)	17.92	ft (2)	8.5	ft (3)	16.58	ft (4)		ft (5)		
Uniform Load Details												
Dead Load	0.00	psf										
Live Load	0.00	psf	LL Reduction =		1.00							
Snow Load	0.00	psf										
W'_u =1.4DL(TW)	0.00	k/ft										
W'_u = 1.2DL(TW)+1.6LL(TW)+0.5SL(TW	0.00	k/ft										
W'_u = 1.2DL(TW)+0.5LL(TW)+1.6SL(TW	0.00	k/ft										
W'_u (unfactored)	0.00	k/ft										
Moment												
Moment	See Mu eqns		NONE			n = 1	M_u = PL/4		n = 4	M_u = 3PL/5		Note: P_u is equal at all locations
M_u (calculate)	760.19	ft-k	0.00	ft-k		n = 2	M_u = PL/3		n = 5	M_u = 3PL/4		Note: The applied loads are evenly spaced in all equations
$M_{u\,Total}$	760.19					n = 3	M_u = PL/2		uneven	M_u = Pab/L		Note: In case n = 1, P_u is not applied in the center
Z	214.64	in^3										
						Manual Calcuated Mom			760.19	ft-k		
Trial Size	**W24x84**											
A_g	24.7	in^2										
d	24.1	in										
Z	224	in^3										
I_x	2370	in^4										
X_1	1950	ksi										
X_2	0.0122	$(1/ksi)^2$										
S_x	196	in^3										
r_y	1.95	in.										
Beam Weight	84	lb/ft										
Recalculate M_u	767.21	ft-k										
Recalculate Z	216.62	in^3										
Is the girder adequate?	**Adequate**											
Local Buckling												
Flange:												
bf/(2tf)	5.90											
Buckling?	No FLB											
Web:												
h/(tw)	45.90											
Buckling?	No WLB											
Is the girder compact?	**Compact**											
Lateral Torsional												
Unbraced Length, L_b	7.92	ft										
L_p	6.89	ft										
L_r	18.63	ft										
ΦM_p critical point	793.33	ft-k										
ΦM_r critical point	555.33	ft-k										
Plastic Region?			ΦM_p	0.00	ft-k							
Inelastic Region?	**Inelastic**		ΦM_n	772.41	ft-k							
Elastic Region?			ΦM_{cr}	0.00	ft-k							
ΦM_n vs. M_u												
Is the beam adequate?	**Adequate**											
Deflection												
Due to P_u	Other			n = 1	$\Delta = (Pa^2b^2)/(3EIL)$			n = 4	$\Delta = (PL^3)/(19.04EI)$			Note: P_u is equal at all locations
Deflection due to P_u	0.601	in		n = 2	$\Delta = (5PL^3)/(648EI)$			n = 5	$\Delta = (PL^3)/(29.716EI)$			Note: The applied loads are evenly spaced in all equations
Deflection due to W_u	0.012	in		n = 3	$\Delta = (3PL^3)/(128EI)$			uneven	$\Delta = (Pa^2b^2)/(3EIL)$			Note: In case n = 1, P_u is not applied in the center
Total Deflection	0.613	in										Note: In case n = 2, P_u is applied at L/3
Δ_{max}	0.862	in		**Manual Calcuated Defl**			0.601	in				
Is the girder adequate?	**Adequate**											

E.4 Axially Loaded Wide Flange Column Design Spreadsheet

Office Design Joist Alternative								
Column Design (20ft Spacing)								
B2 - Basment (w/ K-Brace Design)								
Gravity & Wind Loads								
# floors > column	2	floors	3	floors	1	floors		
Tributary Area	1st/Gym		Office		Living		Roof	
Column TA	440	ft^2	440	ft^2	440	ft^2	440	ft^2
Wall TA		ft^2						
Loads								
Floor Dead Load	79.24	psf (a)	77.84	psf (b)	77.84	psf (c')		
Floor Live Load	100	psf (a)	80	psf (b)	40	psf (c')		
Wind Load	128.88	k (cumulative @ top of column)						
Roof Dead Load	77.84	psf						
Roof Snow Load	35	psf						
Wall Dead Load		psf						
Load Combos								
P_u = (1.4DL)TA	337.37	k						
P_u - (1.2DL)TA + (1.6LL)TA + (0.5SL)TA	634.80	k						
P_u - (1.2DL)TA + (0.5LL)TA + (1.6SL)TA	419.42	k						
P_u - (1.2DL)TA + (1.6SL)TA + (0.8WL)	416.92	k						
P_u - (1.2DL)TA + (1.6WL) + (0.5LL)TA + (0.5SL)TA	608.68	k						
Critical Loading	634.80	k						
Live Load Reduction								
N = 1-0.0008(TA-A_p)	0.73		0.73		0.85			
N = 0.75-0.20(DL/LL)	0.59	(a)	0.56	(b)	0.36	(c')		
N = 0.5 or 0.6	0.60							
Critical LL Reduction	0.73	(a)	0.73	(b)	0.85	(c')		
Revised Load Combos								
P_u = (1.4DL)TA	337.37	k						
P_u - (1.2DL)TA + (1.6LL)TA + (0.5SL)TA	546.26	k						
P_u - (1.2DL)TA + (0.5LL)TA + (1.6SL)TA	391.75	k						
P_u - (1.2DL)TA + (1.6SL)TA + (0.8WL)	416.92	k						
P_u - (1.2DL)TA + (1.6WL) + (0.5LL)TA + (0.5SL)TA	581.02	k						
Critical Loading	581.02	k						
Steel Properties								
F_y	50	ksi						
E	29000	ksi						
ϕ	0.85							
Trial Size								
Trial Size - Table 4-2 AISC	**W14x74**							
A_g	21.8	in^2						
X_1	3290	ksi						
X_2	0.00119	$(1/ksi)^2$						
Z	126	in^3						
S_x	112	in^3						
Column Capacity								
K-value (end condition) X-X	1							
K-value (end condition) Y-Y	1							
Unbraced Column Length X-X	15	ft						
Unbraced Column Length Y-Y	15	ft						
Radius of Gyration X-X	6.04	in						
Radius of Gyration Y-Y	2.48	in						
K_xL_x/r_x	29.80							
K_yL_y/r_y	72.58							
Maximum KL/r	72.58							
Evaluate Capacity								
λ_c	0.96							
Inelastic or Elastic Zone?	**Inelastic**							
F_{cr}	(0.658^(λc^2))*Fy							
F_{cr}	34.02	ksi						
ϕP_n	630.33	k						
Is the beam adequate for axial loads?								
P_u/ϕPn	0.92							
Is the beam adequate?	**Adequate**							

E.5 Reinforced Concrete Slab Design Spreadsheet

Given Information

Live Load?	100	psf
Interior Bay Length?	8.83	LF
End Bay Length?	8.08	LF
Fy?	60000	psi
F'c?	3000	psi

Estimate Thickness of Floor

Required Max Thickness

End Bay: H=	4.04	in
Interior Bay: H=	3.78	in

Trial Thickness

Trial Slab Thickness?	4	in
D (3/4" cover w/ #4 bars)	3	in

Factored Loads

self weight of conc.	50	psf
other dead load?	20	psf
LL	100	psf
Wu=1.4dead+1.7live	268	lb/ft

Note: use a one foot strip

Thickness Required for Moment

1st interior Support

ω	0.2		
φKn	476.28		
Ln	8.455	LF	Note: Ln should be to inside of beam
Mu	1.92	ft-kips/ft	Note: Use max Mu

2nd interior support

ω	0.2
φKn	476.28
Ln	8.83
Mu	1.90

Choose an Mu

Mu	1.92

Compute thickness

bd^2	48.27
B = 12" D =	2.01
Is the thickness adequate	**ADEQUATE**

Thickness Required for Shear

First Interior Support; Vu	103.76	lb/ft of width
Typical Interior Support; Vu	98.60	
φVc	3352.06	lb/ft
Is the thickness adequate	**ADEQUATE**	

Area of Steel Calculations

Lengths

L1 ?	8.08
L2 ?	8.08
L3 ?	8.46
L4 ?	8.83
L5 ?	8.83
L6 ?	8.83

Concrete Properties

fy	60000
f'c	3000

Trial Dimensions

B = ?	12
D =	3
H =	4
Wu ?	268

#4 bar area options	
#4 @15"	0.16
#4 @ 12"	0.2
#4 @ 9"	0.267
#4 @ 8	0.3
#4 @ 6	0.4
#4 @ 4"	0.6

Slab Summary

ln	8.08	8.08	8.46	8.83	8.83	8.83
Wu*ln^2	17.50	17.50	19.18	20.90	20.90	20.90
M coeff	0.04	0.07	0.10	0.06	0.09	0.06
Mu	0.73	1.25	1.92	1.31	1.90	1.31
As req.	0.057	0.097	0.150	0.102	0.148	0.102
As min	0.086	0.086	0.086	0.086	0.086	0.086
rebar ?	#4 @ 12"	#4 @ 12"	#4 @ 12"	#4 @ 12"	#4 @ 12"	#4 @ 12"
area provided ?	0.2	0.2	0.2	0.2	0.2	0.2

Computing As

Mu max (from Slab Summary)	1.92		
As	0.154		
a	0.301	jd	2.849
		j	0.950
As req. coeff.	0.0780	As req =	As req coeff *Mu
As min	0.086		
As temp & shrinkage	0.086	at 12" o/c spacing	
Max spacing	12 in		

E.6 Reinforced Concrete T-Beam Design Spreadsheet

(First Half)

T-Beam Design

Dimensions and Factored Loadings

live load	*100*	*psf*	*ext. length*	*20*	*ft*	*fy*	*60000*	*psi*
dead load	*20*	*psf*	*int length*	*25*	*ft*	*f'c*	*3000*	*psi*
slab thickness	*4*	*inches*	*ext. width*	*10*	*ft*			
			int width	*10*	*ft*			

Influence Area A

length	19	ft	L	104	psf
ext. width	9	ft			
int. width	9	ft			
beam width	1	ft			
Aa	361	ft^2			

Influence Area B

Length 1	19	ft	L	77	psf
Length 2	24	ft			
ext. width	9	ft			
int. width	9	ft			
beam width	1	ft			
Ab	836	ft^2			

Influence Area C

length	24	ft	L	95	psf
ext. width	9	ft			
int. width	9	ft			
beam width	1	ft			
Ac	456	ft^2			

Stem size will be chosen based on influene area B

Loadings

slab DL	50	psf
DL	20	psf
Reduced LL	77	psf
factored loading	229	psf
tributary width	10	ft

factored loading per foot of slab 2.29 k/ft

Estimate weight of Beam Stem

W1	0.229	k/ft
W2	0.457	k/ft

Using B, H and PCF

H1	2	B1	1	W3	0.300	k/ft
H2	2.5	B2	1.25	W4	0.469	k/ft

Select a trial weight in the range given by W1 - W4

Wbeam	0.400	k/ft	trial loading	2.69	k/ft

Compute Actual Size of Beam

Minimum height	16.22 in		
based on Mu 1st int. support		136.03	k-ft
ρ	0.013		
ω	0.26		
φKn	594.3132		
bd^2	2746.554484 in^3		

options	b	d	h
	10.00	16.57	19.07
	12.00	15.13	17.63
	14.00	14.01	16.51

try

b	12.00	in
d	15.50	in
h	18.00	in

Check Shear Capacity

Max Vu	38.62 k			
min Bwd	82.96 in^2			
for b =	12.00 in		d1 =	6.91
if d > d1 the OK	GOOD			

TRIAL SUMMARY

b	12.00	in
d	15.50	in
h	18.00	in
wu	2.69	k/ft

Compute dead Load of Stem and recompute total Load Per Foot

wt. per foot of beam	0.225	k/ft
new wu	2.51	k/ft

Calculate Flange Width at + Moment Regions

choose smallest	60	in
	108	in
	132	in
flange width	60	in

Compute Moments

L1	20	LL1	1.77	k/ft
L2	20	LL2	1.31	k/ft
L3	22.5	LL3	1.62	k/ft
L4	25	trib area1	10	ft
L5	22.5	DL	1.30	k/ft
L6	20			
L7	20			

Moment Calculation Table

ln	20	20	22.5	25	22.5	20	20
wu	3.06	3.06	2.60	2.91	2.60	3.06	3.06
wu*ln^2	1225	1225	1317	1821	1317	1225	1225
Cm	0.04	0.07	0.10	0.06	0.10	0.07	0.04
Mu	-51.04	87.49	-131.72	113.83	-131.72	87.49	-51.04

(Second half)

Design Reinforcement

Point of Max - Moment

Neg. Max Moment	132	kft	
As	2.16	in^2	
a	4.24	in	
a/d = a/dt	0.274		
ab/d	0.503		fs=fy
atcl/dt	0.319		phi = .90
recompute As	0.0166	MU	

Point of Max + Moment

Pos. Max Moment	114	kft	
As	1.72	in^2	
a	0.675		
a/d = a/dt	0.044		
ab/d	0.503		fs=fy
atcl/dt	0.319		phi = .90
recompute As	0.0147	MU	

Compute As Min

Asmin	0.62
As>Asreq?	YES

Bar Properties

Bar #	diameter	in^2
# 3	0.375	0.11
# 4	0.500	0.20
# 5	0.625	0.31
# 6	0.750	0.44
# 7	0.875	0.60
# 8	1.000	0.79
# 9	1.128	1.00
# 10	1.270	1.27
# 11	1.410	1.56
# 14	1.693	2.25
# 18	2.257	4.00

Calculate As and Select Bars

As Selection

Mu	-51.04	87.49	-131.72	113.83	-131.72	87.49	-51.04
As Coeff.	0.0166	0.0147	0.0166	0.0147	0.0166	0.0147	0.0166
As required	0.85	1.28	2.19	1.67	2.19	1.28	0.85
As > As min?	YES	YES	YES	YES	YES	YES	YES
bars	2 #6	3 #6	4 #7	4 #6	4 #7	3 #6	2 #6
As provided	0.88	1.32	2.4	1.76	2.4	1.32	0.88
bw Okay?		YES		Yes		YES	

Reinforcement Distribution

Positive Moment Region

smallest bar #	#6	# of bars	3
As ?	1.32	in^2	
diameter?	0.75		
dc	2.25	in	
A	18	in^2/bar	
z	124		
Okay?	OKAY		

Negative Moment Distribution

distributive length	24	in	
diameter of bars	0.875		
type of bars	#7	# of bars	4
dc	2.31	in	
Max A	10.22		
s	2.21	in	

Shear Reinforcement

ln		20			25	
wu		3.06			2.91	
wl		1.77			1.62	
Cv	1.00	0.15	1.15	1.00	0.15	1.00
wuln/2		30.62			36.43	
Vu	30.62	4.59	35.21	36.43	5.46	36.43
Vn	36.02	5.40	41.43	42.86	6.43	42.86

Exterior of B3

d	15.5	in	from support	Vc	20.3753 kips
Vn	32.07			Vc/2	10.1876 kips
stirrups?	YES				
Max Spacing	14.67	or	7.75		
Max Spacing	7.75				
try	6	in			
# 0f stirrups	20				

Interior end of B3

d	15.50	in		
Vn	36.78			
s	5.13	in o/c	when s =	6
try	4	in o/c	Vn	32.92 kips
use			x	28.34 in

# of stirrups	Spacing		
16	6	in o/c from middle	96 in from middle
6	4	in o/c from end	24 in from end

End Of B4

d	15.50	in		
Vn	39.09	k		
s	4.72	in	when s =	6
try	4	in o/c	Vn	32.921
# of stirrups	Spacing		x	40.9072

19	6	in o/c from middle	114 in from middle
9	4	in o/c from end	36 in from end

Final Summary

b	12.00 in	
d	15.50 in	
h	18.00 in	
rebar		length
Exterior neg bar	2 #6	6.67 ft
1st Int Pos. bar	3 #6	20.00 ft
1st Int Neg. bar	4 #7	8.33 ft
2nd Int Pos. bar	4 #6	25.00 ft

See Directly Above for Stirrup Spacing

E.7 Reinforced Concrete Girder Design Spreadsheet

Girder Design Spreadsheet

20 x 20 Column Spacing **Beam Option 1 - Floors 1-6**

General Information

1st interor bay length	20	ft	concrete wt.	150	pcf
2nd interior bay length	20	ft	fy	60000	psi
Exterior tributary length (long)	10	ft	f'c	3000	psi
Interior tributary length (long)	12.5	ft	dead load	20	psf
Exterior tributary length (short)	5	ft	live load	100	psf
Interior tributary length (short)	10	ft	snow load	0	psf
slab thickness	4	in			

Beam dimensions

type	b	h	
B1	10	18	in
B2	10	18	in
B3	10	18	in
B4	12	18	in
B5	12	18	in
B6	12	18	in

Point loads

from factored loadings

Ple	22.275	kips
Pli	44.550	kips

from slab

Pse	5.625	kips
Psi	11.250	kips

from beam (per beam)

P1	1.875	kips
P2	2.344	kips
P3	1.875	kips
P4	2.813	kips
P5	2.250	kips
P6	2.813	kips

Combined Loading

PA	32.119	kips
PB	60.488	kips
PC	60.863	kips

Choose the largest point load when calculating the self weight of beam

Pmax 60.863

Self Weight of Girder Calculation

10% to 20% of load on span

W1	6.086	kips
W2	12.173	kips

Using span length note: use longest span

length 20 ft

h1	1.600 ft	b1	0.8 ft	W3	3.840	kips
h2	2.000 ft	b2	1 ft	W4	6.000	kips

choose a self-weight from either W1, W2, W3 or W4

Pg 7.50 kips

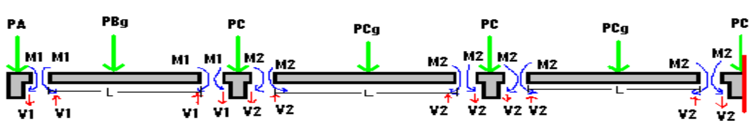
Finalized Point Loads

PA	32.119
PBg	67.988
PC	60.863
PCg	68.363

Forces on Girder and Adjoining Columns

M1 = PBg*L/4
V1 = PBg/2

M2 = PCg*L/4
V2 = PCg/2



M1	169.969	kip-feet	M2	170.906	kip-feet
V1	33.994	kips	V2	34.181	kips
Ms	169.969		Ms	170.906	

Mu Max 170.906

Actual Girder Dimensions

bd^2	4306.021668	φKn	476.28	ω	0.2

Size options	b	d	h	
	12.00	18.94	22.44	in
	14.00	17.54	21.04	in
	16.00	16.41	19.91	in
	18.00	15.47	17.97	in

Choose a b and an h

b	14.000
h	22.000
d+	18.500
d-	19.500

Area of Steel Calculations

1st Interior Span			***2nd Interior Span***		
As -	2.039		**As-**	2.050	
steel options			*steel options*		
	6 #6 bars	2.64 in^2		6 #6 bars	2.64 in^2
	3 #8 bars	2.37 in^2		3 #8 bars	2.37 in^2
	4 #7 bars	2.4 in^2		4 #7 bars	2.4 in^2
try	3 #8 bars		try	3 #8 bars	
As provided	2.370 in^2		As provided	2.370 in^2	
As+	2.149		**As+**	2.161	
steel options			*steel options*		
	4 #7 bars	2.4 in^2		4 #7 bars	2.4 in^2
	3 #8 bars	2.37 in^2		3 #8 bars	2.37 in^2
	5 #6 bars	2.2 in^2		5 #6 bars	2.2 in^2
try	4 #7 bars		try	4 #7 bars	
As provided	2.400 in^2		As provided	2.400 in^2	

1st Interior support		***2nd Interior Support***	
length of - bars	6.667 ft	length of - bars	6.667 ft
length of + bars	15.000 ft	length of + bars	15.000 ft

E.8 Reinforced Concrete Column Design Spreadsheet

Column Design

Columns Between Floors 2 and 3

Notes: You will need to use the forces you found in calculating the Girder dimensions
In this case the values have been tabulated on the Girder Summary worksheet for G2.
Additionally, you will need tables A-6 and A-7 from McGregor

Design of C3 Column

Forces Acting on Column

P1	358.73	K	P1 = PCroof +2* V2roof+2*(Pcfloor+2*V2floor)
P2	358.73	K	P2 = P1
M1	171.38	K-ft	M1 = M2
M2	171.38	K-ft	M2 = M2floor

Select trial properties, trial size and trial reinforcement

fy	60000	psi	Pu	358.73	K	
f'c	3000	psi	Mu	171.38	K-ft	
height	12	ft	min girder	12	in	Pick the largest value of P1 and P2 for Pu and M1 and M2 for Mu

Gross Area Calculation

Ag	204.405	in^2	or	14.297	by same

Trial Column b and h

b	16	in	Do not use less than a 12"x12" column just to be safe.
h	16	in	

Compute e

e	0.478	for 0.2>e/h>0.1 use reinforcing in all 4 sides, for e/h>0.2 use reinforcing in two sides. For e/h <0.1 use cylindrical column.
e/h	0.358	

use reinforcing in two sides

Slenderness

klu/r	27.50	in
34-12*(M1/M2)	46.00	
Slenderness ?	Okay	

Trial Design Summary

b	16	in
h	16	in
fy	60000	psi
f'c	3000	psi
rebar	2 faces	

Compute γ

trial rebar ϕ	1.000
rebar type	#8
trial tie ϕ	0.375
tie type	# 3
γ	0.703

Pu/Ag	1.40
Mu/Agh	0.04

Use tables A-6 and A-7 to find pt

from A-6	pt	0.006	pt must be at least .01 so if values from tables are less than .01 use .01 for pt
from A-7	pt	0.006	
	pt	0.01	

Select Reinforcement

Ast		2.56	in^2
Options	4 #6	1.76	in^2
	4 #7	2.4	in^2
	4 #8	3.16	in^2
try	4 #8	3.16	in^2

Check Minimum load Capacity

ϕPn	638.272	kips
okay?	YES	

Design Lap Splices

Ld	54.77	in	Assume all bars spliced at same location. This means splice length = 1.3Ld
1.3Ld	71.20	in	
try	54	in	

Select Tie Spacing

s1	16	in			
s2	18	in	Choose the smallest of the s's to be your max. tie spacing		
s3	16	in	Suse	16	in

ties at	8	in	for 1st	3	ft
ties at	16	in	for middle	6	ft
ties at	8	in	for last	3	ft

F Lateral Load Calculations

F.1 Lateral Considerations for the Reinforced Concrete Structures

The nature of a reinforced concrete structure is such that the frame accounts for the moments created in the members throughout the structure, and these forces are transmitted down through the load path of the building. This means that when lateral loadings are applied to a structure designed for gravity loads, the member sizes may need to be recomputed, but a lateral bracing system does not need to be implemented.

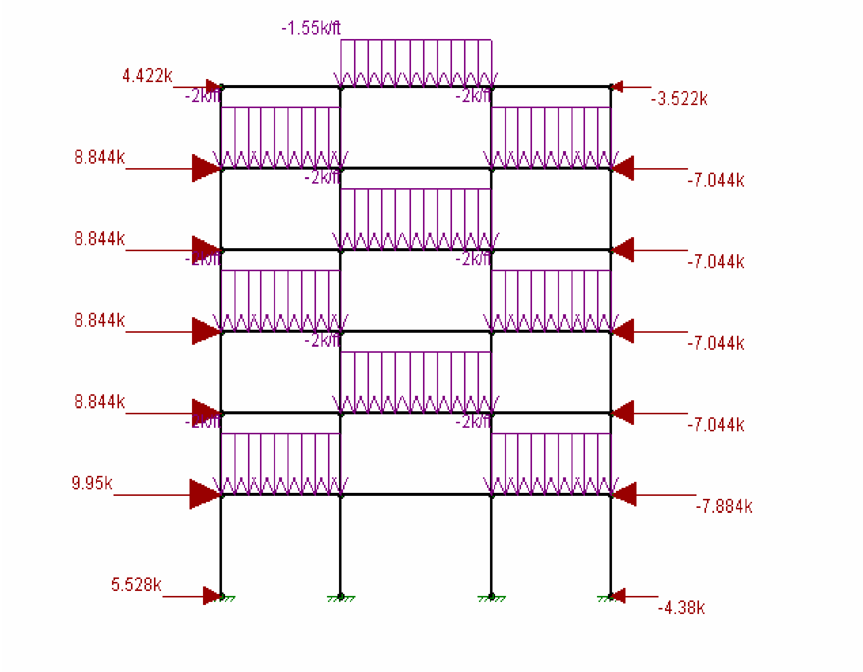
Two major lateral forces affecting buildings are those associated with wind pressures and seismic forces. The first force that was computed was that associated with wind. This is typically the force that governs in this section of the United States. This is because the characteristics of an earth quake that would hypothetically occur in this region are less of a concern in terms of force than that of a severe wind loading being applied to the building.

F.1.1 Office Wind Pressure Loading Calculations

In order to determine the forces acting on the members from wind, the amount of pressure applied to the largest sections of the windward and leeward sides of the building was determined for both the one hundred foot length of the building and the sixty-five foot length. The methods for obtaining the wind loading on both faces of the building are discussed in Section 3.6.1. Once the pressure in pounds per square foot of force had been determined for the windward and leeward sides of the building, the area on which the pressure would be acting was determined. This involved looking at the column spacing on both major faces of the structure and calculating the maximum length of the building on which the pressure would be acting. This length was twenty feet for the force acting on the short side of the building and twenty-two and a half feet for the force acting on the long side of the building.

After the force acting along each face of the building was determined that force was subdivided along each vertical section of the building to which it would be applied. This meant that the height between columns was halved and the amount of pressure applied above and below the head or foot of the column was attributed to that particular column. For example, the base of the first floor column had a force applied to it equal to that of the distributed load placed on the building from the wind pressure multiplied by seven and a half feet, which was half the height of the column above that particular point.

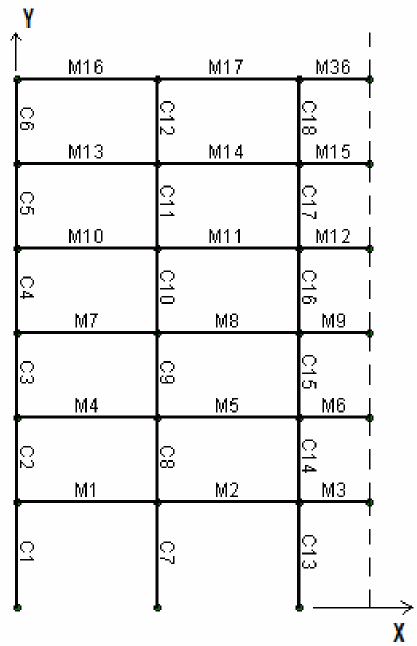
The forces acting along the short side of the building were computed in this manner. In addition to the wind pressure being applied, the worst case floor loading for each floor was incorporated at the same time to determine the worst case member forces for this section of the building. All of the force calculations done for this analysis were completed using RISA2D. The following diagram shows how the combined windward and leeward loadings for this scenario were defined for analysis. In addition this figure shows the worst case gravity load distribution as a distributed load on the structure alternating by floor and bay. This type of loading distribution is worst case because it creates forces in the columns and girders that are not counteracted by the forces acting through the adjacent bay.



Appendix Figure F.1.1 Short Side RISA2D with Windward and Leaward Loading

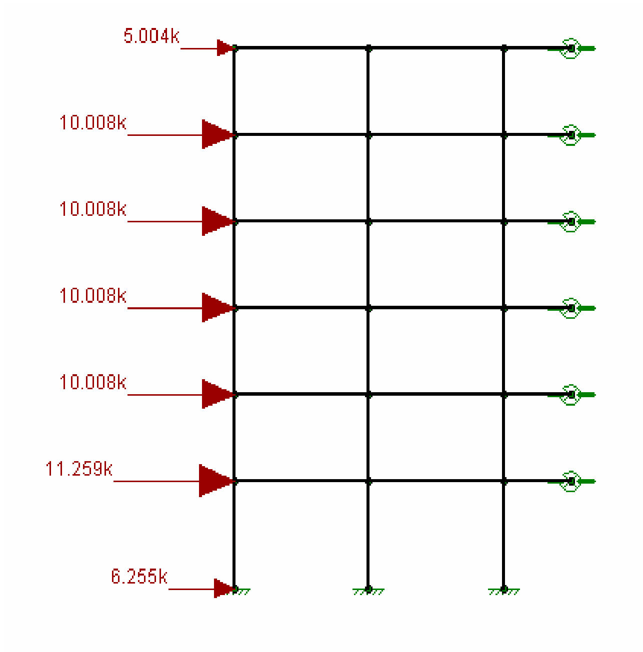
The wind pressure forces were resolved into point forces acting where the columns met the T-beams along these faces of the building. This scenario was analyzed using RISA2D, the member forces for the columns and beams were extracted, and the critical design shears, moments and axial loads for each member were identified. In addition to investigating a simultaneous application of the leeward and windward pressure forces, both forces were applied independently. Once each of these scenarios had been analyzed, the floor loadings were reversed such that the opposite checkerboard effect was created and the three scenarios were run again using this gravity and wind pressure force combination on the frame.

To compute the forces acting upon the short side of the building, thus the long side of the frame, a different method had to be employed. This frame consisted of too many members to be supported by the demonstration version of RISA2D. In light of this, symmetry of the structure can be used to determine the resulting member forces. The figure below shows how the elements of the office structure were set up to facilitate computation.



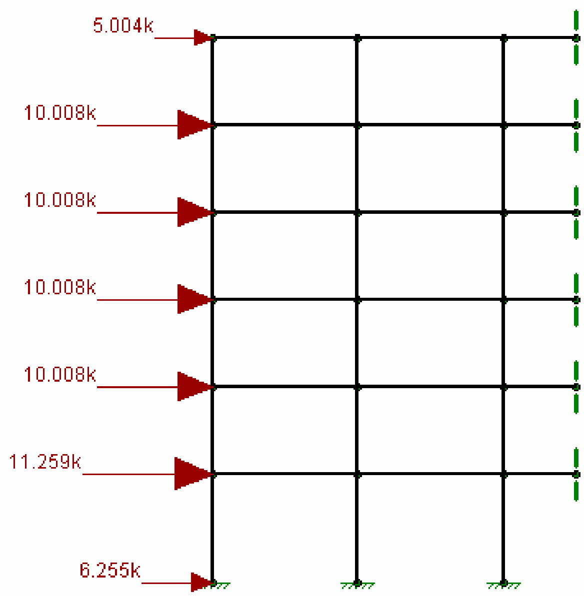
Appendix Figure F.1.2 Long Side Member RISA2D Setup

From this basic set up two different trials were run. The first investigation involved a symmetrical loading of the structure. This meant that the nodes at the end of members M3, M6, M9, M12, M15 and M36 were braced along the X axis and against rotation while movement in the Y axis was permitted. This loading was run for windward pressures because it was the larger of the two pressures, and the values of the pressure forces along the X cancelled out when summed so the greatest effect on the structure would have been from that of the windward forces. Thus a calculation of the leeward wind pressure effects was not run to expedite the lateral analysis and design process. It should be noted that the forces for the windward pressure forces for this side of the building were calculated in an identical method to those acting on the short side of the structure. An example of the frame being loaded for symmetrical windward pressure analysis is depicted below. Note the boundary indicators on the right most members.



Appendix Figure F.1.3 Symmetrically Loaded Long Side RISA2D Example

A second state was investigated for the forces acting on that short side of the building. This state consisted of forces acting in an unsymmetrical manner. In this scenario, the nodes at the ends of members M3, M6, M9, M12, M15 and M36 were braced in the Y direction only, allowing for movement in the X direction and also for rotation. The wind pressure loadings for this scenario were run only for windward pressures with the logic being that the pressure resulting from the windward force was the largest and would lead to the greatest reaction forces in the members. The figure below depicts the loading of the frame in the unsymmetrical state. Note the how the boundary conditions for the rightmost end members have changed from the previous scenario.



Appendix Figure F.1.4 Symmetrically Loaded Long Side RISA2D Example

In both the symmetrical and unsymmetrical loading situations, the frame was loaded in a checkered pattern to mimic the worst case loading possibilities. There were two ways in which this could be done. The first was to apply the loadings applied to the beams and slab along with the dead weight of those members to members M1 and M3 then alternate loadings on the upper floors as shown in both figures above. The second option was to apply the loadings to member M2 and alternate. Both of these options were considered in the analysis of the structures and data was obtained for each loading case.

The outputs of the member forces from all of the RISA scenarios were put into a spreadsheet and analyzed in terms of worst case forces to act on members. The first step in this process was to determine the worst case loadings on the T-beams. This meant evaluating the outputs for the short side loading of the structure for worst case moment and shear forces. The basic premise behind this analysis was to determine the worst case forces at the beginning, middle and end of each member for all of the conditions it could be subjected to. In doing this the forces for the members were selected not as a grouping but rather as simply the highest that the member could experience at any point in time. The reasoning behind this method was based in the idea that the structure could encounter any of the types of forces that it was subjected to in the RISA scenarios. For example, the moment on the end of a beam was greatest when M1/M3 were loaded with the windward pressure force. Yet, the shear and axial forces were greatest when the beam was subjected to loading in the form of M2 being loaded in the combined state of windward and leeward. The maximum moment force for the beam would be taken from the M1/M3 windward loading and the maximum shear and axial forces would be taken from the M2 combined loading. This rationale was applied to all members.

Once the maximum forces the T-beams would be subject to were determined, these forces were compared to the forces resulting from the gravity loads applied to the members. If the forces resulting from the wind and gravity load combination were smaller or equal to those designed for in the gravity loading the member was left unchanged. If the forces were larger, the beams were analyzed to determine first if the sizing of the beam needed to change and secondly if the reinforcement needed to be recalculated. This process was done by using the design spreadsheets from the gravity loading design. The spreadsheets were easily manipulated to facilitate this process by bypassing the input section for dead and live loads and simply typing in the moments and shears found in the wind loading design. The same process was executed for the girders and column on both sides of the building.

F.1.2 Seismic Reaction Calculations

Once the new members had been designed, the total weight of the structure was calculated so that a comparison could be made between the seismic forces on the structure and the wind forces. The calculation of seismic forces was done as specified in *ASCE 07-02*. Essentially, certain characteristics of site, such as soil properties and accelerations due to potential seismic activity, and the height of the building were used to form a coefficient that relates the weight of the building to the forces it would encounter during a seismic event such as an earthquake. The resulting force was a shear force acting on the building. This force was then compared to the wind pressure force acting along the long side of the building. Had the seismic force been greater than that of the wind pressure force, the building would need to be evaluated in terms of the seismic forces acting upon it. Hand calculations for the comparison of seismic loading to wind pressure forces can be found in Appendix F.5.

F.2 Wind Load Calculations – Office

Location Description:

120 acre parcel of land
surrounding land - mostly flatland w/ some partial tree cover
soil type: medium silty-sand w/ the occasional glacially deposited boulder
current ground coverage: grass and low growing brush w/ a few areas of small tree growth

Source: American Society of Civil Engineers
Minimum Design Loads for Buildings and Other Structures
ASCE 7-98 revision of ANSI/ASCE 7-95

- Basic Wind Speed, V
  $V = 100\,mph$ (Fig 6-1)

- Wind Directional Factor, K_d
  $K_d = 0.85$ for buildings: main wind force resisting system (Table 6-6)

- Importance Factor, I
  $I = 1.15$ for category III (includes jails and detention centers) located in non-hurricane prone areas (Table 1-1 and 6-1)

- Exposure Category
  Exposure C – includes flat open country

- Velocity Pressure Exposure Coefficient, K_z or K_h
  $K_z = 1.19$ (Table 6-5)

- Topographic Factor, K_{zt}
  $K_{zt} = 1.0$ (Fig 6-2)

- Gust Effect Factor, G
  - natural frequency, $n = 1/T$ where $T = 0.05H/\sqrt{D}$
    $n = 2.13\,Hz$ $\quad T = 0.469$ for $D = 64$
    $n = 2.67\,Hz$ $\quad T = 0.375$ for $D = 100$
  - building is a rigid structure $n > 1\,Hz$

$$G = 0.925\left[\frac{(1 + 1.7 g_Q I_{\bar{z}} Q)}{1 + 1.7 g_v I_{\bar{z}}}\right]$$

* "g_Q and g_v shall be taken as 3.4"

* B = horizontal dim of building normal to wind direction

- $I_{\bar{z}} = c(33/\bar{z})^{1/6}$ $\quad c = 0.20$, $\bar{z} = 0.6h > z_{min}$ $= \underline{45\,ft} > 15\,ft$ } Table 6.4

$= (0.2)(33/45)^{1/6}$

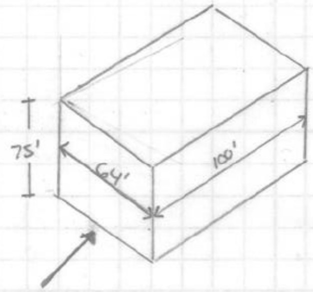
$I_{\bar{z}} = 0.1899$ ← intensity turbulence

- $Q = \sqrt{\dfrac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$

- $L_{\bar{z}} = l(\bar{z}/33)^{\bar{\epsilon}}$ $\quad l = 500\,ft$, $\bar{\epsilon} = 1/5.0$, $\bar{z} = 45\,ft$ } Table 6.4

$= (500)(45/33)^{1/5.0}$

$L_{\bar{z}} = 531.998$ ← integral length scale of turb.



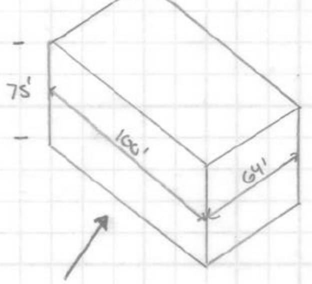
• H = 75 ft
• B = 64 ft

$$= \sqrt{\frac{1}{1+0.63\left(\frac{64+75}{531.998}\right)^{0.63}}}$$

Q = 0.887 ← Background response

$$G = 0.925\left[\frac{1+1.7(3.4)(0.1899)(0.887)}{1+1.7(3.4)(0.1899)}\right]$$

G = 0.87 for wind normal to B = 64 ft



$$\bullet\ Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar z}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1+0.63\left(\frac{100+75}{531.998}\right)^{0.63}}}$$

Q = 0.873

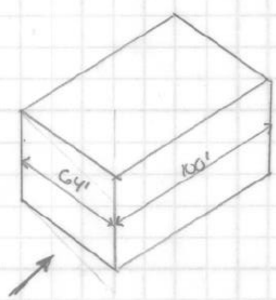
$L_{\bar z}$ = 531.998
B = 100 ft
h = 75 ft

$$\bullet\ G = 0.925\left[\frac{1+1.7 g_Q I_{\bar z} Q}{1+1.7 g_v I_{\bar z}}\right]$$

$$= 0.925\left[\frac{1+1.7(3.4)(0.1899)(0.873)}{1+1.7(3.4)(0.1899)}\right]$$

G = 0.72 for wind normal to B = 100 ft

• Velocity Pressure (B = 64 ft), q_z



• $q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256(1.19)(1.0)(0.85)(100)^2(1.15)$

q_z = 29.78 psf

• Design Wind Pressure (B = 64 ft), p

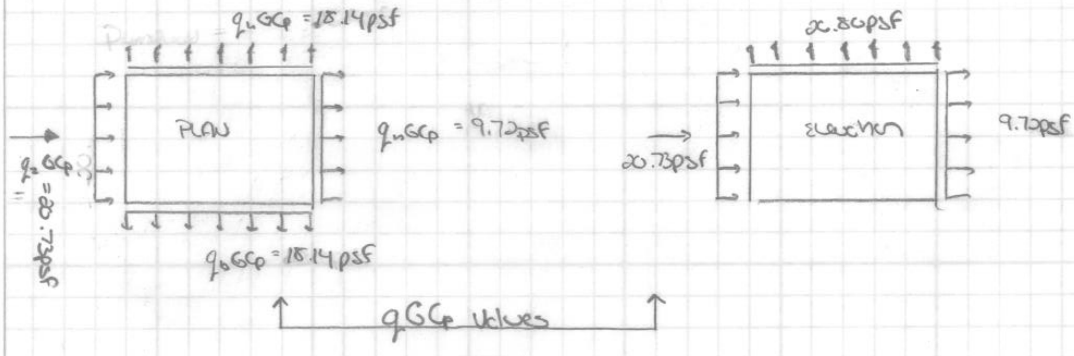
$p = qGC_p - q_i(GC_{pi})$

$q = q_z$ for windward side
$q_i = q_h$ @ height h (= 75 ft) = q_z
$GC_{pi} = \pm 0.55$ for partially enclosed
G = 0.87
C_p = ?

C_p values:

Walls =	windward	0.8	C_p – Fig. 6-3
	leeward	-0.325	
	side	-0.7	
Roof = (θ < 10°)		-0.9	– GC_p – Fig 6-8

• For wind direction normal to $B = 64ft$



• $P_{windward} = q(GC_p) - q_i(GC_{pi})$

$= 20.73psf - 29.78psf(\mp 0.55)$

$P_{windward} = 4.35psf\ ;\ 37.11psf$

• $P_{leeward} = 9.72psf - 29.78psf(\pm 0.55)$

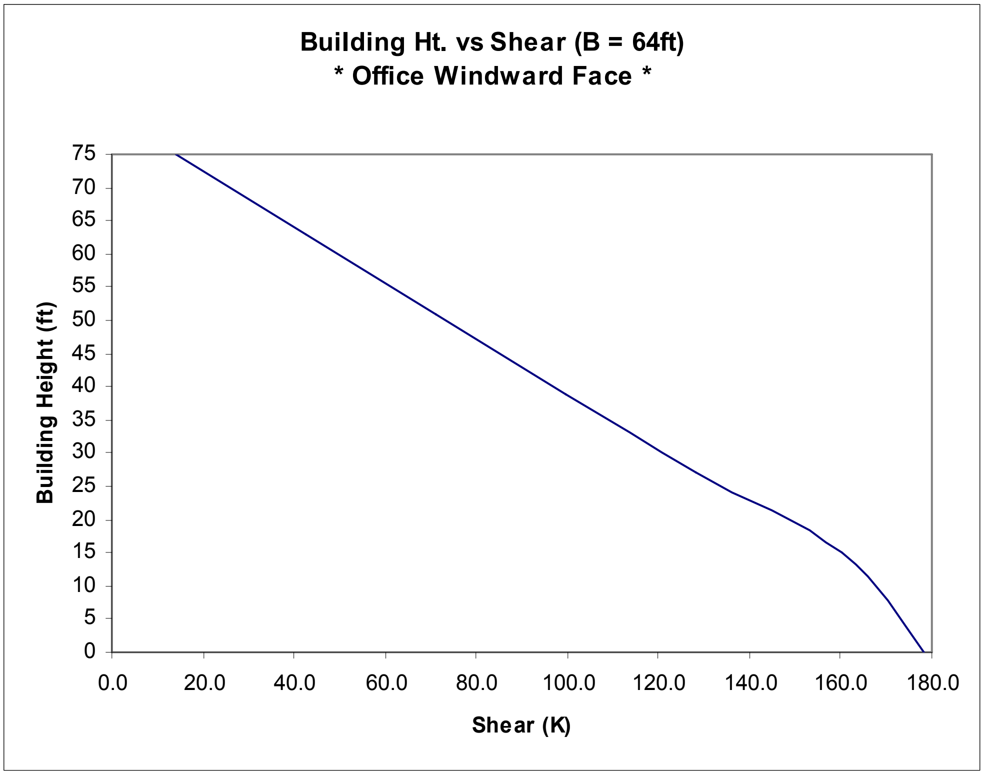
$= -6.66psf\ ;\ 26.10psf$

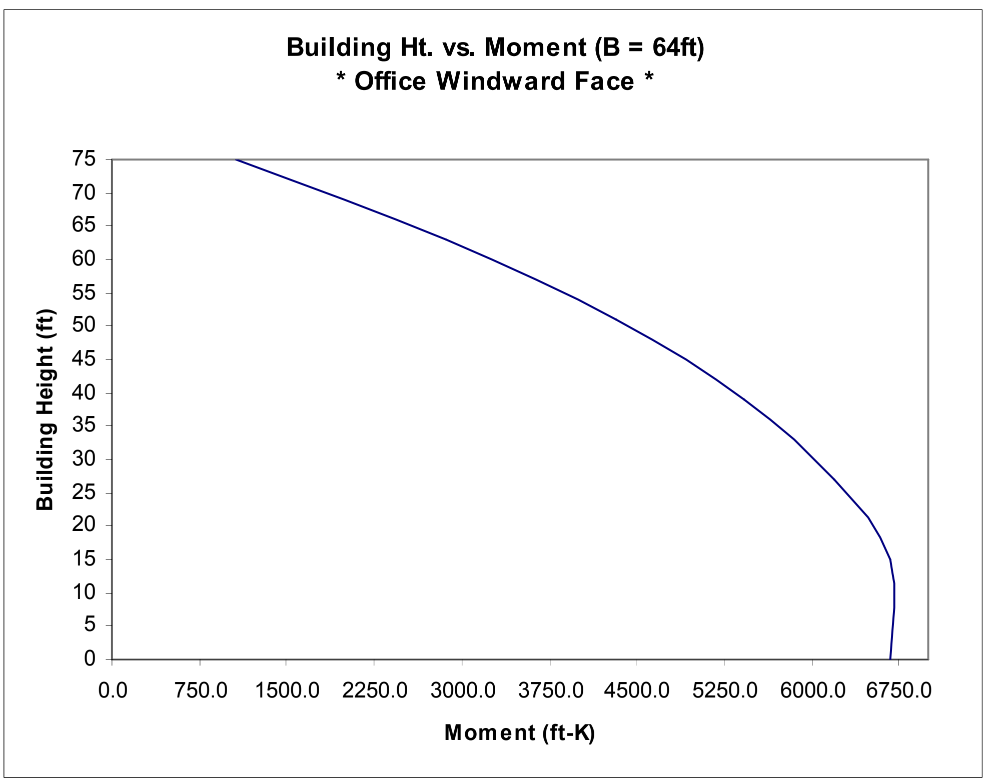
$P_{sidewall} = 18.14psf - 29.78psf(\pm 0.55)$

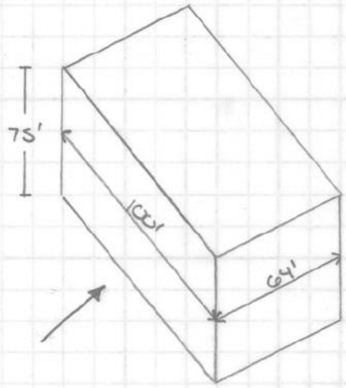
$= 1.76psf\ ;\ 34.52psf$

• $P_{roof} = 26.8psf - 29.78(\pm 0.55)$

$= 10.42psf\ ;\ 43.18psf$


Building Ht. vs Shear (B = 64ft)
* Office Windward Face *
Building Height (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
0.0
20.0
40.0
60.0
80.0
100.0
120.0
140.0
160.0
180.0
Shear (K)


Building Ht. vs. Moment (B = 64ft)
* Office Windward Face *
Building Height (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
0.0
750.0
1500.0
2250.0
3000.0
3750.0
4500.0
5250.0
6000.0
6750.0
Moment (ft-K)



· Gust Factor, G

$$G = 0.925\left[\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right]$$

$I_{\bar{z}} = c(33/\bar{z})^{1/6}$

$c = 0.2$
$\bar{z} = 0.6h > z_{min}$
$= \underline{45ft} > 15ft$
Table 6.4

$I_{\bar{z}} = 0.1899$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}}$ $\ell = 500ft$, $\bar{z} = 45ft$, $\bar{\epsilon} = 1/5.0$

$L_{\bar{z}} = 531.998$ ft

$$= \sqrt{\frac{1}{1+0.63\left(\frac{100+75}{531.998}\right)^{0.63}}}$$

$Q = 0.873$

$$G = 0.925\left[\frac{1+1.7(3.4)(0.1899)(0.873)}{1+1.7(3.4)(0.1899)}\right]$$

$G = 0.86$

· Velocity Pressure (B = 100ft), q_z

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256(1.19)(1.0)(0.85)(100)^2(1.15)$

$q_z = 29.78$ psf

· Design Wind Pressure (B = 100ft), p

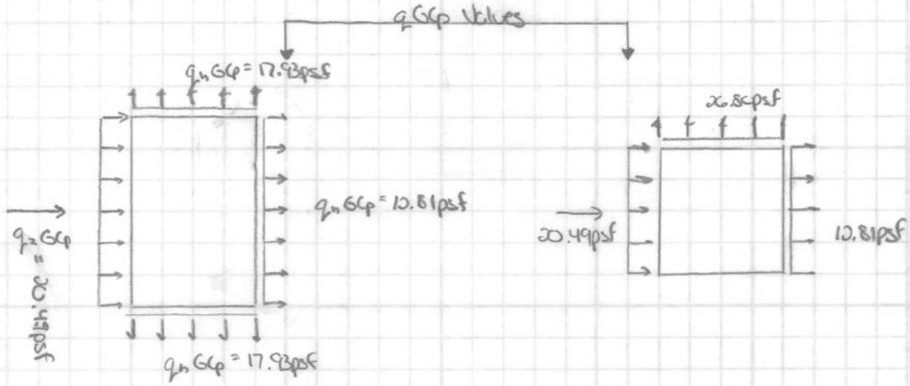
$p = q(GC_p) - q_i(GC_{pi})$

$q = q_z$ for windward side
$q_i = q_h$ @ height h (= 75ft) = q_z
$GC_{pi} = \pm 0.55$ for partially enclosed
$G = 0.86$
$C_p = ?$

C_p Values:

Walls =	windward	0.8
	Leeward	-0.5
	side	-0.7
Roof =	$(\theta < 10°)$	-0.9

s/



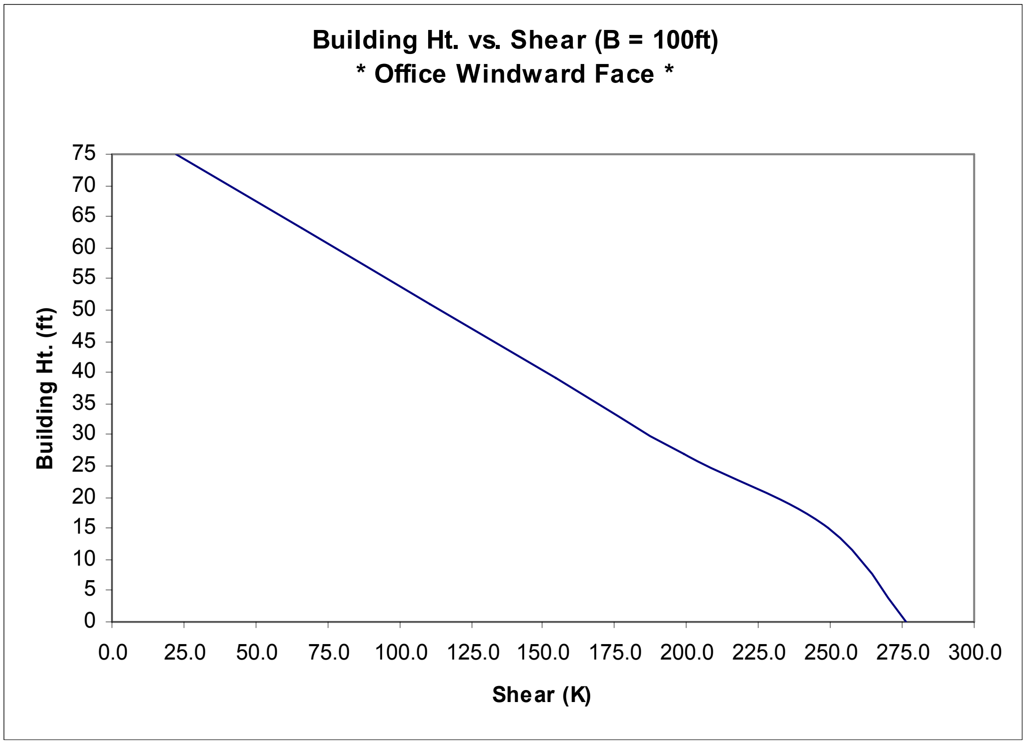
· $p_{windward} = q_z(GC_p) - q_i(GC_{pi})$
$= 20.49psf - 29.78psf\ (\pm 0.55)$

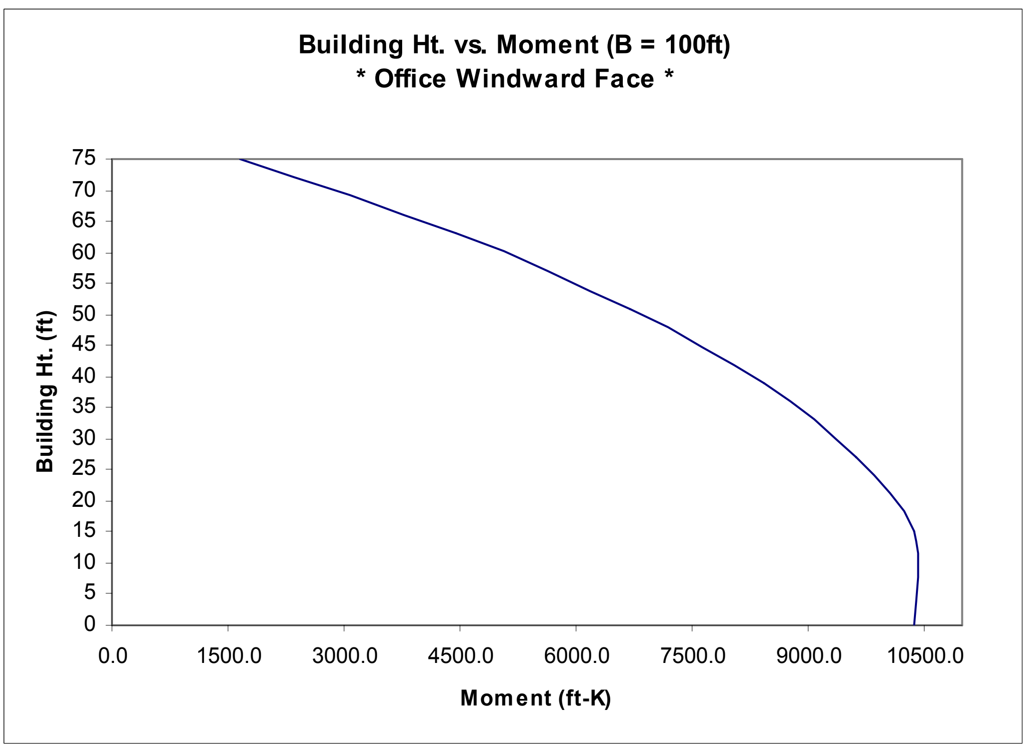
$p_{windward}$ = 4.11psf ; 36.87psf

· $p_{leeward}$ = 12.81 − 29.78 (±0.55)
= −3.57psf ; 29.19psf

· p_{side} = 17.93 − 29.78 (±0.55)
= 1.55psf ; 34.31psf

· p_{roof} = 26.80psf − 29.78 (±0.55)
= 10.42psf ; 43.18psf


Building Ht. vs. Shear (B = 100ft)
* Office Windward Face *
Building Ht. (ft)
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0.0
25.0
50.0
75.0
100.0
125.0
150.0
175.0
200.0
225.0
250.0
275.0
300.0
Shear (K)


Building Ht. vs. Moment (B = 100ft)
* Office Windward Face *
Building Ht. (ft)
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
0.0
1500.0
3000.0
4500.0
6000.0
7500.0
9000.0
10500.0
Moment (ft-K)

F.3 Wind Load Calculations – Prison Superstructure

PRISON CELL BLOCK — WIND LOAD CALCULATIONS — 11/04/05 TA

· Basic wind speed, V:
$V = 100$ mph (Fig 6-1)

· Wind Directional Factor, K_d
$K_d = 0.85$ for building: main wind force resisting system (Table 6-6)

· Importance Factor, I
$I = 1.15$ for category III includes jails and detention centers located in non-hurricane prone areas (Table 1-1 and 6-1)

· Exposure Factor
Exposure C - includes flat open country

· Velocity Pressure Coefficient, K_z or K_h
$K_z = 1.08$ (Table 6-3)

· Topographic Factor, K_{zt}
$K_{zt} = 1.0$ (Fig 6-2)

· Gust Effect Factor, G

· natural frequency; $n_o = 1/T$ where $T = 0.05H/\sqrt{D}$
$= 0.05(48)/\sqrt{184.41}$
$T = 0.18$

$n_o = 1/0.18$
$n_o = 5.66 > 1 Hz \rightarrow$ rigid struct.

$$G = 0.925\left[\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right]$$

· $I_{\bar{z}} = c(33/\bar{z})^{1/6}$ — $c = 0.20$; $\bar{z} = 0.6h > z_{min}$ $= \underline{28.8ft} > 15ft$ (Table 6.4)

$= 0.2(33/28.8)^{1/6}$
$I_{\bar{z}} = 0.205$ ← intensity turbulence

$$\cdot Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

· $L_{\bar{z}} = l(\bar{z}/33)^{\bar{\epsilon}}$ — $l = 500ft$; $\bar{z} = 28.8ft$; $\bar{\epsilon} = 1/5.0$ (Table 6.4)

$= (500)(28.8/33)^{1/5.0}$
$L_{\bar{z}} = 486.57$ ← internal length scale of turb.

* g_Q and g_v taken as 3.4

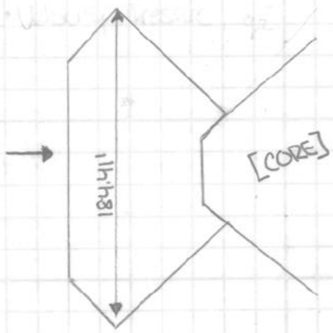
* B = hor. dim. of building normal to wind direction

$$= \sqrt{\frac{1}{1+0.63\left(\frac{184.41+48}{486.57}\right)^{0.63}}}$$

$Q = 0.85$

$$G = 0.925\left[\frac{1+1.7(3.4)(0.205)(0.85)}{1+1.7(3.4)(0.205)}\right]$$

$G = 0.85$ for wind normal to $B = 184.41ft$



- Velocity Pressure, q_z

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$

$= 0.00256(1.08)(1.0)(0.85)(100mph)^2(1.15)$

$q_z = 27.03$ psf

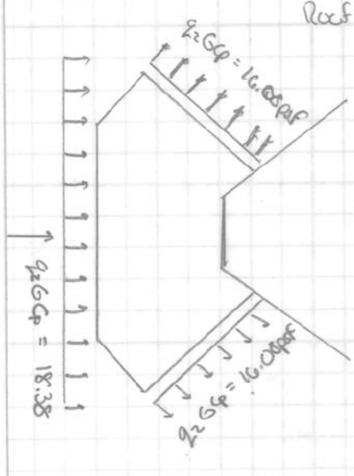
- Design Wind Pressure, p

$p = q(GC_p) - q_i(GC_{pi})$

$q = q_z$ for windward side
$q_i = q_h$ @ height h (=48ft) $= q_z$
$GC_{pi} = \pm 0.55$ for partially enclosed
$G = 0.85$
$C_p = $ ↓

Cp Values:

Walls =	windward	0.8
	leeward	–
	sides	-0.7
Roof =	(θ ≤ 10°)	-0.9



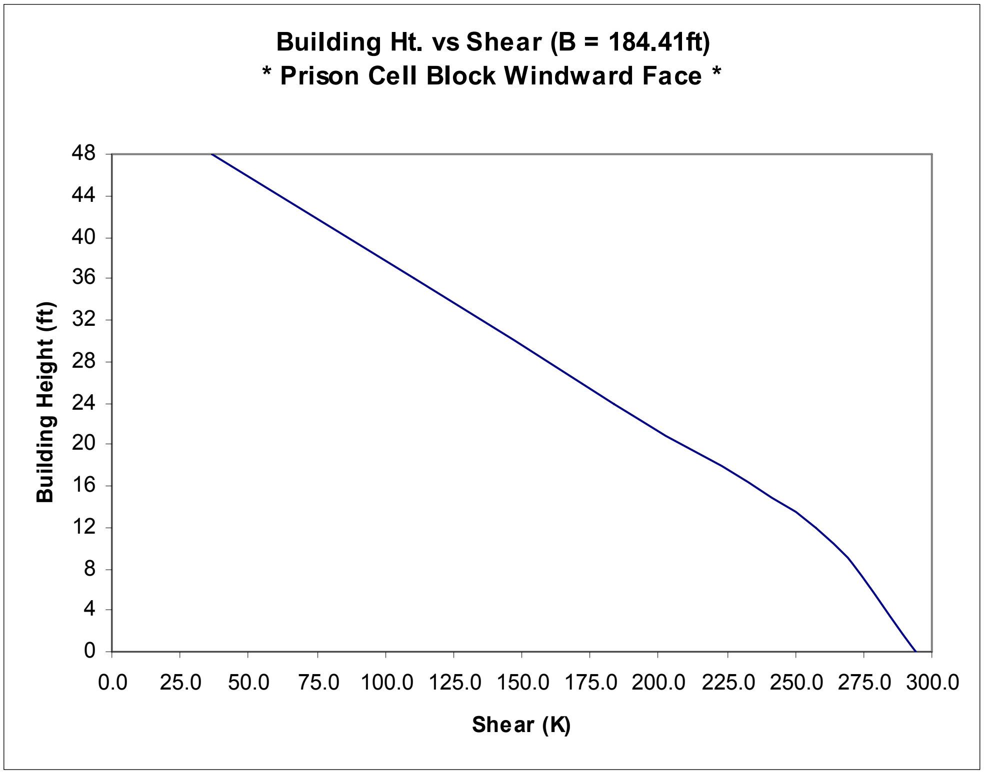
- $P_{windward} = q_z(GC_p) - q_i(GC_{pi})$
$= 18.38$ psf $- 27.03(\pm 0.55)$

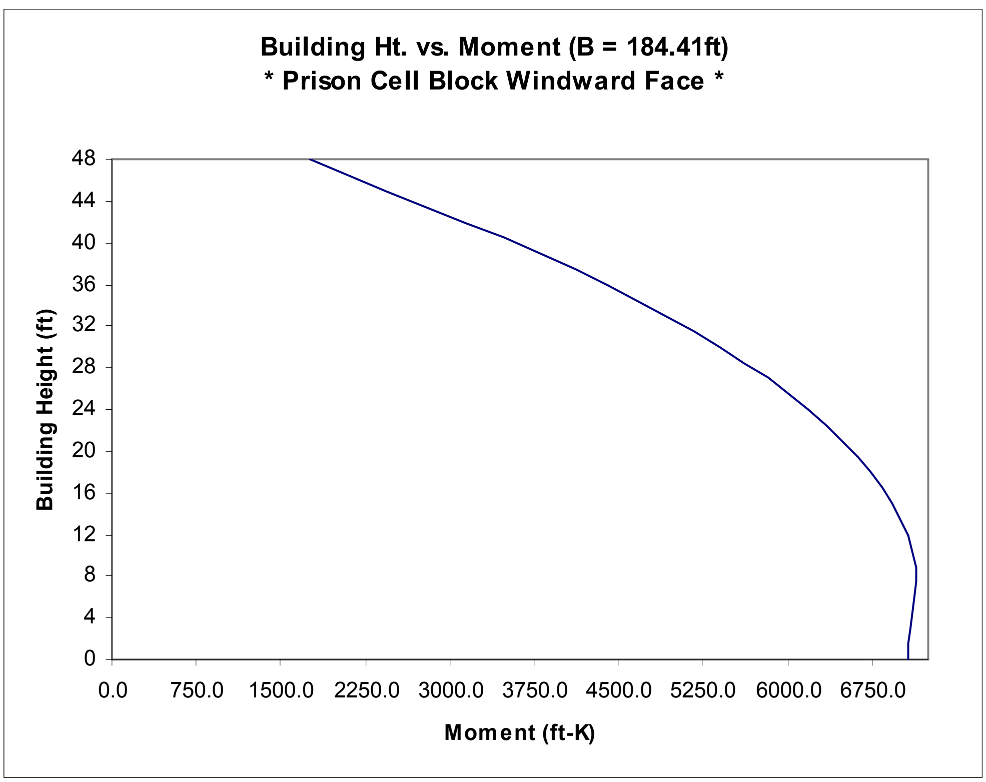
$P_{windward} = 3.51$ psf ; 33.25 psf

- $P_{side} = 16.08$ psf $- 27.03(\pm 0.55)$
$= 1.21$ psf ; 30.95 psf

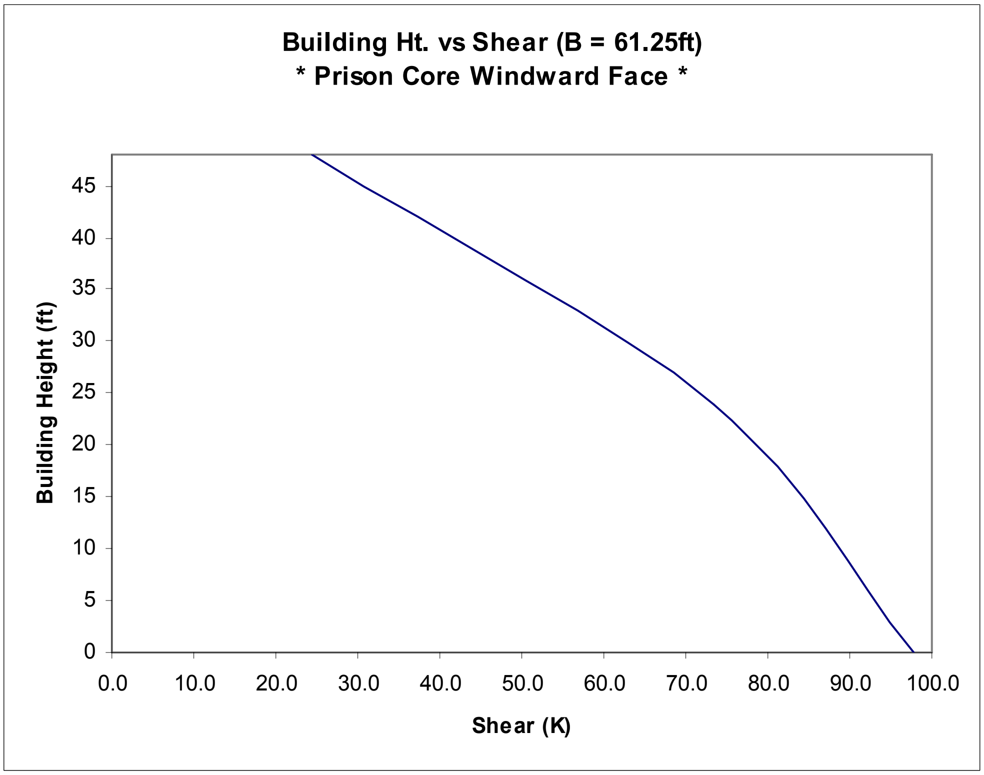
- $P_{roof} = 27.03(0.9) - 27.03 \pm 0.55)$
$= 9.46$ psf ; 39.19 psf

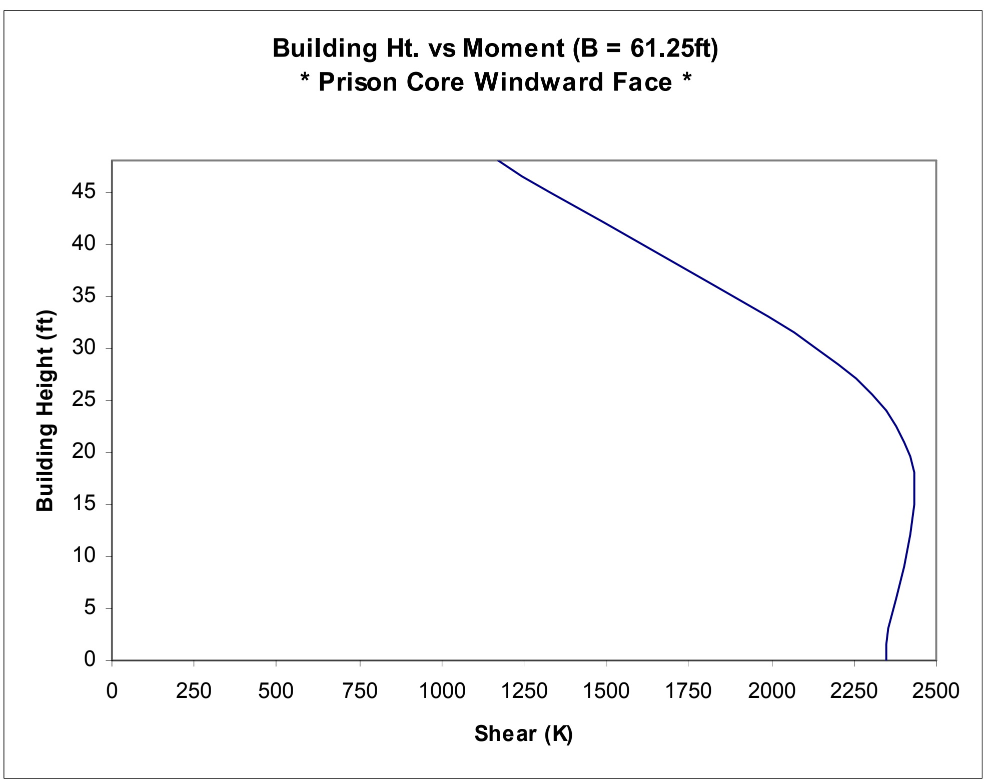
Wind Normal to Prison Cellblock





Wind Normal to Prison Core





F.4 Seismic Load Calculations – Steel

1/14/06 - MQP BT

Seismic Loads - Prison ASCE 07-02

Occupancy Category III

Seismic use group II (9.1.3)

$I = 1.25$ (9.1.4)

Concentrically Braced Frame (CBF)

Ordinary Concentrically Braced Frame (OCBF)

Soil Site Class = C Very Dense Soil / soft rock assumed

$\bar{V}_s =$ 1200 to 2500 ft/s

$\bar{N} / \bar{N}_{CH} > 50$

$S_u > 2000$ psf (9.4.1.2)

$S_s = 0.25g$

$S_1 = 0.1g$

$F_a = 1.2$ (9.4.1.2.4a)

$F_v = 1.7$ (9.4.1.2.4b)

$V = C_s W$ (eq 9.5.5.2-1)

$C_s = \dfrac{S_{DS}}{(R/I)} \not< \dfrac{S_{D1}}{T(R/I)} > 0.044 S_{DS} I$

$S_{MS} = F_a S_s = 1.2(0.25) = 0.3$

$S_{M1} = F_v S_1 = 1.7(0.1) = 0.17$

$S_{DS} = \frac{2}{3} S_{MS} = 0.2$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.17) = 0.1133$

Seismic Continued

$R = 5$

$$C_s = \frac{0.2}{(5/1.25)} = \frac{0.2}{4} = 0.05$$

$$> 0.044\,(S_{DS})I = 0.044(0.2)(1.25) = 0.011$$

$$< \frac{0.1133}{T\,(4)} \qquad T_a \approx C_t h_n^x = 0.02\,(0.055)(80)^{0.75} = 0.03\,s$$

$$< \frac{0.1133}{0.03(4)} = 0.9$$

$C_s = 0.05$

$$V = C_s W \Rightarrow \boxed{V = 0.05\,(W)}$$

F.5 Seismic Load Calculations – Concrete

Seismic Design Calcs for Office Structure

Occupancy category = III   Seismic use group = II (9.1.3)

$I = 1.25$ (9.1.4)

Intermediate Moment Frame (IMF)

Site Class C → very dense soil and soft rock

$$\bar{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}} \quad ?$$

$S_s = 0.25g$ (chart 9.4.1.1(a))

$S_1 = 0.1g$ (chart 9.4.1.1(b))

$F_a = 1.2$ (9.4.1.2.4a)

$F_v = 1.7$ (9.4.1.2.4b)

$S_{MS} = F_a S_s = 1.2(.25) = .3$ (9.4.1.2.4-1)

$S_{M1} = F_v S_1 = 1.7(.10) = .17$ (9.4.1.2.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(.3) = .2$ (9.4.1.2.5-1)

$S_{D1} = \frac{2}{3} S_{M1} = (\frac{2}{3})(.17) = .1133$ (9.4.1.2.5-2)

$R = 5$ → Table 9.5.2.2

$\rho = 1.0$ 9.5.2.4.1

$V = C_s W$ 9.5.5.2-1

$C_s = \frac{S_{DS}}{(\frac{R}{I})}$ 9.5.5.2.1-2 $.044 S_{DS} I < C_s < \frac{S_{D1}}{T(\frac{R}{I})}$

$C_s = \frac{.2}{(5/1.25)} = .05$

need to find T

$T = C_t h_n^x$ — 9.5.5.3.2.1   or $T_a = 0.1N$ – 9.5.5.3.2.1-a

$T = 0.016(75)^{.9}$ $T_a = 0.1(6) = 0.6$

.78

$.044 S_{DS} I < .05 < \frac{S_{D1}}{T(R/I)}$

$.044(.2)(1.25) < .05 < \frac{.1133}{.78(5/1.25)}$

$0.011 < .05 \not< .036$

C_s doesn't need to be greater than .036 so use .036

$V_s = .036W$

$V_s = .036(5035.71)^k = 181.28^k$

$V_{wind} = (100 \times 75)(36.87\ psf) = 276525\ lbs = 276.525^k$ ← wind governs!!

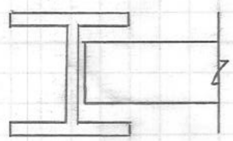
tsubarea

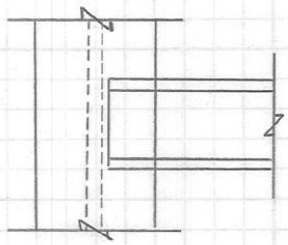
G Connection Design – Steel

G.1 Girder-to-Column Web Bolted Connection

Typical Girder-column Connection
Office - Girder C45

• W16×50 Girder
• W14×74 Column





• W16×50 Girder
$t_w = 0.38in$
$b_f = 7.07in$
$d = 16.3in$
$V_u = 26.46k$

• W14×74 column
$t_w = 0.45in$
$b_f = 10.1in$
$d = 14.2in$

• Verify Shear Resistance of W16×50 girder

$\phi V_n = \phi(0.6F_y)(d t_w)$
$= 0.9(0.6 \times 50ksi)(16.3in)(0.38in)$
$\phi V_n = 167.24K > V_u = 26.46K \rightarrow$ Girder is Adequate

• # Bolts

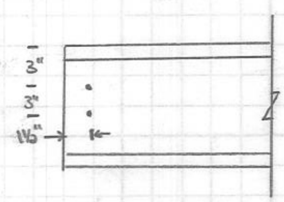
3/4" ϕ A325-X in A36 angles ($F_y = 36ksi$, $F_u = 58ksi$)

• $\phi R_n = \phi F_v(2A_b)$
$= 0.75(60ksi)\left[2\left(\frac{\pi \times (0.75in)^2}{4}\right)\right]$

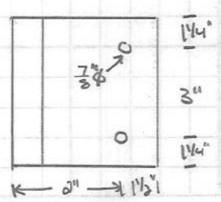
$\phi R_n = 39.76K > V_u = 26.46K \rightarrow$ need 1 bolt $\rightarrow$ use 2 bolts for girder connection

• $\phi R_n = \phi F_v(A_b)$
$= 0.75(60ksi)\left(\frac{\pi \times (0.75in)^2}{4}\right)$

$\phi R_n = 19.88K < V_u = 26.46K \rightarrow$ need 2 bolts (1 each angle) $\rightarrow$ use 2 bolts for each angle



W16×50 Column



• Angle Thickness

$\cdot L_{c1}$ = clear distance between holes
$= 3 - 2[1/2(0.75 + 0.125)]$
$L_{c1} = 2.125in$

$\cdot L_{c2}$ = clear distance @ top
$= 1.25 - 1/2(0.75 + 0.125)$
$L_{c2} = 0.8125in$

$\cdot \phi R_n = 2(\phi 1.2 L_c t F_u)$
$39.76k = 2(0.75)(1.2)(2.125 + 0.8125) t (58ksi)$
$t \geq 0.13in$ →

$\phi R_n = 2[\#bolts \cdot 2.4 \phi d_b t F_u]$
$39.76k = 2[2 \cdot 2.4 \cdot 0.75 \cdot (0.75) t (58ksi)$
$t \geq 0.127in$

use 3/16" thick ∠

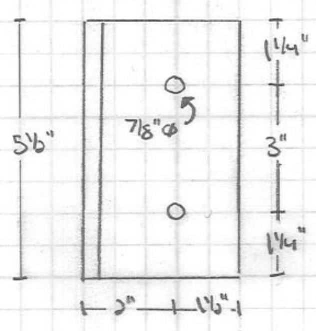
• Shear Yield
$\phi R_n = \#angles[\phi A_g(0.6F_y)]$; $A_g = LT$
$39.76k = 2[0.75(5.5 \cdot T)(0.6)(36ksi)]$
$t = 0.22in$

Shear Rupture
$\phi R_n = \#angles[\phi(0.6F_u)(L_n)(t)]$; $L_n = L - \#bolts(\phi)$
$39.76k = 2[0.75(0.6 \cdot 58ksi)[5.5 - 2(0.75 + 0.125)]t]$
$t = 0.203in$

use 1/4" thick angles ← revised (shear yield governs)

• Shear Resistance of W14×74 column
$t_w = 0.45in > t = 0.22in$
↓
shear resistance is adequate

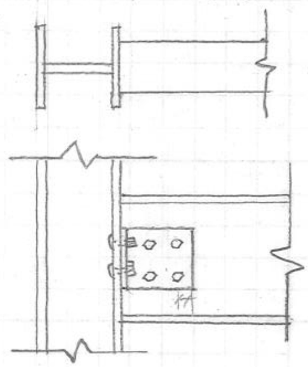
- Use 3½" × 3½" × ¼" plate for girder - column web connection w/ A325-X Bolts



G.2 Beam-to-Column Flange Bolted Connection

Typical Girder – Column Connection

– Connect to Column Flange



W16×50 girder

$t_w = 0.380$ in

$d = 16.3$ in

W14×74 column

$t_f = 0.785$ in

$d = 14.2$ in

end reaction $= \dfrac{52.92^k}{2} = 26.46^k$

$\phi V_n = \phi\, 0.6 f_y\, d\, t_w = 0.9(0.6)\, 50\,\text{ksi}\,(16.3")(0.380) = 167.2^k$

Number of bolts required assume 3/4" A325-X Bolt

$\phi R_n = \phi F_z (2A_b) = 0.75\,(60\,\text{ksi})(2(0.442)) = 39.7^k$

$\phi R_n >$ end reactions

use two bolts on girder

$\phi R_n = 0.75(60)(0.442) = 19.8^k$

use two bolts on each angles

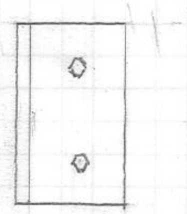
See AutoCAD drawings for details

Angle member selection

3 in C/C bolt spacing

$1\tfrac{1}{4}$ in from top of plate

$1\tfrac{1}{2}$ in from edge



17

Angle thickness

L_{c1} = Cl. dist between holes = $3\text{ in} - \varnothing_b = 2\tfrac{1}{8}\text{ in}$

L_{c2} = Cl. dist to top = $1\tfrac{1}{4} - \frac{\varnothing_b}{2} = \tfrac{13}{16}\text{ in}$

$$\varnothing R_n = 2(\varnothing)(1.2(L_{c1}+L_{c2}))\,t_A\,F_u$$

$$t_A = \frac{\varnothing R_n}{2\varnothing(1.2)(L_{c1}+L_{c2})F_u} = \frac{39.7k}{2(0.75)(1.2)(2\tfrac{1}{8}+\tfrac{13}{16})58ksi} = 0.13\text{ in}$$

$$\varnothing R_n = 2(\text{No. bolts})\,2.4\,\varnothing\,\varnothing_b\,t_A\,F_u$$

$$t_A = \frac{\varnothing R_n}{2(\#\text{bolts})(\varnothing)(\varnothing_b)F_u} = \frac{39.7}{(2.4)2(2)(0.75)(0.75)(58)} = 0.126\text{ in}$$

use 3/16 in A36 angle plate

Shear yield

$$\varnothing R_n = \#\text{angles}(\varnothing)(\text{gross area})(0.6F_y)$$

$$39.7 = 2(0.75)(5\tfrac{1}{2})(t_A)(0.6)(36\text{ ksi}) \Rightarrow 0.23\text{ in}$$

Shear rupture

$$\varnothing R_n = \#\text{angles}(\varnothing)(0.6F_u)(A_{\text{eff}}\text{ Nominal})$$

$$2(0.75)(0.6)(58)(3.75)(t_A) \Rightarrow t_a = 0.202\text{ in}$$

use 1/4 in A36 angle plate

$t_f > t_{angle}$ thus should be sufficient

use L $3\tfrac{1}{2} \times 3\tfrac{1}{2} \times \tfrac{1}{4} \times 5\tfrac{1}{2}$" (length) A36 steel
2 - 3/4" A325-X bolts on girder
4 - 3/4" A325-X bolts on column
See AutoCAD Appendix for Details

21

H Foundation Design

Foundation Design (Square Spread footing)

Reaction from interior column w/in correctional facility

$P_u = 600^k$

Soil Bearing Capacity (IBC 2003)

3000 psf

Dimensions of footing:

$$\frac{3^k}{1\,ft^2} = \frac{600^k}{a_f}$$

a_f = area of footing = 200 ft^2

= 14.14 ft × 14.14 ft

use: 15 ft × 15 ft

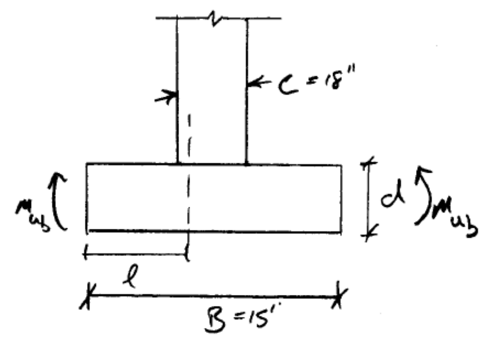
thickness = 36" (table 8.1 Coduto, Donald P.)

Required Area of Steel:

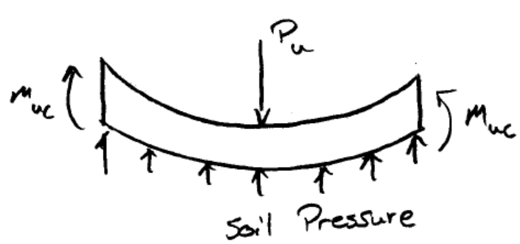
B = 15'

C = 18"

$$\ell = \frac{B-C}{2} = \frac{15' - 1.5'}{2} = 6.75\,ft \Rightarrow 81''$$



$$M_{uc} = \frac{P_u \ell^2}{2B} = \frac{600{,}000\,(81)^2}{2\,(180)} = 10{,}935{,}000 \text{ in-lb}$$



$$A_s = \text{Area of Steel} = \left(\frac{f'_c\, b}{1.176\, f_y}\right)\left(d - \sqrt{d^2 - \frac{2.353\, M_u}{\phi\, f'_c\, b}}\right)$$

$$= \left(\frac{3000\,(180)}{1.176\,(60{,}000)}\right)\left(36'' - \sqrt{36^2 - \frac{2.353\,(10{,}935{,}000)}{0.9\,(3000)\,(180)}}\right)$$

$$= 7.653\,(0.743) = 5.96\,\text{in}^2$$

<u>use</u>: 10 #7 bars (both directions)

$\phi = 0.875''$

$A_s = 6.01\,\text{in}^2$

$$\text{Spacing} = \frac{180}{11-1} = 18''$$

<u>Development</u> <u>Length</u>

$$l_{d_{\text{supplied}}} = 81'' - 3 = 78''$$

$$\frac{C + K_{tr}}{d_b} = \frac{3+0}{0.875} = 3.43$$

$$\frac{l_d}{d_b} = \left(\frac{3}{40}\right)\left(\frac{f_y}{\sqrt{f'_c}}\right)\left[\frac{\alpha\beta\gamma\lambda}{\left(\frac{C+K_{tr}}{d_b}\right)}\right]$$

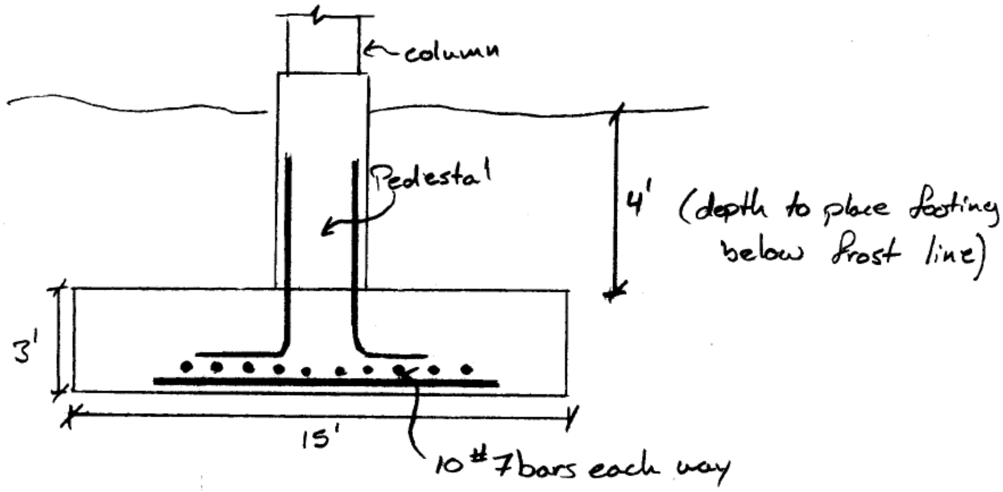
$$= \left(\frac{3}{40}\right)\left(\frac{60{,}000}{\sqrt{3000}}\right)\left[\frac{1\times1\times1\times1}{3.43}\right]$$

$$\frac{l_d}{d_b} = 23.95\,\text{in} \qquad l_d = 23.95(0.875) = 21''$$

$$l_d < l_{d_{\text{supplied}}} \quad \underline{\text{OK}}$$

use: 10 #7 bars in both directions

Spacing = 18" (clear)



I Exterior Wall Design Calculations

I.1 Office Exterior Wall Design

Administration Building — T. Alexander — 1/

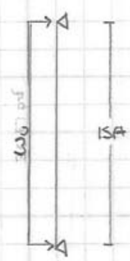
Exterior Wall Design
- Brick Veneer Over Steel Studs

- Story Height = 15ft (1st floor – worst case)
- Wall Length = 20ft

Loads
- Brick Veneer = 40psf
- WL = 36.87psf

Materials
- Norman Brick → nom. size : 12×4×2 2/3
  act. size : 11 5/8 × 3 5/8 × 2 1/4
  f'_c = 2500psi
  $E \approx 750 \cdot f'_c$ → $E = (750 \text{ to } 1000)(f'_c)$
  = 1,875 ksi
- Cold-rolled steel framing – F_y = 33 ksi
  – spacing : assume 16" o.c.

- $\Delta_{MAX} = \dfrac{L}{600}$
  $= \dfrac{(15ft \cdot 12in/ft)}{600}$
  $\Delta_{MAX} = 0.3in$



$w_u = (36.87psf)(\tfrac{16}{12}ft)$
$= 49.16\ lb/ft$

- $\Delta = \dfrac{5wL^4}{384EI}$
  $I_{req'd} = \dfrac{5wL^4}{384E\Delta}$
  $= \dfrac{5(49.16lb/ft)(15ft)^4(12in/ft)^3}{384(29{,}000{,}000psi)(0.3in)}$
  $I_{req'd} = 6.44in^4$

↓

Try CSW Series (Dietrich Curtain Wall / Light-Gauge Structural Framework Products)

CSW 10" stud, 18g : $I_x = 9.080in^4$ $S_x = 1.581in^3$ wt = 2.198lb/ft
CSW 8" stud, 16g : $I_x = 6.577in^4$ $S_x = 1.644in^3$ wt = 2.377lb/ft
CSW 6" stud, 16g : $I_x = 6.762in^4$ $S_x = 2.254in^3$ wt = 4.239lb/ft

- Try CSW 8" – balance between wt and thickness

$\cdot S_{req'd} = \dfrac{w_u L^2}{8 F_b}$

$= \dfrac{(49.16\ lb/ft)(15ft)^2 (12in/ft)}{8 \times 19{,}760.5\ psi}$

$S_{req'd} = 0.84 in^3$

$S_{req'd} < S_{provided} = 1.644 in^3$

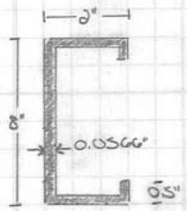
$F_b = \dfrac{F_y}{F.S}$ ← bending strength

$= \dfrac{33{,}000 psi}{1.67}$

$F_b = 19{,}760.5 psi$

$M_n = F_b S_x = \dfrac{w_u L^2}{8}$

★ Use Dietrich CSW 8" cold-rolled steel stud ; $F_y = 33 ksi$

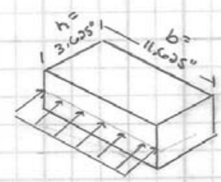


· Strength checks of Brick Veneer

-Norman Bricks (nom size: 12×4×2⅔)

$f'_c = 2500 psi$ $f_r = 180 psi$

$E = 1{,}875{,}000 psi$



·Cracking Load

$W_{cr} = \dfrac{8 M_{cr}}{L^2}$; $M_{cr} = f_r S_x$ $S_x = \dfrac{I}{C}$

$= (180 psi)(25.46 in^3)$ $= \dfrac{1/12 (11.625in)(3.625in)^3}{\frac{1}{2}(3.625in)}$

$M_{cr} = 4582.8 lb{\cdot}in$ $S_x = 25.46 in^3$

$= \dfrac{(8)(4582.8 lb{\cdot}in)}{(12in/ft)(15ft)^2}$

$W_{cr} = 13.58 lb/ft$

$\dfrac{W_{cr}}{1.5ft \text{ tributary width}} = W_{cr} = 9.05 psf$

·Brick Veneer Stiffness

$(EI)_{brick} = (1{,}875{,}000psi)[\tfrac{1}{12} \cdot 11.625in \cdot (3.625in)^3]$

$= 89{,}315.2 k{\cdot}in^2$

·Steel Stud Stiffness

$(EI)_{stud} = (29{,}000{,}000psi)(6.577in^4)(12/16)$

$= 143049.8 k{\cdot}in^2$ ↑ contribution per unit length

· Total out-of-Plane load Resulting in cracking

$$W_{TOTAL} = W_{cr}\underbrace{\left[\frac{(EI)_{brick} + (EI)_{stud}}{(EI)_{brick}}\right]}$$

increased stiffness (total of assembly / brick)

$$= 13.58\,lb/ft\left[\frac{89315.2\,k\cdot in^2 + 143049.8\,k\cdot in^2}{89315.2\,k\cdot in^2}\right]$$

$W_{TOTAL} = 35.3\,lb/ft$

$W_{TOTAL} < W_{wind} = 49.18\,lb/ft$

↓

since $W_{TOTAL} < W_{wind}$ → brick has cracked (i.e. the allowable load is less than the applied load → exceeds the rupture strength of the brick)

· Force to crack brick

$T_{MAX} = W_{cr} * L/2 * \text{trib width}$

$= (9.05\,psf)(\frac{15}{2}\,ft)(16\,in/12\,in/ft)$

$T_{MAX} = 90.5\,lb$

15ft

16"

· Cracked Brick

$T_{MAX} = W_{wc} * L/2 * \text{trib. width} * (1/2)$

↑ limiting factor since bricks have cracked

$= (36.87\,psf)(\frac{15}{2}\,ft)(16\,in/12\,in/ft)(1/2)$

$T_{MAX} = 184.4\,lb$

· Anchor Strength

- ties shall be placed @ a distance no greater than 16in. o.c vertically and 24in. o.c horizontally and shall not support an area greater than 2.67 ft². (The Massachusetts State Building Code (780 CMR), 1405.5.4)

· Tie spacing: 16in. o.c vertically (max. allowable)
16in. o.c horizontally (steel stud spacing)

$\text{Load} = W_{wc} * \text{trib. area} + W_{wind} * \text{trib area}$

$= (36.87\,psf)\left(\frac{16in * 16in}{144\,in^2/ft^2}\right) + (40\,psf)\left(\frac{16in * 16in}{144\,in^2/ft^2}\right)$

$\text{Load} = 136.7\,lb$

Design Summary

· Dietrich CSW 8" 16gauge cold-rolled steel stud, $f_y = 33\,ksi$
· Norman Bricks (12" × 4" × 2⅔") $f'_c = 2500\,psi$; $f_r = 180\,psi$
Ties spaced 16" o-c vertically and horizontally
└ capacity = 136.7lb.

I.2 Prison Cellblock Exterior Wall Design

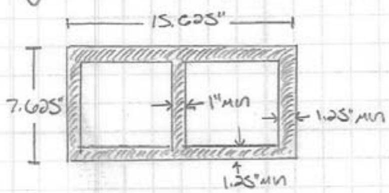
Prison Superstructure | T. Alexander | 1/

Wall Design
-Non-load Bearing

·Prison Cell Block
- Wall height = 12ft → 144in
- Wind load = 33.25psf
- DL, LL = ϕ → non-load bearing

- column spacing = 19.58ft → 235in (worst case)

·Try 8" hollow concrete block, fully grouted



A_g = 119.14in²
A_e = 57in²
wt = 92psf
↑ fully grouted wall, 8" thick, normal wt. concrete units (Schneider, Robert R. p.103)

·Grade I (exposed)
H. 5,000

·f_m = 1400psi
↑ flexural comp. stress in masonry

·f'_m = 2300psi
↑ specific ult. axial comp. stress of masonry assemblage

(Table 2-7, Schneider, p.45)

·f_c = 460 psi
↑ max. allowable working stress

(Table 7-1, Schneider, p.210-211)

·Type M Mortar

·f_{mo} = 3,000psi

·Grade 60 Rebar

·f_s = 60,000psi

·Nominal Moment
- Use WL · shown to govern over EL in lateral design of structural framework

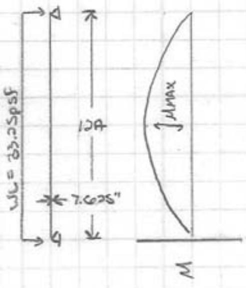
$$\cdot M_{wind} = \frac{w_{wl} \cdot h^2}{8} = \frac{(33.25psf)(12ft)^2}{8}$$

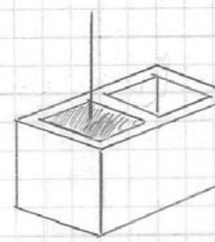
M_{wind} = 598.5 lb·ft / ft wall

$$\cdot M_{weight} = \frac{w_{wt} \cdot h^2}{8} = \frac{(31.74psf)(12ft)^2}{8}$$

M_{weight} = 571.32 lb·ft / ft wall

$w_{wt} = ZICpW$
$= (1.0)(1.15)(0.3)(92psf)$
w_{wt} = 31.74psf
↑ static force equivalent





• Steel Required

$M_s = f_s A_s j d$

$A_s = \dfrac{M_s}{f_s j d}$

$= \dfrac{(598.56\,lb\cdot ft)(12 in/ft)}{(60{,}000\,psi \cdot 1.333)(0.9)(3.8125 in)}$

$\uparrow$ accounts for wind

$A_s = 0.0262\ in^2/ft$ wall

• $j = 0.9$ (assumed) $\leftarrow$ moment arm

• $d = t/2$

$= 7.625''/2$

$d = 3.8125 in$

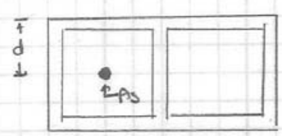
• check to see if bending governs

$\rho = \dfrac{A_s}{bd}$ b = unit length of wall d = effective width of block

$= \dfrac{\text{effective area of reinforcement}}{\text{effective area of concrete block}}$

$= \dfrac{0.0262 in^2}{(12in)(3.8125in)}$

$\rho = 0.000573$



$\dfrac{a_{bal}}{d} = \beta_1\left(\dfrac{87{,}000}{87{,}000 + F_s}\right) > \dfrac{a}{d}$ $\rightarrow$ steel will yield before concrete crushing

$= 0.85\left(\dfrac{87000}{87{,}000 + 60{,}000}\right)$ ($\beta_1 = 0.85$ for $f'_m < 4000psi$) $> \dfrac{0.0262 in^2}{3.8125 in}$

$\dfrac{a_{bal}}{d} = 0.503 > 0.0068 in$ $\rightarrow$ steel will yield!

• $\rho n = \rho\left(\dfrac{E_m}{E_s}\right)$

$= 0.000573\left(\dfrac{1800{,}000psi}{29{,}000psi}\right)$

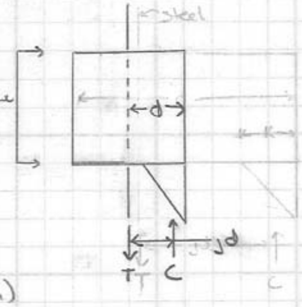
$\rho n = 0.0356$

• k = depth of compressive stress block

$= \sqrt{2\rho n + (\rho n)^2} - (\rho n)$

$= \sqrt{2(0.0356) + (0.0356)^2} - 0.0356$

$K = 0.233 in$



• j = moment arm = $1 - k/3$
= $1 - 0.233/3$
$j = 0.922$

• $C = \frac{f_m}{2}(b\,kd)$
← area block in compression
$f_c/2$ → assume no field inspection – worst case
(if field inspected – use f_m)
$= \frac{460\,psi}{2}(12in)(0.233 * 3.8125in)$
$C = 2451.7\,lb$

• $M_n = Cjd$
$= (2451.7\,lb)(0.922 * 3.8125in)$
$M_n = 8618.2\,lb{\cdot}in/ft$ wall
↳ $718.2\,lb{\cdot}ft/ft$ wall

$M_n > M_{wind}$ → good!!! bending does not govern

• Determine bar sizing and spacing (vertical steel to carry tensile forces from wind)
– reinforcing running vertically shall not be spaced more than 48" OC
(MA building code 6th Ed., 2104.3.2)

#4 @ 48" → $A_s = 0.049\,in^2/ft$ wall
#5 @ 48" → $A_s = 0.077\,in^2/ft$ wall
#6 @ 48" → $A_s = 0.110\,in^2/ft$ wall

– Try #4 @ 48"

• Minimum Reinforcement (Vertical) = $0.0007\,bt$ (MA Building code 6th Ed., 2104.4.3)
$= 0.0007(12in)(7.625in)$
$= 0.0641\,in^2 > 0.049\,in^2$ provided
↓
#4 bars @ 36in OC

• Min. Total Steel Req'd = $0.002\,bt$ → gross sectional area (MA Building code 6th Ed. 2104.4.3)
$= 0.002(12in)(7.625in)$
$= 0.183\,in^2/ft$ wall

• Total horizontal steel
$= 0.183 - \left(\frac{\pi/4 * (0.5in)^2}{3ft}\right)$
$= 0.118\,in^2$
↓
#4 bars @ 18in

Design Summary

- 8" hollow concrete block, fully grouted
  - f_n = 4000psi
  - f'_m = 2300psi
  - f_c = 460psi
- Type M Mortar
- Grade 60 Rebar
  - #4 @ 36in o.c vertically
  - #4 @ 18in o.c horizontally

↓

Recommended to use atleast #4 bars @ 8in o.c horiz. and vert. (Krasnow, Peter p.183-184)

- Wall Lateral Support Requirements (MA Building Code 6th Ed, Table 2108.1)
  - for non-load bearing exterior walls:
    max ratio of wall height or wall length to thickness = 18:1

  - height = 12ft (144in)
  - length = 19.58ft (235in)
  - thickness = 7.625in

$\frac{h}{t} = \frac{144in}{7.625in} = 18.9$ $\frac{l}{t} = \frac{235in}{7.625in} = 30.8$

↳ to satisfy h/t → redesign w/ 10in. wide block

↳ to satisfy l/t → redesign column spacing to ≈11.4ft
→ redesign w/ wider block and altered column spacing