PROJECTE O TESINA D’ESPECIALITAT

Títol

Mètode de construcció “top-down”: impacte en la subestructura de l’Hotel Marriott Marquis

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RESUM

En aquesta tesina s’estudia i s’investiga l’ús del mètode de construcció top-down i com aquest mètode impacta en la subestructura dels edificis.

El mètode de construcció Top-down permet que la subestructura i la superestructura d’un edifici puguin ser construïdes al mateix temps, estalviant temps en la construcció del mateix. A més, té la particularitat de que les lloses de l’edifici poden fer-se servir per apuntalar el mur de sosteniment.

Per analitzar com aquest mètode impacta en un edifici, s’ha fet un estudi de cas amb l’Hotel Marriott Marquis de Washington, DC. Així, en aquesta tesina s’investiga com el mètode de construcció afecta al disseny de les columnes de la subestructura (que han de ser dissenyades com a columnes d’acer i com a columnes mixtes), a les lloses de superestructura (que seran reforçades amb plaques d’acer i bigues) i a les bigues principals.

Paraules clau: Mètode de construcció "top-down", model d’elements finits, mur de sosteniment, columnes mixtes, plaques d’acer.

ABSTRACT

In this thesis, the use of top-down construction and how this method impacts in the below grade structure of the buildings is investigated.

Top-down construction permits the above grade construction to proceed simultaneously with below grade work and permits to use below grade slabs to brace the diaphragm wall as the construction proceeds.

The Marriott Marquis Hotel located in downtown Washington, DC is chosen as a case study in this thesis to analyze these impacts. In this thesis, how the construction method impacts the design of below grade columns (that have to be designed as steel columns and composite columns), below grade slabs (designed to brace the diaphragm wall and reinforced with steel plates and beams) and plate girders is investigated.

Keywords: Top-down construction method, finite elements model, slurry wall, composite columns, steel plates.
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PREAMBLE

This thesis is the result of the “Projet Fin d’études” of the “École Nationale des Ponts et Chaussées”, as stipulated in the learning agreement to obtain the double degree with this university.

This thesis was done during the months from March 2011 to September 2011 in the American company of Thornton Tomasetti, based on Washington DC and was supervised by Mr. Christopher Crilly from Thornton Tomasetti and Joël Raoul from “École Nationale des Ponts et Chaussées”.

This thesis has already been validated in the “École Nationale des Ponts et Chaussées” and this is the final version for the “Escola de Camins, Canals i Ports the Barcelona”, tutorized by Mr. Jose Turmo.

1. INTRODUCTION

The top-down construction method is an innovative construction method permitting the above grade construction to proceed simultaneously with below grade work. This method reduces the schedule of a project and can also be used to brace the diaphragm wall. However, the choice of top-down impacts the design of a building adding unique aspects to a project that the engineer has to deal with.

The Marriott Marquis Convention Center Hotel in Washington DC is a project where top-down is the construction method used. In this project, the choice of the method directly impacts the design of the below-grade structure, which has to be designed for special conditions. The special challenges that top-down add to this project and how these challenges have been solved is described in this report.

In the first part of the report, a general approach to the top-down construction is done, describing the main stages of the method. In this part of the report it is also described why top-down construction is chosen in the Marriott Marquis. The rest of the report investigates how the choice of a construction method can impact in the design of a building and how the possible problems of this methodology have been solved.
2. OBJECTIVES

The main objective of this thesis is to investigate about the top-down construction method and its applications.

During this investigation, the main goals are:

- To understand how the top-down construction method works, when it can be used, and when its use can be mandatory.
- To do a case study of the application of the top-down construction method, applying this method to the construction of the Marriott Marquis Hotel in Washington DC, and see how the choice of this construction method impacts in its below-grade structure.
- To draw conclusions about how this construction methods can impact in the below-grade structure of the case study and how the engineer can deal with this new challenges.

In the case study, the focus on where the construction method impacts the structure is in the below-grade structure. The impacts on the below-grade columns, the steel reinforcement of the below-grade slabs and the design of steel framing in levels S5 and L1 are investigated. Solutions of how to deal with these problems are also proposed.

During this investigation several modeling software tools such as Revit [1], RAM [2] or SAP [3] are used to simulate the impacts of the construction method in the structure.

3. THORNTON TOMASETTI

Thornton Tomasetti [4] is an engineering design company founded in 1956 as Lev Zetlin & Associates (LZA) by Lev Zetlin upon the completion of his PhD at Cornell University. This company was a pioneer in the use of double layer bicycle wheel roof system used in the Utica Civic Auditorium and other structures, as well as the hyperbolic roofs utilized in American Airlines 747 super-hanger in Los Angeles and San Francisco. LZA also embraced the creative use of materials. Thornton Tomasetti really came into existence in 1975 when Charlie Thornton and Richard Tomasetti purchased LZA Associates. At this time, Thornton Tomasetti began to branch out and enter the high-rise market with several innovative designs.

Today, Thornton Tomasetti is a leader in engineering design, investigation and analysis, serving clients worldwide on projects of all sizes and complexity. Thornton Tomasetti has projects on 6 continents (42 countries) and it is an organization with 550 engineers, architects and professionals, distributed in 22 offices, 13 offices in the United States and 9 offices abroad (Abu Dhabi, Christchurch, Dubai, Ho Chi Minh City, Hong Kong, London, Moscow, Mumbai and Shanghai) [5]. Their practices are building structure, building skin, building performance, property loss consulting and construction support services.

Thornton Tomasetti has provided structural design for several of the world’s tallest building structures including the Petronas Towers in Kuala Lumpur (Malaysia), as seen in Figure 1,
Taipei 101 (Taiwan), as seen in Figure 2, as well as multitude office buildings in New York City (New York Times building) and other recognizable structures. The company has collaborated with high profile architects including Cesar Pelli, Rafael Vinoly, Renzo Piano and Santiago Calatrava [4].

The office that I joined and where I have done this thesis is the Washington DC office. This office is composed by 25 engineers and professionals and has participated in projects such as the Nationals Park baseball stadium, The Johns Hopkins Hospital New Clinical Building and the Washington, DC Convention Center [4].

![Figure 1. Petronas towers in Kuala Lumpur, Malaysia [4]](image1)

![Figure 2. Taipei 101 in Taipei, Taiwan [4]](image2)
4. THE CASE STUDY: MARRIOTT MARQUIS

The Marriott Marquis Convention Center Hotel is located in downtown Washington DC, the capital of the United States, between 9th and 10th Street North West, and L Street and Massachusetts Avenue. The hotel is located adjacent to Mount Vernon Square and the Washington DC Convention Center, the largest convention center in the city. The hotel is Metro accessible and will host a big portion of convention center visitors.

The Marriott Marquis is a 15-story building above grade and 7 stories below grade with a building area of 1,000,000 square feet. The hotel includes 1,175 guest rooms (including 46 suites), below grade long-span areas for meeting and ballroom use, 25,000 square foot of retail space and a full-height atrium courtyard. Meeting and event facilities have more than 100,000 square-foot of function space, including a 30,000 square foot grand ballroom, two 10,800 square foot junior ballrooms, more than 53,000 square foot of meeting space, an 18,000 square foot indoor event terrace and a 5,200 square foot rooftop terrace. The penthouse houses mechanical equipment [6]. A render image of the final aspect of the Marriott Marquis can be appreciated in the Figure 3.

![Figure 3. Final aspect of Marriott Marquis [6]](image-url)

In this project, the existing historical Plumber’s union building on the corner of Massachusetts Avenue and 9th Street will remain, becoming a separate guestroom tower. This tower will be connected to the lobby and the 4th floor of the project. The exterior materials and the architectural character of the building will be compatible to the area and to other newer buildings on and around Massachusetts Avenue.

The architectural design is provided by TVS Design and Cooper Carry, Inc. and the owners are Quadrangle Development Corporation and Marriott International, Inc. Total construction cost is estimated to be $520 million. The construction started in November 2010 and is scheduled to end in spring 2014.
Thornton Tomasetti is Structural Engineer of Record (SER) and provides structural design for the below grade structure as well as for the building lateral system. A+F engineers provided structural design of the above grade structure as consultant to Thornton Tomasetti. The new Marriott Marquis is a concrete building, including some steel, that will be built by using the “top down” construction methodology. The 100 foot deep excavation is supported by perimeter slurry walls which are internally braced by below grade floor diaphragms. The below grade diaphragms are perforated by three stories of ballroom spaces and various vertical transportation shafts. Finite element plate analysis is required to design for in-plane stresses, and an innovative composite concrete and steel plate system will be used to resist the highly concentrated stresses at the floor openings. The fifteen stories above grade portion of the building are partially transferred over the ballrooms and will require the plate girders to be pre-loaded to control deflections caused by the staged loading of the construction above. The project was produced in Revit [1] and is also designed to meet criteria for Leadership in Energy and Environmental Design (LEED) Silver certification [7].

4.1. GENERAL STRUCTURE OF THE MARRIOTT MARQUIS

Marriott Marquis is a 15-story building above-grade and 7 stories below-grade. Above-grade and below-grade parts of the building have different structural systems.

The above grade structure is made of concrete. The typical floor-to-floor height is 8’-8” and the typical slab is 8” thick (with a concrete compressive strength of 5,000 psi). The typical columns are concrete columns 28”x20” with a concrete compressive strength of 5,000 psi from level 10 to the roof, 6,000 psi from level 4 to level 9 and 7,000 psi from level 1 to level 3. The typical bay is 27’-0” x 27’-0”, as can be seen in Figure 4.

Figure 4. Typical above grade structure [8]
The below-grade structure of Marriott is different from the above grade structure. The typical floor-to-floor height is 15’ but varies from 11’ to 16’. The slab thickness varies from 16” to 24” inches, and is reinforced with steel plates and steel beams at some locations. Columns used in the below-grade structure are composite steel and concrete columns, typically 28”x24”. The typical bay is 27”-0” x 27”-0”.

Level S3 has 2 junior ballrooms, and level S5 has one grand ballroom where columns cannot be used. Because of the lack of columns, long spans are required to support levels S5 and L1, so steel framing is used to support these 2 slabs.

Figure 5. Revit [1] model created for the Marriott Marquis project [4]
5. TOP-DOWN CONSTRUCTION METHOD

Construction used for the Marriott Marquis Hotel will utilize the Top –Down construction method. Top-down is an innovative method of excavation that allows the above grade construction to proceed simultaneously with below grade work. This removes the foundation from the “critical path”, resulting in an accelerated project schedule compared to conventional temporary shored methods.

5.1. GENERAL ASPECTS OF TOP-DOWN CONSTRUCTION METHOD

The traditional construction sequence of the top-down construction method begins with the installation of the retaining wall that supports soil at both sides. The next step, before any excavation takes place, is to install the columns that will carry the future super-structure. Then, the ground floor slab is built, leaving a construction hole open to allow continuous excavation until next level by mining methods. The excavation, starting at the construction hole, begins once the top floor has gained sufficient strength to support its own weight as well as soil diaphragm loads. Soil under the ground floor is excavated around the basement columns, to make it slightly lower than the first basement floor elevation, in order to allow the installation of the forms for the first level basement slab. Construction holes are left open within each newly formed basement floor slab and the procedure is repeated. Thus, each floor rests on the basement columns that were constructed earlier [9], [10], [11].

In the case of Marriott Marquis Hotel, the sequence is divided into 3 main steps, the construction of the retaining wall (slurry wall), the installation of steel columns and the construction of each floor slab by mining methods. The elements needed for the top-down construction method can be seen in Figure 6.

![Figure 6. Elements needed for the top-down construction method [9]](image)
5.1.1. SLURRY WALL CONSTRUCTION

Slurry wall is a technique used to build reinforced-concrete walls in areas of soft earth close to open water or with a high ground water table. In the case of top-down construction, this method serves to build the diaphragm wall to support the soil. The slurry wall construction consists of the excavation of a trench (following the form of each wall) that is then filled of slurry.

First, a pre-trench is excavated along the wall alignment. Then, inside and outside guide walls (that provide alignment to the slurry wall) are installed along the final location of the slurry wall. Once these guide walls are installed, wall panels are excavated under the slurry. The slurry used is named bentonite slurry and it is a mixture of water and bentonite (an absorbent aluminium phyllosilicate). This slurry has a higher specific gravity than the soil, and prevents the trench from collapsing, providing outward pressure that balances the inward hydraulic forces. It also prevents water flow into the trench. Once the trench is filled of slurry, the steel reinforcement is lowered into the trench, and the trench filled with concrete.

Concrete is placed by tremie method pushing the slurry up to the top. The tremie method consists on placing concrete below water level through a pipe in order to push from bottom up the bentonite slurry, which has a lower density than concrete. This process is shown in Figure 7 [9],[10],[11].

Figure 7. Steps followed to install the slurry wall. 1) Install guide walls 2) Excavate panel 3) Desand excavated panel 4) Install rebar cage 5) Place concrete [9]
5.1.2. STEEL COLUMN INSTALLATION

The second step in top-down construction is the installation of the load bearing elements, columns and caissons, to proceed with below-grade construction and to support above-grade structure during the construction. The sequence to build the load bearing elements is the following.

First, a round shaft is excavated under bentonite slurry (material that can be recycled from one column to another). This excavation has to reach at minimum 2’ into bedrock or bearing strata. Then, the steel columns are installed along with its reinforcing cage tied to the column into the excavated trench. Concrete for the caisson is then placed to final sub-grade and the remaining trench is filled with granular fill or low strength flowfill [9]. In Figure 8, steel columns being installed are shown.

![Figure 8. Steel columns installed in their final location [9]](image)

5.1.3. SLAB CONSTRUCTION BY MINING METHODS

Once the slurry wall and the steel columns are installed, the top-down construction method itself begins. The first step of top-down construction is excavating to the sub-grade of level 1 and welding slab supports to steel columns. Then, the final slab is constructed, leaving a hole to proceed with the top down construction. After that, and using mining methods, soil is excavated under level 1 slab to the sub-grade of first basement slab and the same procedure followed to build level 1 slab is repeated [9], [10]. The excavation of the first below-grade slab by mining methods is shown in Figure 9.
All this procedure for the excavation and construction of level S7 slab has to be repeated for S6, S5, S4, S3, S2 slabs until final sub-grade is reached and level S1 slab is built. The Below-grade structure finished is shown in the Figure 10.
5.2. CHOICE OF TOP-DOWN CONSTRUCTION METHOD IN MARRIOTT MARQUIS

The possibility of realizing the upper-part of a building at the same time that the lower part is the main reason why Top-Down construction method is traditionally chosen. This possibility reduces the schedule of the construction, saving time and money. However, in the case of Marriott Marquis, top-down construction method was not chosen to save time during the construction of the hotel, and actually, the construction of the upper part of the building will not start until the above-grade part of the building is almost finished.

In the Marriott Marquis, top-down construction method was chosen because of the impossibility of digging the entire hole without excavation collapse. This collapse would be produced by the strong forces transmitted from the soil.

5.2.1. SOIL RETAINING SYSTEM IN MARRIOTT MARQUIS

The soil retaining system chosen for the Marriott Marquis is a diaphragm wall, a continuous structure formed and cast in a slurry trench. In this case the trench excavation is supported by bentonite slurry that prevents soil incursions into the excavated trench. Later, this slurry is replaced by concrete (injecting the concrete from the bottom to the top). The term “diaphragm walls” refers to the final condition, when it acts as a structural system, part of the permanent structure. This diaphragm wall has to be braced with traditional support systems such as tiebacks, rakers etc.

Due to the specific location of the Marriott Marquis, traditional support systems cannot be used in this construction site, and the Top-Down construction method was chosen to retain the soil during the excavation process. Specifically, the diaphragm wall is braced with the floor slabs, retaining the soil and avoiding the collapse of the excavation.

A review of some traditional lateral support methods during excavation and the reasons of why these methods do not apply in this construction have to be done. Finally, a brief review of top-down construction method is done as a soil retaining method in deep excavations.

Tiebacks:

In most deep construction sites, tiebacks are used to brace retaining walls in order to resist the lateral pressure of soil and stabilize the excavation. These tiebacks are usually horizontal wires or rods as well as helical anchors, secured to the wall in one end and penetrating the soil with sufficient resistance. The main advantages of this type of bracing are the availability of high-capacity anchor systems and the absence of interior obstructions that permit uninterrupted earth moving [12], [13]. In the Figure 11, a schematic sketch of a tieback is shown.
In the case of Marriott Marquis, with high lateral soil pressure these tiebacks should be really long. Due to the specific location of the Marriott Marquis these tiebacks cannot be installed, because perforation under Massachusetts Avenue is not allowed by the city council (due to the great number of utilities going through the street and also because of adjacency of existing building basements).

**Internal Steel Bracing:**

Temporary internal steel bracing can also be used to increase the resistance of retaining walls. In the case of internal bracing systems, the lateral earth (and water pressures) is transferred between opposing walls through compressive struts. The internal struts are installed from whaler (horizontal member placed against the soil support) to whaler. Usually, the struts are either pipe or I-beam sections and are usually preloaded to provide a very stiff system. Installation of the bracing struts is carried out by excavating soil locally around the strut and only continuing the excavation when preloading is complete.

Nevertheless, this kind of bracing is only used in narrow excavations, due to the large amount of steel that would be required in wider excavations. Marriott Marquis cannot be considered a narrow excavation, so the use of internal bracing was rejected.

Another possibility of internal bracing would be the use of raker bracing. Rakers are inclined structural struts that go from the bottom of the excavation to the whalers installed in the retaining walls. This option was also rejected because, once more, a large amount of steel would be needed to brace the retaining walls. In the case of Marriott Marquis, this amount of steel installed in the excavation would block the construction and interfere in the building work [13].
Top-Down Construction Method:

In the case of the Marriott Marquis the top-down construction method has been chosen because of its possibility of being used as a soil retaining method in deep excavations. In this case, the basement floors are constructed as the excavation progresses, being these floors the structural elements that braces the diaphragm wall (slurry wall). Top-down construction method is used for deep excavation projects where the tieback installation is not feasible and soil movement has to be minimized, being the case of the Marriott Marquis.

5.3. **UNIQUE ASPECTS OF A CONCRETE BUILDING WITH STEEL**

The Marriott Marquis is located in Washington DC. Most of the buildings of this city are made of concrete, due to some city height limitations. In this project, although the above-grade structure of is also made of concrete, a large amount of steel has to be used in the below-grade structure due to the particularities of top-down construction and for the long spans produced by the grand ballroom of level S5 and the two junior ballrooms of level S3.

5.3.1. **HEIGHT LIMITATION LAW IN WASHINGTON DC**

The main reason to use concrete instead of steel to build in Washington DC is the height limitation law that prohibits the construction of tall buildings and skyscrapers in the city.

Contrary to popular belief, this law does not restrict buildings to the height of the United States Capitol or the Washington Monument, although it was conceived to preserve the grandeur of these and others key locations. The current law about height limitation is from 1910 and says that no new building may be more than 20 feet taller than the width of the street in front of it (so height limitation changes from one location to another location around Washington DC). This current law is codified as D.C. CODE ANN. § 6-601.05 [14].

The choice of concrete instead of steel caused by the height limitation is due to the difference between floor-to-floor height when building a concrete or a steel building. Concrete structures will typically have a shallower floor depth (8” to 12” thickness), reducing the total height of the building. On the other hand, steel structures are faster and some times more economic, but have deeper floor-to-floor heights (14” to 24” thickness).

Nevertheless, due to the height limitation low, the difference between floor-to-floor heights of a concrete building and a steel building is mainly important in the above-grade part of the building. In this project, the structural steel has been used in the below-grade structure, and it is used in the temporary steel columns, to reinforce the below-grade slabs (with steel plates and embedded beams) and to frame S5 and S3 slabs.
5.3.2. **STEEL COLUMNS**

The second step in the top-down construction method is to install the final columns into the ground (before any other excavation takes place). These columns are around 100 foot long and there is no possibility of building these long columns of concrete and put them in place. Thus, the use of steel columns is mandatory when using top-down. These steel columns will finally be encased during the process of construction.

5.3.3. **BELOW GRADE SLABS**

Top-down construction is used in this building to brace the diaphragm wall. Because of the forces transmitted from the soil to the slabs, steel plates to resist the shear forces and embedded steel beams to resist the moment and the axial forces in Figure 12 can be seen a slab portion reinforce with a steel plate and an embedded beam.

![Figure 12. Detail of below-grade slab reinforced with steel plate and an embedded plate](image)

5.3.4. **STEEL USED TO FRAME THE BALLROOMS**

S3 hosts two junior ballrooms while S5 hosts a grand ballroom. These ballrooms have longer spans than other typical floors. Thus, special steel framing is required in levels S5 and level 1. Level S5 includes steel trusses and level 1 long plate girders, specially designed for this project.
6. BELOW GRADE COLUMNS DESIGN

Below-grade column design in Marriott Marquis is directly impacted by the use of top-down construction method.

In this project, below-grade columns are composite (steel plus concrete) columns in their final state, and above-grade columns are concrete columns. Nevertheless, due to the use of top-down construction method, below-grade columns are steel columns during the early stages of the construction of the building, because there is no possibility of pouring the concrete when the columns are installed (in the second stage of top-down method). Thus, below-grade columns shall be designed as steel columns for the early stages of the construction and as composite columns for the project final state. This consideration affects the design of these columns, because they have to be designed for these two different situations, adapting materials and loads to the design. In these conditions, some assumptions have been made to design the columns.

The first assumption is that below-grade columns are steel columns during the construction until level 10 of the building is reached. This assumption was made to minimize steel column size (not considering the 15 levels of the building) based on discussions of sequence and schedule with the contractor. Once level 10 is finished and the construction of level 11 starts, these columns are no longer considered steel columns, but composite. Thus, during the construction, below-grade columns shall be encased before the construction reaches level 10. In this case it has been considered that columns are all encased at the same time, situation that will not occur in the field.

In the field, steel columns are installed in their final position as one of the early stages of top-down construction method. They are encased floor by floor as the construction progress, meaning that once the slabs of level S7 and S6 are poured, the part of the columns between these two levels is encased. Thus, when the slab of level S5 is poured, the part of the columns between S6 and S5 is encased, and so on.

6.1. DESIGN OF BELOW-GRADE COLUMNS AS STEEL COLUMNS

In the early stages of the construction of Marriott Marquis below-grade columns are steel columns instead of composite columns (their final state). These columns have to be designed as steel columns during the construction, assuming that the columns are encased when the level L10 is finished. Thus, loads considered for the design are those relative to the construction (live load during temporary condition is construction live load, less than final) and only the first 10 levels are taken into account. All below grade columns have been design individually and following the same process. All the calculations are done according to the AISC steel construction manual [15], the 2009 International Building Code [16] and the Steel design code by Thomson [17].
6.1.1. CONSTRUCTION LOADS FROM LEVEL 1 TO LEVEL 10

First of all, the total gravity load from Level 1 to Level 10 during the construction has been calculated. The design of the above-grade structure was not realize by our company, but we were allow to use the data of the structural engineering company realizing this design. To calculate these loads, a spreadsheet as the one on Figure 13 was used for each single column.

![Figure 13. Screen capture of the spreadsheet use to calculate the construction load of column B.9-3](image)

The construction load needed to design steel below-grade columns is the cumulative ultimate load. To find this load, the load produced by each floor is calculated and all these loads are finally added, applying the necessary load factors.

The inputs introduced in the spreadsheet to calculate the dead and live load of each floor are the tributary load of this column (the area that contributes to their loading), the area of the column, the distributed dead load and the distributed live load. Then, service dead load (D.L) of each floor is calculated as:

\[
D.L = \text{Tributary area} \times \text{Distributed dead load} + \text{Concrete column weight} \quad \text{(ec. 1)}
\]

And the service live load (L.L) as:

\[
L.L = \text{Tributary area} \times \text{Distributed live load} \quad \text{(ec. 2)}
\]

Then, adding the dead load of each floor, the cumulative service dead load (C.D.L) is obtained and the same process is done to obtain the cumulative service live load (C.L.L). Thus, the cumulative ultimate load is calculated as:

\[
C.U.L. = 1.4 \times C.D.L. + 1.7 \times C.L.L \quad \text{(ec.3)}
\]
The distributed dead load considered during the construction for each floor varies based on member self weight while the distributed live load considered is temporary construction live load.

### 6.1.2. SIZE OF STEEL COLUMNS

A new spreadsheet for each below-grade column has been used to calculate the column size when it is considered a steel column. An example of this spreadsheet for column B.9-3 can be seen in Figure 14.

![Figure 14. Screen capture of the spreadsheet used to calculate below-grade columns loads](image)

In this spreadsheet, the global inputs that have to be introduced are the number of floors, the elevation of the bottom floor, the Live Load Reduction Factor (LLRF) code used and KLL, a factor used to calculate the live load reduction.

Once the global inputs have been introduced, some inputs relatives to each column and to each floor are also introduced. These inputs are the story height, the tributary area of each column, the distributed dead load and the distributed live load (both for each level slab). In addition, in the level L1, the cumulative ultimate dead load and the cumulative ultimate live load calculated from level 1 to level 10 are added as miscellaneous dead load and miscellaneous live load. Thus, cumulative dead and live load are calculated as:

**Cumulative dead load for level 1:**

\[
\text{Tributary area } \times \text{ Distributed dead load } + \text{C.D.L from level 1 to level 10} \quad \text{(ec. 4)}
\]

**Cumulated dead load for level } n: \]

\[
\text{T.A. (level n) } \times \text{ Distributed D.L. (level n) } + \sum_{\text{Level } 1}^{\text{Level } n-1} \text{Cumulative dead load} \quad \text{(ec. 5)}
\]

**Cumulative live load for level 1:**

\[
\text{Tributary area } \times \text{ Distributed live load } + \text{C.L.L from level 1 to level 10} \quad \text{(ec. 6)}
\]

**Cumulated live load for level } n: \]

\[
\text{T.A. (level n) } \times \text{ Distributed L.L. (level n) } + \sum_{\text{Level } 1}^{\text{Level } n-1} \text{Cumulative live load} \quad \text{(ec. 7)}
\]
Tributary area x Distributed live load +C.L.L from level 1 to level 10  
(ec. 5)

Cumulated dead load for level n:

T.A. (level n) x Distributed L.L. (level n) + \( \sum_{i=1}^{n-1} \) Cumulative live load  
(ec. 6)

Once each floor cumulative loads are calculated, three different cumulative ultimate loads are calculated varying the value of the coefficients C1 and C2 of the expression:

\[ C_1 \times \text{Cumulated dead load} + C_2 \times \text{Cumulated live load} \]  
(ec. 7)

The different coefficients applying, depending on the code, are shown on Table 1.

<table>
<thead>
<tr>
<th>Code</th>
<th>C1</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASD</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>ACI99</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>LRFD</td>
<td>1.2</td>
<td>1.6</td>
</tr>
</tbody>
</table>

Table 1. Coefficients applying in equation 7 depending on the code used

It is interesting to remark that in this case, the distributed dead load used varies from floor to floor and from column to column. These differences in the distributed dead load are caused by the different slab thickness and because some parts of the slab are reinforced with embedded beams and steel plates. In the other hand, the distributed live load remains constant, being 100 psf.

Once the loads are calculated, the last data needed are the clear height in the strong direction as well as in the weak direction.

With all these data, the available strength in axial compression that each column needs is known, and it is only necessary to find out which column size can provide this strength.

This step was done manually with the use of the AISC steel construction manual [15]. In this manual it is said that, for compression, the requirement is:

\[ P_u < \varphi_c \times P_n \]  
(ec. 8)

Where \( P_u \) is the cumulative ultimate load, \( \varphi_c \) the resistance factor for compression (0.9 in LRFD) and \( P_n \) the nominal compressive strength. In this project, \( P_u \) is the load previously calculated and the value \( \varphi_c \times P_n \) of any column size can be found in the Table 4-1 of the AISC steel construction manual [15]. It is important to remark that each column is divided into two different parts, one from level S1 to level S4, and other from level S4 to Level L1 due to designed splice location. Thus, two different column sizes can be used, as seen in Table 2.
<table>
<thead>
<tr>
<th>Floor</th>
<th>Clear Height (Strong Direction) ft</th>
<th>Clear Height (Weak Direction) ft</th>
<th>Size of steel column</th>
<th>Pu (cumulative ultimate load) ksi</th>
</tr>
</thead>
<tbody>
<tr>
<td>S7</td>
<td>12,0</td>
<td>12,0</td>
<td>W14x257</td>
<td>1049</td>
</tr>
<tr>
<td>S6</td>
<td>15,0</td>
<td>15,0</td>
<td>W14x257</td>
<td>1256</td>
</tr>
<tr>
<td>S5</td>
<td>15,0</td>
<td>15,0</td>
<td>W14x257</td>
<td>1463</td>
</tr>
<tr>
<td>S4</td>
<td>15,0</td>
<td>15,0</td>
<td>W14x283</td>
<td>1670</td>
</tr>
<tr>
<td>S3</td>
<td>16,0</td>
<td>16,0</td>
<td>W14x283</td>
<td>1877</td>
</tr>
<tr>
<td>S2</td>
<td>11,0</td>
<td>11,0</td>
<td>W14x283</td>
<td>2015</td>
</tr>
</tbody>
</table>

Table 2. Column sizes definition for column D.5-8.9

As an example, in column D.5 – 8.9 level S2 cumulated LRFD load (Pu) is 3249 kips and cumulated load in level S4 (Pu) is 2615 kips. If we use the AISC steel construction manual [15] for an effective length of 11 feet (as in level S2) a column size of W14x283 has an available strength in axial compression LRFD (ϕc Pn) of 3480 kips. In the other hand a W14x257 with an effective length of 15 feet (as in level S4) has an available strength in axial compression (ϕc Pn) of 2960 kips. Thus, the results are shown in Table 2.

6.2. DESIGN OF BELOW-GRADE COLUMNS AS COMPOSITE COLUMNS

Below-grade columns are composite columns in the final state of the project. These columns have been designed as steel columns for the early stages of the construction and later they have to be designed to resist the final loads once they are encased. To design the composite columns some assumptions have also been made. All the calculations are done following the AISC steel construction manual [15] and the ACI building code [18].

6.2.1. FINAL LOADS FROM LEVEL 1 TO LEVEL 15

First of all, the cumulative ultimate load from level 1 to level 15 for each below-grade column has to be found. To calculate this load in each column, the process followed is similar to the case of the steel columns. In the case of the steel columns, the cumulative load came from level 1 up to level 10, whereas in this case, the cumulative load comes from level 1 to level 15. The tributary areas of each column remained constant from the case of steel columns, as well as the distributed dead load used in each floor. Nevertheless, the distributed live load used in this case is bigger than the one used for the steel columns. This increase of the distributed live load is due to the columns are considered to be made of steel during the construction of the building, so the live loads are controlled. Nevertheless, below-grade columns are considered to be composite columns in the final state of the buildings, when the live loads are not so easily controlled. To calculate this load, a new spreadsheet as the one of Figure 15 has been created for each column.
6.2.2. SIZE OF COMPOSITE COLUMNS

Once the loads from level 1 to level 15 are obtained, a spreadsheet as the one of Figure 16 is used to design each composite column. The design of each composite column is not done by traditional methods and some assumptions are made due to the special characteristics of top-down construction method.

Figure 15. Screen capture of the spreadsheet use to calculate the final load of column B.9-3 above grade

Figure 16. Screen capture of the spreadsheet use to calculate below grade columns load
To design each column, the first inputs needed are the number of floors, the elevation of the bottom floor, the Live Load Reduction Factor code used and KLL, a factor used to calculate the live load reduction.

Once the global inputs have been introduced, some inputs relating to each column and to each floor are also introduced. These inputs are the story height, the tributary area of each column, the distributed dead load, the distributed superimposed dead load and the distributed live load (both for each slab level). In addition, in the level 1, the cumulative ultimate dead load and the cumulative ultimate live load calculated from level 1 to level 15 in their final state are added as miscellaneous dead load and miscellaneous live load. Then, following the same methodology as for the steel columns, the cumulative load for each floor is calculated. This cumulative load is calculated applying different coefficients to dead and live load, using the ASD, ACI 99 and LRFD combinations.

The same spreadsheet has been used to design the size and reinforcement needs of the concrete encasement. As shown in Figure 17, data about the column encasement as well as about the steel reinforcement have to be introduced.

![Figure 17. Screen capture of the cells used to introduce data to design the concrete encasement](image)

These data are the length corresponding to steel flange (B), the length corresponding to steel depth (H) and the concrete compressive strength. With these data, the young modulus of concrete is calculated as:

\[ E_c = 5700\sqrt{f'_c} \]  
\[ \text{(ec. 9)} \]

The concrete moment of inertia (strong direction) as:

\[ I_{c\text{ Strong}} = \frac{BH^3}{12} \]  
\[ \text{(ec. 10)} \]

and the concrete moment of inertia (weak direction) as:

\[ I_{c\text{ Weak}} = \frac{HB^3}{12} \]  
\[ \text{(ec. 11)} \]
Data about the reinforcement bars are also introduced. These data are the area of steel reinforcement ($A_{sr}$) and the diameter of the bars ($D_{bar}$). Then, the steel moment of inertia (strong direction) is calculated as:

$$I_{s\text{ Strong}} = A_{sr} \left( \frac{H}{2} - 1.5 - 0.5 - \frac{D_{bar}}{2} \right)^2$$  \hspace{1cm} (ec. 12)

The steel moment of inertia (weak direction) is calculated as:

$$I_{s\text{ Weak}} = A_{sr} \left( \frac{B}{2} - 1.5 - 0.5 - \frac{D_{bar}}{2} \right)^2$$  \hspace{1cm} (ec. 13)

At this point, some assumptions have been made to check the capacity of composite columns.

When designing a composite column, the required strength capacity is given by the total strength available in the concrete and the total strength available in the column.

In this case, the procedure of loading of the concrete columns is different from the standard loading process (when the load comes to the steel and to the concrete at the same time). Below-grade columns are, first of all, steel columns and they are charged with the construction loads from the above-grade levels from L1 to L10. Later, as composite columns, they are charged with the rest of the load.

Thus, to design the composite columns, it has been assumed that the strength provided by the steel before the columns are encased is not later re-distributed to concrete. When calculating the composite column, it has been considered that the yield strength is reduced from 50 ksi to $(50 - \text{the strength already taken by the steel column})$ ksi. Thus, each column is calculated with this reduced yield strength ($F_{y}'$).

In order to not over-estimate the size of the column the cumulative ultimate load was reduced from the cumulative ultimate load calculated from floor 1 to floor 15. Thus, as the yield strength capacity of the steel has been reduced, the loads taken by the steel have been subtracted from the cumulated LFRD load. The assumption made was that:

Cumulative LRFD used load = C.U. LRFD Load – C.D.L from level 1 to 10  \hspace{1cm} (ec. 14)

With these two assumptions (reduction of yield strength capacity and reduction of cumulated LRFD load) the composite column check is made by the traditional method, by the use of a spreadsheet shown in the Figure 18.

![Figure 18. Screen capture of the spreadsheet used to check a composite column](image)
As well as in the steel column, in a concrete column it has to be checked that:

\[ Pu < \varphi_c \ Pn \]  \hspace{1cm} (ec. 15)

Where \( Pu \) is the cumulative ultimate load, \( \varphi_c \) the resistance factor for compression (0.75 for a composite column) and \( Pn \) the nominal compressive strength of a composite column. The parameters needed to find \( Pn \) are the following:

The area of steel:

\[ As = \text{Area of steel column} \]  \hspace{1cm} (ec. 16)

The area of concrete:

\[ Ac = BH - As - Asr \]  \hspace{1cm} (ec. 17)

The coefficient \( C_1 \):

\[ C_1 = \text{MIN}\left(0.3; 0.1 + 2 \cdot \frac{A_s}{A_s + Ac}\right) \]  \hspace{1cm} (ec. 18)

The efficient EI (young modulus per the moment of inertia) in the strong axis:

\[ EI_{\text{eff}} S = E_s I_{x \text{steel column}} + \frac{E_s A_{sr \text{Strong}}}{2} + C_1 E_c I_c \text{Strong} \]  \hspace{1cm} (ec. 19)

The efficient EI (young modulus per the moment of inertia) in the weak axis:

\[ EI_{\text{eff}} W = E_s I_{y \text{steel column}} + \frac{E_s A_{sr \text{Weak}}}{2} + C_1 E_c I_c \text{Weak} \]  \hspace{1cm} (ec. 20)

Then the nominal axial strength at zero eccentricity \( P_0 \) is calculated as:

\[ P_0 = As F_y + 60 Asr + 0.85f'c \ Ac \]  \hspace{1cm} (ec. 21)

Then, the Euler Buckling Load in the strong direction is calculated as:

\[ P_{e \text{Strong}} = \frac{\pi^2 EI_{\text{eff}} S}{(12 \text{ Clear Heigh}\ S)^2} \]  \hspace{1cm} (ec. 22)

And:

\[ P_{e \text{Weak}} = \frac{\pi^2 EI_{\text{eff}} W}{(12 \text{ Clear Heigh}\ W)^2} \]  \hspace{1cm} (ec. 23)

Finally, the value of the Nominal Compressive Strength \( Pn \) has to be found for the strong and for the weak axis.

For the strong axis, if \( P_{e \text{Strong}} > 0.44 P_0 \) then:

\[ P_{n \text{Strong}} = P_0 \frac{P_0}{0.658 P_{e \text{Strong}}} \]  \hspace{1cm} (ec. 24)

Otherwise:

\[ P_{n \text{Strong}} = 0.877 P_{e \text{Strong}} \]  \hspace{1cm} (ec. 25)
For the strong axis, if $P_{e \text{ Strong}} > 0.44 P_0$ then:

$$P_{n \text{ Weak}} = P_0 0.658P_{e \text{ Weak}}^{P_0} \quad (\text{ec. 26})$$

Otherwise:

$$P_{n \text{ Weak}} = 0.877P_{e \text{ Weak}} \quad (\text{ec. 27})$$

Then, the final value of $P_n$ is the minimum between the values of $P_{n \text{ Strong}}$ and $P_{n \text{ Weak}}$.

With all this values, it is checked for each column and in each stage that

$$P_u < \varphi_i P_n \quad (\text{ec. 28})$$

If not, the values of $A_{sr}$, $D_{bar}$, $B$ and $H$ shall be modified.

### 7. BELOW-GRADE SLAB DESIGN

The complexity in the design of the below grade slabs is one of the unique aspects to consider in this project due to the use of top-down construction method.

The diaphragm wall (slurry wall) used as a retaining wall has to resist the soil pressure during the construction and when the hotel is finished. This diaphragm wall uses the slabs from S1 to S7 as bracing system so the slabs have to be designed to resist the shear forces transmitted from the slurry wall, and not only the gravity loads as usual.

Below grade slabs are made of concrete. The steel reinforcement has to be designed to resist the shear forces transmitted from the soil to the diaphragm wall.

#### 7.1. SHEAR FORCES TRANSMITTED FROM THE SLURRY WALL TO THE SLABS

The first step in order to find the steel reinforcement in the concrete slabs was to determine the shear forces transmitted from the slurry wall to the slabs.

To find the shear forces in each floor slab, the forces transmitted from the soil to the slurry wall must be considered. As these forces are not equal in the entire building perimeter, the perimeter was divided into 6 different sections as shown in Figure 19. Each one of these sections have different soil forces and conditions (such as different floor levels) and the calculation to find the shear forces transmitted to the slab is done for each of the 6 sections.

Once the perimeter has been divided, the shear forces transmitted from the diaphragm wall to the slabs have been calculated for 7 different load cases, depending on the wall load
conditions. These 7 different wall load cases are due to the top down construction method [19].

As the construction progress, the use of the top-down method, changes the load conditions in the wall. The first step in the top-down method is the installation of the slurry wall, before any other excavation begins. In this situation, the slurry wall is covered by soil at its two sides and the soil pressure is compensated. In this case no load is applied to the wall.

![Figure 19. Different soil pressure sections](image)

When the excavation of the first below grade floor begins, one of the wall sides starts to be loaded by the soil pressure. This load changes as the excavation continues. To determine the shear forces transmitted to the slabs during this process, the wall has been considered as a concrete beam in which every slab (once it has been installed) acts as a support [19]. The reaction of this support (slab) at the different load cases is considered as the shear force transmitted to the slab.

To simplify the calculation, 7 load cases have been considered. The first load case is when the soil is excavated until level S7 and the slab of this floor is installed. In this case, this is equivalent to a cantilever beam with just one support.

Thus, each new load case corresponds to a new level excavation and the installation of its slab. The second load case corresponds to the moment when the excavation reaches level S6 and the slab of this floor is installed. In this case, the problem to solve is equivalent to one beam
that spans from one support (slab S6) to another (slab S7) and that cantilevers (until the end of the slurry walls) as seen in Figure 20. The load cases from 3 to 6 correspond to the excavation and installation of levels S5, S4, S3 and S2 slabs. The equivalent structure to solve is the same than in the load case number 2 adding a new span in each load case.

![diaphragm wall](image)

**Figure 20. Equivalent structure number 2 for the soil model**

The last load case (also called at rest condition) corresponds to the final wall load condition. In this case the equivalent structure to solve is a beam (slurry wall) that has 6 spans (the supports are the 7 floors slabs) and a right cantilever (from S7 to the top of the wall), as shown in ¡Error! No se encuentra el origen de la referencia.1.

![diaphragm wall](image)

**Figure 21. Equivalent structure number 7**

With all these data, a spreadsheet (one for each of the 6 different perimeter section considered) provides the result of the calculations. An example of spreadsheet can be seen in the ¡Error! No se encuentra el origen de la referencia.2.

![Spreadsheet](image)

**Figure 22. Summary of staging analysis (Section C) [19]**
In this spreadsheet, the 7 load cases correspond are listed as stages from 2 (first load case) to 8 (end of below-grade excavation) each one corresponding to a different row. The first 8 columns represent the horizontal reactions at slab level (so the shear forces transmitted to each slab), one for every slab level (El. +77 corresponds to Level 1 slab and EL. -16.5 to Level S1 slab). Level 1 slab doesn’t participate in the bracing of the slurry wall, so any result is extracted. In the other columns the slab reaction for every load case is found, being remarked the higher reactions for every slab. Finally three columns provide the maximum wall moment (inside and outside) and the maximum wall shear for the equivalent structures described above in each load case.

7.2. SAP [3] SLAB MODEL

The next step to determine the steel reinforcement needed in each floor was done by creating a SAP model of each floor. SAP [3] is software that allows analyzing any structure using the finite elements method. Thus, each slab was modeled by the use of finite elements and then analyzed. In ¡Error! No se encuentra el origen de la referencia. SAP model from slab S5 is shown.

Figure 23. SAP model of slab S5
Once each slab is modeled, the loads have to be introduced. Seven load cases were created, one for each load case previously considered, and the shear force found in the previous analysis were introduced into the model.

In order to calculate the steel reinforcement, it is necessary to know the maximum factored axial force $Pu$, the maximum factored moment $Mu$ and the maximum factored shear force, $Vu$. These maximum values change substantially from one section to another of the slab. The use of SAP allow the designer to create as many section cuts as required, to find the values of maximum moment, axial force and shear force in each section.

Thus, once each slab was modeled in SAP [3] and the loads were introduced, several section cuts were introduced in each model. The location of these section cuts was chosen in these places having special conditions (narrower slabs, openings etc.). All these section cuts were labeled, and the maximum axial force, shear force and moment obtained for each section and for each load case. An example of how section cuts are distributed can be seen in Figure 24.

![Figure 24. Section cuts introduced in the SAP model of Slab S6](image-url)
7.3. SLAB REINFORCEMENT TO RESIST SHEAR FORCES

Each floor slab has to be designed to resist the shear forces transmitted from the slurry wall. The American Concrete Institute (ACI) 318 Building Code [18] says that the design of cross sections subject to shear shall be based on:

\[ \varphi V_n \geq V_u \]  \hspace{1cm} (ec. 29)

Where \( V_u \) is the factored shear force at the section considered and \( V_n \) is the nominal shear strength computed by:

\[ V_n = V_c + V_s \]  \hspace{1cm} (ec. 30)

Where \( V_c \) is nominal shear strength provided by concrete and \( V_s \) is nominal shear strength provided by shear reinforcement. According to the code, the minimum value of \( V_c \) (unless a more detailed calculations is made) shall be computed as:

\[ V_c = 2 \sqrt{f_{\text{cb}} b_w d} \]  \hspace{1cm} (ec. 30)

This value of \( V_c \) is conservative. In the Marriott Marquis project, the slabs have been considered as walls. This assumption has been done due to the deepness of the slabs that is equivalent to a wall and the value of \( V_c \) has been calculated to resist shear in plane of wall. In this case, the value of \( V_c \) is permitted to be the lesser of the values computed from

\[ V_c = 3.3 \sqrt{f_{\text{c}} t d} + \frac{N_u d}{4 l_w} \]  \hspace{1cm} (ec. 31)

Or

\[ V_c = \left[ 0.6 \sqrt{f_{\text{c}}} + \frac{l_w \left(1.25 \sqrt{f_{\text{c}}} + 0.2 \frac{N_u}{l_w} \right)}{\frac{M_u}{V_u} \frac{l_w}{2}} \right] t d \]  \hspace{1cm} (ec. 32)

Where \( l_w \) is the length of the section cut, \( t \) the thickness of the slab, \( d \) shall be taken equal to 0.8\( l_w \) and \( N_u \) is positive for compression and negative for tension. If \( (M_u/V_u - l_w/2) \) is negative, this last equation shall not apply.

The values of the shear strength provided by concrete have to be calculated for each section cut in all the slab floors. Thus, a spreadsheet was created for each floor slab in order to find the slab reinforcement. In these spreadsheets, represented in ¡Error! No se encuentra el origen de la referencia.S each row represents a different section cut in a different load case, so seven rows for each section cut.
The first data introduced in the spreadsheet is the yield strength of the steel used ($f_y$), the concrete compressive strength ($f'_c$), and the value of $\phi_{\text{Shear}}$.

Then, in each row (representing a section cut in a load case) the results obtained in the SAP [3] model are introduced. These results obtained are the thickness of the slab ($t$), the length of the section cut ($l_w$), the factored shear force ($V_u$), the factored axial force ($N_u$) and the factored moment ($M_u$).

In this spreadsheet the first value to be checked is the minimum thickness of the wall. This is calculated as:

$$ t \geq \frac{V_u}{\phi_{\text{Shear}} \sqrt{f'_c d}} $$

(ec. 33)

According to the ACI building CODE [18], $V_n$ at any horizontal section for shear in plane of wall shall not be taken greater than $10 \sqrt{f'_c d t}$. Thus, the minimum thickness has to be checked.

The following step is to calculate the values of $V_c$ per the equations of $V_c$ described above. The final value of $V_c$ is the minimum of the 2 values calculated to resist shear in plane of wall.

As per the ACI building code [18], where $V_u$ is smaller than $\phi V_c$, the minimum steel reinforcement is enough. In the case that $V_u$ exceeds $\phi V_c$ horizontal shear reinforcement shall be provided. In order to calculate the area of steel needed to reinforce the slab, the necessary shear strength provided by the steel is calculated as:

$$ V_s = \frac{V_u - \phi V_c}{\phi} $$

(ec. 34)

Once the value of $V_s$ is found, the area of horizontal shear reinforcing ($A_v$) required to reinforce the slab is found as the maximum value of:

$$ A_v = \frac{V_s s}{f_y d} $$

(ec. 35)

Or
Where $s$ is the spacing of the shear reinforcement, 12” in the case of the Marriott Marquis.

Once the theoretical value of reinforcement steel needed in each section is found, this value has to be studied. As said before, the value of $V_n$ at any horizontal section for shear in plane of wall shall not be taken greater than $10 \sqrt{f'c}d$ t. Thus, it has to be checked that:

$$10 \geq \frac{V_u}{\phi \sqrt{f'c}d}$$  \hspace{1cm} (ec. 37)

When, this equation is not fulfilled the section is reinforced with a steel plate and this steel plate will be found according to the steel construction manual. In the case that a plate is not required to reinforce the concrete steel, the diameter of the bars is found.

### 7.3.1. BELOW GRADE SLAB PLATES

Some parts of the slab need to be reinforced with a steel plate to resist the shear forces transmitted by the slurry wall. In this case, the plate is supposed to resist all the shear force transmitted from the wall, while the concrete does not resist any load. Thus, the American Institute of Steel Construction (AISC) code [15] is used to calculate the shear resistance of the plates. For the code, the selected plate has to satisfy the following equation:

$$V_u \leq \varphi V_n$$  \hspace{1cm} (ec. 38)

Where $V_u$ is the factored shear force at the section and $V_n$, the nominal shear strength, has to be found as per the AISC code [15]. To find the nominal shear strength and check if a chosen plate can resist the given shear force, a spreadsheet as the one in [Error! No se encuentra el origen de la referencia.6](#) is used.

![Spreadsheet](image.png)

**Figure 26.** Screen capture of the spreadsheet used to calculate the steel plates needed to reinforce slab S6.
In this spreadsheet, the global inputs introduced are the \( \varphi \) shear (0.9 LRFD as per the AISC code [15]), the young modulus of the steel and the yield strength of the steel used. In the Figure 27 a screenshot of a section cut needing a reinforcing plate is shown.

![Figure 27. Screen capture of the spreadsheet used to calculate steel plate reinforcement in section SCUT02 of S6](image)

All the plates used to resist the shear forces have studs, required to transfer load from concrete to plate and avoid buckling, so when a plate is needed, as in ¡Error! No se encuentra el origen de la referencia.7, the inputs introduced by the designer are the plate thickness \( t \) and the studs spacing in the plate \( s \). Then, the value of \( V_n \) for each plate is found following the same process.

First of all the relation \( h/t_w \) is found as:

\[
\frac{h}{t_w} = \frac{\sqrt{2}s}{t} \quad \text{(ec. 39)}
\]

In this case the value of \( h \) is \( \sqrt{2}s \) due to the studs being equally spaced at both directions of the plate, the bigger unbraced length is the diagonal between two rows of studs, as shown in ¡Error! No se encuentra el origen de la referencia.8.

![Figure 28. Unbraced length in steel](image)

Then, as per the AISC code [15], the value of the web shear coefficient \( (C_v) \) has to be determined following the Table 3.

<table>
<thead>
<tr>
<th>Case</th>
<th>Value of web shear coefficient ( (C_v) )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( h/t_w \leq 1.10 \sqrt{k_v}E/Fy )</td>
<td>1.0</td>
</tr>
<tr>
<td>( 1.10 \sqrt{k_v}E/Fy \leq h/t_w \leq 1.37 \sqrt{k_v}E/Fy )</td>
<td>( \frac{1.10 \sqrt{k_v}E/Fy}{h/t_w} )</td>
</tr>
<tr>
<td>( h/t_w &gt; 1.37 \sqrt{k_v}E/Fy )</td>
<td>( \frac{1.51 E_k_v}{(h/t_w)^2 F_y} )</td>
</tr>
</tbody>
</table>

Table 3. Value of the shear coefficient according to the AISC code [15]
Then, the value of $V_n$ is calculated as in the Table 4.

<table>
<thead>
<tr>
<th>Value of $C_v$</th>
<th>Value of nominal shear strength ($V_n$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>$0.6 \cdot F_y \cdot t \cdot l_w \cdot C_v \cdot (1 - 0.2)$</td>
</tr>
<tr>
<td>Otherwise</td>
<td>$0.6 \cdot F_y \cdot t \cdot l_w \cdot C_v \cdot (1 - 0.15)$</td>
</tr>
</tbody>
</table>

Table 4. Value of the nominal shear strength depending on $C_v$ according to the AISC code [15]

It can be noticed that in the case that $C_v$ is 0.5, the 80% of the total capacity is considered while for any other value of $C_v$, the 86% of the capacity is used. This reduction of the capacity is due to the plates will be installed by parts and welded. This weld reduces the strength resistance of the plates.

With this value of $V_n$ the resistance of the plate to the shear force can be checked. In the case that the resistance is not enough, the values of thickness and stud spacing can be modified.

Finally, to check the factored shear force ($V_{u,trib}$) that every stud has to resist (in order to choose the studs), the value of $V_u$, trib is calculated as:

$$V_{u,trib} = \frac{V_u}{l_w}$$  \hspace{1cm} (ec. 40)

Thus, the required brace strength ($P_{br}$) is calculated as:

$$P_{br} = 0.01 \cdot V_{u,trib}$$  \hspace{1cm} (ec. 41)

### 7.3.2. STEEL PLATES DIMENSIONS

Once the sections needing steel plate reinforcement are identified, the dimension of these steel plates has to be determined. To determine the dimension of these steel plates, the SAP [3] model is used.

Once the SAP [3] model is analyzed, a stress diagram in the slab can be drawn. As the location of steel plates has been determined before, to determine the size of each plate the stress diagram is used. Thus, when a steel plate is required, it is used in all the area that has the same stress than sections needing plate reinforcement. Furthermore, a development length is also required, because the plate cannot take all the stress from 0 to 100%, so the plate is also extended to a certain range of lower stresses.

In ¡Error! No se encuentra el origen de la referencia.9 can be seen a screenshot of a stress diagram from S6 SAP [3] model. In ¡Error! No se encuentra el origen de la referencia.30 the structural drawings from slab S6 where steel plates are needed is shown. Comparing these two figures, it can be seen the relation between the maximum stresses and the plates locations.
Figure 29. Screen capture of the stress diagram of S6 in SAP [3] model
Figure 30. Section of the structural drawing from slab S6 [8]
7.4. STEEL REINFORCEMENT TO RESIST FLEXURE AND AXIAL LOADS

Steel reinforcement has also to resist moment and axial forces, so it has to be checked if the steel reinforcement calculated to resist the shear forces is also valid to resist moment and axial forces. The axial force and the moment introduced by the soil in each section cut were also found with the SAP [3] model.

To check if the reinforcement used to resist the shear is enough to resist also moment and axial forces, the slab sections have been considered as columns under axial force and moment. Then, each of these columns has been checked with the software PCA Column [20].

7.4.1. PCA COLUMN [20] SOFTWARE

PCA Column [20] is software permitting to calculate the reinforcement in a concrete column under axial forces and moments.

Thus, as seen in ¡Error! No se encuentra el origen de la referencia.31, the first is to introduce the material properties for concrete (strength, elasticity, maximum stress, beta and ultimate strain) and for reinforcing steel (strength and elasticity).

![Material Properties]

Figure 31. Screen capture of PCA column [20], showing the window dialog to introduce Material properties

Then, the type of reinforcement has to be chosen. For the first attempt, the type of reinforcement chosen was “sides different” type, which has to be completed as in ¡Error! No se encuentra el origen de la referencia.2.
Finally, the factored loads obtained in the SAP [3] model for every different load case are also introduced. Each case load includes an axial load and Y-Moment.

Once all data have been introduced, PCA Column [20] creates the interaction diagram for the column, and checks if all load cases are inside the diagram. If this is the case, the steel reinforcement calculated to resist the shear force is enough. If not, as in ¡Error! No se encuentra el origen de la referencia.3, a new steel reinforcement is calculated.
The new reinforcement used to reinforce slab section that cannot resist the axial force and the moment is always a W steel beam. To simulate the steel beam in PCA Column [20], the area of steel of the column is introduced using irregular reinforcement as a type of reinforcement.

As an example, slab section 2 of floor S6 is reinforced with two W14x233 steel beams as shown in ¡Error! No se encuentra el origen de la referencia.4.

Figure 34. Detail of structural drawings W14x33 [8]

8. BELOW-GRADE STEEL FRAMING

Level S5 of Marriott Marquis includes a grand ballroom of 30,000 square foot. This ballroom is a big open space on Level S5 up to Level S7 with no columns. The large spans, due to the lack of columns inside the ballroom, requires steel framing to be used to support the slab of level S5 and the level L1 Slab. The steel framing used to support level L1 also has to support some of the concrete columns used in the above grade structure so plate girders have been specially designed to support the above grade structure. To avoid cumulative deflections in these plate girders as the construction goes up, plate girders number 2 and number 9 have been preloaded by the use of a jacking system attached to temporary columns. The preloading will minimize the column settlement, reducing the additional slab moments the structure must be designed for.

8.1. LEVEL S5 STEEL FRAMING

Level S5 slab is supported by steel framing due to the two junior ballrooms included on level S3. These two large open spaces require long span trusses to be used to frame the slab and a steel framing as the one in ¡Error! No se encuentra el origen de la referencia.5 has been designed with this objective.
Figure 35. Section of structural drawings of slab level S5, steel framing designed to support S5 slab [8]

The steel framing designed in this case is composed of trusses as the one in Figure 36. Detail of beams composing Truss 1 [8] that spans from grid lines MB, MC, MD and ME.

These trusses are framed with W18x35 beams every 9 feet as can be seen in Figure 36. Detail of beams composing Truss 1 [8].
8.2. DESIGN OF PLATE GIRDERS

The design of the plate girders and transfer girders of level 1, to support the long spans of level L5 ballroom was done in two steps.

First of all, a RAM model of level L1 steel framing was created. In this RAM model all the loads from the above-grade part of the building used to calculate the steel and composite below-grade columns, as well as other live loads included in level L1 were added. Thus, the maximum factored moment (Mu) and factored shear force (Vu) in each plate girders were found.

Then, a spreadsheet as the one in ¡Error! No se encuentra el origen de la referencia. was used to design each of the plate girders.

![Figure 37. Screen capture of the spreadsheet used to design the plate girders](image)

In this spreadsheet, the inputs introduced to calculate the moment resistance (ϕ Mn) and the shear force resistance (ϕ Vn) of the girder were the span of the girder, the depth of the section (d), the flange width (bf), the thickness of the loaded flange (tf) and the web thickness (tw). In ¡Error! No se encuentra el origen de la referencia. these values are represented.

![Figure 38. Detail of plate girder dimensions](image)

With these data, and following the AISC code [15], this spreadsheet calculates the resistance of the plate girder.
First of all, the basic properties of the plate girder are calculated. Then, following the AISC code [15], the spreadsheet analyzes the design of the plate girder for flexure (explained in the Appendix F of the AISC steel code [15]), checking the lateral torsional buckling and the flange local buckling. The analysis of the design of the plate girder to support the shear has also to be done (Appendix F of the AISC steel code [15]). As per the code, the nominal shear strength has to be calculated.

Once these analyses are completed, it has to be checked that for the shear as well as for the moment, these two equations are satisfied:

\[ Vu \leq \phi V_n \quad \text{and} \quad Mu \leq \phi M_n \]  

(ec. 41),(ec. 42)

If these two equations are satisfied, the plate girder designed can be used. In the case that these two equations are not satisfied, the dimensions of the plate girder can be modified until the requirements are satisfied.

8.3. PRE-LOADING OF PLATE GIRDERS PG2 AND PG9

When a plate girder is used to solve a long span in a building, the deflections of these plate girders have to be controlled, in order to not exceed a certain acceptable level.

What is done in these cases is to slightly camber the plate girder (create an upward deflection of the girder) so when the plate girder deflects, this concavity disappears and the beam comes down to its supposed position under building self weight. In the case of Marriott Marquis, the camber of each beam is 80% of the previously calculated self weight deflection. The deflection is not taken at 100% because in the case that the forces are lower than predicted, it is preferable to have a small downward deflection than an upward concavity in the slabs.

Plate girders PG2 and PG9 used to frame the level L1 will support also some columns from the above grade part of the building. In this case, the deflections in the girder have to be controlled during the construction process, in order to avoid large deflections when the building is completed.

The reason why deflections have to be avoided in these two plate girders is that, as these two plate girders support concrete columns from the above grade structure, as the construction goes up, the load in the girders will be increased and consequently the deflections. With the increase of these deflections, the above grade columns will displace down, and the slabs of these columns will slope. As the construction continues, the load increases (so the deflection) and the slabs continue sloping. To avoid slabs to slope and maintain its original position, the deflection of the plate girders PG2 and PG9 has to be eliminated.

The solution chosen to avoid the deflection was to pre-load the camber plate girder through 4 jacks attached to temporary columns (on grids MB, MC, MD, and ME). The pre-loading forces applied for the jacks simulate the final loads applied to the plate girder, charging the girder before any above construction starts. Then, once the construction progresses and the real
loads are applied to the girders, the force made by the jacks will be reduced as the real loads increase. At the end, when the final loads are applied, any force will be made by the jacks and these (As well as the temporary columns) can be dismantled.

8.3.1. CALCULATION OF JACK FORCES TO PRELOAD THE PLATE GIRDERS

In order to avoid deflection in the plate girders during the construction process, PG2 and PG9 will be preloaded by 4 provisional jacks simulating their final load and deflections. The design of the pre-loaded plate girders assumes that the preload force is concentric to the plate girder centerline and that the 4 jacks apply the same force to the plate girder. The process followed to find the preload-force needed to simulate the final deflection was the following.

First of all, a RAM model [2] was created with the loads applied to the plate girder. Then, with the ram model, we found the point loads applied to the plate girders, dividing the loads of the plate girder into 30 different point loads. Once the point loads were found, a spreadsheet was created in order to find the preload forces needed. The Figure 39 shows a screenshot of the spreadsheet used to find the preloading forces.

![Figure 39. Screen capture of the spreadsheet used to find the preloading forces](image)

In this spreadsheet, the global inputs introduced were the length of the girder, the modulus of elasticity and the moment of inertia of the girder. Then, the 30 point loads found in the RAM model [2], are introduced in different rows of the spreadsheet, including its value in kips and the distance from the beginning of the line. Then, the deflection due to this point load in the four points where the jacks are installed is calculated as shown in Table 5. The distance a shown in Table 5 is shown in Figure 40.
Once the deflections produced by each of the 30 point loads are calculated in the points distant 27.1 ft, 54.2 ft, 81.3 ft and 108.4 ft, these deflections are added in order to obtain the final deflection in the girder.

Then, using the same equations, it is simulated the deflection created by 4 point loads (the load value it is assumed equal in the 4 points) at grids MB, MC, MD and ME. The final forces are obtained minimizing the difference between the deflections created by the real forces, and the deflections created by the temporary jacking columns.

With this assumption, in both plate girders (PG2 and PG9) the force applied to the plate girders have to be 675 kips in each jack.

### 8.3.2. PRE-LOADING SEQUENCE FOR PLATE GIRDERS PG2 AND PG9

The pre-loading sequence for plate girders PG2 and PG9 during the construction is the following.

First of all, all the structural steel at level 1 including floor framing, transfer girders, plate girders and diagonal bracing has to be erected. Then, and following the sequence of top-down construction method, the concrete two-way slab from north of grid MA has to be poured. This slab is used to brace the ends of the long span plate girders along grid MA.

The next step is connecting the jacking assembly between plate girders PG2 and PG9 and the respective temporary plunge columns below as indicated in ¡Error! No se encuentra el origen de la referencia.1.
Once the jacking assembles are connected, the plate girders PG2 and PG9 have to be preloaded with a force of 675 kips in each of the 4 points indicated in the Figure 41. The expected deflection of the plate girders under the full-preload at the jacking locations are also shown on Figure 41. During the construction, the jacking force and the plate girder deflections shall be measured and recorded to control that the actual deflections don’t deflect more than a 10% from the expected ones. Then, the level 1 composite slab have to be poured (the upper level and lower level slabs have to be part of the same pour). The weight of these pours on the plate girders will help take out girder camber prior to above grade construction.

Once this process of pre-loading is completed, the above grade construction can be started. The self-weight of the above grade construction imposed on the plate girders PG2 and PG9 below will incrementally replace the preload force, resulting in a decreased force in the jack. During this process, the forces in the jack and the plate girder deflections have to be measured after each above grade floor is completed (Concrete pour). If the results provide to the Structural Engineer of Record were not the expected ones, the jack force could be adjusted.

Finally, after the roof topping out on both towers, the pre-loading jacking assemblies have to be dismantled and the extent of the temporary plunge columns above the S5 level removed.
9. CONCLUSION

The main objectives of this thesis were to study the top-down construction method and see how this construction method impacts in the below grade structure of a building.

With these objectives, the Hotel Marriott Marquis in Washington DC has been chosen as a case study, and after simulating the impact of the soil forces and the use of top-down construction on its structure, some conclusions can be drawn.

The first conclusion drawn is that when a deep excavation is done, the choice of the construction method can impact directly in the design of the structure. In this case study, the top-down construction method has been chosen because of the impossibility of bracing the diaphragm wall with any other system (after studying the other possibilities). The advantage of the top-down construction method is that it allows using the slabs as bracing system. As seen in the thesis, the construction method is then forced to be the top-down and the impacts on the structure are a major issue, comparing the steel reinforcing needed to build with top-down against down to top.

In the case study, the first part of the structure studied were the below grade columns. If this same building was not built with the top-down method, concrete columns could have been used. However, because of construction issues (the columns are placed before any other excavation take place), the use of steel columns is mandatory (they are the only columns that can be installed), and to reduce their size when the live load is applied they are encased. Taking into account these factors, first of all the below grade columns have been designed as steel columns (considering only the construction process) and as composite columns in their final state. Thus, and collaborating with the contractor, some assumptions have to be made. These assumptions are at what point columns are no longer supposed steel columns and they are considered concrete columns (decision that affects the size of the steel columns) and how the strength is later distributed between the concrete and the steel when they are composite columns (affecting the encasement of the columns). It has been studied that changing these two parameters, the steel size of the column and the concrete volume vary drastically.

The second part of the below grade structure of the Marriott Marquis studied were the below grade slabs, key point in this case study, because they have to resist the soil forces transmitted to the diaphragm wall and brace this wall. In this case, a SAP model has been created to simulate the effects of these forces on the slabs and the steel reinforcement has to be calculated at this effect. Thus, the steel reinforcement was designed first to resist the shear forces and then to resist flexure and axial forces. To resist the shear, the steel reinforcing needed was found, but it was noticed that in some cases, the strength added by the steel was higher than a maximum value allowed by the American Concrete Institute, so the reinforced planned was not allowed. When this happened, the decision of reinforcing the slab with a steel plate was made, using the SAP model to determine the dimensions of the steel plates and assuming and modeling how these plates will take the efforts transmitted by the concrete. In the case of flexure and axial forces, some models have also been created to simulate the behavior of the slab. It was noticed that in some areas the reinforcing added for the slab was not enough to resist flexure and axial forces. In these cases, the slab was considered as a
continuous beam in the direction perpendicular to the forces and the slab was reinforced with W beams.

The last part of the case study reflects how the use of the top down construction method can add possibilities in the design of the structure. Because of the use of top-down, some columns in their temporary condition are longer that in their final state (for instance in the case of the columns that support the ballrooms. Thus, it is investigated how some of these columns can be used to preload the plate girders PG2 and PG9, avoiding the hot or cold pre-cambering. The forces needed to preload these girders have been calculated to simulate the deflection due to the final loads and the results analyzed.

Finally, it has been concluded that for the case study of the Marriott Marquis Hotel and even if the impacts on the below-grade structure cannot be considered a minor issue, the advantages and the possibilities of the top-down construction in this case made that this construction method is chosen as the more efficient one.
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