POLITECNICO DI TORINO

Master of Science in Civil Engineering

Hoboken Bridge:

Preliminary Design of a Pedestrian Cable-Stayed Bridge

Master Thesis

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Executive Summary

This master thesis has the objective to investigate from an economic and a technical perspective the design of a new pedestrian bridge across the Hudson River, between Manhattan and Hoboken.

Three design alternatives were analyzed considering economics, constructability, environmental impact, and maintenance. The cable-stayed bridge was identified to best serve the design challenges. The main challenge was found to be the geometry of the bridge, extremely narrow and long, as a consequence, the preliminary design was driven by the dynamic analysis and the need to decouple the vertical and the torsional mode.

In the second part of the research both an economic and financial analysis are performed, to better understand the benefit and cost that this infrastructure would bring to the community and the monetary return for a possible private investor. Moreover, an investment strategy is proposed to prove how it would be possible to finance the bridge through a Public-Private-Partnership.

1 Introduction

1.1 Problem Statement

New York City is the most sought-after destination for people all over the world to come and pursue their careers and dreams. NYC is the place to be, it is the city where people are ready to invest in projects and believe in each other. However, the side effect of this is that NYC happens to be extremely overpopulated and one of the most expensive cities in the world. Nowadays, many people would rather work in a city and then live outside, like in New Jersey, to reduce living expenses. This certified trend has greatly challenged the interstate connectivity and mobility. Ridership and traffic have dramatically increased. For example, to get to New York and get back to New Jersey seems impossible during rush hours and the facilities and infrastructures are overloaded. The PATH System and the Hudson Tunnel are overburdened with ridership.

Hudson Tunnel suffers from severe deterioration and it is in urgent need of accurate maintenance. The decrepit century-old tunnels that currently carry 500,000 daily passengers could fail at any time which leaves no other choice to the passengers but using the PATH as an alternative to get to New York City and get back. The PATH itself is not designed for the amount of traffic to which it is subjected and moreover, it is not able to face the dramatic increase in the ridership. In October 2017, the monthly ridership was 7,537,344, which increased by 1,345,976 from October 2012[1]. The daily delays and congestions are already important reasons to justify the need for improving the mobility between the two states.

Moreover, it has been recently shown how the connectivity between NYC and NJ is a big weak point with respect to resiliency and safety conditions; this problem clearly emerged during the 9/11 terrorist attack. Right after the attack to the Twin Towers, NYC was paralyzed by shutting down subway lines, crippling cellular phone service. The mayor closed lower Manhattan to make way for emergency vehicles. Thousands of people left by walking across the Brooklyn Bridge. All the commuters from New Jersey to NYC were left with no option to get back to their homes and let their families know that they were fine because there was no available infrastructure safely connecting the two states.

[1] (The Port Authority, 2018)

From this perspective, a new infrastructure, such are the herein proposed pedestrian bridge would help to deal with all the critical concerns previously mentioned. It would help to relieve some of the pressure on the current connectivity and transports, offering a new access point to both Manhattan and New Jersey and allowing for thousands of commuters daily to walk or bike to work and home.

A pedestrian bridge will bring multiple positive effects to the city: reduce traffic, pollution, promote a healthy lifestyle and meet the increasing demand for environment-friendly modes of transportation, New York City Department of Transportation has promoted many projects and systems around NYC to address and improve these topics. Improvements such as reforming traffic lines to allow for bike lanes, developing parks around the city and expanding Citi Bike throughout NYC and NJ are designed to promote cycling and walking. The proposed facility fits completely this new healthy and "green" way of approach life; it will be a key infrastructure in the connection of two communities, make walking and bicycling an easy means of commuting. This bridge will give commuters the choice of making the 20-30-minute walk or the 5-10-minute bicycle ride instead of driving or taking the PATH System.

1.2 Bridge Alternatives

Once that the need for a bridge is clear and identified, there are a lot of different alternatives important to consider for the final design. The possibilities depend on the site constraints, client needs, available budget and weather condition. In this project phase, three main bridge types were considered, an arch bridge, a suspension bridge, and a cable-stayed bridge.

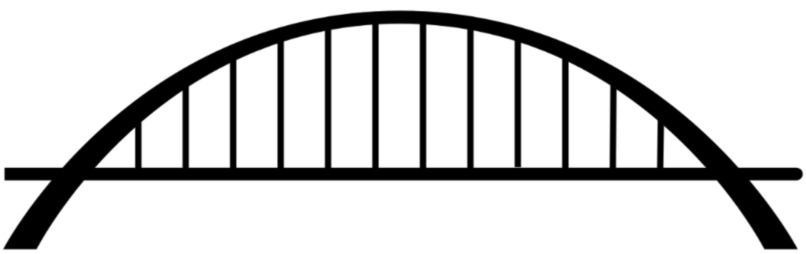

Figure 1-1 (Source: Design and Construction of Steel Bridges[2])

[2] (Gosh, 2006)

The arch structure is one of the most used shapes in history. The bridge is composed of three elements: the deck with the supports, the hangers and the arch itself. All the forces and moments are carried by the arch which transmits them to the supports, critical point of the structure. This alternative offers pro and cons. Firstly, it allows a high level of strength and the new cable-stayed arch bridge can be considered also structurally sound and redundant. However, the span length is limited compared to a cable-stayed bridge or a suspension bridge and they require great effort in the foundation construction, especially in multi-span arch bridge, due to the load distribution.

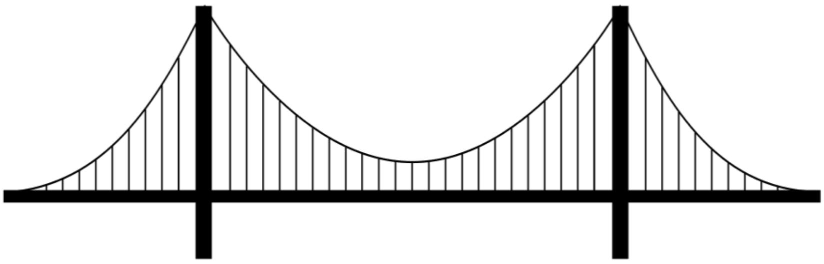

Figure 1-2: Suspension bridge (Source: In the Wake of Takoma[3])

The modern suspended bridge is a type of bridge in which one cable is suspended between two towers, at these massive cables are attached to other cables (typically vertical) which aim to hang and support the deck. This typology of bridges is the one that is able to span longer distances.

The most iconic bridge of this kind is the Brooklyn Bridge in New York, which is actually a suspended bridge with an addition of some stayed that help the bridge to carry the loads and improve the lateral efficiency. The bridge was started in 1869 and finished fourteen years later in 1883. It connects Manhattan to Brooklyn, from which the name, it is 5,229 feet long with a main span of 1,594 feet and have one of the heaviest towers ever built entirely made of limestone and granite.

The load path is slightly different from the arch bridge because the live loads are transferred to the cables and then to the towers. The anchorage system in this alternative is of primary importance and in case of absence of bedrock huge counterweights must be used.

[3] (Scott, 2015)

The great advantage of this solution is the possibility to design a gigantic main span that can be easily around 4000 ft and arrive, thanks to a particular design approach to 6000 feet. In fact, this solution is particularly fascinating for the Hoboken Bridge because it would allow the construction of no peer in the Hudson, with a significant reduction of the construction cost. On the other hand, the maintenance cost would be higher compared to an arch bridge or a cable-stayed because the substitution of the catenary requires the complete lifting of the deck.

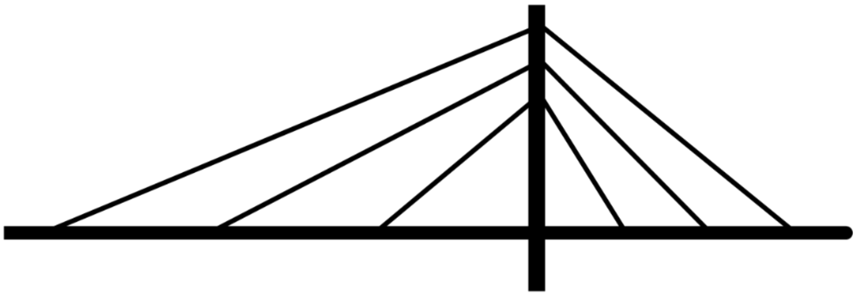

Figure 1-3: Cable-Stayed bridge (Source: In the Wake of Takoma)

Lastly, a cable-stayed bridge is a continuous girder with one or more towers erected above piers in the middle of the span. From these towers, cables stretch down diagonally (usually in both directions) and support the girder. These bridges are optimal for spans longer than cantilever bridges and shorter than suspension bridges.

Cable-Stayed bridges are much easier to build, once the towers are erected the deck can be constructed by cantilevering out from them and the cables act both as temporary and permanent supports to the bridge deck. These bridges are not just easier to build and consequently cheaper but are also much stiffer than the suspension one, so the deformation of the deck under live load is strongly reduced. The only problem with this bridge is that the inclinations of the cables produce a high compression into the deck especially in the last cables which are the ones with a smaller angle between the deck and the cable.

In these days usually, the design of a cable-stayed bridge is preferred to a suspended one, due to the construction simplicity and costs reduction, while suspended remains the best solution if large distances must be spanned.

There are three main typologies of cable-stayed bridges, the differentiation depends on the arrangement of the cables. The three categories are shown in Figure 1-4 and are introduced in the following paragraph.

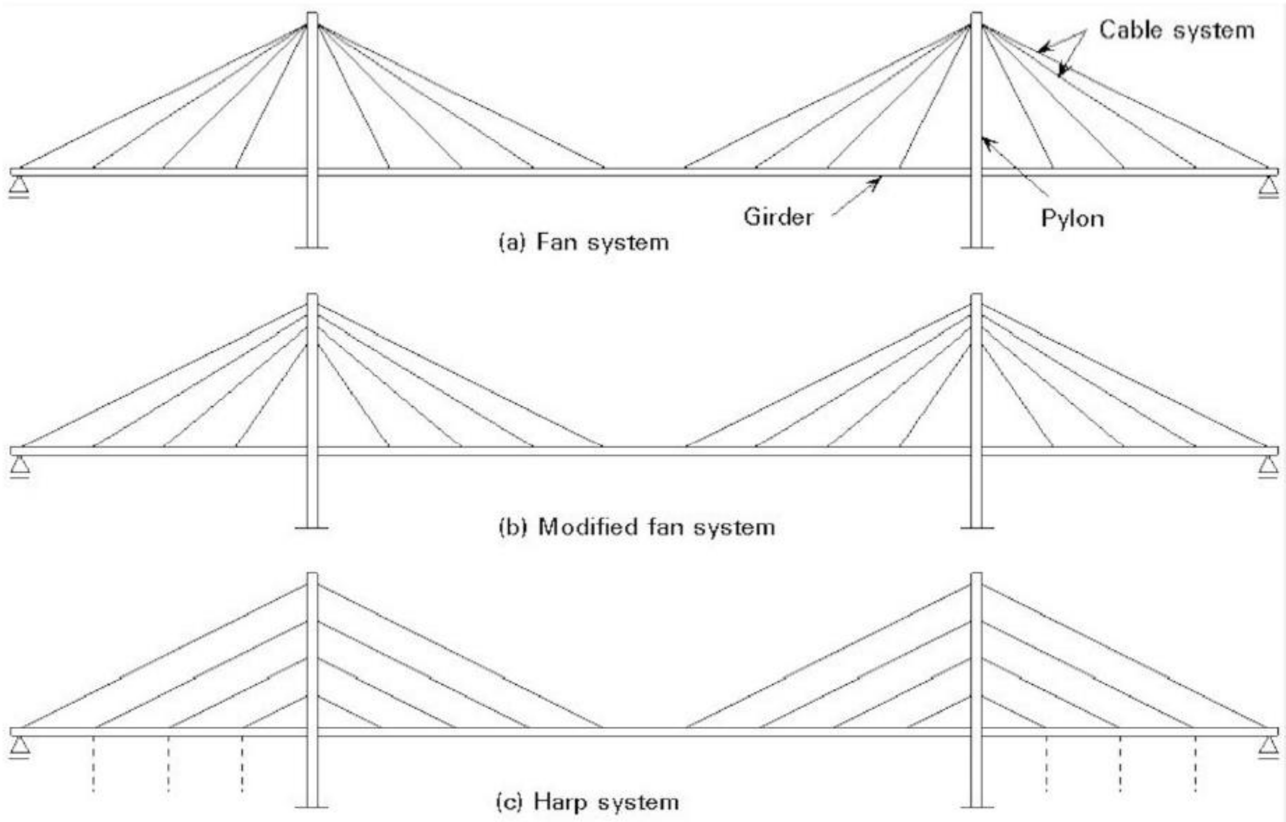

Figure 1-4: Cables Arrangement

Fan Arrangement:

In this pattern, all the stay cables are attached to a single point at the top of each pylon as shown in Figure 1-4a. the smaller slope of the stay cables results in a smaller horizontal component which reduces the compression force in the deck. Moreover, this relatively steep slope of the cables results in a smaller cross section of the cables. However, by increasing the length of the span, and consequently the number of cables, the connection at the top of the pylon became strongly difficult to realize and are hence suitable for moderate spans with a limited number of stays.

Semi-Fan Arrangement:

This type of arrangement is in between the fan and the harper. As shown in Figure 1- 4b. the cables are mainly anchored to the upper part of the pylon. This arrangement allows the engineers to have similar propriety of the fan arrangement without heaving the problem of joining in the same point a large number of cables. The real issue of the fans arrangements is that there is the need of really stiffness pylon as it acts as a cantilevered tower as all the cables are anchored at the top.

Harp Arrangement:

In this arrangement cables are almost parallels and they are attached to different points along with the tower (Figure 1-4c). As mentioned before this arrangement produce a higher compression into the deck and larger cable cross section is needed. However, the different position of the cables along the pier, which cover almost all the length of it, reduce the displacement of the tower and the rigidity is so four times higher than the normal cantilever and allow the tower to be much more elastic. In addition to that, architects use to prefer this solution as it gives a more pleasant and regular view.

Cable-stayed bridges differ also depending on the tower shape, in Figure 1-5 the most used shapes are presented.

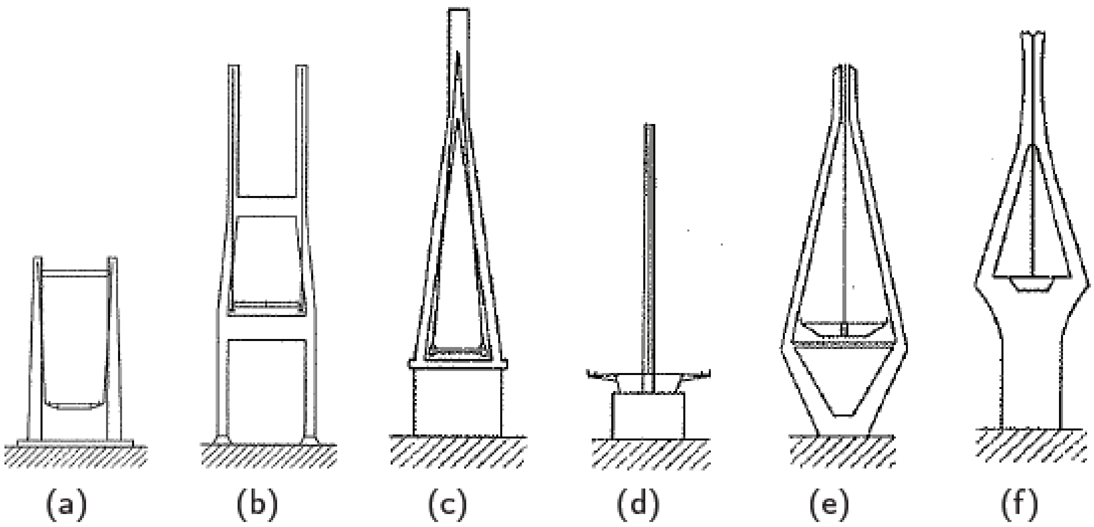

Figure 1-5: Tower Shape (Source: Gimsing, 1983)

The selection of the tower type depends on possible constructability advantage, the cable arrangement, the type of the deck used and from the general structural behavior of the bridge[4].

[4] (Gimsing, 1983)

In the following pages, it will be possible to understand how this choice influence the load distribution and the modes of the structure. In fact, one of the purposes of the research is to show how to select which shape best fits the scenario, through a simple dynamic analysis.

The most used shapes are the H, Figure 1-5b., and A shape, Figure 1-5e. The first one is more used on the wide bridge and it is generally cheaper to build. The second one is characterized by a better torsional behavior and it is usually preferred on a long and narrow bridge[5].

1.3 **Bridge Concept**

The proposed bridge is a cable-stayed non-vehicular bridge that will connect Hoboken, New Jersey to 14th Street, New York City. The proposed bridge spans 4160 ft long. It consists of three spans, with the middle span movable, and a modified A-frame tower.

This long span and movable pedestrian bridge are one of its kind and it is going to be costly. However, the bridge is designed considering optimized construction methods and reducing construction cost. On the other hand, the bridge will help improve the economy, especially on the New Jersey side. There are many potential direct and indirect revues markets. A small fee to charge for entering the bridge is reasonable and it is considered since the bridge serves as a platform for other alternative modes of transportation. These fees would go toward the movable equipment operation and maintenance. Moreover, events and attractions could be held on the bridge at an additional cost such as using the bridge to watch fireworks, fun walks, runs and more. Furthermore, more commuters will use the Citi Bikes which is indirect revenue that would count for.

The cable-stayed bridge design is chosen for the advantages it offers compared to another structural system such as suspension or truss bridges. Some of these advantages are a long span, lightweight, constructability, and maintenance. Cable-stayed bridges are very efficient structural system due to their lightweight deck and large lever arms. Compared to the traditional suspension bridge design, the cable-stayed bridge uses fewer cables and they are capable of handling more pressure allowing the deck to maintain its shape under

[5] (Davalos, 2000)

live load. As a result, the overall system has more rigidity. Moreover, they are ideal for long spans and their construction is more efficient and faster. Both, the tower and superstructure can be built parallel saving tremendous time which translates into saving on both labor and equipment cost. In a cable-stayed bridge, the strands are replaceable making the bridge more maintainable compared to a suspension bridge. Most importantly, the cable-stayed bridge is aesthetically stunning and would be a nice addition to the iconic structures of New York.

1.4 **Preliminary Design Philosophy**

The preliminary design of the bridge is composed of an iterative approach of 4 main steps: alignment, section assumptions, 3D modeling to perform dynamic optimization and structural assessment[6].

The first step required a lot of different sites visit of the Hudson riverside to understand the best location of the bridge, both from a structural and social point of view. Secondly, following the advice of Prof. T. Zoli and using previous bridge design projects, preliminary sections dimensions were hypothesized.

Through a dynamic analysis, the best span length and the tower shape was identified, the new aspect that the research wants to highlight is how it possible to reach a preliminary design starting from a dynamic perspective. The excessive sway motion of the Millennium Bridge in London has focused a great deal of attention on serviceability issues associated with Pedestrian-Induced Vibration (PIV).

A recent CEB-FIP publication Bulletin 32 Guidelines to the Design of Footbridges (November 2005) devotes an entire chapter to dynamics but includes little or no discussion of any other aspect of footbridge dynamics (wind, seismic, member loss). While PIV represents a critical aspect of design and dominates current footbridge research, a number of recent major footbridges in the US have been very much influenced by other dynamic design considerations[7]. Lastly, different load combinations were applied to the model and

[6] (Salvadori, 1981) (Siegel, 1961)
[7] (Zoli, et al., 2010)

four different structural design assessments were performed, under ultimate limit state condition.

2 Constraints

2.1 Vessel Track

The Hudson River is one of the busiest rivers in New York carrying different kind vessels characterized by various dimension. The vessels vary from personal boats, ferries to enormous cruises. This puts a great constraint on the number of piers in the water and their dimensions. Moreover, there is a strict constraint on the clearance height in correspondence of the middle span where the higher vessels are supposed to navigate. The height of the deck from the waterline in correspondence of the middle span is 150 ft and, therefore, the middle span is designed to be lifted up to a maximum of 80 ft, in order to reach a total height of 230 ft which is compatible with the maximum clearance that needs to be available for the big cruise ships to access the river. This height is also in accordance with the clearance of the Verrazano Bridge which is 228 ft high. Vessel tracking website and the vessel traffic service manual for NYC were consulted to check the constraints on the beamwidth of vessels and the room for maneuver that need to be considered for these vessels when driven by auxiliary ships in two the channel. One of the largest vessel ships that pass through the Hudson River cruise is the Oasis of the Seas. This huge cruise ship presents a maximum beam of 198 ft and in order to give room for maneuver, a minimum of 500 ft need to be considered on both sides of the middle pier allowing enough clearance.

2.2 Social

Pedestrian bridges in general play a great role in connecting communities and increasing footprint in the surrounding areas. Building a pedestrian bridge between NYC and Hoboken will benefit both areas by attracting new businesses and investments such as shopping centers and restaurants. Particularly it will make the connected areas even more valuable leading to improvements for reactional facilities and residential housing. Nowadays young people are forced to leave NJ and it is mainly because of the lack of walkability and street connectivity. One of the goals of designing the new bridge is to increase the employment rate in the New Jersey area and attractiveness for the local community.

The bridge will be constructed in one of the busiest areas in New York and New Jersey areas. This has risen many constraints and drove the design approach and construction methods. Businesses, Stevens Institute of Technology, 11th Avenue and vessel traffic on the river will be disturbed during construction. However, the bridge is designed to optimize the construction stages and building bridge fast, while minimizing the noise and environmental disturbance.

The cable-stayed bridge is clean, elegant and stunning. One of the reasons that drove the choice towards the cable-stayed option is its fluidity and how it is going to beautifully blend with structures and parks on both sides of NJ and New York City. The construction constraints are mostly related to the areas near the plazas, in between the interchanges. However, on the NYC side, there is an old unused pier that can be utilized. There is also the Hudson River Greenway which the bridge will connect to. On the New Jersey, the Steven Institute of Technology would welcome this connecting infrastructure opening the opportunity to new students from NYC to join the school given the commute will be more efficient and much faster. Furthermore, there are many urban planning design options that could occur here, such as parks and sports fields. Overall, the visual impact carried by the bridge could increase the value of both cities from an architectural point of view.
The bridge promises to bring new life into the area, connect the community and allow people to commute faster and at better conditions and increase healthier lifestyle. It is important that our project reinforces the compatibility of residential and industrial areas of the existing neighborhoods. The land use will surely change but this can benefit the local economy.

2.3 **Geotechnical Report**

Foundation is one the driving cost for a bridge. Therefore, a study was done to examine the depth to rock and the capacity on NJ, NYC and Hudson River. The geotechnical information is derived examining the Hudson Tunnel geotechnical report:

STV kindly provide the results of the geotechnical test performed for the Hudson Tunnel, which location is less than one mile from Hoboken Bridge. Thanks to this information we are able to prove that the soil under the bridge has good mechanical properties and no particular consolidation is needed with the related extra cost.

Rock Information – ISRM Method				
Location	Depth to the rock Range	Rock Description	Bulk Specific	Unit Weight, pcf
Hoboken	34-130	Dark Red-Brown Siltstone	2.59	149
Hudson River	37-165	Red-Brown,Gray and Arkosic Sandstone	2.49	145
Manhattan	10-125	Dark Gray Schist	2.77	161

Table 2-1 Geotechnical Information (Source: STV Corporation)

3 Preliminary Design

3.1 Alignment

The process of selecting the bridge alignment, the location, the geometry, accessibility, and the location constraints were considered and studied. The purpose of the bridge is to serve as many people as possible. It is to get the commuters as close as possible to other modes of transportation such as subways and buses or close enough to allow them to continue walking or biking to their work. As a result, three alignment alternatives are examined. The first alignment is alignment A as shown in Figure 3-1: Possible Alignment. In this option, the bridge would the downtown area of Manhattan side to Jersey City, NJ. However, this alternative is very close to the PATH System and the Holland Tunnel which make pointless to build this kind of infrastructure there. The second alternative connects the Hudson Yards area on the Manhattan side with the Hudson County on the New Jersey side. However, the Hudson yard is very dense with several stakeholders and constructions in progress. Building a bridge there will require dealing with a lot of stakeholders and it will be very expensive to build, due to the property value, too, due to the interference and complication that may be caused by the ongoing construction in the area. This alignment is also close to Lincoln tunnel which is already serving the connection between those areas.

Therefore, the chosen alignment is the third alternative. Bridge Alignment - C connects Chelsea, NY with Hoboken, NJ. After examining and studying the area, several advantages emerge in building along this alignment comparing to the two mentioned earlier. For example, the Hoboken routing would greatly improve the access to the Hoboken and Jersey City waterfront business district. These areas are an important economic engine for Hudson County. The building of the pedestrian bridge will serve both cities with a combined population of nearly 300,000 people and this just on the New Jersey side. It will also advance NJDOT's goal of making New Jersey "a state where people choose to walk and bicycle."

Furthermore, another benefit of choosing this location is taking advantage of the elevation height at Stevens Institute of Technology's Castle which 87 feet (Figure 3-2). This helps to achieve 150 ft clearance in the middle span before moving the middle span upward by another 80 feet and maintaining a slope to only 4°. This is an option will connect to the Greenway where commuters could continue walking or biking. It also connects to other modes of transportation nearby as shown in Figure 3-2. There is also the ability to connect the bridge to High Line as a feature of a green infrastructure if that comes to be desirable at any time by the client.

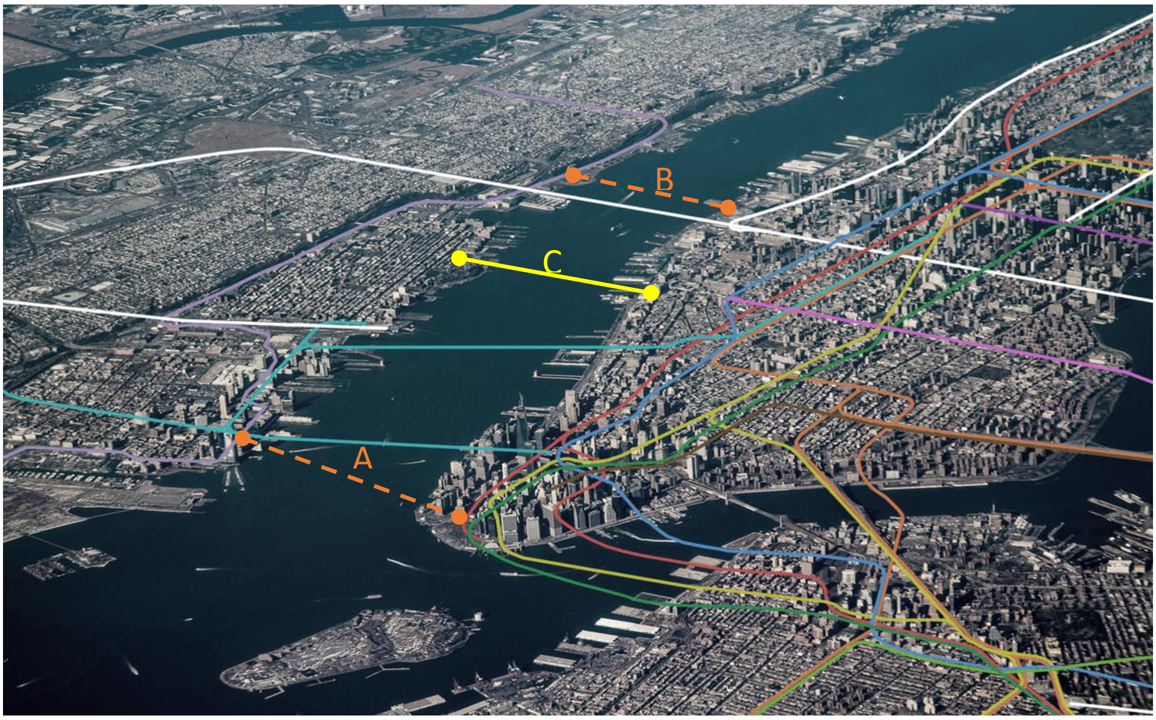

Figure 3-1: Possible Alignment (Figure made by the author)

Figure 3-2: Chosen Bridge Alignment (Figure made by the author)

3.2 **Cad Design**

The first step in the structural design is the raw initial sketch of the bridge. In this project, the initial proposal is a cable-stayed bridge which presents an overall height of the deck above the waterline in correspondence of the middle pier of 150 ft. The number of the piers, their placement and consequently the span lengths are driven by the main objective of reducing the possibility of having vessels' impact on the piers. This aim is achieved reducing the number of piers to the minimum and having the piers themselves far enough from the vessels' track lines. All the three piers are 300 ft high above the deck and the spans considered as the length of the deck before and after each pier are the same and equal to 1360 ft. The first proposal is then mainly symmetric (Figure 3-3).

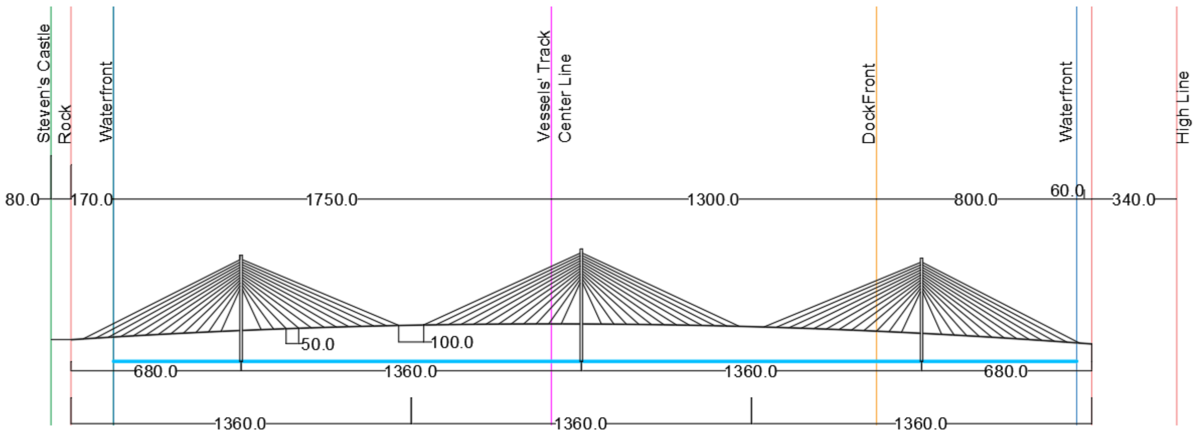

Figure 3-3 First Sketch-Big Middle Span Option (Figure made by the author, SAP2000)

Since one of the main features of this infrastructure is the movable middle span, a key point in the designing process is the optimization of its configuration finalized of making the lifting operation as easier as possible. This objective leads to the second option for this bridge, which presents a middle smaller span (Figure 3-4).

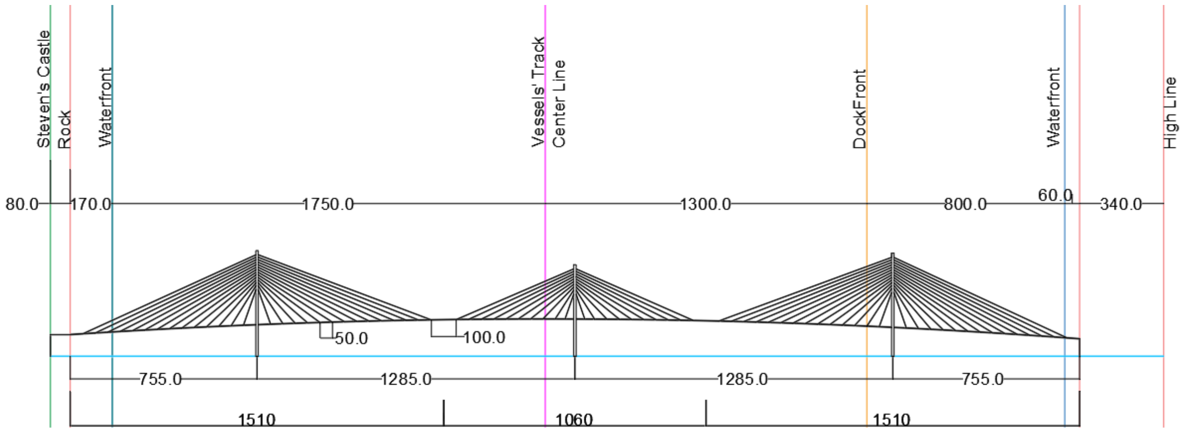

Figure 3-4 First Sketch-Small Middle Span Option (Figure made by the author, SAP2000)

3.3 Materials and Sections

In the definition of the bridge models, the first step is defining the materials, section and section properties associated with all the elements used in those models.

The chosen materials for this project are:

- Concrete 5 ksi lightweight. This material is used exclusively for the deck elements.
- Steel A992F_y50: This material is used for the piers and for the girders
- Steel A416G_r270. This material is used exclusively for the cables.

The principal mechanical properties of these materials are listed in Table 3-1

Each one of these materials is later on associated with one or more elements used to build the bridge models: longitudinal and transversal girders, the towers, the deck, and the cables. The chosen sections for these elements with the corresponding material are presented in Table 3-2

Material	Young's Modulus E	Unit Weight	Poisson v
	kN/m^2	kN/m^3	
Steel A416G$_r$270	1.96E+08	76.97	0.3
Steel A992F$_y$50	1.99E+08	77.01	0.3
Concrete 5 ksi Lightweight	2.11E+07	18.85	0.2

Table 3-1: Material Properties (Table made by the author)

Elements	Material	Section Type	Section Dimension	Values
Cables	Steel A416G$_r$270	Circular	Diameter (ft)	0.15
Towers	Steel A992F$_y$50	Square Tube	Thickness (ft) Outer Length(ft)	0.5 10.00
Longitudinal Girders	Steel A992F$_y$50	I/Wide Flange	W36x231	-
Transversal Girders	Steel A992F$_y$50	I/Wide Flange	W30x124	-
Deck	Concrete 5 ksi Lightweight	Membrane	Thickness (ft)	1.00

Table 3-2: Section Properties (Table made by the author)

		Stainless Steel Structural Cables						Stainless Steel Wire Rope			Galvanized Structural Cables						Galvanized Wire Rope				
		Standard Type 316			High Strength 316			Type 316			Standard Strand (ASTM A586)			Full Locked			(ASTM A603)				
Diameter inch	mm	Product Number	Breaking Strength kips	kN	Product Number	Breaking Strength kips	kN	Product Number	Breaking Strength kips	kN	Product Number	Breaking Strength kips	kN	Product Number	Breaking Strength kips	kN	Product Number	Breaking Strength kips	kN	Diameter inch	mm
0.250	6.4	AS10-0250	7	31				AS15-0250	6	27										0.250	6.4
0.375	9.5	AS10-0375	15	67				AS15-0375	12	53							AS25-0375	13	58	0.375	9.5
0.500	12.7	AS10-0500	26	116				AS15-0500	23	102	AS20-0500	30	133				AS25-0500	23	102	0.500	12.7
0.625	15.9	AS10-0625	41	182				AS15-0625	35	156	AS20-0625	48	214				AS25-0625	36	160	0.625	15.9
0.750	19.1	AS10-0750	48	214				AS15-0750	50	222	AS20-0750	68	302				AS25-0750	52	231	0.750	19.1
0.787	20.0				AS11-0787	67	298							AS22-0787	83	368				0.787	20.0
0.875	22.2	AS10-0875	64	235	AS11-0875	80	356	AS15-0875	67	298	AS20-0875	92	409	AS22-0875	100	445	AS25-0875	70	311	0.875	22.2
1.000	25.4	AS10-1000	89	396				AS15-1000	85	375	AS20-1000	122	543				AS25-1000	91	405	1.000	25.4
1.024	26.0				AS11-1024	112	498							AS22-1024	140	621				1.024	26.0
1.125	28.6	AS10-1125	116	516	AS11-1125	129	574	AS15-1125	106	472	AS20-1125	156	694	AS22-1125	174	773	AS25-1125	116	516	1.125	28.6
1.181	30.0				AS11-1181	149	663							AS22-1181	193	858				1.181	30.0
1.250	31.8	AS10-1250	139	618	AS11-1250	169	752	AS15-1250	129	574	AS20-1250	192	854	AS22-1250	219	976	AS25-1250	144	641	1.250	31.8
1.375	34.9										AS20-1375	232	1,032				AS25-1375	176	783	1.375	34.9
1.417	36.0				AS11-1417	214	952							AS22-1417	277	1,230				1.417	36.0
1.500	38.1										AS20-1500	276	1,228	AS22-1500	310	1,380	AS25-1500	208	925	1.500	38.1
1.575	40.0													AS22-1575	342	1,520				1.575	40.0
1.625	41.3										AS20-1625	324	1,441				AS25-1625	246	1,094	1.625	41.3
1.750	44.5										AS20-1750	376	1,673	AS22-1750	450	2,000	AS25-1750	286	1,272	1.750	44.5
1.875	47.6										AS20-1875	432	1,922	AS22-1875	510	2,270	AS25-1875	328	1,459	1.875	47.6
2.000	50.8										AS20-2000	490	2,180	AS22-2000	585	2,600	AS25-2000	372	1,655	2.000	50.8
2.250	57.2										AS20-2250	620	2,758	AS22-2250	728	3,240	AS25-2250	470	2,091	2.250	57.2
2.500	63.5										AS20-2500	752	3,345	AS22-2500	919	4,090	AS25-2500	576	2,562	2.500	63.5
2.750	69.9										AS20-2750	904	4,021	AS22-2750	1,099	4,890	AS25-2750	694	3,087	2.750	69.9
3.000	76.2										AS20-3000	1,076	4,786	AS22-3000	1,297	5,770	AS25-3000	824	3,665	3.000	76.2
3.150	80.0													AS22-3150	1,437	6,390				3.150	80.0
3.500	88.9										AS20-3500	1,448	6,441	AS22-3500	1,778	7,910	AS25-3500	1,110	4,938	3.500	88.9
3.740	95.0													AS22-3740	2,048	9,110				3.740	95.0
4.000	101.6										AS20-4000	1,850	8,229	AS22-4000	2,271	10,100	AS25-4000	1,460	6,494	4.000	101.6

Figure 3-5: Cable Properties (Source: https://www.tripyramid.com/downloads/TriPyramid_catalog.pdf)

3.4 **Models Proposals**

Initially, two different bridge models are built and carried on, which correspond to the two different design of the bridge itself presented in section: the dynamic behavior of the two bridge models is observed and analyzed in order to make a more informed decision about the proposed final model. The investigation of the dynamic behavior of these proposals leads to several versions of the same model, each of which presents an improvement with respect to the previous one.

1. BIG SPAN – VERSION 1

As previously mentioned, this option consists in having two big central spans of 1360 ft each and two smaller lateral spans of 680 ft. The bridge is completely symmetric in terms of spans and the three towers have approximately the same dimensions (Figure 3-6). For the towers, an H global shape is chosen in Figure 3-7.

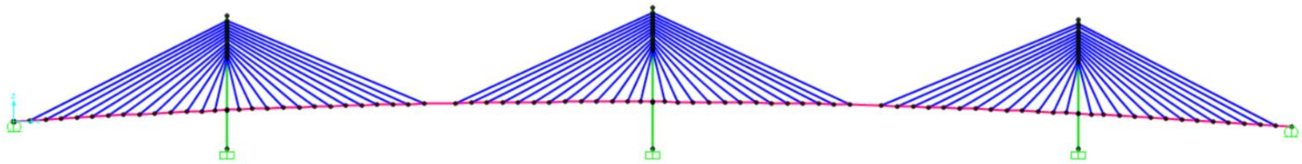

Figure 3-6 Big Span Model - Version 1, Front View (Figure made by the author, SAP2000)

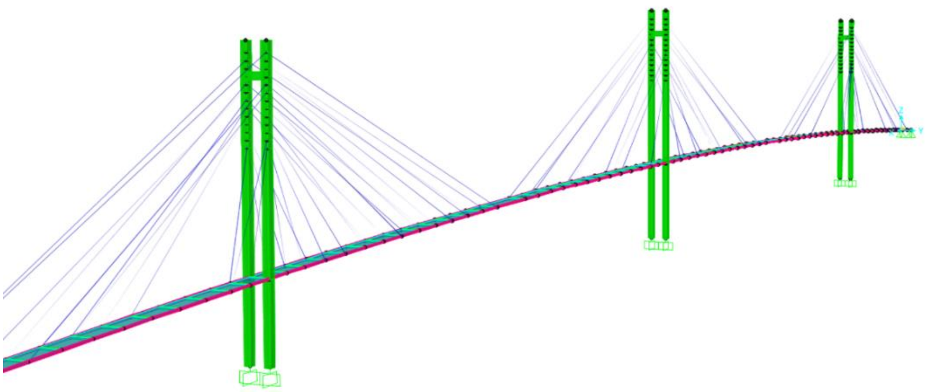

Figure 3-7 Big Span Model - Version 1, H Towers (Figure made by the author, SAP2000)

The deck is 24 ft wide and consists of a lightweight concrete slab supported by a steel grid made of longitudinal and transversal girders (Figure 3-8).

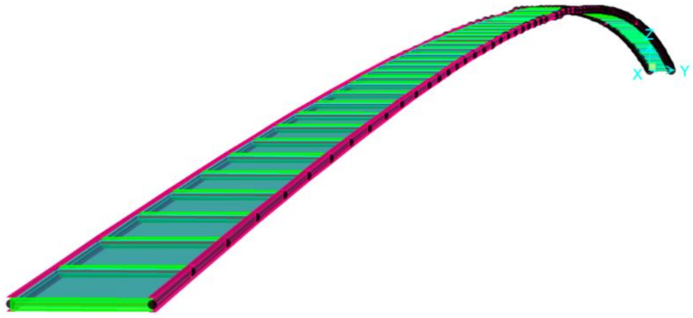

Figure 3-8 Big Span Model - Version 1, Deck (Figure made by the author, SAP2000)

2. SMALL SPAN – VERSION 1

This second option consists in having the movable span smaller with respect to the lateral fixed ones. This option presents the same general design features. Even if the heights of the three towers, in this case, is not the same, the global shape is always an H section (Figure 3-10). The deck is the same as the previous model: 24 ft wide and consists of a lightweight concrete slab supported by a steel grid made of longitudinal and transversal girders (Figure 3-8).

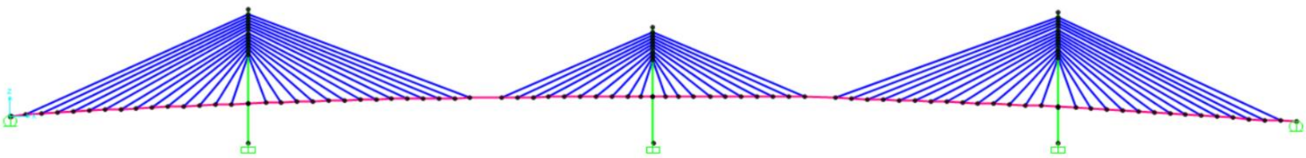

Figure 3-9 Small Span Model - Version 1,Front View (Figure made by the author, SAP2000)

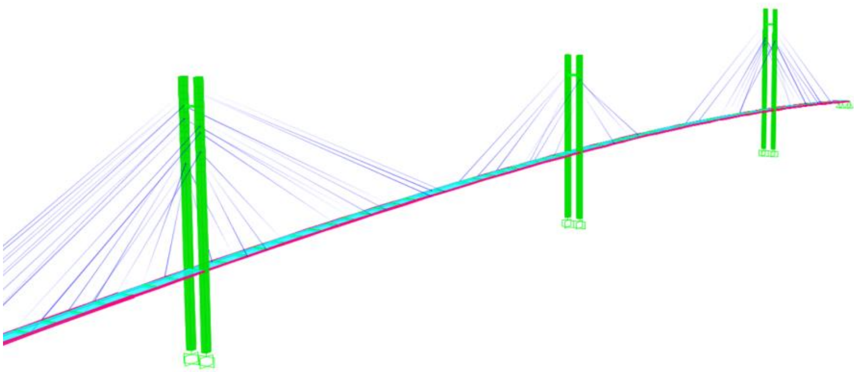

Figure 3-10 Small Span Model - Version 1, H Towers (Figure made by the author, SAP2000)

The existence of this model is driven by the necessity of having the middle movable span as light as possible in order to reduce the dimension of the lifting system, optimizing it.

4 Comparison between Models

In the following chapter, the modal analysis carried on with the program SAP and the related results are presented. The investigation of the dynamic behavior of the starting models leads to the subsequent optimized model, up to the final chosen configuration for the bridge proposed in this project.

4.1 SAP Analysis

1. VERSION 1

		Period	UX	UY	UZ	RX	RY	RZ
Mode	1	6,58	0%	29%	0%	25%	0%	0%
Mode	**2**	**6,51**	**25%**	**0%**	**0%**	**0%**	**3%**	**0%**
Mode	**3**	**6,24**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**
Mode	4	6,14	1%	0%	0%	0%	1%	0%
Mode	5	5,93	0%	6%	0%	5%	0%	36%
Mode	6	5,80	0%	0%	0%	0%	0%	0%
Mode	7	5,68	0%	25%	0%	16%	0%	0%

Table 4-1 Big Span Model-Version 1, Modal Analysis (Table made by the author, Sap 2000)

Looking at the modal analysis of this model, presented in Table 4-1, it can be noticed how the first vertical mode (green) and the first torsional mode (blue) are extremely close within each other. In the design process of bridges like this one, one of the main concerns regards the effect of the wind load on the structure, particularly the flutter phenomenon. In order to avoid this behavior, it is important to keep these two modes apart considering a ratio between the corresponding periods of at least 1.6. In this case, the ratio is way smaller, equal to 1.04 which is not sufficient to guarantee a safe condition with respect to the flutter phenomenon.

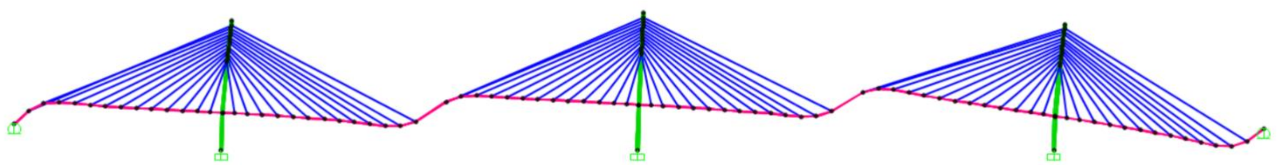

Figure 4-1 Big Span Model - Version 1, First Vertical Mode (Figure made by the author, SAP2000)

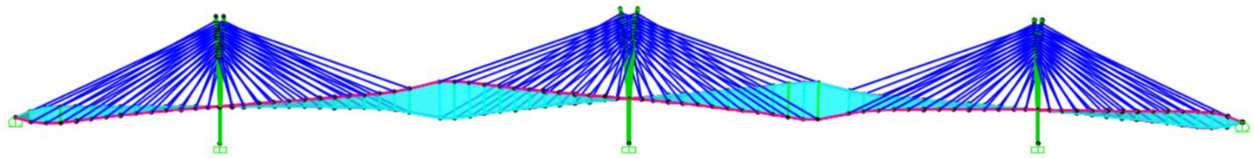

Figure 4-2 Big Span Model - Version 1, First Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{6.51}{6.24} = 1.04$$

The same analysis is carried on for the model with the small middle span. In this case, the first vertical period (green) (Figure 4-3) is higher with respect to the previous model and this is due to the higher flexibility of the model with the smaller middle span. The torsional first mode is actually split in two local torsional modes (blue) (Figure 4-4 and Figure 4-5) related to the side spans because in this case, the dynamic of the bigger side spans starts to become important and independent. Also, for this model, the ratio between the first vertical mode and the first two torsional modes is too small, equal to 1.27, which not guarantee also in this situation the safe conditions with respect to the flutter phenomenon.

		Period	UX	UY	UZ	RX	RY	RZ
Mode	**1**	**7,68**	**17%**	**0%**	**0%**	**0%**	**3%**	**0%**
Mode	2	6,56	0%	0%	0%	0%	0%	0%
Mode	**3**	**6,04**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**
Mode	**4**	**6,00**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**
Mode	5	5,57	0%	17%	0%	19%	0%	35%
Mode	6	5,32	0%	31%	0%	29%	0%	15%
Mode	7	5,03	0%	1%	0%	1%	0%	7%

Table 4-2: Small Span Model-Version 1, Modal Analysis (Table made by the author, SAP2000)

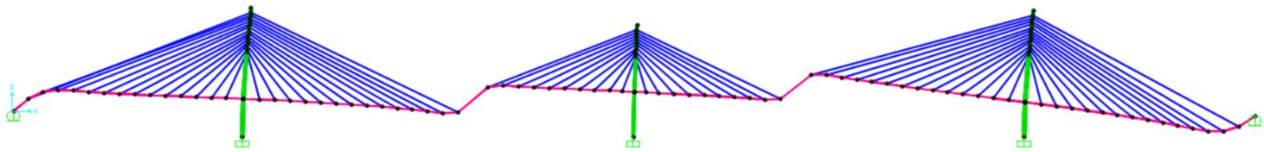

Figure 4-3 Small Span Model - Version 1, First Vertical Mode (Figure made by the author, SAP2000)

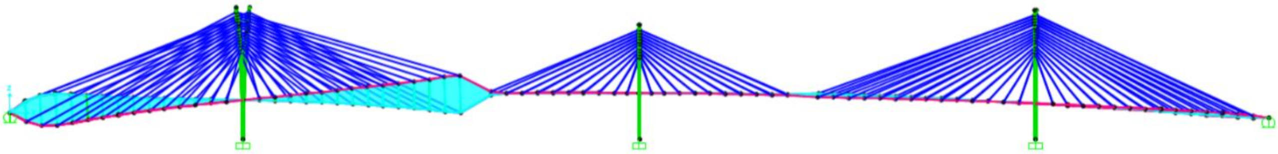

Figure 4-4 Small Span Model - Version 1, First Torsional Mode (Figure made by the author, SAP2000)

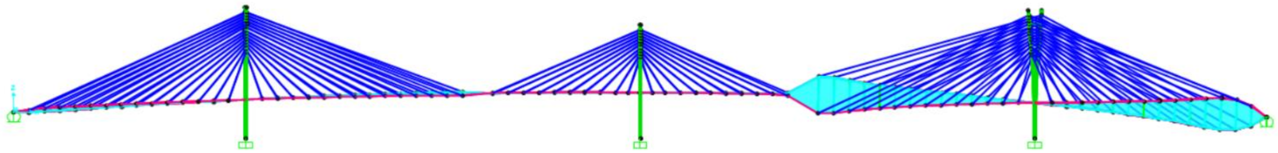

Figure 4-5 Small Span Model - Version 1, Second Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{7.68}{6.04} = 1.27$$

2. VERSION 2

Since the dynamic behavior of the previous two models is not satisfying, some modifications are introduced in these two models. The object is to make more torsional stiff the model in order to keep apart the first vertical and the first torsional mode. In order to achieve this aim, the deck is slightly modified, introducing outriggers. The side longitudinal girders are located outside the concrete deck slab itself and the transversal connection is given by the transversal smaller girders.

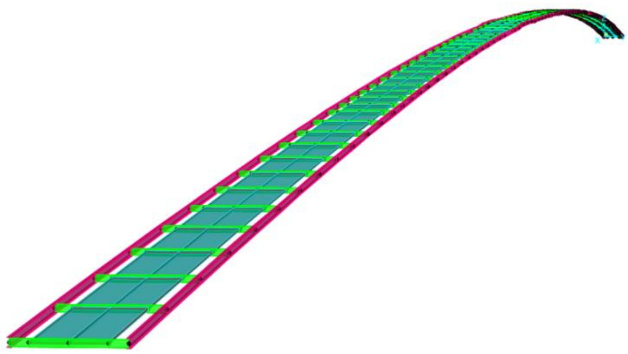

Figure 4-6 Outrigger Deck (Figure made by the author, SAP2000)

This modification makes the deck globally wider (from 24 ft to 36 ft) and consequently stiffer. Moreover, the openings between the concrete slab deck and the side girders offer a way to reduce the dynamic effect of the wind on the deck itself, smoothing the flattering effect. The stiffening effect of this modification can be easily read in the modal analysis of this model. It is evident how the first torsional mode (blue) (Figure 4-8) moves away from the first torsional mode (Figure 4-7) offering a ration between these two modes of 4.33 which represents an extremely, not optimized, safe condition, regarding the flutter phenomenon.

		Period	UX	UY	UZ	RX	RY	RZ
Mode	1	7,74	0%	0%	0%	0%	0%	4%
Mode	**2**	**6,23**	**27%**	**0%**	**0%**	**0%**	**2%**	**0%**
Mode	3	6,17	0%	15%	0%	1%	0%	0%
Mode	4	6,08	4%	0%	0%	0%	1%	0%
Mode	5	5,68	0%	0%	0%	0%	0%	0%
Mode	6	3,92	0%	19%	0%	22%	0%	0%
Mode	7	3,49	0%	23%	0%	15%	0%	35%
Mode	8	3,31	0%	23%	0%	13%	0%	33%
Mode	9	2,98	0%	0%	0%	0%	0%	2%
Mode	10	2,73	0%	2%	0%	8%	0%	0%
Mode	18	1,55	0%	0%	2%	0%	0%	0%
Mode	**19**	**1,44**	**0%**	**0%**	**0%**	**0%**	**0%**	**1%**

Table 4-3: Big Span Model - Version 2, Modal Analysis (Table made by the author, SAP 2000)

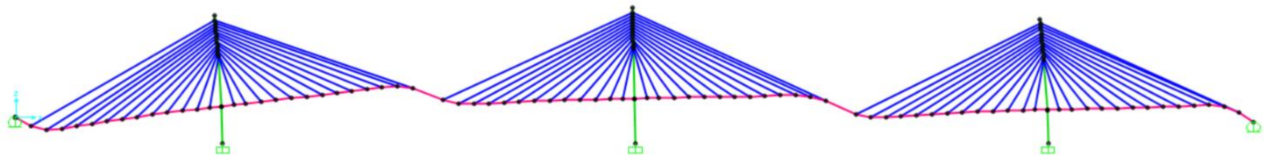

Figure 4-7 Big Span Model - Version 2, First Vertical Mode (Figure made by the author, SAP2000)

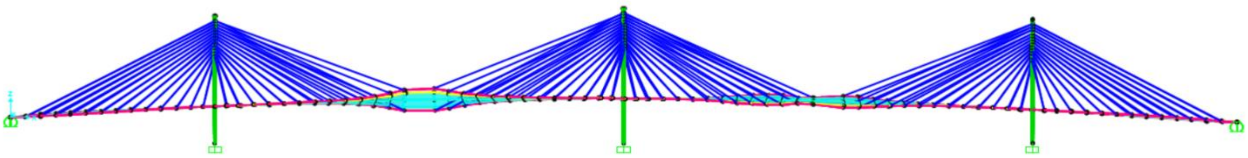

Figure 4-8 Big Span Model - Version 2, First Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{6.23}{1.44} = 4.33$$

The same shift happens also for the model with the smaller middle span even if the effect is less marked. The first torsional mode (Figure 4-10) moves away from the first vertical mode (Figure 4-9) as well but not of a comparable shift.

		Period	UX	UY	UZ	RX	RY	RZ
Mode	**1**	**7,26**	**19%**	**0%**	**0%**	**0%**	**3%**	**0%**
Mode	2	6,89	0%	0%	0%	0%	0%	5%
Mode	3	6,39	0%	0%	0%	0%	0%	0%
Mode	4	5,65	0%	41%	0%	13%	0%	5%
Mode	5	5,43	0%	0%	0%	12%	0%	27%
Mode	6	5,22	0%	14%	0%	26%	0%	22%
Mode	7	4,47	5%	0%	0%	0%	0%	0%
Mode	**8**	**4,36**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**
Mode	9	4,36	0%	0%	0%	0%	0%	0%
Mode	10	4,10	0%	15%	0%	12%	0%	0%
Mode	11	3,13	0%	1%	0%	2%	0%	9%
Mode	12	3,01	0%	5%	0%	9%	0%	1%

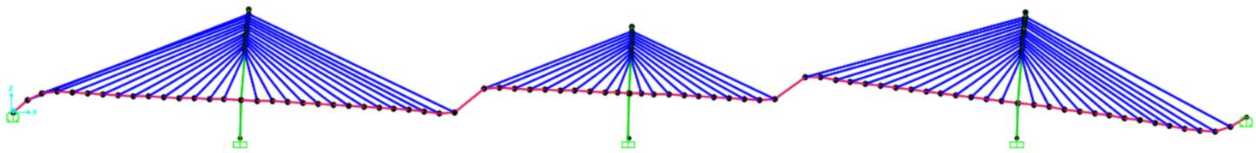

Figure 4-9 Small Span Model - Version 2, First Vertical Mode (Figure made by the author, SAP2000)

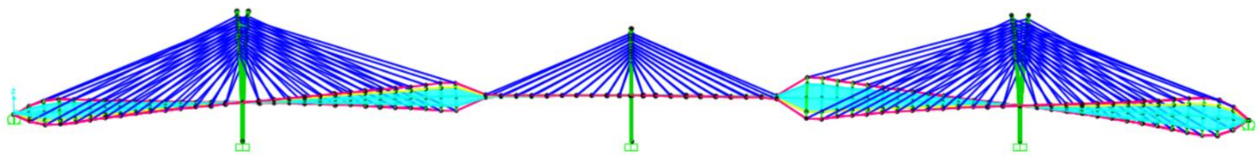

Figure 4-10 Small Span Model - Version 2, First Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{7.26}{4.36} = 1.66$$

The ratio between the two modes, in this case, is equal to 1.66 which is close to the optimal solution.

3. VERSION 3

The third version of these two models is presented. In this case, an important change of the global section of the tower is introduced, which from an H section changes to an A section.

This type of towers gives more stiffness to the model and this decision is made because the models presented until now are extremely flexible.

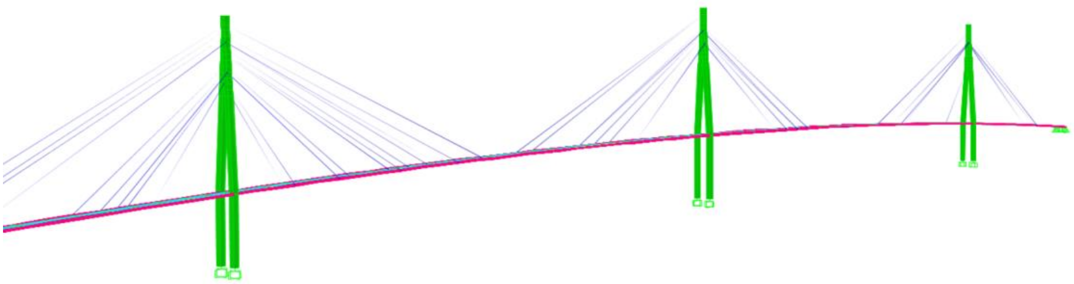

Figure 4-11 Big Span Model - Version 3, A Towers (Figure made by the author, SAP2000)

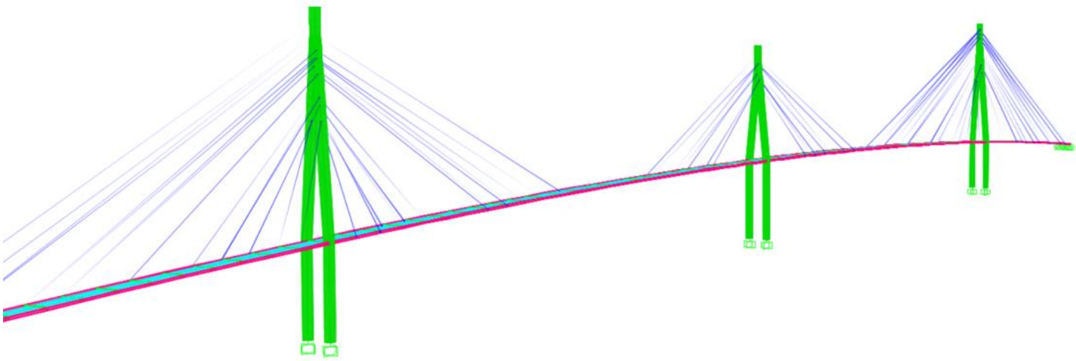

Figure 4-12 Small Span Model - Version 3, A Towers (Figure made by the author, SAP2000)

The stiffening effect of the A section tower is clear from the reduction of the period for the first vertical mode (Table 4-5) both for the model with the big middle span and the model with the smaller middle span.

		Period	UX	UY	UZ	RX	RY	RZ
Mode	1	7,62	0%	0%	0%	0%	0%	4%
Mode	2	6,09	0%	13%	0%	1%	0%	0%
Mode	**3**	**5,93**	**17%**	**0%**	**0%**	**0%**	**2%**	**0%**
Mode	4	5,82	8%	0%	0%	0%	2%	0%
Mode	5	5,43	0%	0%	0%	0%	0%	0%
Mode	6	2,96	0%	0%	0%	0%	0%	1%
Mode	18	1,54	0%	0%	2%	0%	0%	0%
Mode	**19**	**1,42**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**

Table 4-4: Modal Analysis A towers

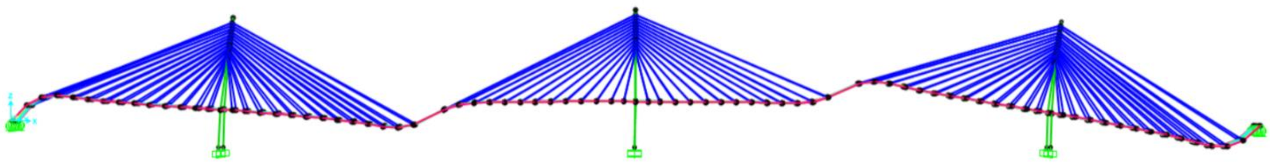

Figure 4-13 Big Span Model - Version 3, First Vertical Mode (Figure made by the author, SAP2000)

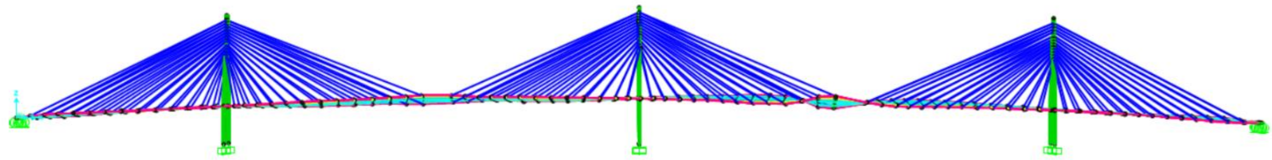

Figure 4-14 Big Span Model - Version 3, First Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{6.23}{1.44} = 4.17$$

For the model with the big span the reduction of the period of the first vertical mode lead to an improvement in the ratio between the periods associated with the first vertical and the first torsional mode, which however remain pretty high. The same happens for the model with the small span where the ratio remains pretty high as well.

Table 4-5 Small Span Model - Version 3 (Modal Analysis)

		Period	UX	UY	UZ	RX	RY	RZ
Mode	1	8.70	0%	1%	0%	2%	0%	4%
Mode	**2**	**6.63**	**0%**	**0%**	**0%**	**0%**	**0%**	**7%**
Mode	3	6.11	8%	0%	0%	0%	2%	0%
Mode	4	5.61	2%	0%	0%	0%	1%	0%
Mode	5	5.44	0%	15%	0%	2%	0%	3%
Mode	6	4.01	8%	0%	0%	0%	1%	0%
Mode	22	1.34	0%	0%	0%	0%	0%	0%
Mode	**23**	**1.31**	**0%**	**0%**	**0%**	**0%**	**0%**	**0%**

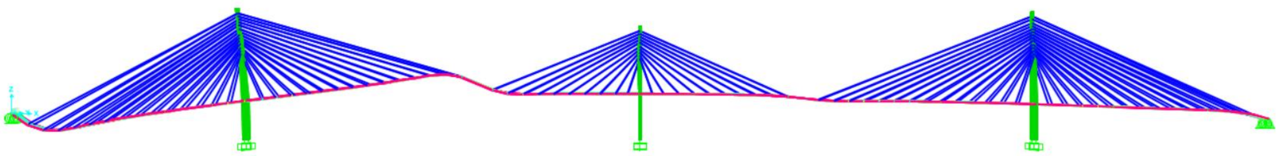

Figure 4-15 Small Span Model - Version 3, First Vertical Mode (Figure made by the author, SAP2000)

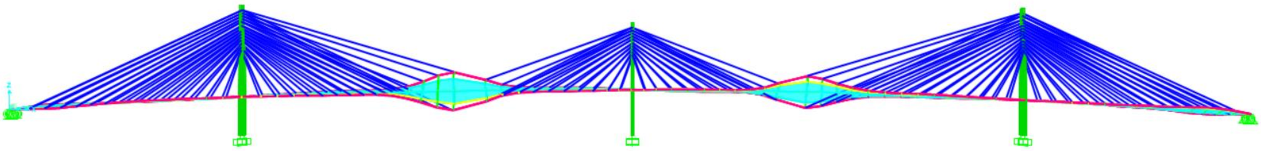

Figure 4-16 Small Span Model - Version 3, First Torsional Mode (Figure made by the author, SAP2000)

$$\frac{V}{T} = \frac{6.63}{1.31} = 5.06$$

4.2 Benefits of the Final Solution

Instead of leaning towards one of the two options presented in the previous section, the final proposed model is optimized more. Among the model with the big and the small span, the choice is taken considering the movability of the middle span. Then in order to make easier the lifting operation of that span the final model starts from the option with the smaller middle span. Moreover, to make lighter the middle span, the section of the deck for that specific span is reduced from 1 ft height to 0.5 ft.

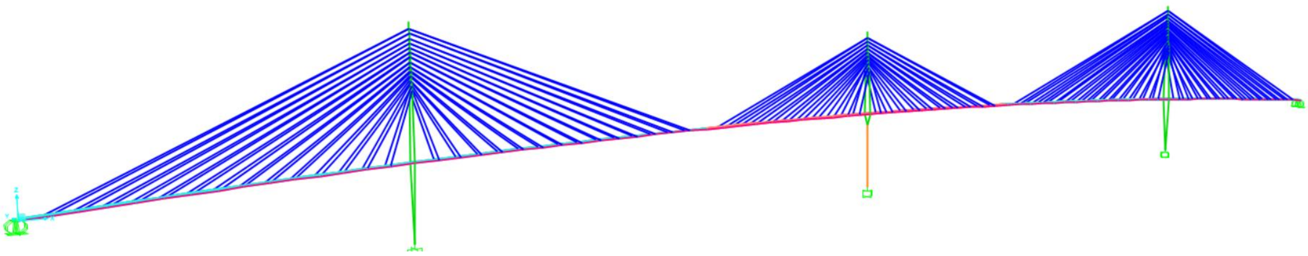

Figure 4-17: Final Model (Figure made by the author, SAP2000)

Another important issue is taken into account and solved in this model. One of the main constraints of this project is given by the extremely busy navigation affecting the Hudson River which makes this bridge very sensitive to vessel impact. In order to reduce the possibility of vessel impact to happen, it is important to reduce the obstruction in the water due to the bridge piers. Because of that, the central pier associated with the central span is modified, adopting a single leg pier with a circular section. Also, the side spans present converging section at the base of the pier reducing the obstructing room in the water and offering the possibility of reducing the dimension of the foundations.

		Period	UX	UY	UZ	RX	RY	RZ
Mode	1	8.90	0%	26%	0%	10%	0%	0%
Mode	**2**	**7.56**	**27%**	**0%**	**0%**	**0%**	**3%**	**0%**
Mode	3	6.59	0%	0%	0%	0%	0%	7%
Mode	4	6.03	0%	0%	0%	0%	0%	0%
Mode	5	5.46	0%	23%	0%	5%	0%	2%
Mode	6	5.06	0%	7%	0%	12%	0%	43%
Mode	7	4.82	0%	26%	0%	26%	0%	23%
Mode	8	4.31	13%	0%	0%	0%	0%	0%
Mode	9	2.84	0%	0%	0%	0%	0%	7%
Mode	10	2.76	0%	4%	0%	10%	0%	0%
Mode	11	2.50	0%	0%	7%	0%	0%	0%
Mode	12	2.44	10%	0%	0%	0%	9%	0%
Mode	13	2.24	32%	0%	0%	0%	0%	0%
Mode	14	2.15	0%	0%	1%	0%	0%	0%
Mode	15	2.02	5%	0%	0%	0%	2%	0%
Mode	**16**	**2.00**	**0%**	**0%**	**0%**	**0%**	**0%**	**1%**

Table 4-6: Modal Analysis final solution

The modal analysis of this final proposed model is also satisfying the requirement regarding the ratio between the first vertical and torsional mode. The ratio is smaller than the ones obtained for the third version of the model presented in the previous section, showing that the modification introduced lead to an optimized model which present a safe configuration with respect to the flutter phenomenon.

$$\frac{V}{T} = \frac{7.56}{2.00} = 3.78$$

5 Final Model

A set of different static and dynamic analysis is carried on the final adopted bridge model, in order to observe and analyze the behavior of the proposed solution to the different type of external excitations: cable loss, pedestrian dynamic load, seismic load. Moreover, before investigating the bridge dynamic response to these load conditions, the cables' pre-stressing process is simulated.
Mathcad and DynamAssist are used to derive the time-history functions for the seismic analyses and the resulting graphs are implemented into SAP.

5.1 Prestressing

The goal of the prestressing procedure in SAP is to neutralize the dead load deflections of the structure. By applying negative temperature loads to each cable, we are applying a compressive thermal strain that effectively shortens the cables. We can use these temperature loads as a way of determining the appropriate cable length for each cable so that the bridge sags into a visually desirable position under its self-weight. It must be done this way simply because we draw the bridge in SAP the way we intend for it to look. It is not possible for us to know what length each cable should be before we know the deflected shape of the whole bridge.

There are a few factors that make this process difficult to implement. One of the main challenges of doing this is that on fan-style cable-stayed bridges, each cable is under a different amount of load. As such, each cable needs a different amount of "prestressing" in order to come back to zero deflection. Also, the outer cables are longer, so they will have a larger change in length than the inner cables. Another challenge comes from the fact that cable-stayed bridges are highly coupled systems. Prestressing one cable affects the amount of prestressing that the cables near it require. As such, we used a recursive method of determining prestressing temperature loads. The end goal is to bring the bridge from its deflected shape, shown in Figure 5-1, back to its original shape. The peak dead load deflection before prestressing is 7.5 feet.

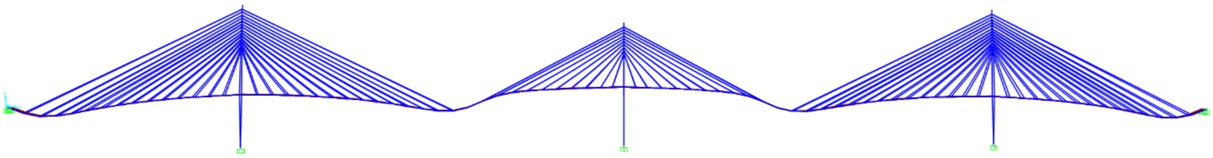

Figure 5-1: Dead Load Deflections, without prestressing (Figure made by the author, SAP2000)

Given then the geometry of our bridge, the deflected shape behaves differently in different regions along the length of the deck. Since the center tower is the smallest of the three, and the bridge is roughly symmetric about the center tower, we can treat the two half-spans of the center tower symmetrically, as shown in Figure 5-2.

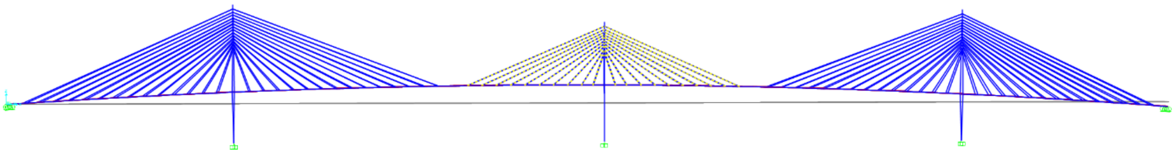

Figure 5-2: Center Tower Cable Group (Figure made by the author, SAP2000)

For the outer towers, one half-span tie into either side of the center tower, and the other half-span ties into the support conditions on either end of the whole bridge. As such, each of these half-spans behaves differently. We treat the inner half-span and the outer half-span of each outer tower separately, but symmetrically, as pictured in Figure 5-3 and Figure 5-4.

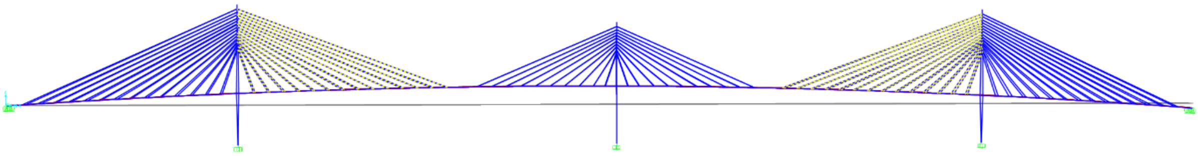

Figure 5-3: Inner Half-spans of Outer Towers Cable Group (Figure made by the author, SAP2000)

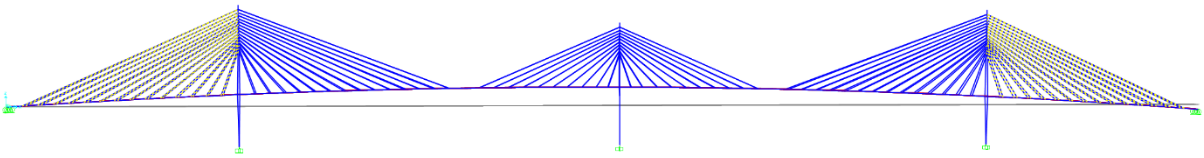

Figure 5-4: Outer Half-spans of Outer Towers Cable Group (Figure made by the author, SAP2000)

To implement the temperature loads, we use different functions for each of the groups pictured above. We assign the temperature loads to each cable using spreadsheets. First, we define a trial function for each grouping, then we apply the trial functions (temperature vs cable) to each group symmetrically. We did this over and over again, changing coefficients and functions until we were able to match to the deflected shape of the prestressing loads to the inverse of the deflected shape of the dead loads. Since this is being treated linear-elastically, SAP simply superimposes the dead and prestressing load patterns to produce the final shape of the bridge. The final functions used for each cable group are shown in Figure 5-5, Figure 5-6, and Figure 5-7. The cable temperature is shown as a function of the cable number (from tower outwards), for one half-span.

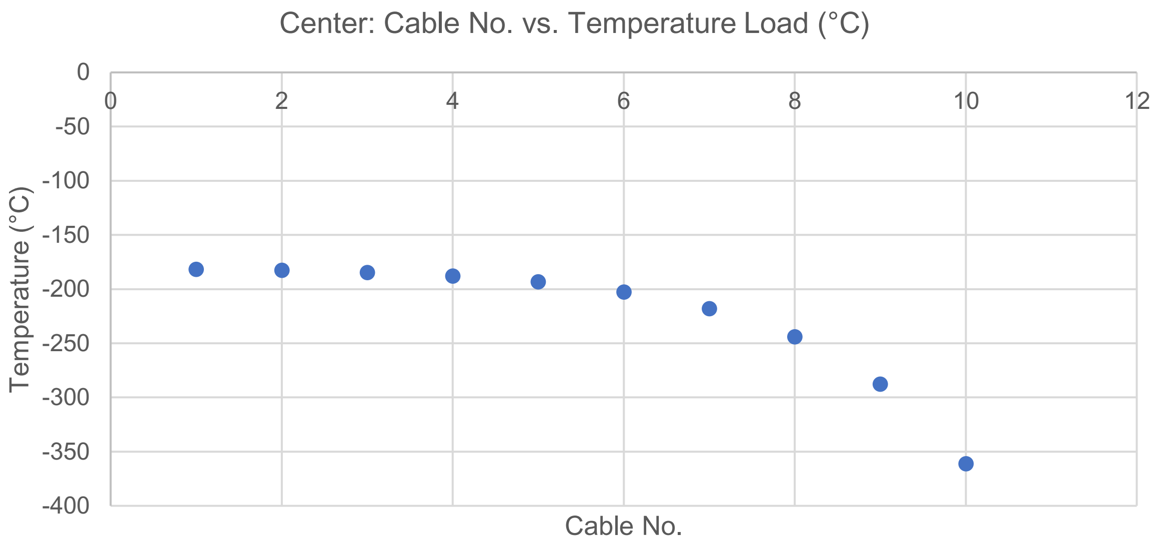

Figure 5-5: Temperature Loads for Center Tower (Figure made by the author, SAP2000)

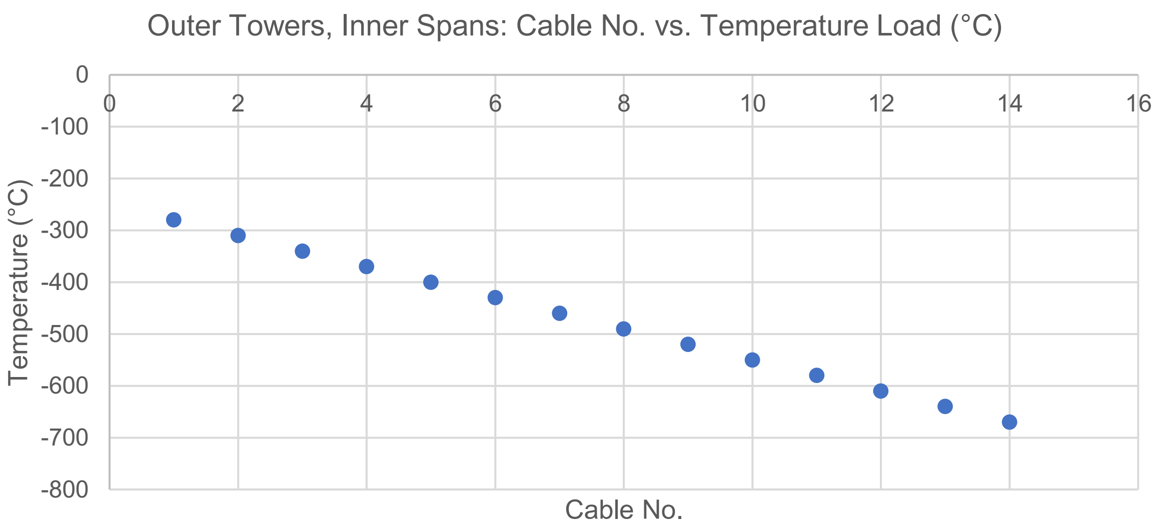

Figure 5-6: Temperature Loads for Inner Spans of Outer Towers (Figure made by the author, SAP2000)

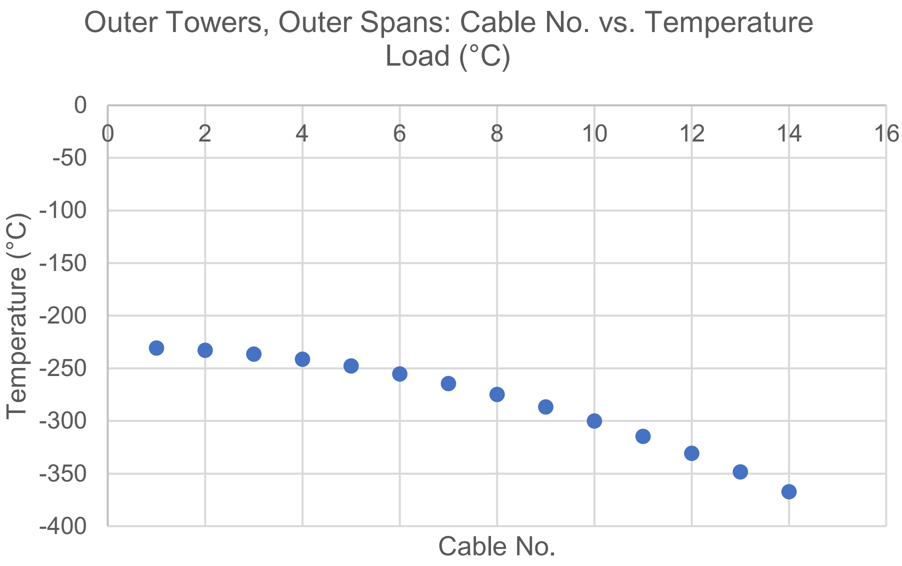

Figure 5-7: Temperature Loads for Outer Spans of Outer Towers (Figure made by the author, SAP2000)

The center tower has the shortest span, so the deflection curve is relatively flat in the center and drops off towards the ends. As such, we found that an exponential function fit this tower best. As for the group containing the inner spans of the outer towers, a linear function fit the best. The outer spans of the outer towers worked well with parabolic functions. This excel spreadsheet will be submitted in the submission package. The final results of the prestressing are pictured in Figure 5-8, with a final maximum dead load deflection of eight inches, down from almost eight feet.

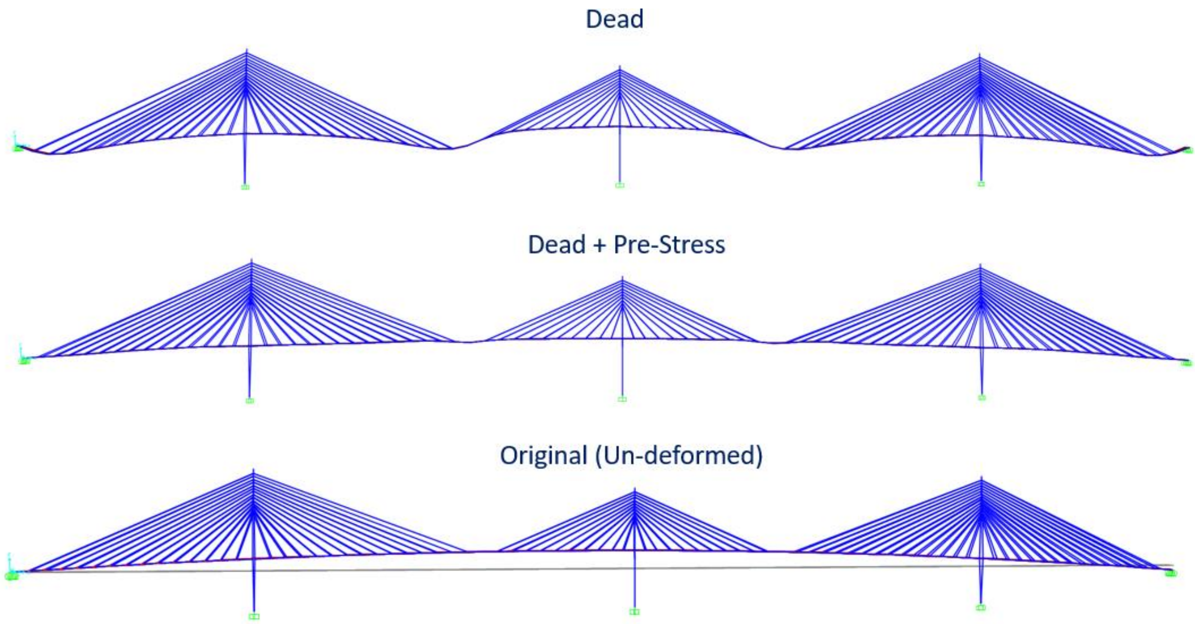

Figure 5-8: Results of Prestressing (Figure made by the author, SAP2000)

5.2 **Seismic Analysis**

The acceleration time history which represents the seismic loading design condition for this bridge in its NYC location is derived directly from DynamAssist. In this program is possible to define the location (in terms of latitude and longitude) of the structure itself and the soil conditions which characterize the site. The program, using this information, is able to derive the seismicity of the site and gives back all the principal information and the design response spectrum. The analysis is carried on considering the seismic load applied the final presented model considering both the big and small middle span. This choice is made to double check the dynamic behavior of the two models and to support the decisions made on the final design of the bridge itself.

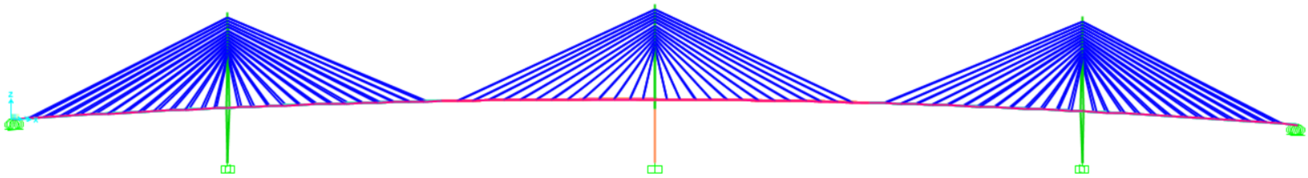

Figure 5-9 Final Big Span Model, Front View (Figure made by the author, SAP2000)

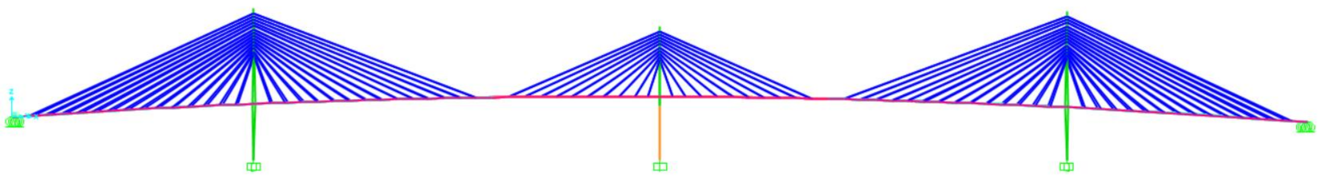

Figure 5-10 Final Small Span Model, Front View (Figure made by the author, SAP2000)

The response spectrum for this specific case presents both a PGA and a maximum acceleration pretty low and this is consistent with the seismicity condition of New York City and the excellent geotechnical conditions of the soil which don’t give any amplification to the ground seismic acceleration. Moreover, the first three periods of our models are extremely long and that means positioning them on the spectrum itself that the acceleration which will affect the bridge is even smaller than the PGA.

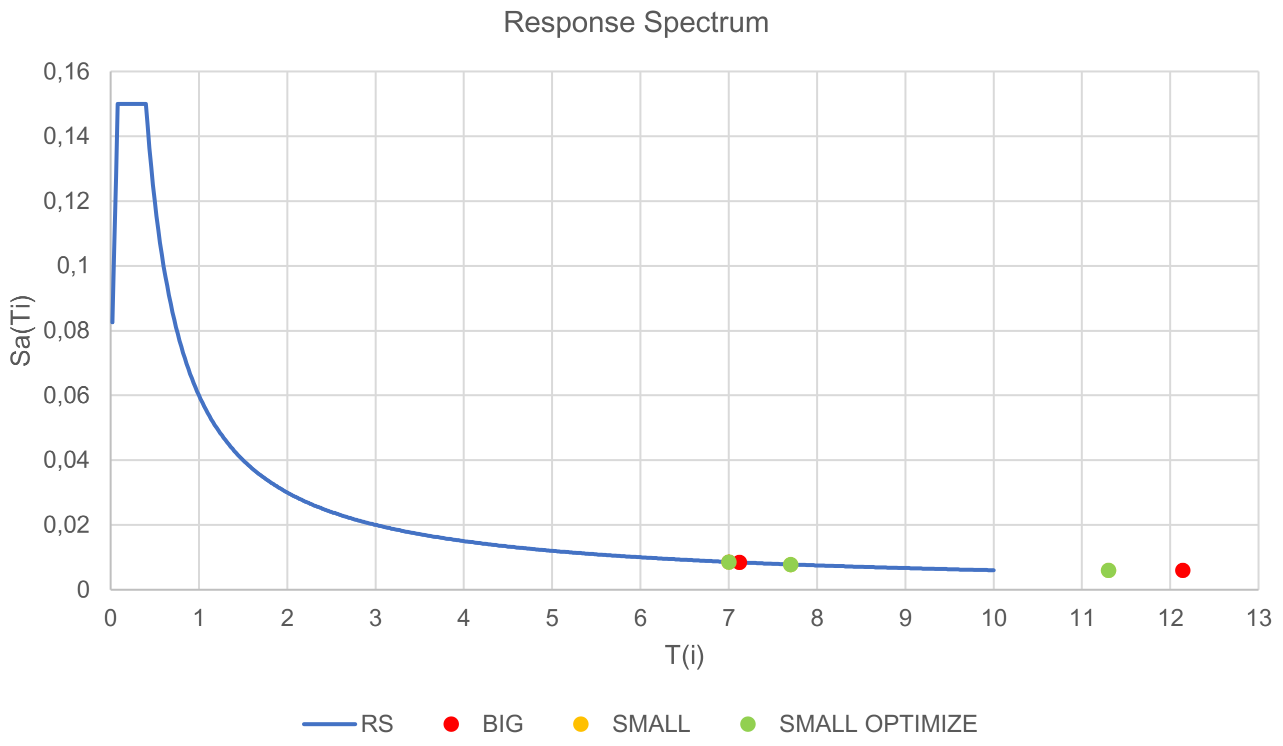

Figure 5-11 Response Spectrum (Figure made by the author, SAP2000)

DynamAssist offers also the possibility of deriving the time history of acceleration artificially generated from the response spectrum. This is later used as the effective seismic load applied to the model. The acceleration is applied equally in the longitudinal and transversal direction of the bridge and it is reduced to 2/3 for the vertical loading condition.

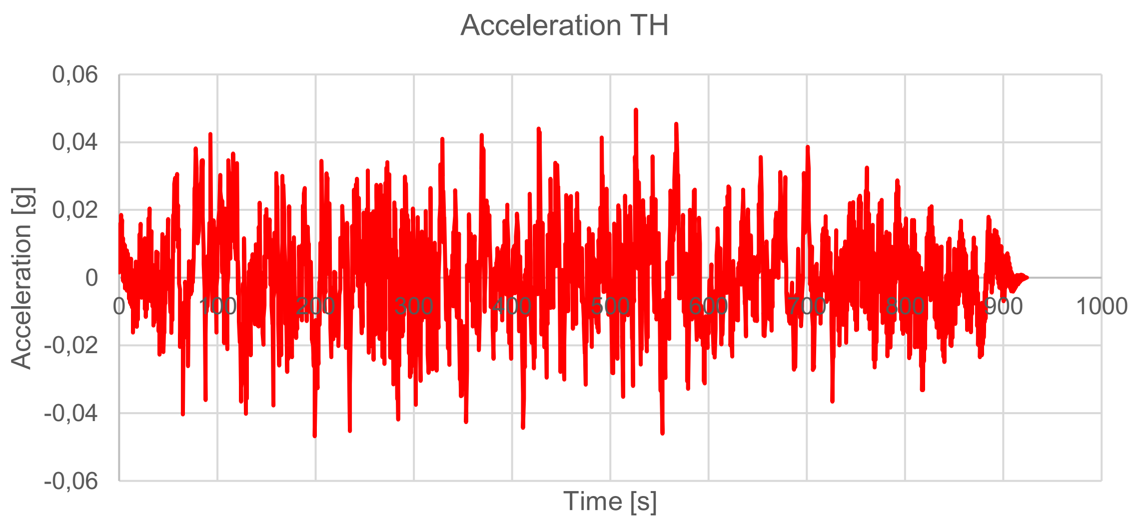

Figure 5-12 Generated Time History (Figure made by the author, DynamAssist)

In Figure 5-13 and Figure 5-14 the moments computed at the base of the three piers are presented. It can be noticed that for the model with the big span the bending moment along the longitudinal axis is smaller than the one computed in the model with the smaller middle span.

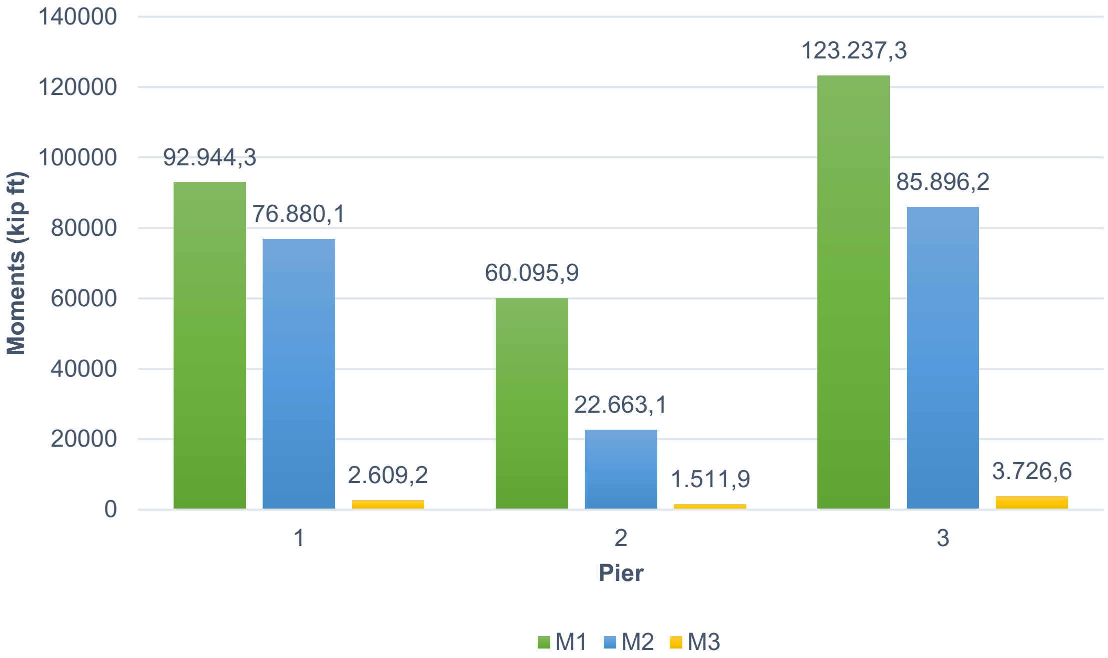

Figure 5-13 Base Reaction Moments- Big Span Model (Figure made by the author, Excel)

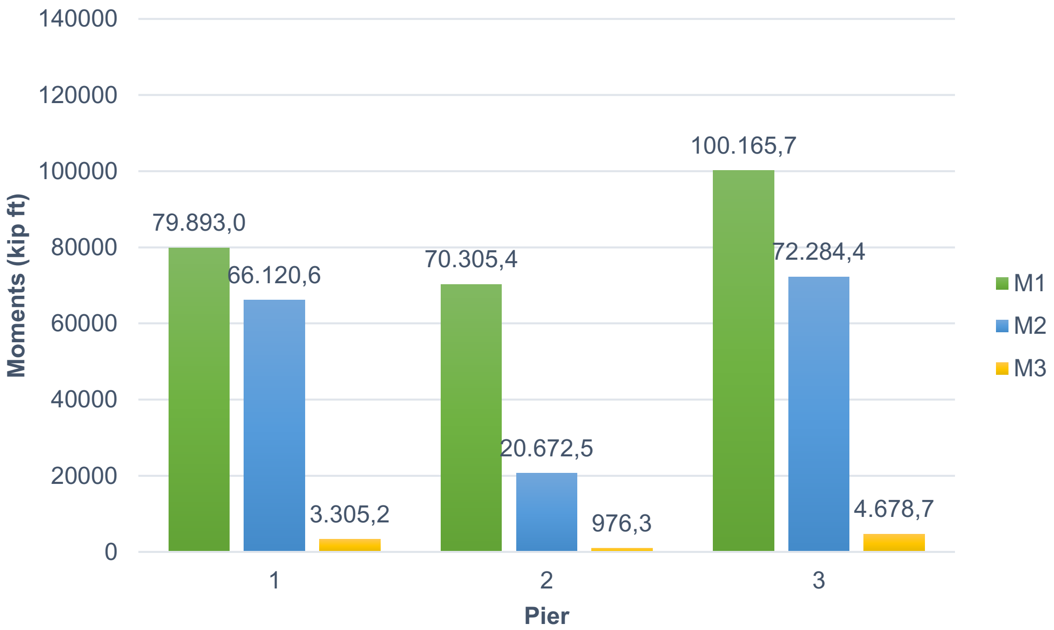

Figure 5-14 Base Reaction Moments- Small Span Model (Figure made by the author, Excel)

The model is optimized once again in order to reduce the bending moment in the model with the smaller middle span since one of the objects of this model is also to keep small the foundations of the piers. To achieve this aim the section dimension of the middle pier are reduced by 30%. This section reduction leads to a decrement of the moments with respect to the longitudinal and transversal axis of the 40% and this allows to reduce significantly the cost of the foundations for the model with the smaller middle span which is the model that collects the higher number of positive characteristics.

Figure 5-15 Base Reaction Moments- Optimized Small Span Model (Figure made by the author, Excel)

5.3 Wind Analysis

This section of the report presents the analysis of the wind climate and wind properties undertaken at the Hoboken Bridge. The results presented in this section are used in subsequent analyses to assess the aerodynamic stability of the bridge and to determine the wind loads for structural design.

5.3.1 Wind Speed

5.3.1.1 Source of Data

The wind statistics used to determine the design wind speeds and directionality at the bridge site were based on the surface wind measurements taken between 1931 and 2018 at the Newark Liberty International Airport, located about 11 miles southwest from the bridge site. Figure 5-16 shows the location of the airport in relation to the proposed bridge.

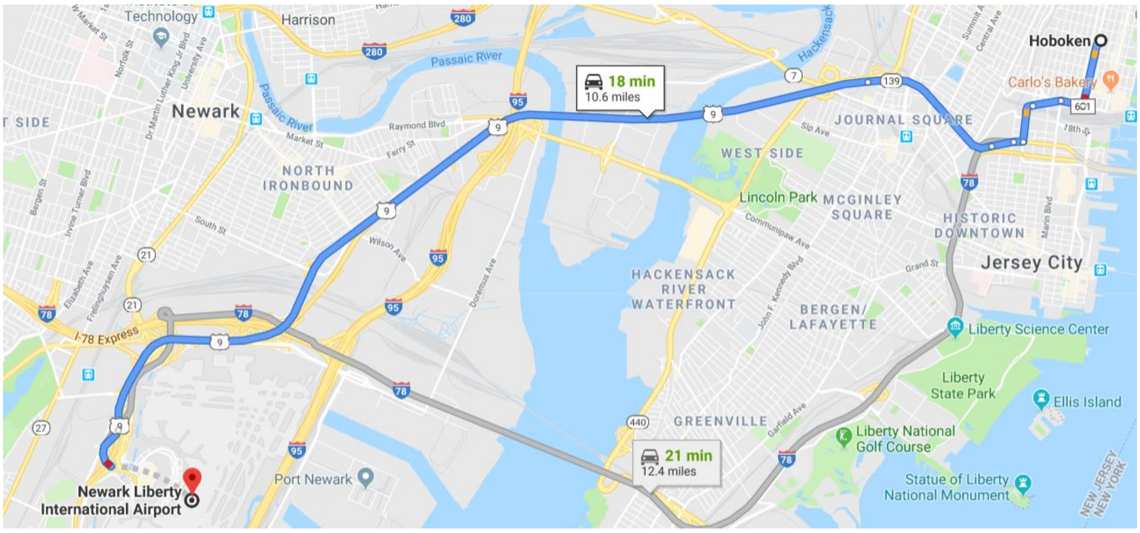

Figure 5-16: Distance between Hoboken and Newark (Source: Google Maps)

5.3.1.2 Wind speed at deck high

The wind analysis follows the directional procedure prescribed in AASHTO LFRD chapter 3.8.

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right)\ln(\frac{Z}{Z_0})$$

- Z = height of a structure at which wind loads are being

calculated as measured from low ground, or from water level, > 30.0 ft

- V_0 = friction velocity, a meteorological wind characteristic is taken
- Z_0 = friction length of upstream fetch, a meteorological wind characteristic is taken

The 100-year mean hourly velocity at a height of 92 ft was calculated to be 71 mph.

5.3.2 Wind Load

Due to the dimensions and the location of the infrastructure a wind tunnel test would be required, as suggested also by the AASHTO, to obtain more accurate data along the deck, the cables, and the towers. A wind tunnel test would have been extremely expensive and not justified for a preliminary design at this stage.

The two commonly used methodologies were both inefficient for our project, the first one from a technical perspective and the second one from a financial one. So, a different strategy was applied, use an existing wind tunnel test on a bridge with similar characteristics and subjected to similar wind speed. Through the support of HNTB Corporation, I was able to use the results of the I-70 Mississippi River Bridge, which geometry is reported in Appendix A.

The wind tunnel test provides us different pressures along the deck, on the towers and on the cables for different load case in three different directions. The following formulas are used to obtain the loads per unit length from the pressures.

1) **Bridge Deck**

Lateral loads Fz=pz*D

Along the deck Fx=px*D

Vertical loads Fy=py*B

Torsional loads Mx=pm B^2

- Deck depth D
- Deck Width B

2) **Tower**

Lateral loads Fz=pz*D

Longitudinal loads Fx=px*B

Vertical loads Fy=py*B

- Tower depth D
- Tower Width B

3) **Cables**

Fz=22*D

- The diameter of the cables D

Once the loads have been evaluated the interpolation is necessary to distribute the forces along with the structural elements because the wind tunnel test gives us punctual results at the intersection of each node. All the calculations are reported in Appendix A. The two pictures below show the pressure distribution both on I-70 Bridge and on Hoboken Bridge calculated through the interpolation process.

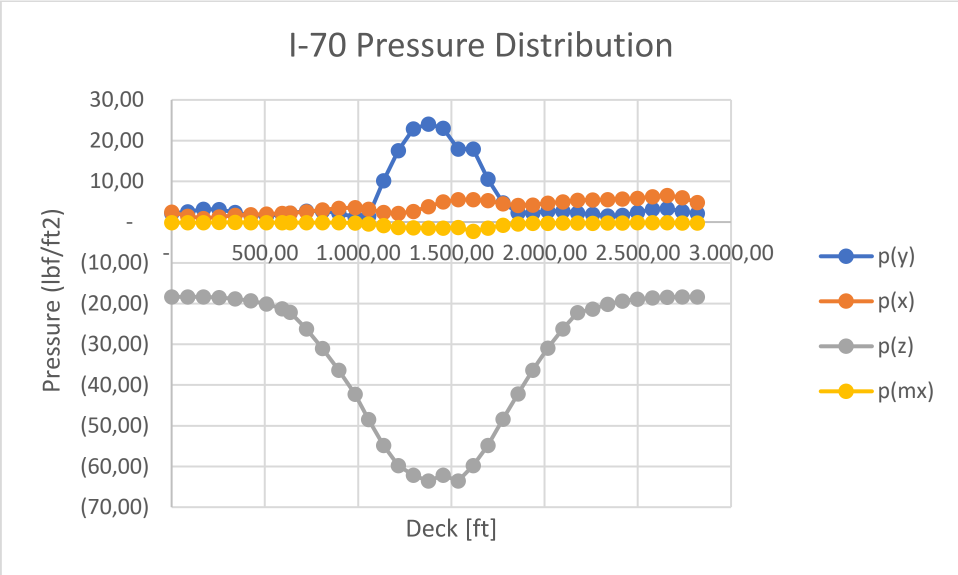

Figure 5-17: Pressure distribution (Figure made by the author, Excel)

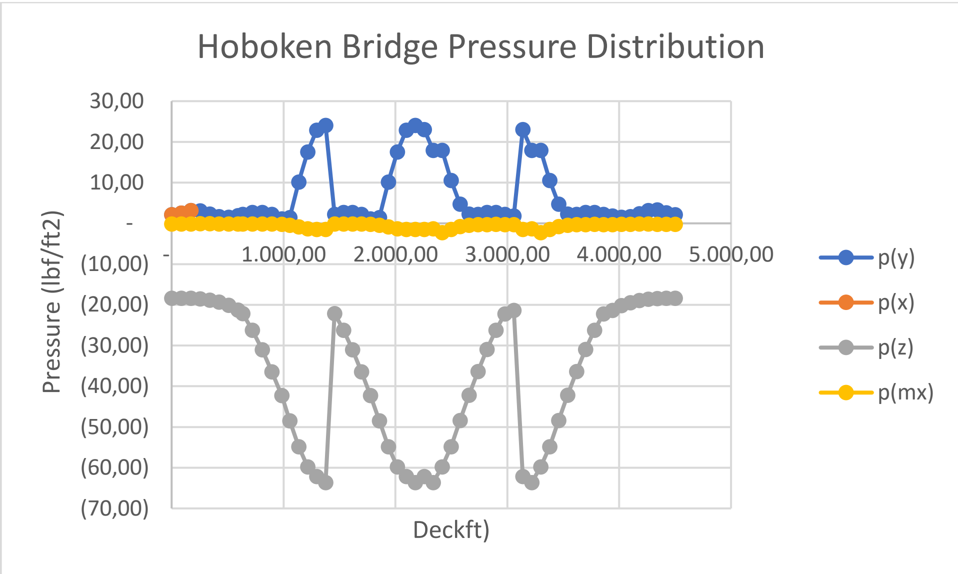

Figure 5-18: Pressure distribution (Figure made by the author, Excel)

As it is possible to notice the wind pressure is particularly high in the middle of the spans and it is almost symmetric due to the geometry of the two bridges and the surrounding area. It is possible to notice the same behavior in Figure 5-20, where a picture of the 3D model is reported. It is also important to underline that these two graphs were made considering the worst load case (load case 1).

In fact, the same procedure reported in Appendix A is performed for 10 different load cases, for everyone the load for unit length is applied to the Sap model, and the worst scenario is taken into account.

5.4 **Load Combination**

Before completing the assessments in the following sections of the thesis, it is important to define the worst strength load combination. The different load combinations are applied to the model, following the AASHTO LRFD Bridge Design Specifications.[8]

										Use One of These at a Time				
Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	*EQ*	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.4 0	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.4 0	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.3 0	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.7 0	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I— *LL, IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue II— *LL, IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Figure 5-19: Load Combinations (Source: AASHTO LFRD, section 3)

The worst-case scenario, as expected in the New York area is the Strength limit state III, 1 Dead load + Prestressed + 1.4 Wind. As reported in the previous chapter, the analysis was done following the results of the I-70 Mississippi River Bridge wind tunnel test. The wind speed at which the test was performed was 69 mph, slightly different from the one in New York, 71 mph. To minimize the uncertainty and obtain the most reliable results we increased the wind factor.

[8] (American Association of Transportation, 2014)

$$WS = 1.4 * (^{71}/_{69})^2 = 1.06$$

This is an empirical formula, used by HNTB Corporation to perform preliminary design on bridges where the wind tunnel test is not present. It has some limitations but has proven to be reliable for a preliminary design study.

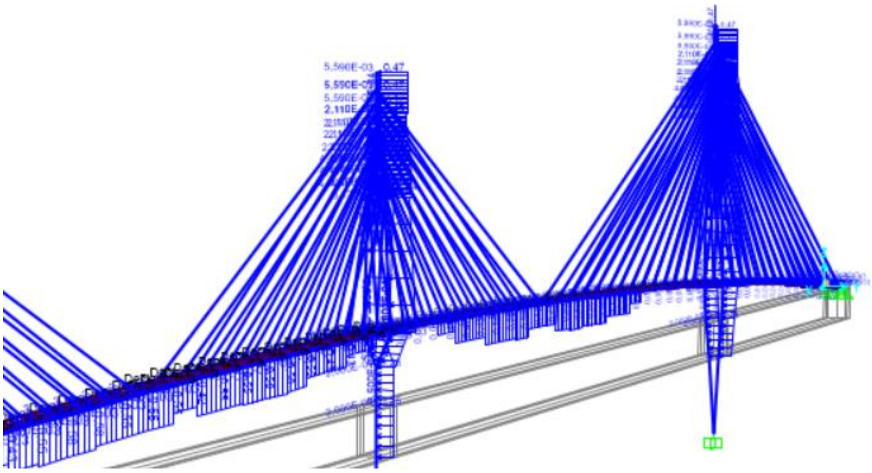

Figure 5-20: Wind Load applied to the model (Figure made by the author, SAP2000)

5.5 **Section Assessment**

Once that the worst load combination is evaluated (Strength limit state III, 1 Dead load + Prestressed + 1.4 Wind) a preliminary section assessment is performed for the cables, the towers and the longitudinal and transversal girders of the deck. Following the procedure performed by HNTB Corporation for the towers and the deck, a moment assessment is done, while for the cables we analyzed the breaking strength.

Starting from the cables, the ultimate criterion that we wanted to be sure to satisfy is that:

Axial Force < 65% ultimate breaking strength

This criterion that we want to satisfy is defined by the AASHTO LFRD and it is the first check performed by HNTB to design the cables of a cable-stayed bridge.

The most stressed cables are the ones close to the middle span, and they have a maximum axial force of 252 kips that is reasonable and can be accepted by the criterion.

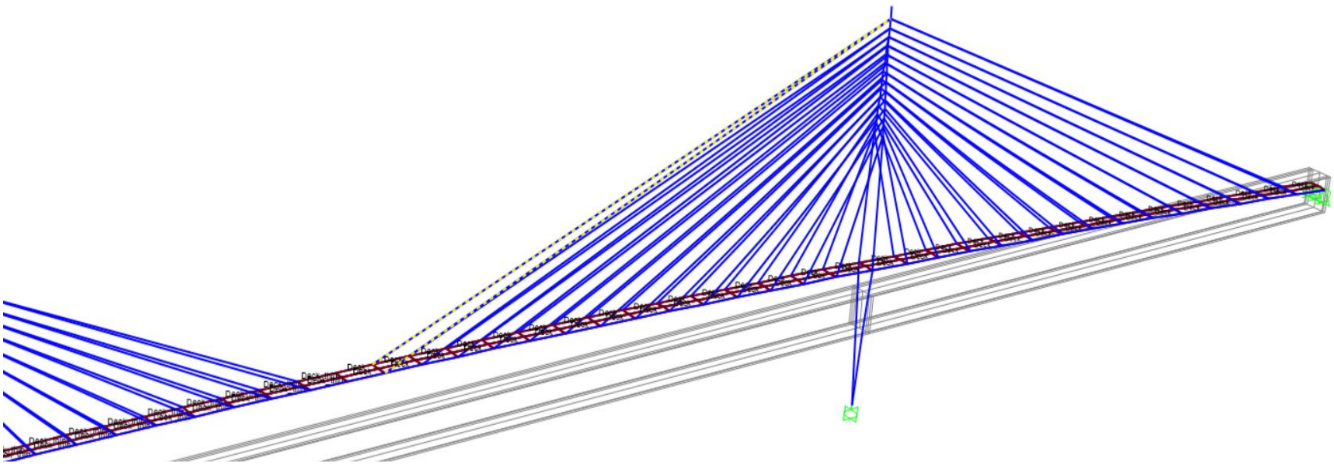

Table 5-1: Maximum loaded cables (Figure made by the author, SAP2000)

Fmax	252	kips
Fbs	510	kips
65% Fbs	331,5	kips

Table 5-2: Cables axial force (table made by the author)

As a preliminary analysis for the other 3 elements, the resisting moments of the sections was compared to the acting moments. Due to an initial huge difference between these two quantities, an optimization process was performed for the towers, longitudinal and transversal girders.

$$f_{yd} = \frac{f_{yk}}{y_s}$$

- f_{yd} = steel calculation strength
- f_{yk} = characteristic resistance to yielding steel
- y_s = steel safety factor, 1.05

$$M_{rd} = W_{pl} * f_{yd}$$

- W_{pl} = Design plastic modulus, directly evaluated through SAP2000
- f_{yd} = Material design tension
- M_{rd} = Resisting moment

First model				
1) Central Tower				
Outer Length	10	ft		
Thickness	0,5	ft		
fy	50,00	ksi		
Ys	1,05	Reduction Factor		
fyd	47,62	ksi		
W	17,00	ft^3	29.376,00	in^3
Mr	1.398.857,14	Kip*ft		
Ma	218.000,00	Kip*ft		
2) Lateral Tower				
Outer length	10	ft		
Thickness	0,5	ft		
W	122	ft^3	210.816,00	in^3
Mrd	10.038.857,14	Kip*ft		
Ma	110.000,00	Kip*ft		

Table 5-3: Resisting moment first model (Table made by the author)

Second model				
1) Central Tower				
Outer Length	D=5	ft		
Thickness	0,4	ft		
fy	50,00	ksi		
Ys	1,05	Reduction Factor		
fyd	47,62	ksi		
W	3,5	ft^3	6.048,00	in^3
Mr	288.000,00	Kip*ft		
Ma	200.000,00	Kip*ft		
2) Lateral Tower				
Outer length	4	ft		
Thickness	0,3	ft		
W	3	ft^3	5.184,00	in^3
Mrd	259.200,00	Kip*ft		
Ma	114.000,00	Kip*ft		

Table 5-4: Resisting moment second model (Table made by the author)

In the table above all the calculations are shown, both for the central and lateral tower. The initial dimensions of the towers were overestimated. After many tries, it was possible to dramatically reduce the diameter of the towers and the width of the steel. Probably there would be more margin to reduce the size of the towers but considering only the moment it is better to stay on the safe side. The same procedure is done also for the longitudinal and transversal girders and all the calculations are reported in Appendix A.

5.6 **Cable Loss Study**

The goal of this cable loss study is to model the effects of a snapped cable dynamically. We want to examine the behavior of the bridge due to the impulse of abrupt cable loss. A static model cannot capture the instantaneous effects of cable loss, so we utilize the time history analysis feature in SAP to simulate the event dynamically.

The first step is to remove the desired cable from the bridge. We chose to remove the cable with the highest load, next to the jump span (the worst-case scenario for member deflection). The location of the removed cable is shown in Figure 5-21.

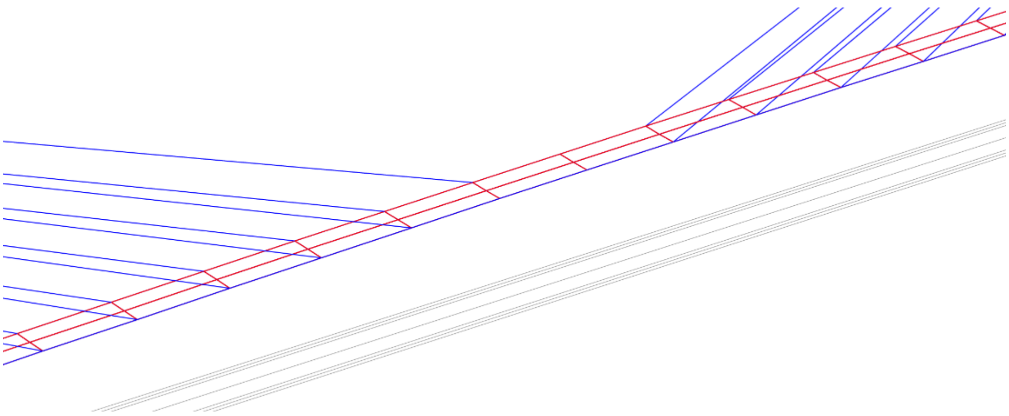

Figure 5-21: Deleted Cable (Figure made by the author, SAP2000)

We then apply point loads, in place of the cable, to the deck and tower. These point loads (in X, Y, and Z) correspond to the resultant forces that the cable was applying to the structure in its undamaged state. As we gradually apply the dead and prestressing loads to the structure, we will also gradually apply these point loads to simulate the deleted cable still taking the load. The time history functions for all of the loads is called a cycloidal front. It is a sinusoidal function that applies the loads to the structure in a way that minimizes the dynamic effects of the load application. The cycloidal front is given by the function in Figure 5-22. The sinusoid ramps from zero to one over the rise time, τ, which in this case is two times the fundamental period of the structure, approximately 18 seconds.

Cycloidal front:

$$F_i(t) = \begin{cases} \dfrac{F_i}{2\pi}\left(\dfrac{2\pi \cdot t}{\tau} - \sin\dfrac{2\pi \cdot t}{\tau}\right) & [0 \le t \le \tau] \\ F_i & [\tau \le t] \end{cases}$$

where:
τ = Rise time
T = Fundamental period

Figure 5-22: Cycloid Front

The cycloidal time history serves as a multiplicative factor for the magnitudes of all of the loads on the bridge. The dead loads, prestressing loads, and point loads from the deleted cable all follow the same upward ramp. However, to simulate the abrupt cable break, the time history for the point loads suddenly drop to zero, while the rest stay at one (Figure 5-23).

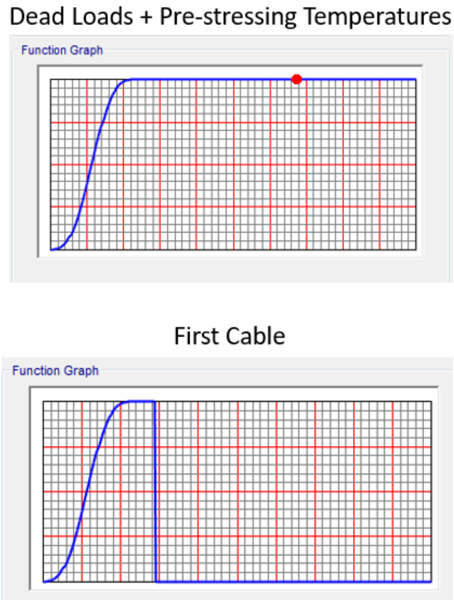

Figure 5-23: Time History Function for Single-Cable Loss

The animations generated by SAP2000 are enclosed in the submission package.

We also want to consider the scenario in which the bridge loses one cable, then loses another cable, sequentially. To do this, we must first conduct static analysis of the bridge with only one cable deleted and recover the loads in the second cable (the one we wish to delete) at steady state. This will be the load that the second cable settles to before breaking.

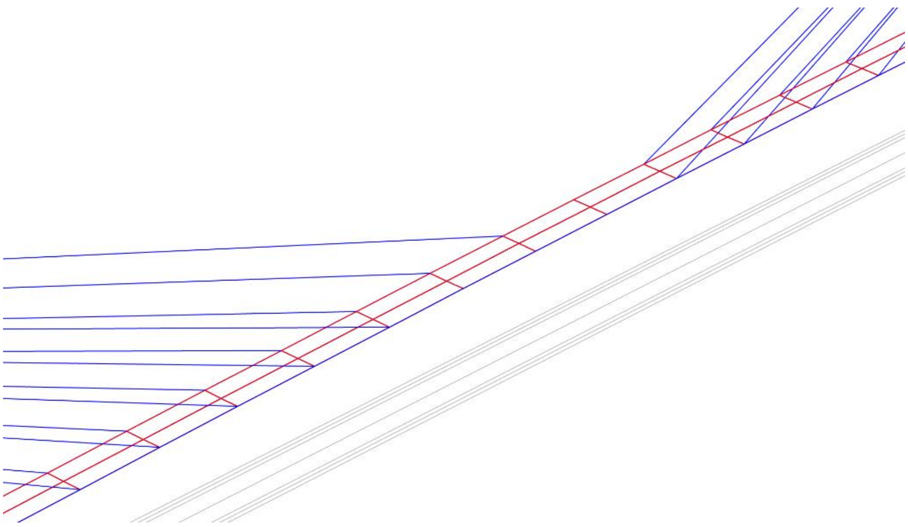

Figure 5-24: Two Lost Cables (Figure made by the author, SAP2000)

We then remove the second cable and apply its point loads (from the un-damaged bridge) back to the structure, as before. This time, when the first cable breaks, the second cable momentarily picks up double the load it had before (cycloid magnitude two), vibrates, and settles to 1.5 times the load it had before (from the static analysis).

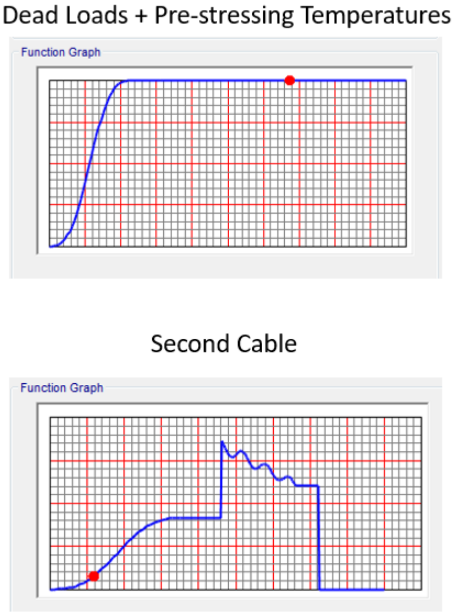

Figure 5-25: Second Cable Cycloid (Figure made by the author, SAP2000)

The time history function for the second cable is shown in Figure 5-25. The cable sits at 1.5 times its original load for a few seconds before it sheds all load. As before, the animations from SAP2000 are attached in the submission package.

5.7 **Pedestrian Load**

In this section, the analysis of the structure subjected to the dynamic pedestrian load is presented. The pedestrian load is characterized by low-intensity forces, which applied to very stiff and massive structures hardly make them vibrate. However, technical and technological developments lead to more slender and flexible structures and pedestrian bridges are following this general trend. As a result, they more frequently require a thorough dynamic analysis to better catch their behavior under pedestrian load. Moreover, footbridges are submitted to the simultaneous actions of several persons and this makes the corresponding dynamic much more complicated. In fact, each pedestrian has its own characteristics such as, weight, frequency and speed and, according to the number of persons on the bridge, pedestrians will generate loads which are synchronous with each other[9].

The dynamic analysis carried on in this section is intended to investigate how much the dynamic response of the structure is amplified when excited with the pedestrian load. To simulate the resonance condition it is important, as the first step, to study the modal analysis of the structure itself since it is fundamental to identify the structural modes affected by the walking/running frequencies.

Activity	Vertical frequency	Lateral Frequency
Walking	1.00 Hz to 2.80 Hz	0.75 Hz to 1.40Hz
Running	1.50 Hz to 4.30 Hz	1.00 Hz to 1.50 Hz

Table 5-5: Pedestrian-Load

Analyzing the results coming from the modal analysis, it was clear that since the structure is very flexible with a high period, the affected vertical and lateral modes are higher modes. Particularly the 65th (Figure 5-26) and the 73rd (Figure 5-27) modes are the ones mainly excited by the pedestrian load since the corresponding frequencies, 1.47 Hz (for the vertical) and 1.36 Hz (for the lateral) fall into the interesting walking frequency range.

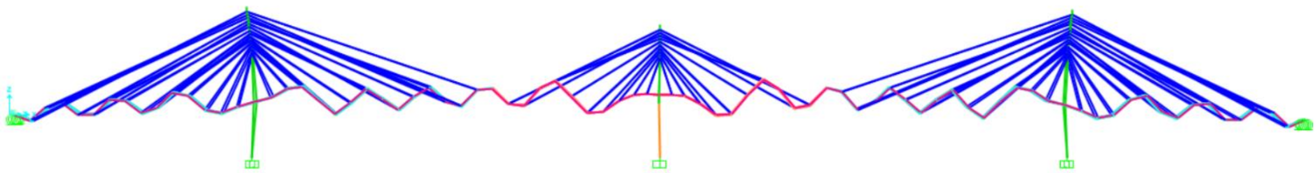

Figure 5-26 Vertical Mode-65th (Figure made by the author, SAP2000)

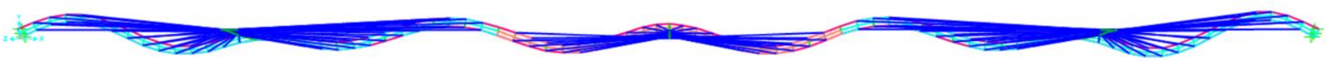

Figure 5-27 Lateral Mode-73th (Figure made by the author, SAP2000)

To simulate the resonance condition, the pedestrian load is applied in the areas of the bridge which present the higher modal displacements in the investigated modes. The value adopted for the distributed load is 9 psi and it is applied alternatively vertically and laterally in the two separate resonance analysis.

[9] (Kala, 2009)

It is worth noticing that the load is applied dynamically following a particular cycloidal ramp function (Figure 5-28). The function presents an initial cycloidal front which, once multiplied by the 8psi pedestrian load, allows to slowly load the interesting area of the bridge. Once the full loading condition is reached, a sinusoidal function is applied which oscillates in between [-1,+1] around the mean amplitude (9psi) and with a forcing frequency which is chosen to be equal to the vertical or lateral frequency of the two investigated modes (Figure 5-26 and Figure 5-27) depending on the resonance condition that needs to be simulated.

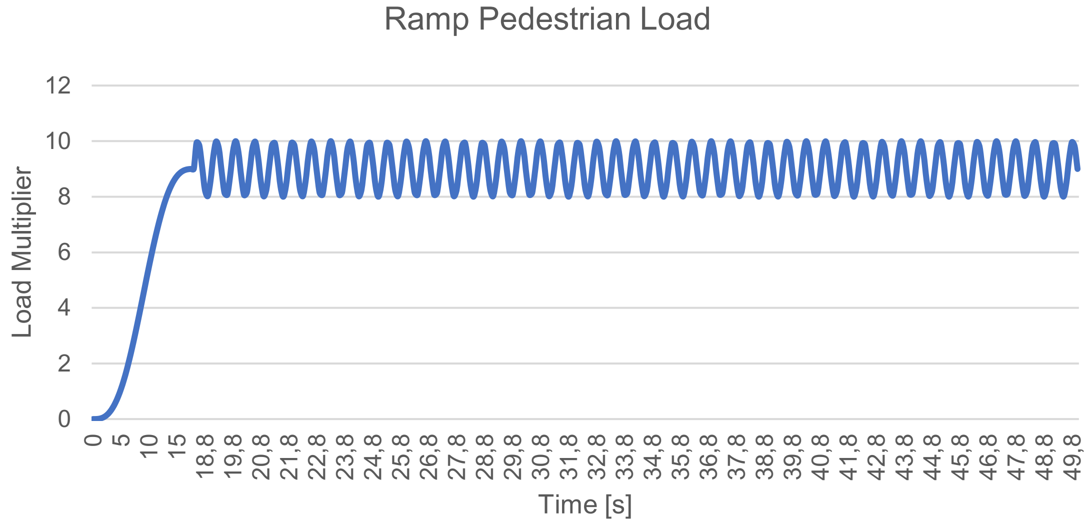

Figure 5-28 Cycloidal Function for Pedestrian Load (Figure made by the author, DynamAssist)

Once the analysis is set it is important to derive the acceleration response of the bridge itself in the two different resonance conditions and check if the acceleration level exceeds the comfort threshold somewhere along the bridge (3.2 ft/s^2). Different joints are monitored along the bridge and the acceleration time history is computed in all these locations considering also different damping conditions for the bridge (Figure 5-29). The damping is set to 0% and 1% damping considering a Classical Rayleigh damping is viscous damping which is proportional to a linear combination of mass and stiffness.

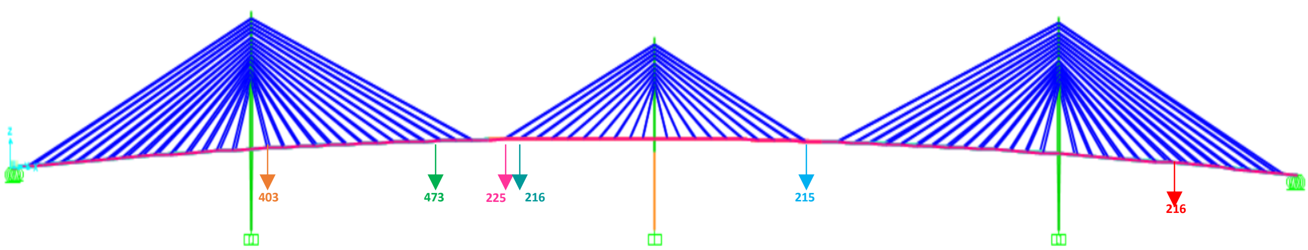

Figure 5-29 Monitored Joints Location (Figure made by the author, SAP2000)

For the 0% damping it is clear, looking at the acceleration values that the areas of the bridge close to the applied load zone present values of accelerations that exceed the comfort threshold. This effect wears off moving away from the loading area. To reduce the acceleration response of the bridge it is important to introduce and consider the damping effect on the bridge and for a structure like this one is reasonable to consider 1% damping. Introducing the damping in both the analysis, the lateral and vertical, it can be noticed (Figure 5-33 and Figure 5-35) that the accelerations values reduce in general and also that the values converge to zeros quicker with respect to the no damping condition. This assures the comfort safe condition with respect to the pedestrian dynamic load.

0% Damping - Vertical Load

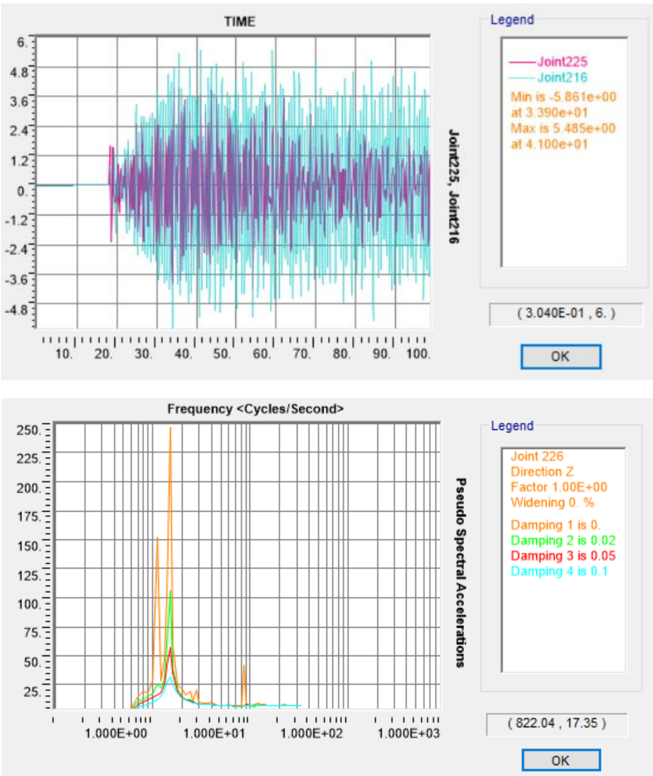

Figure 5-30 Acceleration Time History and Resonance Curve, Damping 0%-Vertical Load (Figure made by the author, DynamAssist)

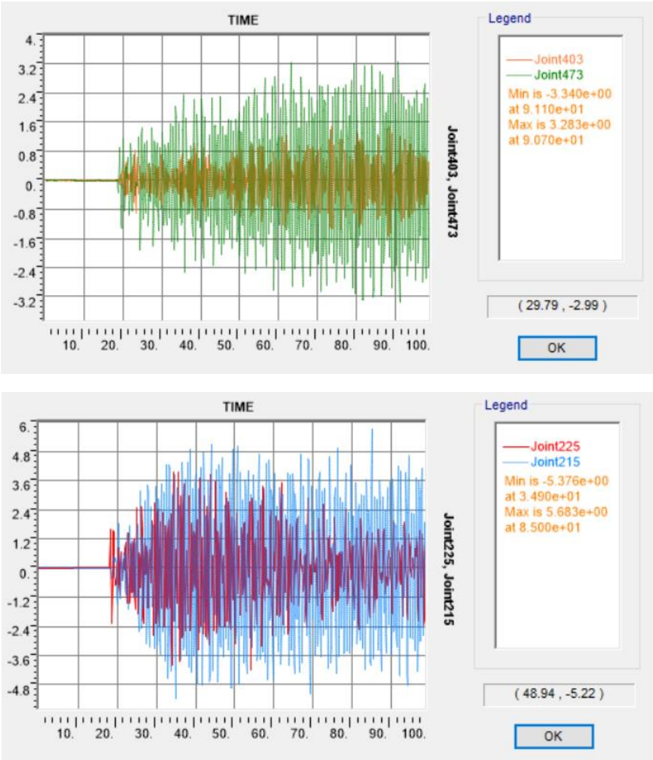

Figure 5-31 Acceleration Time History and Resonance Curve, Damping 0%-Vertical Load (Figure made by the author, SAP2000)

1% Damping - Vertical Load

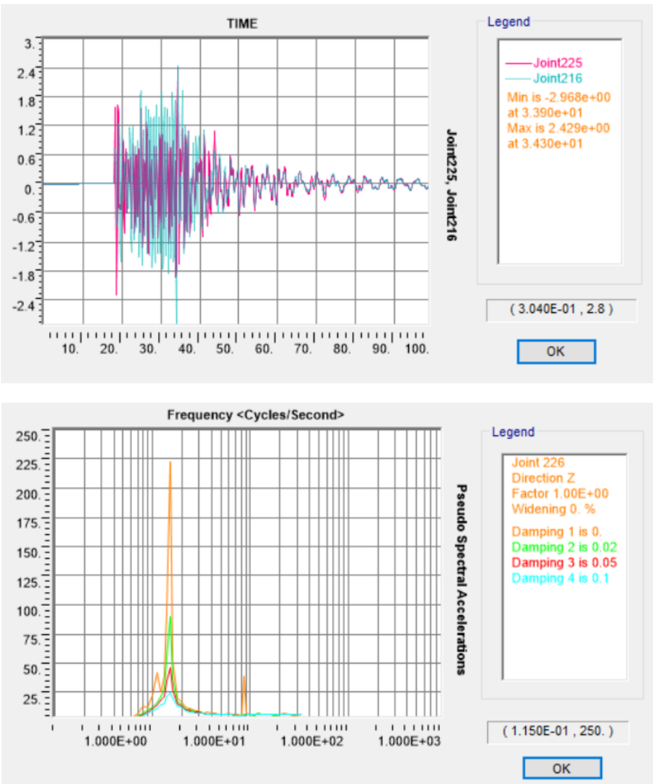

Figure 5-32 Acceleration Time History and Resonance Curve Damping 1%-Vertical Load (Figure made by the author, SAP2000)

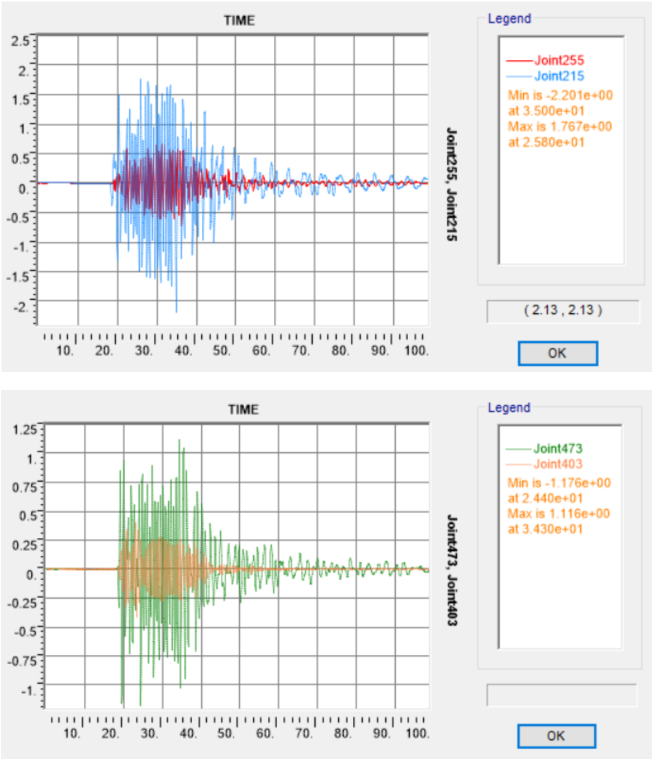

Figure 5-33 Acceleration Time History and Resonance Curve Damping 1%-Vertical Load (Figure made by the author, SAP2000)

1% Damping - Lateral Load

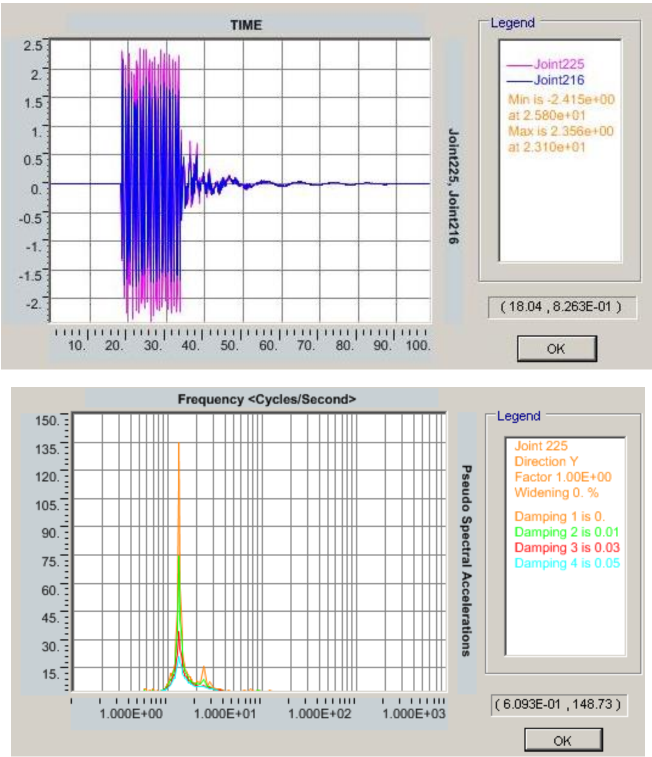

Figure 5-34 Acceleration Time History and Resonance Curve, Damping 1%-Lateral Load (Figure made by the author, SAP2000)

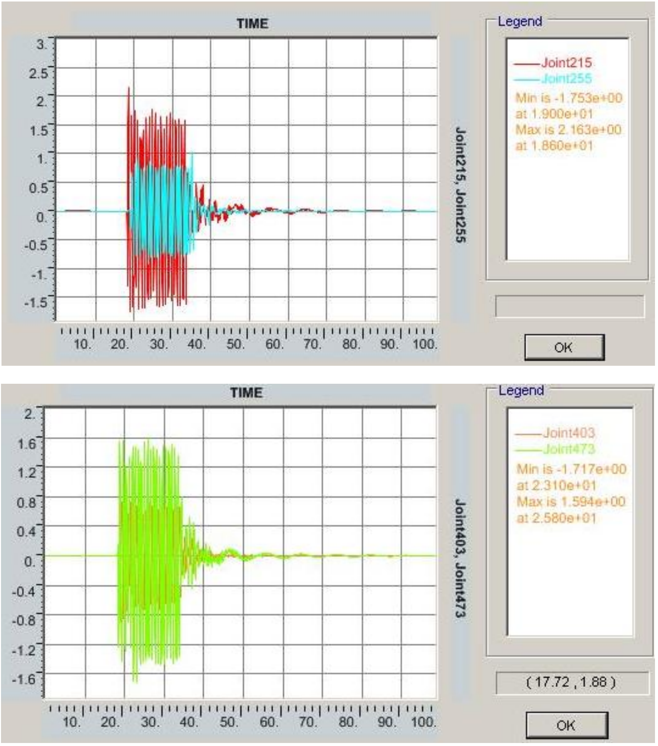

Figure 5-35 Acceleration Time History and Resonance Curve, Damping 1%-Lateral Load (Figure made by the author, SAP2000)

5.8 Blast

The time history function for the blast assessment is derived using Mathcad and DynamAssist. The test is performed with a blast of 50 lb located at 2 m high on the central tower considering a panel of 9 m^2 at two different distances: 3m and 6m.

Figure 5-36 Time history 3m (Figure made by the author, DynamAssist)

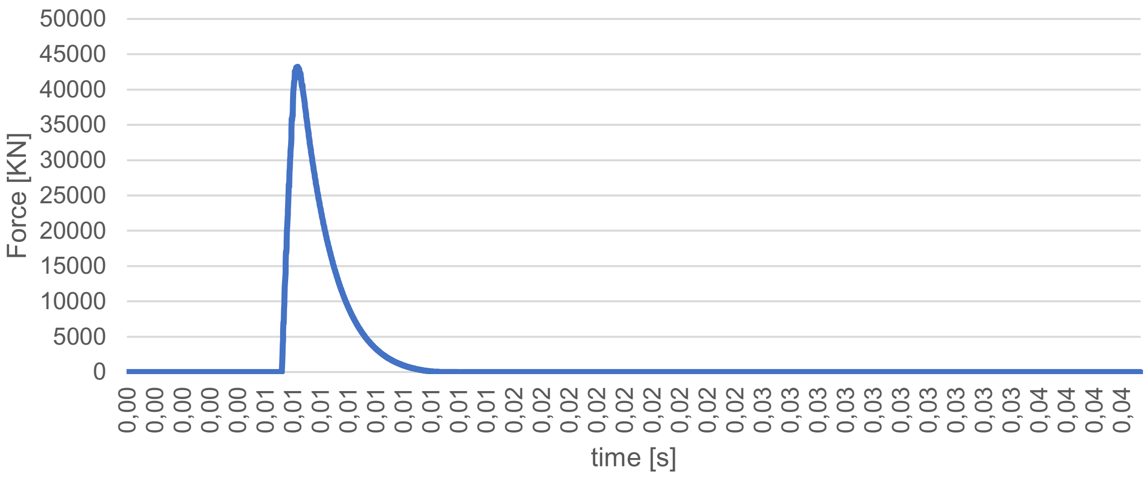

Figure 5-37 Time history 6m (Figure made by the author, DynamAssist)

It is important to notice the great difference between the two-time histories, doubling the distance in the second graph the force is almost 4 times less than the first one.

As expected, the effects of the deflagration are extremely local and even if the all the bridge's modes are excited the distance is playing a fundamental rule, in fact also the moment in the central tower are considerably different.

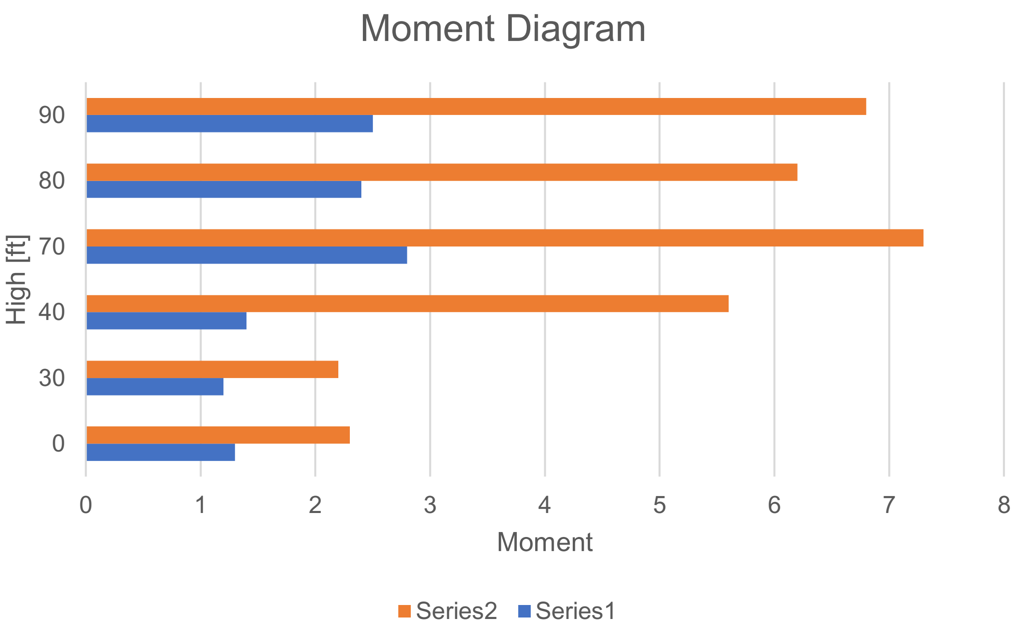

Figure 5-38 Moment central tower (Figure made by the author, DynamAssist)

Analyzing the results, it is possible to conclude that it is important to avoid the direct contact between the structure and the blast, to do this we came out with a possible structural solution where the distance between the deck and the towers is incremented.

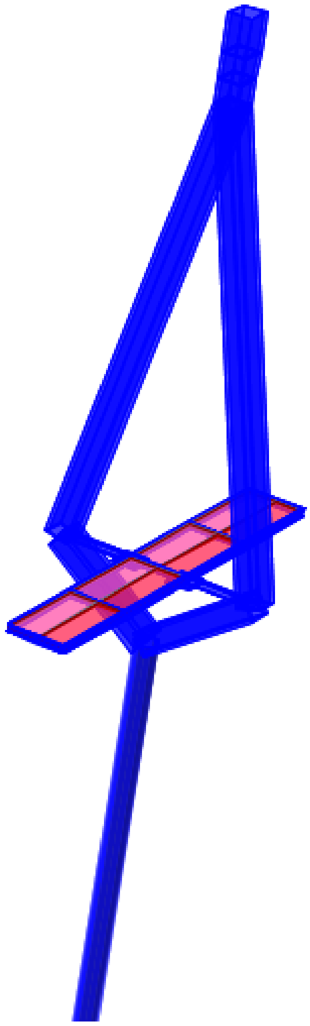

Figure 5-39 Structural solution (Figure made by the author, SAP2000)

6 Optimization and Efficiency

6.1 Lifting the Bridge

Since we want to keep the bridge fully ADA compliant, the highest deck elevation is 150 feet above water level (allowing us to keep the maximum slope of the arced deck below five degrees. We also need to allow clearance for the largest ships to pass underneath the bridge. We aim to have the same clearance as the Verrazano Narrows Bridge, 220 feet. The only way to achieve this is to make the center tower capable of an 80-foot lift.

A moveable cable-stayed bridge of this span has never been done before, so the movability has to be ingrained into the fundamental design of the bridge. Our center tower is shorter and of a smaller span than the other towers with this idea in mind. The smaller center span has to be as light as possible, so the tower is a circular steel tube. We choose to use a single tower so that there is only one mechanical power source necessary. This mitigates any need for synchronization between motors, and any possibility of jamming the towers on an angle due to poor synchronization. The circular tower sits inside of a large, concrete shaft, and has shear keys to resist torsion. A cross-section of the tower is shown in Figure 6-1.

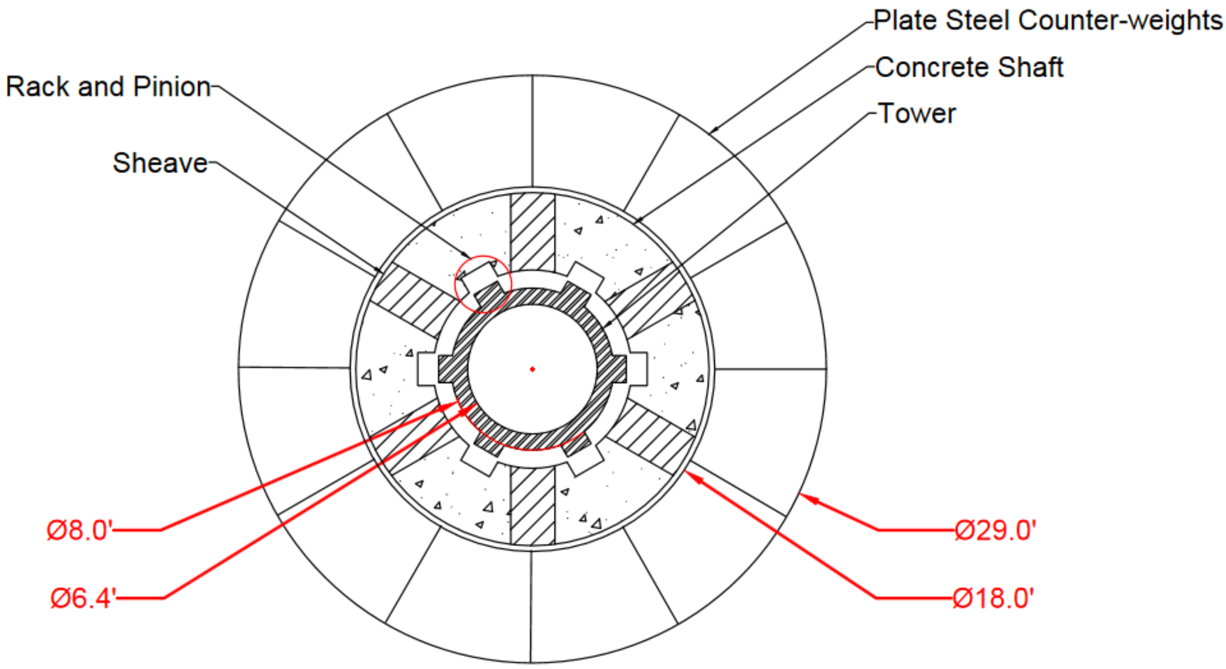

Figure 6-1: Lift Tower Cross Section (Figure made by the author, AutoCAD)

The counterweight sits around the concrete shaft. The concrete shaft holds the six sheaves that tie the counterweight to the steel tower with cables. This shaft must hold twice the weight of the bridge (counterweight + center span), so it is 18 feet in diameter, with five-foot-thick walls. The counterweight is made out of segments of plate steel and is 29 feet in diameter with 5.5-foot wall thickness. In order to balance the dead weight of the center tower, the counterweight must be 38 feet tall and weigh in at 7,700 kips. Because of the counterweight, the mechanical system does not have to lift the weight of the bridge – it only needs to fight the internal friction and inertia of the system. We are using six simple rack and pinion systems to move the bridge, shown in Figure 6-2.

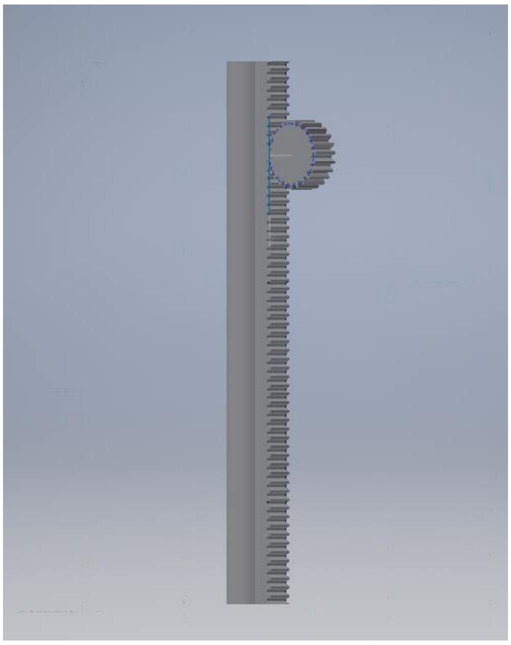

Figure 6-2: Rack and Pinion (Figure made by the author, Revit)

The pinion gears are embedded in the concrete shaft. They are attached to the motor via gearboxes and driveshafts, while the rack is attached to the tower. In this way, the entire mechanical system can remain stationary while the pinion gears lift the tower. This greatly simplifies the mechanical detailing. As the pinion gears drive the racks upward, the counterweight slides down along the concrete shaft. This is illustrated in Figure 6-3. There are multiple pinion gears along with the height of the shaft for each rack. Having multiple opinions not only decreases the load on each pinion but also provides the tower with moment resisting couples in all lateral directions.

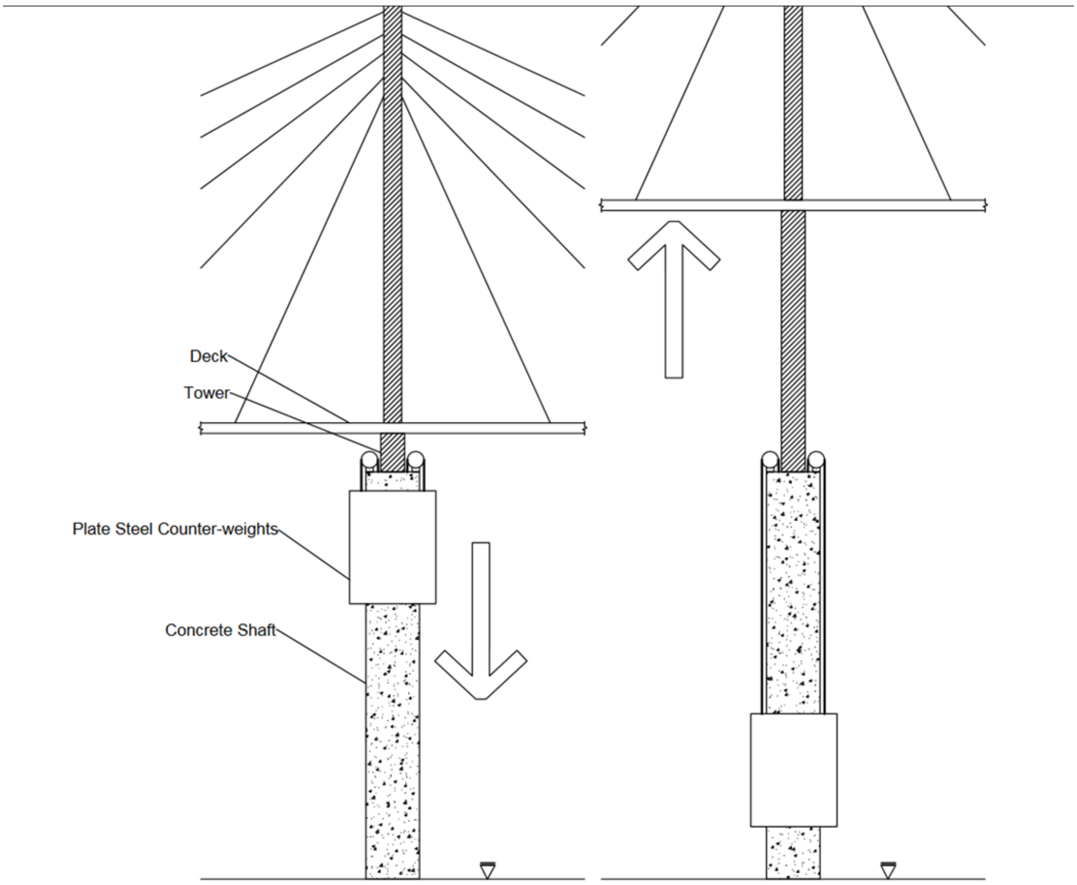

Figure 6-3: Moving the Center Span (Figure made by the author, AutoCAD)

The current design allows for lifting the main span without the presence of a live load. If in the future we wish to make an event out of having limited live load during lifting, we can also implement the use of ballast tanks on the counterweights. These tanks would sit dry during normal operation. However, if we want to lift with live load, the tanks can be pumped full of water in order to counter-weigh the load discrepancy between the weight of the loaded span and the weight of the steel counterweight.

7 Constructability

The bridge will be build following a traditional cantilever approach as for most of the cable-stayed bridge. It consists of a step by step process where each component of the deck is cantilevered and then connected to the corresponding cable before proceeding with the other portion.

The deck is a composite structure that needs to be built in two steps: first, the steel elements are assembled by cantilevering to the previous segment already built and then the concrete slab is cast over them and they stay attached and tensioned. The decision to have a composite deck was mainly due to a structural region because the spans were too long for the prestressed concrete deck and the behavior would not have justified a steel plate configuration.

The best construction process would be to construct simultaneously the three towers and then the deck on both sides of each tower until they will meet in the center. On the other hand, it is important to underline that the asymmetric construction process is not needed in a cable-stayed bridge as it is in a suspension bridge. This is giving great flexibility to the all construction process and in an area, like New York City it is extremely useful.

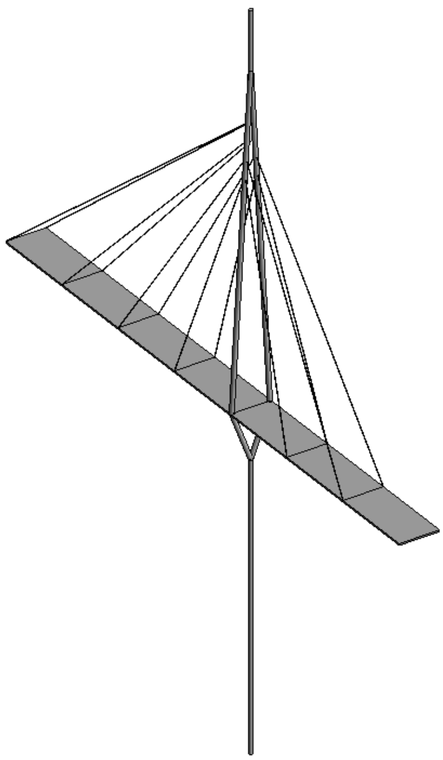

Figure 7-1 Cantilever Construction Process of the middle span (Figure made by the author, Naviswork)

7.1 Fabrication and Transportation

Due to site location and to optimize the construction process it is ideal to limit the operation on site. The central tower is composed of 10 prefabricated components while the two laterals in 12 to make each of them less than 48 feet and ensure the transportability following the New York regulation.

The metal component of the deck is the same for all the section allowing a standard production, transportation, and construction, no transportation precautions should have to be taken in this case.

A possible solution could be to order entire pre-made lengths of the girder sizes for the three spans, cut them immediately after the production to the sizes we need for transportation and erection. Since the girder size is exactly the same along the deck, we would choose a girder size that the steel mill has enough length of, and plan to use that in our design.

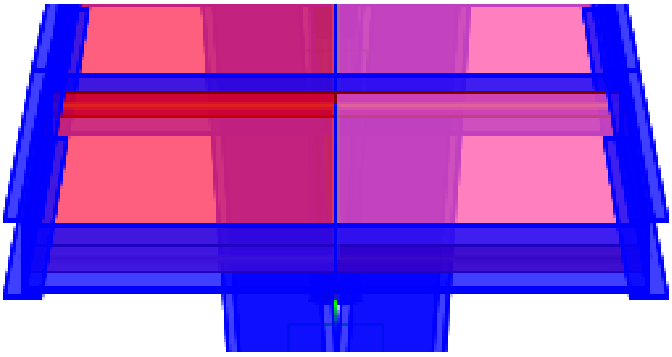

Figure 7-2 Deck (Figure made by the author, SAP2000)

8 Project Evaluation

Engineers involved in any kind of project have always to deal with financial and economic analysis, they are both fundamental to reach the final goal, design a sustainable infrastructure and efficient for a long period. Even if they look at the project from a different perspective, they are both equally important.

The financial analysis is necessary to understand the investment that will be needed to build, operate and maintain the project, so it is strictly connected with the availability of money and cash flow, cost of capital and consequently the possible return on investment.

On the other hand, economics is a broader concept because inside this analysis there are many different thoughts that can be quantified monetary, with some assumption.
We can say that it is related to the cost and benefit of a certain project and is important to underline that a good project has always higher benefit than cost, also from a socio-economic perspective. Some examples of economic benefits are the reduction of travel time, reduction of accidents, creation of jobs and a lot of others that can be also indirectly created by our project[10].

The private sector is more concentrated in financial analysis and public on the economic one because the primary goals are different. For the first one, the main objective is to make a profit and the other components of the project such as socio-economic benefit and environmental issues are seen more as constraints than as real objectives because everything is finalized to the return on the investment.

For the second one, the main objectives are equity distribution of cost and benefits across regions and group of people, aesthetic and integration in the community, disruption related to construction and quality of life. This means that for the public the most important goals are what the private usually consider as constraints to reach a final profit

[10] (Florio, 2003)

To perform a wide-ranging project evaluation, it is extremely important to look at the project both from an economic and a financial perspective, especially if both the private and public parties are involved as it happens for Hoboken Bridge.

8.1 Supply and Demand Analysis

The study of supply and demand is always a very important factor for project evaluation, especially when it comes to deal with transformation interventions. It is therefore important to know at a preliminary stage what is the need of the population living in a specific area and which offers similar to ours are already available. In this way, we can use the study and the design evolution to satisfy the requests of that specific typology of users, which most need the infrastructure[11].

What we will do in this part of the thesis will be to analyze the socio-economic characteristics of the citizens living near the infrastructure and how they commute to Manhattan, identifying the number of people and any other information that will be useful to know the potential future users of the bridge. It will also be very important to study the tourist flow of the areas affected by the intervention.

Firstly, a complete analysis of all the commuters from New Jersey to Manhattan is performed, considering a number of commuters, commuting transportation method and major crossing paths used. The results obtained from the Census Transportation and Planning are shown in the following images and tables.

[11] (Pearce, 1983)

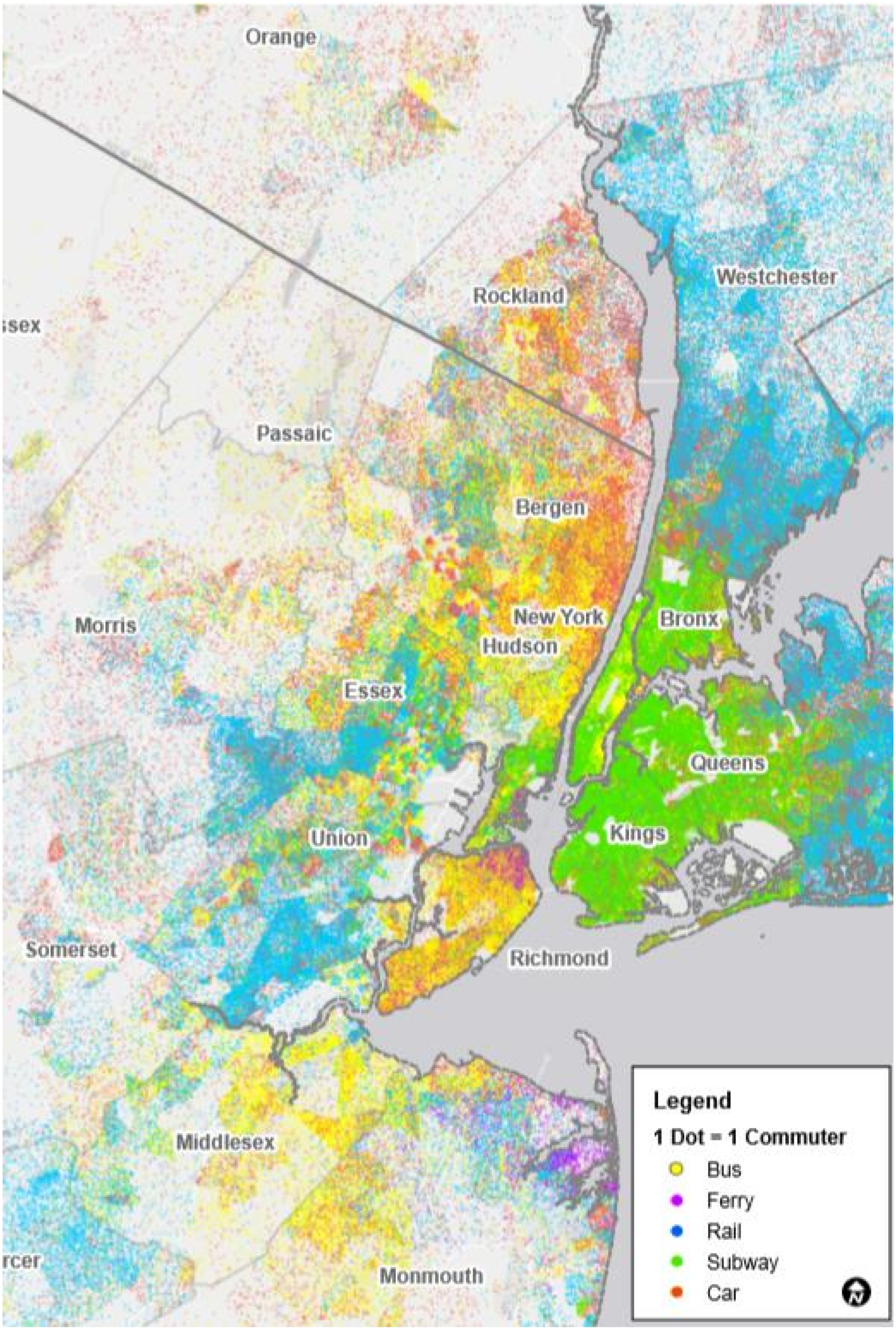

Figure 8-1: Mode split by origin for commuters (Source: PANYNJ, Community Survey 2006-2010)

Figure 8-2: Major paths between NY and NJ (Source: Trans-Hudson Commuting Capacity Study)

	Crossing/Facility	1990	% Share	2015	% Share	Absolute Change	Percent Change
Auto	Holland Tunnel	121,982	14.9	106,165	9.0	-15,817	-12.97
	Lincoln Tunnel	167,383	20.5	142,484	12.1	-24,899	-14.88
	Total	289,365	35.4	248,649	21.1	-40,716	-14.07
Bus	Holland Tunnel Bus	15,162	1.9	29,243	2.5	14,081	92.87
	Lincoln Tunnel Bus	233,777	28.6	426,931	36.2	193,154	82.62
	Total	248,939	30.4	456,174	38.7	207,235	83.25
Hudson Rail Tunnel	NJT	52,698	6.4	172,419	14.6	119,721	227.18
	Amtrak	19,196	2.3	21,958	1.9	2,762	14.39
	Total	71,894	8.8	194,377	16.5	122,483	170.37
PATH	Downtown	118,205	14.4	109,785	9.3	-8,420	-7.12
	Uptown	75,778	9.3	135,752	11.5	59,974	79.14
	Total	193,983	23.7	245,537	20.8	51,554	26.58
Ferry	Total	14,109	1.7	34,887	3.0	20,778	147.27
Total Transit		528,925	64.6	930,975	78.9	402,050	76.01
Total Trips		818,290	100.0	1,179,624	100.0	361,334	44.16

Figure 8-3: Daily trips (Source: New York Metropolitan Transportation Council)

It is important to underline, as it is possible to notice from Figure 8-3, that George Washington and Tapped Zee bridges are not considered in the analysis because too far from Hoboken and Midtown area. The commuters from NJ to NY are constantly growing and especially the uptown trips almost double, showing a great need for new infrastructures. As it is possible to understand from the data we are dealing with a great number of potential users and only a small fraction of all of them would be enough to justify the construction of the bridge.

Moreover, recently the Port Authority identified several deficiencies for each of the infrastructures, supporting the thesis that the construction of a bridge on this side of Manhattan, would not only improve the resiliency of the city but also help the transportation system.

Related to the Hudson River Tunnel*:*

- *"The Tunnels and Highline (a series of structures that elevate the NEC over the Meadowlands from Secaucus Junction to Newark Penn Station) reached their maximum rush-hour capacity years ago and cannot accommodate more trains and passengers. Until new capacity is built to supplement trans-Hudson train travel, the region's economic growth will be artificially capped.*
- *Several bridges on the Highline are long past the end of their useful life and must be replaced.*

The weight of the Hudson River fluctuates with the tides and as a result, the tunnels compress at high tide and expand at low tide. More than 100 years of this daily abuse has taken a significant toll on the tunnels' engineering.

- *In 2012, the tunnels were flooded with water from the Hudson River for the first time ever by the 14-foot storm surge from Superstorm Sandy. The brackish saltwater was quickly pumped out but left a coating of chlorides and sulfates which cause long-term corrosion to reinforce steel and concrete. In 2014, an engineering study found that more than $350 million is needed to fix the damage. Fortunately, the study also found "negligible, if any chemical impact" to the tunnel's outer cast iron ring, so they can continue to safely support operations for the time being"[12].*

Related to the Lincoln Tunnel

- *"The lack of PABT capacity causes overflow operations, including NJT and other bus carriers, plus less formal ("jitney") bus and van services to operate on the neighboring streets. This exposes passengers to the elements, creates congestion on sidewalks and local streets, air and noise pollution and is an eyesore for neighborhood residents.*
- *Peak hour bus traffic through the Lincoln Tunnel has grown rapidly and is projected to grow from 730 buses to 1,000 by 2040. This leads to backups and delays at its entrance in the Meadowlands. Passengers are losing time and travel reliability suffers.*
- *There is not enough storage and layover space for buses in Manhattan. The result is that many buses must be stored in New Jersey during the day and then battle rush-hour traffic in the evening using only the two eastbound lanes available to access the PABT. This leads to countless delays for buses and for evening peak period passengers.*
- *The terminal's operating deficiencies affect the customer experience. Long lines are the result of the unreliable deployment of buses and inefficient design of boarding areas. Waiting areas are insufficient and unpleasant and expose commuters to bus fumes."*

Related to the PATH:

- *"Many platforms, especially on uptown PATH, are short and/or narrow reducing the ability to expand to longer trains to add passenger capacity.*
- *The track configuration (geometry) and interlockings in Jersey City slow trains down and limit throughput.*

[12] Cf. (The Port Authority, 2016), pag 3

- *Lack of peak hour storage at the Newark Penn Station terminal and inability of the inefficient terminal to turn trains quickly limits capacity too.*
- *PATH is not well integrated with the rest of the region's urban transit network, especially the NYC subway system. In Manhattan, two Uptown PATH stations lack direct underground connections to nearby NYC subway stations.*
- *The two systems have different railcars (even though PATH is compatible with NYCT Division A or the "numbered" line cars) and fares are administered separately*.[13]"

Related to the Ferries

- *"Ferries operate in a private sector environment, at least as far as operations go, but the Port Authority has subsidized the construction of docking facilities. To cover operating costs, ferries have had to raise their fares faster than the PATH system, which has reduced ferry ridership.*[14]"

To better understand the demand more accurate research has been performed for Hoboken. Which is a residential area of New Jersey highly populated that in the last ten years has rapidly grown to reach 55,000 residents. Despite his dramatic growth the transportation system mainly relies on the Path that provides a great service for the commuters that are going downtown but not for the upper part of Manhattan. More than 22,000 people are commuting from Hoboken to uptown every day.

The main goal of the new Hoboken Bridge is to satisfy that need and provide a service for this region, it would be the only direct way to go to uptown and it would have not any other competitive facility. For this reason, it is reasonable to think that it can rely on a constant flux that can justify the initial investment.

Lastly, in Manhattan, there are more than 400,000 tourists per day and 10,000 walks on the Highline. It is reasonable to think that a new bridge in one of the most touristic destinations worldwide will become an attraction, especially for tourists that are already walking on the Highline every day.

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[13] (The Port Authority, 2016), pag 4
[14] (The Port Authority, 2016), pag 5

The final results are summarized in the following table, where we compared the potential users (all the commuters from NJ to Manhattan) and the effective users, a realistic percentage of all the commuters. To understand which percentage could effectively commute by bike or walking, I studied the flux of people commuting every day from Queens and Brooklyn to Manhattan. The results are extremely positive and surprising because they represent 11% of all the total commuters.[15] As it is possible to notice from the table the percentage assumed for Hoboken Bridge is conservative and the total is only 5%, way lower than the one on the other side of Manhattan.

Crossing Alternatives	Total Users	Percentage	Effective Users of Hoboken Bridge
Holland Tunnel	67.500,00	1%	675,00
Ferry	17.500,00	5%	875,00
Lincoln Tunnel	284.000,00	5%	14.200,00
Path Uptown	67.500,00	20%	13.500,00
Path Downtown	54.500,00	1%	545,00
Hudson Rail Tunnel	97.000,00	1%	970,00
High line	10.000,00	10%	1.000,00
Tot	588.000,00	5%	31.765,00

Table 8-1: Potential Users (Table made by the author, Excel)

In Table 8-1: Potential Users (Table made by the author, Excel) the first column shows all the crossing alternatives, then in columns two and three are compared the total users to the potential users with a percentage assumed by the author from the previous studies[16].

8.2 Service Life and Cost

The physical life of infrastructures is only one side of the service life, which can be extremely complicated to evaluate because of its dependency on technology, regulation, and economic and social conditions. Moreover, service life is directly related to performance, the ability to provide a service that the community expects. Which is a function of effectiveness, reliability, and cost.

It is possible to understand that the direct evaluation of the service life of a structure is almost impossible and for this reason, the useful service life for the cash flow and benefit-cost

[15] (NYCEDC, 2012)
[16] (Barone, 2017)

analysis was assumed to be 70 years, following the standard used by the New York City DOT. This value can be considered a good approximation, and 70 years is considered the amount of time in which no important rehabilitation and extraordinary maintenance are needed.

Another important parameter to consider is the service life in relationship with the cost, every project is subjected to different costs during its service life. To perform a complete project evaluation is important to evaluate all the expenses, starting from the initial design cost, to construction and operation and maintenance.

This study has been performed using as a guideline a cable-stayed bridge designed by HNTB Corporation, Bob Kerry Pedestrian Bridge, in Omaha, USA. The decision to follow this project as a guideline is not only due to the similarity between the two projects but also to the availability of data provided by Professor Theodore Zoli.

The total construction cost is assumed to be $250.000.000 with an initial design cost that can be assumed around 10% of the total bridge cost. The other important expenses are related to operation and maintenance. As reported in a study of Northwestern University, conducted by Professor Yingchun Zang this cost can be assumed around 1% of the construction cost per year, with an increment of 4% per year[17].

Service life	70	Years
Construction Cost	$ 250.000.000,00	
O&M Cost	$ 2.500.000,00	
Design Cost	$ 25.000.000,00	
Construction Duration	2	Years

Table 8-2 Cost Assumptions (Table made by the author, Excel)

[17] (Zang, 2003)

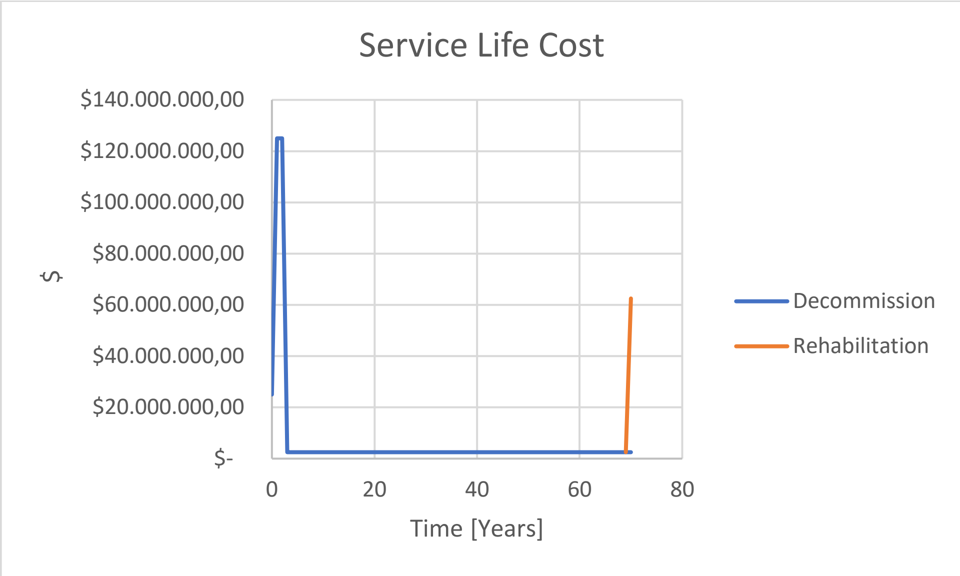

Figure 8-4 (Figure made by the author, Excel)

The chart above represents the distribution of cost during the service life of the infrastructure, assumed to be 70 years. Two different scenarios are considered, the decommission after the end of the service life or rehabilitation to enhance the physical and socio-economic condition of the bridge. This means the possibility to substitute the cables or expand the deck or structural enhancement due to a change in the regulation.

On the y-axis the annual investment along the service is represented, the initial investment is very high because the design cost and the construction costs are all concentrated in the first three years, while from year 3 to year 70 the capital cost is low because it consists only of the operation and maintenance expenses.

This concept can be well described by Lemer theory[18] that is summarized by the following graph, aging of the structures is normal and has to be considered from the beginning. Through a capital programming approach, we could identify, program, budget, and account the capital need for infrastructure. Avoiding unexpected expenses and issues during the daily operation of the infrastructure. The intersection between the expected performance curve is defined as the service life, it is shown as the expectation is naturally rising during the years and a change in the regulations (new standard applied) could dramatically reduce the service life of the infrastructure. It is immediately possible to notice from Figure 8-5 that after a certain amount of time, depending on the infrastructure, an important aging component will appear.

[18] (Chang, 2018)

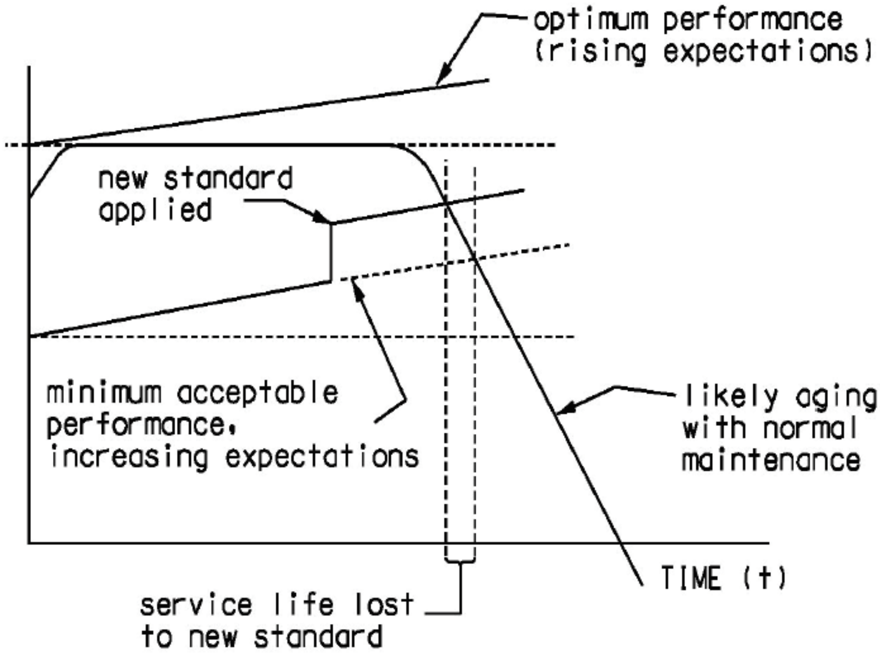

Figure 8-5: Lemer Theory (Source: Capital facility and Planning class material, Columbia University)

8.3 Profit

Entrepreneurs and owners are concerned with the financial success of a project. Three main questions are of interest. First, will the project be profitable? Second, will the profit be sufficient to justify the investment required? Third, once the project is completed, will it be worth more than it cost to build it?

The project will be profitable if the revenues received from the project are sufficient to cover its costs. The revenue could include subsidies from government agencies as well as revenues from users of the project. Revenue from users will depend upon the price that is charged and the value of the project to potential users. For projects that add capacity within a competitive market, such as most real estate projects, the prices that can be charged will rise and fall with market forces. For projects where competition is difficult, such as new bridges or toll roads, the prices can to some extent be established by the owner. In the competitive situation, the question is whether the project can be constructed and operated so that it achieves a profit given expected market prices. In a monopolistic situation, the challenge is to choose the prices that will maximize profits, assuming that there is, in fact, a range of prices that could be profitable[19].

[19] (Martland, 2011)

Hoboken Bridge can be considered in the middle between a monopolistic and a competitive situation. There is no direct competition because the closer bridge is 9 mile north, the George Washington Bridge, but there is an indirect competition from other infrastructure and transportation methods, first of all, cars and busses through the Lincoln Tunnel, the PATH and the old ferry system between New Jersey and Manhattan.

The first important assumption that has to be made at this step of the project evaluation is how can the infrastructure be profitable and secondly which is the price that a user could be willing to pay for the service. To understand these parameters analysis of the other infrastructure in the area was performed. The following prices were obtained: average price for a round trip on the ferry system is around $10, for the path $5.5 and for the Lincoln Tunnel $20. Using these values as reference a first analysis can be performed assuming an average toll price of 5$ considering the fact that walking at 3mph from Hoboken to Mid-Town Manhattan the bridge would be faster than any other option. This assumption can be considered cautionary and different sensitivity analysis will be performed in the financial study to understand better how the return on investment would change with the toll price.

Firstly, the profit was studied using the break-even volume and find a reference number of users that would allow the bridge to be profitable, without any public financial support, at 20 years from the construction (a reasonable time of return for a pension fund and for infrastructure investment). As shown in Figure 8-6 the break-even volume is around 3 Million users per year, around 2% of the total number of commuters from New Jersey to New York.

The blue line represents the revenue, constantly growing with the users, and the orange line the cumulative costs for the first three years, considered to be independent of the number of users. The intersection point, with coordinates $300 million and 3.2 million users, is an approximate value of the traffic volume sufficient to justify the cost of the Hoboken Bridge.

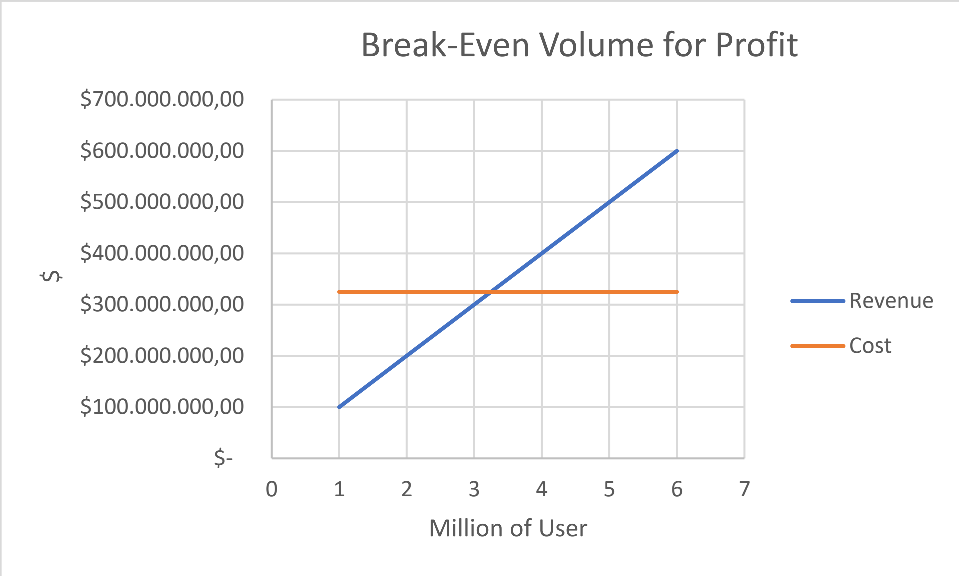

Figure 8-6 (Figure made by the author, Excel)

8.4 **Economic Analysis**

The economic analysis also considers the costs and benefits not deriving from direct monetary returns. In other words, it must consider those costs and benefits generated by external or indirect factors, considering the positive and negative impact on the community.

Briefly, it can be said that the purpose of financial analysis is to present a valid financing plan and assess what the financial situation will be for the management and maintenance of the work, while the purpose of cost-benefit analysis is to verify the degree of utility. Through cost-benefit analysis, it is possible to measure the increase in global well-being of the community brought about by the realization of the work through the identification, evaluation, and comparison of the complex of social costs and benefits linked to the intervention and environmental externalities.

The estimate of social costs and benefits is based on economic analysis. The variables considered in this analysis are valued at their account price, this is the not distorted price by possible market inefficiencies.

The different steps of the analysis usually are:[20]

- Identification of the effects of the project
- Definition of monetary effects

[20] Cf. (Martland, 2011)

- Construction of cash flow
- Taking a discount rate
- Elaboration of economic performance criteria
- Formulation of the final choice

Once all costs and benefits have been identified and estimated in monetary terms, the true evaluation takes place by aggregating the repetitive items and comparing them. In principle, for each year of the project's life, a total social benefit is calculated to which a total social cost will be subtracted. The series of all the benefits and costs for each year of life must be aggregated into a final value that allows us to evaluate the project. The problem is that a direct aggregation of all the items is not possible as there is a temporal inhomogeneity of both costs and benefits and therefore the simple sum cannot be applied. It is necessary to update these values at a time through a weight, the discount rate (DR) so that the relevant values in recent times have a greater weight than those relevant in remote times[21].

$$S_n = \frac{1}{(1+r)^n}$$

The following formula shows as each term as to be discounted depending on *n*, the numbers of years from the beginning of the investment. The choice of the discount rate is a critical step of the analysis and it is extremely hard to determine because dependent on many factors. Some of the most common hypothesis are:

- Use the interest rate applied by the banks for short term period loan
- Use public agencies reference DR

For this reason a brief review of the estimates of values to be attributed to the discount rate shows how for some countries, with a specific focus of some contributions on France and the United Kingdom, on a definition of two factors in the part of pure preference (temporal impatience and life expectancy), and a hypothesis of utility elasticity based on economic analyses of wage tax data at different income levels, the range for EU countries is between 2.3 and 6.8%, with consumption growth as the main source of variability. The range defined for the elasticity factor is between 1.3-1.6, the consumption growth (historical) varies from 1% (Denmark) to 2.7% (Austria), a pure preference rate between 0.8 and 1%, calculated

[21] (Martland, 2011)

assuming zero the impatience value. Specifically, in two large European countries, France and Great Britain, in the first case, an appropriate 3.8% rate is defined[22].

These examples show the great variability of the values that can be attributed to the discount rate in the economic analyses of the CBAs, whether they are constant values for the entire time horizon of analysis or they are variable as a function of time, so the debate on the choice and on the how-to estimate the correct social discount rate is alive and currently constitutes one of the most active strands of research in this branch of economic theory.

In the economic analysis for Hoboken Bridge we decided to use a DR of 7%, the one required in the United States for a federal grant, another possible solution could have been to use 3.6%, the one required by the DOT of New York[23]. The decision to take the 7% is to stay on the safe side of the analysis and make the bridge eligible also for a federal grant. After the 9/11 attack and the terrible consequences that had on the Trans-Hudson connections, make this bridge important not only from a local perspective but also from a regional one, so federal grant could be more than justified.

The development method chosen for Hoboken Bridge is a Public-Private Partnership, further explained in the following chapter. So instead of using the economic analysis in the traditional way, it has been applied to understand which the public support for the investment could be. Which depends on the direct and indirect economic benefit to the community.

The economic benefits of a pedestrian bridge are multiple, ranked in Table 13, the main three factors are the gain in resiliency, time savings for commuters and environmental advantages for the emission reduction that would be saved.

Also, other benefits have been identified but only the first three where quantified and monetized, the main reason is the possibility to directly evaluate the first three benefit in the ranking while the others would require a level of analysis, software's and studies not available. Through Table 8-3: Benefit ranking (Table made by the author)all the possible benefits were sorted, and a brief description is provided in the second one, the third column classifies the benefit depending on the capacity to give them a monetary value.

[22] (Evans, 2004)
[23] (Trasportation, 2017)

Benefit	Description	Quantified
Faster Travel	Faster time to reach destination when the bridge is open	✓
Emission Reductions	Lower GHG emissions when the additional distance is avoided	✓
Safety	Difference between the number and severity of injuries on the alternate route and the build route	*No*
Resilience	Faster evacuations during emergencies	*No*
First Responder Access	Faster emergency response time	*No*
Recreation	Enjoyment of crossing bridge, or view of the bridge, or view from the bridge, etc	*No*
Property Values	Change in a nearby property values depending on presence of bridge	*No*

Table 8-3: Benefit ranking (Table made by the author)

Firstly, we considered the actual commuting scenario from Hoboken to Manhattan, without the infrastructure. As results, we obtained an average commuting time of 50 minutes, through at least 2 different transportation methods, an average cost for a daily trip of $7 (obtained through the average of the subway and ferry tickets).

Secondly, we simulated the commuting using the Hoboken Bridge, the average time to walk for 4080+1000= 5080ft at 4 ft*sec is 22 minutes. This would represent a time saving of (50-17)*2 = 58 minutes and a monetary saving of 7-5= 2$ per day for the community. According to Federal Government BCA[24] guidance, this number should be halved for nonbusiness travel. Inflating this number from May 2015 to June 2016 by the Consumer Price Index resulted in an hourly wage of $14.56/hr[25].

[24] (Trasportation, 2017)
[25] (Trasportation, 2017)

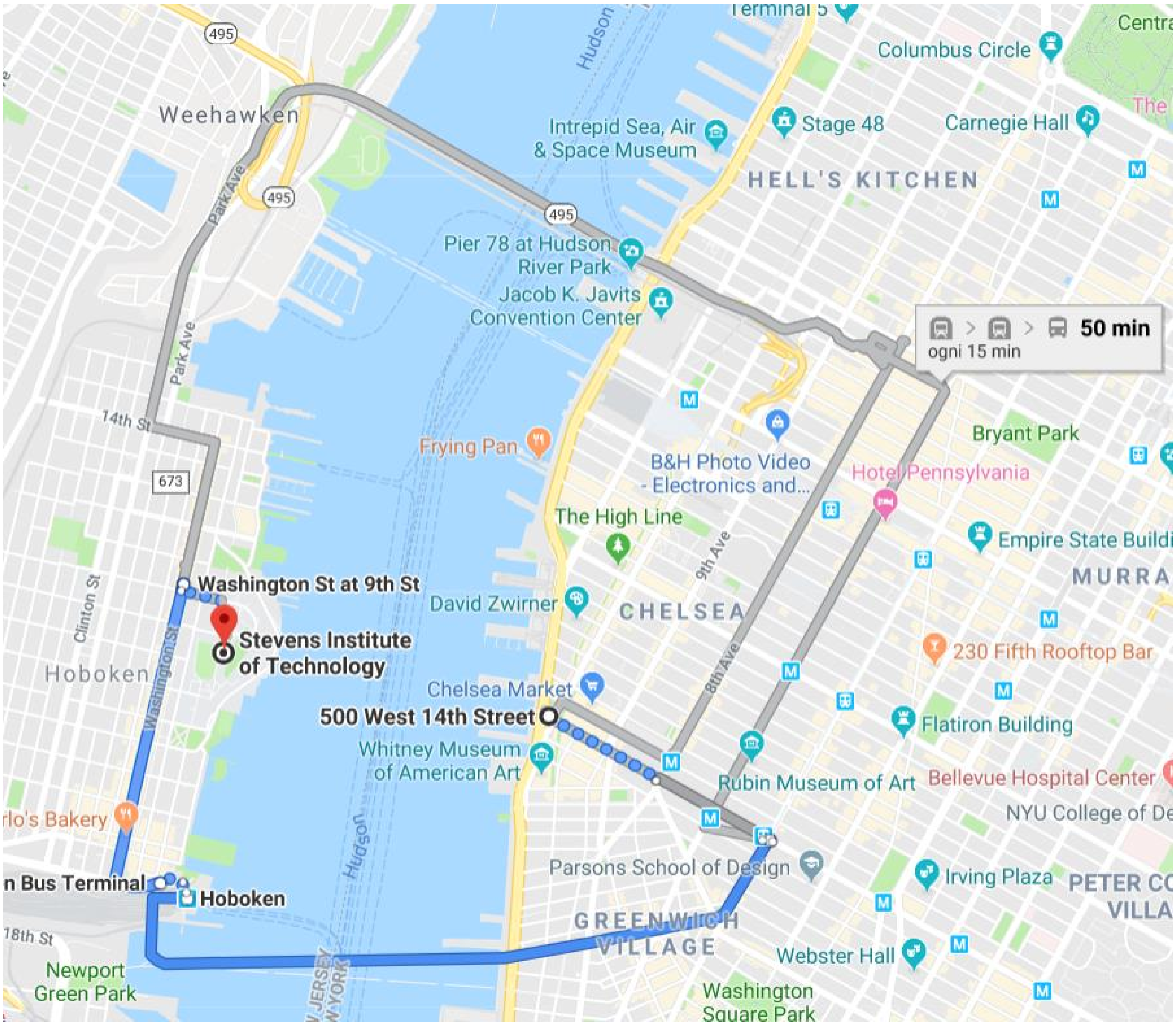

Figure 8-7: Alternative ways (Source: Google Maps)

Assumptions		
Number of users per day	10.000,00	
Bridge length	5.080,00	ft
Speed	4,00	ft*sec
Time without the Bridge	50,00	min
Average annual Hoboken salary	14,56	$*hr
Time with the Bridge	21,17	min
Ticket without the Bridge	7,00	$
Ticket with the Bridge	5,00	$
DR	7,00	%

Table 8-4: Assumption for the Economic analysis (Table made by the author, Excel

Results		
Time saved per day	57,67	min
Monetized annual travel time benefit	51.077.288,89	$
Annual tickets savings	7.300.000,00	$
Emission reduction	2.918.864,44	$
Total benefits	61.296.153,33	$
NPV(Benefit)	752.574.192,15	$
NPV(Cost)	301.887.022,00	$
BCR	2,492900116	

Table 8-5: Monetized benefit (Table made by the author, Excel)

Using the assumptions previously described the monetized annual travel time benefit was evaluated (MATTB = 365days* 58min* n of Users) and also annual ticket savings for the commuters were evaluated considering the average ticket price that commuters would spend using the subway or the ferry system (ATS = 2$ * n of users).On the other way, the emission reduction was monetized using the 5% of the total benefit, this value is suggested by the U.S. Department of Transportation Benefit-Cost Analysis Resource Guide[26] for a pedestrian bridge.

The Results of the analysis are extremely positive with a benefit PV over the 30 years of operations of $ 650,000,000 and a BCR (that also include the construction, design and operation cost) of more than 2.

Benefit-Cost Ratio:

$$BCR = \frac{\sum_{i=n}^{n} \frac{Bi}{(1+r)^n}}{\sum_{i=n}^{n} \frac{Ci}{(1+r)^n}}$$

At the numerator, there is the sum of all the benefits and the denominator the sum of all the costs, during the first 30 years of operation, discounted by the DOT rate, previously discussed. The ratio between these two quantities is usually considered positive if bigger than one[27].

[26] (Trasportation, 2015)
[27] (Martland, 2011)

It is important to underline that this result is conservative because the other benefit that were not monetized could be an addition important value for New York, as the resilience which in the last years had a tremendous impact on the urban planning of Manhattan as reported by Daniel Zarrilli, Senior Director for Climate Policy & Programs and the Chief Resilience Officer in the Mayor's Office "Through our entire $20 billion resiliency program, we are finding ways to make our investments stretch further to buy down risk for the benefit of all New Yorkers[28]"

8.5 Financial Analysis

The Financial analysis is a fundamental step of project evaluation and is a method for analyzing the monetary flows generated by an investment. It allows assessing the sustainability and the financial return through the aggregation of the balance sheet items in an integrated account plan. The objective is to establish whether the project will be able to guarantee a certain flow of revenue such as to justify the private investment. It places itself at the center of the feasibility analysis as well as providing specific indications regarding the sustainability of the intervention, it acts as a bridge between demand analysis, technical and economic feasibility analysis, monetizing and inserting in the project context all the information.

The most commonly used method for aggregating financial analysis values is the Discounted Cash Flow Method. All the actual disbursements or monetary revenues generated by the project over the project lifetime are recorded and aggregated, discounting future values, with an appropriate discount value.

This analysis model uses the information on cash flows, most of which are already available, and reorganizes them to allow the assessment of financial sustainability. The main data for the editing concern:

- Total investment flows
- Operating costs and revenues
- Sources of financing

[28] (NYC, 2017)

The hypotheses on the consistency and on the trend of these variables are formulated in the demand analysis and in the analysis of technical feasibility. The analysis of the demand determines the solvent demand, the returns believed possible, in order to formulate hypotheses based on the self-financing capacity of the project or in the case of the eventual budget imbalance.

The demand analysis, therefore, is used to determine the amount of the revenues. The technical feasibility analysis describes the technical-functional and dimensional characteristics of the investment to be realized.

The financial analysis required the estimate of the following factors:
- Monetary flows of investment
- Operation and maintenance flows
- Project lifetime

8.5.1 Return Indices

Profitability indicators represent the economic-financial-social framework of the investment are the fundamental criteria for the choice of the project. The main three used indices are Net Present Value (NPV), Benefit-Cost Ratio (BCR) and Internal Rate of Return (IRR). NPV and BCR were already introduced and used in the economic analysis and gave us a general understanding from the pubic perspective.

The choice to develop a Financial Analysis on the IRR was due to public-private partnership investment. In fact, the choice of a DR in a project of this type, for the private return, can be extremely hard to determine and in the past, we have seen tremendous mistake due to wrong evaluation of the discount rate. A significant example could be Dallas Greenway[29].

The IRR is the value of the discount rate for which the PV is null. This rate makes the investment value equal to the initial cost. The DR, in this case, is the unknown to be found in the mathematical equation. This concept is represented both graphically and numerically

[29] (Miller, 2012)

in the following page. The graph shows how the present value changes with the lower value of the discount rate, on the x-axis.

$$\sum_{t=1}^{n} \frac{F_t}{(1+r)^t} = 0; r = IRR$$

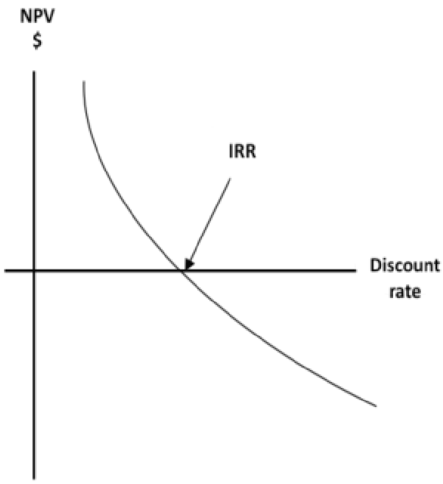

Figure 8-8: Discount Rate (Source: material of Capital Facility and Planning class, Columbia University)

The value of r obtained from the resolution of this equation is defined as the internal rate of return on the investment and represents the maximum value of the interest rate that the economic characteristics of the investment would allow satisfying in the case of loan financing.

8.5.2 Cash Flow Model

Using all the assumptions and studies performed in the previous chapter, especially the cost, and demand analysis, as reported in Table 8-6. We performed a 30 years cash flow model, a time considered acceptable in the USA for a public-private concession. All the calculations are reported in Appendix A.

The final concept is based on a project that is financed through a 30% equity provided by the main private investor or other possible private equity funds, while the rest is financed using two permanent fully amortized loans of 30 years. The first one is a public loan that covers 20% of the total investment, which is released by the Department of Transportation and the Port Authority of New York at a very advantageous rate (3,5%), the discount rate used by these two public agencies. Thanks to the economic analysis and the BCR obtained is reasonable to believe that the public would be interested to invest in the infrastructure at this rate. As a consequence, it would be possible to low the WACC and obtain a higher return on the equity invested.

The second loan is a pension fund permanent loan, pension funds are investors used to infrastructure investments and longtime return period. Moreover, they use a reasonable rate of returns (7-10%) which seem compatible with our cash flow model and financial plan.

N years	20,00
Construction Cost	250.000.000,00
Equity	30%
Design Cost	25.000.000,00
Interest Rate Pension Fund	8,0%
Interest Rate DOT	3,50%
O&M	2.500.000,00
O&M Increase	2%
Lease Payment	10.000.000,00
Toll	$ 5,00
Toll Increase factor	0,5%
Users	11.594.225,000
Users Increase factor	1,0%

Table 8-6: Cash Flow Assumptions (Table made by the author)

IRR on Project	20%
IRR on Equity	34%

Table 8-7: IRR results (Table made by the author)

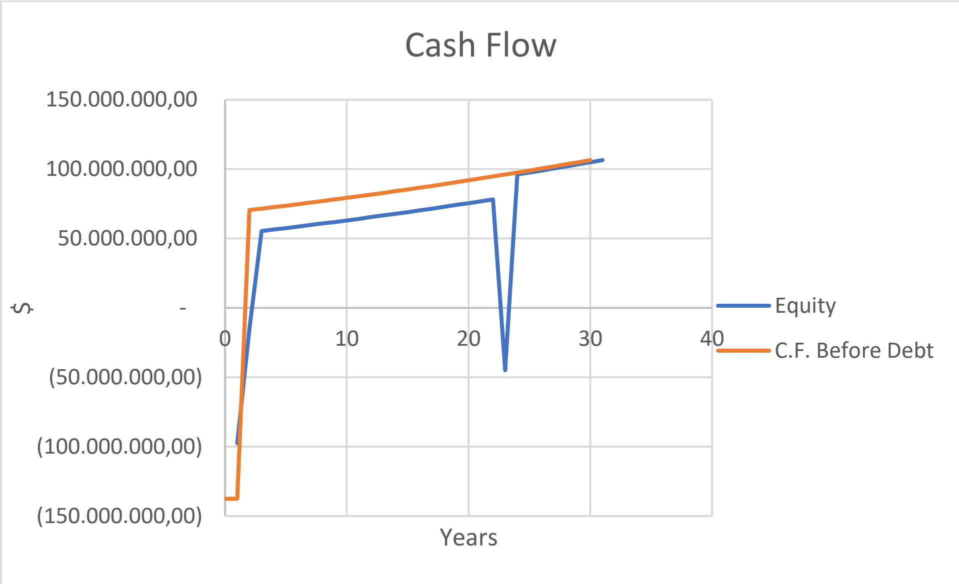

Figure 8-9: Cash Flow results (Figure made by the author, Excel)

The IRR as shown in Table 18 is evaluated both on the project and considering also the debt service, as expected, the return on the project is lower than on equity because of the presence of larger negative number at the beginning of the cash flow.

As we did for the economic analysis, the net present value on the project has been calculated to have a clear understanding that the project is more than feasible and to prove that all the initial investment, especially the equity, can be returned with the appropriate interest. Differently, from the economic analysis, where the federal government is providing a certain discount rate, the choice of the DR can be extremely difficult. Because dependent on micro and macroeconomic factors of the project and of the New York and New Jersey region. The weighted average cost of the capital model is used to find a possible discount rate to be used in the cash flow. In fact, the weighted average cost of capital allows a company or an investor to establish the cost of capital by analyzing all its components and therefore allows discriminating between an acceptable or less than expected return on investment.

$$WACC = k_E\left(\frac{E}{D+E}\right) + k_D(1-t)(\frac{D}{D+E})$$

Where:

- k_E = Cost of equity
- k_D = Debt cost
- t = Corporate tax rate
- D = Level of debt
- E = Level of equity

The tricky component is to evaluate the cost of equity that differently from the debt component has no direct and fixed cost. It depends on the risk associated with the investment and on micro and macroeconomic components related to the project. To overcome this problem, we decided to use a different approach and study the DR used by a similar project in the past[30]. In most of the case, studies read the cost of equity for the project in an urban area is between 15% and 20%. To continue to be on the safe side we preferred to assume 20%.

WACC	7%
k_E	20%
k_D	7%
E	82.500.000,00
D	192.500.000,00
t	20%

Table 8-8: WACC (Table made by the author, Excel)

In Table 8-8 are reported all the assumptions and the final result, a WACC of 7% is an acceptable result and in line with the percentage used in the construction industry as reported by the New York University[31]. This result is then used as the DR for our model and the NPV obtained is more than 400 million as reported in Appendix B- Cash Flow.

Both the results are extremely positive and such a high return on equity would allow the developer not only to have a very high margin but also to attract private capital such as hedge fund or private equity fund. Moreover, it is important to underline that in the cash flow we didn't consider the residual value of the bridge after 30 years that can be assumed to be 60% of the initial bridge cost, based on the fact that the service life is 70 years.

8.6 Sensitivity analysis

Sensitivity analysis helps to understand how sensible is the model that we have built to the different variables. Through this process, we want to reduce the uncertainty when it is possible, or at least takes it into account. In fact, due to all the assumptions that we have done during the project evaluations the level of uncertainty in the model is high and many different scenarios are possible.

[30] (Miller, 2012)
[31] (Damodaran, 2019)

In the following table, the main variables were summarized, and a range of value is chosen.

Variable	***Base Price***	***Value for Sensitivity Analysis***
Ticket Price Percentage	5$	-40%,-30%,20%,10%,0%,10%,20%30%,40%
Construction Time	2 years	2,3,4,5
Capital Cost	7%	4%,5%,7%,8%,10%,12%,14%
Traffic Volume	30,000	15,000 20,000 25,000 30,000 35,000

Table 8-9: Sensitivity Analysis Variables (Table made by the author, Excel)

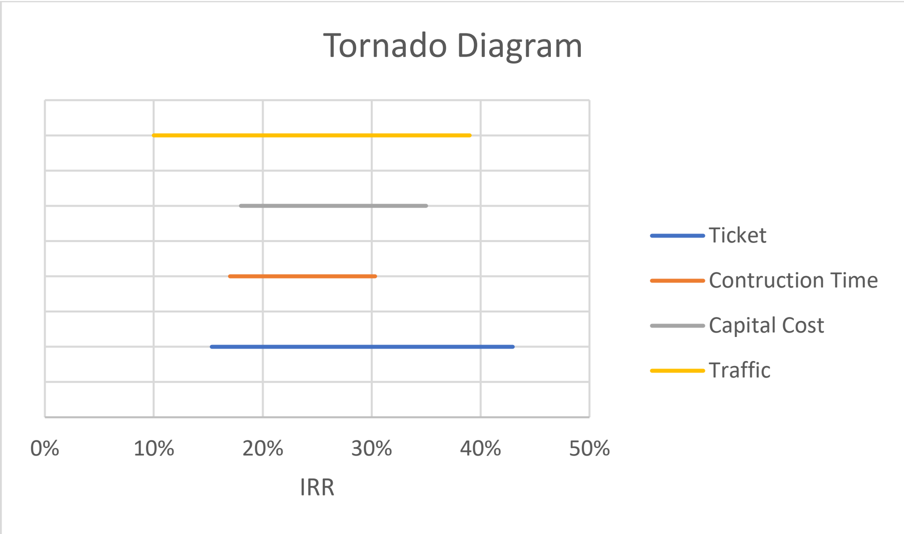

Figure 8-10: Tornado Diagram (Figure made by the author, Excel)

To understand how each variable influences the model, we plotted the IRR on a tornado diagram. Which highlights how each variable influences the return on the project, in fact, longer is the line, more influence has the variable on the model.

It is important no notice that traffic, volume, and tickets have the greatest influence on the internal rate of return on the project, in fact, a traffic volume of 10,000 and a reduction of the ticket price of 40% results in a deeply decrease on the project return. Tornado diagram is useful to have a preliminary understanding, so in the following tables, it is possible to look in the specific at each variable.

Ticket price variation	
Percentage	IRR
-40%	15%
-30%	19%
-20%	23%
-10%	27%
0%	30%
10%	34%
20%	37%
30%	40%
40%	43%

Construction Time	
Years	IRR
2	30%
3	24%
4	20%
5	17%

Capital Cost	
Interst	IRR
4%	35%
5%	32%
7%	30%
8%	28%
10%	26%
12%	23%
15%	18%

Traffic Volume	
Daily Volume	IRR
15.000,00	10%
20.000,00	17%
25.000,00	25%
30.000,00	30%
35.000,00	35%
40.000,00	39%

Figure 8-11: Sensitivity Analysis Results (Figure made by the author, Excel)

The cash flow that we have already modeled is considered the most probable, for certain variables also very conservative, and the return obtained is are already satisfactory, especially from an equity point of view. For this reason, an optimistic scenario analysis was not performed.

On the other hand, the four variables shown in Figure 77 were combined to understand the behavior of the investment in a pessimistic scenario.

Worst Case Scenario	
Ticket price variation	-20%
Construction Time	4
Capital Cost	10%
Traffic Volume	15.000,00

Table 8-10: Worst Case Scenario Variables (Table made by the author, Excel)

IRR on Project	6%
IRR on Equity	0%

Table 8-11: Worst Case Scenario Results (Table made by the author, Excel)

The worst-case scenario results show that the model is robust and even with a dramatic reduction of the four variables there would be almost no loss in the investment.

A sensitivity analysis on the discount rate is still missing but to do it we have to change the reference variable, so from the IRR, we move to the NPV. This test on the DR is extremely important because for the reasons explained in the previous page its evaluation could be tricky and can lead to huge mistakes and to completely wrong project evaluation.

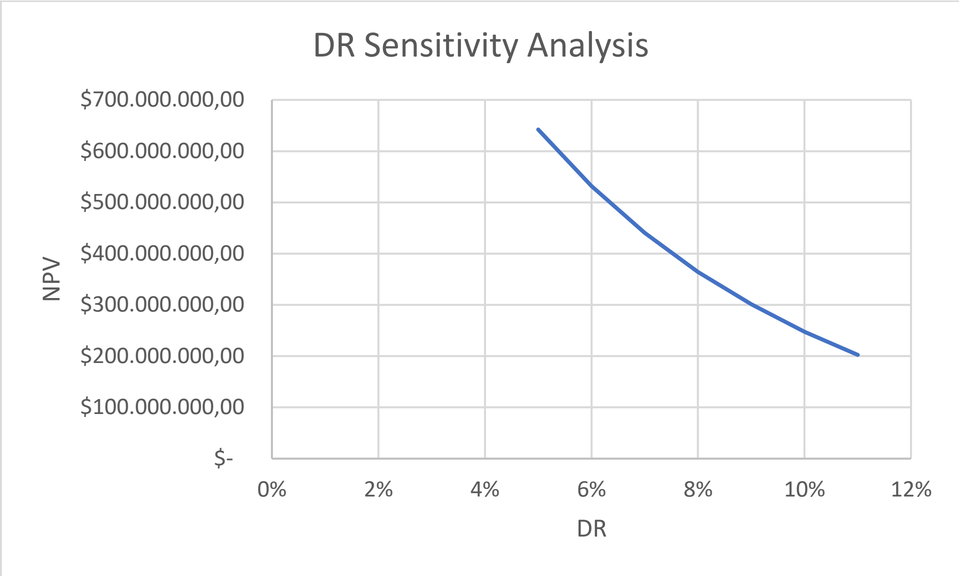

Figure 8-12: DR Sensitivity Analysis (Image made by the author, Excel)

On the graph above it is shown the NPV variation on the y-axis and how it changes with a different value of the DR on the x-axis. From this image, it is possible to understand that even if the NPV decreases very fast with higher DR we have a great margin of error in the evaluation of the variable, more than 60%. In fact, also with a discount rate of 11%, that for this type of project and the way it is financed it is really high, the present value is 200 Million that would be sufficient to justify the investment.

8.7 **Public-Private Partnership Development**

Public-private partnerships are a collaboration between private and public entities that can be set at different levels and with different duties. In general, we can say that it always involves the funding and operation of projects, but this can happen in various ways and with different assets: Brownfield and Greenfield.

A similar approach, with different financial strategies, was already used for the Old London Bridge in 1300, an ancestor of the actual form of Public-Private-Partnership. Nowadays it is broadly applied: the Indiana Tool Road was built using a PPP and probably without this partnership, it couldn't have been completed because financial cooperation between private and the public was fundamental in this DBFO project. The other important example that we have seen was the Golden Gate Bridge that can be considered as DBB financed by revenue bond taxes.

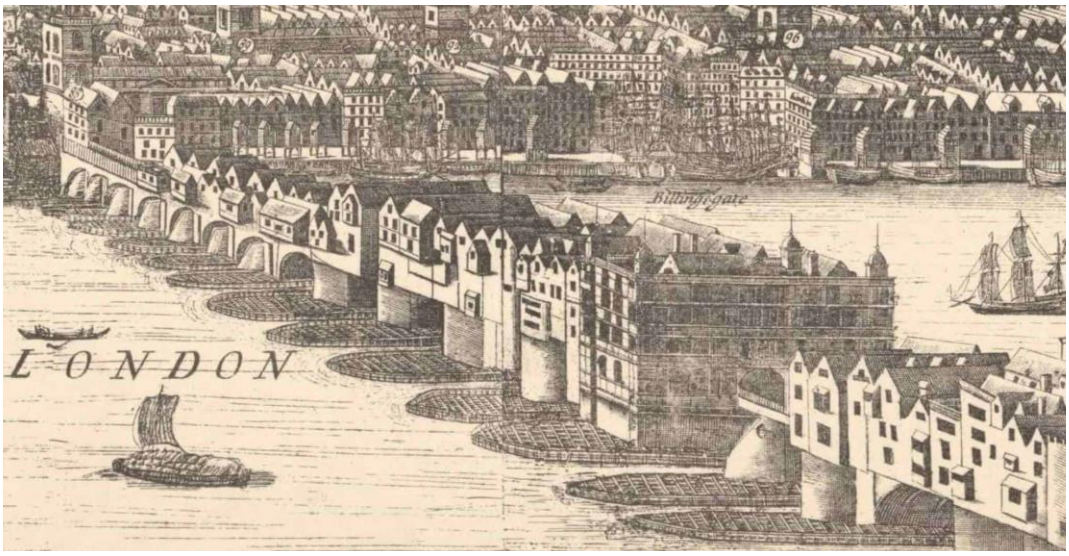

Figure 8-13: Old London Bridge

In fact, if performed in the right way the PPP can be an advantage for both the public and private sector because there is a sharing of investments, risks, and benefits even if their aims are different. For the public splitting the investment means, in some cases to be less exposed and be able to deal with a larger project with the same amount of money, while in others to give all the financial responsibilities to the company that developed the project, as happened in JFK, and partially also in Hoboken Bridge. Secondly, being involved with the private can make the procedure, operation and maintenance of the structure more flexible, even if we don't have to forget that the public main goal is to satisfy the citizen's needs.

On the other hand, the private can make an investment that usually provides stable cash flow and the risk of the project can be considered lower than private investment.
Moreover, the lower risk of the project has multiple advantages and repercussion on bond and possible investors. Usually, PPP in infrastructure is considered attractive for long term investment as pensions or insurances.

To conclude, the PPP if performed in the right way and with the correct upfront evaluations, concerning term of concessions, environment, tools increase, can bring great advantages to both the players even if the goals of the two are partially different and in certain case opposite. We must remember that each case is unique and has to be analyzed as such.

Even if differently from Old London Bridge, Hoboken Bridge could be the perfect candidate for a Public-Private investment, both the economic and the financial analysis showed positive results for investors and citizens. This partnership would allow to a potential “Win-Win”, the private would reach easier a steady annual cash flow, thanks to the advantageous pubic loan and thanks to what this loan would appear to other private investors, a sort of certificate of quality given by institutional investors. This would result in a significant reduction in capital cost. Differently, the public would have greater flexibility with all the procedures and from a financial perspective, it would be exposed only with the loan at 3,5% rate.

It is important to underline that a 3,5% rate loan for a greenfield investment is low, however, given the NPV of the benefit provided to the community by the bridge this loan is more than justified. Moreover, from the analysis, no specific cost for the citizens appeared.

9 Appendices

9.1 Appendix A – Preliminary Structural Design

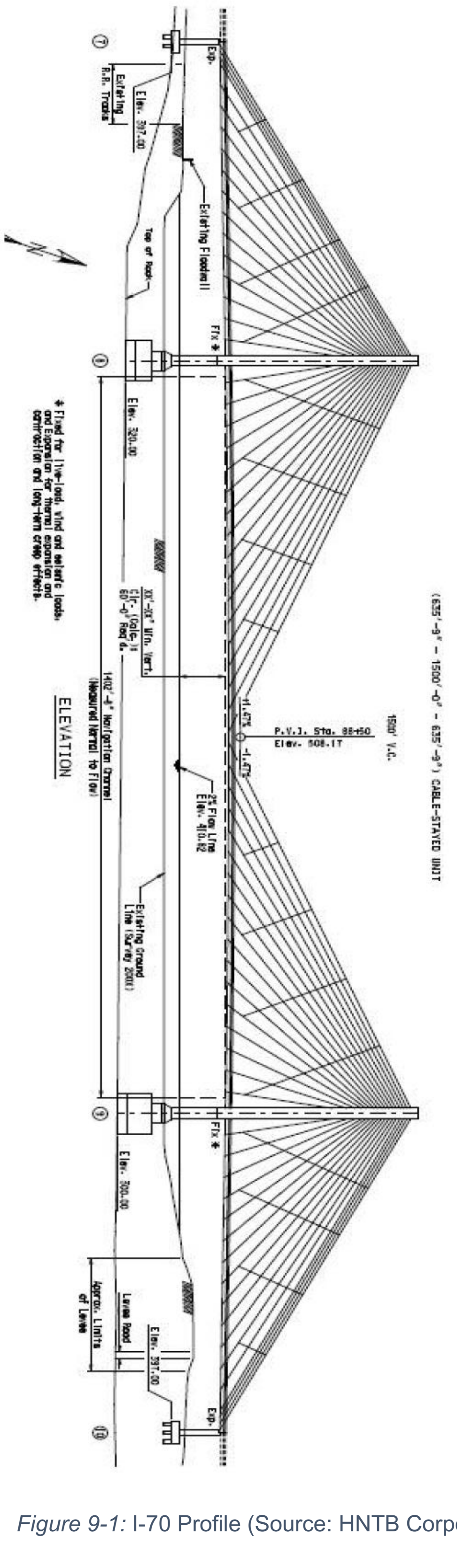

Figure 9-1: I-70 Profile (Source: HNTB Corporation)

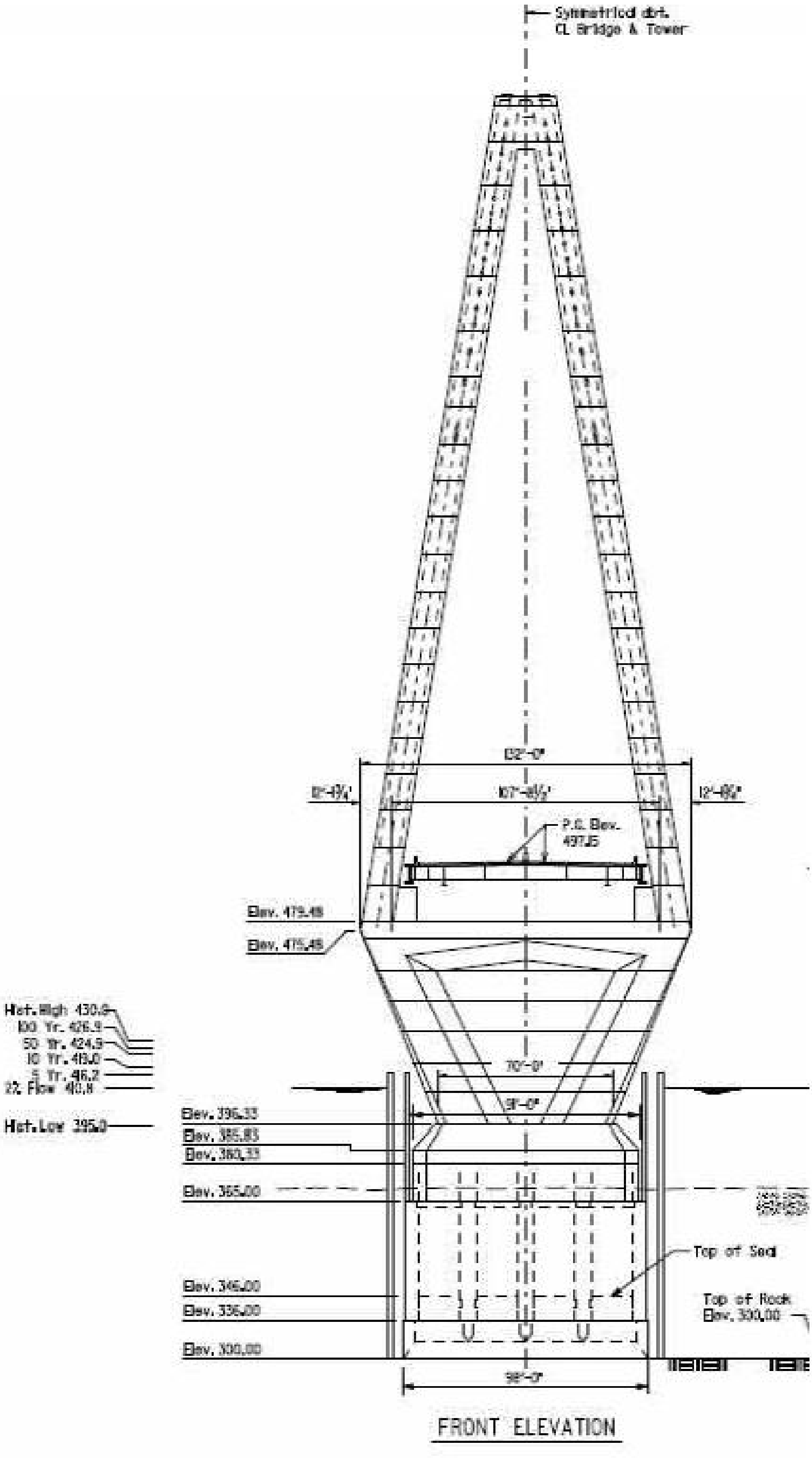

Figure 9-2: I-70 Front Elevation (Source: HNTB corporation)

Span 1 Loads Distribution

I-70						
B[ft]	D[ft]	X[ft]	p(y)[lbf/ft2]	p(x)[lbf/ft2]	p(z)[lbf/ft2]	(mx)[lbf/ft2]
24,00	5	-	2,13	2,48	-18,40	-0,14
24,00	5	85,75	2,51	1,449	-18,342	-0,10
24,00	5	170,25	3,16	0,905	-18,36	-0,10
24,00	5	254,75	3,05	1,279	-18,551	-0,07
24,00	5	339,25	2,34	1,635	-18,857	-0,09
24	5	423,75	1,67	1,86	-19,334	-0,111168
24	5	508,25	1,53	2,017	-20,093	-0,134784
24	5	592,75	1,88	2,135	-21,282	-0,151488
24	5	635,00	2,19	2,211	-22,12	-0,146304
24	5	722,00	2,72	2,573	-26,216	-0,13824
24	5	809,00	2,71	2,976	-30,984	-0,153792
24	5	896,00	2,21	3,403	-36,401	-0,169344
24	5	983,00	1,16	3,546	-42,241	-0,235008
24	5	1.055,73	1,47	3,165	-48,485	-0,441792
24	5	1.135,85	10,16	2,408	-54,837	-0,835776
24	5	1.215,97	17,54	2,12	-59,765	-1,304064
24	5	1.296,08	22,88	2,623	-62,131	-1,430784
24	5	1.376,20	24,06	3,8	-63,59	-1,469376
24	5	1.456,32	23,06	5,008	-62,124	-1,434816
24	5	1.536,43	17,94	5,543	-63,59	-1,314432

Hoboken Bridge				
X[ft]	p(x)[kip/ft]	p(y)[kip/ft]	p(z)[kip/ft]	p(mx)[kip]
0	0,01242	-0,091995	0,051168	-0,139968
50	0,007245	-0,09171	0,060216	-0,104832
100	0,007245	-0,09171	0,060216	-0,104832
150	0,004525	-0,0918	0,075888	-0,097344
200	0,004525	-0,0918	0,075888	-0,097344
250	0,006395	-0,092755	0,073248	-0,069696
300	0,008175	-0,094285	0,056208	-0,088128
350	0,008175	-0,094285	0,056208	-0,088128
400	0,0093	-0,09667	0,040128	-0,111168
450	0,0093	-0,09667	0,040128	-0,111168
500	0,010085	-0,100465	0,0366	-0,134784
550	0,010085	-0,100465	0,0366	-0,134784
600	0,010675	-0,10641	0,045072	-0,151488
650	0,011055	-0,1106	0,052632	-0,146304
700	0,012865	-0,13108	0,065304	-0,13824
750	0,012865	-0,13108	0,065304	-0,13824
800	0,01488	-0,15492	0,065136	-0,153792
850	0,01488	-0,15492	0,065136	-0,153792
900	0,017015	-0,182005	0,053112	-0,169344
950	0,01773	-0,211205	0,027936	-0,235008
1000	0,01773	-0,211205	0,027936	-0,235008
1050	0,015825	-0,242425	0,035256	-0,441792
1100	0,015825	-0,242425	0,035256	-0,441792
1150	0,01204	-0,274185	0,243888	-0,835776
1200	0,0106	-0,298825	0,421008	-1,304064
1250	0,0106	-0,298825	0,421008	-1,304064
1300	0,013115	-0,310655	0,54912	-1,430784
1350	0,013115	-0,310655	0,54912	-1,430784
1400	0,019	-0,31795	0,577392	-1,469376
1450	0,02504	-0,31062	0,553536	-1,434816
1500	0,027715	-0,31795	0,430464	-1,314432
1550	0,027715	-0,31795	0,430464	-1,314432

Figure 9-3: Load distribution along span 1(graph made by the author, Excel)

Span 3 Loads Distribution

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B[ft]	D[ft]	X[ft]	p(y)[lbf/ft2]	p(x)[lbf/ft2]	p(z)[lbf/ft2]	p(mx)[lbf/ft2]
24	5	1244	17,541	2,12	-59,765	-2,264
24	5	1331	22,88	2,6	-62,13	-2,484
24	5	1385	24,064	3,8	-63,6	-2,551
24	5	1439	23,064	5	-62,12	-2,491
24	5	1526	17,936	5,54	-59,75	-2,282
24	5	1613	10,584	5,28	-54,8	-1,471
24	5	1700	4,766	4,51	-48,405	-0,783
24	5	1787	2,306	4,1	-42,18	-0,416
24	5	1874	2,256	4,2	-36,35	-0,297
24	5	1961	2,726	4,62	-30,96	-0,267
24	5	1048	2,729	5,01	-26,23	-0,24
24	5	2135	2,194	5,36	-22,19	-0,254
24	5	2177	1,87	5,42	-21,36	-0,263
24	5	2261	1,508	5,51	-20,18	-0,233
24	5	2346	1,673	5,64	-19,428	-0,191
24	5	2430	2,402	5,849	-18,95	-0,159
24	5	2525	3,18	6,19	-18,631	-0,129
24	5	2599	3,314	6,552	-18,419	-0,173
24	5	2684	2,586	5,956	-18,36	-0,186
24	5	2767	2,134	4,836	-18,4	-0,241

Hoboken Bridge

X[ft]	p(x)[kip/ft]	p(y)[kip/ft]	p(z)[kip/ft]	p(mx)[kip]
0	0,0106	0,420984	-0,298825	-1,30406
50	0,013	0,54912	-0,31065	-1,43078
100	0,013	0,54912	-0,31065	-1,43078
150	0,019	0,577536	-0,318	-1,46938
200	0,019	0,577536	-0,318	-1,46938
250	0,025	0,553536	-0,3106	-1,43482
300	0,0277	0,430464	-0,29875	-1,31443
350	0,0277	0,430464	-0,29875	-1,31443
400	0,0264	0,254016	-0,274	-0,8473
450	0,0264	0,254016	-0,274	-0,8473
500	0,02255	0,114384	-0,242025	-0,45101
550	0,02255	0,114384	-0,242025	-0,45101
600	0,0205	0,055344	-0,2109	-0,23962
650	0,021	0,054144	-0,18175	-0,17107
700	0,0231	0,065424	-0,1548	-0,15379
750	0,0231	0,065424	-0,1548	-0,15379
800	0,02505	0,065496	-0,13115	-0,13824
850	0,02505	0,065496	-0,13115	-0,13824
900	0,0268	0,052656	-0,11095	-0,1463
950	0,0271	0,04488	-0,1068	-0,15149
1000	0,0271	0,04488	-0,1068	-0,15149
1050	0,02755	0,036192	-0,1009	-0,13421
1100	0,02755	0,036192	-0,1009	-0,13421
1150	0,0282	0,040152	-0,09714	-0,11002
1200	0,029245	0,057648	-0,09475	-0,09158
1250	0,029245	0,057648	-0,09475	-0,09158
1300	0,03095	0,07632	-0,093155	-0,0743
1350	0,03095	0,07632	-0,093155	-0,0743
1400	0,03276	0,079536	-0,092095	-0,09965
1450	0,02978	0,062064	-0,0918	-0,10714
1500	0,02418	0,051216	-0,092	-0,13882
1550	0,02418	0,051216	-0,092	-0,13882

Figure 9-4: Load distribution along span 3 (Figure made by the author, Excel)

First model Longitudinal Girders				
Dimensions	W36x231			
fyd	47,6	ksi		
W3	0,56	ft3	968	in3
W2	0,1	ft3	172	in3
Mr3	30.076,80	kip*ft		
Mr2	8.187,20	kip*ft		
Ma3	1.200,00	kip*ft		
Ma2	58,73	kip*ft		

Second model Longitudinal Girders				
Dimesions	W21x147			
fyd	47,6	ksi		
W3	0,21	ft3	362,88	in3
W2	0,05	ft3	86,4	in3
Mr3	12.273,09	kip*ft		
Mr2	4.112,64	kip*ft		
Ma3	1.500,00	kip*ft		
Ma2	70,00	kip*ft		

First model Transversal Girders				
Dimensions	W30x124			
fyd	47,6	ksi		
W3	0,24	ft3	414	in3
W2	0,03	ft3	51	in3
Mr3	15.706,40	kip*ft		
Mr2	2427,6	kip*ft		
Ma3	472,1044	kip*ft		
Ma2	40	kip*ft		

Second model Transversal Girders				
Dimensions	W21x93			
fyd	47,60	ksi		
W3	0,12	ft3	207,36	in3
W2	0,02	ft3	34,56	in3
Mr3	7.870,34	kip*ft		
Mr2	1.645,06	kip*ft		
Ma3	500,00	kip*ft		
Ma2	50	kip*ft		

Table 9-1: Moment Assessment (Tables made by the author, Excel)

9.2 Appendix B- Cash Flow

				Gross	Net Revenues					
Year	Purchase Price	Equity	Toll	Gross Toll Revenue	Users	O&M	Lease Payment	Lease Reimb.	Residual Value	Net Operating Income afeter O&M
0	55.000.000,00	82.500.000,00								
1	137.500.000,00									
2	0		5,00	57.971.125,00	11.594.225,00	2.500.000,00	10.000.000,00	10.000.000,00		55.471.125,00
3	0		5,03	58.843.590,43	11.710.167,25	2.550.000,00	10.000.000,00	10.000.000,00		56.293.590,43
4			5,05	59.729.186,47	11.827.268,92	2.601.000,00	10.000.000,00	10.000.000,00		57.128.186,47
5			5,08	60.628.110,72	11.945.541,61	2.653.020,00	10.000.000,00	10.000.000,00		57.975.090,72
6			5,10	61.540.563,79	12.064.997,03	2.706.080,40	10.000.000,00	10.000.000,00		58.834.483,39
7			5,13	62.466.749,28	12.185.647,00	2.760.202,01	10.000.000,00	10.000.000,00		59.706.547,27
8			5,15	63.406.873,85	12.307.503,47	2.815.406,05	10.000.000,00	10.000.000,00		60.591.467,80
9			5,18	64.361.147,30	12.430.578,50	2.871.714,17	10.000.000,00	10.000.000,00		61.489.433,13
10			5,20	65.329.782,57	12.554.884,29	2.929.148,45	10.000.000,00	10.000.000,00		62.400.634,12
11			5,23	66.312.995,80	12.680.433,13	2.987.731,42	10.000.000,00	10.000.000,00		63.325.264,38
12			5,26	67.311.006,38	12.807.237,46	3.047.486,05	10.000.000,00	10.000.000,00		64.263.520,33
13			5,28	68.324.037,03	12.935.309,84	3.108.435,77	10.000.000,00	10.000.000,00		65.215.601,26
14			5,31	69.352.313,79	13.064.662,93	3.170.604,49	10.000.000,00	10.000.000,00		66.181.709,30
15			5,33	70.396.066,11	13.195.309,56	3.234.016,58	10.000.000,00	10.000.000,00		67.162.049,53
16			5,36	71.455.526,91	13.327.262,66	3.298.696,91	10.000.000,00	10.000.000,00		68.156.830,00
17			5,39	72.530.932,59	13.460.535,29	3.364.670,85	10.000.000,00	10.000.000,00		69.166.261,74
18			5,42	73.622.523,12	13.595.140,64	3.431.964,26	10.000.000,00	10.000.000,00		70.190.558,86
19			5,44	74.730.542,09	13.731.092,05	3.500.603,55	10.000.000,00	10.000.000,00		71.229.938,55
20			5,47	75.855.236,75	13.868.402,97	3.570.615,62	10.000.000,00	10.000.000,00		72.284.621,13
21			5,50	76.996.858,07	14.007.087,00	3.642.027,93	10.000.000,00	10.000.000,00		73.354.830,13
22			5,52	78.155.660,78	14.147.157,87	3.714.868,49	10.000.000,00	10.000.000,00		74.440.792,29
23			5,55	79.331.903,47	14.288.629,44	3.789.165,86	10.000.000,00	10.000.000,00		75.542.737,61
24			5,58	80.525.848,62	14.431.515,74	3.864.949,18	10.000.000,00	10.000.000,00		76.660.899,44
25			5,61	81.737.762,64	14.575.830,90	3.942.248,16	10.000.000,00	10.000.000,00		77.795.514,48
26			5,64	82.967.915,97	14.721.589,21	4.021.093,12	10.000.000,00	10.000.000,00		78.946.822,85
27			5,66	84.216.583,11	14.868.805,10	4.101.514,99	10.000.000,00	10.000.000,00		80.115.068,12
28			5,69	85.484.042,68	15.017.493,15	4.183.545,29	10.000.000,00	10.000.000,00		81.300.497,40
29			5,72	86.770.577,52	15.167.668,08	4.267.216,19	10.000.000,00	10.000.000,00		82.503.361,33
30			5,75	88.076.474,72	15.319.344,76	4.352.560,52	10.000.000,00	10.000.000,00		83.723.914,20

Figure 9-5: Cash Flow (Figure made by the author, Excel)

Debt Service					
Annual Payment DOT	Annual Payment PF	Total Debt Cost	DSCR	Net Annual C.F. Before Debt	Equity
				-137.500.000,00	-82.500.000,00
2.909.770,64	12.014.442,92	14.924.213,56		-137.500.000,00	-14.924.213,56
2.909.770,64	12.014.442,92	14.924.213,56	3,72	55.471.125,00	40.546.911,44
2.909.770,64	12.014.442,92	14.924.213,56	3,77	56.293.590,43	41.369.376,88
2.909.770,64	12.014.442,92	14.924.213,56	3,83	57.128.186,47	42.203.972,91
2.909.770,64	12.014.442,92	14.924.213,56	3,88	57.975.090,72	43.050.877,17
2.909.770,64	12.014.442,92	14.924.213,56	3,94	58.834.483,39	43.910.269,83
2.909.770,64	12.014.442,92	14.924.213,56	4,00	59.706.547,27	44.782.333,71
2.909.770,64	12.014.442,92	14.924.213,56	4,06	60.591.467,80	45.667.254,25
2.909.770,64	12.014.442,92	14.924.213,56	4,12	61.489.433,13	46.565.219,58
2.909.770,64	12.014.442,92	14.924.213,56	4,18	62.400.634,12	47.476.420,56
2.909.770,64	12.014.442,92	14.924.213,56	4,24	63.325.264,38	48.401.050,82
2.909.770,64	12.014.442,92	14.924.213,56	4,31	64.263.520,33	49.339.306,78
2.909.770,64	12.014.442,92	14.924.213,56	4,37	65.215.601,26	50.291.387,70
2.909.770,64	12.014.442,92	14.924.213,56	4,43	66.181.709,30	51.257.495,75
2.909.770,64	12.014.442,92	14.924.213,56	4,50	67.162.049,53	52.237.835,98
2.909.770,64	12.014.442,92	14.924.213,56	4,57	68.156.830,00	53.232.616,44
2.909.770,64	12.014.442,92	14.924.213,56	4,63	69.166.261,74	54.242.048,18
2.909.770,64	12.014.442,92	14.924.213,56	4,70	70.190.558,86	55.266.345,30
2.909.770,64	12.014.442,92	14.924.213,56	4,77	71.229.938,55	56.305.724,99
2.909.770,64	12.014.442,92	14.924.213,56	4,84	72.284.621,13	57.360.407,58
2.909.770,64	12.014.442,92	14.924.213,56	4,92	73.354.830,13	58.430.616,58
2.909.770,64	12.014.442,92	14.924.213,56	4,99	74.440.792,29	59.516.578,73
2.909.770,64	12.014.442,92	14.924.213,56	5,06	75.542.737,61	60.618.524,06
2.909.770,64	12.014.442,92	14.924.213,56	5,14	76.660.899,44	61.736.685,89
2.909.770,64	12.014.442,92	14.924.213,56	5,21	77.795.514,48	62.871.300,93
2.909.770,64	12.014.442,92	14.924.213,56	5,29	78.946.822,85	64.022.609,29
2.909.770,64	12.014.442,92	14.924.213,56	5,37	80.115.068,12	65.190.854,56
2.909.770,64	12.014.442,92	14.924.213,56	5,45	81.300.497,40	66.376.283,84
2.909.770,64	12.014.442,92	14.924.213,56	5,53	82.503.361,33	67.579.147,78
2.909.770,64	12.014.442,92	14.924.213,56	5,61	83.723.914,20	68.799.700,64

Figure 9-6: Cash Flow (Figure made by the author, Excel)

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