DESIGN OF LONG SPAN MODULAR BRIDGES FOR TRAFFIC DETOURS

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Submitted to the Department of Civil and Environmental Engineering
In Partial Fulfillment of the Requirements for the Degree of

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The oncoming large amount of bridge replacements in the next 10 to 20 years called for a detailed examination of available replacement schemes which can have variable impact on user costs. Detouring traffic with a modular bridge proved to be the most desirable scheme in terms of user costs such as traffic delays, detour distances, ultimate highway geometrics, construction crew safety, and safety of drivers.

Criteria that encompassed modular bridge design were defined and two companies in North America – Acrow and Mabey – were found to provide bridges within those parameters. A brief analysis of Acrow bridges showed that maximum span lengths range in the order of 100m; this is fairly short compared to spans of many bridges that will be have to soon be replaced.

The current bridge system with which modularity is achieved is a set of truss panels which are supported by abutments or piers. In order to span crossing over 100m, piers would have to be placed in the channel or on the head-slopes which is a costly and undesirable construction process. Therefore, a modular bridge which could achieve longer spans was proposed for a 2 lane and a 3 lane wide bridges using as many existing Acrow components as possible.

The scheme encompasses a harp cable-stay bridge with cables spaced and sized such that they are fully interchangeable between the various bridge widths and can be built up to any span. These cables are the only additional component as the towers, the girders, and the decks are all made out of existing Acrow components. The pylon is balanced with an anchoring cable and ideas for modular foundations for the anchor are presented. A span of approximately 183m is possible for a 3 lane bridge limited by the maximum axial tower capacity and 250m for the 2 lane bridge based on lateral vibrations. The design fully reflects modularity and should promote the use of modular bridges for longer span crossings.

Thesis supervisor: Jerome J. Connor

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INTRODUCTION

North America, particularly the United States has a vast transportation network which drastically facilitates the movement of goods and people within its boundaries. A key component of this network is the Eisenhower Interstate System. The Eisenhower Interstate System originated in 1956 when President Eisenhower signed the Federal-Aid Highway Act of 1956; it is valued as the greatest public works project in history. Figure 1 shows the proposed interstate layout.

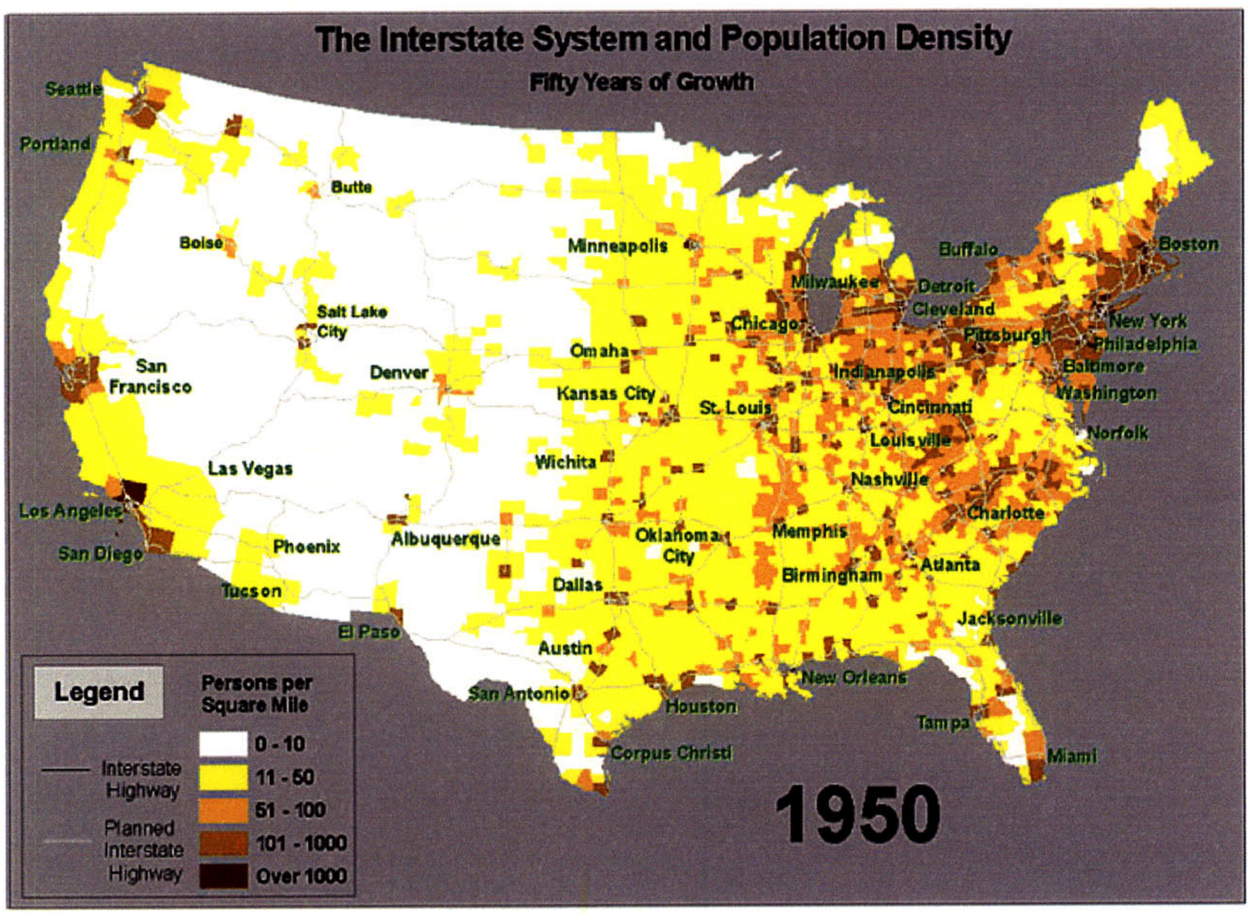

FIGURE 1: PROPOSED INTERSTATE LAYOUT IN 1956 [WWW.FHWA.DOT.GOV]

The speed at which the project was getting completed is noteworthy. Figure 2 displays how much of the Interstate System was completed by 1970. [6]

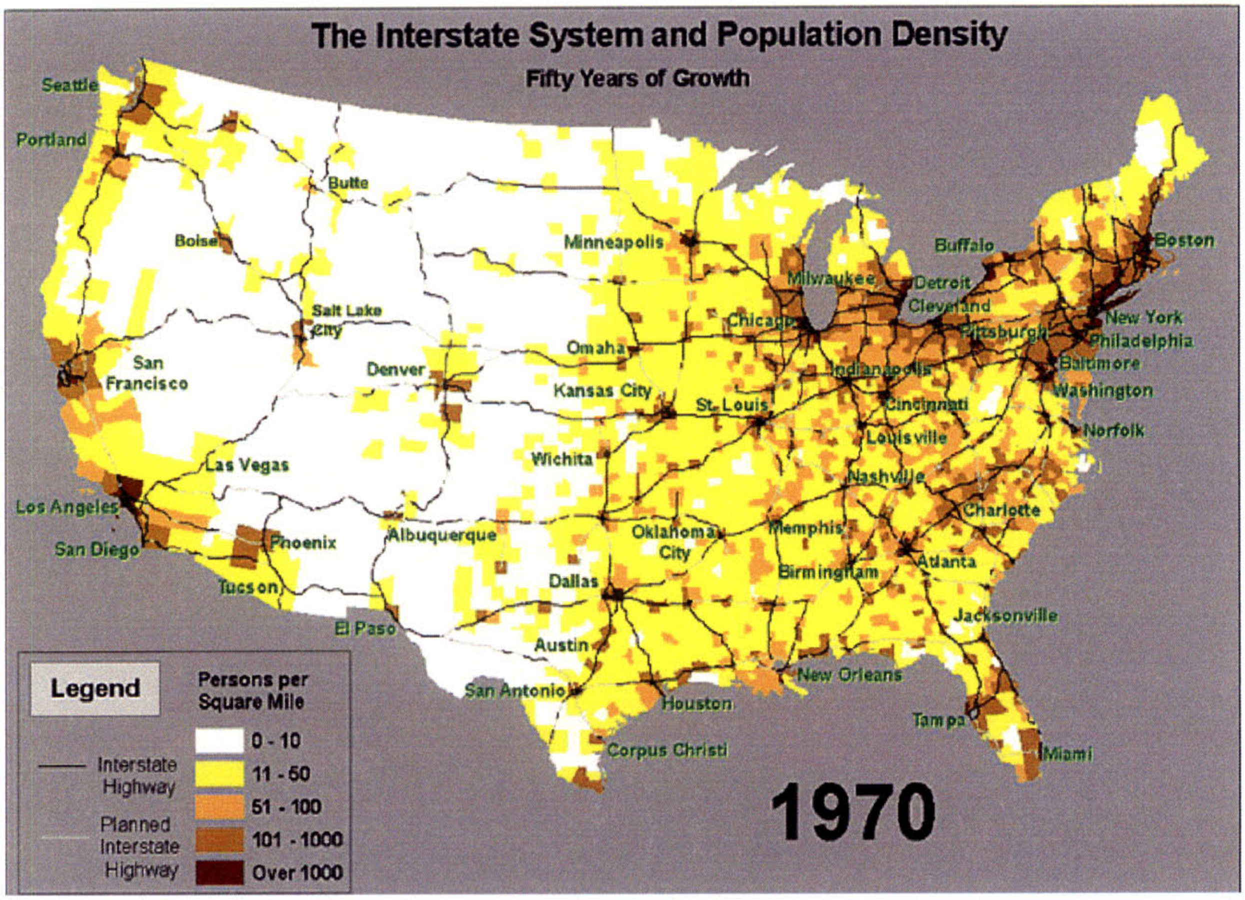

Figure 2: Completed Interstates by 1970 [www.fhwa.dot.gov]

Of course, no transportation network can exist without bridges; these allow for crossings over water, other highways, or other transportation networks such as railroads. Bridges are the most expensive and complex features of a road and as they are structures that are exposed to many natural and human induced loads (wind, flood, ice, vehicular), they present a maintenance challenge to transportation network authorities. The number of bridges just under the federal highway administration is nearing 600,000 and a large portion of those bridges was constructed as part of the Eisenhower Interstate System sometime in between 1950 – 1970 [9].

As with any structure, a bridge has a lifecycle or a design life. In the 1950's, a typical bridge design life was approximately 50 years – this was limited by issues like technology of the day and availability of durable materials. When a structure like a bridge approaches the end of its design life, it usually has major rehabilitation done to it, or it is fully replaced. A full replacement is more typical as the older bridges not only suffer structurally, they are also deficient functionally (for example, they are not big enough to carry all the traffic, or the lanes are not wide enough for newer wider trucks).

Since the majority of the bridges were built approximately 40 - 50 years ago, and their lifespan is approximately 50 years, this means that a large number of bridges are due for replacement at once. Aside from the obvious issue this poses on the budgets of the federal and local authorities, this also presents a challenge to those who are concerned with minimizing the user costs.

To elaborate more on the second point of user costs: users in this case are the public who cross the bridge. The cost to them of having a bridge replaced is the traffic delays associated with replacement. User cost can also include a reduction in safety due to proximity of construction or suboptimal driving conditions from having only one narrow lane open on the bridge. Since bridge replacements take 1 – 2 years, these user costs can be quite significant. Another user cost that is less apparent is the negative impact bridge replacements have on environment, which ultimately negatively affects the user. Of course, these user costs are much harder to quantify when it comes to deciding which strategy to use for a bridge replacement, but nonetheless they are taken seriously by project sponsors.

Given that there will be a significant amount of bridge replacements done in the near 10 – 20 years, this thesis examines various bridge replacement construction strategies, and how they impact user costs. It then further explores an optimal bridge replacement scheme, in particular the idea of a modular traffic detour bridge. A specific modular bridge design is then presented for long-span bridges, as it currently does not exist in the North American industry.

1.0 BRIDGE REPLACEMENT STRATEGIES

There are many bridge replacement schemes that exist and bridge construction can be very site specific. However, they can be somewhat grouped in a few categories with the main differential being each scheme's means of dealing with traffic since that is the most direct user cost.

Four basic options of dealing with traffic exist. Traffic can be detoured through an alternate route, using another existing bridge. Traffic can be kept on the bridge during replacement through a complex staging technique or an innovative construction method. A new bridge can be built along side the old one (while the traffic is on the old one) and then rerouted to the new bridge after construction. Finally, a detour bridge can be constructed alongside the actual bridge, and traffic re-routed on it during construction; afterwards the detour bridge would be demolished or deconstructed.

Each strategy is examined in detail below and advantages and disadvantages of each strategy analyzed. In order to demonstrate the replacement schemes in detail, a real site is used. Sagamore Bridge was selected because of its proximity to the area and majority of the readers being familiar with the site.

1.1 ALTERNATE ROUTE

Sagamore Bridge carries HW6 over Cape Cod Canal. As illustrated in Figure 3, the closest detour (outlined in red) would be via the Bourne Bridge. This detour is approximately 8 miles out of the way. On top of the extra driving distance, the capacity of the Bourne Bridge should be examined. The traffic demand of both the Sagamore and Bourne Bridge exceeds the capacity – anyone who has traveled to Cape Cod in the summer time is too familiar with this issue. If Sagamore traffic is detoured via Bourne Bridge, the delays in peak hours will be unacceptable.

This situation can be extrapolated to any general bridge site – an alternate route option is usually too much extra driving distance and it increases the traffic on the bridge used for detour.

FIGURE 3: ALTERNATE ROUTE FOR SAGAMORE BRIDGE REPLACEMENT [WWW.GOOGLE.COM]

1.2 TRAFFIC ON THE BRIDGE DURING REPLACEMENT

Keeping traffic on the bridge during replacement can be done by either innovative construction practices or a complex staging scheme. It has been shown by past MIT students that Sagamore Bridge can be replaced with minimal traffic interruption by using an innovative construction practice and an arch design that accommodated the construction scheme [8].

Innovative construction methods usually require that the contractor works together with the bridge design team – and this implies a design-build mechanism for delivering projects. Although design-build projects frequently drive innovation and are very common in the private industry, many government agencies still have rules against design-build delivery mechanisms. The reason traditional delivery mechanisms of separating designers and contractors are preferred is that it allows more firms to participate, and thus drives up the competition and hence reduces the bids on the tenders. Needless to mention, most bridges are owned by governments since they are on public roads.

Staging schemes are very site specific and depend on the type of bridge system used. The Sagamore Bridge has a superstructure that is laterally connected on both sides (Figure 4) which makes it very difficult to split the bridge into two halves and thus work on one side while passing traffic on the other (a typical method).

FIGURE 4: SAGAMORE BRIDGE SUPERSTRUCTURE [WWW.BOSTONROADS.COM]

The bridge would most likely have to be shut down during a few critical phases completely (the replacement of the main arch) and night time construction would have to be involved. The construction of the deck and secondary members can most likely be achieved by moving traffic from one side to the other side of the bridge.

From an overall perspective, even if staging is possible – there are many disadvantages to the user. The bridge capacity is reduced, therefore there are traffic delays. Usually a much slower speed has to be observed on the bridge and frequent stops occur. The proximity to construction – regardless of the precautions taken by the contractor – puts the drivers at a higher risk safety-wise (there is an increased probability that either the construction can have an accident and affect the driver or the driver can be distracted by the site and cause an accident). Usually night time construction is involved – this is difficult on the construction crew, and they might be more prone to error because of the lack of light; it is also very costly.

1.3 NEW BRIDGE PLACED AT AN OFFSET ALIGNMENT

A practice that is used sometimes is building a replacement bridge on an offset horizontal alignment. While the new bridge is being constructed, the traffic remains on the old bridge, and after it is finished, the traffic is slightly interrupted while the new approaches are being tied in (not a significant time). After the approaches are finished, the traffic is moved to the new bridge and the old one is demolished.

Interstates usually have speed limits of 60mph and above – this speed limit dictates the maximum road curvature that is acceptable (based on allowable centrifugal forces generated by negotiating a certain curve at a certain speed). Since an offset in horizontal alignment requires curves, this construction scheme is heavily dependent on the surrounding road geometry and allowable curvatures.

By AASHTO Policy on Design of Highways and Streets (the governing code for all American highways), radii of approximately 1500 ft or more have to be maintained on interstates and a minimum distance of 230 feet between reverse curves is required for speeds over 60mph. These criteria are used to layout a potential offset scheme for the Sagamore Bridge and another site to show a comparison of how surrounding road geometry can drastically affect the ultimate road layout with the offset bridge construction alternative.

Interstate 93 over Neaponset River near Ashmont, MA illustrated in Figure 5 shows where the offset strategy is favorable in terms of improving road geometrics.

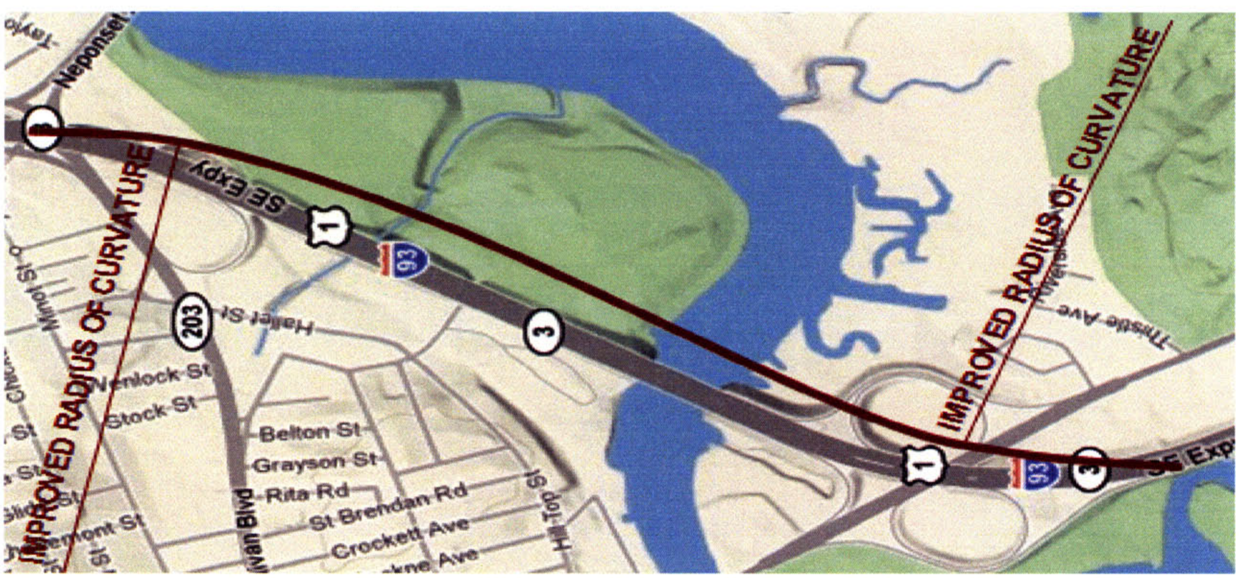

FIGURE 5: FAVORABLE ROAD GEOMETRICS FOR NEW OFFSET BRIDGE [WWW.GOOGLE.COM]

Notice that the offset alignment uses the existing curvature in the road to tie back into the existing alignment and the shift in the road actually improves the existing radii of curvature.

This concept of improving the radii is pretty important: when changing the alignment of the road, as with any design, it is considered bad practice to reduce the service level from its original design (i.e. create some unfavorable/ suboptimal geometry that did not exist there before).

These criteria are used to layout a potential offset scheme for the Sagamore Bridge. The result is illustrated in Figure 6.

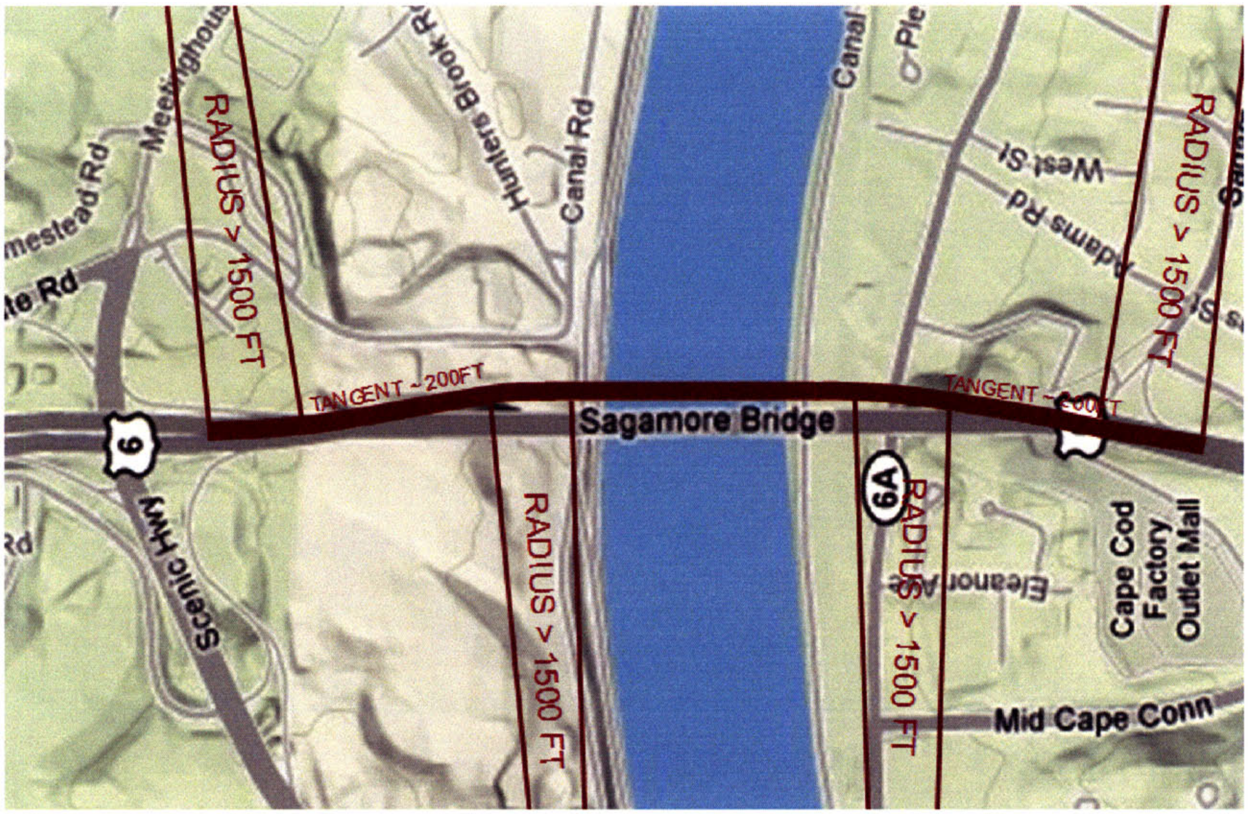

FIGURE 6: UNFAVORABLE HIGHWAY GEOMETRICS FOR AN OFFSET PERMANENT BRIDGE [WWW.GOOGLE.COM]

Since the alignment is straight, the only way to offset it is by putting a "kink" in the road. This basically deteriorates the existing alignment, which is unfavorable for the user and government agencies try to avoid it. Furthermore, because of the stringent requirements for the curves and distances between them, this results in a large amount of extra roadwork.

Finally, both in the Interstate 93 and Sagamore Bridge case, offset schemes require a lot of extra land. In fact, the propped offset alignment of Interstate 93 cut directly through one of its approach ramps and affects another overpass bridge on the road. The whole interchange would have to be redesigned and shifted, which comes at a high price.

To summarize, building a new bridge on an offset alignment is ideal in terms of traffic accommodation. However, many sites (like ones with a straight alignment) have road geometry which will result in a "kink" in the road, or another form of suboptimal horizontal alignment which the users will have to live with for the lifespan of the bridge. Furthermore, in order to accommodate the stringent requirements of highway geometrics (min. radii of curvature, etc), a large portion of surrounding land and approach road has to be disturbed. Frequently this is not an option since the real estate near by could be very valuable (within cities for example) and surrounding structures hard to relocate.

1.4 DETOUR BRIDGE PLACED AT AN OFFSET ALIGNMENT

The final strategy of dealing with traffic is similar to that of the last one discussed – the offset bridge. In this strategy, however, the offset bridge is a detour bridge. The construction sequence is as follows: a detour bridge and its approaches are constructed, traffic rerouted to the detour structure, old bridge replaced with a new one on the same alignment, traffic is then put back on the old replaced bridge, and finally the detour bridge is deconstructed. This method is shown in Figure 7.

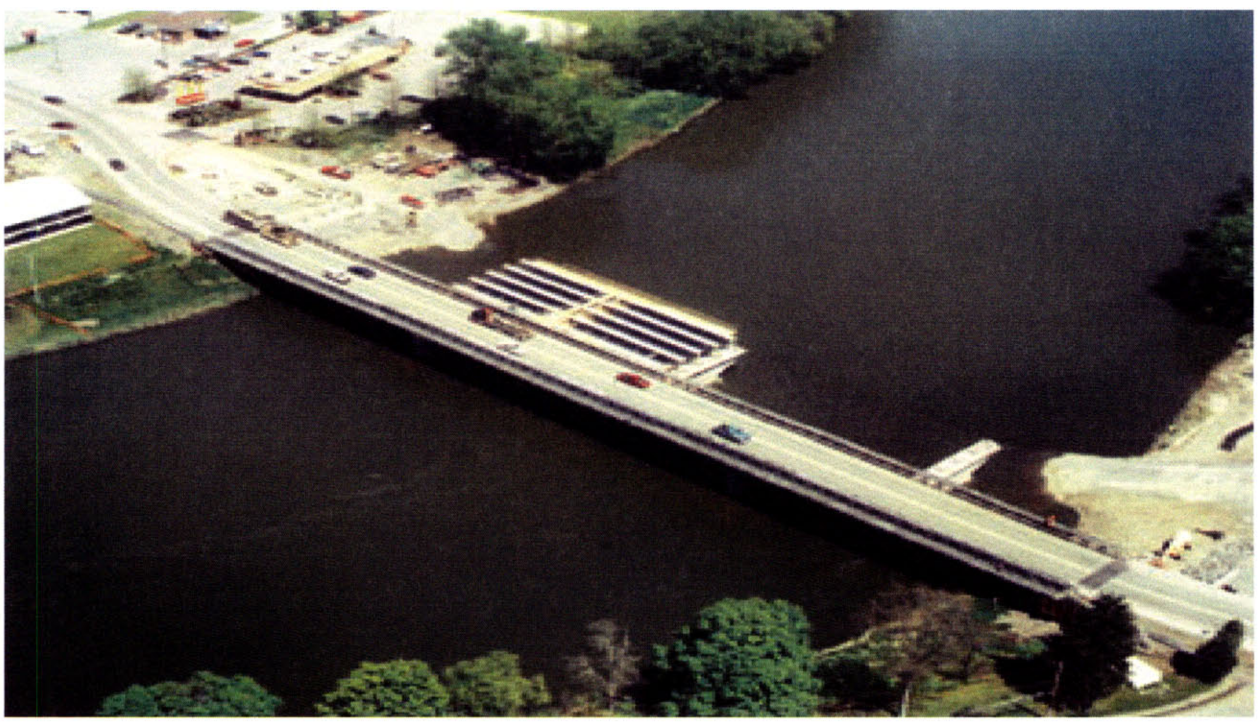

FIGURE 7: DETOUR BRIDGE USED ON AN OFFSET ALIGNMENT [WWW.MAYBE.COM]

The main advantage of this scheme is that the offset bridge – or the "kink" in the road – is now a temporary structure, not the permanent solution. This notion of it being a temporary solution opens the discussion of the level of service which it needs to provide; the project sponsor is likely to agree to lower than existing level of service for the traffic detour bridge if necessary. Ultimately what this implies is that there is a lot more leeway in following the geometric criteria and thus a lot less land could be used, and surrounding expensive structures/land can be preserved. This is demonstrated for both the Sagamore Bridge and HWY93 crossing in Figure 8 the orange line shows the temporary detour bridge alignment in comparison to the red line which is the alignment for an offset permanent structure as discussed in Section 1.3.

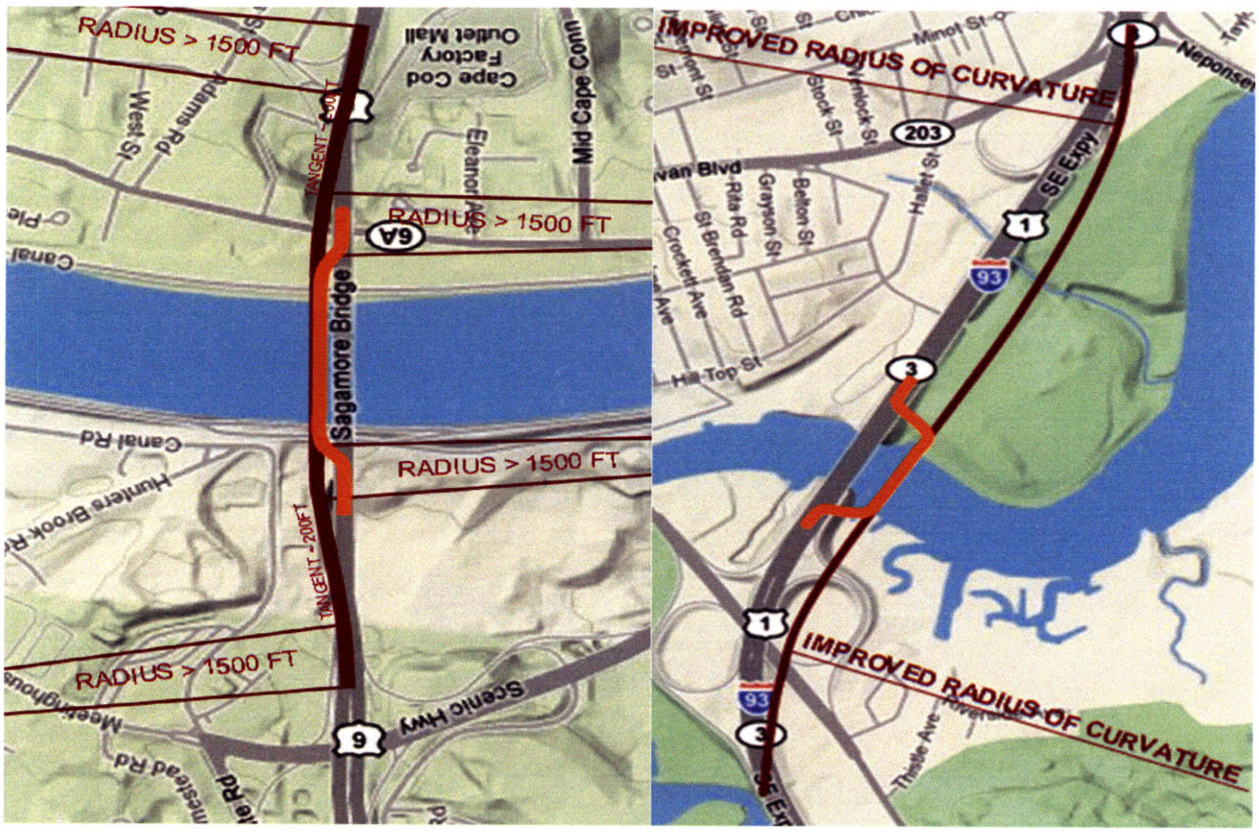

FIGURE 8: SAGAMORE OFFSET DETOUR BRIDGE (LEFT) & HWY 93 DETOUR BRIDGE (RIGHT) [WWW.GOOGLE.COM]

Of course, this scheme also has a shortcoming – two bridges are constructed instead of one. If a normal bridge is used this scheme is not only a waste of money but also a highly unsustainable way to use materials. However, if the detour structure is not constructed as a permanent structure, but thought of more as modular construction equipment that can be reused, then this strategy is optimal in terms of user costs for the sites where it is not favorable to offset the permanent structure.

The following section discusses what parameters this detour structure would have to respond to in order to fill the criteria of acting as modular construction equipment as opposed to a permanent structure.

1.5 WHAT MAKES A BRIDGE MODULAR?

The detour bridge can be thought of as construction equipment. The main goal is to use it on multiple sites – this is the only way to make the detour bridge scheme sustainable and cost effective. This then implies that the bridge has to become a modular system. What, then, is the definition, or goals of this modular bridge system?

Firstly, the ability of the bridge to be easily constructed and deconstructed should be considered. This would imply connections that can be reused such as bolted, slotted, or bearing; welding should not be used. Furthermore, structural schemes such as a plate girder with a concrete slab on top that is made composite through use of shear studs should not be used – these are too hard to disassemble.

Looking at the ability to easily construct and deconstruct from another angle, the bridge should also be designed such that the construction process by which it is built is simple since that is the process that will be used continuously. Given that the goal of this bridge is to be used by many different projects, the construction process should be familiar to all bridge construction crews and should use equipment that is already onsite to construct the actual bridge.

Out of the three most available building materials – steel, wood, and concrete – the choice is pretty obvious. Concrete is too hard to join and disconnect afterwards, and wood capacity is well surpassed by steel. Steel is frequently used in bridge construction, but measures must be taken to protect it from corrosion. Aluminum is lightweight but is not strong enough to bridge loads.

The design of the bridge should also be adaptable to various sites without having to re-fabricate elements of it. This is one of the most challenging aspects of a modular bridge design. How do you create a system which is applicable for various bridge spans, heights, and widths?

The concept can be rationalized in the following manner. A bridge can be thought of as a beam supported by columns, because ultimately that is what it is. The beam needs to be flexible in terms of length – different sites have different distances to span, different places where pylons (columns) can be placed. The varying span and width also result in different maximum stress on the bridge; therefore sometimes the area and moment of inertia of the beam element need to be varied.

Not only does each crossing vary horizontally in span, it also varies vertically in depth. Therefore the columns that support it need to be flexible in terms of height. Column moment of inertia and area also need to have the flexibility since the loads on the column vary, and as column height goes up, the moment of inertia needs to be increased for buckling purposes.

This narrows down the options to some steel elements that are in panel configuration and can be connected. These pieces must be of dimensions that are transportable by a typical truck – not every site will have access by barge or other special transportation mechanisms.

Right away, a truss system that has clear panel separations such as a Pratt, a Howe or a K-truss comes to mind (Figure 9). Truss panels can be easily adapted to various span lengths (use more panels) and doubled height-wise or thickness-wise to respond to increased loading. Moreover, pylons can be constructed out of the same elements and their moments of inertia can be increased in the same way as the superstructure. Steel panels can be easily bolted together, and the sections that are connected can be small enough to transport and conveniently store when not in use.

Incidentally the idea of using truss panels to create modular bridges and use them for traffic detours is not original to this thesis. This only confirms that this is an economically viable and desired way of dealing with bridge replacements and thus the idea is further investigated.

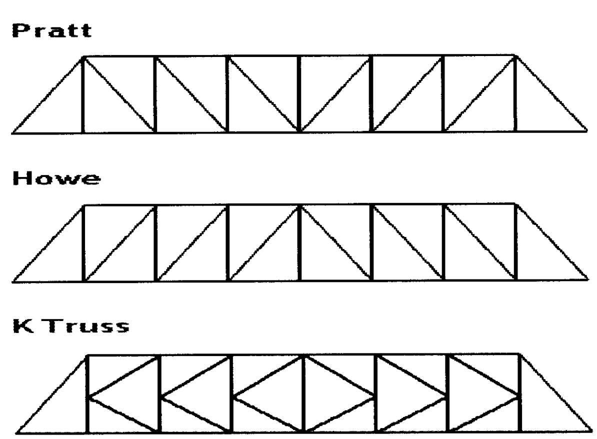

FIGURE 9: TRUSSES WITH MODULAR CONFIGURATION

2.0 Existing Modular Bridges

There are some companies today that provide modular bridges for detouring traffic. Although quite a clever scheme for bridge replacements, it wasn't the construction industry that invented a modular bridge.

2.1 BAILEY BRIDGES

The invention of the modular bridge dates all the way back to 1935. It was invented by A.M. Hamilton for military operation, particularly quick replacement of destroyed bridges and access to isolated locations. Figure 9 shows Hamilton's original design which was named the Callender- Hamilton System.

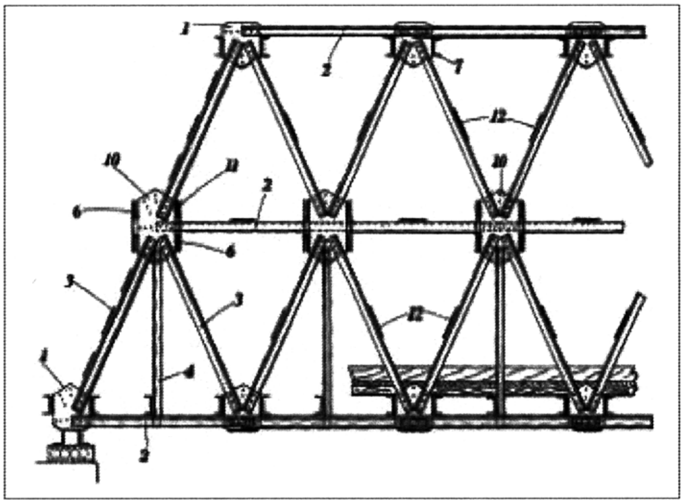

FIGURE 10: CALLENDER HAMILTON SYSTEM, 1935 [FHWA.DOT.GOV]

According to Federal Highway Administration, the design was "augmented" and improved by Sir Donald Bailey in 1944 (some Wikipedia users claim that it was a patent violation). Bailey's design (Figure 10) could span longer distances – its versatility allowed to stack panels laterally and vertically. Furthermore, it was designed such that it could be cantilevered out (launched) or erected by a crane and each panel could be carried by a crew of maximum six men.

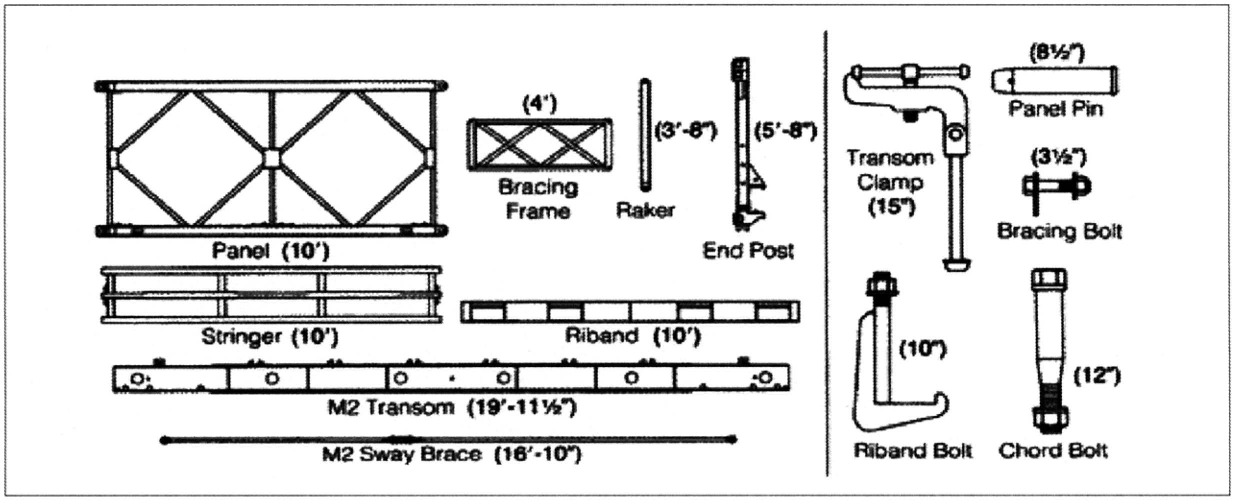

FIGURE 11: DONALD BAILEY'S SYSTEM, 1944 [WWW.FHWA.DOT.GOV]

Furthermore, it was designed such that it could be cantilevered out (illustrated in Figure 11) or erected by a crane and each panel could be carried by a crew of maximum six men. The Bailey Bridge formed a basis for the modular systems used today and the name is still commonly used in the industry. [6]

FIGURE 12: BAILEY BRIDGE CONSTRUCTED BY CANTILEVERING OUT [WWW.FHWA.DOT.GOV]

2.2 EXISTING MODULAR BRIDGE INVENTORY TODAY

Currently, two companies in North America supply modular bridges – Acrow Bridges and Mabey Bridge and Shore. Both have very similar design (Figure 12) which is composed of a deck that rests on secondary members referred to as transom beams. The transoms frame into truss panels. Both companies rent and sell the panels; some State Departments of Transportation have purchased these panels for their own inventory.

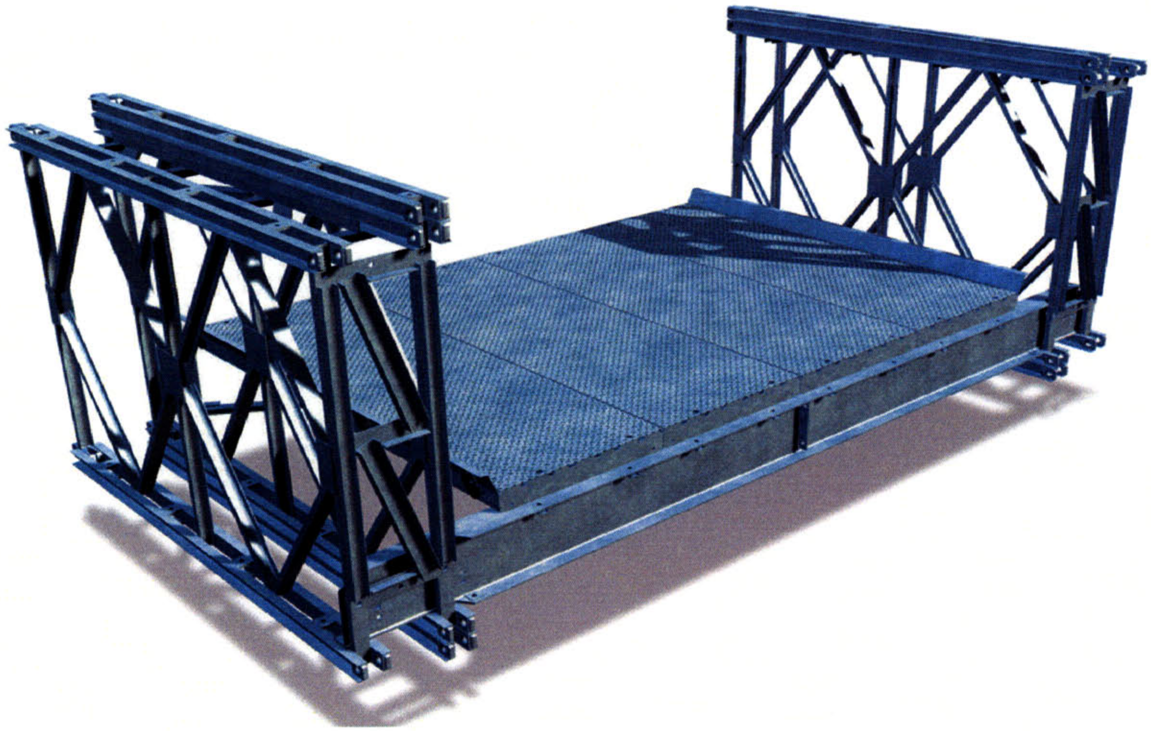

Figure 13: Mabey Modular Panel and Deck [7]

The following section uses information and figures from the Acrow 700XS Technical Handbook, or the "Acrow Manual" in order to describe the system in more detail [1].

The Acrow Panel 700XS panel has three subtypes: AB701 Basic Panel, AB702 Shear Panel, AB708 High Shear Panel. All the panels have the same dimensions of 2.286m depth x 3.048m width (Figure 13), but different resisting shear values. AB701 can handle up to 534kN in shear, AB702 can resist 801kN and AB708 takes 1023kN.

The 700XS panel is the most important component of modular bridges; it is now on its 3rd evolution of design and has improved by 50% in bending and 20% in shear from its original design [6].

The versatility of the modular panel allows for construction of many other bridge components. Figure 14 illustrates the versatility of Acrow panels.

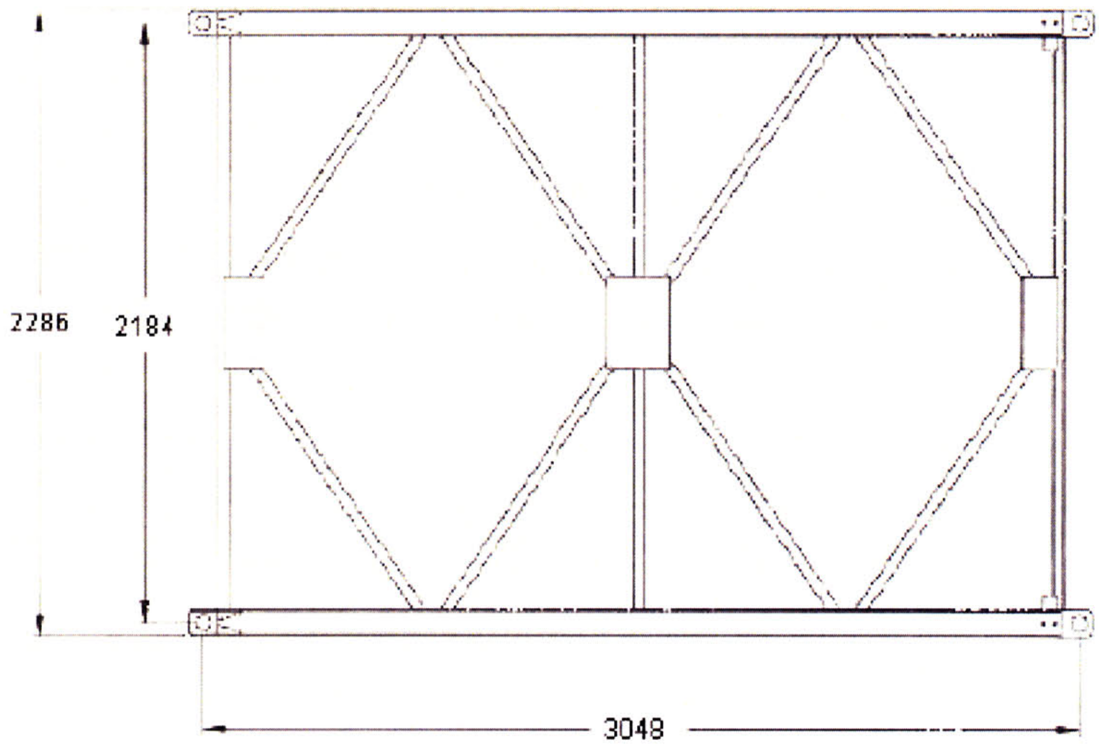

FIGURE 14: ACROW 700XS PANEL [1]

FIGURE 15: VERSATILE APPLICATIONS OF ACROW PANELS [2]

Just like the Bailey Bridge panels, the Acrow truss panels can be stacked up to four times laterally and two times vertically. All the various configurations and their nomenclatures are shown in Figure 15 below.

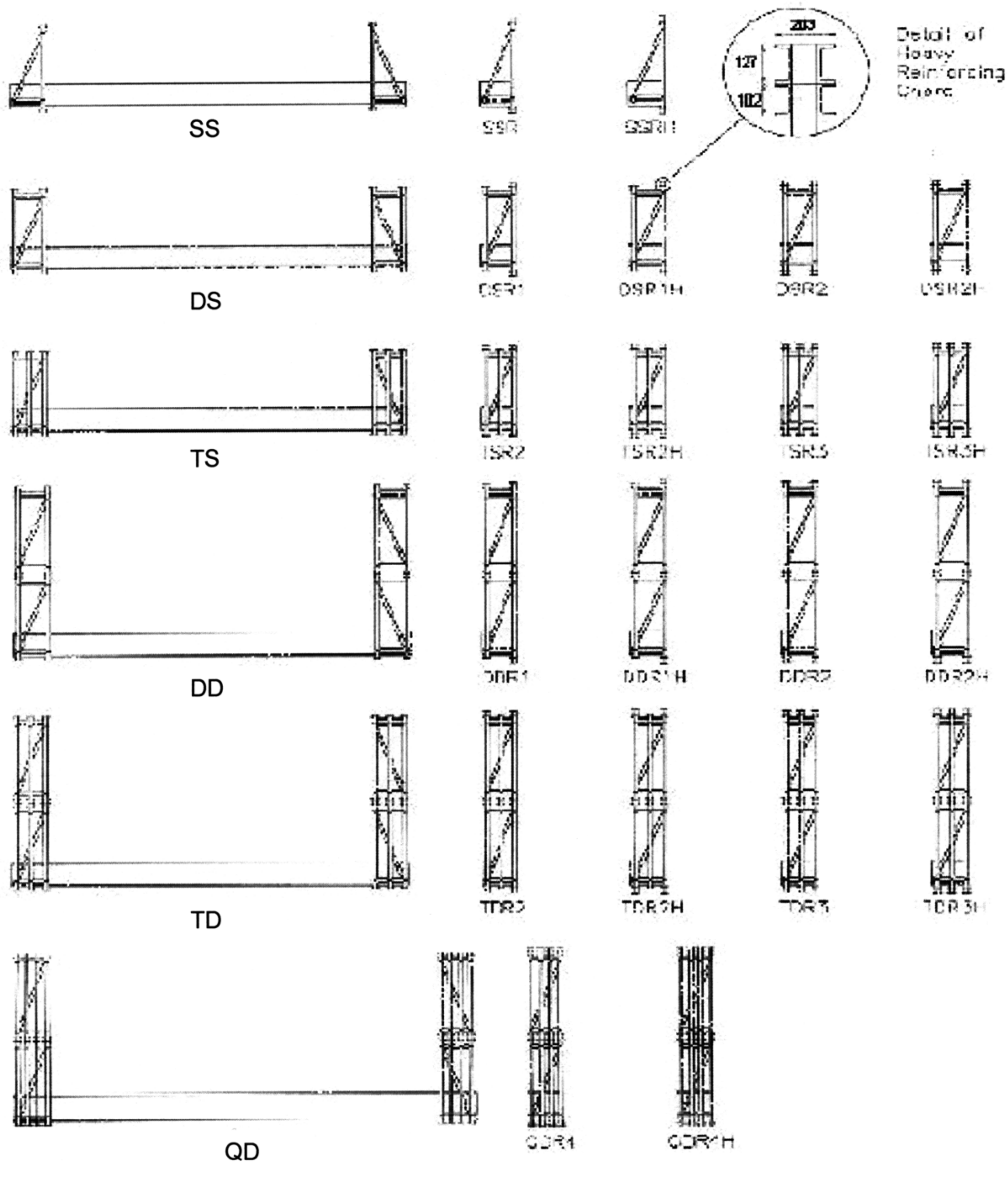

FIGURE 16: VARIOUS ACROW PANEL CONFIGURATIONS [1]

Figure 16 demonstrates a double-double scheme connection detail which is critical to the idea of modularity. The bolting together of male and female panel parts allows for ease of assembly and disassembly without damaging the components.

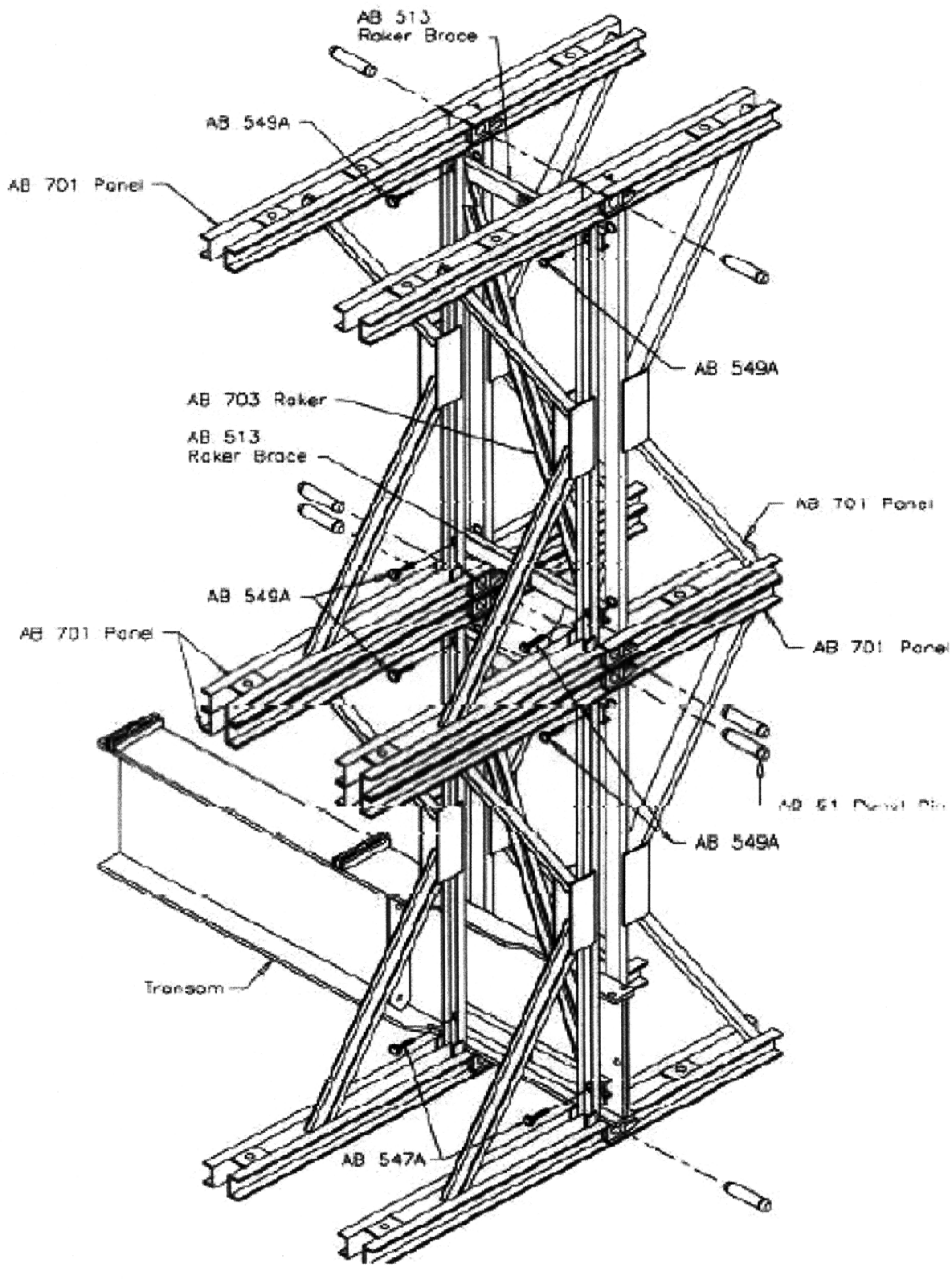

FIGURE 17: DOUBLE-DOUBLE CONFIGURATION DETAIL [1]

The deck is bolted to the transom beam as shown in Figure 17; this detail once again reinforces the modularity of the bridge. The width of the deck can be up to three lanes (based on a 3.65m wide lane) and with or without shoulders.

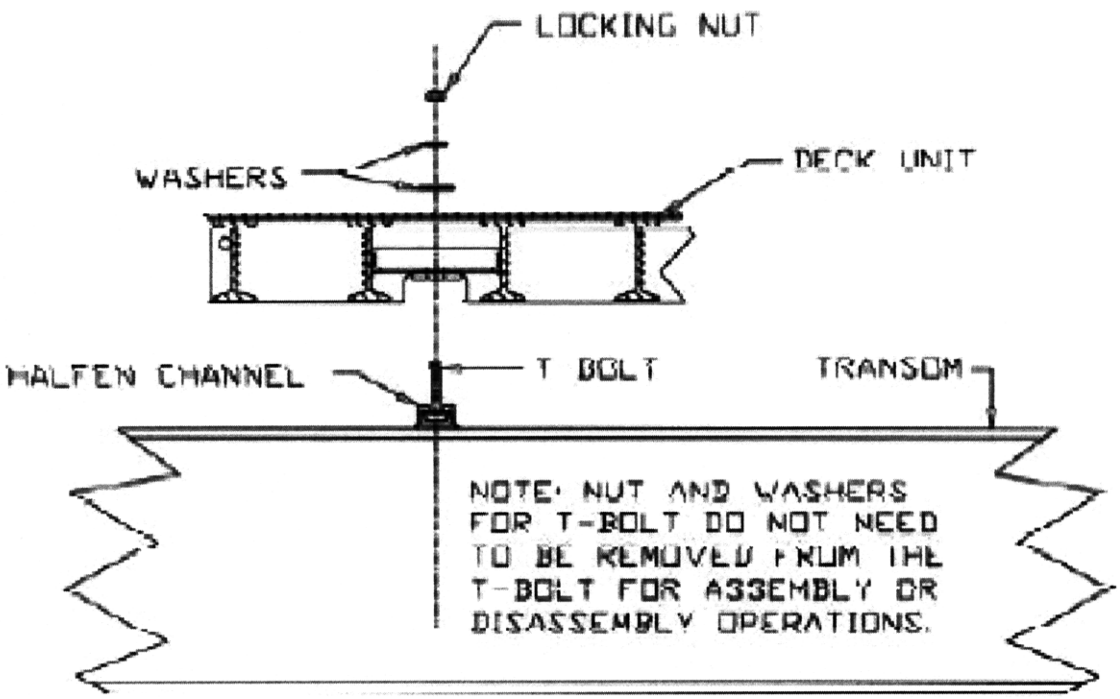

FIGURE 18: TRANSOM AND DECK ASSEMBLY [1]

As with Bailey Bridges, two construction methods are available. The deck can be launched (cantilevered out) or it can be installed with the help of cranes.

2.3 NEED FOR LONGER BRIDGES AND LARGER CLEAR SPANS

As demonstrated before, a modular bridge is a very advantageous alternative for bridge replacements. However, these bridges currently have one disadvantage: because of the structural scheme used for modularity, they are limited in the maximum clear space they can span.

The length of many existing bridges far exceeds what the modular bridges can span. Even though on some crossings temporary piers can be placed through the crossing to reduce the length between supports, a lot of times clear spans can not be violated. This could be due to navigability issues or a wide highway passing under the crossing where the traffic can not be disturbed.

Some environmentally sensitive streams also do not allow work inside the stream during fish spawning seasons and require very expensive and restrictive permits for all the other days of the day that the steam can be worked in. For example, the map in Figure 18 below shows some streams near Edmonton, the capital of Alberta.

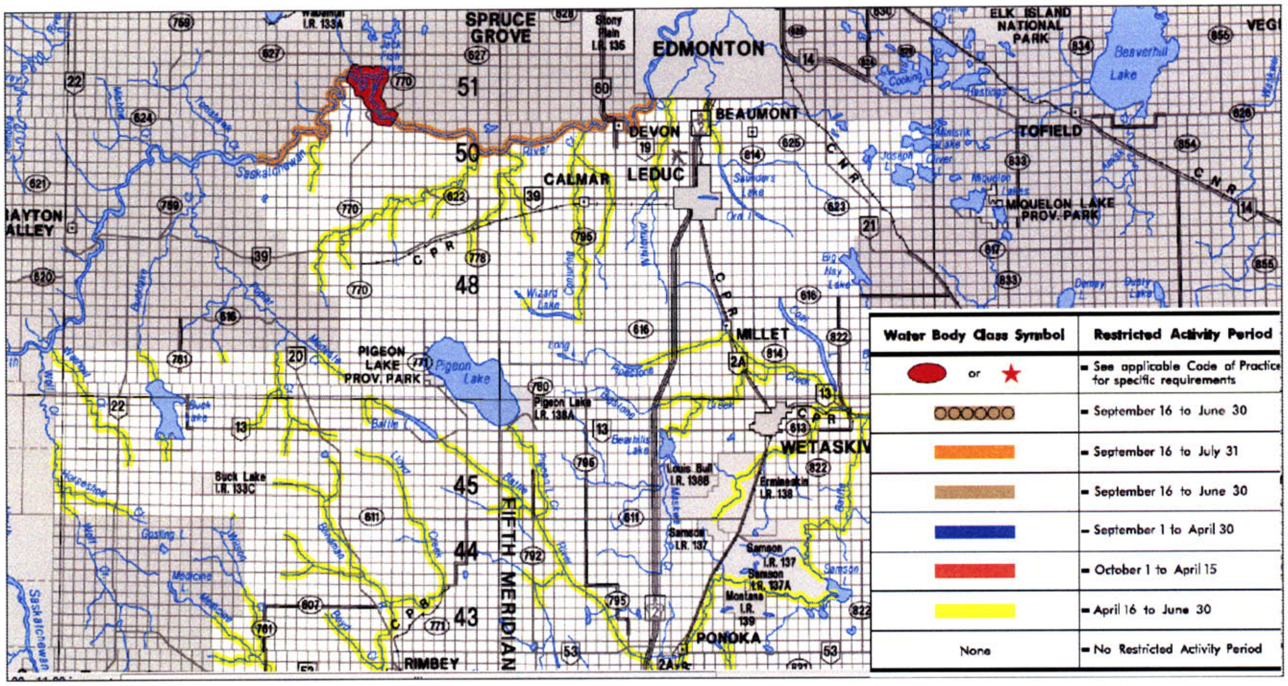

FIGURE 19: STREAM ACTIVITY RESTRICTION PERIOD NEAR EDMONTON, AB [ENVIRONMENT.ALBERTA.CA]

The stream in orange only allows construction in water from July 31st to Sep 16th; that is only 45 days, therefore all the construction activities will have to be scheduled around that. The majority of the other streams are yellow – in these the construction in the water can start after June 30th. At first, this seems reasonable; however, since the province is in the north, construction season usually ends by the end of October. So in fact, building piers in water puts a huge scheduling constraint on most of the crossings in that province.

Furthermore, if the decision was made to put piers in water, a significant amount of money has to be paid to have an environmental engineer approve construction activities and use expensive equipment to monitor water turbidity and sediments going into the water. The environmental approval process also requires a significant amount of paper work. This is a large expense for building a pier in the water and could potentially stop the contractor from using a temporary bridge that is limited in span on a larger crossing.

Moreover, even if it is possible to build a pier in the water – it is still one of the more expensive construction items. Typically, the process involves building berms (piles of dirt) in the water, so that the foundation area can be isolated, dewatered, and trucks with concrete can be brought in to pour the foundations.

Placing foundations takes away from the modularity of a temporary bridge as well. The goal is to reuse all the materials utilized in constructing the temporary structure– for cost efficiency and environmental effects. However, it would be impossible to reuse concrete foundations, and no contractor will even consider taking them out of the water (leaving behind a disturbed fish habitat) after the modular bridge is disassembled.

Of course, other means of supporting piers have been explored. Since Bailey Bridges have been around, the army has been using floating foundations (Figure 19).

FIGURE 20: BAILEY BRIDGE SUPPORTED BY FLOATING FOUNDATIONS [WWW.BTINTERNET.COM/~IAN.A.PATERSON]

This is a reasonable solution for crossings that are not too affected by water fluctuations (short period of use), and the joints at the abutments can be flexible. The detour bridge would have to be in function for one to two years and since the general public is using them at fairly high speeds (they would be paved) the joints cannot be flexible and the level of the deck can't change with water. Therefore hydraulic systems would have to be used to maintain a constant elevation which are expensive pieces of equipment and have mechanical components in them that would likely require intensive maintenance.

To reiterate, modular bridges are not currently designed to span long distances, but there is a clear need for it; therefore, means of achieving longer spans are worthwhile investigating.

2.4 SPAN LIMITATIONS OF EXISTING MODULAR BRIDGES

As previously mentioned, the span length of a modular bridge depends on which configuration of panels is used. Clearly when panels are double stacked, or stacked vertically, the capacity of the bridge is increased or it can span a longer distance for the same applied load.

The maximum span was calculated for every bridge configuration using the information on the inertia and the dead weight from the Acrow Manual. Typical highway vehicular lane load (10kN/m) and factors of 1.6 for live and 1.2 for dead were used. The results are listed in Table 1 below (as a number of 3.048m bays). The sections were checked for deflection as well as shear and moment capacity. Moment governed unless stated otherwise in the table.

Panel Configuration	Moment of Inertia cm^4	Maximum Span (number of 3.048ft bays)	
		2 lane	3 lane
SS	1,312,960	8	6
SSR	2,881,237	11	shear 8
SSRH	3,299,383	12	shear 8
DS	2,625,921	11	9
DSR1	4,194,198	13	10
DSR1H	4,612,344	13	shear 10
DSR2	5,762,474	15	12
DSR2H	6,598,767	16	13
TS	3,938,881	13	11
TSR2	7,075,435	16	13
TSR2H	7,911,727	16	14
TSR3	8,643,712	18	15
TSR3H	9,898,150	19	16
DD	10,996,834	15	12
DDR1	16,999,974	shear 16	shear 11
DDR1H	18,521,383	shear 15	shear 10
DDR2	23,003,113	21	shear 15
DDR2H	23,045,931	shear 21	shear 15
TD	16,495,251	18	15
TDR2	28,501,530	23	shear 17
TDR2H	31,544,348	shear 23	shear 17
TDR3	34,504,669	25	shear 21
TDR3H	39,068,897	26	shear 21
QD	21,993,669	20	17
QDR4	46,006,226	27	24
QDR4H	52,091,862	29	25

Table 1: Maximum Possible Spans (Given as Number of Bays)

From the table it can be seen that by stacking 4 panels side by side and then repeating twice upwards (QDR4H configuration) the maximum span of 88.4m can be achieved for a 2 lane bridge and 76.2m for a 3 lane bridge.

These are fairly long spans; however the means to achieve them are fairly inefficient. The amount of self weight of the structure required to carry the highway load constitutes 60% of total load. A dead load to total load of 40% is a more typical ratio. Table 2 tabulates the dead to total ratio for all Acrow configurations which were previously described in Figure 15.

Panel Configuration	Percent of Dead Weight to Total Load	
	2 lane	3 lane
SS	38 %	38 %
SSR	39 %	39 %
SSRH	39 %	39 %
DS	41 %	41 %
DSR1	47 %	45 %
DSR1H	48 %	45 %
DSR2	48 %	46 %
DSR2H	44 %	42 %
TS	44 %	42 %
TSR2	52 %	49 %
TSR2H	53 %	49 %
TSR3	47 %	45 %
TSR3H	48 %	45 %
DD	46 %	44 %
DDR1	47 %	45 %
DDR1H	48 %	45 %
DDR2	48 %	46 %
DDR2H	49 %	46 %
TD	51 %	47 %
TDR2	52 %	49 %
TDR2H	53 %	49 %
TDR3	53 %	49 %
TDR3H	54 %	50 %
QD	54 %	50 %
QDR4	57 %	52 %
QDR4H	58 %	53 %

Table 2: Percent of Dead Load to Total Load for Each Configuration

It can be seen that the truss panels are heavy, and as soon as the arrangement is deviated from the basic single truss (SS), the dead weight dominates the loading. The maximum span of the SS panels is 36.6m for a 2 lane bridge and 24.4m for a 3 lane bridge.

3.0 DESIGN OF A LONG SPAN MODULAR BRIDGE

3.1 LONG SPAN ALTERNATIVES

Now that the problem of designing a long span modular bridge has been identified, what are the means of solving it and what are some of the design parameters to be fulfilled? As discussed in Section 2.5 "What Makes a Bridge Modular?" the following main sections still have to hold true:

1. Easily constructible and de-constructible
2. The parts must be re-usable
3. Design should be such that modules can be configured to make various bridge lengths and widths (the parts from a smaller bridge can be used to make a bigger bridge)

In addition, because a large inventory of modular panels exists, the goal would be to reuse as many of existing pieces as possible thus a realistic solution that can actually be used in the industry can be generated, as opposed to creating a brand new system that would require a large up-front investment.

With the criteria for design established the question of what the options are to making longer spans remains.

The alternative to putting a pier under the deck to support the span from under is to put a cable on top to grab it from above. There are two typical means of bringing the load through cables to foundations: it can be done by either cable-stays which bring the load directly to the pylons or cables which transfer the load to another main cable which is then anchored to towers.

Another solution could be to put cables through the bottom cord and prestress them to create compression, thus counteracting the tension forces created by dead weight (similar to putting post-tensioning or pre-stressing into a concrete slab). This would increase the capacity of the existing panels and thus allow for a longer span under the same load.

Active control mechanisms, such as hydraulic linear actuators, could also be used to put opposite forces into truss panels to counteract the live load. Just like the pre-stress, this would increase the maximum possible span.

Mechanisms such as active control can be ruled out at this point, as they are expensive and will constantly be assembled and disassembled during construction which is tedious and could damage the mechanical and electrical components. Prestressing cables would need some kind of bonding to the truss members, large anchors at abutments, and each crossing would require a different cable size. Suspension cables involve difficult construction methods which is the opposite of what is desired.

Cable stays on the other hand reflect modularity: the larger the span, the more cables you can add and the taller the tower can be. The balanced sequential span construction is fast and does not require any shoring because the modular panels can be constructed out with the cantilever technique that they are designed for.

The only additional item to the inventory would be the cables. The towers can be constructed out of existing panels, and thus their height can be kept modular. The girders of course would be made out of the truss panels, but instead of getting picked up by piers, they would be supported by abutments.

3.2 CABLE-STAYED SOLUTION

A cable-stay bridge is clearly the best alternative for creating longer span bridges. Figure 20 illustrates a number of cable arrangements that are possible: fan-shaped (orange), harp shaped (yellow), and mixed.

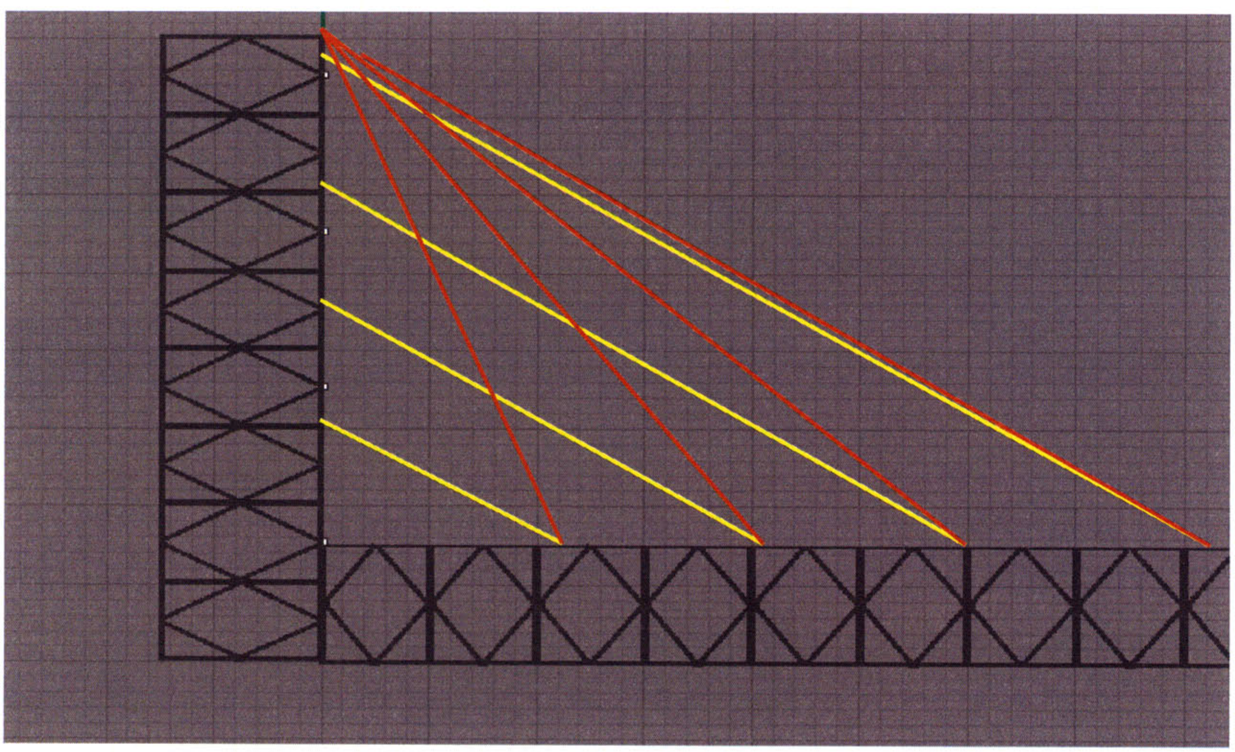

FIGURE 21: FAN AND HARP CABLE ARRANGEMENTS

The fan-shaped arrangement is very efficient because it has a high stiffness towards the middle of the bridge; however it is challenging to make the fan design modular. The difference in span would either require a change in the tower height, cables area, or cable spacing. A change in the tower height ultimately means a change in the cable length. Thus, it would be very difficult to reuse the same cables for various bridge configurations and maintain the same desired deflection limit and moment capacity. The mixed arrangement is partially a fan arrangement, and therefore has the same obstacles in terms of modularity.

The harp arrangement, on the other hand, seems a lot more promising. If a maximum span bridge is designed, smaller bridges can be made out of it without being over-designed. The cable area would be kept constant and spacing would vary for bridges that take higher loads.

The harp arrangement was therefore selected to preserve modularity. Since the tower is made out of slender steel elements, tower buckling could possibly limit the maximum achievable bridge span. Therefore the lowest recommended angle for cable stay bridges (based on effectiveness of cable) of 20 degrees was selected which would allow the use of the shortest tower.

Acrow bridges come in 4 widths: 1 lane, 2 lane, 2 lane plus shoulder, and 3 lane. They are used for a range of applications, from military applications to pedestrian crossings in parks; 2 lane and 3 lane widths were selected as the most desirable configurations for traffic detours. As demonstrated in Section 3.3 "Span Limitations of Existing Modular Bridge", single-single panel configuration was the most efficient in dead weight to total weight ratio, therefore it was selected as the spanning truss scheme in-between the cables.

3.3 BRIDGE LAYOUT

Cable stayed bridges are systems that self equilibrate the horizontal forces on the pylon – typically this is achieved by a backspan. However, if a backspan is included, construction within the trapezoid defined by a river ravine or a highway underpass would have to take place. This construction on a steep slope or at the bottom of the ravine is undesirable for numerous reasons such as difficult access and extra costs of building pylons under the bridge. The implications of building piers in water were discussed in Section 3.3 and the goal of designing a long span bridge is to avoid going into the ravine/channel. For this reason a backspan was eliminated for the bridge design and instead the balancing forces are provided by anchors similar to those of a suspension bridge. This cable-stay scheme is illustrated in Figure 21.

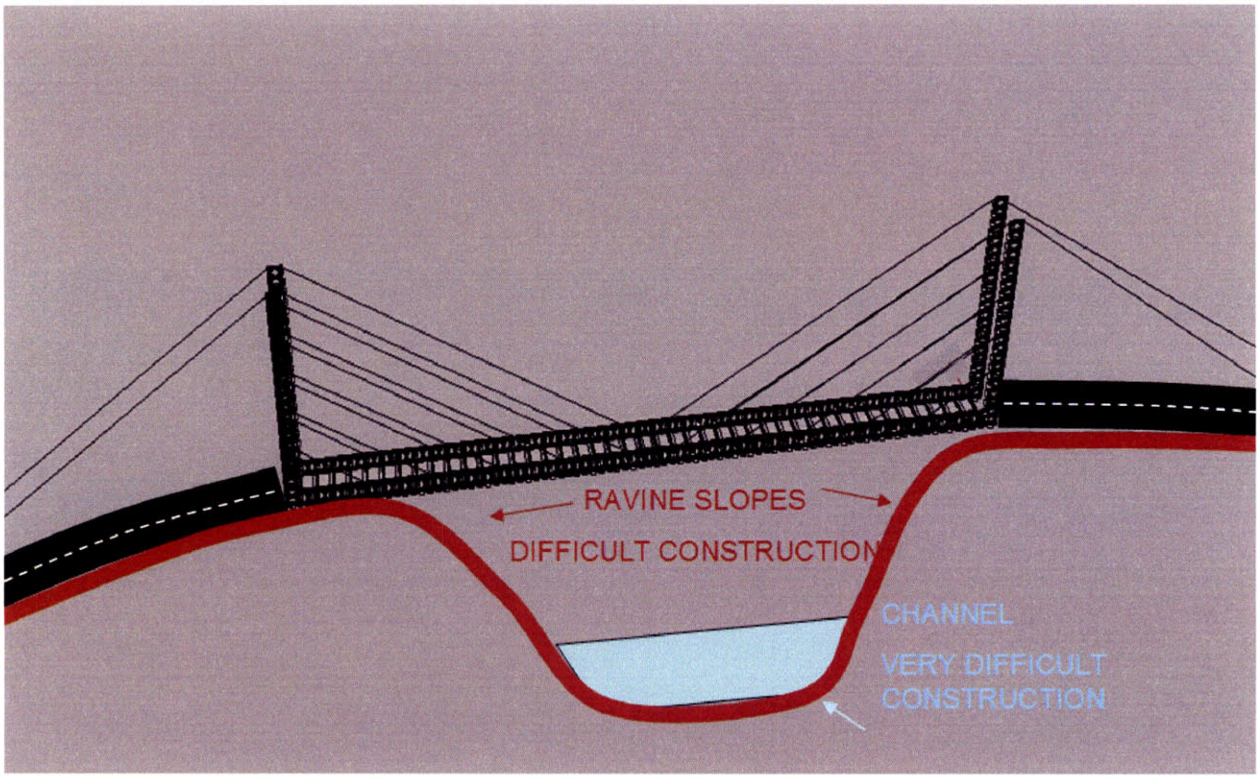

FIGURE 22: BALANCING CABLE-STAY MID SPAN WITH ANCHORS

Of course, these anchors require heavy foundations which from initial assessments could take away from the modularity of the scheme. Thus, ideas for modular foundations are presented in Section 3.10.

3.4 LOAD DEFINITION

The loads applied were in accordance with AASHTO Bridge Design Specifications [4]. The preliminary cable spacing and design was done using the dead weight of the deck with a wearing surface and its secondary transom beams, the truss panels, and a vehicular live load of 10kN/m/lane.

After the spacing was determined, a more detailed check on the capacity of the girders and cables was performed applying a116kN load for shear and 80kN for moment in the worst possible location (more details in Section 3.4 "Analysis") along with the 10kN/m/lane vehicular load. A moving HS20 truck was also applied at the detailed design check.

Load factors of 1.6 for live and 1.2 for dead were used. A deflection limit of L/800 was maintained on all structural members. This limit is typically used for permanent structures but it was confirmed as good design criteria for temporary structures with Massachusetts Highways Bridge Manager, Alexander Bardow [10].

3.5 CABLE SELECTION

There is a variety of cables available in today's industry: spiral, locked coil, structural rope, and parallel wire strand. Spiral strand, illustrated in Figure 22, was selected for the stays because of its versatility, high breaking strength, and a good weight to strength ratio.

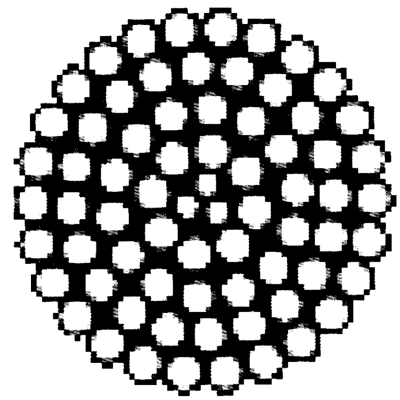

FIGURE 23: SPIRAL STRAND CABLE [WWW.CORUS.COM]

Corus, the supplier of the cable, recommends the following parameters:

- Available sizes: 13mm – 143mm
- Breaking stress: 1177MPa (largest diameter) – 1288MPa(smallest diameter); an average of 1200MPa was used
- Modulus of Elasticity, E: 165GPa
- Spiral configuration material density: 70% of steel 7900kg/m^3

3.6 CABLE AND DECK ANALYSIS: PRELIMINARY DESIGN

The cable design was very challenging as this is a process that has many unknowns and thus assumptions were made and then various iterations performed. Furthermore, because the design was for a modular bridge with various configurations, it significantly increased the complexity of the design and optimization approach.

Since the inertia of the section was fixed, the original strategy for spacing cables was to use the maximum that the trusses can span. Anchorage of cables was assumed to be at the end of bays because the Acrow panels are currently designed to accommodate that (the bridges are designed to terminate at the end of a bay and floor-beams carry the load from the deck to points on the trusses that correspond to end of bays). Therefore the spacing of the cables was taken as number of bays.

The maximum span of trusses was computed with a fixed-fixed connection because the trusses are continuous at cable supports. For a 2 lane bridge this resulted in a 10 bay maximum span (30.48m) and an 8 bay (24.384m) for a 3 lane bridge.

The effective modulus of elasticity, Eeff, was taken as 80% of E to be conservative. The required vertical stiffness Kv was calculated based on motion based design (MBD) and strength based design (SBD) using the following equations:

Motion Based Design

$$Kv = \frac{\Delta L * W_l}{v_{allow}}$$

v_{allow} = total bridge span, L/ 800, a conservative span of 200m was used for preliminary design

Strength Based Design

$$Kv = \frac{\Delta L \; * E_{eff} W_{factored} \sin(\Theta)\cos(\Theta)}{x_d \sigma_{ult}}$$

x_d = approximated as 100m to be conservative assuming that the maximum cable-stay modular bridge span would be approximately 200m

σ_{ult} = breaking strength, 1200MPa

Different widths of bridges carry different amount of load, therefore modular design became a difficulty at this point. Out of the two configurations, 2 lane and 3 lane, 3 lane bridge was selected as a starting point.

MBD governed resulting in a 3000kN/m vertical stiffness. Note that during preliminary design, all the values of spring stiffness were taken to be the same (invariant of x_d) which was a conservative approach since stiffness increases as x_d decreases.

The main goal of the first step was to select the cable spacing that ensured the model function as a beam on elastic foundation.

To model the SS configuration in SAP2000 a section was selected with inertia equivalent to the effective inertia of 0.02625m^4 given in the Acrow Manual and essentially treated as an equivalent beam. The SS configuration is to be supported by two planes of cables, but for simplicity the cable stiffness was added and the equivalent beam has one set of springs supporting it with a k_{eff}.

The first model shows that beam on elastic foundation behavior is in fact occurring. The deflection profile (Figure 23) follows the deflection profile computed using the same parameters and beam on elastic foundation theory (Figure 24).

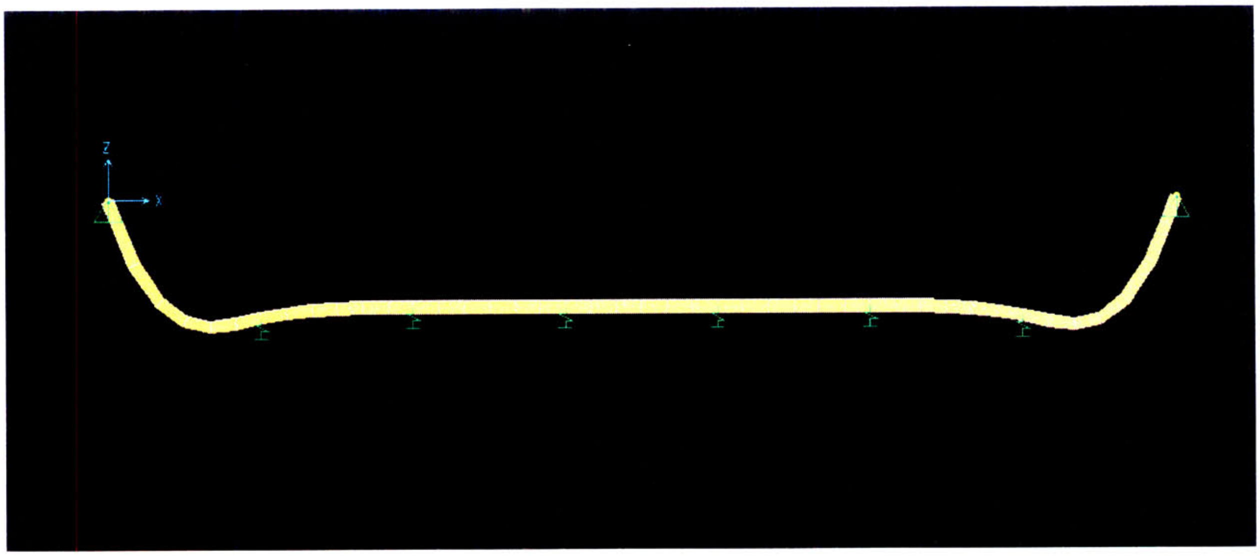

FIGURE 24: MODULAR BRIDGE DEFLECTION PROFILE GENERATED WITH SAP

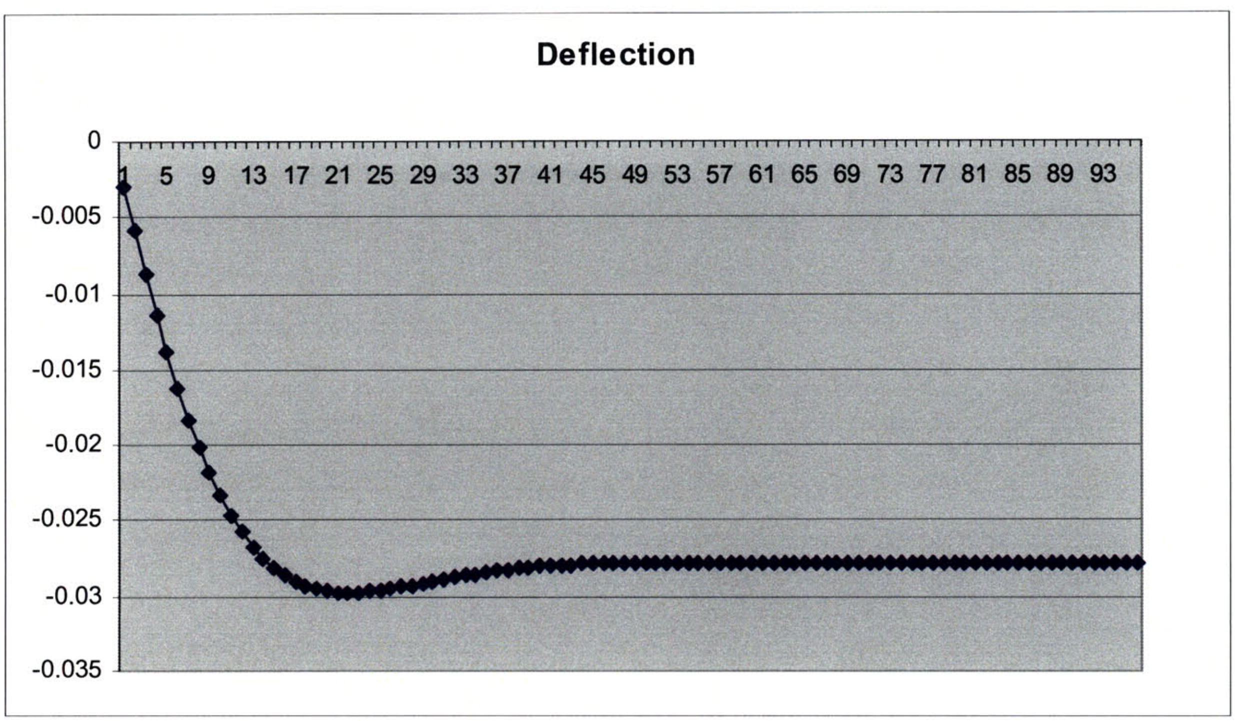

FIGURE 25: DEFLECTION PROFILE GENERATED WITH EXCEL BASED ON BEAM ON ELASTIC FOUNDATION

The behavior is not only confirmed by the profiles corresponding, but also by the fact that the springs pick up the entire load for their tributary area instead of allowing some of the shear to be transferred into the supports. This was checked by looking at the spring forces illustrated in Figure 25 against the total live distributed load multiplied (30kN/m) by the cable spacing of 24.384m.

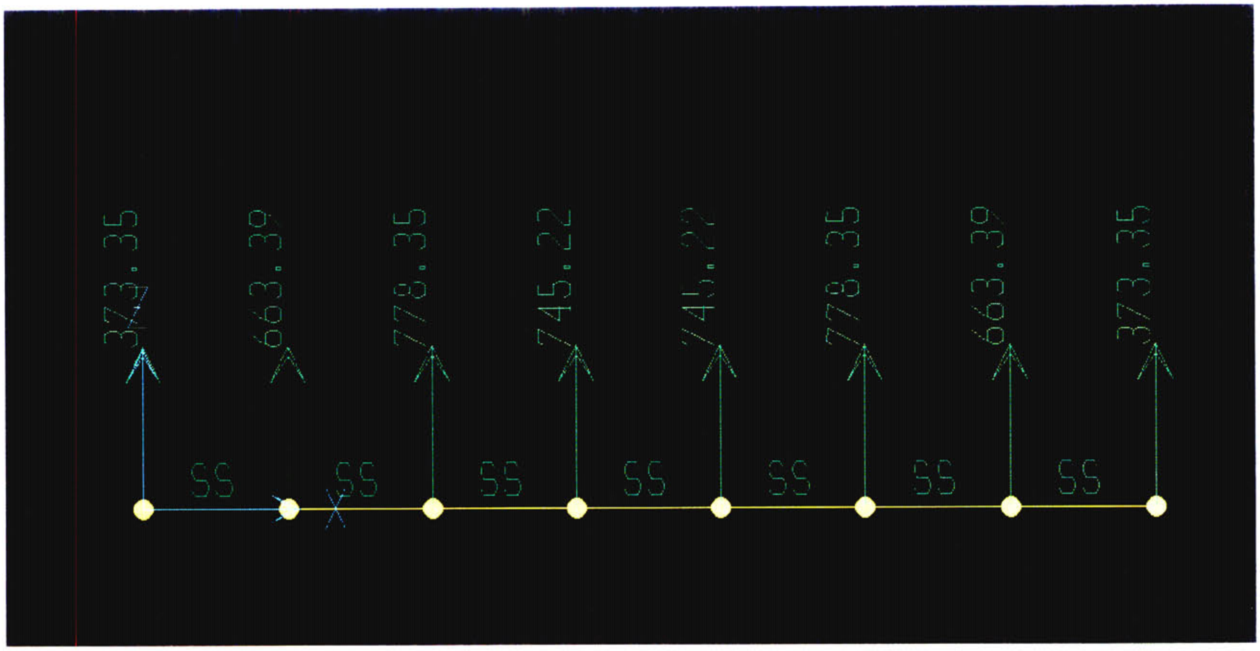

FIGURE 26: SPRING FORCES UNDER FULL LOADING

Furthermore, different span lengths were tried, and the overall deflection stayed the same for the same stiffness. The effect of going outside the ideal spacing region was evaluated, and the deflection profile is illustrated in Figure 26 below.

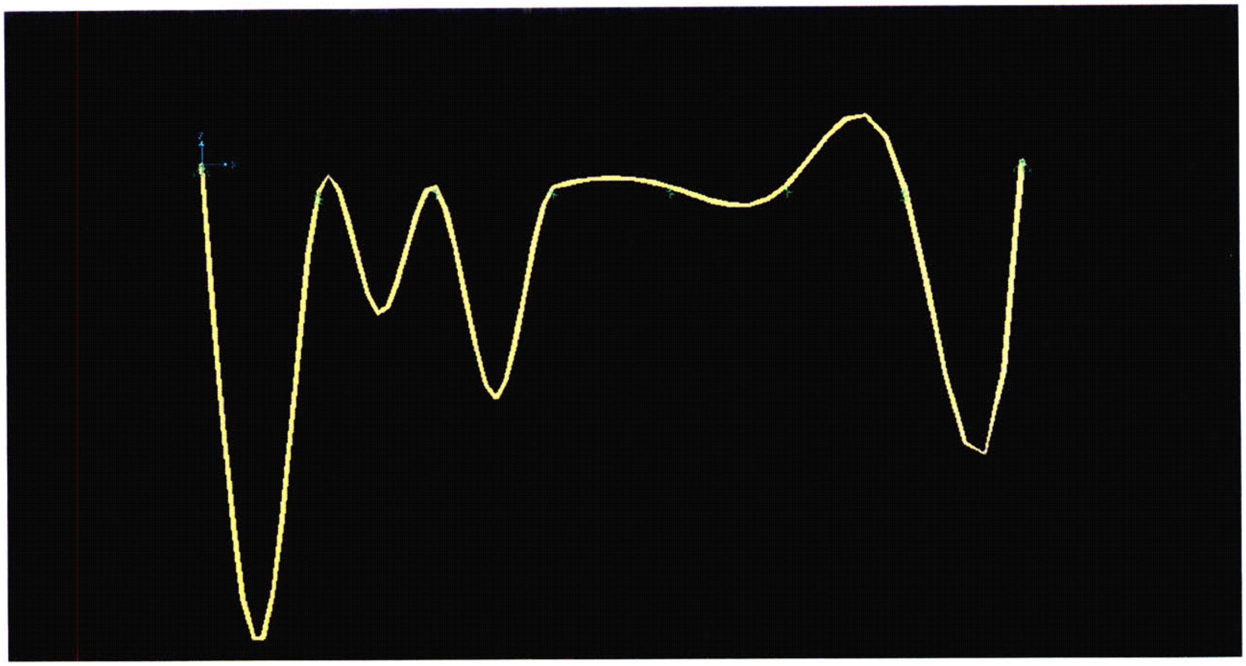

FIGURE 27: DEFLECTED SHAPE WHEN BEAM ON ELASTIC FOUNDATION SPACING IS EXCEEDED

Shear and moment distribution in the beam were drastically amplified. The local deflection in between cables was also beyond the allowable L/800 limit.

The original spacing of 8 bays and stiffness of 3000kN/m met the required deflection criteria. The shear in the section was also within the allowable limits. The moment, on the other hand, was very close to the allowable limit in the middle of the beam and exceed the allowed capacity in the region circled in red in Figure 27.

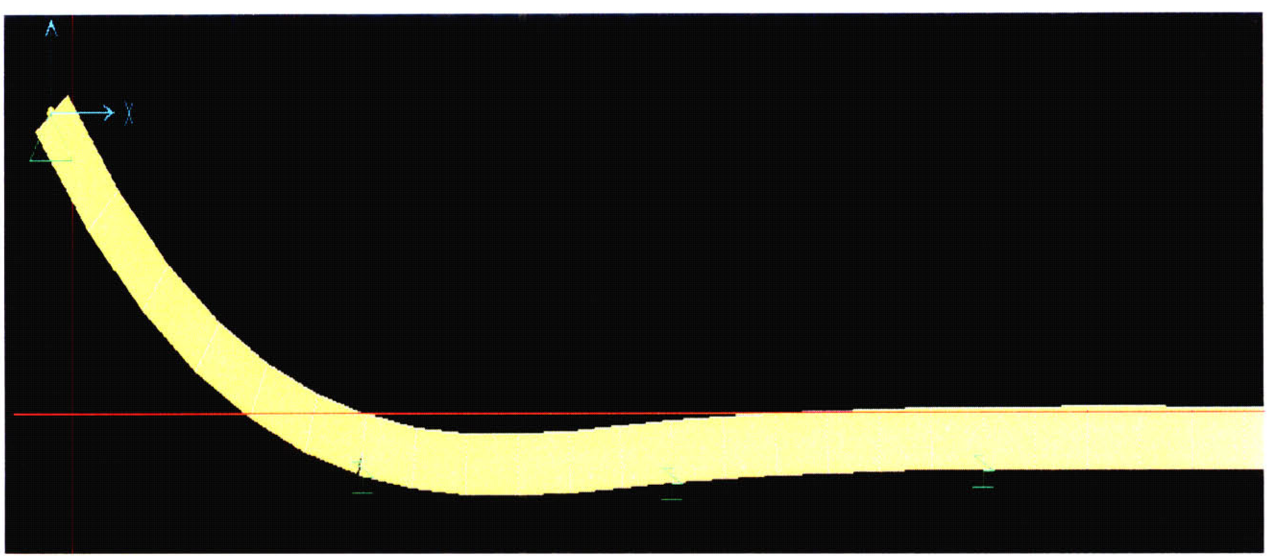

FIGURE 28: LOCAL DEFLECTION BEHAVIOR NEAR SUPPORTS

The allowable moment was computed using the section modulus obtained from the Acrow Manual and the following formula:

$$M_{allow} = S_x F_y$$

F_y = 350MPa

M_{allow} = 4014kNm

As previously mentioned, maximum spans were estimated based on the assumption that they were fixed-fixed and thus a moment given by the formula below was used.

$$M = \frac{wl^2}{12}$$

However, it was apparent from the moment diagram (Figure 28) that there was an additional moment generated in the effective beam section.

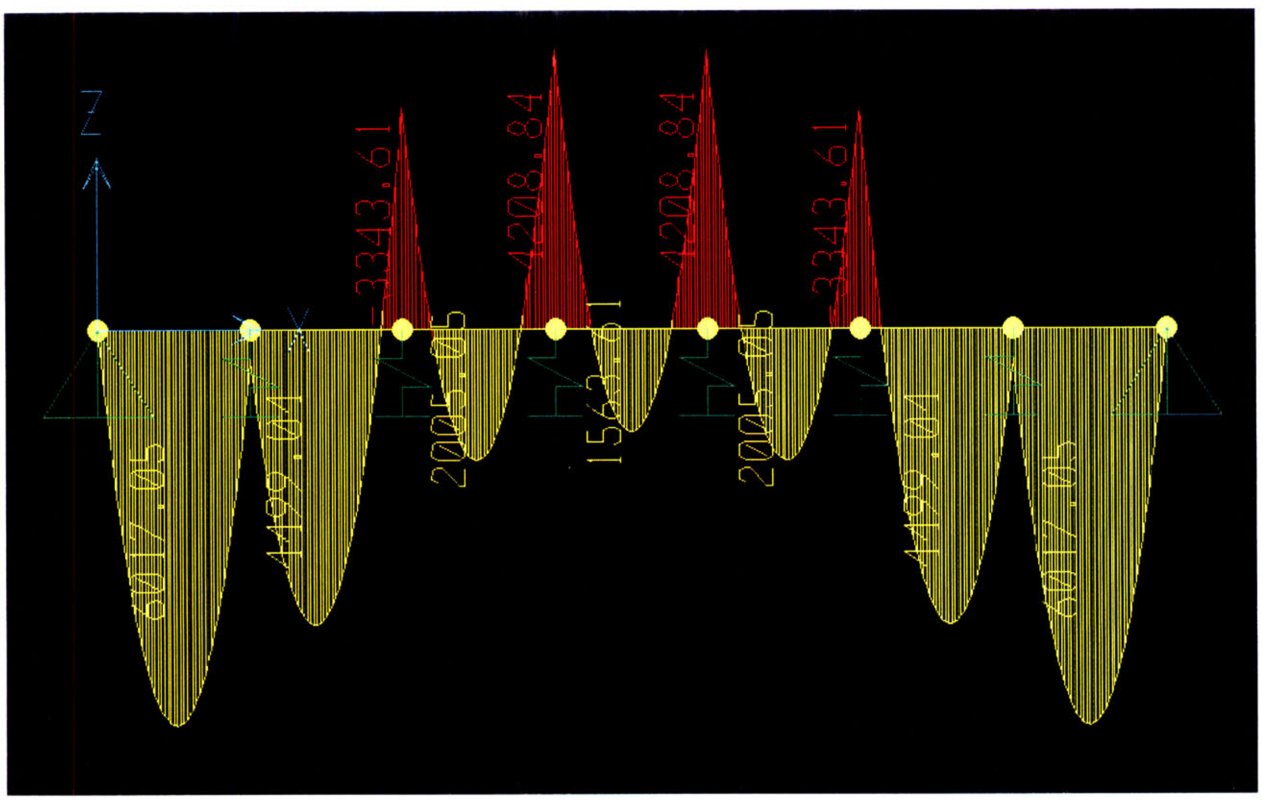

FIGURE 29: MAXIMUM MOMENT EXCEEDING CAPACITY

The beam is continuous between supports, but the supports are flexible; thus it is also affected by differential settlements of supports, and the magnitude of the additional moment is given by:

$$M_{diff.sett} = \frac{6EI * differential\_settlement}{L^2}$$

Of course, since the end supports are rigid and the cables expand, the maximum differential support settlement happens in that zone closes to the rigid supports.

The design at this point had to be adjusted such that the maximum moment was brought within allowable limits. The second span was not the only one under concern; when the bridge is loaded such that one span is under a full load and the surrounding two are not loaded differential deflection of cables will occur and again the moment in the beam will be increased. This is a likely scenario if there is a red light before the bridge and when the light turns green a platoon of vehicles equivalent to a span spacing will travel across the bridge together.

Since the inertia of the equivalent beam could not be increased, two other methods were identified to mitigate this issue. Cable spacing could be reduced (costing more because more cables will be used to span a single crossing) or stiffness could be increased (costing more because the area of the cables will be increased).

It was selected to increase the stiffness until the moment was reduced to the allowable limit. The necessary area of the cable was then computed using:

$$A_c = \frac{K_v x_d}{E_{eff} \sin^2(\Theta)\cos(\Theta)}$$

x_d =100m to be conservative

The resulting required cable diameter was approximately 250mm which, unfortunately, exceeds the typical largest cable in the industry.

The following approaches were considered to reducing the moment: the cable angle could have been increased because the cables were not being used effectively or the cable spacing could have been changed. Since the goal is to maximize the span which will most likely be controlled by the tower buckling, the tower has to remain as short as possible. Therefore, it was selected to keep the angle at approximately 20 degrees and reduce the spacing.

The spacing was reduced by 1 bay to 7 bays (21.336m); also stiffness values were adjusted so that each cable area was the same. Maintaining the same cable area and incorporating a variable value for x_d instead of a worst case scenario resulted in a stiffness which increased for each cable as they got closer to the supports. This step ensured that the deflection of L/800 was maintained for a bridge with any number of bays. This was not only helpful at mitigating end moments but it was also an important solution in terms of modularity. Equal cable sizes gave a variable stiffness which was a function of x_d, distance from the origin; however, since the deflection limit is also a function of x_d (span = 2* x_d), the L/800 criteria was met for any bridge span.

The combination of reducing spacing and adjusting stiffness values such that x_d was accounted for lowered the moment in the section to allowable limits. Furthermore, the beam was checked under asymmetrical loading for maximum positive and negative moments inbetween the cables and this was also within the allowable 4100kNm limits (Figure 29).

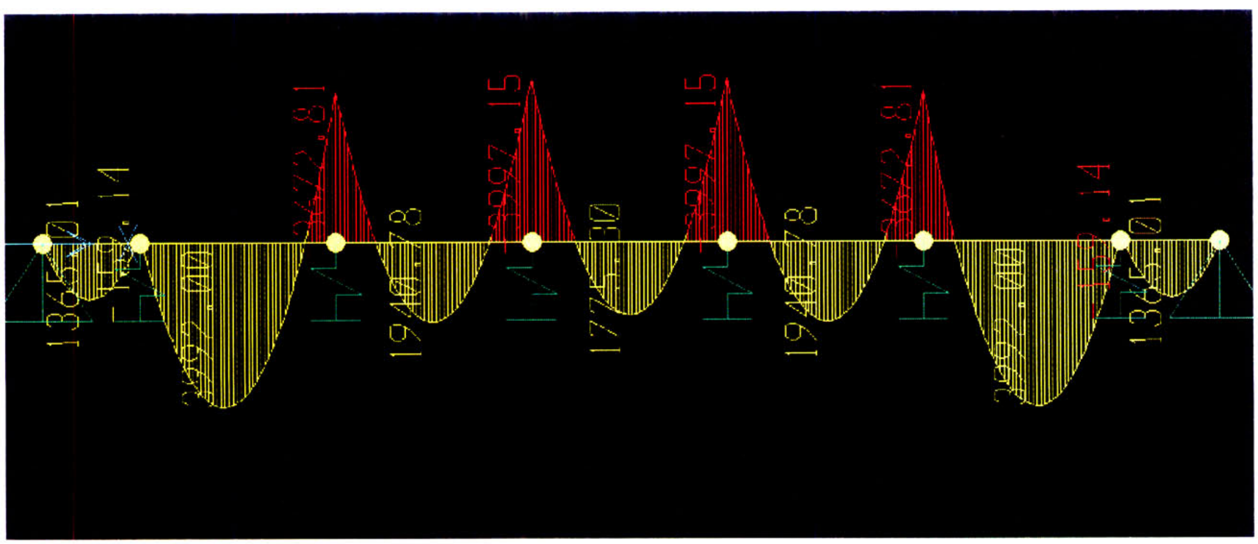

FIGURE 30: MOMENTS BROUGHT WITHIN ALLOWABLE 4100KN*M RANGE

3.7 CABLE AND DECK ANALYSIS: ADDRESSING MODULARITY

Once the general strategy (as described in Section 3.4 above) was determined, modularity had to be addressed, particularly looking at the 2 lane bridge since the preliminary design was done for a 3 lane bridge.

The two lane bridge carries less loading per panel; therefore the maximum possible cable spacing based on section capacity is larger than for the three lane bridge. Based on looking just at the moment due to distributed load on a continuous section the maximum spacing was computed to be 10 bays (30.48m). However, from previous discussion, differential settlement of supports had to be considered, and so reducing the maximum spacing to 9 bays would probably resolve the moment capacity as it did for the three lane bridge.

Recalling modularity design criteria, particularly keeping as many elements as possible to be interchangeable, the cable lengths were examined. Since various spans lengths were already addressed, the goal now was to make the cable spacing such that the cables could be interchanged between the two and three lane bridge.

Seven and nine are not a multiple of each other, and even if they were anchored to the pylon at the same angle, the same cable length would only occur in every 63rd cable.

The factor of safety concept for cables was brought in at this point. This is an important design issue, since the factor of safety for cables is quite high – values anywhere in between 2 and 5 are recommended. Because the cable design was governed by MBD, the safety factor was determined as allowed deflection divided by existing deflection.

Iteratively working with attempting to synchronize the spacing of the two bridges to a multiple of each other and providing a safety factor of 2.5 or more, cable spacing was selected for the two bridges such that the cables were as interchangeable as possible. An Excel spreadsheet that computed vertical stiffness and maximum spacing based on every available cable diameter was used to select an adequate cable area.

The results were 143mm cables spaced at 4 bays 12.192m for the 3 lane bridge and same cable diameter spaced at 8 bays 24.384m for the 2 lane bridge (Figure 30). This resulted in a factor of safety of 3 for the 2 lane bridge and 4.5 for the 3 lane bridge.

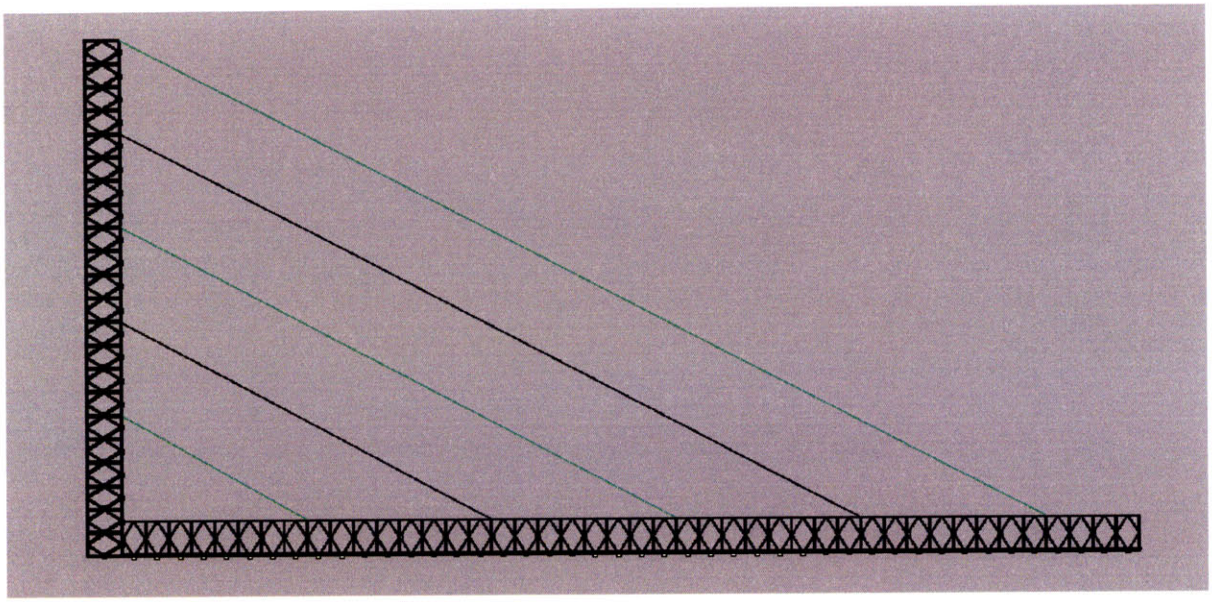

FIGURE 31: CABLES FOR 3 LANE AND 2 LANE BRIDGE FULLY INTERCHANGEABLE

3.8 CABLE AND DECK ANALYSIS: DETAILED DESIGN CHECK

Once the spacing was set, a detailed design check was performed. In addition to analyzing stresses under a distributed load, both of the bridges were modeled for a 116kN load for shear and 80kN load for bending moment in the effective beam section.

Furthermore, a moving truck load with a varying cab as described by AASHTO was applied on the bridge. The effective beam was well within capacity and the factor of safety barely changed for the cables.

The shear diagram was examined in detail and the shear capacity of the AB701 panels (532kN) was exceeded near the cable connections of the 2 lane bridge by approximately 100kN; therefore AB702 high shear panels (capacity of 800kN) should be used in the two bays around the cable supports.

3.9 LATERAL LOAD AND TOWER ANALYSIS: MAXIMUM SPAN

Detailed lateral load and tower design analysis were not feasible because the exact section sizes or inertias of panels in the lateral direction was not provided in the Acrow Manual and was not possible to obtain because of proprietary information.

Instead, the following strategy was employed to estimate lateral inertia and to then check wind loads, tower buckling, and dynamic stiffness of structure to see how those related to span length.

LATERAL AND TOWER INERTIA

The lateral moment of inertia of the bridge was not provided in the Acrow Manual; however, the weight of the panel and its height and width dimensions were known. The panel was approximated as a rectangular plate (Figure 31) with the same dimensions as the truss in height and width, and the mass of the panel was divided by the density of steel to find the volume of steel and then divided by the height and width to find an effective thickness of the rectangular plate.

The deck thickness and tower moment of inertia were estimated in the similar manner; the secondary floor beams did not contribute to the moment of inertia of the deck section.

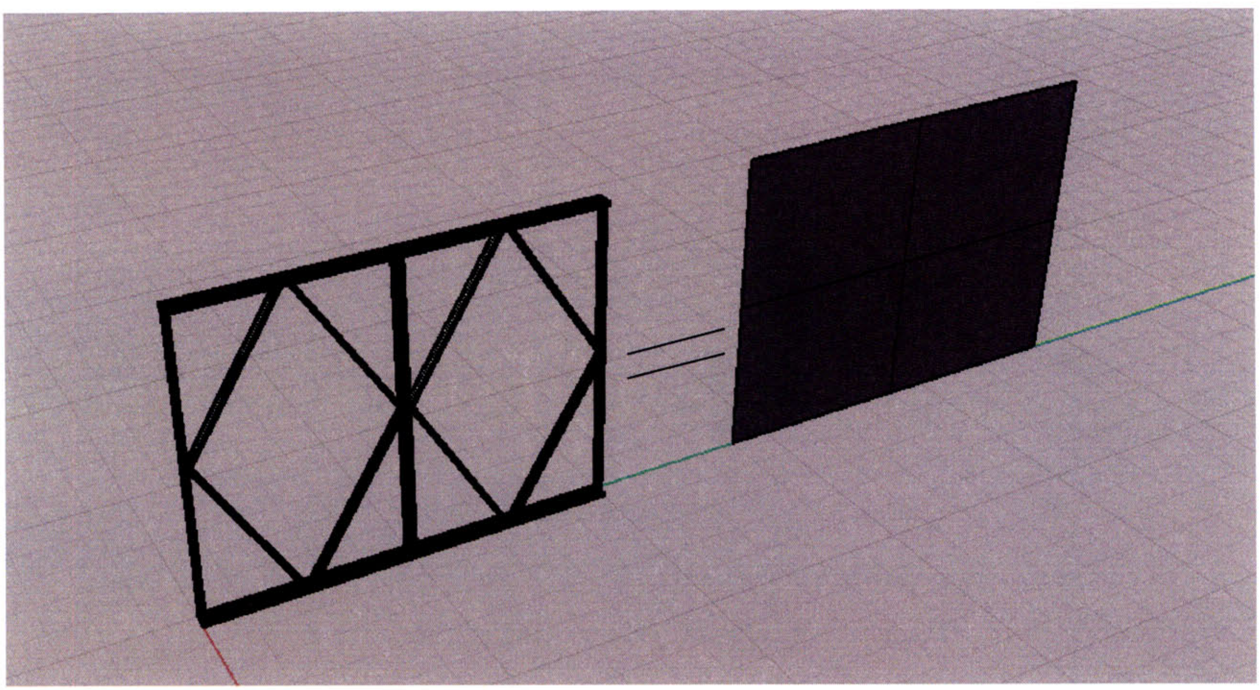

FIGURE 32: PANELS APPROXIMATED AS EFFECTIVE PLATES TO FACILITATE INERTIA CALCULATIONS

WIND LOAD ANALYSIS

The bridge was approximated as an equivalent beam with the lateral moment of inertia estimated as described above. The deflection of the section was limited to L/800 and the end conditions were assumed pin-pin to be conservative. This resulted in maximum allowable span lengths of over 2000m for each bridge.

TOWER BUCKLING

The towers can be assembled out of the Acrow panels as demonstrated in Figure 32.

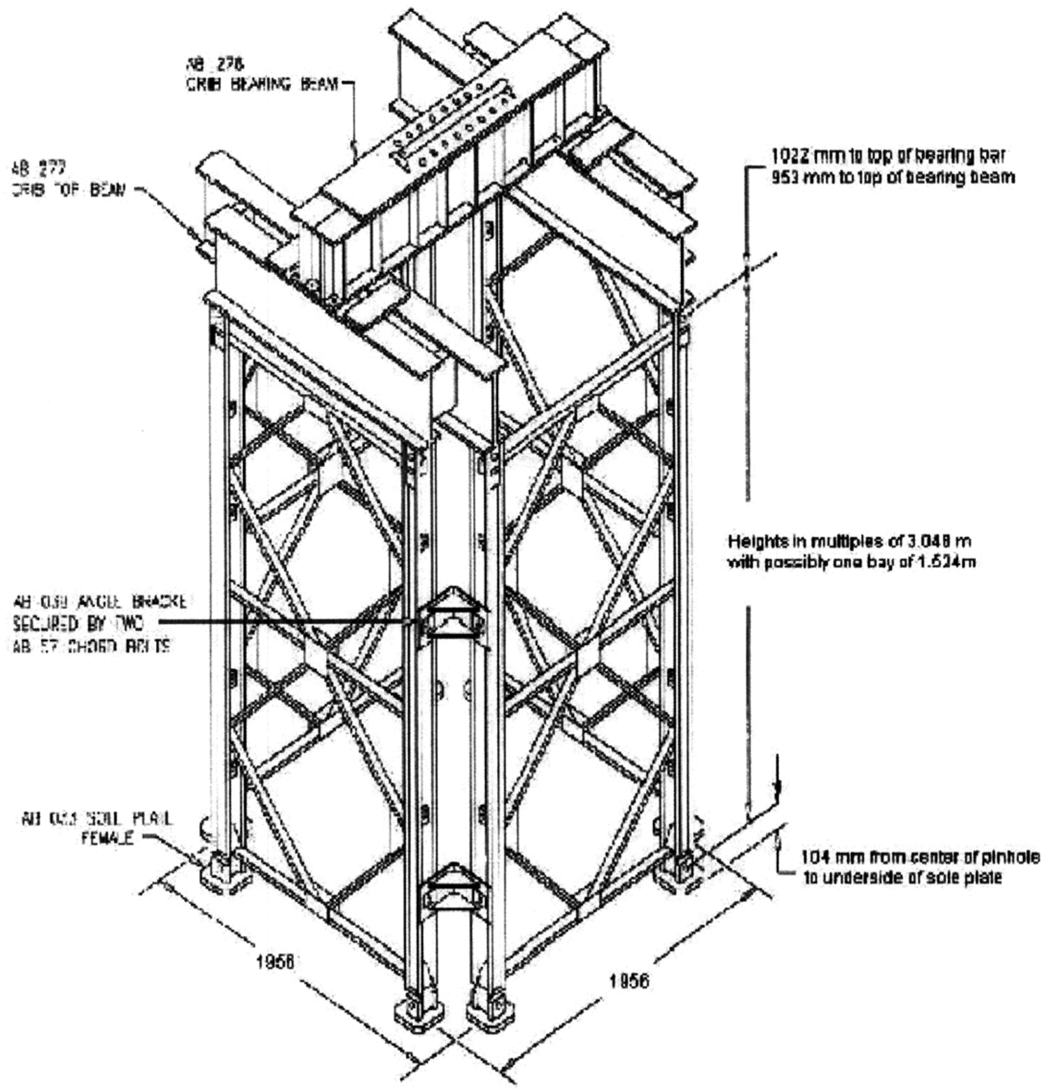

FIGURE 33: TOWER OUT OF ACROW PANELS [1]

The height of the tower is directly related to the maximum span that can be achieved because of the harp cable configuration and the fact that the cables lose their efficiency after a certain angle; therefore the longer the span needs to be the higher the tower should be. The height of the tower was assumed to be governed by buckling under maximum compressive load which was equal to fully factored lane distributed load multiplied by total span and divided by two towers (one on each side).

Based on tower buckling, a span of 400m can be achieved in a 3 lane bridge with just using single-single configuration of panels. More can be done with a different configuration. The 2 lane bridge can span even longer.

This dimension translates back to a 200m tall tower. This number seems a bit high, especially since the Acrow Manual specifies that the maximum length that can be achieved for an unsupported tower is approximately 18m.

Another approach was attempted. The maximum axial load of the tower was given by the Acrow Manual as 3558kN. This value (multiplied by 4 towers) was compared to the maximum load the bridge would carry under each span length. Thus a maximum span of 277m for a 2 lane bridge was computed; this corresponded to approximately 90 bays (5 cables on each side). Similarly, the maximum span for a 3 lane bridge was computed to be 183m or 60 bays (6 cables on each side).

Towers of adequate length (approximately 130m) to provide the maximum span were assumed to be feasible with built up panels (similar to the idea of double-single configuration where another set of panels gets added on to double the inertia).

DYNAMIC LOADS

To give a rough approximation of the effect of dynamics on the structure, the natural fundamental period of each bridge for each span was computed. The main concern was the structure becoming too soft with increasing span and thus having the natural frequency match the forcing frequency of wind gusts thus bringing it to resonance.

The period for which the structure was considered too soft was taken as 3 seconds because wind gusts are usually anywhere between 3 and 6 seconds.

The bridge was modeled as a continuous beam with the same lateral moment of inertia as the one computed for the wind loads with a span in between the pylons. Lateral cable stiffness was considered to be negligible. Both fix-fix and simply supported ends conditions at pylons were examined.

The 2 lane bridge became soft (T = 3.2sec) when the spans went over 250m (84 bays). The 3 lane bridge had a much bigger inertia because it had a wider steel deck; it only reached a period of 3sec at approximately 300m.

Detailed calculations of this section can be examined in Appendix A.

3.10 ANCHORAGE OF BACKSPAN AND CONSTRUCTION SEQUENCE

The anchorage of backspan for a cable-stayed bridge involves intensive foundations. Because foundations are typically difficult to make modular and reusable, this could be used as an argument against using a modular bridge as a traffic detour scheme. Therefore, a modular foundation design is presented below. Although a detailed numerical calculation is beyond the scope of this thesis, some design ideas and a qualitative analysis are included.

Typically, a foundation that provides anchorage for this type of a scenario consists of a large concrete block buried underground. The vertical load to resist, which is equal to the span loading, is balanced by the weight of the block and the soil above it. The lateral load to resist depends on the angle at which the cable is attached and is balanced by the lateral soil pressure or by a strut between the deck and the anchorage point as shown if Figure 34 (this would reduce the foundation requirements at the tower and at the anchor point).

A way to make this modular is to replace the concrete block with a container out of steel elements that can be assembled and disassembled easily and fill the container with a material that can be removed and reused. The idea is that company like Acrow would own this modular foundation or portions of it and it would be part of the rental.

A few materials can be considered. Water can be used if a watertight container is provided; on river bridge replacements it is available locally and free. Monitoring the water content to make sure it is not decreasing would be necessary but since the bridge is temporary and there is a construction crew always present during its use that should not be a problem. Reusable pre-fabricated concrete blocks (more dense than water), steel plates (more dense than concrete), or even led blocks (more dense than steel) can be used.

Another material option could be the rock that is usually brought in to cover the banks and the head-slopes as an erosion protection measure. This is usually the final step in constructing a bridge, thus it would not be needed until after the detour bridge is disassembled.

A reasonable size of container (based on various parameters such as transportation, ability to handle with a crane, etc.) would then guide the material choice based on material densities.

Two schemes for anchorage using modular containers are proposed. The first one illustrated in Figure 33 would use a single anchor cable. For this scheme, the container size for the foundation would vary based on the bridge span.

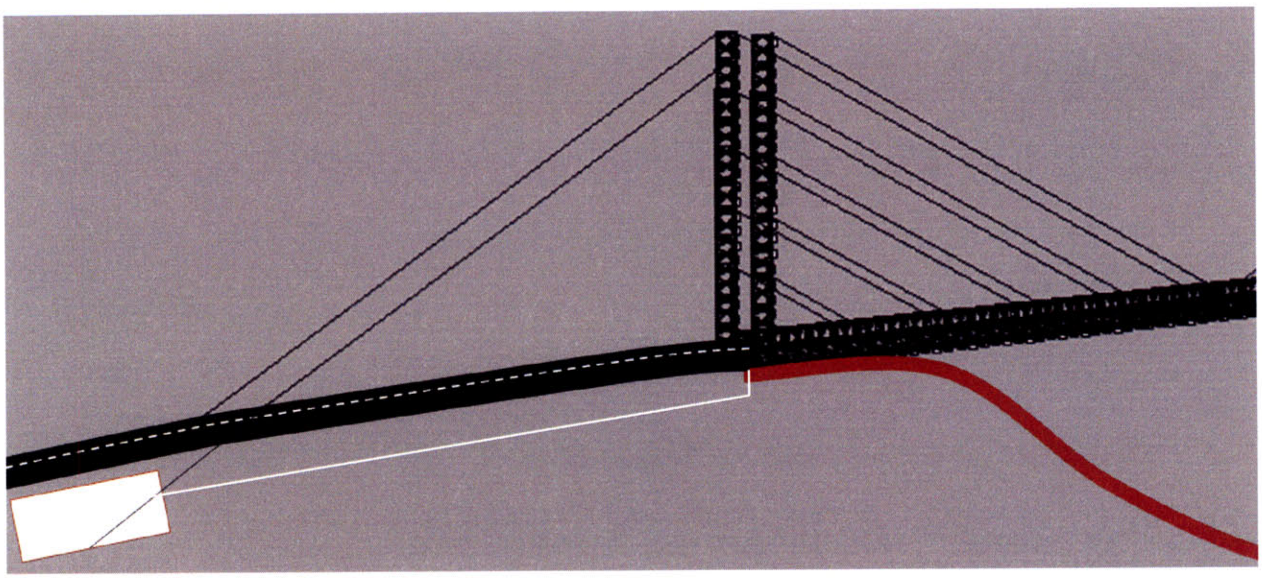

FIGURE 34: ANCHORING SCHEME WITH ONE CABLE

The second scheme, demonstrated in Figure 34, would be using multiple anchoring cables and thus multiple containers. Since each container can be sized exactly to support the maximum load from each cables tributary area, this scheme would be a more modular approach but require more cables.

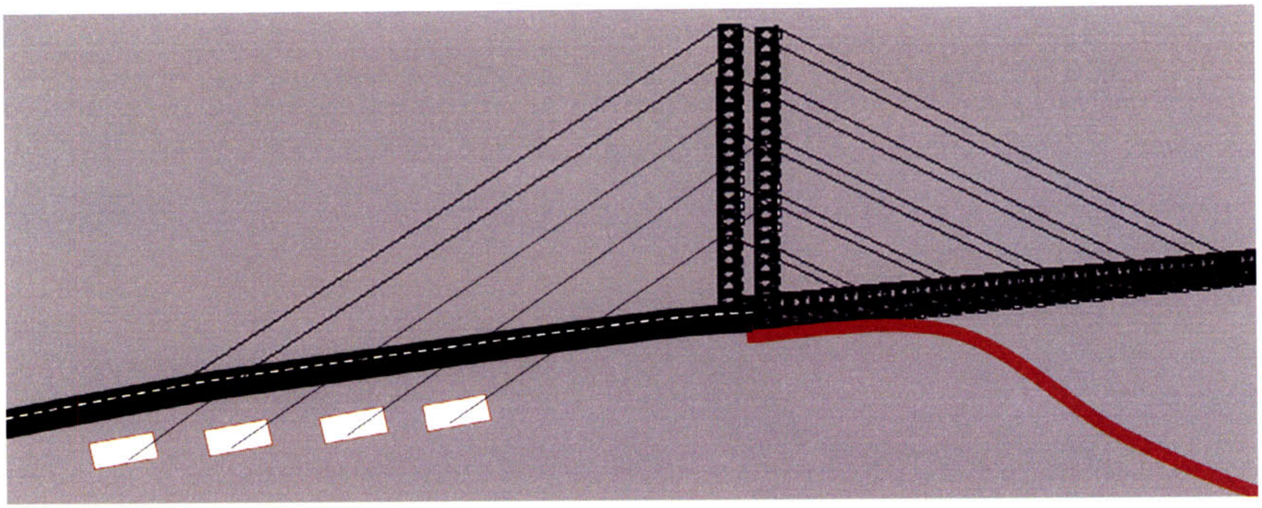

FIGURE 35: ANCHORING SCHEME WITH MULTIPLE CABLES

4.0 CONCLUSION

The oncoming large amount of bridge replacements in the next 10 to 20 years called for a detailed examination of available replacement schemes which can have variable impact on user costs. Detouring traffic with a modular bridge proved to be the most desirable scheme in terms of user costs such as traffic delays, detour distances, ultimate highway geometrics, construction crew safety, and safety of drivers.

Criteria that encompassed modular bridge design were defined and two companies in North America – Acrow and Mabey – were found to provide bridges within those parameters. A brief analysis of Acrow bridges showed that maximum span lengths range in the order of 100m; this is fairly short compared to spans of many bridges that will be have to soon be replaced.

The current bridge system with which modularity is achieved is a set of truss panels which are supported by abutments or piers. In order to span crossing over 100m, piers would have to be placed in the channel or on the head-slopes which was proved to be a costly and undesirable construction process. Therefore, a modular bridge which could achieve longer spans was proposed for a 2 lane and a 3 lane wide bridges using as many existing Acrow components as possible.

The scheme encompasses a harp cable-stay bridge with cables spaced and sized such that they are fully interchangeable between the various bridge widths and can be built up to any span. These cables are the only additional component as the towers, the girders, and the decks are all made out of existing Acrow components. The pylon is balanced with an anchoring cable and ideas for modular foundations for the anchor are presented.

Not only does the design fully reflect modularity, it also achieves a longer span with only the addition of cables and modular foundations to the existing inventory. A span of approximately 183m is possible for a 3 lane bridge; this is limited by the maximum axial tower capacity. The 2 lane bridge can span up to 277m based on tower capacity, however it behaves like a "soft" structure in terms of stiffness when spans reach beyond 250m (the period T is over 3sec). Thus with an addition of only cables to the inventory, maximum attainable spans can be doubled. This makes the use of modular bridges more cost effective in location with a wider crossing and thus more likely to be selected as the replacement strategy which would drastically ameliorate user costs associated with bridge replacements.

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APPENDIX A: MAXIMUM SPAN CALCULATIONS

Dynamic Analysis

# Lanes	Deck Width	Weight tonnes/bay	Effective Thickness(m)	Total Inertia m^4	Area
2	7.31	0.721	0.014971428	1.613801976	0.2426357
3	10.96	0.721	0.014971428	4.541168374	0.3338857

E	2.00E+11	Pa
Steel Dens	7847	kg/m^3
Bj - simply s.	3.141593	
Bj - fixed	4.712389	
2 lane	T (fixed)	T(s.s)
12.192	0.00323	0.007268
36.576	0.029072	0.065413
60.96	0.080756	0.181702
85.344	0.158282	0.356135
109.728	0.261651	0.588714
134.112	0.390861	0.879436
158.496	0.545913	1.228304
182.88	0.726807	1.635316
207.264	0.933543	2.100472
231.648	1.166121	2.623773
256.032	1.424542	3.205219
280.416	1.708804	3.844809
3 lane		
6.096	0.000565	0.001271
18.288	0.005083	0.011436
30.48	0.014118	0.031766
42.672	0.027672	0.062261
54.864	0.045743	0.102922
67.056	0.068332	0.153747
79.248	0.095439	0.214738
91.44	0.127064	0.285894
103.632	0.163206	0.367215
115.824	0.203867	0.4587
128.016	0.249045	0.560352
140.208	0.298741	0.672168
152.4	0.352955	0.794149
164.592	0.411687	0.926295
176.784	0.474936	1.068607
188.976	0.542704	1.221084
201.168	0.614989	1.383725
213.36	0.691792	1.556532
225.552	0.773113	1.739504
237.744	0.858952	1.932641
249.936	0.949308	2.135943
262.128	1.044182	2.34941
274.32	1.143575	2.573043
286.512	1.247485	2.80684
298.704	1.355912	3.050803
310.896	1.468858	3.304931

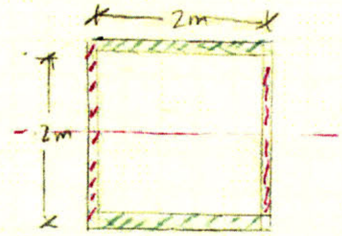
Tower Design

$P_{cr} = \frac{\pi^2 EI}{L_{eff}^2}$

variable — variable

$L_{eff} = L$ assume pin-pin (provide lateral strut @ top)

$E = 210$ GPa



inertia of pink sections about pink axis is given for bridges

Inertia of the green

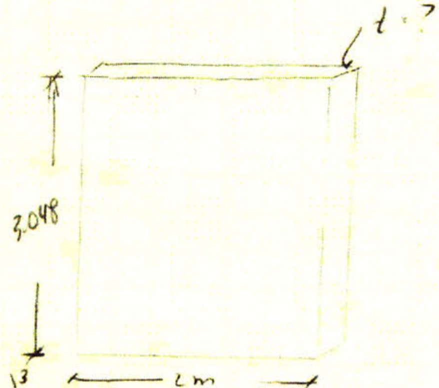
· have to back calculate because I don't have the info

approximate as rectangular plate

weight of panel

(SS) = .721 tonnes

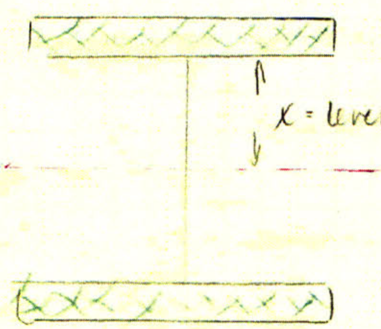
steel density = 7.9 g/cm³



$\therefore$ Volume steel = $721\,kg\left(\frac{1\,cm^3}{.0079\,kg}\right)\left(\frac{1\,m}{100\,cm}\right)^3$

$V_{steel} = .09127\ m^3$

Thickness of effective plate = $t = \frac{.09127\,m^3}{3.048\,m \times 2\,m} = .015\,m$



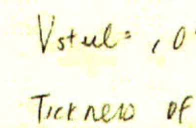
$I_{green} = \cancel{\frac{1}{12} bh^3}^{\sim 0} + Ad^2$
$+ (t \cdot 2m)(1m)^2$

Total Inertias tabulated in Excel

$L_{eff} = \sqrt{\frac{\pi^2 EI}{P_{cr}}} = \sqrt{\frac{\pi^2 \times 210\times10^9 \frac{N}{m^2} \times I\ m^4}{W_{FACTORED} \times \frac{SPAN\ (kN) \times 1000}{2} \times 2}}$

2 towers one per side

backspan anchor



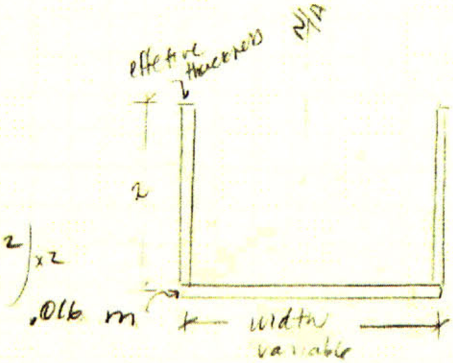
Max span = $2 \times L_{eff}$

Based on 3558 kN ⇒ span = 3558 kN × 4 towers / $W_{factored}$

2 lane — $w = 51.4$ kN/m — span 277 m ≅ 90 bays ≅ 11 cables ≅ 5 on each side

3 lane — 183 m — 60 bays

Lateral Inertia



width of deck 2lane : 7.31m
3lane : 10.96m

$$I = \frac{1}{12}(.025)(\text{width})^3 + \left(2 \cdot \text{thick eff} \cdot \left(\frac{\text{width}}{2}\right)^2\right) \times 2$$

thick-eff of SS : .015 m

$I_{2lane} = .92 \ m^4$

effective thick = $\frac{\text{panel weight} \times \text{density}}{\text{height} \times \text{width}}$

$I_{3lane} = 2.65 \ m^4$

Wind

$$\frac{L}{800} > \frac{5wL^3}{384EI} \Rightarrow span_{max} < \sqrt{\frac{384EI}{800 \times 5 \times w}}$$

$\Delta d_{max} = span/2$

$w \Rightarrow$ using conservative AASHTO - because any site...

$w = 300$ plf

$$300 \frac{lb}{ft} \left(\frac{1N}{.2248 lb}\right)\left(\frac{1ft}{.3048m}\right) = 4.3 kN/m$$

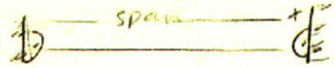
$L < 2076$ m (2 lane)

$L < 3000$ m (3 lane)

Dynamic Analysis

Dynamic

checking the fundamental mode for the span that would give a soft structure $T > 3-4$.



assume pin-pin

$$\frac{2\pi}{w} = T$$

$$w_j = \frac{\beta_j^2}{L^2}\sqrt{\frac{EI}{\rho A}}$$

$E = 200\,GPa$

$\rho_{STEEL} = 490\,\frac{lb}{ft^3}\left(\frac{1N}{.2248\,lb}\right)\left(\frac{1ft}{.3048m}\right)^3 \times \left(\frac{s^2}{9.81m}\right) = 7847\,\frac{kg}{m^3}$

L = variable

$\beta_i = (j=1) \cdot (j + 1/2)\pi = 3\pi/2$ (fixed-fixed) $\quad j=1$

$= \pi$ (if simply supported)