Structural design considerations for super-tall buttressed core structure - the Burj Khalifa

by

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Abstract

The intent of this report is to illustrate the design and feasibility considerations for super-tall buttressed core structures in respect to the Burj Khalifa. Utilizing the latest construction surveying equipment and engineering practices has led to taller structures and more sophisticated methods of analysis. With realization of such heights, the Burj Khalifa poses as a benchmark for structural analysis methods and future structures.

Dynamic considerations for structures vary from many factors such as overall stiffness, damping, and mass. For better understanding of global behaviors, early integration of aerodynamic shaping and wind engineering played a major role in mitigating and taming the dynamic wind effects. Understanding the structural and foundation system behaviors of the tower were the key fundamental drivers for the development and execution of a state-of-the-art survey and structural health monitoring programs. These programs will be further described as well as their ability to measure accelerations, deflections, strains, concrete shortening, and settlements of structural members.

Furthermore, this report describes the foundation design process adopted for the Burj Khalifa. The foundation system is a compensated piled raft, founded on very heterogenous soil deposits. Geotechnical investigations, field and laboratory testing programs, and the design process will all discuss how various design issues were addressed. The various design issues addressed include: ultimate capacity, overall stability under wind and seismic loadings, and settlement and differential settlements.

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Chapter 1: Introduction

In the last two decades, building heights have grown significantly. Eighty-six of the world's 100 tallest structures were built within the last 22 years (Highsmith, 2021). Shown in Figure 1, the Burj Khalifa is currently the tallest structure in the world standing 828 meters (2,716.5 feet) tall located in Dubai, United Arab Emirates (UAE). With 200 plus stories and 160 habitable floors, the Burj Khalifa has a total area of 527 thousand square meters (5.67 million square feet) consisting of a hotel, corporate offices, and residential living space. Supertall buildings have very unique design considerations such that these buildings needed their own name. The term "Megatall" was coined by the Council on Tall Buildings and Urban Habitat (CTBUH) for a building 600 meters (1968.5 feet) or taller (Highsmith, 2021). Megatall buildings are different from traditional mid-rise structures as they need extensive wind analysis and dynamic behavior assessments for design of the lateral force resisting system (LFRS) and the foundation. This report examines how the structural behaviors and location shaped the buttressed core system utilized in the design of the Burj Khalifa.

The concept of building "Megatall" buildings is not new. On 16 October 1956, Frank Lloyd Wright unveiled his design for the mile-high (1609 meter) tripod spire. Wright's vision was to spread urbanization upwards rather than outwards – 'a city-within-a city' (Lobner, 2020). This single skyscraper would reduce the amount of land occupied by a building and eliminate the need for other large skyscrapers in its vicinity. This concept was sought to mitigate a cities geographic footprint while preserving the natural environment around it. Similar concepts are becoming realized by "The Line" in Saudi Arabia. While the Burj Khalifa may not follow Frank Lloyd Wright's vision completely, it brings awareness of how his vision may be incorporated in the future.

With a significant number of skyscrapers constructed and continuing to be constructed in the Middle East, all disciples of engineers are required to utilize more sophisticated methods of analysis and design. Lateral loadings pose numerous difficulties in mitigating dynamic effects not only for the structural materials, but also for occupant comfort. In addition, once all loadings are considered, geotechnical engineers become faced with new challenges due to geological location and higher loadings. This report will provide design considerations for such structures in hopes of reaching the mile-high vision of urban living in the future.

Five chapters are contained within this report. Chapter One provides the introduction, objective, and scope of report. Chapter Two introduces the background for dynamic considerations, wind engineering and analysis, habitability of upper floors, and serviceability and design criteria. Chapter Three outlines and describes concrete buttressed core structures, core and buttressed elements, outriggers, coupling beams, and the structural monitoring system utilized. Chapter Four presents the foundations geotechnical technical investigation, considerations, models & analysis, design, and pile testing. Chapter Five describes the findings and conclusion of the report.

Figure 1: Dubai Skyline (BLUEBIRD, 2023)

Chapter 2: Dynamic Considerations

2.1 Background

Dynamic considerations for a structure vary from many factors. A few points of interest for most structures are overall stiffness, damping, and mass. Stiffness of a structure can be related through $k = AE/L$, where k is the stiffness of an element, A is the area, E is the modulus of elasticity, and L is the length. Damping of a structure is restraining vibratory motion, or dissipating energy as a building moves. However, more importantly, aerodynamics considerations will widely affect a structures dynamic behavior. Simple quasi-static treatment of wind loading, which is universally applied to design of typical low and mid-rise structures can be unacceptably conservative for very tall buildings. Such a simplified treatment for deriving lateral loads does not address key design issues including dynamic response, wind directionality, and cross wind response (Mendis, 2007).

Buildings are subjected to several dynamic loading conditions including live, wind, and seismic loads, which all vary with time, and excitation function. Additionally, the natural frequency with respect to excitation forces must be considered when examining the dynamic behavior of the structure. A structures natural frequency directly relates to its stiffness and mass and can be calculated, without damping, as $f = sqrt(k/m)$, where f is the natural frequency, k is the stiffness and m is the mass. Another important consideration when assessing dynamic effects are mode shapes. A mode shape describes the component deformation when vibrating at its natural frequency. Knowing the natural frequencies and associated modes of a building structure enables engineers to evaluate dynamic parameters necessary to predict the displacements, velocities, and accelerations due to an excitation function or response spectra. This analysis becomes increasing important in megatall structures for acceleration, and total displacement of a structure. Typical wind drift limits in common usage vary from *H*/100 to *H*/600 where *H* is the building height (Abdulqader, 2019). With the Burj Khalifa's height of 828 meters (2716.5 feet), drift limitations would be 11.4-17.3 centimeters (4.5-6.8 inches). In the case of the Burj Khalifa, habitability of the upper floors and the inhabits' perception of the motion (acceleration, displacement) needed to be within proper limitations for occupant comfort. The next sections introduce the concepts of how habitability, wind loading, and structural dynamics are assessed for megatall buildings.

2.2 Wind Engineering & Analysis

The Dubai Municipality (DM) specifies Dubai as a UBC97 Zone 2a seismic region (the same seismic zone as North-East Kansas). With such a tall building and low seismic region, wind loads governed design of the lateral force resisting system (LFRS). Wind engineering, resisting dynamic wind loading, is one of the primary challenges in design of megatall building design planning. From fluid mechanics, wind, when in contact with a cylindrical shaped object, induces a pressure on the object that reduces its speed due to friction. In laminar flow, wind moves around the object symmetrical in force and shape as shown in image one of Figure 2. However, as fluid velocity increases, turbulent flow comes into effect and the fluid undergoes irregular fluctuations, shown in image two and three of Figure 2. This phenomenon is known as Karman Vortex Street. Karman Vortex Street is caused by vortex shedding or vortex-induced vibration (VIV); as wind comes in contact with a cylinder it slows down due to friction at the surface and causes a spin in flow on either side of the cylinder that leads to oscillation. Knowing if a structure will see such velocities becomes important if the vortex shedding frequency synchronizes with the natural frequency of the building or object, which will lead to larger deflections (drift) and amplitudes.

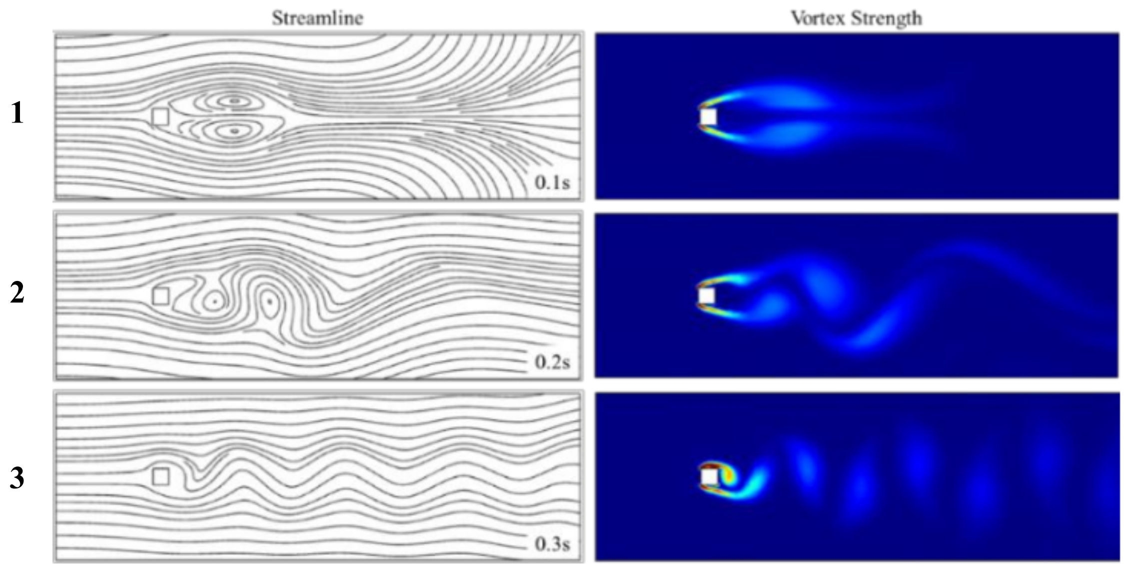

Figure 2: The Formation and Development of Karman Vortex Street (Ke, 2014)

For a building with uniform width, similar to 432 Park Avenue in New York, a critical wind velocity can occur almost at the same time over the building's upper portion where the changes of mean wind speed are relatively small. However, for a tapered building, similar to Burj Khalifa, the critical windspeed decreases with height. Therefore, it is unlikely to occur simultaneously over the height. This means a uniform building may experience a very intensive wind excitation within a relatively narrow range of wind speeds. While a tapered building will have an overall moderate excitation but within a wider range of wind speeds. This can be better understood using the expression $N = SU/b$, where N is the vortex frequency, S is the Strouhal number, U is the wind speed, and b is the building width (Irwin, 2008). Tapering changes building width, thus varying vortex frequencies at different heights. Similar to tapering, varying cross-section, can affect the width but may also affect the shape. In this case the Strouhal number S varies with height causing different frequencies at different heights. Figure 3 shows sections of the upper floor plans and how the width differs for each.

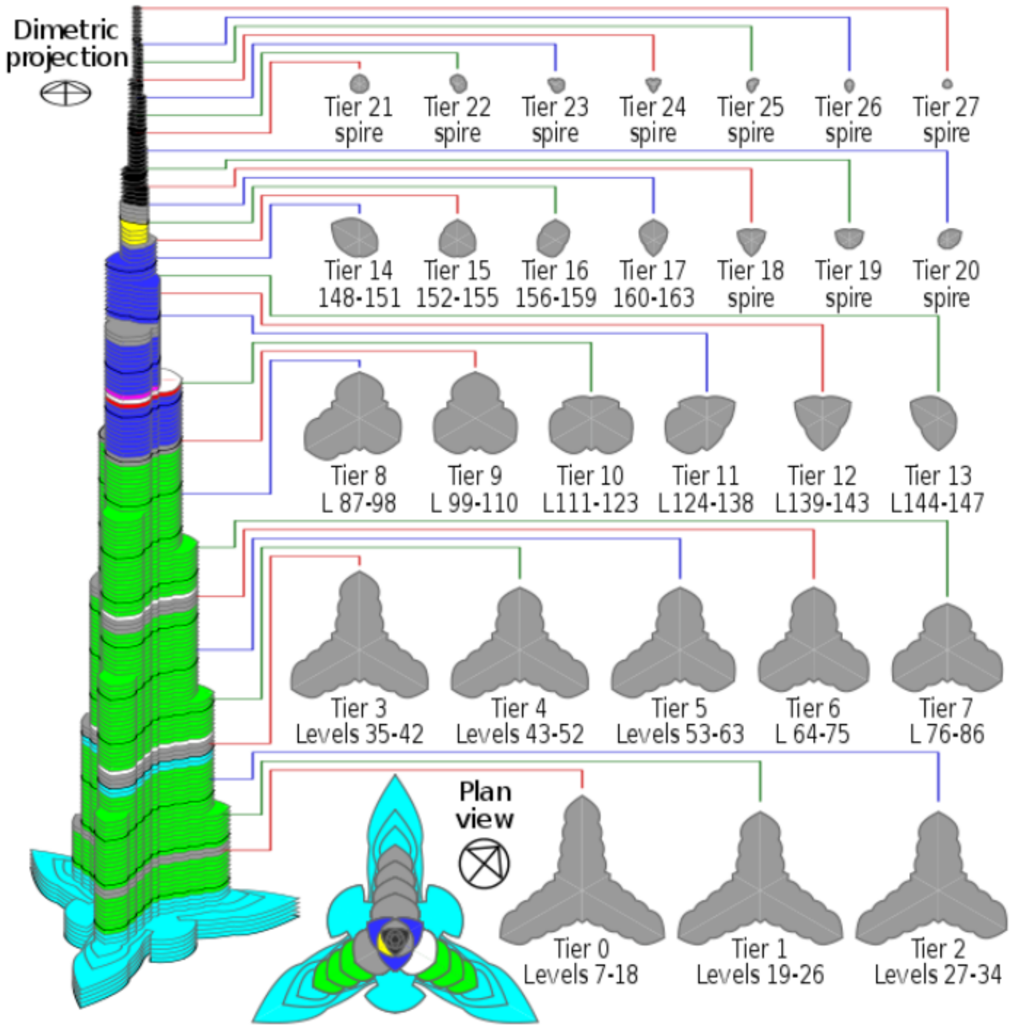

Figure 3: Various sections of upper floorplans (ArabianHorizons, 2021)

Twisting of a structure as used for the Turning Torso in Malmo, Sweden is another consideration for aerodynamic favorability. The effectiveness of twisting increases with increase of twisting angle, altering susceptible wind directions. (Xie, 2014) studied five twisting angles for supertall structures and found that twisting tends to equalize the aerodynamic characteristics over wind directions. On the one side this makes the maximum wind loads considerably reduced, but on the other side the minimum wind loads are increased.

To mitigate vortex shedding for the Burj Khalifa, several methods were utilized: changing shape of building, designing taper or sudden shrinkage of section, orientation of structure. Aerodynamic optimization of the building geometry can be divided into two categories: minor modifications in the plan size, such as, corner recessing, roundness, and chamfering, and major modifications in the elevation size, such as, tapering, setback and twisting (Xie, 2014). The Burj Khalifa's structure not only uses sudden shrinkage of section, but also has an unpredictable outline combined with a loose spiral upward breaking the big vortexes into smaller ones and changing their frequencies (Ke, 2014). Figure 4 depicts the loose spiral upwards (left) of the building as well as vortex frequency with respect to wind velocity and floor level (right). Figure 4's right image highlights six floors including: floor 34 (dark purple), floor 64 (black), floor 87 (blue), floor 99 (red), floor 124 (violet), floor 160 (orange). Knowing the frequency at each level with respect to the wind velocity comes into play when understanding if the natural frequency at such level will correspond with the vortex frequency, causing resonance. The following paragraphs explain how the shape and orientation effects crosswind excitations that directly affect accelerations and deflections of a structure.

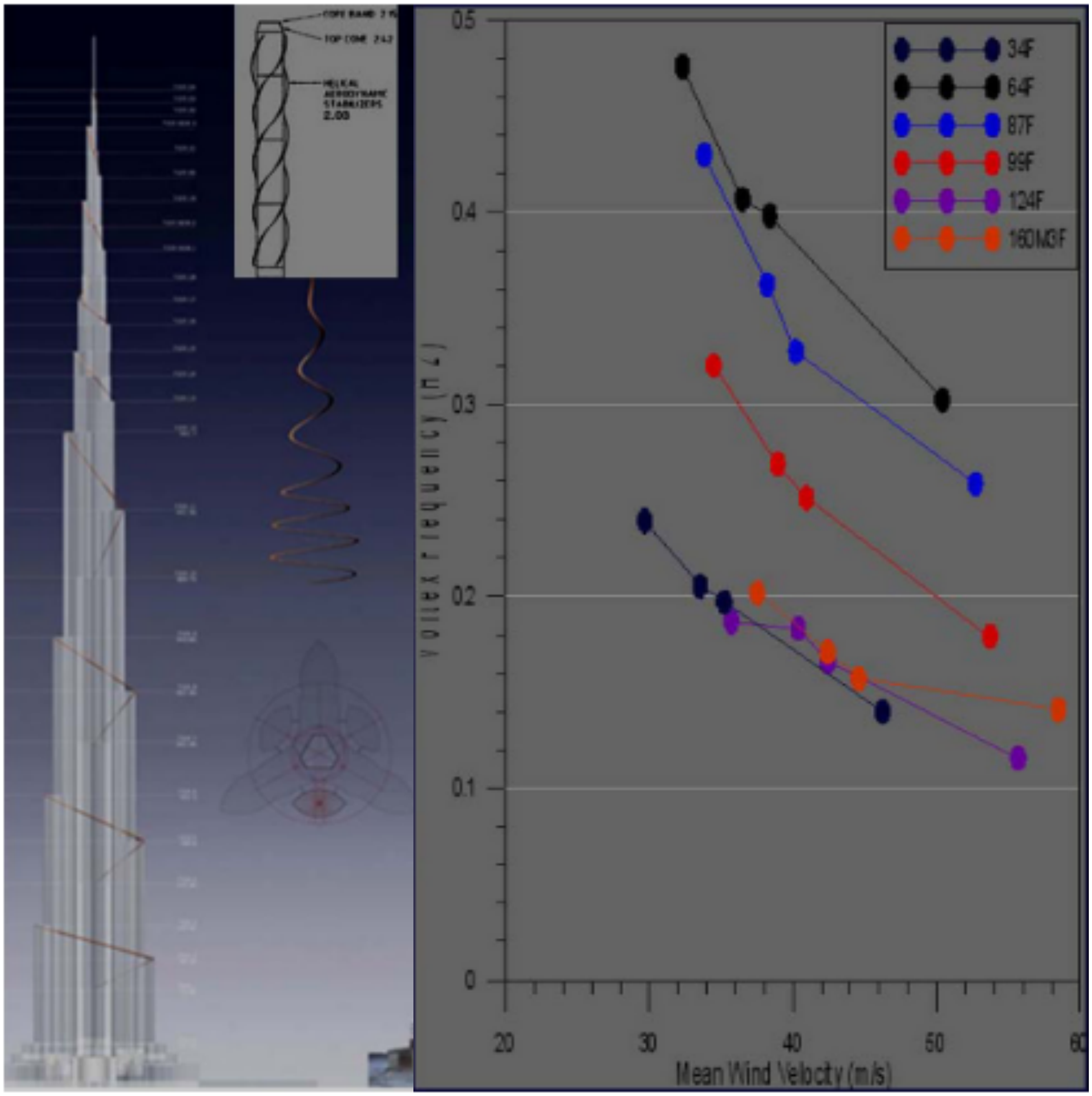

Figure 4: Vortex shedding formation, with different resonance frequencies, along building height (Abdelrazaq, 2012)

One of the critical phenomena that effects tall slender towers is vortex excitation, which causes strong fluctuation forces in the crosswind direction. This is probably the main behavior that distinguishes tall towers from mid-rise buildings. The well-known expression of Strouhal gives the frequency *N* at which vortices are shed from the side of the building, causing oscillatory across-wind forces at this frequency.

The Strouhal number typically ranges from 0.1-0.3, with rough values for square cross sections being 0.14 and cylinders being 0.20. When this number matches any natural frequency of the building, resonance occurs and amplifies the crosswind response. Figure 5 indicates the effect of vortex shedding as wind velocity increases. However, this plot may be moved to the right as the building's natural frequency increases. If it can be moved far enough to the right, where wind velocities will not occur, vortex shedding will be mitigated. The traditional method is to add stiffness to the structure; however, this can become very expensive if the peak is moved far enough to the right. Another consideration is the height of the peak vortex shedding and how a buildings shape and orientation can substantially reduce or eliminate the peak (Irwin, 2008).

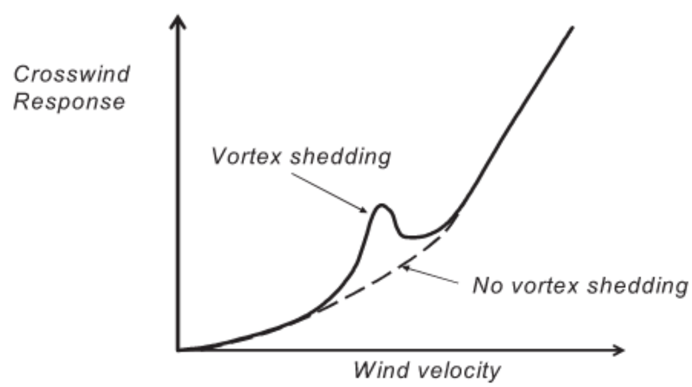

Figure 5: Effect of vortex shedding on response (Irwin, 2008)

The Y-shape of the Burj Khalifa provides six important wind directions, Nose A, B, C and Tail A, B, C. (Irwin, 2009) observed from the force balance test that the force spectra for different wind directions showed less excitation when impacting the nose end of each wing rather than the tail. Figure 6 is an exterior floor plan view of the Burj Khalifa with the nose of each wig being closest to the letters A, B, and C, seeing lower impact. The tails of A, B, and C follow the nose back and to the opposite side of the building, seeing higher impact.

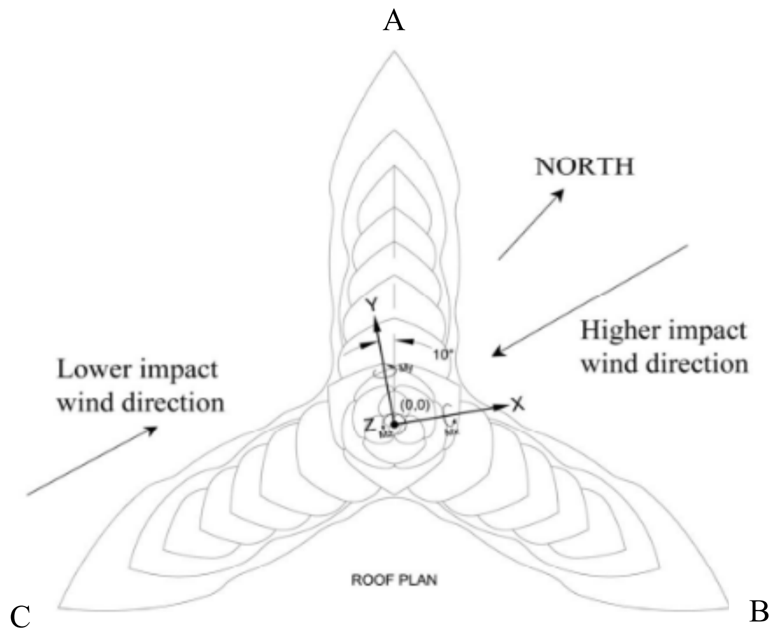

Figure 6: Wind Orientation and Labeling Plan View (Abdelrazaq, 2012)

To understand the wind loading on the Burj Khalifa, the high frequency force balance (HFFB) method and aeroelastic method were utilized. The HFFB method has a base technique of estimating wind loading information from the overturning and torsional moments measured at the base of a rigid building model (see Figure 7). However, this method obtains static measurements, thus cannot account for a structure's dynamic parameters. These forces may then be used to predict dynamic loads and responses (Yip, 1995). This approach does have some major limitations such as: results are valid for buildings with linear sway mode shapes and corrections are required for sway mode shapes, results are valid for buildings with uniform torsional mode shape assuming correction values are approximated, aeroelastic effects are not measured, only fundamental modes are included and higher mode effects are neglected, effects of cross-correlation of wind forces on modal forces are neglected, it is nearly impossible to estimate details of the modal forces for coupled three-dimensional modes (Yip, 1995).

Using the HFFB method a direct relation between shape and wind induced forces was utilized and several rounds of force balance tests were undertaken to optimize the shape. Rowan Williams Davies & Irwin (RWDI) was the company responsible for such testing and concluded that the tower should be oriented relative to the most frequent strong wind directions for Dubai: northwest, south, and east (Irwin, 2005).

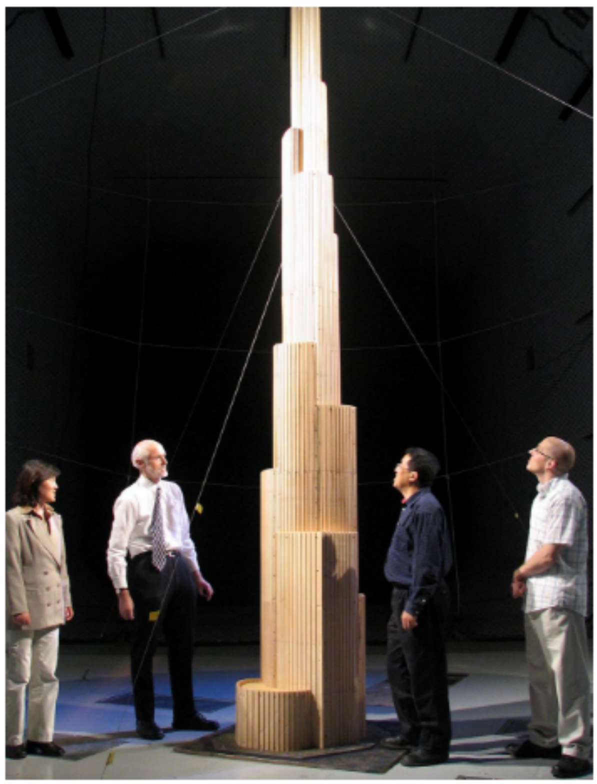

Figure 7: Rigid Model at 1:50 Scale (Irwin, 2005)

With wind-induced forces now known from the high frequency force balance method, an aeroelastic model was created to further improve dynamic predictions. An Aeroelastic model allows for the full response to be measured, higher order modes, aerodynamic damping (damping provided by wind), and the use of a higher Reynold's number than that of high frequency force balance method. Having a large Reynold's number allows for higher wind velocities, thus providing more accurate behaviors due to vortex-induced vibration.

The aeroelastic test utilizes an elaborate aeroelastic model to obtain wind-induced oscillations, fluid-structure interactions, and high mode shapes. Typically, the model is designed to have the exterior geometry of the structure, as well as dynamic parameters such as stiffness, mass, and effective damping (Zengshun, 2020). However, the disadvantages of an aeroelastic test include: design and construction of model is time-consuming, during the test dynamic parameters may vary with oscillating amplitude, wind loading information cannot be obtained. Figure 8 depicts how damping can be controlled through the use of oil. As more damping is needed the plate will become more immersed in oil. The mass and frequency can be adjusted by changing the position of the weight and spring

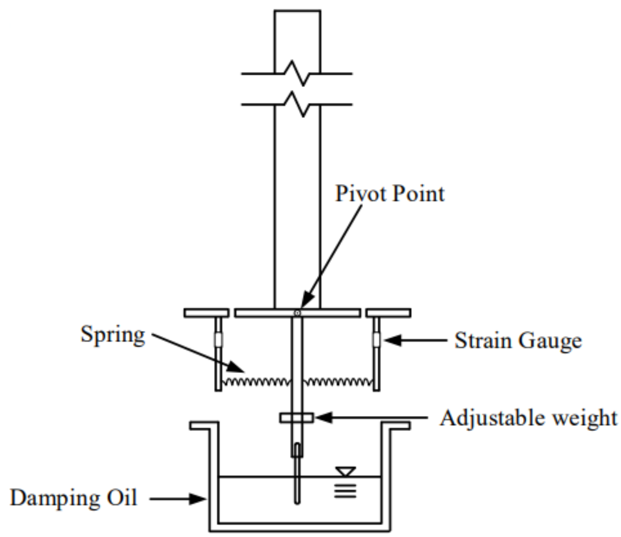

Figure 8: Aeroelastic Stick Model (non-multi-degree-of-freedom model) (Chen, 2020)

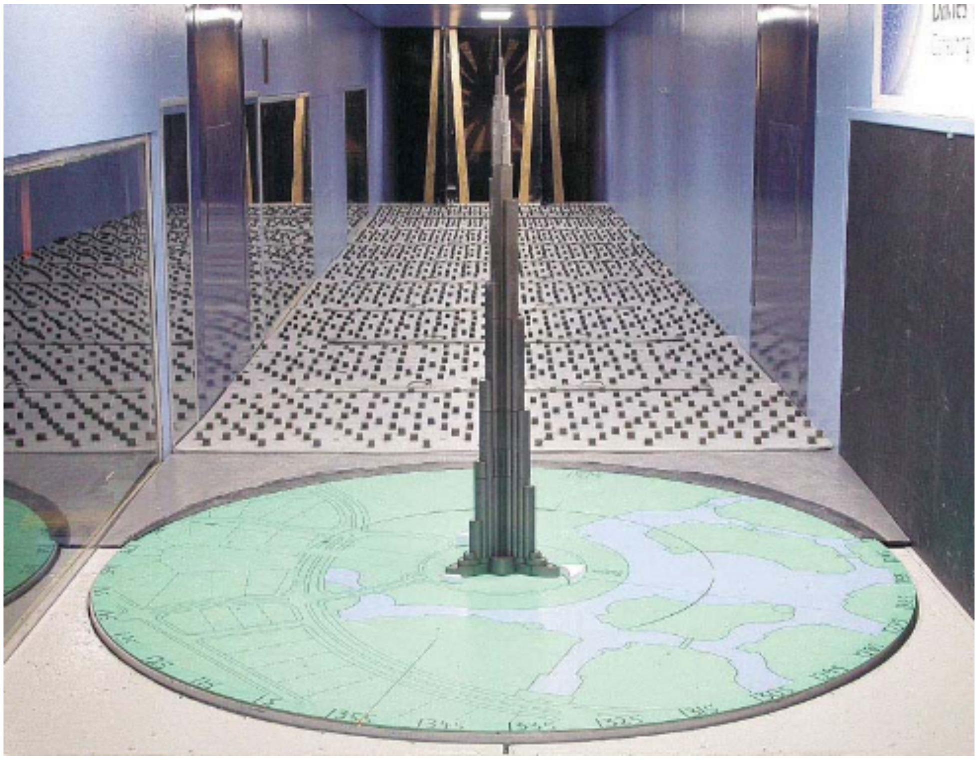

Figure 9: Aeroelastic Model at 1:500 scale (Irwin, 2005)

For the Burj Khalifa's aeroelastic model the modal deflection shapes were similar to those of a tapered column. Thus, it was possible to obtain excellent agreement between frequencies and mode shapes of the model compared to those predicted at full scale using a single machined metal spine with an outer shell attached to it (Irwin, 2005). The aeroelastic model was able to model the first six sway modes as well as measure the accelerations in the upper levels. Comparing the aeroelastic model results with the force balance results it was found that the base moment and the accelerations in the upper levels were significantly lower in the aeroelastic model results (Irwin, 2005). The next section presents why wind engineering and accelerations are necessary for habitability of the upper floors of the Burj Khalifa.

2.3 Habitability of Upper Floors

The past few decades many super-tall and megatall buildings have been constructed all over the world. With advances in engineering materials, structural design, and wind engineering, designers can ensure buildings meet strength and constructability requirements. However, as buildings become taller, occupant comfort further refines the design to ensure habitability of upper floors. Through wind engineering, designers can better estimate building drift, and resonant vibration due to the dynamic and varying action of wind as previously discussed in Section 2.2. These resonant motions become perceptible to occupants. Thus, two factors need to be considered in order to maintain an acceptable environment for occupants: the mitigation of fear for safety and the elimination of discomfort (Burton, 2015).

Wind-sensitive buildings can experience excessive vibrations that cause discomfort and interruption of the activities of inhabitants of the buildings. To ensure desired serviceability, codes and standards have proposed perception curves that limit the peak acceleration of buildings. However, these criteria do not consider the probability that wind-induced vibration is perceived within a service period (Pozos-Estrada, 2010).

Humans in general experience and perceive their environments within buildings in vastly different ways depending on occupational task. Mixed use buildings that have residential, hotel, and office spaces may need to satisfy several serviceability requirements. Several cues (visual and acoustical) may trigger the perception of motion for building occupants. Internal visual examples may include, swinging lights, moving blinds, swaying plants, liquids, or movement of suspended objects. Additionally, external visual cues may trigger perception including: swaying of trees, extended flags, or other indications of high wind speeds. Similarly, the most common acoustic cue is blinds impacting windows and structural creaking (Burton, 2015).

2.4 Wind Serviceability & Design Criteria

Building codes and standards are to promote safety of buildings and serviceability issues such as accelerations, velocities, and deflections but are often regarded more to the quality of the building rather than safety. This allows for owners and designers to negotiated design depending on desired level of quality attributed to the building. Meaning designers may allow for smaller accelerations and deflections, but such implementation will directly affect an owner's overall cost. As a result, acceleration criteria receive diverse considerations within each code or standard utilized. The original criteria for tall building design was the introduced in the National Building Code of Canada (1977) which suggested limiting the peak building accelerations occurring once every 10-years to 1-3% of gravity (0.098*g*-0.294*g*). After various investigative research, the Council of Tall Buildings and Urban Habitat and International Organization for Standardization (ISO) adopted three tentative guidelines from ISO 10137 (2007): *Bases for design of structures – Serviceability of buildings and walkways against vibrations*. American Society of Civil Engineers (ASCE) and CTBUH technical committees suggested three ranges of acceleration to be, 5-7 milli-*g* for residential buildings, 7-9 milli-*g* for hotels and 9-12 milli-*g* for office buildings, and 1.5 milli-rad/sec for torsional velocity all within one-year return periods (Burton, 2015).

Figure 10 compares several codes and standards according to a structure's peak acceleration and frequency. Since frequency is the inverse of period, tall buildings will appear on the far-left with lower frequencies. Higher accelerations in taller, lower frequency structures may consider the fact that in order to decrease accelerations, structural cost would significantly increase. Average perception thresholds can be seen in Figure 10 by various lines from Burton, Denoon, Kanda & Shioya, Kanda & Tamura, and Chen. As for design criteria, ISO 6897 has the highest peak acceleration for low frequencies and nearly matches Australian Standard (AS) 1170.2 peak acceleration at higher frequencies. A difference to note is that office and residential lines are parallel but different in nature. This is due to the effect of human sensory and occupancy (Mendis, 2007). Meaning that occupants are less effected by accelerations when doing office work rather than in the comfort of their own personal apartment or residence.

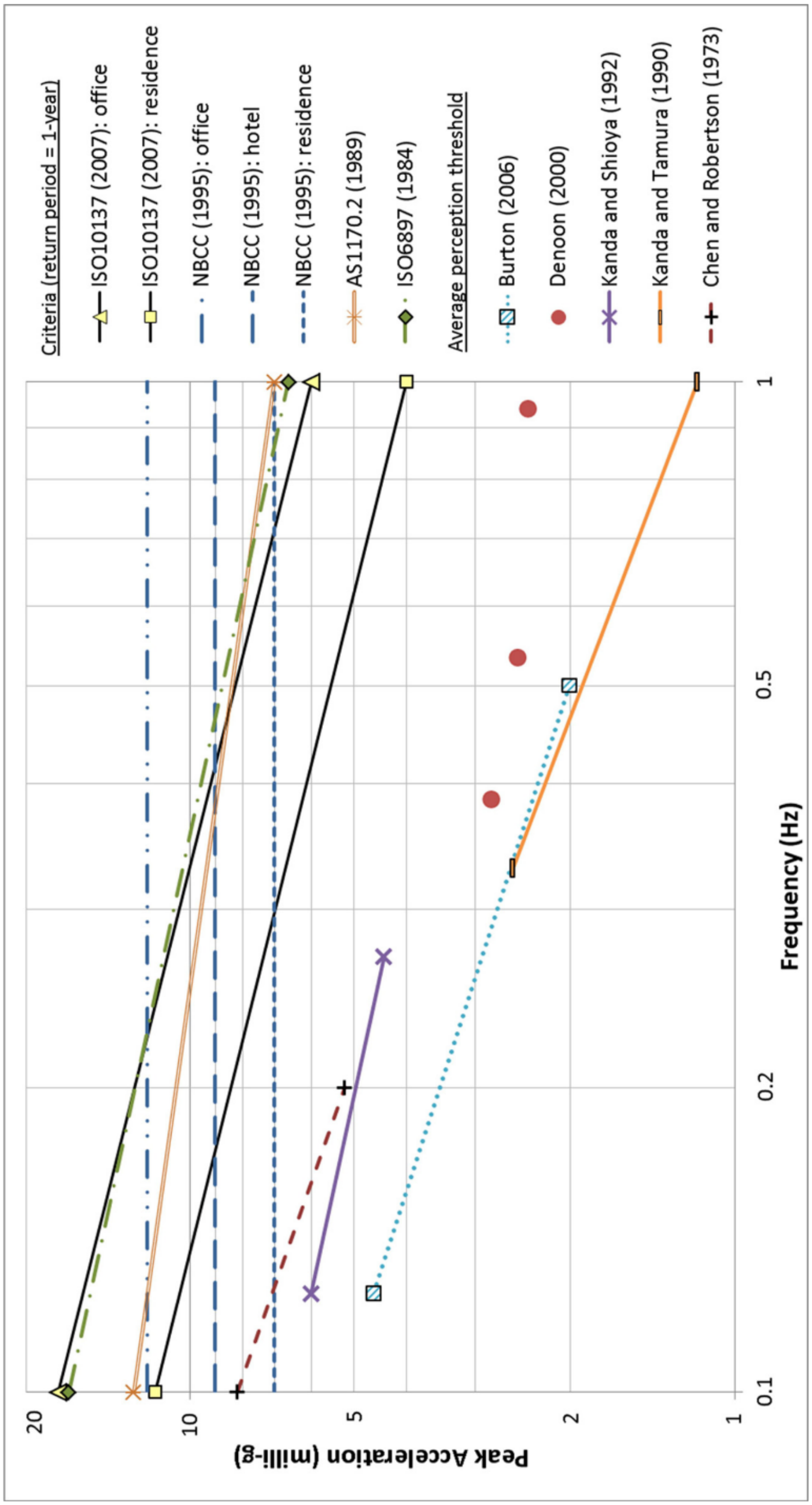

Figure 10: Comparison of average perception thresholds, and suggested criteria, for the 1-year return period (Burton, 2015)

Based on the high frequency force balance test results the Burj Khalifa utilized local wind statistics allowing the prediction of peak accelerations for various return periods ranging from one to ten years. Initial predictions resulted in 37 milli-*g* for the five-year return period. A return period is the probability that a severe event occurs once in every certain amount of years. Generally, higher return periods are associated with larger loads as the probability to see a severe event in one year is lower compared to ten years. While these predictions were at the foreground of the wind engineering design process, the final iteration resulted peak acceleration had come down to 19 milli-g. Subsequently, when the aeroelastic model became available predictions were further improved. Part of this was due to the ability of the aeroelastic model to have seen a higher range of Reynolds numbers and also aerodynamic damping. In total the acceleration was able to be brought down into the 12 milli-*g* range, meeting the three tentative guidelines provided in the paragraph above. With levels 112-154 being corporate suites, peak acceleration was able to be kept in the highest classification of the guidelines.

Chapter 3: Concrete Buttressed Core

3.1 Background

Throughout the history of tall buildings, structural engineers have invented the means to go higher. In the 1970's Fazlur R. Khans tube concept was a dramatic shift from the traditional portal frame system used on such structures as the Empire State Building (Ellis, 2003). Later developments, including the core plus outrigger system (Figure 11), also provided architects with the tools to design taller, more efficient buildings. However, the resulting growth was gradual, each innovation marking a point on the progressive scale of the tall building. The buttressed core is different in nature as it permitted a dramatic increase in height. The buttress core design employs conventional materials and construction techniques and was not precipitated by a change in materials or construction technology. Buttressed core is a structural system for high-rise buildings consisting of a hexagonal core reinforced by three buttresses that for a Y shape. It is an inherently stable system in that each wing is buttressed by the other two. The central core provides the torsional resistance for the building, while the wings provide the shear resistance and increased moment of inertia (see Figure 11 typical floor plan of the Burj Khalifa). The buttressed core represents a conceptual change in structural design whose evolutionary development began with Tower Palace III, designed by Chicago-based Skidmore, Owings & Merrill LLP (SOM). Figure 11 provides a visual to describe the system and is further discussed in the following paragraphs.

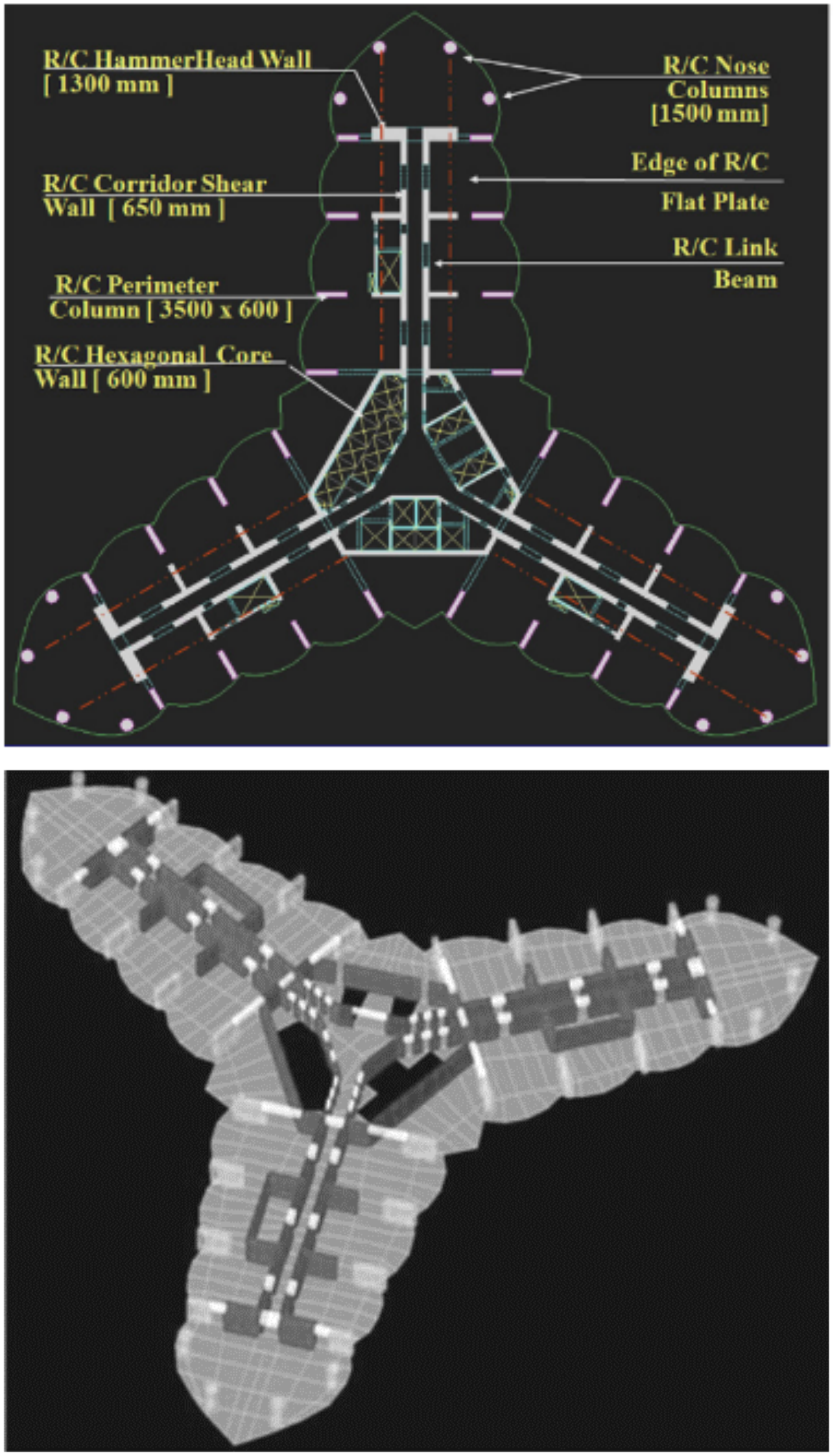

Figure 11: Typical floor framing at typical hotel level (Abdelrazaq, 2012) & (Lee, 2008)

The buttressed core, a structural system for tall buildings, consists of a hexagonal core reinforced by three buttress elements that form a Y-shape, was first used in 2004 by Skidmore, Owings & Merrill LLP (SOM) as the structural system of Tower Palace III, South Korea (WF Baker, 2012). While the buttressed core was first used in the Tower Palace III, this report discusses its application within the Burj Khalifa.

The system utilizes buttress members comprised of corridor shear walls and link beams to support a 600 millimeter (25.6-inch) thick hexagonal core. The central core provides torsional and axial resistance for the building, while the wings provide shear resistance as well as an increased moment of inertia (WF Baker, 2012). Increased moment of inertia allows for higher forces to be applied when resisting rotation. Furthermore, 120 degrees between wings provides greater views, privacy, and amounts of natural light for occupants. To better understand how a buttressed core performs several elements must first be observed: outriggers and their effects on the structural system, coupling beams and why they are necessary to keep buttress elements continuous, the hexagonal core and buttressed elements and considerations taken for strict construction tolerances, and finally the state-of-the-art health monitoring system utilized to validated all design assumptions as well as observe structural behaviors.

3.2 Outriggers

Outriggers are rigid horizontal structures designed to improve building overturning stiffness and strength by connecting the building core or spine to distant columns (Choi, 2017). Their purpose is to act as stiff arms engaging outer columns, when a central core tries to tilt, its rotation at the outrigger level induces a tension-compression couple in the outer columns, acting in opposition of that movement. Thus, a restoring moment acts on the core at that level. An outrigger system is frequently selected for the LFRS, where overturning moment is large, and where building flexural deformations are major contributors to lateral deflections such as story drift, which improves occupant comfort during high winds (Choi, 2017). Figure 13 depicts the concrete core wall and LFRS at various levels. Outriggers can be seen in several locations connecting an exterior group of columns to the centralized structure.

Figure 12 and Figure 14 shows a simple core with outriggers to help conceptualize how this system effects structural behaviors. High-rise buildings are subjected to large lateral deformation either due to wind or seismic loads. The core structure of a high-rise building is subjected to cantilever deformation, while the frame structures surrounding the core are subjected to portal type deformation. This causes high drift and overturning effects on the structure. The incorporation of outriggers efficiently controls the excessive drift due to lateral loads. This prevents structural and non-structural damage in the structure. However, as outriggers are added the structure is able to transfer the overturning moment of the building through the outriggers attached to each side of the core. These very stiff elements then share the moment of the core with perimeter columns reducing story drifts and deflections. Outriggers have the potential to reduce drift up to 30% through the use of the horizontal belt trusses that tie the frame to the core (Fu, 2018).

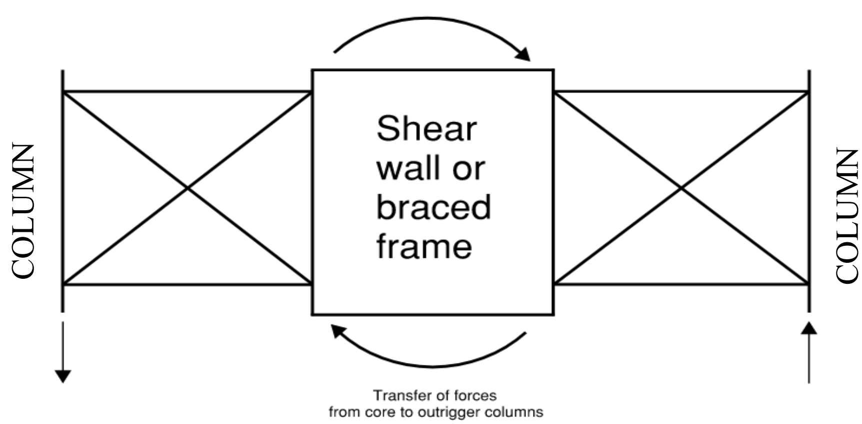

Figure 12: Force Transfer in Conventional Outriggers, adapted from (Moon, 2018)

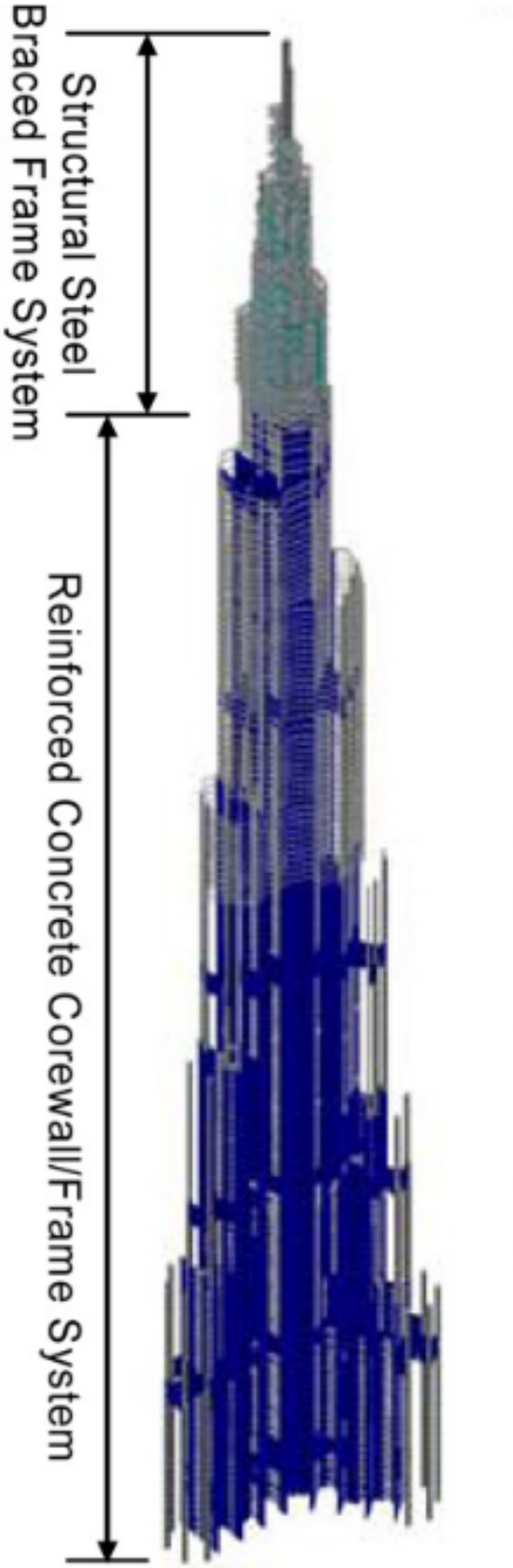

Figure 13: Outriggers at various levels (Abdelrazaq, 2012)

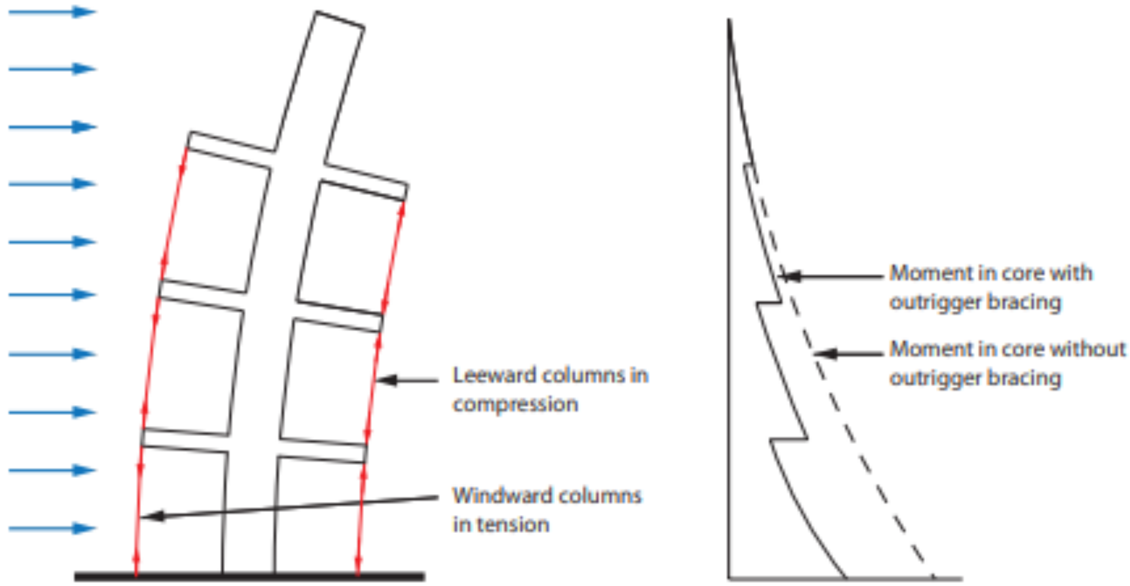

Figure 14 : Reduction of core moment with outrigger (Choi, 2017)

The Burj Khalifa utilizes outriggers as shown in Figure 13 and Figure 15, the reinforced concrete core walls are linked to the exterior columns through a series of reinforced concrete shear walls (corridor and hammer head). The structural action of outrigger systems in high-rise buildings is based on the tension-compression couple induced in the outer column. While most wall corridor shear walls and link beams are tied together at every floor, mechanical levels have outrigger walls and nose columns tied together through four story shear wall panels to engage all vertical member in the lateral system and to allow for better gravity load and stress distribution between them (Abdelrazaq, 2012). To help visualize this, compare Figure 15 to Figure 11 and note the differences seen in the tips of each wing.

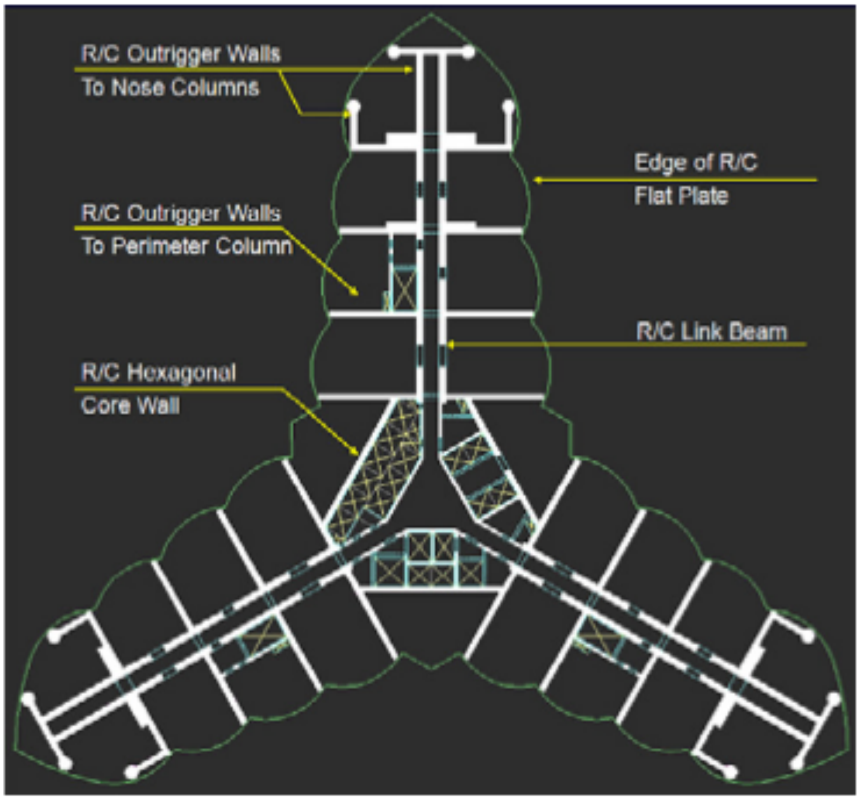

Figure 15: Outrigger structural floorplan (Abdelrazaq, 2012)

However, outriggers can be problematic. For instance, architectural constraints may prevent placement of outrigger columns where structural efficiency would be maximized. Connections of the outrigger trusses or diaphragms to the core can be complicated. To be effective, outriggers need to be extremely stiff to attract forces from the core. However, outriggers must also remain ductile in the case of inelastic behavior as to not damage the core or have brittle failure within the element itself. Finally, certain connections may need to be made after the building has topped out to mitigate differential shortening between the core and perimeter columns. In most instances, the core and outrigger columns will not shorten equally under gravity load. With the outrigger's high stiffness, severe stress can occur as they try to restrain the differential shortening between the core and outrigger columns (Nanduri, 2013).

3.3 Coupling Beams

Coupling beams are a lateral force resistant component of a structure. Conventionally, coupling beams are used to link individual walls in a stiff lateral-load resisting system. These members work in double curvature with high shear forces and often feature small span-to-depth rations ($a/d < 2.5$) where a is the span length and d is the depth of the member (Mihaylov, 2017). Their main function is to maintain stiffness, strength, and continuity of the lateral load system of a structure. Meaning these elements must be adequate to carry load from lateral force resisting system without excessive deformations in order for proper load path to be achieved. Such short members are susceptible to shear failures that occur along wide diagonal cracks with yielding in the transverse reinforcement, depicted on the left in Figure 16. To mitigate such effects coupling beams are provided with large amounts of stirrups or diagonal reinforcement. However, with transverse failure suppressed, failure develops in the end sections and is characterized by crushing of the concrete prior to or after the yielding of the flexural reinforcement depicted on the right in Figure 16 (Mihaylov, 2017).

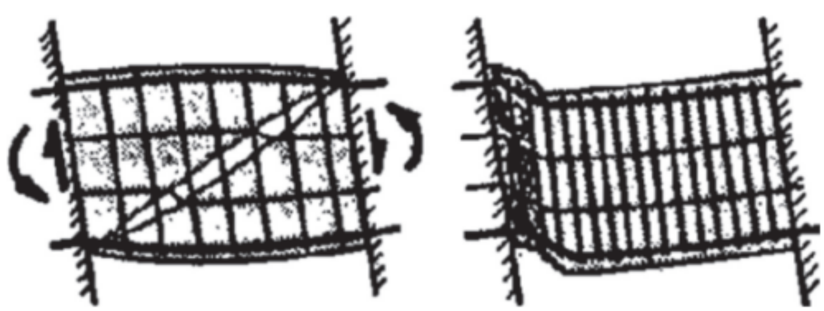

Figure 16: Transverse and sliding failure (Mihaylov, 2017)

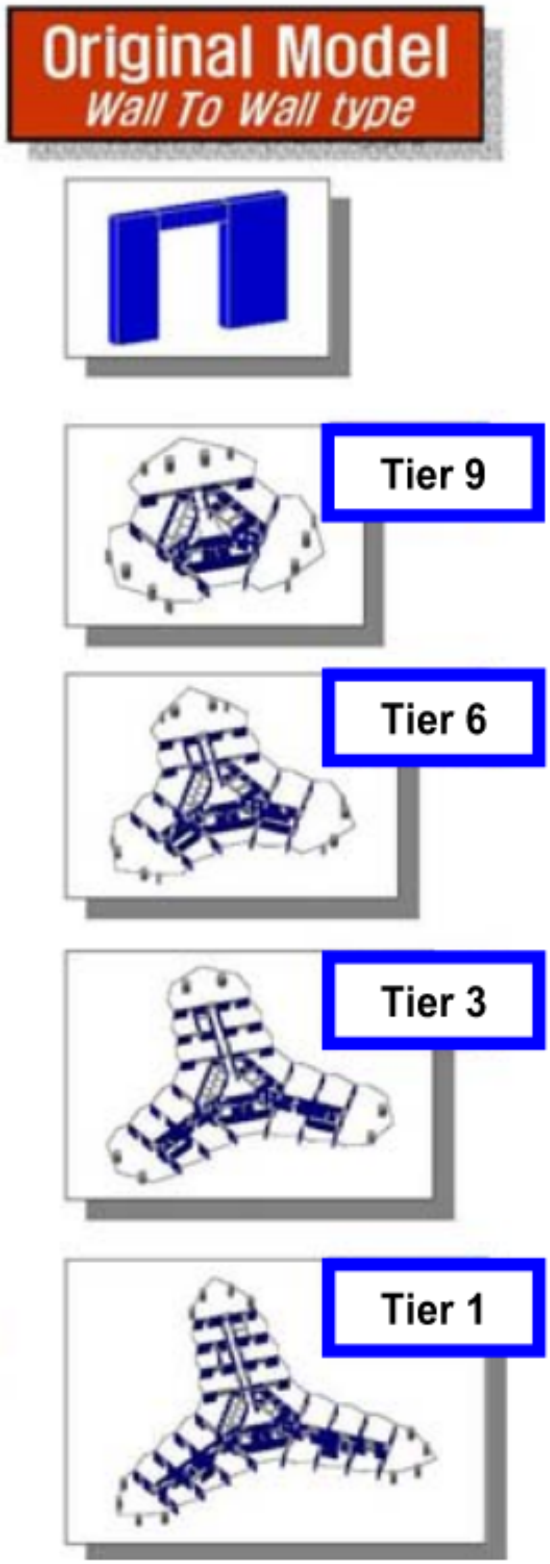

Figure 17: Lateral Force Resisting system (Abdelrazaq, 2010)

As depicted in Figure 17**Error! Reference source not found.**, coupling beams are a necessary element at many levels where larger floorplans occur. In total several thousand reinforced concrete coupling beams were used to interconnect structural walls. "In some cases, the factored shear forces in the link beams were up to three times the traditional nominal American Concrete Institute (ACI) shear force strength limit. Due to tapering of the tower, the primary demand on the link beams is from the gravity load redistribution, flow from the taller core to the perimeter of the structure" (Lee, 2008). Where the tower sees setbacks, large amounts of load are transferred through the coupling beams of the lower stories. To overcome the traditional nominal ACI strut-and-tie shear force strength limit, several tests were conducted to better understand how coupling beams would act under the high loadings provided by the Burj Khalifa.

Strut-and-tie models define two types of zones in a concrete component depending on the characteristics of stress fields at each location. The two types of zones are B-regions and D-regions. B-regions are zones that remain in the "plane section" assumption. This assumption states that as a member becomes loaded that it remains in one plane and does not break it, neglecting the influence of shear distortions/strains. D-regions are zones where this assumption does not apply. Typically, D-regions are assumed at portions of a member where discontinuities (or disturbances) of stress distribution occur because of abrupt changes in geometry (Beres, 2007). In such regions the complex flow of internal forces is transformed to a truss-like structure carrying the imposed load. These truss-like elements are composed of struts, ties and nodes. Struts consist of fan or bottle shaped compression zones, ties are tensile regions, and nodes are points where at least three forces gather for force equilibrium.

Table 1: Design Details for Analyzed Beams, adapted from (Lee, 2008)

Beam ID	Geometry			Factored loads		Design method used
	Width, mm	Depth, mm	Span, mm	Shear, kN	Moment, kN*m	
LB1	650	825	1400	1705	1194	Conventional (ACI 318-99, Section 11.8)
LB2	650	825	1400	2805	1164	Strut-and-tie (ACI 318-02, Appendixes A and C)
LB3	650	825	1400	3750	2625	Steel Plate
LBB	650	825	1400	5250	3675	Built-up steel I-beam

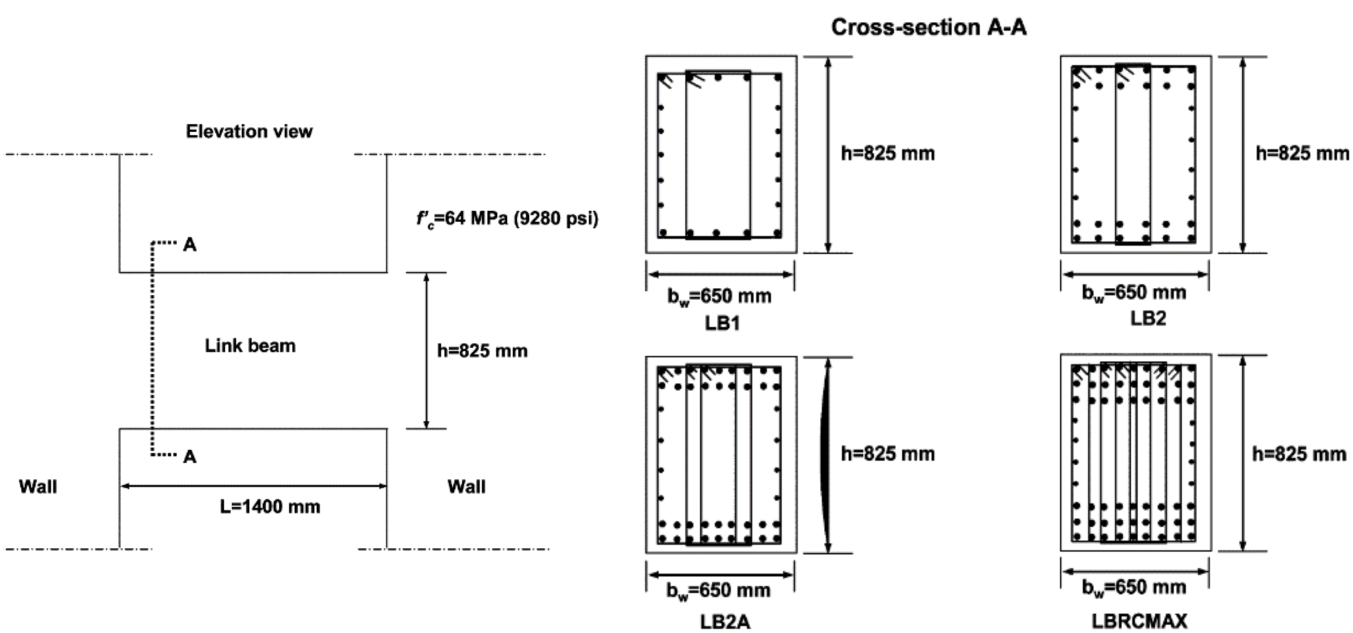

Figure 18: Design details for analyzed beams, Loading, & Predicted load-deformation response of LB2 (Lee, 2008)

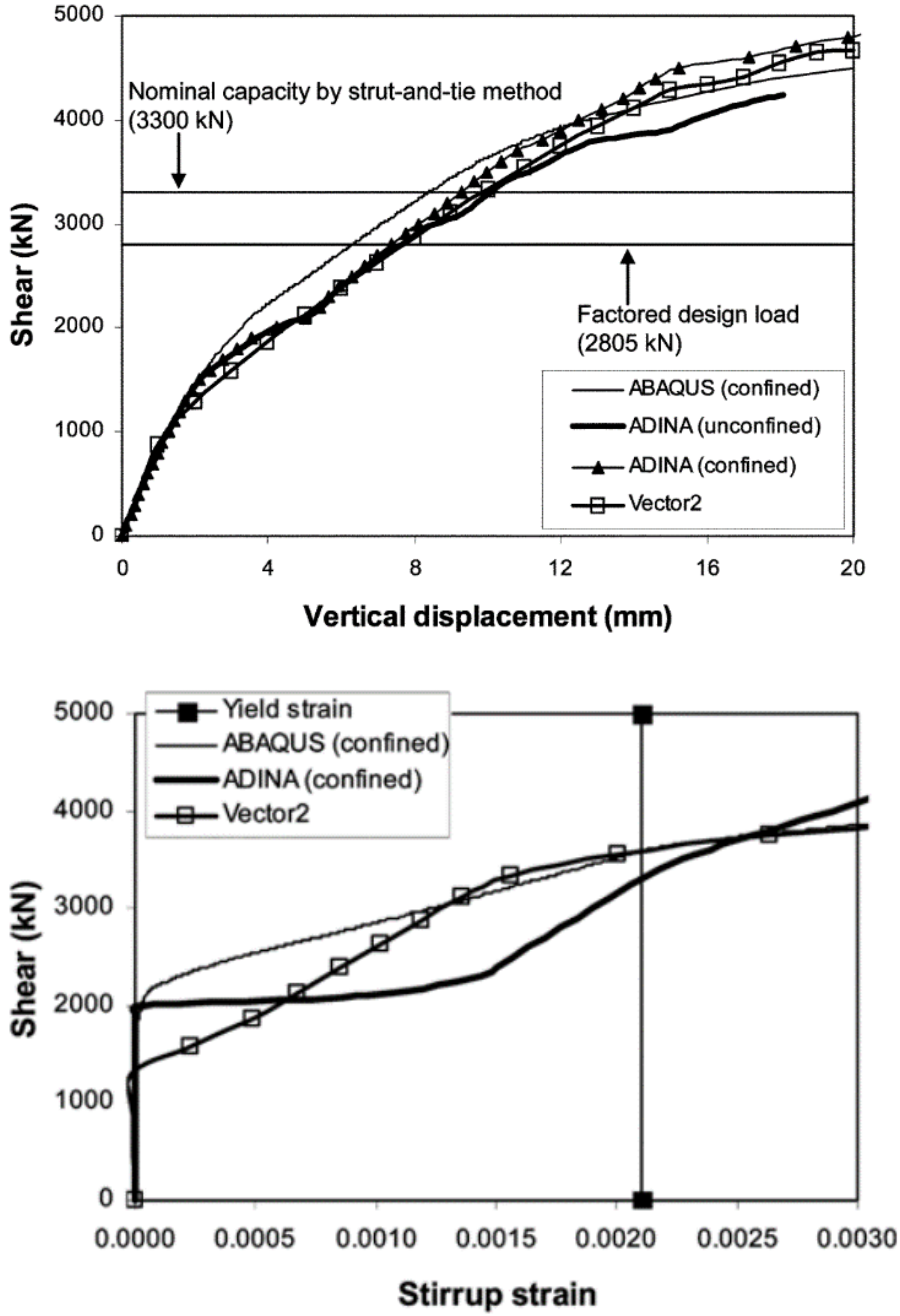

Figure 19: Predicted load-deformation response & reinforcement strains in LB2 (Lee, 2008)

Figure 18 shows typical design arrangements for reinforcement in coupling beams as well as the governing loads and design methodology used. The strut-and-tie method was utilized on the basis that externally applied loads can flow directly to a near support. Comparatively, traditional beam design uses geometric and material properties of the entire cross section. The strut-and-tie method becomes advantageous due to the elements resistances is less dependent on the properties of the entire cross section and more dependent on the configuration of the struts, ties, and nodes in D-regions (Tuchscherer, 2014). With large shear loads present, direct flow to nearby supports was crucial.

Figure 19 depicts the vertical displacement and strain as LB2 was loaded. For beam LB2, the strain in the transverse reinforcement reached yield before yielding of the longitudinal reinforcement. Leading to the capacity and failure mode of LB2 predicted to occur at the point of yielding of both the longitudinal and transverse reinforcement (Lee, 2008). It was concluded that the strut-and-tie method using ACI 318-02 and Appendixes A and C were very conservative for shear design of LB2 and that the beams would be adequate. While Lee also tested LB2A and LBRCMAX, the constructability of said members was deemed too congested and composite steel-concrete members were used.

3.4 Hexagonal Core & Buttressed Elements

3.4.1 Differential Shortening

High rise buildings are subjected to a large number of load increments during the construction process. As concrete is continuously loaded, elastic shortening is a direct effect followed by shrinkage, and creep over time. Historically, engineers have determined the behavior of concrete structures using linear-elastic finite element analysis for vertical elements.

However, as building heights increase conventional methods may diverge from actual behavior. When time-dependent effects of creep, shrinkage, variation of concrete stiffness with time (Young's Modulus), sequential loading, and foundation settlements are not considered, may cause the predicted forces and deflections to be inaccurate (Baker, 2007).

As walls and columns have different sections, support different amounts of load with time, a differential shortening between vertical elements can be expected (Bliuc, 2009). This can create serviceability and structural problems for horizontal elements (slabs and beams). Cladding and services may also be affected by unexpected amounts of differential shortening between vertical elements. Differential shortening consists of three components: elastic (instantaneous), creep, and shrinkage. This relationship is depicted in Figure 20.

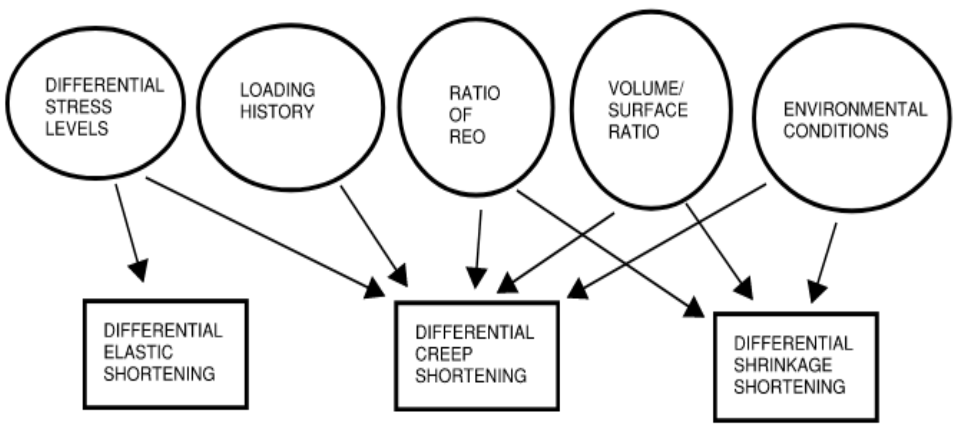

Figure 20: Relationship between different factors affecting concrete shortening, adapted from (Bliuc, 2009)

Differential elastic shortening can be explained using Hooke's law which states that a force, F, needed to extend or compress a spring by some distance, x, is proportional to that distance. That is $F = kx$, where k is the spring stiffness. From structural analysis axial loaded members are treated as truss elements and have stiffnesses $k = AE/L$, where A is the area, E is the modulus of elasticity, and L is the length of member. From this relationship, vertical elements of different sizes having the same stress level will shorten similarity, whereas vertical elements of the same size but different stresses will shorten differently. The change in distance *x* will increase linearly with length even if stresses are constant. Therefore, the taller the building, the greater the shortening amount (Matar, 2017). With the Burj Khalifa using perimeter columns as outriggers, Section 3.2, these elements may attract significant axial loads due to wind, but otherwise have similar loading as the interior walls. This must be considered during design and can be alleviated by the use of outrigger walls to engage all elements depicted in Figure 15 to equalize stresses over vertical elements.

To mitigate the effects of differential column shortening due to creep between the perimeter columns and interior walls, perimeter columns of the Burj Khalifa were sized such that self-weight gravity stress matched the stress on the interior corridor walls. Since shrinkage occurs more quickly in thinner smaller volume to surface area ratios, walls and columns matched the typical corridor wall thickness to ensure columns and walls will generally shorten at the same rate due to concrete shrinkage. "The creep and shrinkage prediction approach are based on the Gardner-Lockman GL2000 (Gardner, 2004) model with additional equations to incorporate the effects of reinforcement and complex loading history" (Baker, 2007). Appendix A of Baker's report gives all the additional equations incorporated in the effects of reinforcement and complex loading history. The predicted vertical shortening for the Burj Khalifa is shown in Figure 21. The purple dashed line indicates the elastic displacement subsequent to casting of the concrete, starting at zero and having the largest value of 50 millimeters (1.97 inches) at approximately the 100th story. After this the elastic displacement reduces to approximately 25 (.98 inches) millimeters at the 160th story. The green short dashed lines indicate the shrinkage displacement subsequent to casting.

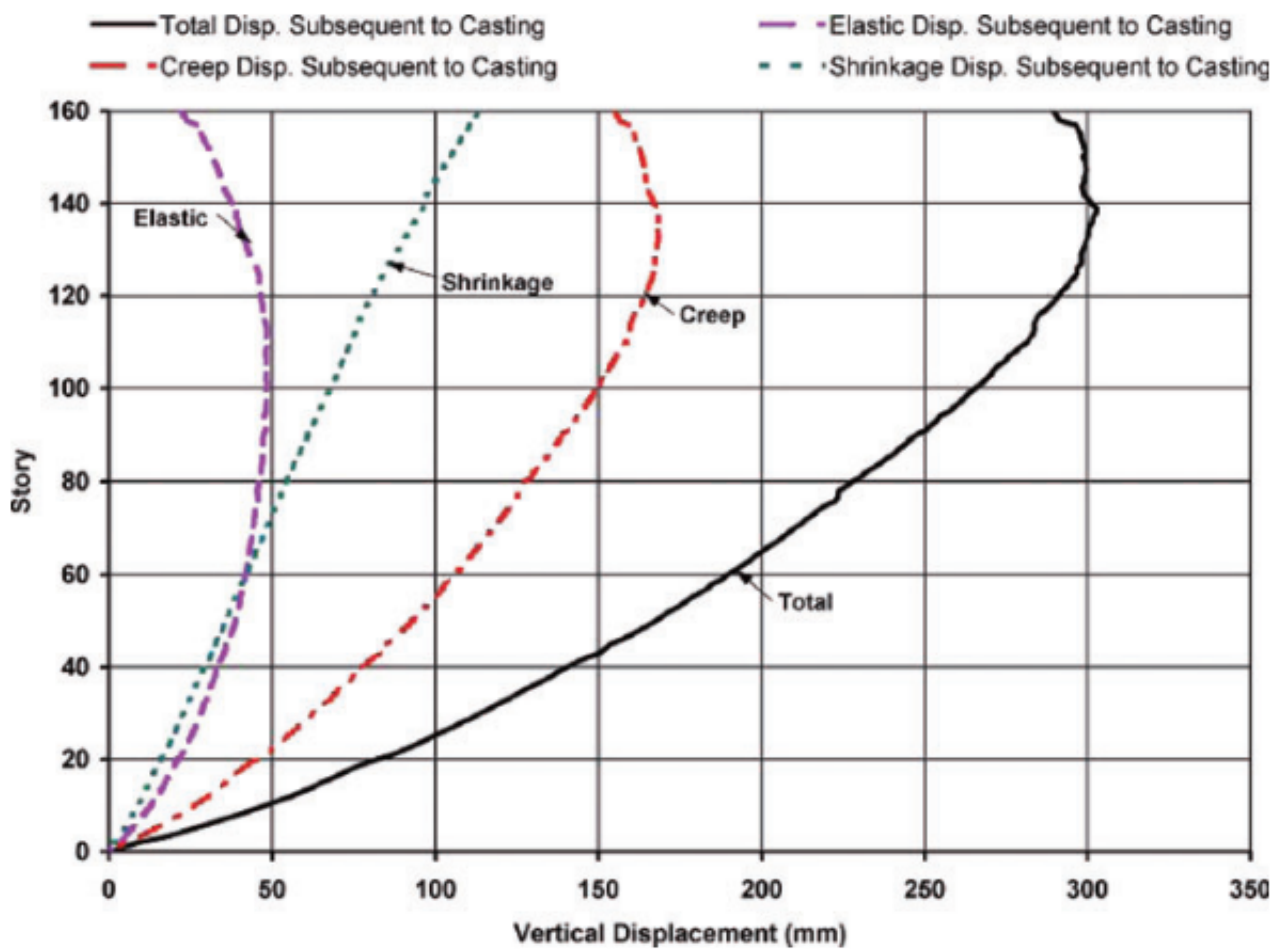

Figure 21: Predicted vertical shortening vs. story at 30 years (subsequent to casting) (Baker, 2007)

All the following explanations are based out Appendix A of *Creep and Shrinkage and the Design of Supertall Buildings-A Case Study: The Burj Dubai Tower*. Elastic shrinkage, as defined in Baker's Appendix A, depends on incremental gravity load applied at a time, transformed section area, and concrete modulus of elasticity at a time. Shrinkage depends on steel reinforcement correction factor (steel reinforcement ratio and modular ratio at a time), shrinkage strains at initial and final time, and is dependent on volume of concrete without the impact of external forces. Creep depends on strain at a time due to load, steel reinforcement correction factor at a time, and is the effect of continuous sustained loading applied for a considerable time. Consideration for each effect is important as every floor plan is not stacked on top of each for aerodynamic efficiency stated in Section 2.2 Wind Engineering & Analysis. Meaning loading may not be equalized over the structural system until an outrigger in reached. This effect could be seen in the creep and elastic shrinkage of a vertical element. In regard to shrinkage, if volumes of elements are not similar at each floor plan, vertical shortening would be affected. With strict construction tolerances, vertical displacement was a critical consideration.

3.5 Structural Health Monitoring System

3.5.1 Background

With the Burj Khalifa being the tallest building in the world, construction planning as well as the construction process, was challenging in every aspect. Several factors making the construction process challenging include: dynamic wind excitations, large and concentrated crane loads at the uppermost constructed level, foundation settlement, column shortening due to elastic, creep, and shrinkage effects, daily temperature fluctuation (150 millimeter (5.9 inch) change in height over a six hour period), uneven solar effects that could result in building tilt, lateral drift of the building under gravity loads due to asymmetrical load distribution relative to the tower center of rigidity, building construction sequence, and mix of concrete (Abdelrazaq, 2012). With all these considerations, utilization of the latest advances in construction methods to build the tower to high degree of accuracy required the implementation of state-of-the art survey and structural health monitoring systems. The systems were comprised of extensive survey monitoring program, strain gages, temporary real-time health monitoring program, and permanent real-time structural health monitoring.

3.5.2 Survey monitoring programs

Several detailed survey programs were developed for the construction of the tower that used the latest GPS technology and instruments to measure sloping distance, horizontal angles, and vertical angles (inclination sensors and clinometers). This system was used for every level being made up of three GPS antenna/revivers with tillable cylinders underneath for self-leveling, and a total station. A total station is a piece of surveying equipment that is used to measure angles and distances electronically to give position coordinates in a space. This paired with eight clinometers at approximately every 20 floors were used to track the towers lateral and torsional movements as well as center the auto climbing formwork system (ACS) at every level. With a strict tolerance of 15 millimeters (.59 inches), this integrated system was necessary to stay within limits and keep the structure centered.

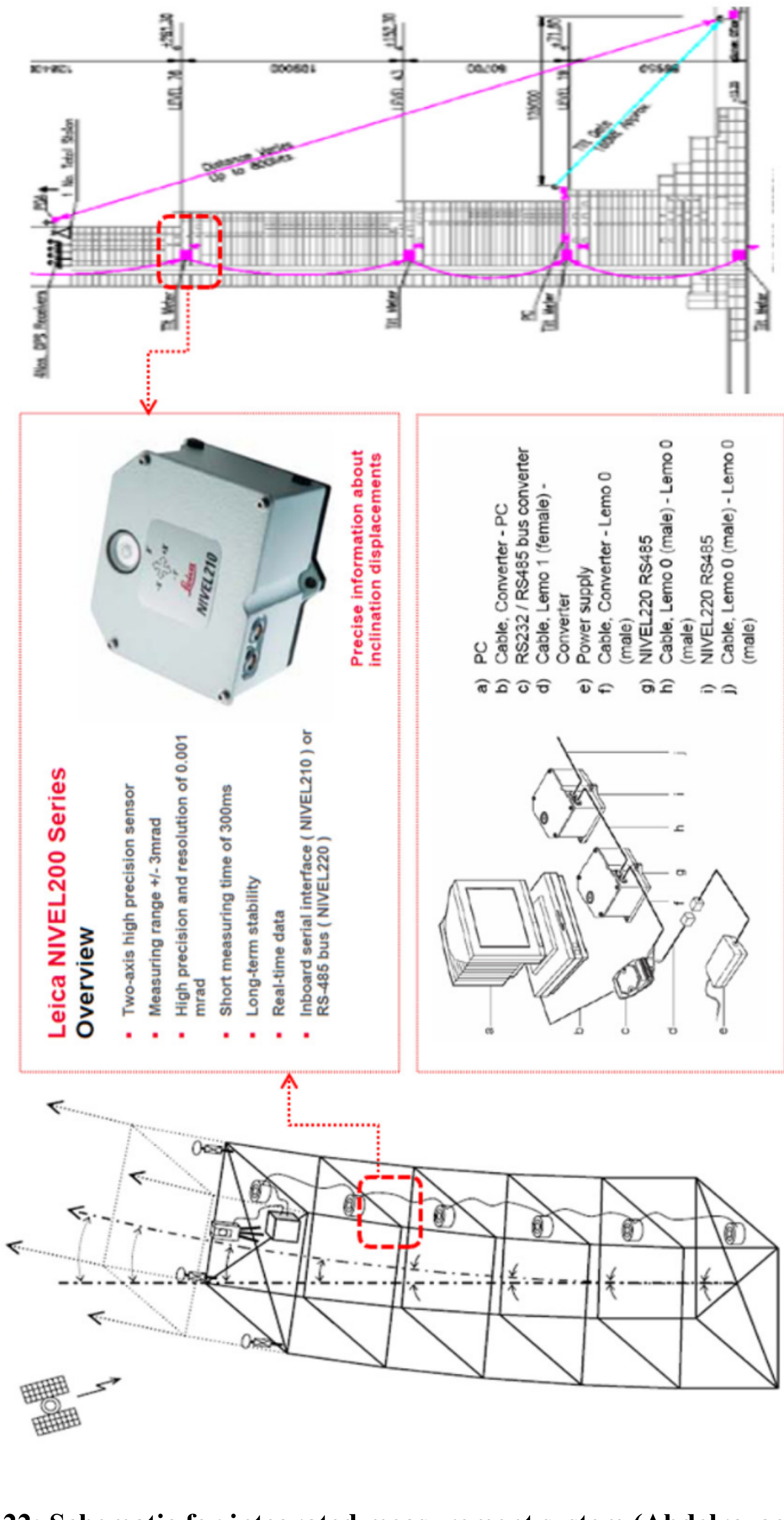

Figure 22: Schematic for integrated measurement system (Abdelrazaq, 2012)

With the surveying monitor program now able to measure the actual building movements and displacements, a three-dimensional finite element analysis model was then constructed to considered material properties and foundation flexibility provided from the geotechnical consultants. This paired with sixteen survey points on top of the raft foundation provided a comparison between the predicted settlements as well as calibrated the 3-dimensional analysis model. From both sets of data the intent was to predict all factors affecting the construction process stated above, as well as during and after completion of the structure. Comparison between the predicted natural frequency of the three-dimensional finite element analysis and the measured frequency were within 2-3%, including higher modes (Abdelrazaq, 2012). Figure 23 is the recorded data at all column/wall locations and center core wall shortening at all setback locations. Understanding the shortening at setback locations is necessary as setbacks occur at different levels for each wing of the structure. It is interesting to note that the displacement of the survey points (Figure 23) follows the shape of the shrinkage trend from Figure 21.

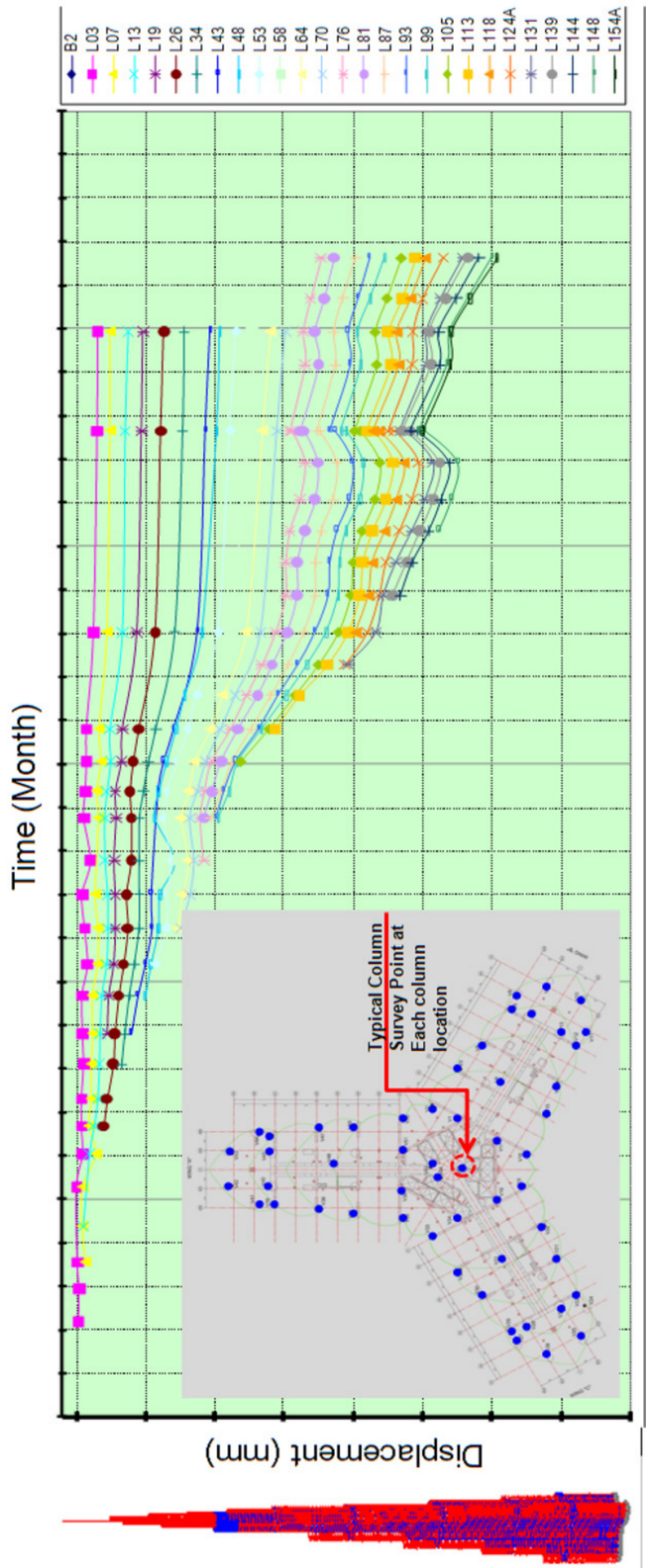

Figure 23: Typical survey points at all column/wall locations and center core wall shortening (Abdelrazaq, 2012)

3.5.3 Permanent real time structural health monitoring program

For the permanent real time structural health monitoring program an immense amount of strain gauges, along with other instruments, had to be placed all over the Burj Khalifa. The following lists all instruments used for monitoring the structural health of the structure: 197 electrical resistance strain gauges, 197 electronic extensometer vibrating wire strain gauges (both in the tower's concrete), 24 embedment vibrating wire strain gauges, 3 gauge rosettes, and 2 gauge rosettes at the load cells (all placed in the raft foundation), 3 pairs of accelerometers at foundation level, 6 pair of accelerometers at levels 73, 123, 155, 160M3, Teir 23A, and top of pinnacle, a GPS system, 23 sonimometers, and finally a weather station at level 160M3. For definitions of each instrument and how it integrates in the system, see the glossary of terms at the end of the report.

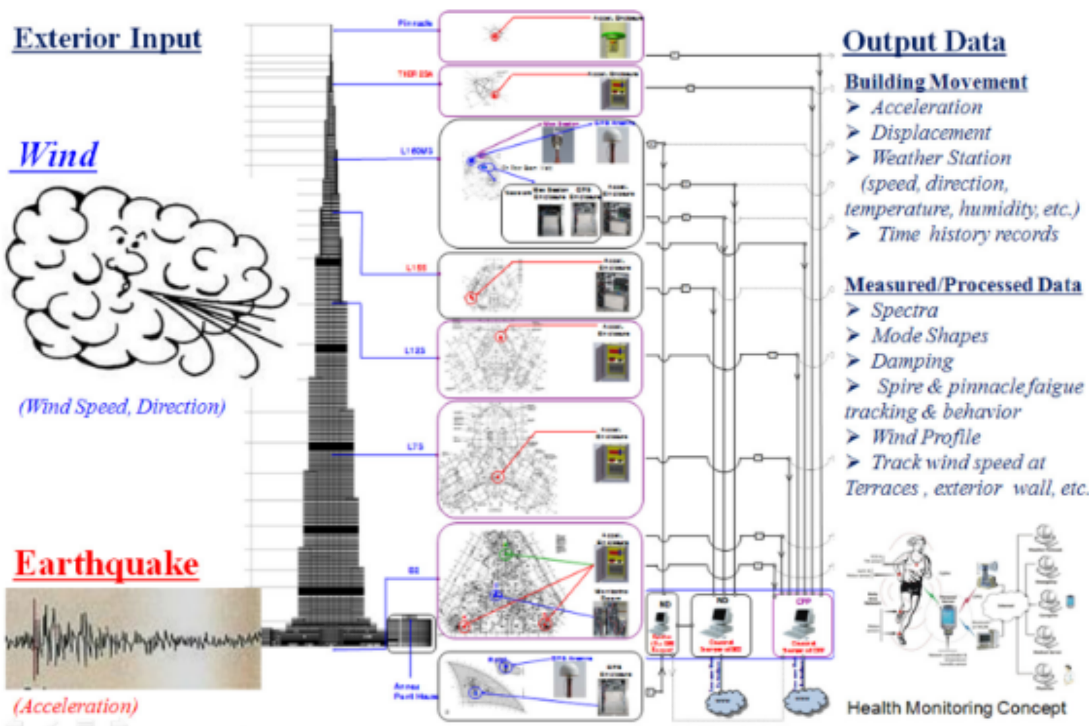

Figure 24: Permanent real-time structural health monitoring program (Abdelrazaq, 2012)

Since the completion of the installation of the structural health monitoring program, the structural systems characteristics have been identified. The following behaviors and measurements have been recorded: building acceleration at all levels, building displacement at level 160M3, wind profile along the building height, building dynamic frequencies, including higher modes, expected building damping at low amplitude wind and seismic events, and time history records at the base of the tower.

Samsung C&T provided 100% funding of the temporary real time monitoring programs and partial funding of the permanent real time monitoring programs. Thus, findings must remain confidential. However, comparison between predicted building behavior and the in-situ measured responses have been excellent (Abdelrazaq, 2012)

Chapter 4: Foundation Design

4.1 Background

Numerous factors must be taken into consideration for the design of any tall building and their foundations. The buildings weight, lateral forces imposed by wind loading, cyclic induced vertical and lateral loading, seismic action, dynamic response, location, and relative cost all must be assessed for effective foundation design (Poulos, 2016). With such tall buildings, total dead and live loads over the height of the structure will be substantial. Furthermore, building weights increase non-linearly with height affecting settlements. This will cause greater settlements initially as larger floorplans will be placed closer to ground level.

Dynamic forces imposed by wind and seismic loading can pose sizeable lateral loads and overturning moments on the foundation system. Thus, moments can impose increased vertical loads on the outer piles as well as introduce cyclic loading on a foundation system. In many cases wind-induced lateral loads govern design due to location of tall structures. Subsequently, wind-induced cyclic loading may degrade foundation capacity and cause increased settlements. Whereas dead and live loads pose strictly compressive loads to a foundation, cyclic loads have the potential to reverse loading and cause uplift. This action may in turn increase settlements through repetitive loading and unloading of soils.

Numerous factors effect foundation design for tall buildings such as the Burj Khalifa. Foundation and building behavior are highly interactive, foundation movement is caused by the building loads, which in turn influences the building behavior. Foundation design of high-rise buildings is a performance-based soil-interaction issue not limited to traditional methods where a safety factor is applied to a bearing capacity. The type of foundation system is dependent on the main design elements as well as local construction conditions, cost and project program requirements. Common foundations for high-rise buildings are: raft or mat foundations, compensated raft foundations, pile foundations, and compensated piled raft foundations. Compensated foundations occur when excavation takes place before the construction of a foundation has begun. The soil in turn acts as overly-consolidated for the first stage of the building's construction. Meaning the soil will not be loaded past a stress it has already been subjected to.

A compensated piled raft foundation was used to support the Burj Khalif. Located in the United Arab Emirates, the tower is founded on a 3.7 meter (12.1 foot) thick raft supported on 194 bored piles, 1.5 meters (4.9 feet) in diameter and extend nearly 50 meters (164 feet) into the soil. Foundation design can be difficult in middle eastern countries, especially for high-rise buildings. The following factors are present in such climates: weak soils with variable cementation (void volume), highly heterogenous, deposits extremely loose in nature, chemically aggressive ground conditions, and ground conditions that do not improve with depth (Poulos, 2009). These factors required a thorough geotechnical investigation that included testing. The coming sections illustrate the geotechnical investigation, foundation considerations, foundation models and analyses, foundation design, as well as the pile load testing and analysis used for the foundation design of the Burj Khalifa.

4.2 Geotechnical Testing & Investigation

The geotechnical investigation was conducted in several phases as follows: 23 boreholes, in situ standard penetration tests (SPT's), 40 pressure meter tests in three boreholes, installation of four standpipe piezometers, three geophysical boreholes, laboratory testing, and specialist laboratory testing. Several more boreholes, SPT's, and piezometers were installed and conducted months later but were not part of the initial site investigation. A brief description of each test is provided to better understand their application within the foundation analysis, selection of foundation type and design parameters.

Pressure meter tests consist of two parts, a read-out unit and a probe that is inserted into the borehole. The probe is designed to apply uniform pressure to the borehole walls by inflating a membrane. Once this membrane inflates and is held at a constant pressure, the borehole walls will deform and the increase in volume required to maintain constant pressure is recorded. From this, load-deformation diagram and soil characteristics can be assumed by the measurement of pressure to change in volume of the membrane. Thus, providing a stress-strain relationship of the soil which in turn helps understand the modulus of elasticity of the soil from each borehole. The modulus of elasticity is necessary for the modeling of soils or associated stiffness applied to the raft and piles.

A piezometer is used to identify water table levels. The instrument uses a simple standing pipe with a porous tip as to allow the flow of water to reach equilibrium inside the instrument. From here the fluctuation in water table and pore pressure may be assessed when considering settlements.

The SPT is one of the oldest and most common in situ tests. First a drill digs to the designated level of testing so that a split spoon (collecting tube) may be driven into the exposed soil. The number of blows corresponds with the *N* value, which provides an indication of the relative density of the subsurface soil and an empirical geotechnical correlation to estimate the approximate shear strength properties of the soils. It is from here an approximate stiffness for each soil layer can be determined.

Some of the more sophisticated tests include, stress path triaxial, resonant column, cyclic undrained triaxial, cyclic simple shear, and constant normal stiffness direct shear were all conducted by various commercial, research, and university laboratories. These tests examine the effective stresses, pore pressure, degradation, and shear stresses under cyclic loading (Hyodo, 1991).

From the above testing and collected samples, geotechnical design parameters were assessed and are summarized in Table 2, Figure 25, and Figure 26. Results from Figure 26 are further discussed in Section 4.4 where modeling assumptions were conducted. After testing, the selection of a foundation system occurs.

Table 2: Summary of Geotechnical Profile and Parameters, adapted from (Poulos, 2008)

Strata	Sub-Strata	Subsurface Material	Level at top of Stratum (m DMD)	Thickness (m)	UCS (MPa)	Undrained Modulus E_u (MPa)	Drained Modulus E' (MPa)	Ult. Comp. Shaft Friction (kPa)
1	1a	Medium dense silty Sand	+2.50	1.50	-	34.5	30	-
	1b	Loose to very loose silty Sand	+1.00	2.20	-	11.5	10	-
2	2	Very weak to moderately weak Calcarenite	-1.2	6.10	2.0	500	400	350
3	3a	Medium dense to very dense Sand/Silt with frequent sandstone bands	-7.30	6.20	-	50	40	250
	3b	Very weak to weak Calcareous Sandstone	-13.50	7.50	1.0	250	200	250
	3c	Very weak to weak Calcareous Sandstone	-21.00	3.00	1.0	140	110	250

4	4	Very weak to weak gypsiferous Sandstone/calcareous Sandstone	-24.00	4.50	2.0	140	110	250
5	5a	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-28.50	21.50	1.3	310	250	285
	5b	Very weak to moderately weak Calcisiltite/ Conglomeritic Calcisiltite	-50.00	18.50	1.7	405	325	325
6	6	Very weak to weak Calcareous/ Conglomerate strata	-68.50	22.50	2.5	560	450	400
7	7	Weak to moderately weak Claystone/ Siltstone interbedded with gypsum layers	-91.00	>46.79	1.7	405	325	325

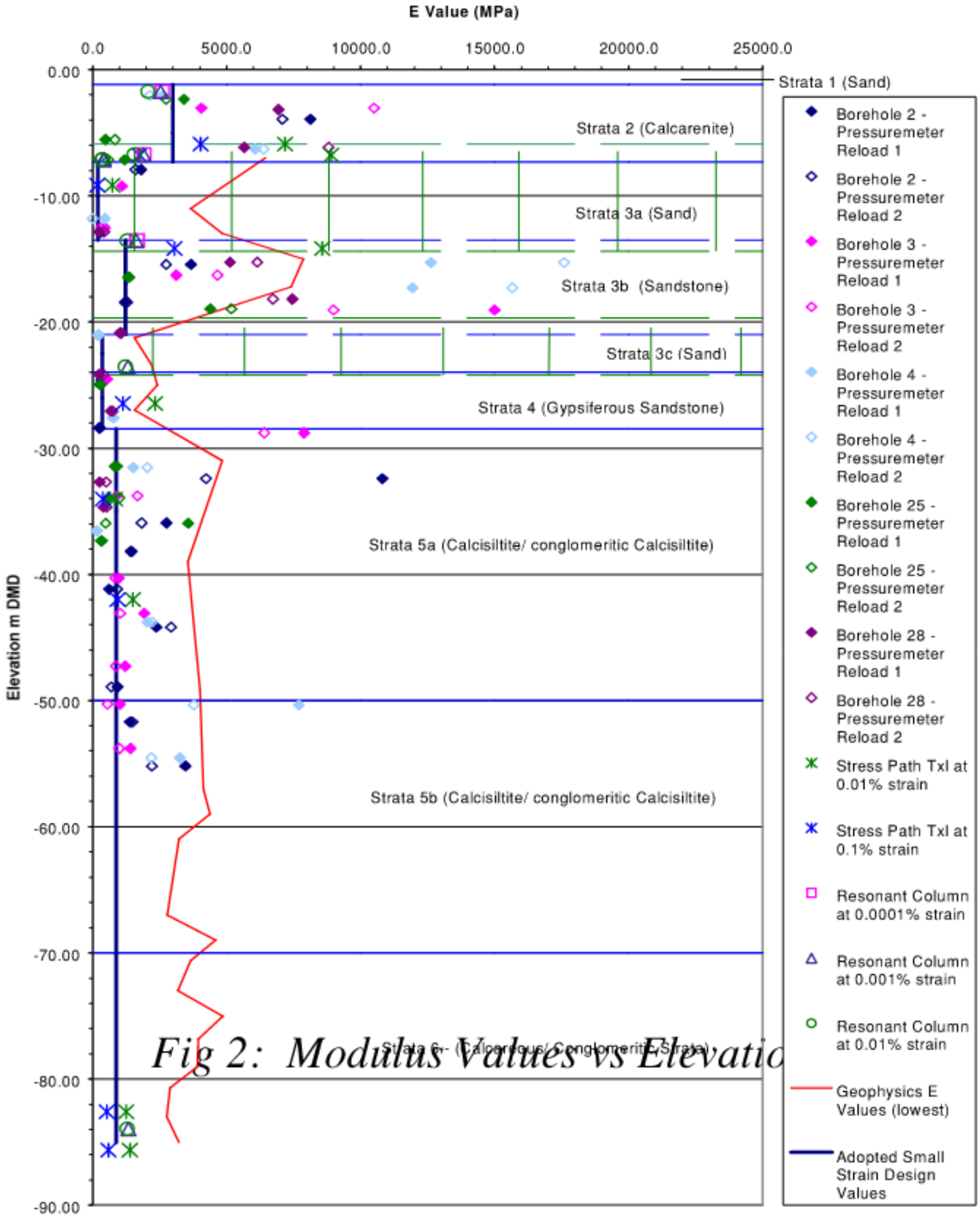

Figure 25: Summary of Geotechnical Profile and Parameters (Poulos, 2008)

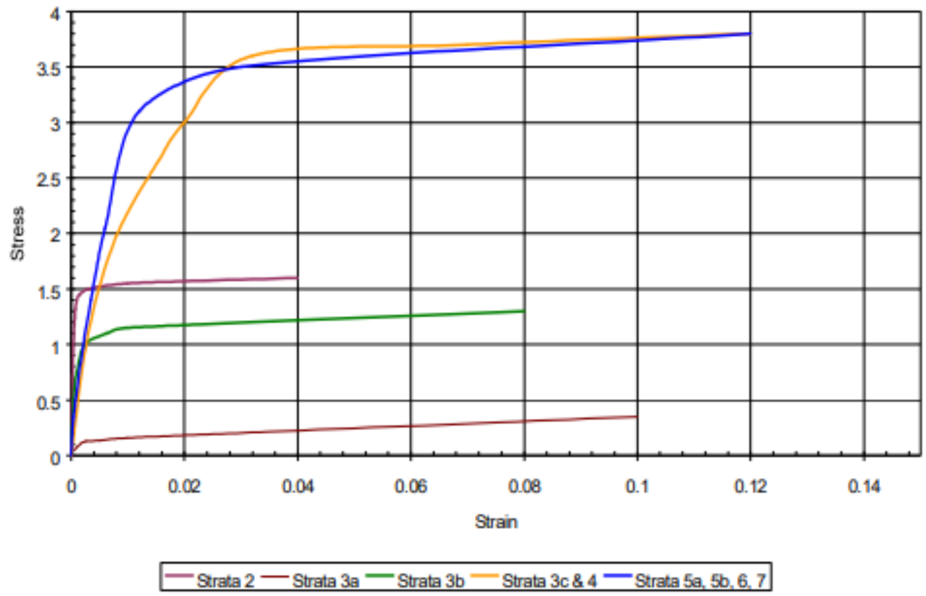

Figure 26: Non-linear Stress-strain Curves (Poulos, 2008)

4.3 Foundation Considerations

Soil assessment for foundation design is critical and without understanding the behavior of in-situ subsoil layering, uneven or unpredicted settlement of the foundation may occur and even foundation failure may occur. Several factors affect the foundation selection: subsoil layering and classification, water table level, pore pressure, and modulus of elasticity of each soil type. In the case of a more severe event, such as an earthquake, foundation failure may be caused by liquefaction of soils, lateral loading, or cyclic loading effects. The geotechnical and foundation engineers take all these characteristics into account and apply their best engineering judgement as to how the soils and foundation will interact under loadings.

Soil is made up of three components: a volume of soil, water, and air. When soils are below a water table, they are fully saturated and the volume of air in the soil is zero. When earthquake waves occur in fully saturated soils, the water-filled pore spaces in the soil collapse, which decreases the overall volume of the soil. This forces the water pressure to increase between individual soil grains, thus allowing the grains to move freely in a watery mix. Such effects are known as liquefaction, and can cause significant damage or failure of a foundation due to the sudden loss of support from below. In the case of the Burj Khalifa, the water table on site was 2.5 to 3 meters below ground level. Knowing this information, paired with granular soils due to the arid climate, the possibility of liquefaction became present. Layers between -7.3 meters and -11.75 meters posed a possibility to liquify, but due to the confining stress of the foundation, it was considered to have negligible effect on the foundation (Poulos, 2008).

Cyclic loading effects and tests were carried out and posed a potential for degradation of stiffness and residual shear strength. However, such tests assumed large strain values that would likely not be present in field conditions. Several piles tests were undertaken before and during construction to validate such assumptions and are discussed in Section 4.5.

4.4 Foundation Settlement Models & Analyses

Several models were utilized to predict the response of the foundation settlement. The main model, ABAQUS (FEA), was a Finite Element program and run by a specialist company KW Ltd. Four other models (REPUTE, PIGLET, VDISP, SOM) were created to validate the results. When modeling, boundary conditions, correct assignment of elements, and material properties all play important roles for global analysis. Geotechnical conditions play an important role in the modeling phase as elements will use assumptions from the soil data. Based on soil data, engineering judgement can be applied and a global assumption may be made. In the ABAQUS model, soil strata, tower piles, podium piles, tower and podium loadings, tower raft submerged weight, tower shearing action, and building stiffness effect were idealized based off a series of assumptions made in (Poulos, 2008).

Soil strata was modeled as a Von Mises material, or independent of hydrostatic stress. Here the uniaxial stress is compared to the Von Mises stress, and if Von Mises stress is exceeded, the material will yield. Von Mises stress is not a true stress but is a theoretical value that allows the comparison between the tridimensional stress with the uniaxial stress yield limit. Comparison of stress values were based off of the non-linear stress-strain curves presented in Figure 26.

Tower piles and podium piles were modeled as beam elements connected to the soil strata by pile-soil interaction elements. The purpose of modeling piles as beams elements, although they receive axial loading, is to isolate the frictional resistance at the interface and exclude end-bearing effects. As for tower (green portion of Figure 27) and podium (blue portion of Figure 27) loadings, each were applied as concentrated loadings at the column locations assuming the piles would not engage the raft. Meaning, the piles would have adequate capacity to carry the structure without any assistance from the raft. Tower shearing action was applied as a body load to the raft elements to coincide with the assumed directional wind action. Body loads are distributed loads applied to modeled elements rather than geometric surfaces or surfaces defined as collections of element faces (ABAQUS, Inc., 2006). Figure 27 gives a visual of the podium versus tower elements of the structure.

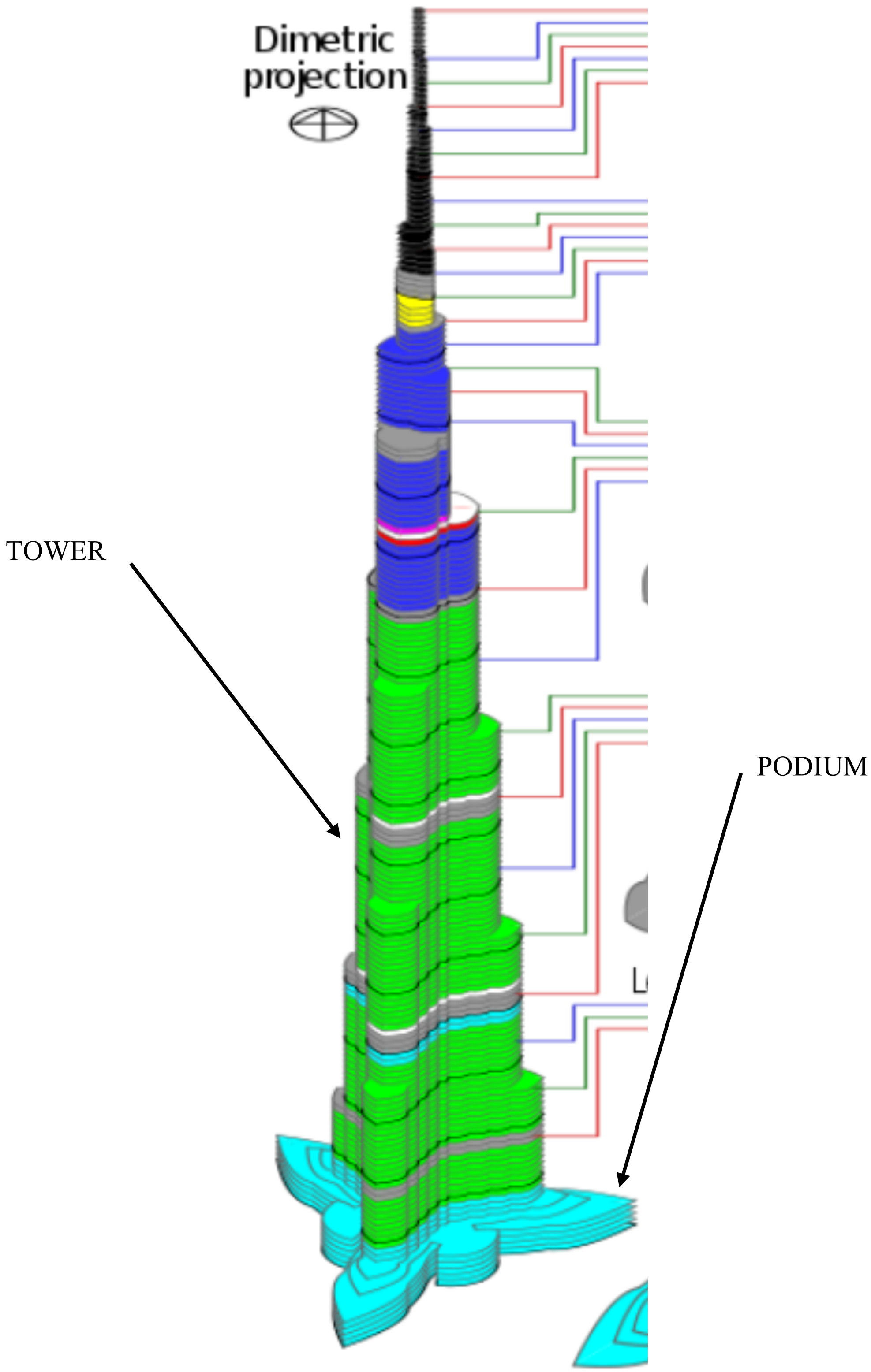

Figure 27: Tower and podium (ArabiaHorizons, 2021)

Lastly, building stiffness effects were modeled with beam elements overlaid on the tower raft elements. This allowed for the moment of inertia of each beam element to be modified to simulate the stiffening effect of the tower (specified by SOM). The stiffening effect of the tower can be thought of as a cantilever beam with a given stiffness depending on modulus of elasticity, length, and moment of inertia. In the case that the cantilever is a wide flange shape, it would have different stiffnesses depending on the direction of loading. While the modulus of elasticity and length may stay the same, the moment of inertia will be different. Similarly, the tower has different stiffnesses depending on the direction of load applied.

4.5 Foundation Design

In the initial design phase an assessment of the pile capacity was needed to adopt an ultimate compressive unit shaft friction value for each soil strata. Once this equation was reviewed, the assessed pile capacity was provided to SOM to provide layout, number, and diameter of the piles. Tower piles were specified to be 1.5 meters (4.92 feet) in diameter and 47.45 meters (156 feet) long founded 7.55 meters (24.8 feet) below ground level. Podium piles were 0.9 meters (2.95 feet) in diameter and 30 meters (98.4 feet) long founded at 4.85 meters (15.9 feet) below ground level. Thickness of the raft was 3.7 meters (12.1 feet).

Five separate models were created to validate and analyze the settlement and behavior of the Burj Khalifa's piled raft foundation. With strains assumed to stay within the small strain region of the non-linear stress strain curves, elastic modulus values were adopted in PIGLET, REPUTE, and SOM models. As for the ABAQUS and VDISP models, the non-linear stress strain curves were utilized for the soil strata. For the piles themselves, a flexible and rigid pile cap comparison was also made. With the piles being 1.5 meters in diameter and attached to a 3.7

meters thick raft, engineering judgment leaned towards the assumption of a rigid pile cap. Table 3 and Figure 28 indicate output settlements of each model with the assumptions previously stated.

Table 3: Computed Settlements, adapted from (Poulos, 2008)

Analysis Method	Load case	Settlement mm	
		Rigid	Flexible
FEA	Tower Only (DL+LL)	56	66
REPUTE	Tower Only (DL+LL)	45	-
PIGLET	Tower Only (DL+LL)	62	-
VDISP	Tower Only (DL+LL)	46	72

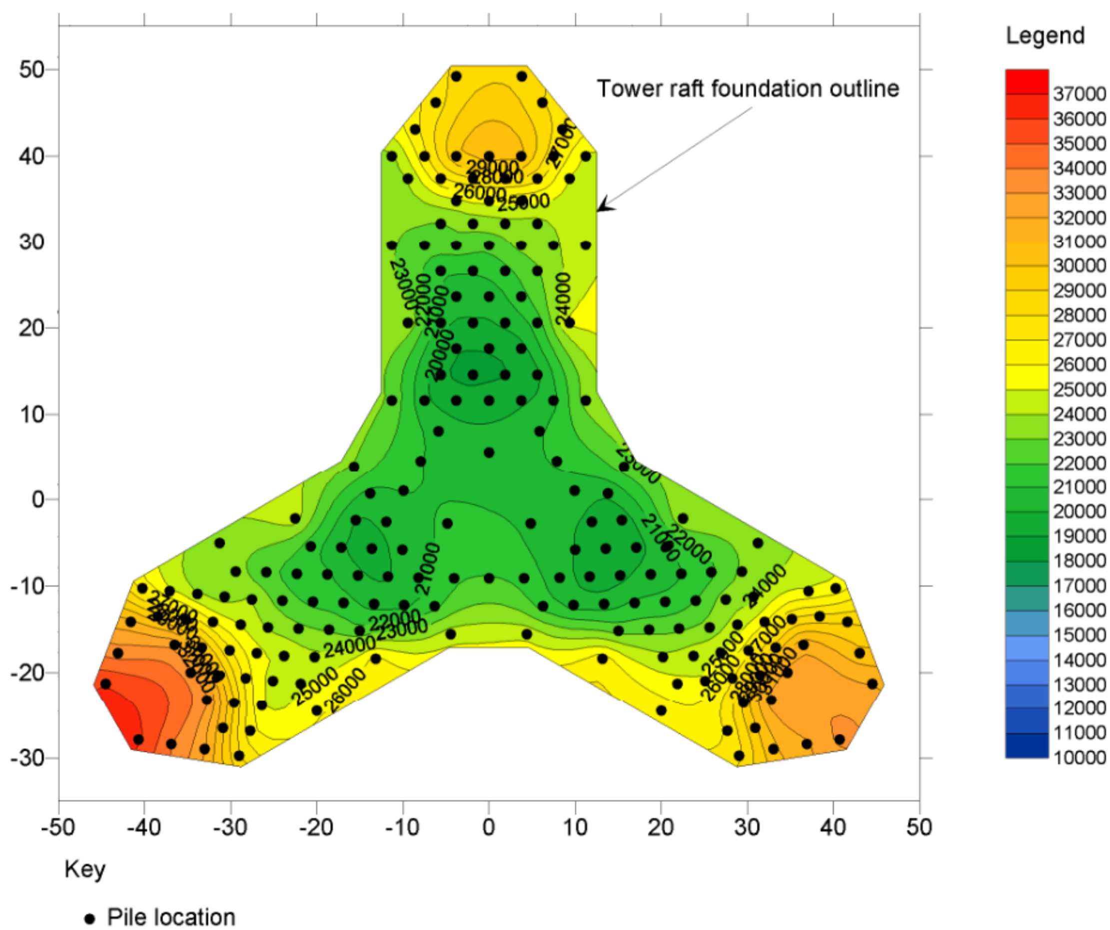

Figure 28: Computed settlements & maximum axial load in MN (Poulos, 2008)

Interestingly, all models used the same loading but computed different settlements in different locations. Figure 28 presents larger loads and settlements on the exterior of the three wings, whereas SOM's model was the inverse. Higher settlements and loads were present in the center under the tower. Several factors to consider would be the different assumptions made for each model, and the fact that SOM modeled the soil as springs connected to the raft and piles. With the initial ABAQUS runs indicating small strains in the soil strata, SOM's springs would more accurately model the seemingly elastic soil properties. Compared to the Von Mises stress assumption in ABAQUS. Another difference may have come from the superstructure stiffening effects on the foundation. SOM was able to more accurately model such effects.

Design of the foundation was found to be governed by the tolerable settlement rather than the overall allowable bearing capacity of the foundation. With settlements ranging from 45-62 millimeters (1.77-2.44 inches), the results were considered to be within an acceptable range (Poulos, 2008).

4.6 Pile Load Testing & Analysis

The main purpose of pile testing is to assess load-settlement behavior, and to verify design assumptions. Two static load test programs were utilized in the Burj Khalifa project. Seven trial piles prior to the foundation construction and eight piles during the construction phase. Dynamic pile testing was also carried out on 10 of 31 piles under the podium, together these programs allowed for the following factors to be considered: effects of increasing pile shaft length, effects of shaft grouting, effects of shaft diameter, effects of uplift loading, effects of lateral loading, and effects of cyclic loading (Poulos, 2008).

Figure 29: 6000 Ton Pile load test setup (The Constructor, n.d.)

For pile construction an immediate obstacle was the sandy subsurface soils. Upon excavation, unstable side walls would collapse. To address this issue the use of a polymer drilling fluid, rather than bentonite, was utilized. In Poulos's *Foundation design for the Burj Dubai,* he mentions how higher stiffness values and lower settlements could have been attributed to the use of polymer drilling fluid. While bentonite and polymer drilling fluid have very similar settlements and shear stresses to a certain load, the true difference appears at higher loadings. These finds can be seen in (Lam, 2015), and played an important role for cyclic loading, overall settlement, and the chemical environment of the Burj Khalifa's foundation.

Table 4 presents the datum for the axially tested piles and in the preliminary pile group. The 1.5 m diameter piles were all loaded to twice the working load while the 0.9 m diameter piles were loaded to 3.5 times the working load. None of the piles indicated any signs of geotechnical failure but instead gave sufficient evidence that the piles were more than adequate. TP5 was even loaded to four times the working load. This pile showed a rapid increase in settlement at max load however, this settlement was due to structural failure of the pile itself.

From these tests a safety factor in excess of three was assumed, giving a comfortable margin of safety.

All piles tested in Table 4 received strain gauges to observe ultimate shaft friction and when it was reached for each pile. The datum showed that skin friction down to about 30 meters below ground level appeared to be ultimate values and values below 30 meters to not have reach ultimate. Ultimate skin friction values for the piles deeper than 30 meters would be assumed to be the same values as those above the 30-meter depth.

Table 4: Pile load tests & results, adapted from (Poulos, 2008)

Pile No.	Pile Diam. (m)	Pile Length (m)	Side grouted?	Test Type
TP1	1.5	45.15	No	Compression
TP2	1.5	55.15	No	Compression
TP3	1.5	35.15	Yes	Compression
TP4	.9	47.1	No	Compression (cyclic)
TP5	.9	47.05	Yes	Compression
TP6	.9	36.51	No	Tension
TP7A	.9	37.51	No	Lateral

Table 5: Pile settlements & stiffnesses, adapted from (Poulos, 2008)

Pile Number	Working Load (MN)	Max Load (MN)	Settlement at W. Load (mm)	Settlement at Max Load (mm)	Stiffness at W. Load (MN/m)
TP1	30.13	60.26	7.89	21.26	3819
TP2	30.13	60.26	5.55	16.85	5429
TP3	30.13	60.26	5.78	20.24	5213
TP4	10.1	35.07	4.47	26.62	2260
TP5	10.1	40.16	3.64	27.45	2775
TP6	-1	-3.5	-0.65	-4.88	1536

Another attribute piles tests predict are uplift and cyclic loading. The ultimate skin friction in tension is generally reduced due to Poisson's ratio of the pile, differences in stress field, and changes in mean effective stress (De Nicola, 1993). In the case of the Burj Khalifa, a calculated ratio of skin friction to compression skin friction was about 0.6. However, a conservative assumption was made and the skin friction was assumed to be 0.5 that of compression. Similar to skin friction in tension, compressive skin friction will see the same factors effecting values. However, where concrete may slightly elongate in tension due to Poisson's ratio, compression will cause an expansion. This expansion will cause additional skin friction as a pile is loaded.

All piles under the tower (TP1, TP2, TP3 see table Table 6) were subjected to a cyclic load of 1.5 (about 45 MN). However, the medium length pile (TP1) had the best settlement after *N* cycles dived by the settlement after one cycle. Meaning that less settlement was seen during the cyclic loading. From this datum sufficient evidence was provided that cyclic loading effects would more than likely be insignificant for this building (Poulos, 2008).

Table 6: Displacement accumulation for cyclic loading, adapted from (Poulos, 2008)

Pile Number	Mean Load/Working Load	Cyclic Load/Working Load	No. Cycles (N)	Settlement after N cycles/Settlement after 1 cycle
TP1	1.0	±.5	6	1.12
TP2	1.0	±.5	6	1.25
TP3	1.0	±.5	6	1.25
TP4	1.25	±.25	9	1.25
TP5	1.25	±.25	6	1.3
TP6	1.0	±.5	6	1.1

A summary of settlements, shown in figure 30, from March 18, 2007 provides an estimate of 75% of the dead load acting on the foundation. However, these number do not represent the impact of the raft, cladding, and live loading. Together these total 20% of the overall mass of the structure. "The settlements measured during construction are consistent with, but smaller than, those predicted, and overall, the performance of the piled raft foundation system has exceeded expectations to date" (Poulos, 2008).

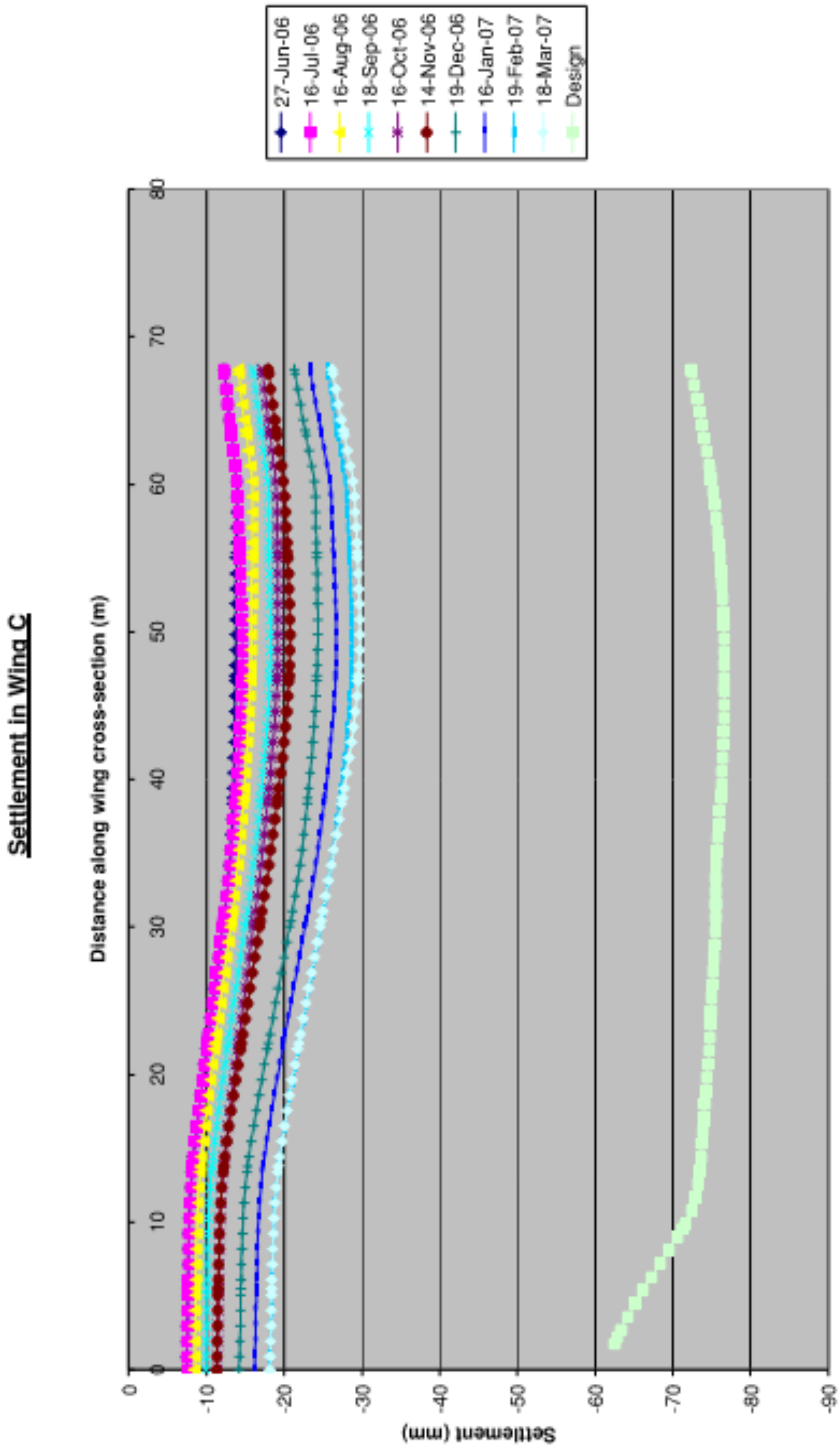

Figure 30: Measured and Computed settlements for Wing C. (Poulos, 2008)

Chapter 5: Conclusion

The Burj Khalifa is the world tallest building standing 828 meters (2,716.5 feet) high with a structural system comprised of a buttressed core. This system was first utilized in the Tower Palace III, but through engineering planning and design, was realized to have greater potential. Through wind engineering, geotechnical investigations, testing, and an adequate structural health monitoring system, crucial loading information was provided for the completion of the Burj Khalifa.

With wind loads governing design of most super-tall structures, it is important to consider dynamic affects. Wind engineering can provide a safe and accurate understanding of a building's dynamic behavior, as well as provide designers with valid wind loads to define shape and orientation of a building. With sufficient structural materials and design, the governing case for tall structures becomes serviceability or habitability requirements. It is from the insights of wind tunnel testing that engineers for the Burj Khalifa could provide a safe and comfortable experience for their clients and occupants.

The concrete buttressed core provides an adequate LFRS for high-rise structures. In the case of the Burj Khalifa, outriggers were necessary to reduce the overturning moment of the structure. This concept paired with coupling beams, increased moment of inertia, survey monitoring programs, and a structural health monitoring system were all integrated together for the completion of the concrete buttressed core.

Designing foundations for high-rise structures in the Middle East presents numerous challenges from a geotechnical viewpoint. With foundations seeing large vertical and lateral loadings, weak founding conditions, and very heterogenous soil deposits, systems must provide adequate design to withstand such parameters. This report has provided several processes in the foundation design and analysis as well as testing needed to verify assumptions during the planning and design phase. In summary, better than expected settlements, deflections, and accelerations were seen by the world's tallest building, the Burj Khalifa.

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Request for Permission: Foundation Design for the Burj Dubai

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Glossary of Terms

Accelerometers – an accelerometer uses electromagnetic sensing that measures the vibration of a structure. The force that is caused by the vibration causes the accelerometer to produce an electrical charge indicating how much force was exerted. Simply put, an accelerometer converts acceleration to measurable signals.

Clinometers/dual axis clinometers – an instrument used for measuring the angles of slope/tilt and elevation/depression of an object with respect to gravity. Measurements can either be given in angular measurements or as a percentage with reference to a level zero plane. They monitor the effect of gravity on a small mass suspended in an elastic support structure so that when the device tilts, the mass moves and causes a change in capacitance between the mass and the support. The tilt angle can be calculated form the difference in measured capacitance. A simple protractor and weighted line may act a uniaxial clinometer.

Electronic extensometer – measure the extension, compression, or shear deformation of a material sample when force is applied. Often used for finding a specimen's strain.

Electro-optical total stations – an optical surveying instrument that uses electronics to calculate angles and distances. It integrates a transit with an electronic distance meter to read distances from the instrument to a point. Data can be downloaded from the total station to a computer and supplication software to generate a map of the surveyed area.

GPS control points – points on the ground marked as known coordinates.

Leica Geosystem – versatile positioning suitable for construction sites.

Reference base station – controls the receivers, downloads data, and computes the network to determine the positions of the antennas.

Sonimometers – for ambient wind sensing in harsh environment industrial applications as a replacement for conventional mechanical propeller and cup anemometers. A sonimometer operates on the principle of measurement of the speed of sound in air.

Strain gauge/Electrical resistance strain gauge – a resistance element which changes resistance when subject to strain. As a specimen is stressed or deformed this causes the electronic wires to be thrown out of balance and a change in output voltage is recorded.

Strain gauge rosettes – an arrangement of two or more strain gauges that are positioned closely to measure strains along different directions of a specimen.

Vibrating wire strain gage – a gauge constructed so that the wire held is in tension between two end flanges. When the distance between the two flanges changes the result is a change in tension of the wire. An electromagnet is used to excite the wire and measure the resulting frequency of the vibration.

Weldable strain gauge – a standard weldable strain gage specially designed for spot welding to structures and components. Also capable of strain measurements when placed under elevated temperatures.