CIVIL ENGINEERING PRACTICE •

JOURNAL OF THE BOSTON SOCIETY OF CIVIL ENGINEERS SECTION / ASCE

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Otis Reservoir Dam / Bridge Rehab

Curved Girder Bridges in Vermont

Remembering Don Goldberg

Volume 29, Number 1

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President's Message

Into the Unknown

The future is, by definition, unknown…what is still to come but not yet. These days it seems as though the mystery of that later time is stretching back to overshadow the present. Traditional approaches to classic problems have always been challenged by subsequent generations, fostered by human ingenuity to be faster, easier, more efficient. But now, better begins to mean more informed, more conscientious, more resilient. We most likely started with a focus on sustainability, but seem to have lost some of that attention along the way.

We, as engineers, work to establish solutions to critical system concerns. According to Merriam-Webster, we carry through an enterprise by skillful or artful contrivance. And our calling is most often born out of necessity. Many of a community's basic needs are fulfilled by civil engineering works, including access to clean water, proper sanitation, safe transportation and reliable communication. All of the systems we design and contribute to require expertise to support, operate and maintain. They require continuous investments in time, energy and resources. And they require competent leaders to champion those efforts.

But the empirical "constants" that our predecessors have employed to solve for their desired variables are changing. The tools have evolved through exponential advances in technology. How we work has been altered by the threat of pandemic disease. The factors that affect change are taking new shapes. So, too, must we adapt and improve. We have a duty to utilize the vast foundation of knowledge that those who came before us have painstakingly built, and advance our own understanding of the impacts we effect on the natural environment and ourselves.

As we embark on this second edition of the third incarnation of the Civil Engineering Practice Journal, there are many hopeful signs on the horizon.

With President Biden's $1.2 trillion dollar bipartisan Infrastructure Investment & Jobs Act (H.R. 3684) recently passing the Senate of the U.S. Congress, we are on the eve of one of the largest federal investments in our nation's roads, bridges, airports and waterways.

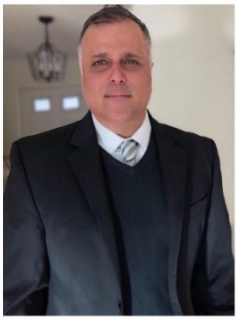
Building on the long history of surveying wastewater for viruses, civil engineers are currently collaborating with biological engineers to track SARS-CoV-2 and generate actionable information.

As evidenced by the thoughtful articles that follow, our local community is striving to share the valuable lessons they have learned on improving design accuracy, managing cost, and providing observed data to support future innovations. We also pay tribute to one of our brightest whose legacy lives on.

And lastly, many thanks to our Editor-in-Chief, Dr. Gautham Das and the volunteer editorial board for all their tireless efforts to produce this next issue of the journal. Your dedication and generosity set an example for the rest of us to follow.

Shallan Fitzgerald, PE
President of Boston Society of Civil Engineers, ASCE Section
(2021-2022)

Message from the Editor-in-Chief



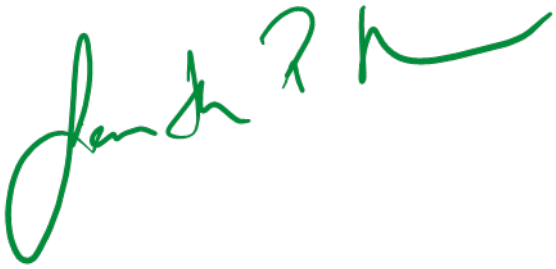
I am happy to announce the 2nd online edition of *BSCES Civil Engineering Practice.* This peer reviewed journal, has been in publication for over 100 years, includes innovative research and practice-oriented work in the field of civil engineering.

The BSCES Journal has been a repository of the amazing work that has been done in the New England/Boston Area. This journal is targeted for working professionals and for academics who want to publish their innovative projects and research with applications in civil engineering.

With the Covid-19 pandemic continuing to surge in the US and all over the world, civil engineering, construction, and infrastructure renewal/maintenance are essential to the safety, and economic well-being of our communities. *BSCES Civil Engineering Practice i*s committed to keeping the profession informed during this unprecedented period.

We hope *BSCES Civil Engineering Practice* will become your primary platform to share findings and discuss all aspects of civil engineering in the development of future innovations that will benefit the community. We are now welcoming submissions for future issues of the journal

Sincerely,



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Remembering Don Goldberg

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Abstract

Don Goldberg, co-founder of GZA GeoEnvironmental, Inc. (GZA), passed away on February 23, 2021 at the age of 92. He was one of the true pioneers in the field of geotechnical engineering starting his own consulting firm in 1964 when there were few other geotechnical engineering firms in the country. Starting out locally in the Boston community, his impact ultimately extended far beyond Boston and far beyond geotechnical engineering. Over the course of his career, he touched thousands of lives and forged a path for thousands in developing their careers within geotechnical, geoenvironmental and construction practice throughout New England, and in the Mid-Atlantic and the Great Lakes regions. Throughout his 30 years of practice, he built an enduring legacy of excellence, trust, and doing the right thing for his employees, clients and community at GZA, which continues to thrive today with 700 employees operating out of 30 offices around the country.

Keywords: Geotechnical Engineering

1. Early Life and Education

Don was born in Winthrop in 1928 and attended Tufts University (then College) in the 1940's with the original intent of becoming a doctor. His professional focus shifted after graduation and eventually pointed him in the direction of MIT. His initial interest had been aeronautical engineering, but a few of his classmates suggested that civil engineering was where it was really at. One of those was Bill Zoino, a commuting buddy from Brockton. Don received a Bachelor of Science in Civil Engineering in 1954 and a Master of Science in Civil/Geotechnical Engineering in 1957 from MIT.

2. Pre-GZA

Don spent his early professional years at Mueser Rutledge in New York and Haley & Aldrich (H&A) in Cambridge. Harl Aldrich was an early mentor. At the time, H&A was the only geotechnical firm in Boston and geotechnical consulting was in its infancy as a profession. In the fall of 1964, Don left H&A to pursue his desire to start his own firm. He reached out to Bill Zoino, who was working for Ebasco at the time in New York City to join him, and they co-founded Goldberg-Zoino & Associates (GZA). It was a big risk personally and professionally, but he always said he figured that if it didn't work out, he could always get another engineering job somewhere. It did work out.

3. GZA

GZA quickly built a reputation for providing clients with innovative and practical solutions. Don's willingness to pioneer the use of cutting-edge geotechnical engineering solutions and new technologies solidified GZA's position as one of the top firms in the profession. The MBTA Red Line subway extension in Cambridge, Massachusetts, a cut-and-cover deep excavation in sensitive clay, was one of those defining projects. Don proposed the first-ever use of slurry walls for both temporary excavation support and permanent support of basement walls. He applied new technologies in below-ground construction, such as first use of tiebacks to support deep excavations in soft soils, and the installation of "record" quantities of lightweight concrete fill to relieve load and stabilize sea walls/basement walls for Commonwealth Pier (now Seaport World Trade Center) in Boston. Many of the technologies first utilized on Don's projects are now standard practice in the industry.


Figure 1. Don Goldberg 1960

Don also was lead author of a 1976 FHWA technical manual called "Lateral Earth Support Systems and Underpinning." The document has received world-wide acceptance and remains a fundamental resource for geotechnical and structural engineers today.

For those who worked with him at GZA, Don was perhaps the brightest, most accomplished, most tenacious, and most complete individual they had ever known. He was held in the highest regard by all those around him. He always gave his best and brought out the very best in others.

From an engineering standpoint, Don was exceptional and his understanding of engineering principles and their practical application was phenomenal. Typically, in any meeting with a group of engineers, his technical knowledge and understanding of the project surpassed all. He was not only brilliant but quick witted. He was hard to keep up with (and always with just a pencil, paper and slide rule). He set the standard for technical excellence and professional practice at GZA. Anyone, who worked with him knew they would be pushed, and no one ever wanted to let him down. But they also knew that if they faltered, he always had their back.

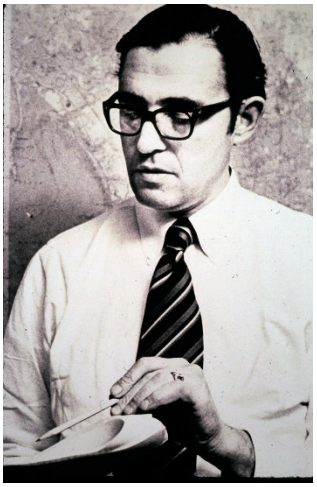

Figure 2. Don Goldberg at GZA

From a business standpoint, he was always thinking of the next thing, looking for the next wave. He felt the firm had to keep trying new things so that it could thrive. With Bill Zoino, and later with John Ayres, he constantly moved the company in new directions and typically was not afraid to be the first to try something new and in doing so, they established a geotechnical instrumentation division and were the first local geotechnical firm to provide environmental engineering/consulting services focused on hazardous waste studies and remediation (1970s); they became one of the first consulting firms in the Northeast to offer in-house test boring and environmental laboratory services (1980s); and the first consulting firm in Boston to perform dynamic pile testing when that was a brand new field in the 1980s.

But underlying that drive was a strong commitment to family, community service, and professional service. Before "work-life balance" was a term, Don would always try to be home by dinner time to spend time reading with the kids, helping with homework, doing household chores, and just being there for his family. He encouraged his employees to put family first. He initiated GZA's first charitable giving program and encouraged participation by all employees. In his time at GZA, Don served as President of the Massachusetts Section of the American Society of Civil Engineers prior to its merger with the BSCES. He served on the Board of Directors of both organizations. He served as chair of the Structural and Geotechnical technical groups of BSCES. He served as President of the American Council of Engineering Companies of New England and was on its Board of Directors. Don was also a founding member of The Engineering Center in Boston and served a term as its President.

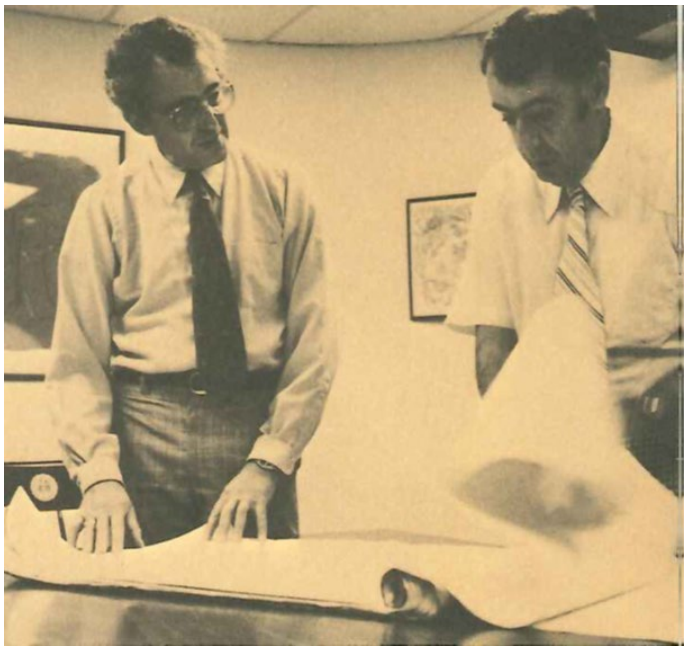

Figure 3. Don Goldberg and Bill Zoino

Figure 4. Don Goldberg at job site.

But Don's most enduring impact, and the key to the long-term success of the company, is the culture that he and Bill established. So much of the culture was and still is – even after all these years - a reflection of Don's character, personality and extremely strong moral compass. He believed that doing the right thing was more important than maximizing profits. He cared totally and thoroughly about GZA's employees first and foremost and in providing opportunities for GZA staff to grow and advance their careers without having to go elsewhere. He mentored them, worried about them, helped them and taught them. He believed that what we accomplished at GZA was always a team effort. He hated to hear someone say, "I did this" or "I did that" or "my client." For him, it was always "we did this" and "our client." He

demanded that his employees show respect for others and operate always with integrity. He believed that clients expected us to do more than just deliver the expected result but to deliver an unexpected and better result. Don was especially good at that.

Those core values of operating with integrity, thriving on challenges, providing opportunities for all, caring for our communities, and supporting employee ownership, are still GZA's Core Values today - 57 years later.

In recent years, Don along with Bill (and their wives and families sometimes) visited with the company on a number of occasions so new generations of GZA employees could get a chance to know them. Over time, it began an annual tradition which we now call "Founders Day" – a day when Don, Bill, their families, and other key contributors to GZA visit the company and discuss its roots. It is always like a visit from engineering royalty. Everything in the company came to a halt. At moments like these, they were able to directly impart the meaning of GZA, and we most felt the connection with them.

Figure 5. Don Goldberg at Engineering Center, Boston.

3. Family and Legacy

Don leaves behind his wife Maxine, his daughters Anne and Lisa, son Jonathan, stepchildren Robert and Laura, thirteen grandchildren, two great grandchildren, and the loving memory of his first wife Joan who was an important part of GZA in its first three decades before passing away of cancer in 1992. He also leaves behind a legacy of the 700 people that currently work at GZA and the hundreds of others who have passed through its doors – and have been made better professionals by the example he provided and the core values he instilled in all of us.

On behalf of grateful professionals who have had the pleasure of being mentored by you and working for the firm you and Bill established in 1964 – thank you, Don.

Figure 6. Don Goldberg

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BIM Quantity Take-offs Analysis for Accurate Estimates of Concrete Volume in Buildings

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Abstract

Precise design of concrete parametric objects for building concrete structures using Building Information Modelling (BIM) provides a new basis for cost estimation and accurate determination of concrete quantities needed in the construction phases. Most BIM software available on the market nowadays offer features, which has the ability to enhance the materials take-off/schedules through acceptable and flexible modifications, as they may occur at the progressive stages of the 3D design model during the pre-construction phases or even more during the building lifecycle. Additionally, the concrete quantities may be assembled based on the concrete type, construction elements, building levels and other criteria. Authors of this manuscript were examining three-dimensional models of structurally different commercial buildings, in order to compare, extract and compute only the quantities of all concrete elements involved in the construction of foundations, columns, framing elements, floors, and walls. The extraction of these quantities is helpful to a construction contractor in determining the cost estimates of all concrete elements and for further analysis on the variances of these quantitates with the final quantities (volumes) which are placed in the buildings during the construction phases. For the three cases selected, the findings revealed a relatively close range of accuracy of positive or negative one to three percent, and therefore it allows generation of savings during procurement process for all stakeholders.

Keywords: Estimating Accuracy, Quantities, Concrete Structures, Variance, BIM

1. Introduction and background for this study

Quantity take-off (QTO) is becoming known as an exhausting work process that requires a long time to be completed. In the traditional way of preparing the estimating sheets and the quantity take-offs in general, both processes need to be done by the same person or at least someone who is using the same procedure because there are different ways of reading 2D plans in order to obtain the materials quantity take-off sheets. Therefore, each construction business has its own method in performing the quantity take-off. In addition, for large projects, the work needs to be divided between the take-off estimator team members, in order to follow the same method of reading these 2D plans.

In the process of obtaining precise quantities, it is critical to avoid recounting or missing elements in assemblies. All these considerations are enough reasons to embrace new technologies which may reduce errors in the QTO work. Many software packages are being used for the quantities take offs and estimating purposes such as On-Center, e-Takeoff, Sage Estimating, Bluebeam Revu, Autodesk Revit, etc. Most of the software is sharing many advantages of saving time and giving certain accuracy for the quantity take-off estimators. In contrast, not all the software is supporting the 3D models' representations and some of the QTO software are required to draw on the plans to obtain the quantity sheets with the values on materials to be used in the jobsite. As a reference, Autodesk Revit Software has the features of providing the quantities take off in association

with the 3D model. The 3D model is created on Autodesk Revit by architectural and/or structural building objects, placed on the basis of the QTO object type required, hence there is no need to redraw 2D plans in order to obtain the quantity take-offs. This approach is new to some estimators who are quite used to extracting quantities only from 3D parametric models.

In the construction industry, there is a demand for precise materials' estimates as they are used for two main reasons. First, these estimates are involved in the bidding process. Second, they are needed to create a valid and as accurate as possible work plan to the Owner, Contractor and potential Subcontractors' jobs working on the respective buildings. It should provide them a fairly good idea about all the materials and how much of them are needed for the project. There is a persistent pressure in the construction market for estimation work to be done faster and as accurate as possible by all construction trades. The QTO accuracy may be inexact when it is sought during different design phases. On one hand, a preliminary version of the QTO helps in providing rough quantities and this could be needed in the early design process. On the other hand, very accurate and detailed QTO sheets are able to be presented when the design is completed, and a contractor is work-ready to proceed.

According to a previous study by Sylvester and Dietrich in 2010 [13], BIM provides a more accurate estimation by visualizing all the construction elements, linking them with the cost databases and reducing human errors, therefore reducing time to acquire a more precise estimate. In many of BIM software markets, the material quantities are automatically adjusted through association of any bi-dimensional changes and/or updates performed to the respective three-dimensional models. For example, the Autodesk Revit users are benefitted to save more time during the pre-construction phases instead of re-computing the quantities of the modified building 3D components in their models. In addition, if known at the time of the actual estimate, the users can attach prices per measuring units and create formulas to compute specific construction element work costs, then adjust the price units in the quantity take-off tables if needed. Other authors' work concludes that while it is possible to adapt the model to extract quantities according to existing specifications for manual-based measurements, the adjustments are not without its consequences in other model applications such as visualization or drawings. Therefore, take-offs specifications should be revised in order to account for BIM's features and accordingly minimizing its limitations [10].

2. Literature Review

Traditionally, the quantity take offs work requires going through every single plan of the 2D drawing sets (architectural, structural, MEP, landscape, fire protection, etc.) to account for all the existing materials and measure every unique instance of certain objects, as first step. For next step, the traditional estimating method requires reviewing the QTO sheets and assuring there is no adding or missing of any building elements, systems, assemblies, connections or other type of attachments. Unfortunately, these processes consume a lot of time and effort [4].

As there is a demand for QTO's in the construction industry, Elbeltagi [4] also explained the QTO significances and requirements are different for each Owner and also from the Contractor's perspective. The significance of the QTOs perspective is illustrated in the following:

a. From the Owner perspective:
   - Provide primary estimates for the project at the different stages,
   - Prepare bill of quantities as they are required in the contract documents,
   - Issue the contractor payments after estimating the work done.
b. From the Contractor perspective:
   - Provide pricing for the different work items,
   - Identify the needed resources (Labour, Equipment, etc.),
   - Prepare the project schedule,
   - Prepare invoices for work done,
   - Prepare the subcontractors' payments,
   - Review and control of crews' production rates.

All these are contributing to the need for a realistic QTO and BIM software can provide the means to extract efficiently these quantities for construction purposes.

Nowadays, construction projects are more readable as a set of databases more than a drawing set of plans which affects the take-offs and the cost estimating work process [6]. For this purpose, there is a need to work with new software capable of reading the project as a set of databases. As mentioned before, there are many BIM software packages currently available in the market to work within preconstruction estimates.

According to an independent research report by McGraw-Hill Construction [9], in North America, it is shown that the adoption of BIM has increased from 17% to 70% between 2007 to 2012. Furthermore, Autodesk website shared great news during 2013 and commented on one of the previous Smart Market Reports at that time, concluding that "for the first time ever, more contractors (74%) are using BIM than architects (70%)" [1]. This represented a great leap for construction estimates too, especially because during that period of time BIM software was not heavily used in construction estimates or for quantity take off purposes.

The more recent National BIM Report [11] from Great Britain (2018) showed that there is growth in the levels of BIM adoption over recent times. The report analyzes this growth into three distinct categories of adoption, and they are shown in Figure 1, as: aware and currently using BIM, just aware of BIM and neither aware nor using BIM at all. In 2018, there is only 1% of the total companies surveyed which were unaware of BIM

comparing to 43% in 2011. Similarly, the growth of BIM usage went from 13% in 2011 up to 74% during 2018.

According to another study done by Bečvarovská and Matějka (2014), both methods (traditional and BIM-based) were applied to one QTO project; as a result, BIM saved up to 80% of time comparing to the traditional quantity take off methods [2]. Table 1 represents the actual time differences between using BIM and traditional way spent to obtain the QTOs for each of the categories in Table 1.

Other study by Olsen and Taylor (2017) supported the BIM advantage of time-saving in general, besides many other advantages such as the accuracy of the quantity take offs [12]. They explained that the probability of the omissions and errors are decreased when projects involving BIM are more detailed. Figure 2 presents the responses of the survey about defining the advantages of QTO and estimation based on BIM. The visualization and the speed are the top two BIM-based QTO and estimating advantages, according to their findings.

However, this study finds in particular two main disadvantages of BIM-based QTO: the time spent in creating the models and examining them carefully against the design intent, also the complexity of software as being used for the purpose of estimating challenged sometimes by the lack of details (Olsen, D. and Taylor, M., 2017). On a different note, Khosakitchalert et. al. (2019) study proposes a method to improve the accuracy of the extracted quantities of compound elements from BIM models that are incomplete or incorrect by using information from BIM-based clash detection to eliminate excess quantities and add missing quantities [7]. Their method (validated through four case studies) claims that accurate material quantities can be delivered (with a deviation of up to 3%), and therefore time used to edit the BIM models is saved.

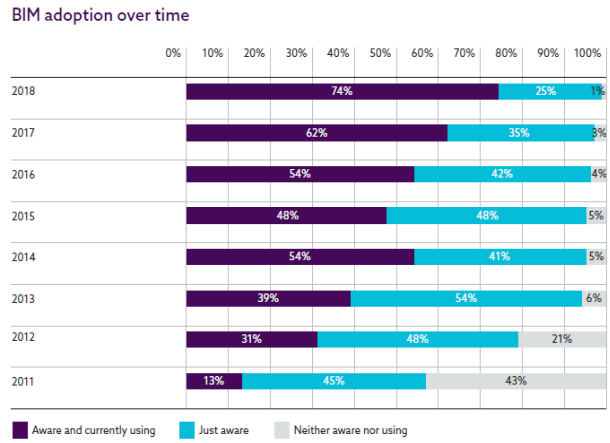

Figure 1. NBS, the National BIM report (adopted from [11] to represent on BIM implementation)

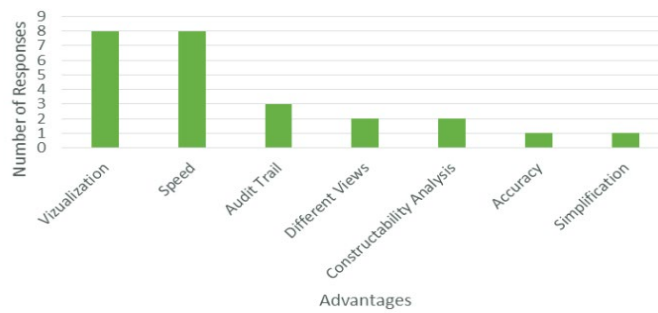

Figure 2. Advantages of BIM-based QTO [12]

Table 1. Tme Saved using BIM Tool for Quantity Take-Off [2]

		Time (minutes)	
Category	**Contents**	**Manual**	**Revit**
Table of rooms	Floor and ceiling composition	40	20
Foundations	Pads, belts, slabs	125	20
Construction phase	Walls, pillars, girders	420	10
Vertical constructions	Masonry, isolation	450	30
Wall surface	Plasters, paints	330	20
Ceiling composition	Ceiling composition	20	15
Floor composition	Floor composition	40	15
Wall composition	Wall composition	100	20
Façade	Façade	240	20
Roof composition	Roof composition	20	5
Balcony composition	Balcony composition	50	15
Groundwork	Excavations, embankments, removal	120	-
Staircases	Staircases	60	-
Other construction and work	Cleaning, shining, covering	70	20
Tables for take-offs		--	130
Total		2085	340

3. Materials used in the study and methodology

Three cases of study were analyzed in conjunction with their concrete-based Autodesk Revit files formats – as-build projects – which were shared by a General Contractor working in the south-east United States (Georgia and Florida mainly) and they were used to perform volumetric concrete quantity take-offs. The following breakdown was considered for each individual project with their concrete elements (Figures 3, 4, and respectively 5):

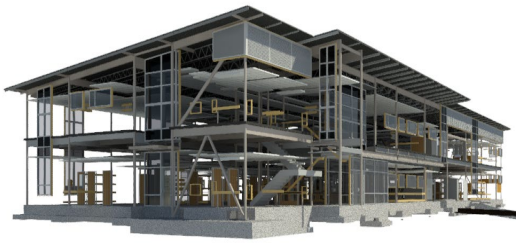

Figure 3. Health Center, Georgia Southern, GA

Figure 4. Grove Station Tower, Miami, FL

Figure 5. Georgia Heights, Athens, GA

Case Study 1: Georgia Southern University Health Center: specifically, foundations, columns, floors, and walls;

Case Study 2: Grove Station Tower (Miami): in this case foundations, columns, framing, floors, and walls;

Case Study 3: Georgia Heights (Athens): specifically, foundations, columns, framing, floors, and walls.

More than a decade ago, earlier adopters and critics reported at that time a number of BIM programs on the market, including Revit Structures, VICO Constructor, Bentley Structures and Tekla Structures having some quantity take-off features and capabilities for concrete volume calculations and to analyze scenarios in preconstruction phase for concrete work [14]. However, as the BIM software was used for all calculations (Autodesk Revit) in all three case studies, the actual software was employed to compute only the concrete quantities for estimation purposes. It is noted that Autodesk Revit has successfully provided ample opportunities to measure a variety of materials quantities for pre-construction needs. Therefore, it offers several types of schedules and quantity take-offs for building elements and various components (objects) to help in computing the total costs of construction. Autodesk Revit also has other tools for scheduling purposes that are shown in Figure 6, these tools help in the estimation phases or processes. For these particular cases, the scheduling tool used in computations was accessed from the View Tab > Create Panel > click Schedules drop-down > then, choose Schedule/Quantities.

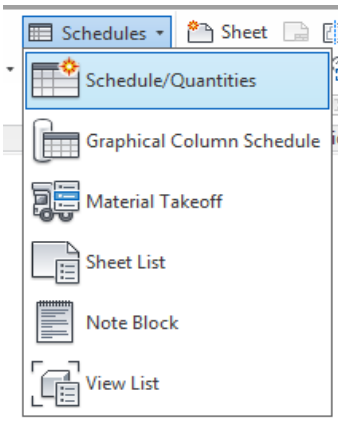

Figure 6. The Schedule/Quantities Revit tool used for computing the QTO structural elements' materials

Using the feature of material take-off in all the concrete structural elements that may have concrete (volumes) among its components from the respective Autodesk Revit models will reveal its quantities as designed in the respective families, including the foundations, columns, framing components, floors, and walls. Because each of these elements has a different structural function in the building, the elements' materials were different too. For example, the columns structural schedule might have a combination of materials such as concrete, steel, and wood. However, for this analysis, the only purpose was to compute the concrete quantities from the different parts of the structure in each Revit file/model of the three respective projects (cases of study). The concrete type and its strength were accounted for separately. Furthermore, the unit costs are going to be different too. This concern is addressed in the chosen BIM software by using the "grouping" feature, based on the concrete/materials type. After this step is finalized, the concrete structural "schedules" are ready to be exported from Autodesk Revit software using Export> Schedule > (*.txt) file feature, as seen in Figure 7. One more step is required in order to obtain the concrete QTO which helps in editing or/deleting the non-

concrete elements from the Excel sheets. This step will be easily applied because all the non-concrete material elements are grouped/gathered and have a sub-heading that indicates the materials type of each group. As a result, the consequent schedule will have concrete material only. In the end, the quantity take-offs will be multiplied with the unit cost/price (CF or CY cost) whenever the final estimation numbers are needed by the Contractor.

Figure 7. Software workflow used for concrete QTO computations

Case Study 1. Georgia Southern University Health Center [for the computational process of this building the following structural elements were considered: foundations, columns, floors, and walls and all of them have different types of materials. The materials quantities take-off schedules are counted separately based on the structural elements of the project. To detail in this case study, the BIM structural elements containing concrete modelled in the 3D design were foundations, columns, floors, and walls. Each one of these structural elements has different types of materials besides the concrete]. Each structure element schedule was exported from BIM software to (*.txt) file extension in the initial process of this computation. Then, all this information was transferred in Microsoft Excel spreadsheets where it was used to edit the output file and remove all the non-concrete materials (Figure 9 and 10). The final concrete structural type elements and its quantities for this specific structure are listed below in each designated category.

Foundations:

1. Concrete cast in place concrete: 772.88 CF (21.88 m^3);
2. Concrete cast in place concrete 3,000 PSI: 7,679.62 CF (217.46 m^3).

Columns:

1. Concrete cast in place concrete 3,000 PSI: 30.52 CF (0.86 m^3).

Floors:

1. Concrete cast in place concrete 3000 PSI: 7,998.13 CF (226.48 m^3);
2. Concrete cast in place concrete 3500 PSI: 9,739.62 CF (275.79 m^3).

Walls: Concrete cast in place concrete 3,000 PSI: 91.84 CF (2.60 m^3).

From these structure elements' materials, it was clear that there were only three types of concrete used for construction of the structure, according to the design information placed in the BIM software. These concrete types quantities are summed up based on their concrete strength. Therefore, the final computation performed in this case have led to the following quantities:

1. Concrete cast in place concrete: 772.88 CF (7.87 m^3).
2. Concrete cast in place concrete 3000 PSI: 7,679.62+30.52+7,998.13+91.84 = 15,800.11 CF (447.41 m^3).
3. Concrete cast in place concrete 3,500 PSI: 9,739.62 CF (275.79 m^3).

Finally, these results of different types of concrete went through a materials' filtering process using the software filtering tools. These available features within this particular software, including the take-off feature, are unable to manage/edit all grouped family materials and process this data. For this reason, it is sometimes necessary to also use other software. (Monteiro and Martins, 2013). All the structural elements schedules of case study 1 have various types of materials in their assemblies, not only concrete.

Structural Foundations Schedule: it has only two types of the structural material, which are: Concrete cast in place concrete, and Concrete cast in place 3,000 PSI (Figure 9). Obviously, both of these materials are concrete based.

Subheading in Revit

Type	Structural Material
20 x 20	Concrete - Cast-in-Place Concrete 3000 PSI
20 x 20	Concrete - Cast-in-Place Concrete 3000 PSI
20 x 20	Concrete - Cast-in-Place Concrete 3000 PSI
20 x 20	Concrete - Cast-in-Place Concrete 3000 PSI
Concrete - Cast-in-Place Concrete 3000 PSI: 4	
HSS4X4X1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular
HSS4X4X1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular
HSS4X4X1/2	Metal - Steel - ASTM A500 - Grade B - Rectangular

Figure 8. Revit take-off table showing the non-concrete columns materials

HSS3X3X1/4	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square
HSS4X4X1/4	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square
HSS4X4X1/4	Metal - Steel - ASTM A500 - Grade B - Rectangular and Square
A500 - Grade B - Rectangular and Square: 182	
2x4	Softwood, Lumber 2x4
2x4	Softwood, Lumber 2x4
2x4	Softwood, Lumber 2x4

The non-concrete materials in Excel

Figure 9. Microsoft Excel table illustrating the non-concrete columns materials

Structural Columns Schedule: it has various types of column materials, such as Softwood Lumber, Metal Steel ASTM, and Cast in Place Concrete. The first two types of columns materials are not needed in this quantity estimation because of their specific material as being different than concrete. The only type considered in this quantity calculation is the Cast in Place Concrete for columns.

Structural Floors Schedule: it has only three types of the structural material which are: Metal Roof Deck, Concrete cast

in place concrete 3,000 PSI, and Concrete cast in place concrete 3,500 PSI.

Structural Walls Schedule: Curtain Wall Type, Louver System Type, Sliding Type 11", Exterior Concrete Masonry Units, Gypsum Wall Board, Metal Stud & Rigid Insulation Layer, Metal Stud & Sound Attenuation Layer, Toilet Partition "Plastic", and Concrete cast in place concrete 3000 PSI.

Unfortunately, the authors have not identified at this time a method to delete the non-concrete materials from the actual outputted Revit schedule. In order to place the quantities into a friendly spreadsheet, one more step is needed to output the total quantity of the related concrete element volumes for each building. This step was to export the schedule of obtained quantities into Microsoft Excel sheets and to further modify the material columns in the resulted spreadsheet(s). The structural columns with non-concrete materials were deleted in all the structural elements schedules (Figure 10). The step did not take a long time because the building elements were listed and grouped based on the material type. As a final result, the schedule contained only the structural concrete materials. However, and just as a constructability note, there is no consideration of construction method information to be taken into account when performing generation of quantities for constructability purposes [8].

Case Study 2: Grove Station Tower (Miami). The concrete quantity for this case study is grouped based on the structural elements of the building which are foundations, columns, framing, floors, and walls.

Structural Foundations Schedule:

Concrete cast in place concrete: 36,385.36 CF (1030.32 m^3).

Concrete, cast in place gray: 35,026.25 CF (991.83 m^3).

Columns: Concrete cast in place concrete: 24,056.75 CF (681.2m^3).

Framing: Concrete cast in place concrete: 6,046.38 CF (171.21 m^3).

Floors: Concrete, cast in place gray: 184,409.36 CF (5,221.89 m^3).

Walls: Concrete, cast in place gray: 48,410.33 CF (1,370.83 m^3).

For this structure, there are only two types of concrete as outputted for final calculations:

1. Concrete cast in place concrete: 36,385.36+24,056.75+6,046.38 = 66,488.49 CF (1882.74 m^3).
2. Concrete cast in place gray: 35,026.25+184,409.36+48,410.33 = 267,846.94 CF (7584.58 m^3).

In this case, the following classifications for structures were identified.

Structural Foundation Schedule: Concrete cast in place concrete, and Concrete, cast in place gray.

Structural Column Schedule: it has only one type of material which is Concrete cast in place concrete.

Structural Framing Schedule: it has only two types of materials which are Concrete cast in place concrete, and Metal - Steel - ASTM A572.

Structural Floor Schedule: it has only one type of material which is Concrete Cast-in-place gray.

Structural Wall Schedule: Concrete Masonry Units, Concrete Cast-in-Place gray, Metal Stud Layer.

Case Study 3: Georgia Heights (Athens). The concrete quantities for this case study are grouped based on the structural elements modelled, which are foundations, columns, framing, floors, and walls.

Foundations: Concrete cast in place concrete: 54,066.73 CF (1530.99 m^3).

Concrete cast in place gray: 4,721.32 CF (133.69 m^3).

Columns: Concrete cast in place concrete: 23,075.91 CF (653.44 m^3).

Framing: Concrete cast in place concrete: 6,078.58 CF (172.13 m^3).

Floors: Concrete cast in place concrete: 154,809.74 CF (4,383.72 m^3).

Walls: Concrete, normal weight, 4 ksi: 28,287.77 CF (801.02 m^3).

For this structure, there are only three types of concrete outputted in the final calculations:

1. Concrete cast in place concrete: 54,066.73 + 23,075.91 + 6,078.58 + 154,809.74 = 238,030.96 CF (6,740.28 m^3).
2. Concrete, cast in place gray: 4,721.32 CF (133.69 m^3).
3. Concrete, normal weight, 4 ksi: 28,287.77 CF (801.02 m^3).

To synthesize and in order to clearly understand the schedules obtained for all structural elements involved in the three case studies, they are aggregated in the followings:

1. Structural Foundation Schedules: Concrete Cast in Place Concrete, Concrete, Cast-in-Place gray.

2. Structural Column Schedules: Concrete Cast in Place Concrete, Metal - Steel - ASTM A500.

3. Structural Framing Schedules: Concrete Cast in Place Concrete, Metal - Steel - ASTM A500.

4. Structural Floor Schedules: Concrete Cast in Place Concrete.

5. Structural Wall Schedules: Concrete Masonry Units, Concrete, Normal Weight, 4 ksi.

4. Results and Preliminary Discussion

A comparison of the actual concrete quantities which were provided by the General Contractor (in CY) with the concrete quantities obtained virtually from the BIM models (in CF) was performed. These quantities were compared (in the end) based on the same unit provided by the contractor for placing and reporting finished concrete, which was worked out in these buildings (in CY). The following Table 2 below displays the total calculated concrete quantity for each case study comparing them to the reported (actual) total quantities of concrete used by the General Contractor in their construction phases. A discussion follows reflecting on the findings for each case study.

*For **Case Study 1***- the total quantity of the concrete is divided into three parts, based on the concrete strength: 1-cast in place concrete is 772.88 CF (21.89 m^3); 2- cast in place concrete compressive strength is 3000 PSI (20.68 MPa) and the volume is 15,800.11 CF (447.40 m^3); 3- cast in place concrete strength is 3,500 PSI (24.13 MPa) and volume is 9,739.62 CF (275.79 m^3). Accordingly, the total actual quantity for concrete reported by the General Contractor for all structural elements using concrete of various strengths in Case Study 1 were: 1,003 CY (766.85 m^3), meaning foundations, column block outs, and equipment pads: 383CY (292.82 m^3), Slab-On-Grade - Section 1: 190 CY (145.27 m^3) + Section 2: 197 CY (150.62 m^3), and the elevated Slab: 233 CY (178.14 m^3).

Table 2. Comparative results of the three Case Studies for Concrete Quantities

Case studies/ Classification Type	Case study 1: Georgia Southern University Health Center	Case study 2: Grove Station Tower - Miami	Case study 3: GA Heights - Athens
Concrete type 1 and its quantity	Cast in place concrete: 772.88 CF (21.89 m^3)	Cast in place concrete: 66,488.49 CF (1882.74 m^3)	Cast in place concrete: 238,030.96 CF (6740.29 m^3)
Concrete type 2 and its quantity	Cast in place concrete compressive strength 3000 PSI: (20.68 MPa), and the volume: 15,800.11 CF (447.40 m^3)	Cast in place gray: 267,846.94 CF (7,584.58 m^3)	Concrete, cast in place gray: 4,721.32 CF (133.69 m^3)
Concrete type 3 and its quantity	Cast in place concrete strength 3500 PSI (24.13 MPa) and volume: 9,739.62 CF (275.79 m^3)	----------	Concrete, normal weight, 4 ksi (25.57 MPa): 28,287.77 CF (801.02 m^3)
Total quantity from BIM QTO	974.54 CY (745.08 m^3)	12,382.79 CY (9,467.32 m^3)	10,038.52 CY (7,675 m^3)
Total concrete quantities from General Contractor	1,003 CY (766.85 m^3)	12,173 CY (9,306.92 m^3)	10,316.35 CY (7,887.41 m^3)
General Contractor vs. BIM quantities, Error variances results (< +3%; -2% >)	**+2.92%**	**-1.72%**	**+2.77%**

For Case Study 2 - the total quantity of the concrete is calculated based on the two different strengths: 1- cast in place concrete is 66,488.49 CF (1,882.74 m3); 2- cast in place gray is 267,846.94 CF (7,584.58 m3). Reported actual/total quantities of concrete from the General Contractor for this case is 12,173 CY (9,306.92 m3).

For Case Study 3 - the total quantity of the concrete has three parts and they are classified based on the concrete type: 1-cast in place concrete: 238,030.96 CF (6,740.29 m3); 2- concrete, cast in place gray: 4,721.32 CF (133.69 m3); 3- concrete, normal weight, 4 ksi (25.57 MPa): 28,287.77 CF (801.02 m3). Total quantity reported by General Contractor in this case after concrete construction phases were completed is: 10,316.35 CY (7,887.41 m3).

Working through the facts as resulted from the totals' comparison, the researchers have found that the error variances for all three case studies are in the range of [- 2.0 to + 3.0] % for the total concrete quantities needed to be used in each case, therefore a comfortable accurate estimate for the type of concrete work from the General Contractor's point of view. The errors generated in these estimates may be caused by several factors and therefore some aspects considered in the extraction of these totals for the quantities which were placed in the respective buildings (when erected) can be reasoned after all. Nevertheless, just by working on this study and considering the time spent to calculate the total quantities of each case study, the authors may empirically imply that the BIM quantity take-off procedure exhibited in this manuscript is an efficient estimation method and therefore may lead to a shorter-time determination needed to compete for real quantities to be priced (and placed as orders) for concrete vendors.

Furthermore and with certainty, the authors are acknowledging that the Level of Details (LoD) and effort the BIM drafters were corroborating into the virtual design and their "constructed" models are critical to the accuracies obtained relative to the parametric-scaled families within the software used (as referred also by Choi et. al. in 2015, [3]).

5. Conclusions and recommendation for further research

The pre-construction quantity take-off (along with the procedural cost estimation) in each of the presented cases are typically involving exhausting work and a great level of attention to details, if performed in a traditional way. However, the BIM quantity take-off as integral method to the procedure represents a great alternative process for extracting the material (in this case volumetric concrete) construction quantities. As a result, BIM quantity take-off generation saves more time and money by providing relatively accurate quantities and therefore an acceptable and accurate cost estimation process for the entire project.

As more details are revealed with their accurate representations for any project in the 3D models, the project quantity take-off and the cost estimation are becoming closer to the factual reality. For example, the concrete details in the presented cases were included in the concrete material descriptions, concrete strength level/type, degree of fire and/or humidity resistance, thermal insulation levels, etc. All these details will have finally an effect on the cost for the quantity unit considered in the estimate.

As limitation for cost estimates, labour costs were not included in the total prices for these case studies, so the sole purpose of calculating concrete material quantities was served as a basis for comparison between quantity take-off/estimated and placed masses of concrete - as volumetric (CY) quantities. Also, as mentioned by other authors [5] in 2017, during the cost estimating phase, the man-hours input from the cost estimator should be lower, as the take-off and parts of the cost estimate can be done automatically and semi-automatically with specialized software tools. The relationship on the labour costs assigned to the quantities needed to be placed is by itself a matter of research and further efficacy on every construction contractor's side.

On a different note, and as a potential further research direction on the presented study, concrete masonry units could be extracted from the completed digital models in each case and be compared with the total quantities used and evidenced by company reports after completion of their masonry work.

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Rehabilitation of the Otis Reservoir Dam: Improving Cost Effectiveness by Including Bridge Placement

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Abstract

The Massachusetts Department of Conservation and Recreation (DCR) owns and operates the Otis Reservoir Dam in Otis, Massachusetts, for recreational purposes. In 2006, the 145-year-old, 31.5-foot-tall, earthen embankment dam with downstream masonry wall and stone masonry spillway was found to be in "Poor" condition due primarily to the deteriorating/leaking spillway and downstream masonry wall and the erodibility of the "emergency spillway" over the main embankment section of the dam. To address most of the main dam safety concerns, a reconstructed spillway with a hydraulically actuated crest gate was chosen as the preferred alternative. The crest gate option provided additional hydraulic capacity while also improving the DCR's ability to manage the reservoir level, most importantly, during the annual winter drawdown. During the planning stages of the project, maintaining access to the west side of the reservoir during construction was identified as a critical component in the viability of the project. At the time, the bridge over the spillway was a one and one-half lane, temporary "Bailey bridge" installed in 1995. It was supposed to have been replaced with a permanent bridge in 1998. Given that the dam is a large, high hazard structure, the dam repairs needed to move forward to protect the public safety. To expedite the project, the DCR elected to incorporate the bridge replacement into the dam rehabilitation project. The dam rehabilitation project was transformed, by necessity and creative planning, into a combined dam and bridge replacement project that ultimately benefited the DCR and the town of Otis.

Keywords: Spillway Design Flood, probable maximum flood, Potential Failure Mode Analysis

1. Project Background

Otis Reservoir Dam is located within Tolland State Forest on the Fall River in the town of Otis, in Berkshire County (see Figure 1). The dam was originally constructed in 1866 by the Farmington River Water Power Company in order to provide supplemental water to power mills on the Farmington River in Connecticut. The Commonwealth acquired the reservoir in 1967. Otis Reservoir is a 1,000-acre impoundment with a 16-square-mile drainage area that encompasses portions of three towns (Otis, Tolland and Blandford). Having a storage capacity of 22,000 acre-feet (7.1 billion gallons) at normal pool, it is the largest recreational body of water in the Commonwealth of Massachusetts.

Tolland Road, owned by the town of Otis, passes over the dam and provides the only access to the western side of the reservoir during the winter months. The majority of the residents on the western side of the impoundment are seasonal, with only four full-time residents present during the winter. Because of the recreation industry supported by the reservoir, the population in the town of Otis swells from a few thousand in the winter to near 10,000 during the summer months.

The Otis Reservoir Dam consists of an earthen embankment with a downstream stone masonry wall. The dam has a maximum structural height of about 31.5 feet and a length of about 630 feet. The exposed portion of the downstream masonry wall of the dam is approximately 480 feet in length. Tolland Road, a paved public roadway, traverses the top of the dam. In 1955/1956, after Hurricane Diane, the top of the embankment was lowered by about 3 feet to create an "emergency spillway," although no erosion protection, other than the asphalt pavement roadway, was provided. The dam was originally built with a 38-foot-long stone masonry primary spillway located near the dam's left (west) abutment. The spillway was divided into two, 19-foot-long segments by a stone masonry pier.

In 1995, MassHighway replaced the deteriorating original bridge over the spillway with a temporary one and one-half lane Bailey bridge. This bridge was intended to be in service for three years. A permanent, two-lane bridge was slated for construction in 1998. However, the project was delayed and postponed,

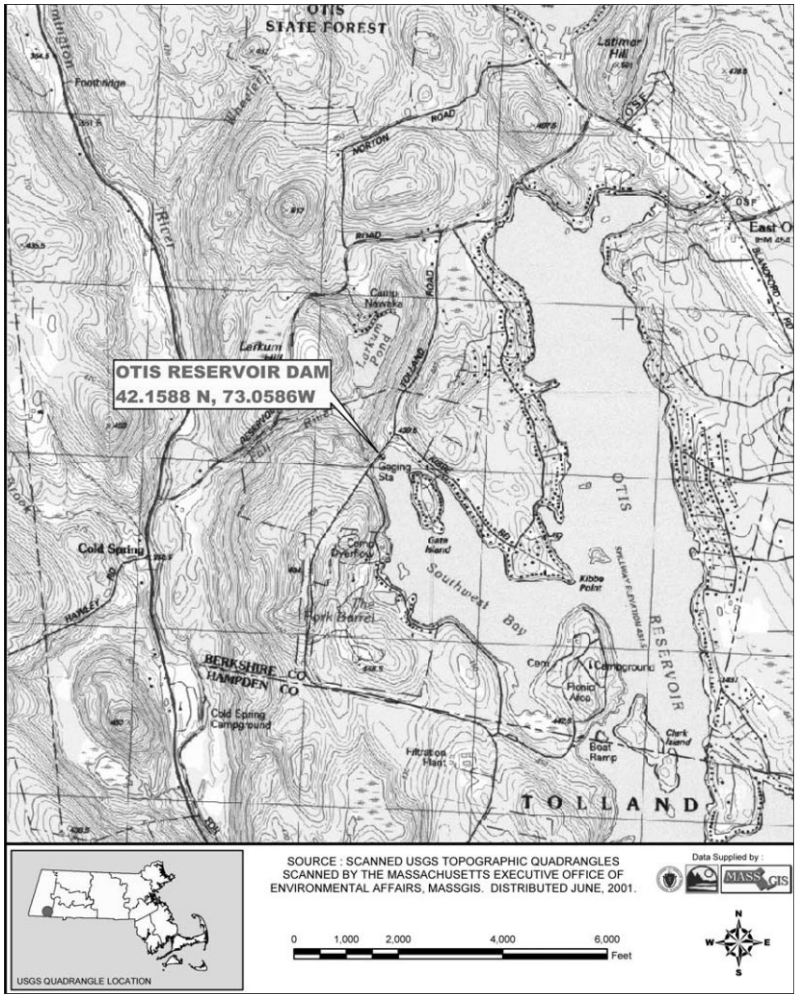

Figure 1. The location of the Otis Reservoir and dam.

apparently due to MassHighway's bridge project prioritizations. By 2009, the permanent bridge replacement at the Otis Reservoir Dam had been officially postponed until 2014.

Prior to the recent dam rehabilitation, there were two gatehouses/outlet structures at the dam. The original gatehouse, located in the middle of the upstream side of the embankment, was taken out of service in 1984 when the outlet works at the dam were rehabilitated. The original sluice gates, stems and operators were removed and a 4-foot-high by 8-foot-wide intake conduit, with an upstream steel trash rack, was installed within the reservoir. The intake conduit extends into the reservoir about 40 feet upstream of the dam. A downstream outlet tower was constructed in 1984 with a second gatehouse opposite to the original upstream gatehouse. The outlet tower has two, 48-inch-square slide gates. One gate serves as a low-level outlet and the other serves as a mid-level outlet. The outlet tower also has a 10-foot-wide by 3-foot-high overflow weir at the same elevation as the primary spillway crest (with the flashboard in place). The two original 30-inch-square stone sluiceways through the dam had been retrofitted with 26-inch-square steel conduits and then grouted in place.

The Otis Reservoir is operated by the DCR's park staff according to an order of conditions from the Otis Conservation Commission, which defines the target seasonal reservoir levels and governs the rates of reservoir drawdown and refill. In October of every year, the DCR opens the low-level outlet gate to provide a flow of around 300 cubic feet per second (cfs) into the Fall River for an annual weekend canoe race. The DCR subsequently dials back the release to lower the reservoir to about 2 inches per day until the winter pool elevation (6 to 8 feet below the primary spillway crest elevation) is reached. The purpose of the annual drawdown is to allow for the inspection of the dam structure, to prevent ice damage to the shoreline docks and piers, and to provide meltwater storage and discharge control during the spring freshet. Beginning in January of each year, the DCR begins to raise the reservoir level. The reservoir is raised to within 4 feet of the summer pool level by April, depending on the ice conditions on the reservoir. The goal is to have the reservoir level restored to the summer pool level by July 4th each year. The 1984 outlet works configuration requires constant adjustment to manage the reservoir level, especially prior to or after storm events.

With the winter pool being maintained by releasing water from the low-level outlet in the downstream outlet tower, the DCR could not easily monitor the water level below the ice. An excessive water release could cause the development of a void between the bottom of the ice and the surface of the reservoir. The presence of such a void would be a major safety concern, given the year-round recreational use of the reservoir, which includes ice fishing and snowmobiling on the reservoir. The DCR's staff would use a chain saw to cut holes through the ice in order to evaluate the reservoir level. Obviously, this practice was not an ideal approach. So, the DCR indicated that the dam rehabilitation project should also include provisions to improve the reservoir operation procedures.

2. Engineering Evaluations & Alternative Analyses

Following an inspection of the dam in May 2006 by an engineering consulting firm, the dam was judged to be in "poor" condition. The primary deficiencies of the dam were:

- The deteriorated masonry and leakage conditions at the spillway (see Figure 2) and the downstream masonry wall; and,
- The erodibility of the "emergency spillway" over the main embankment section of the dam.

Figure 2. View of the deteriorated and leaking conditions at the spillway prior to rehabilitation.

In 2007, the engineering consultant performed a Phase II Engineering Evaluation and Alternatives Analysis of the dam. This study included field investigations covering wetlands delineation, a topographic survey of the dam and nearby areas, rare species determinations, diving inspections of the intake conduit and interior of the downstream outlet tower, subsurface explorations with test borings and taking additional readings from the existing instrumentation (piezometers and inclinometers) at the dam. The consultant's engineering evaluations included interpretation of the subsurface conditions from the test borings, geotechnical laboratory testing (grain size analyses) and existing instrumentation readings, as well as conducting engineering analyses including detailed hydrologic and hydraulic analyses, liquefaction and seismically induced settlement analyses, seepage analyses, slope stability analyses and gravity wall stability analyses.

With Otis Reservoir Dam categorized as a large, high hazard dam per the Massachusetts Dam Safety Regulations, its spillway design flood (SDF) is one-half of the probable maximum flood (½ PMF). The results of the detailed hydrologic and hydraulic analyses revealed that the original spillway was capable of passing only 10 percent of the SDF and overtopping of the dam by about 3.3 feet was predicted. The seepage and stability analyses indicated that the dam met most of the stability requirements set forth in the dam safety regulations except for the case where the dam was overtopped during the SDF event. The factor of safety against sliding for the downstream masonry wall was below the recommended minimum value of 1.3 due to the likelihood for erosion occurring at the base of the wall attributed to overtopping during the SDF.

In concert with the detailed engineering analyses, a potential failure mode analysis (PFMA) was performed with several members of consultant's dam engineering team and representatives from the DCR who were responsible for the operation and maintenance of the dam. The objective of the PFMA was to assess possible failure modes and to determine the most likely failure mechanisms, thereby allowing the design of the rehabilitation project to address each of these deficiencies.

Using the information obtained from the detailed engineering evaluations and the PFMA, an alternatives analysis was performed to evaluate the repair/rehabilitation options that could address the dam safety issues. These alternatives included:

- no action;
- breach/remove;
- raise the dam;

- construct an emergency spillway; and,
- spillway modification/reconstruction.

Spillway reconstruction, along with raising the crest of the dam, were selected as the preferred alternatives.

3. Selection of the Preferred Alternatives

To address both the dam safety issues and operational issues, reconstructing the spillway with a bottom-hinged, 7.5-foot-tall by 38-foot-wide steel crest gate was selected as a preferred alternative. When the crest gate is in the "up" or "closed" position, the top of the gate is at the current normal summer pool elevation for the reservoir. The invert of the new spillway (i.e., with the crest gate in the "down" or "open" position) was set at the winter pool elevation so that once the reservoir was drawn down it could "self-regulate" the reservoir level without constant assessment and adjustment by the DCR staff.

To safely pass the SDF, the DCR staff would need to lower the summer reservoir level by 2 feet in advance of the ½ PMF event and then operate the gate throughout the storm to prevent overtopping of the dam. Without proper gate operation, the dam will be subject to overtopping during the SDF. However, the proposed dam modifications would allow the spillway to safely pass the 500-year flood event with about 2 feet of freeboard without lowering the crest gate.

Typically, relying on human operations to pass the SDF is not a recommended dam safety practice. The reason it is not recommended is the potential for improper or lack of operation that can result from human error. However, Otis Reservoir Dam is not a typical dam since it has a full-time dam operations staff in the DCR's Tolland State Forest Office at the right abutment of the dam. The DCR also has staff who live locally who are "on-call" should an emergency situation develop at the dam. Therefore, the design of the new spillway was able to take advantage of the DCR's somewhat unique on-site staffing situation.

Other improvements included in the dam rehabilitation, as shown in Figure 3, were:

- adding a new gatehouse on the left abutment of the dam;
- raising the top of the dam by adding up to 3 feet of embankment fill;
- repointing the downstream masonry wall;
- extending the downstream toe drain;
- adding a reinforced concrete splash pad at the base of the downstream masonry wall;
- restoring the rip-rap slope protection on the upstream and portions of the downstream slopes; and.
- installing new slide gates on the inside upstream face of the downstream outlet tower.

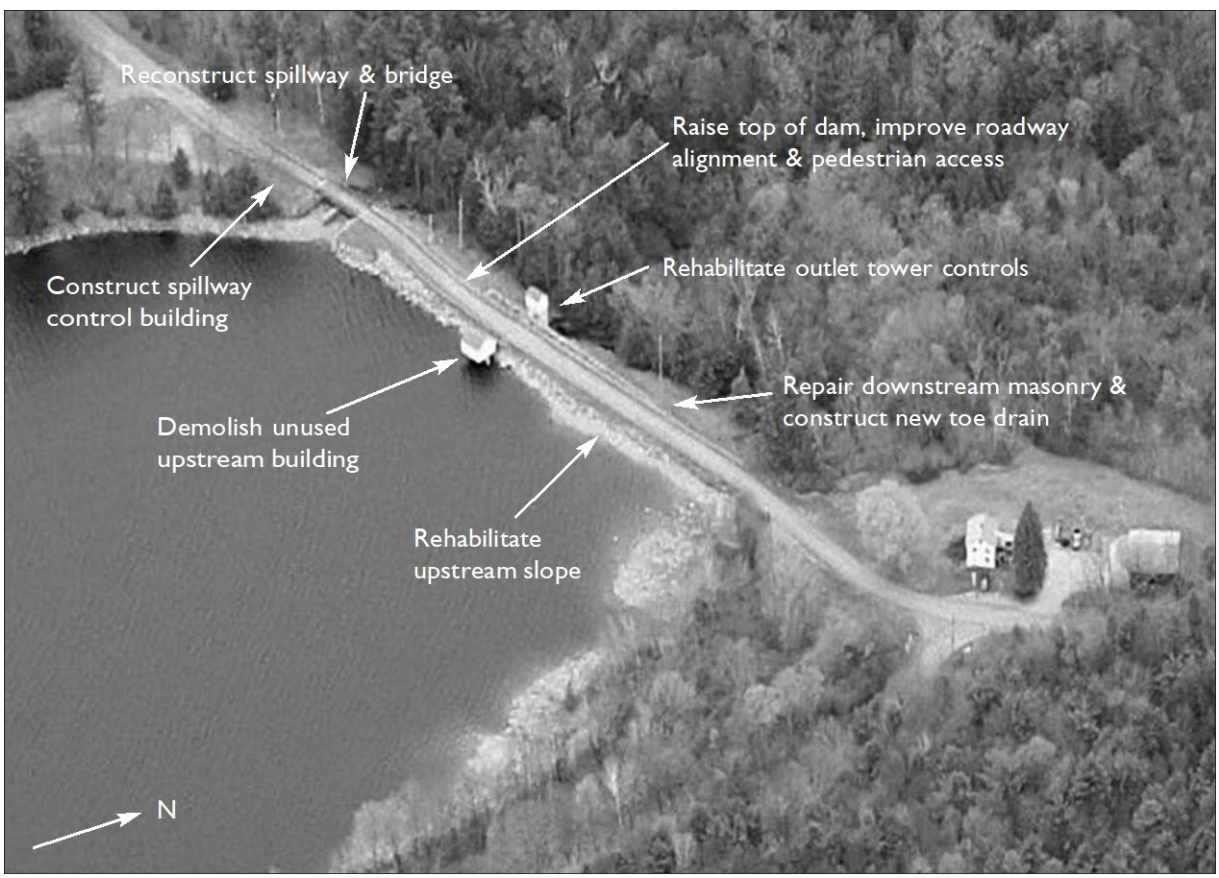

Figure 3. The locations of repairs/improvements on the dam.

4. Solving Resident Access Issues

Access to the west side (left side of the dam) of the reservoir was identified as a critical issue early in the project development. The towns of Otis and Tolland fully understood the benefit of the project. However, residents of both towns were concerned about the potential for short-term impacts on the recreational-based economy. Several concepts and options were evaluated by the project team in order to address this concern.

Initially, a bypass route upstream of the spillway was evaluated. A temporary earthen embankment was considered that would convey traffic around the spillway construction while also serving as a temporary cofferdam for the work area. A significant amount of fill would have been needed to be placed in wetlands area fro the embankment. This option was eliminated when the constructability, cost and permitting requirements were evaluated more closely.

A second approach that was considered was to construct the spillway in two segments: upstream and downstream. The existing Bailey bridge would be left in place initially to allow the upstream portion of the spillway to be constructed. The Bailey bridge would then be moved to the upstream side of the dam and the downstream half of the spillway would be constructed. However, this option was also eliminated when the costs associated with the phased construction, the extended construction duration, the need for a more robust temporary cofferdam capable of remaining in place throughout the higher summer pool season and the additional risks were evaluated more closely.

To limit the impact of the project on the recreational use of the reservoir, it was decided that the majority of the work needed to be performed in off-season (i.e., winter construction). It was also obvious that with the significant improvements being considered for the spillway, it did not make sense to reinstall the temporary Bailey bridge over the newly reconstructed spillway. Therefore, the DCR decided to incorporate a permanent bridge replacement into the dam rehabilitation project. In exchange for adding the new bridge to the project, the DCR was granted permission from the towns of Otis and Tolland to close Tolland Road for five months (from October 2010 to March 2011) in order to facilitate construction.

To address the access issue to the west side of the reservoir, the DCR offered to establish and maintain a detour route around the reservoir during the five-month period when the bridge was closed. The detour route (see Figure 4), which was about 8 miles in length, included a 2.8-mile section of gravel road that was typically not plowed during the winter. The DCR included a construction contract provision for liquidated damages to be levied against the contractor if the bridge over the spillway was not reopened on time. The town of Otis accepted the DCR's offer for the new bridge and the temporary detour route, and granted the DCR permission to close Tolland Road for five months.

5. Dam & Bridge Design Considerations

The design of the dam rehabilitation and the bridge replacement occurred between 2007 and 2010. The engineering consultant retained another engineering firm to provide structural, mechanical and electrical engineering design services for the new crest gate and for two new slide gates in the outlet tower. Under a separate contract, the Massachustts Department of Transportation (MassDOT) engaged another consultant to perform the bridge design. The DCR supervised the bridge designer and reported progress to MassDOT. As overall project manager and the dam engineer-of-record, the contracting engineering consultant was responsible for providing engineering design for the dam, including the hydrologic/hydraulic, geotechnical and civil designs, in addition to coordinating the other project team design responsibilities and merging of the contract documents into a single bid package.

A performance-based specification was created for the crest gate. Two Massachusetts-based gate manufacturers were consulted to provide input in establishing the crest gate design criteria and to understand the implications of the DCR's intended operation procedures. The DCR was committed to providing redundancy with the crest gate operations. The DCR wanted the gate to have twin top-mounted actuators, but the DCR also wanted to have the ability to operate the gate with a single actuator in the event that one failed. Based on this requirement, a hydraulically actuated operation system was selected for the project. Velocity fuses (i.e., a hydraulic valve used to stop flow if the maximum speed of the fluid is exceeded) were also added to the system to prevent the gate from opening unintentionally in the event of a power loss or a break in the hydraulic system. The gate system also included provisions to raise or lower the gate manually in an emergency situation. A new gatehouse was also added on the left (west) abutment of the dam to house the crest gate's hydraulic system.

With the reservoir's annual drawdown schedule, there is a need to operate the gate in cold-weather months. However, the rubber side and invert seals could be damaged if the crest gate is operated while it is subjected to ice accumulation. As such, robust electric side seal and invert heaters were incorporated into the structure. Operational procedures to turn on the heaters in advance of any winter gate operations were also written into the dam's new operations and maintenance plan. The new gatehouse constructed near the spillway to house the crest gate controls included provisions for connecting a portable generator if spillway operations were required during power outages.

The existing masonry spillway was removed and replaced with a reinforced concrete structure to support the crest gate and to convey the spillway flow through the dam. The new spillway structure served dual purposes: spillway training wall and bridge abutments. (In addition to the hydraulic loads imparted due to the spillway flows, the new concrete walls were also required to support the lateral loads from the embankment, vertical dead forces from the bridge structure and HS-25 vehicle

Figure 4. The 8-mile detour route that was used during construction.

loading, and earthquake loads.) The training walls/abutments were to be founded directly on bedrock, which provided adequate bearing capacity and erosion resistance.

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- Several types of superstructure for the new bridge were considered in the early part of the project. Initially, the bridge types considered included:
- Single span, precast, prestressed concrete spread box beams with cast-in-place concrete deck.
- Single span, rolled steel stringers with a cast-in-place deck.
- Single span, built-up steel plate girders with a cast-in-place deck.
- Single span, rolled steel stringers with a timber deck.

The initial recommendation for a bridge type was for a single span, precast concrete spread box beams with a cast-in-place deck. However, the winter construction schedule and the necessity for limited road closure precluded any of the bridge types that used cast-in-place decks. Therefore, a decision was made by the project team to only consider bridge types that used prefabricated elements, since they could be brought on-site and lowered into place once the spillway training walls/bridge abutments were completed. The prefabricated bridge types that were considered included:

- Prefabricated concrete/steel composite superstructure units (formally known as Inverset);
- NEXT beam system;
- Fiber reinforced polymer (FRP) deck on steel stringers;
- Full depth precast concrete deck panels on concrete or steel stringers; and,
- Butted boxes/deck slab with no cast-in-place deck.

The effects of each system on the hydraulic performance of the spillway, their relative maintenance costs and their impact on construction schedule (if any) were evaluated. The first alternative — prefabricated concrete/steel composite superstructure units with an asphalt wearing course — was chosen. The composite system included steel stringers and a precast concrete deck. The main benefits of this bridge type included design flexibility, adaptability to the winter construction schedule and quick installation.

5. Improving Constructability by Modifying the Spillway Design

The winter-only construction schedule, coupled with the five-month road closure limitation, dictated that the total construction schedule for the project occur over a two-year period. And, with the crest gate design, submittal review process and fabrication requiring up to six to eight months, the crest gate installation was not scheduled until the second winter construction season. This schedule would have not been possible if the project had been awarded in the fall since the spillway construction would have needed to have been completed during the winter road closure of the first construction season. In order to accommodate this scheduling issue, the decision was made to add stop logs bays to the spillway upstream of the proposed crest gate. The addition of the upstream stop logs provided the following benefits:

- The stop logs provide a permanent mechanism to allow the crest gate to be taken off line for servicing and maintenance.
- The stop logs could be installed early in the spillway construction in order to limit the amount of time needed for a temporary cofferdam for the project.
- The stop logs could be installed and kept in place to serve as the water control mechanism until the crest gate was fully installed. Doing so would allow the crest gate design, submittal review and fabrication to proceed without the added pressure of being the critical path element for the project.
- The stop logs would also be used to create a controlled upstream water condition that would allow testing of the crest gate and the training of the DCR operations staff.

Figure 5. Improvements to the dam crest and upstream slope.

6. Combined Dam & Bridge Design Benefits Project Financing

With the project design and constructability issues resolved, the next step was to figure out how the project would be funded. The initial construction cost estimate was on the order of $2 million. The DCR Office of Dam Maintenance used this figure in their capital planning for FY2010/2011. However, the initial cost estimate did not include a new bridge nor did it consider a two-season construction schedule. Consequently, the updated project cost estimate was about $1 million higher than originally estimated. In order to address the cost increase, the DCR needed to look at other funding options.

Because this "dam" now included a new bridge, it became obvious that it should qualify for "bridge monies." The bridge and portions of the spillway construction that served a dual capacity (both spillway training walls and bridge abutments) were funded via the MassDOT Accelerated Bridge Program and not solely through the DCR's Office of Dam Maintenance dam rehabilitation budget. Second, by extending the project to two construction seasons, the project was also extended over two fiscal years. Consequently, the DCR was able to spread the project budget out over two years, which provided the DCR with greater fiscal flexibility when compared to the initial approach when the dam rehabilitation was scheduled to occur over one construction season.

Even though the project scope and budget were increased by the addition of the bridge replacement to the project, the DCR, the town of Otis and the general public benefited in a greater way with the expanded project scope and the DCR's ability to see the big picture.

7. First-Year Construction Highlights

The project was advertised for bid in the summer of 2010. The project was awarded to the low bidder, with a bid price of $3,057,496.50. Seven prequalified contractors submitted bids on the project. The four lowest bidders were within $150,000 of each other, and all four of these bids came within $300,000 (roughly 10 percent) of the engineering consultant's estimate for the project.

On September 15, 2010, the DCR and the contractor began first construction season. Approximately one month later, the portion of Reservoir Road across the top of the dam was officially closed and the detour route around the site was established. The first-year construction work included:

- the construction of the toe drain and splash pad;
- raising of the embankment crest by 3 feet;

Figure 6. Bedrock excavation at the spillway.

- placement of new upstream and downstream riprap slope protection (see Figure 5);
- demolition of the masonry spillway;
- construction of the new reinforced concrete spillway/bridge abutments;
- installation of the new stop logs;
- installation of the new bridge;
- demolition of the upstream gatehouse; and,
- construction of the new crest gate gatehouse at the left abutment.

Questions of interest to state transportation policymakers and DOT officials relate to the potential impacts or consequences of such approaches. Examples of these questions are:

- What will be the capital and operating costs to implement toll-based approaches on interstate highways on which tolls are not currently charged?
- What are the potential levels and natures of the revenues that can be collected with these tolls and how do these revenues compare to other funding approaches such as fuel taxes?
- What changes in demand can be expected? Will mode shifts and route diversion occur and at what levels?
- Will there be equity and privacy concerns that may lead to additional challenges in gaining public acceptance?

Because the invert of the new crest gates was almost 8 feet lower than the original spillway crest, bedrock excavation and removal within the spillway footprint was required for foundation construction. The excavation required for the new spillway foundations was approximately 100 by 80 feet in area, and required cuts into the bedrock of up to about 7 feet.

Excavation of the gneissic bedrock was accomplished with a combination of ripping with an excavator and hoeramming. Due to the fracture patterns in the bedrock, controlled blasting was not required. Figure 6 shows an example of typical rock excavation at the spillway. Because the bedrock was fractured and rippable, the contractor utilized line drilling techniques on the outer perimeter of the excavation area to control the limit of the rock removal. Construction-induced vibrations were limited to the criteria set forth in 527 CMR 13.00, which provide vibration limits based on a relationship between peak particle velocity and frequency. Continuous and event-specific vibration monitoring was performed for the existing downstream masonry face of the dam, the downstream gate ouse/outlet tower, and at locations where new concrete was being poured.

The bedrock conditions in the area of the spillway were generally less competent than it was anticipated during design. Because the top of competent bedrock was generally about 1 to 4 feet deeper than the design bottom of footing grades, approximately 150 additional cubic yards of concrete were required to reach the design footing subgrade elevations. The final bedrock surface was cleaned out with compressed air prior to the placement of the concrete. At the east bridge abutment, a "shear key" into the bedrock subgrade was added because of the lower than anticipated competent bedrock surface. This shear key was added to increase the passive resistance for the training wall/bridge abutment foundation.

Figure 7. Concrete work during winter 2010 / 2011 at the spillway.

December 2010 marked the beginning of concrete placement for the new spillway. It also marked the beginning of one of the coldest winters in recent years in the Berkshires.

As shown on Figure 7, concrete was formed and poured in heated tents, and was typically allowed to cure for at least three days within the heated tents prior to stripping forms. Diligent maintenance of the heating system by the contractor allowed for concrete placement to occur relatively unimpeded despite the cold weather. Rock excavation was not allowed for 24 hours after concrete had been poured on the site, and backfilling was not permitted until the concrete had achieved its required 28-day compressive strength. Field-cured cylinders were used to determine whether required 28-day compressive strengths were achieved, which, thus, allowed backfilling to be performed at an accelerated schedule. The stop log bays foundations, crest gate foundation, as well as upstream and downstream training walls/bridge abutments were all poured throughout the winter in this manner with great success. Side seals and heating elements for the hydraulic crest gate were installed as part of the training wall construction.

By February 2011, work on the spillway had progressed sufficiently to allow backfilling between the existing embankment and spillway training/abutment walls. Freezing temperatures required a diligent effort on the contractor's part to provide ground heaters and frost blankets. The specifications for backfill material were geared toward providing control of seepage through the embankment, as opposed to the free-draining materials commonly associated with roadway and bridge construction. Because of the relatively high fines content (between 15 and 30 percent) of the off-site embankment fill material, moisture content significantly impacted the contractor's ability to compact the material to the required density. The backfill material was brought from an off-site borrow pit to the site with a moisture content that was typically well over its optimum moisture content. This condition, in addition to the rainy/wet weather that became more prevalent in the early spring of 2011, caused delays in the backfilling operation. The wet backfill issues were addressed primarily by providing better control of moisture in both the on-site and off-site borrow area stockpiles. In some instances where moisture could not be properly controlled, the amount of fill placed in a single day had to be limited to allow porewater to dissipate prior to the placement of the next lift of fill. Despite the adverse conditions, the backfilling was completed, the stop logs were installed and the temporary cofferdam was removed in time for the scheduled bridge/road reopening in March 2011.

The new bridge has a 20-foot-wide roadway (curb-to-curb) and a 6-foot-wide sidewalk on the north side, resulting in an overall width of 29 feet. The bridge can accommodate two travel lanes, where the previous "temporary" Bailey bridge could not accommodate travel in both directions. The wider bridge required the location of the new bridge to be shifted southward, with respect to the previous bridge, which resulted in a straightening of the layout of Tolland Road.

Figure 8. New spillway bridge installation.

The bridge installation was performed using a 60-ton crane set up at the east (right) abutment of the spillway (see Figure 8). Completing the spillway backfilling operation was critical since it needed to be completed ahead of the bridge installation in order for the bridge and Tolland Road to be reopened to the public by March 2011. The bridge was fabricated in the three sections, which were trailered individually to the dam and staged on the dam crest. The crane was able to set each bridge section into place in one day. The grouting of the bridge sections and the installation of the bridge railings were completed over a two-week period and the bridge was opened on schedule on March 18, 2011.

8. A Little More Than Just a "Wet Test"

Between the first and second construction seasons, on August 27-28, 2011, Otis Reservoir received between about 7 and 10 inches of rain from Tropical Storm Irene. In anticipation of the storm, the DCR began to lower the reservoir level with the outlet gate two days prior to the storm's arrival in the area. The outlet tower slide gate was kept open during and after the storm to help control the reservoir level. By the time the storm hit, the DCR had removed up to three rows of stop logs (1.5 feet). The reservoir level eventually rose to about 12 inches over normal pool (as seen in Figure 9) on August 29, 2011, which is about 6 inches above the maximum reservoir level used to design the stop logs. On September 1, 2011, the DCR reported that the three rows of stop logs had been replaced and gate in the outlet tower was closed. The DCR inspected the spillway and did not observe any noticeable damage.

9. Second Year Construction: The Home Stretch

The contractor re-mobilized to the site fulltime in October 2011 in order to begin construction on the last parts of the project. Final pavement of the road and bridge was placed and the pedestrian railings were completed. Construction of the new gatehouse began concurrently with the installation of the hydraulically actuated crest gate. The new gatehouse was constructed on the left (west) side of the spillway:

- to house equipment and controls associated with the hydraulic crest gate;
- to house the remote water level instrumentation system; and,
- to provide secure storage for the aluminum stop logs.

The crest gate (shown in Figures 10 & 11) was installed by the end of December 2011. The hydraulic system was installed and tested as part of the "dry test" in January 2012. A "wet test" of the crest gate system was also performed shortly thereafter. Although the reservoir was at the winter pool elevation, water could be pumped from the reservoir into the area between the stop logs and crest gate. So, once again, the inclusion of the stop logs provided a benefit to the project that went beyond their primary objective.

The crest gate system was substantially complete by mid-March 2012 when a training session was held for the DCR personnel who would be operating the new system.

The new slide gates were installed "in-the-wet" inside the downstream tower on its upstream face. The new slide gates included new electric actuators (with manual backup). These new slide gates were installed with new nchor bolts drilled into the downstream face of the outlet tower, with cement grout bedding installed between the gate flange and the outlet tower wall. However, the contractor had problems during the installation of the new gates. During startup testing, the slide gates leaked significantly through the grout bedding and flowed behind each gate's flange and through several of the anchor bolt holes. The reason for the leakage was attributed to problems encountered by divers used during the installation of the grout bedding. Several repair attempts were made. However, the leakage remained beyond the specified allowable limit. As of 2013, the DCR is evaluating repair options to address the slide gate leakage.

9. Summary

The Otis Rehabilitation Dam and Bridge Rehabilitation Project (see Figure 12) was a success. The planning efforts, which were initiated by the dam safety inspections, led to a project that ultimately benefited the DCR, the town of Otis and the general public. The crest gate spillway provides the needed hydraulic capacity to the dam in order to mitigate overtopping and the potential failure of a high hazard dam. The crest gate also provides a self-regulating winter pool level that significantly improves the DCR's reservoir operations. The new bridge replaces a temporary one-lane bridge that was in place for approximately 15 years beyond its intended service life and provides a permanent two-lane bridge that will benefit the users of the Otis Reservoir. Even though the project initially started out with the primary goal of addressing the dam safety issues at the dam, the evolution of the project ultimately provided a broader and more substantial benefit to each of the project stakeholders.

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Curved Steel Girder Integral Abutment Bridges in Vermont, USA

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Abstract

The Vermont Agency of Transportation has constructed four curved steel girder integral abutment bridges of varying details that have been in service from three to twelve years. These designs are unique in the New England region and documentation of these designs and performance may aid wider implementation of these bridge types. Spans range from 86 ft. to 140 ft., which are typical of many bridge replacement projects. Arc span divided by girder radius, ranges from 0.067 to 0.358 radians and girder stiffness varies across the spans. Therefore, these structures could not be designed as straight structures per AASHTO guidelines, and published curved bridge design guidelines do not apply to integral abutment structures. Instead, these bridges required special design considerations which were addressed through straightforward design assumptions. Each has distinct characteristics that are notable, from curvature of 23.87 degrees with superelevation and grade change at the Danby Bridge, 30 degree skew and curvature of 2.75 degrees at the Bradford Bridge, and two-span structure with curvature of 11.25 degrees and twelve years in service of the Stockbridge Bridge. Site visits were made to all four structures and the current condition was documented, including any observed cracking. Any distress noted was not attributable to the bridge curvature, but was common to the detailing used. The condition of all four of these bridges is very good, based on author site visits and bridge inspection reports., and they are performing as expected.

Keywords: bridge, jointless, integral abutment, curved, performance, inspection, case study

1. Introduction

Integral abutment bridges (IABs) are widely used throughout the United States as efficient and durable structures. By eliminating expansion joints within the bridge span, the inherent problems associated with failing bridge joints are avoided. In states with harsh winters, such as in New England, failed bridge joints lead to exposure of the superstructure and substructure to anti-icing salts and result in significant corrosion, spalling and other deterioration. IABs can also result in significant initial cost savings due to the minimally invasive foundations, which avoid cofferdams, dewatering requirements, and environmental impacts. Therefore, in this and many regions of the country, IABs become the bridge of choice when appropriate for the design conditions. In addition, IABs are generally more resilient structures, reported to perform well under earthquake excitations and flood conditions. (Figure 1 shows an example where an IAB allowed for a resilient response after Tropical Storm Irene in 2011 in an area where many

There are complications with IABs, however, as making the superstructure integral with the substructure introduces frame action of the entire assembly and introduces significant effects of soil-structure interaction. Prior to completion of the abutments the structure behaves identically to a conventional bridge, as there is no restraint so long as the bearings are designed similarly, which is standard practice in VT. Therefore, superstructure construction issues of girder fit up, cross frames, and dead load distribution can be designed similarly to non-integral designs. Designs of straight IABs of moderate spans can still be done with straightforward assumptions. However, when large skew or any curvature is introduced, appropriate design assumptions are not clear and simplified methods developed for jointed structures are not applicable. This has led many Departments of Transportation (DOTs) to place restrictions on skew or curvature when designing an IAB.

Figure 2. Resilience of an IAB in Vermont after 2011 Tropical Storm Irene (Photo Courtesy of VTrans)

The AASHTO (2017) Bridge Design Specifications definition of "straight" bridge analysis 4.6.1.2.4b for steel I girders requires the following: concentric, bearing not skewed more than 10 degrees from radial, stiffness similar in all girders and arc span (distance between centers of adjacent bearings) divided by the girder radius in feet less than 0.06 radians. Bridges not meeting these criteria must use more advanced design methods, including approximate analysis such as V-Load or M/R methods or full finite element methods (FEM). However, the approximate analysis methods are not applicable to IABs and a full FEM is not typical in design of a short to moderate span structure. ''Guide Specifications for Horizontally Curved Highway Bridges'' (AASHTO (2003)) and the "Development of LRFD specifications for horizontally curved steel girder bridges" (NCHRP Report 563 (2006)) do not address effects of integral abutments on curved bridge behavior.

A survey by Maruri and Petro (2005) collected responses from 39 state DOTs and reported allowable maximum IAB curvature ranging from zero to ten degrees. More recently in summer 2019, the lead author of this paper (Civjan) consulted Bridge Design Manuals and contacted Bridge Engineers for each of the six New England and New York State DOTs, and found that requirements have changed in the past 15 years, but there are inconsistencies in how curvature in IABs is addressed. In 2019, one DOT has a horizontal curvature upper limit for IABs of 5 degrees while another limits curvature to the definition of a straight bridge in AASHTO. The other states either did not limit curvature but considered it on a case-by-case basis or omitted any reference to curved IABs, with one specifically stating that curvature was allowed for IAB designs. Of the six New England states, only Vermont responded that they have constructed curved girder IABs (two state DOTs did not respond but their guidelines were the most restrictive on curvature limitations), even though several states have considered curved IAB designs in past projects. Nationwide, other curved girder IABs have been constructed, such as the three span structure reported and modeled in Hoffman and Phares (2014).

Kalayci et al. (2012) performed a parametric FEM analysis of a specific two span, curved girder IAB. The results indicated that designers should consider curvatures up to 10 to 20 degrees for design, so long as differences from straight girder IABs are accounted for. Deng et al. (2015) used FEM to study a specific three span, curved girder IAB and conclude that IABs with 10 degrees of skew and 0.06 radian arc span length can be designed as a straight bridge if 10 percent increases in stresses are allowed. Both of these studies indicate that curved girder IAB design is practical and advantageous for certain conditions. It should be noted that single span behavior of IABs differs from multi span behavior, so some caution should be exercised when generalizing results.

The Vermont Agency of Transportation (VTrans) initiated a research project in 2009 that evaluated three IABs through extensive instrumentation and monitoring during construction, load testing with loaded dump trucks, and approximately five years of long term monitoring (Civjan et al. (2014a and 2014b)). This included the first curved girder IAB in VT (Stockbridge Bridge), along with two straight girder IABs (one with skew). Based on this study it was concluded that the two span curved girder IAB (Stockbridge Bridge) was performing as designed, was only marginally different in response predicted (from calculations and FEM) from a straight bridge of similar dimensions, and differences from straight IABs could be accounted for in design. Subsequently, VTrans has designed and constructed three additional curved steel girder IABs.

This paper provides case studies of four curved girder IABs constructed by VTrans since 2009, including any unusual features, and information from walk-through evaluations of each structure on June 5, 2019 and July 22, 2021. These structures are in very good condition and have distinct advantages to jointed structures or straight IABs that could have been constructed at these sites. It is hoped that this information will be useful to other DOTs considering the design of curved girder IABs and provide evidence of the acceptable performance of these structures.

2. Bridge Design Concepts

The curved steel girder IABs described were designed in-house by VTrans and are typical of VTrans IAB designs, with steel girders and abutments supported on steel piles bending about their weak axis in the longitudinal direction of the structure (as opposed to Figure 1 which was an older IAB with strong axis pile orientation). All piles are embedded 2.0 ft. into the abutment and driven to bedrock for the bridges described, though friction piles would also be allowed.

The typical design process in VTrans straight IAB structures is to design the superstructure and substructure independently. The superstructure dead load is evaluated similarly to a jointed bridge since both structure types are similar until the abutment is completed. The superstructure is also designed as a jointed bridge for live load, which has been reported to be a conservative approach in Dicleli and Erhan (2009) and Dicleli (2010) for single span straight pre-stressed girder IAB bridges. Simple calculations of coefficient of thermal expansion times length are then performed to determine the bridge expansion and contraction under thermal load. These thermal results are then used as a top of pile deformation and a soil interaction model is completed of an individual pile using a standard nonlinear finite element analysis

program used by VTrans for pier design. If pile yielding is noted, the pile evaluation is either re-run assuming a pin at the top of pile to introduce full yielding or a larger pile can be selected. Only compact H-pile sections are used.

The curvature in these curved girder IABs was evaluated through straightforward design, using conservative assumptions. The superstructure was designed as a non-integral structure for dead and live loads. For thermal load, the typical pile design was modified to include pile displacements in both longitudinal and transverse directions based upon the angle of the anticipated bridge movement versus the orientation of the abutment. Movement occurring along the chord line between abutments, or to piers if a multi-span bridge, was calculated as the coefficient of thermal expansion times chord length. The longitudinal component of this movement was considered similarly to a straight IAB bridge, and transverse component was used to apply biaxial pile foundation effects, though transverse movement could be ignored if found to be negligible. This analysis results in the inclusion of biaxial bending in pile design.

The design of the Bradford Bridge and Jamaica Bridge, which have minimal to moderate curvature, also relied on the findings of Civjan et al. (2014a and 2014b)) to determine that the effects of these values of curvature were not expected to be significant. Designers were aware of possible additional load demands arising from the girder curvature, but designed the bridges similarly to straight IAB designs.

For the high curvature, superelevation, and grade change at the Danby Bridge much more attention to these effects was included, but the overall design process focused on the modifications to the traditional VTrans IAB design process described above rather than incorporating a significantly more detailed process. A longer span of this curvature would have considered a full three-dimensional finite element model including soil-structure effects, but the focus of the selected bridges was to use simple models for design assumptions. Transverse pile forces and deformations due to curvature were still estimated based on geometric relationships, with more attention paid to the biaxial bending effects in piles. All movement was assumed to occur at the lower bridge elevation, though the higher abutment was subsequently designed to accommodate some movement as well. Vuggy rock conditions required piles be drilled through the vuggy zone and placed in pre-excavated holes that reached competent rock. These site conditions precluded the use of shallow foundations.

The Stockbridge Bridge was the first curved girder IAB constructed by VTrans. This bridge is a two span structure with moderate curvature. As the initial design of its kind, significantly more detailed evaluation of the expected bridge movement was completed to evaluate the curvature than in the other bridges. This included a design concept of orienting the wingwalls such that the total backfill reactions would act in a radial direction along the superstructure, and some methods employed to minimize variation of backfill pressures. These were each accomplished through geometric calculations in order to develop detailing and provide conservative estimates of forces and deformations, but the overall structure was designed in line with typical VTrans IAB design of straight girder bridges at the time.

The designs of these structures were completed without any significant modifications to typical bridge design practices. Construction and detailing of the structures followed a combination of practices for curved jointed bridges and straight IABs, with some conservative assumptions made to accommodate the non-typical components. These methods can be incorporated into typical design practice without requiring complicated three-dimensional FEM for short to moderate span curved girder bridges of low to moderate curvature.

3. Expected IAB Performance

IAB designs are selected due to their expected improvements to durability and resilience of the structures. By eliminating bridge joints many of the long term corrosion issues with New England bridges can be minimized. However, the resulting soil-structure interactions result in stress distributions and movements that differ from jointed bridges. For VTrans IAB designs some minor cracking in the abutments and wingwalls is typical. These will be noted in the bridge photos, but have had minimal effects on long term performance and appear to stabilize with time in existing IABs.

In the abutment back wall at the bottom flanges of girders there is often a 45 degree crack propagating downward from the corner of bottom flanges. Cracks between an integral wingwall and abutment above the construction joint can occur. These all appear to be stable with time. Minor transverse cracks at regular intervals at the edge of the deck are typical, sometimes propagating across the deck, which can be larger in heavy integral curb or parapet barriers when included along the edge of the deck. Earlier designs (such as the Stockbridge Bridge) exhibited a minor vertical abutment crack between girders. VTrans policy (Structures Engineering Instructions SEI 17-001) since 2017 is to reduce abutment horizontal reinforcing spacing on the interior face to 6 in. to reduce these cracks. Structures with subsequent designs have not exhibited these cracks.

When skew is included in an IAB design some cracking at the acute corner wingwall above the construction joint has occurred. These cracks appear stable with time and are not expected to affect service life as corrosion resistant reinforcing steel is included above the construction joint. Curvature is expected to introduce similar effects as skew, with thermal expansion between corners of the bridge causing out of plane rotations of the structure. This could potentially lead to service issues such as bridge joint deterioration and additional stresses and cracking in the abutments if not accounted for in design.

4. Bridge Descriptions and Evaluations

Each of the structures is presented independently in the following sections. For each bridge, general details are provided including any unusual features of the design, followed by description of any deterioration noted on the walk-through of June 5, 2019, and any new observations from the walk-through of July 22, 2021. The initial evaluations took place the day after a

rainstorm, allowing for visual observation of any moisture penetration, while the latter followed a dry day. General details of the bridges are found in Table 3.

The definition of curvature used in this paper is the angle in degrees per 100 feet of central girder arc length

$$\left(Curvature = \frac{180}{\pi}\left(\frac{100ft}{Radius}\right)\right)$$

The AASHTO designation of arc span divided by girder radius will also be included in bridge descriptions.

4.1 Bradford Bridge

The Bradford Bridge (Figures 2 and 3) has a very small curvature and relatively short span but high skew. The bridge replaced a traditional jointed bridge that had a curved deck on straight steel girders. The incremental cost for curved girders rather than straight was approximately 5 cents per pound, so was determined to be worthwhile to improve aesthetics. The single span is 140 ft. between centerline of abutments and has a central radius of 2083 ft. resulting in arc span to radius of 0.067 radians and curvature of 2.75 degrees. There is a superelevation of less than 4 percent (varies along span) at road level and minimal vertical elevation difference between start and end of the bridge. The bridge has a 9 in. exposed deck with longitudinal grooving. Abutments are skewed 30 degrees supported on six HP 12X84 piles oriented with the bridge span that extend approximately 70 ft. and 25 ft. below the Abutments 1 and 2, respectively. The six steel girders are metalized due to the low clearance to the stream water level, which would prohibit weathering steel (less than 10 ft. clearance). Girders are supported by steel reinforced elastomeric pad bearings with a single line of anchor bolts and embedded into the abutments. Girder sizes are 1/2X35 webs and top and bottom flanges of 2X16 and 2-1/2X16 for the four girders at the inside of the curve (Girders 1 through 4), 2-1/2X16 and 3-1/2X16 for the two girders at the outside of the curve (Girders 5 and 6), respectively. Width of the bridge is 31.33 ft. to outside of fascia with galvanized bridge rails. Abutments are 3.5 ft. thick, approximately 10 ft. tall and constructed from a precast lower section and cast in place upper section that also fills the voids in the lower precast section that encase the pile tops. Wingwalls are also precast concrete sections that are 2.0 ft. thick and extend 7 to 8 ft. from the abutment centerline, connected at the abutment face by dowels and mortar. The bridge was constructed in 2017.

The bridge had been in service for 2 and 4 years when evaluated. The overall condition is very good. There are some minor deck cracks approximately 15 feet and 30 feet from the West abutment and at midspan (Figure 4). These do not progress across the entire deck but are allowing some moisture penetration (there is no waterproofing on this deck).

As is typical in many IAB abutment backwalls, a 45 degree crack propagating from the bottom flange of most girders was observed, on the side of the obtuse angle between girder and abutment (Figure 5). There is a filled joint between the abutment and precast wingwalls, with dowels between the two components providing rigidity. No distress was noted in these details (Figure 6). Due to cost, an asphalt plug joint was used rather than an installed joint option at the end of the approach slab. These joints are failing due to the high skew (Figure 7), and would normally not be used for this skew angle. The joint condition is not attributed to the curved IAB design. The follow up evaluation in 2021 observed slightly more scaling on the top of the abutments (Figure 8), continued deterioration of the asphalt plug joint (Figure 7b), and a very slight increase in efflorescence at cracks, though cracks were less noticeable due to the dry conditions.

Overall, the condition of this bridge is very good and performing as expected. Official bridge inspection reports were completed in 2017 and 2019 and indicated 8 (Very Good) condition ratings for deck, superstructure, and substructure.

Table 3. Bridge Details

Bridge Name	Location	Lanes carried	Span length (feet)	Curvature (degrees)	Arc Span to Radius (radians)	Skew (degrees)	Year Open
Bradford Bridge	43.987525, -72.138899	2	140	2.75	0.067	30	2017
Jamaica Bridge	43.099619, -72.782144	2	130	8.16	0.185	N/A	2013
Danby Bridge	43.339344, -73.039001	2	86	23.87	0.358	N/A	2018

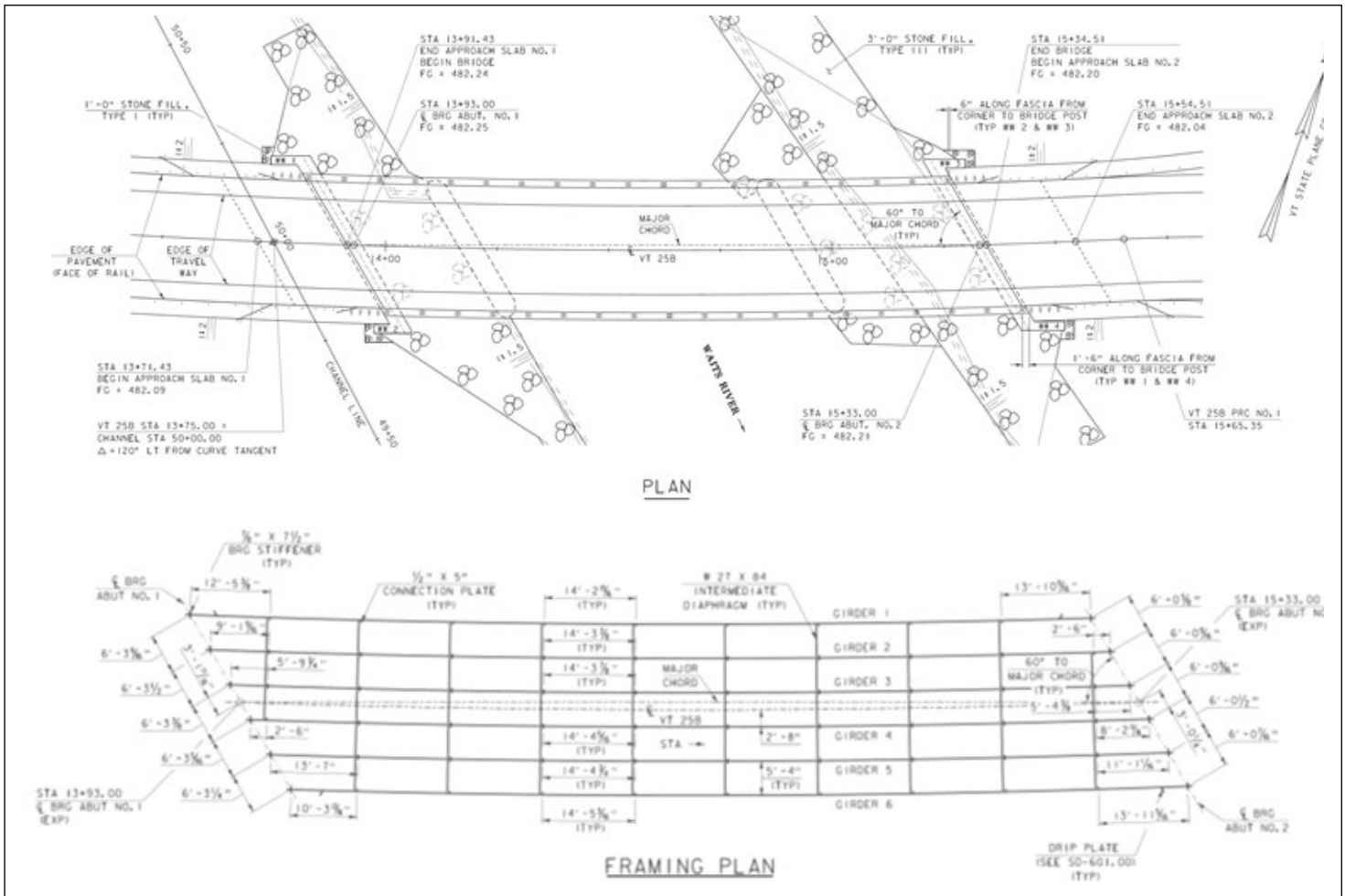


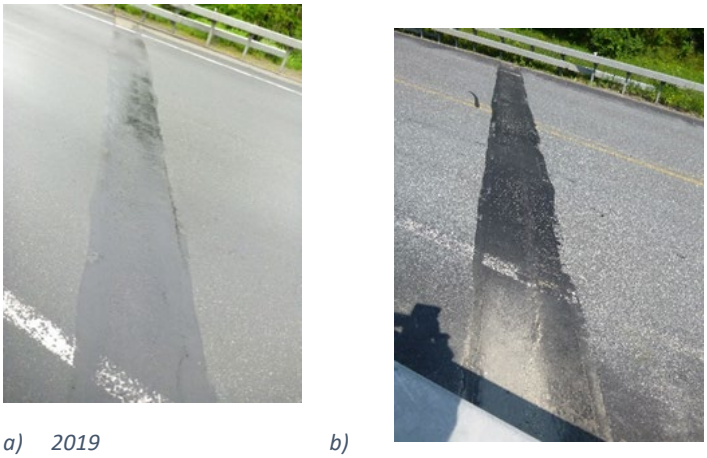
Figure 4. Plan Views of Bradford Bridge (VTrans - Used by Permission)

Figure 3. Overview of Bradford Bridge

Figure 5. Typical Deck Cracking at Bradford Bridge

Figure 6. Crack in Abutment at Girder Bottom Flange at Bradford Bridge

Figure 7. Doweled Precast Wingwall to Abutment at Bradford Bridge



a) 2019 b) 2021

Figure 8. Deteriorating Asphalt Plug Joint at Skewed Bradford Bridge

Figure 9. Scaling at Top of Abutment at Bradford Bridge (2021)

4.2 Jamaica Bridge

The Jamaica Bridge (Figures 9 and 10) has a moderate curvature with no skew. The bridge replaced a two span rolled beam bridge washed out during Tropical Storm Irene. The new single span is 130 ft. between centerline of abutments and has a central radius of 702.5 ft. resulting in arc span to radius of 0.185 radians and curvature of 8.16 degrees. There is a superelevation of 3.2 percent at road level and a vertical elevation difference of 4.05 ft. between start and end of the bridge. The bridge has an 8-1/2 in. deck with a membrane and bituminous pavement. Abutments and wingwalls are cast in place and integral with each other. Abutments are supported on seven HP12X84 piles that extend approximately 25 ft. and 17 ft. below the abutment at Abutments 1 and 2, respectively, and socketed 3 ft. into bedrock. The seven steel girders are of weathering steel and supported by steel reinforced elastomeric pad bearings with no additional constraint, and embedded into the abutments. Girder sizes are 1/2X48 webs and top and bottom flanges of 1X20 and 1-1/4X20 for the four girders at the inside of the curve (Girders 1 through 4), 1-1/4X20 and 2X20 for the next two interior girders (Girders 5 and 6), and 1-1/4X20 and 2-1/4X20 for the exterior girder at the outside of the curve (Girder 7), respectively. Width of the bridge is 41.00 ft. to outside of fascia with galvanized bridge rails integrated with a concrete support and a sidewalk on one side of the structure. Abutments are 3.0 ft. thick and range from approximately 11.5 ft. to 13.5 ft. tall. Wingwalls vary at the two ends of the bridge. Abutment 1 has parallel wingwalls extending the abutment from 4.4 ft. and 12.0 ft. from the fascia, with an additional pile under the longer section. Abutment 2 has perpendicular 1.5 ft. thick wingwalls that are tapered, from approximately 12 feet to 6 feet in height and extending 11.5 ft. from the abutment centerline. However, the connection of the wingwalls differ significantly at Abutment 2. Wingwall 1 has a cold joint with the abutment and specifies mechanical splices and epoxy filler between the wingwall and abutment. Wingwall 2 instead relies on an integrally cast wingwall and abutment. The intent was to compare the two details. The bridge was constructed in 2013.

The bridge had been in service for 6 and 8 years when evaluated. The overall condition is very good. The bridge includes a partial concrete barrier below the railing, curb, and sidewalk. As appears typical of these designs for both IAB and non-IAB bridges, cracks have formed at approximately 6 ft. spacing that are wider at the top of barrier and narrower as they extend toward the deck, with some extending to the bottom of deck (Figure 11). At these crack locations, there is efflorescence at and below the top of the road surface. These cracks are at almost every location of form inserts and at a few intermediate rail anchor locations (noted in Figure 11). Additional cracking is found at the sharp transition in concrete barrier height near the abutments. The underside of the deck exhibits some minor map cracking, which may indicate deck curing problems (Figure 12), but was far less noticeable in dry conditions. Both of the in-line wingwalls at Abutment 1 do not show any signs of distress. At Abutment 2, wingwalls are at right angles to the abutment and integral Wingwall 2 at the interior of the curve has minor cracking at 45 degrees at the interior of the abutment and wingwall interface, typical of other IABs (Figure 13a). Wingwall 1 on the outer exterior of the curve has a mechanical splice, epoxy filler and bevel. A large crack has formed at this location, extending through the entire interface, though not following the cold joint, along with some chipped concrete along the outer edge of the abutment and wingwall (Figure 13b). No change was noted in 2021. This detail has poor results and should not be used. It is noted that a relatively similar detail was used at the Bradford Bridge with no distress noted. It should be further investigated whether the difference is due to the tapered wingwall design or detailing implemented. The asphalt plug joints are performing adequately though the thin asphalt seal at the approach slab to bridge deck connection is showing some deterioration. The follow up evaluation in 2021 observed an increase in crack size on parapet locations, slight increase in efflorescence at cracks, though cracks were less noticeable due to

Figure 10. Overview of Jamaica Bridge

the dry conditions. It was noted that the cracks in the underside of the deck were far less noticeable in the dry conditions.

Overall, the condition of this bridge is very good and performing as expected. All distress noted is typical of these details in general, and is not an effect of the curved IAB structure. Official bridge inspection reports were completed in 2013, 2015, 2017, and 2019 and indicated 8 (Very Good) condition ratings for deck, superstructure, and substructure.

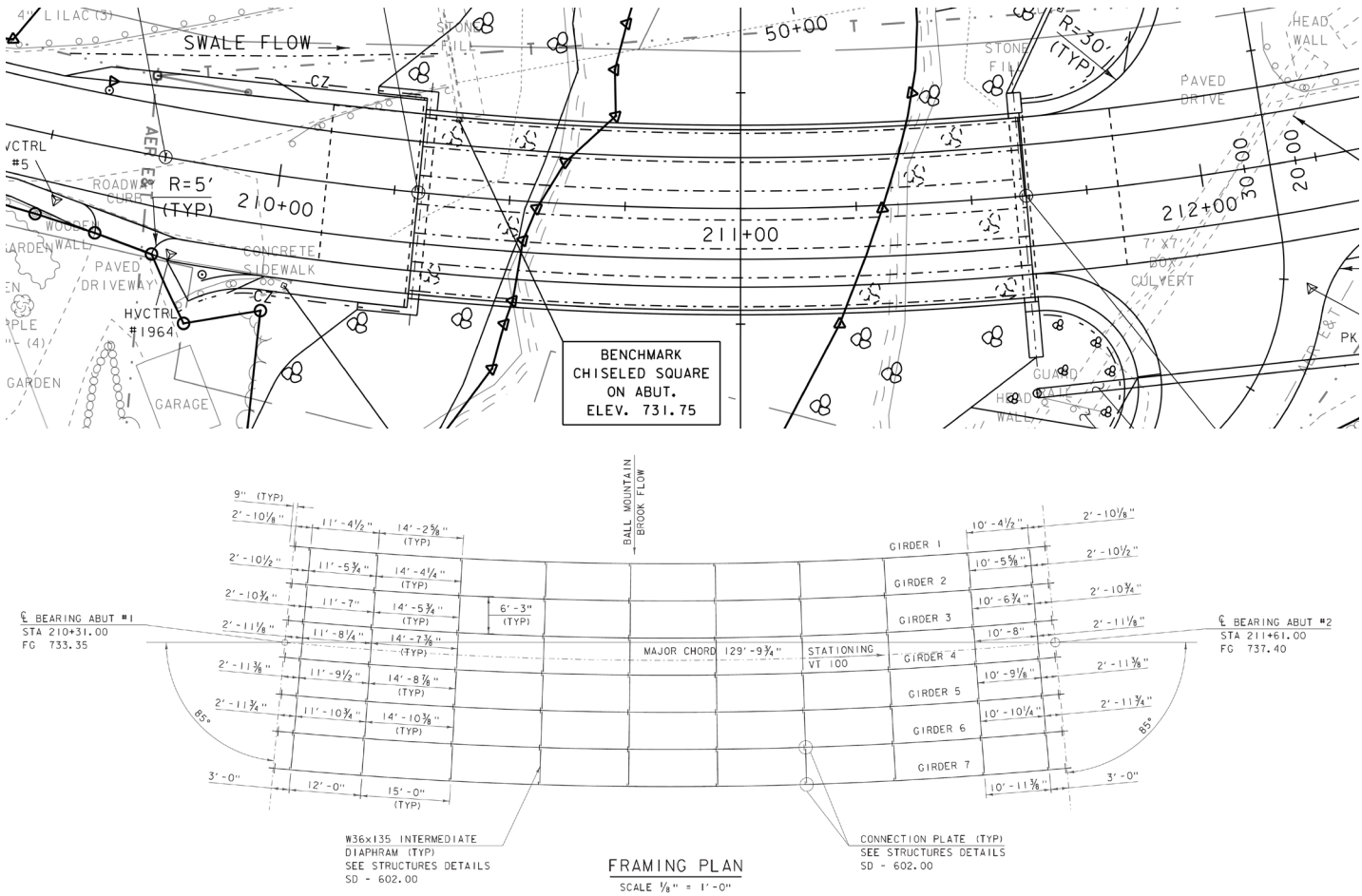


Figure 11. Plan Views of Jamaica Bridge (VTrans - Used by Permission)

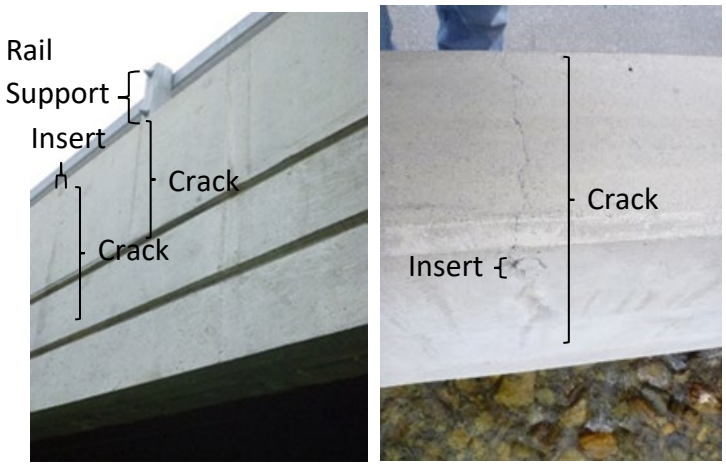
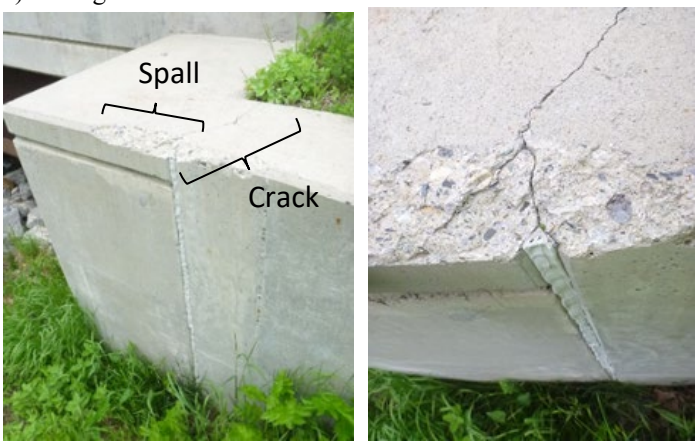


Figure 12. Typical Barrier Cracking at Jamaica Bridge

Figure 13. Underside of Deck at Jamaica Bridge

a) Integral Connection



b) Dowelled Connection at Jamaica Bridge

Figure 14. Wingwall to Abutment Cracking at Jamaica Bridge

4.3 Danby Bridge

The Danby Bridge (Figures 14 and 15) has the highest curvature of these structures and is expected to push the limits of curved girder IAB design, relatively short span, no skew, and high superelevation. The bridge replaced a 74 ft. single span rolled beam bridge constructed in 1933.The new single span is 86 ft. between centerline of abutments and has a central radius of 240 ft. resulting in arc span to radius of 0.358 radians and a curvature of 23.87 degrees. There is a superelevation of 5.6 percent at road level and a vertical elevation difference of 4.02 ft. between start and end of the bridge. The bridge has a 9 in. exposed deck with longitudinal grooving. Abutments are cast in place and integral with wingwalls at Abutment 2. Abutments are supported on four HP12X74 piles that extend approximately 20 ft. below the abutment. Conditions at the site included a layer of vuggy rock which was deemed unsuitable for shallow foundations. Holes were drilled through this layer and pile tips placed on competent bedrock. The four steel girders are of weathering steel, supported by steel reinforced elastomeric pad bearings with a single line of anchor bolts, and embedded into the abutments. Girder sizes are 1/2X42 webs and top and bottom flanges of 7/8X18 for the twor girders at the inside of the curve (Girders 1 and 2) and 7/8X18 and 2-1/4X18 for the two girders at the outside of the curve (Girders 3 and 4), respectively. The significantly different cross sections from the inner to outer girders are due to the sharp curvature. Width of the bridge is 25.33 ft. to outside of fascia with galvanized bridge rails attached to the outer fascia. Abutments are 3.0 ft. thick and range from approximately 13.0 ft. to 14.5 ft. tall. Abutment 1 (higher elevation) has 1.5 ft. thick precast modular retaining walls perpendicular to the abutment extending 12.5 ft. and 14.5 ft. from the abutment, with a flexible joint between the walls and abutment. Abutment 2 (lower elevation) has integral wingwalls, both extending 8.0 ft. from the abutment centerline, with one perpendicular to the abutment and the other skewed 30 degrees. The bridge was constructed in 2018.

The bridge had been in service for 1 and 3 years when evaluated. The overall condition is excellent with only a few minor cracks at the girder bottom flange (Figure 16). To date, there is no cracking at the integral wingwall to Abutment 2 detail (Figure 17). The retaining walls at Abutment 1 are not flush with the back wall, with vertical misalignment approximately 1 in. at the top of the abutment (Figure 18). It appears that this was the condition at the completion of construction as there is no distress of the sealant between the components or soil at the base of the walls. The follow up evaluation in 2021 noted that the misalignment was nearly identical (at similar ambient temperature), with the only change being that the sealant had dried and pulled away from the joint. This will be monitored in future years to look for signs of bridge twisting in plan view. Some new, but minor, efflorescence at cracks bottom flange corners and staining from weathering steel was also observed.

Overall, the condition of this bridge was excellent and performing as expected. Official bridge inspection reports were completed in 2018 and 2020 and indicated 8 (Very Good) condition ratings for deck, superstructure, and substructure.

Figure 15. Overview of Danby Bridge

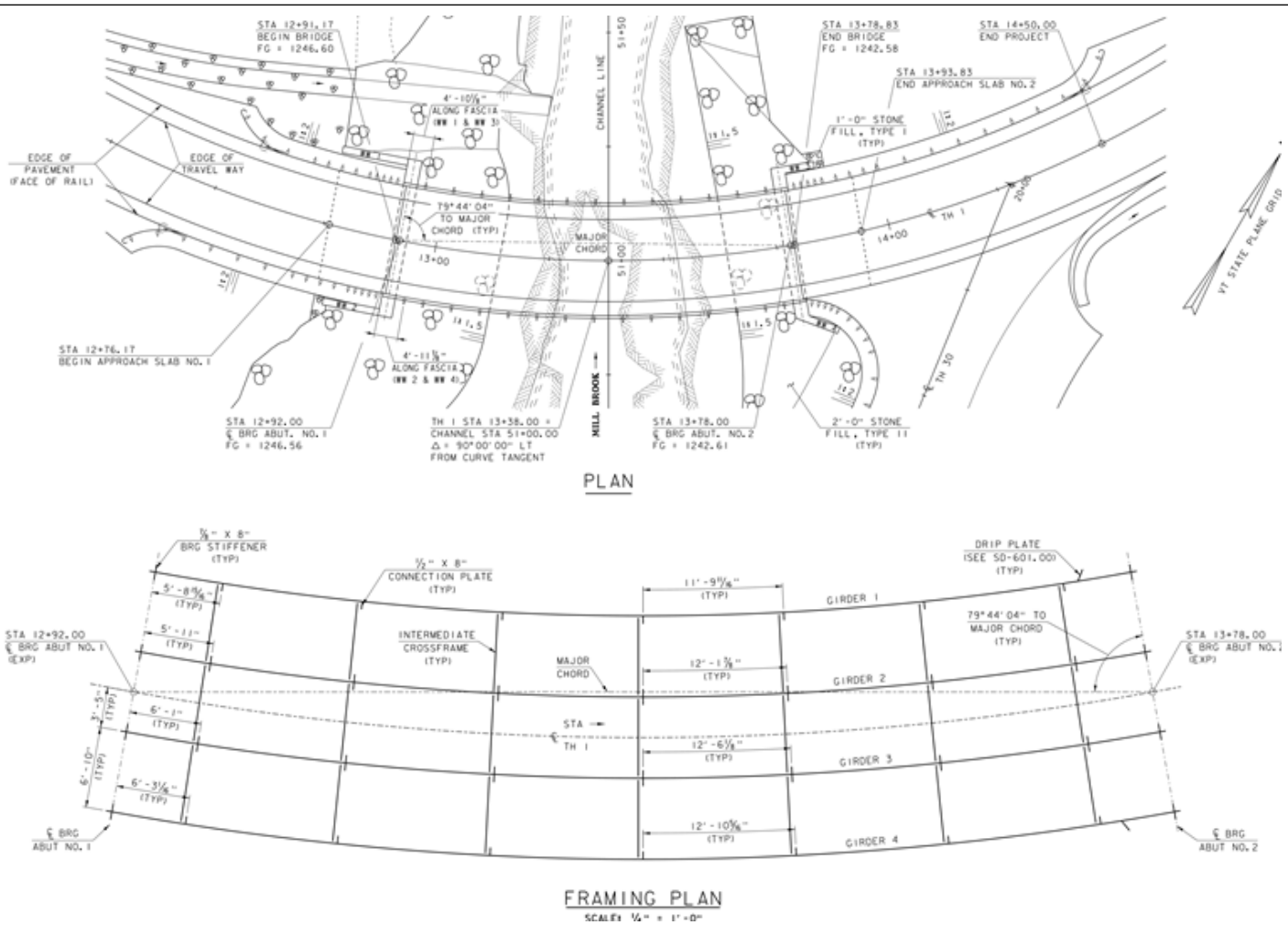


Figure 16. Plan Views of Danby Bridge (VTrans - Used by Permission)

a) 2019

b) 2021

Figure 17. Crack in Abutment at Girder Bottom Flange at at Danby Bridge

4.4 Stockbridge Bridge

The Stockbridge Bridge is a two span structure that has a moderate curvature and no skew (Figures 19 and 20). The bridge replaced a traditional jointed three span curved I-Girder bridge. This bridge is notable for several reasons. It was the first curved girder integral abutment bridge constructed by VTrans and was part of an extensive instrumentation program (results of this instrumentation and monitoring are reported elsewhere (Civjan et al. (2014a and 2014b)). The bridge was constructed in 2009, so has twelve years of service, significantly more than the other structures. It was also in service during Tropical Storm Irene, which overtopped the structure and resulted in a large tree and other debris impacting the side of the structure. It is noteworthy that many other bridges along the White River were washed out in the event, but the Stockbridge Bridge was fully functional post-event. The design of this structure also had some interesting features that make it non-typical. Geofoam was installed between the back wall and backfill soil at each abutment, with the intention of evening out and lowering abutment soil pressures. From instrumentation readings this appeared to be effective, as backfill soil pressures were lower than those obtained in the two other structures that were instrumented which each had significantly shorter spans (though those were single span structures) and pressures were lower than predicted from finite element modeling. In addition, the wingwall design was completed with the specific intent of counteracting curvature effects.

Figure 18. Wingwall to Abutment 2 Connection at Danby Bridge

a) Downstream b) Upstream

Figure 19. Retaining Wall to Abutment 1 Detail at Danby Bridge

Figure 20. Overview of Stockbridge Bridge

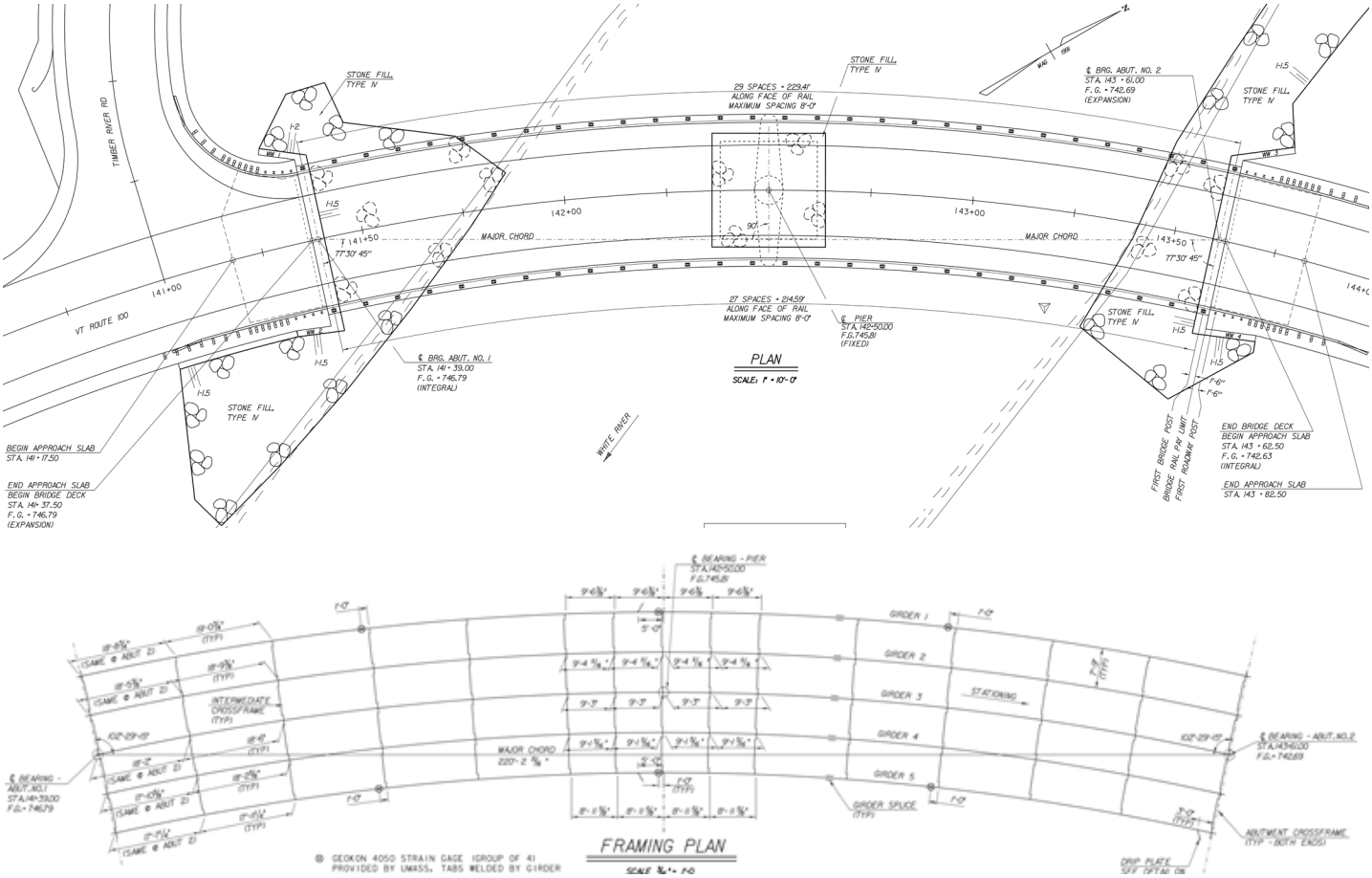


Figure 21. Plan Views of Stockbridge Bridge (VTrans - Used by Permission)

The bridge length is 222.0 ft. along its curved centerline, equally divided between the two spans, with a center radius of 509.3 ft., resulting in arc span to radius of 0.218 radians and curvature of 11.25 degrees. There is a superelevation of 6.0 percent at road level and a vertical elevation difference of 4.16 ft. between start and end of the bridge. The bridge has an 8-1/2 in. deck with membrane and bituminous pavement. Abutments and wingwalls are cast in place and integral with each other. Abutments are supported on five HP14X117 piles that extend approximately 75 ft. below the abutment. The five steel girders are of weathering steel and supported on mortar pedestals with a single line of anchor bolts that were loosened prior to placing concrete when embedded into the abutments. At the pier, the outer three girders are supported by guided bearings transverse to the span and the interior two girders are supported on fixed bearings. Girder sizes vary for the outer spans and over the pier, with splices located approximately 80 ft. from the abutment supports. All webs are 5/8X46. Outer span top and bottom flanges are 1X16 and 1-1/2X16 for the three girders at the inside of the curve (Girders 3 through 5), 1-1/4X18 and 2-1/4X18 for the two girders at the outside of the curve (Girders 1 and 2), respectively. Note that this bridge has numbering of girders (outside to inside of curve) that differs from the others. Over the pier, the top and bottom flanges are similar, 2X16 for the three girders at the inside of the curve (Girders 3 through 5) and 2-1/4X18 for the two girders at the outside of the curve (Girders 1 and 2). Width of the bridge is 37.00 ft. to outside of fascia with galvanized bridge rails integrated with a 12 in. tall concrete curb support. Abutments are 3.0 ft. thick and range from approximately 19.0 ft. to 20.5 ft. tall. Wingwalls are 1.5 ft. thick, integral with the abutment and vary at each location. At each abutment, the one wingwall angle is acute to the abutment, 5 degrees shy of perpendicular, and the other is obtuse at 20 degrees beyond perpendicular. Abutment 1 wingwalls extend 10 feet from the abutment centerline, while the dimension is 14 ft. at abutment 2. Wingwalls are each tapered differently at the bottom with resulting height reductions of 2 ft. to 10 ft.

The bridge had been in service for 10 and 12 years when evaluated. The overall condition is very good. The underside of the bridge shows some transverse deck cracks (Figure 21) that are larger near the abutment. These appear to line up with the cracking in the concrete barrier (Figure 22), regularly spaced approximately every 4 ft. to 8 ft. and often originate at the location of guardrail anchors. Vertical cracks in the abutment backwall between the girders were noticed (Figure 23), but were not significant. Newer VTrans designs decrease the spacing of abutment reinforcement to 6 in. in order to prevent these cracks in all IABs (Structures Engineering Instructions SEI 17-001 (2017)), and newer designs appear to avoid this cracking. At the abutments, which are integral with the structure, there is typically cracking at the intersection between the wingwall and abutment (Figure 24). Erosion prior to Tropical Storm Irene and greatly exacerbated by the event is apparent at the upstream precast boxes used to house instrumentation adjacent to the abutment. The asphalt plug joints are failing, which is typical for this type of joint after 10 years of service (Figure 25). As noted, geofoam was placed behind the abutments in this structure and data indicates that it reduced backfill soil pressures from those expected without geofoam and avoided peaks in backfill pressure values. There is no indication that this geofoam has any deleterious effects on bridge durability at the backwall and approach slab locations. Most significantly, all signs of distress (cracking) aside from the asphalt plug joint was nearly identical to that reported at the conclusion of the instrumentation project in 2014, indicating that the cracking is stable after the first few years of service pending any corrosion of reinforcement. While efflorescence was present at some cracks, no other staining was observed. The follow up evaluation in 2021 indicated minimal if any changes to condition from 2019 (cracks were less noticeable due to the dry conditions), with the following exceptions. At the intersection between the wingwall and retaining wall the closed cell foam was slipping out of the joint, resulting in some sluffing of ground cover aggregate at this location (Figure 26). The plug joint and end of approach slab both showed expected continued deterioration of the asphalt materials.

Overall, the condition of this bridge is very good and performing as expected. Official bridge inspection reports were completed in 2011, 2013, 2015, 2017, and 2019 and indicated 8 (Very Good) condition ratings for deck, superstructure, and substructure.

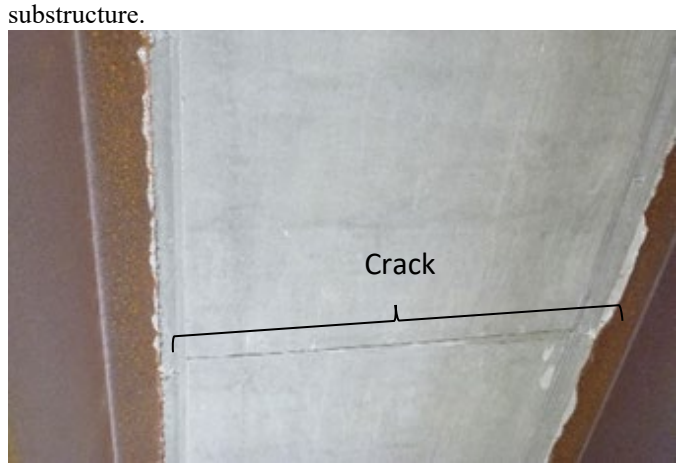


Figure 22. Deck Crack at Stockbridge Bridge

Figure 23. Barrier Crack at Stockbridge Bridge

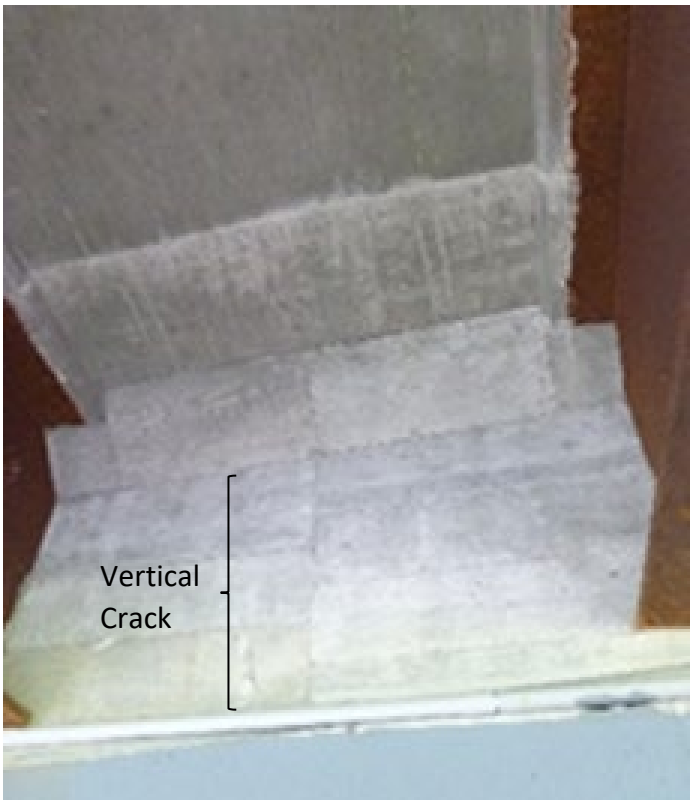


Figure 24. Vertical Abutment Crack at Stockbridge Bridge

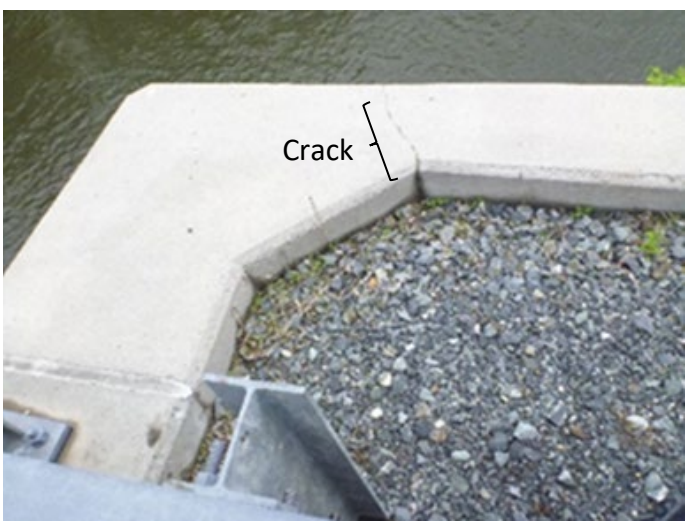
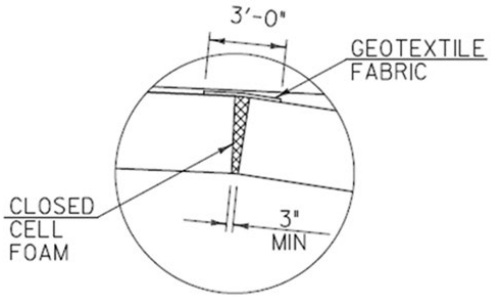


Figure 25. Abutment to Wingwall Connection Crack at Stockbridge Bridge

Figure 26. Deteriorated Asphalt Plug Joint at Stockbridge Bridge



a) Construction Detail

b) Missing Foam 2021

Figure 27. Abutment to Wingwall 2021 at Stockbridge Bridge

5. Estimated Cost Savings

Integral abutment bridges can result in significant cost savings compared to a traditional design, due to the minimally invasive foundations that avoid cofferdams, dewatering requirements and environmental impacts. VTrans provided estimated savings for the Stockbridge Bridge at the time of construction and noted total savings of $821,000 on a total as bid project cost of $4,155,879. The savings were itemized as follows: Cofferdam ($150,000), Substructure ($308,000), Steel Piling ($310,000), Backfill ($33,000) and Excavation ($20,000). It was noted that the savings on this single structure exceeded the total research expenditures used to instrument, model, collect data, and evaluate the performance of this and two other (non-curved) IABs. Through the evaluation of the Stockbridge Bridge, cost savings have now been realized on the three additional curved IABs (reported in this paper) VTrans has constructed. The savings were estimated at 30 percent or more on the Bradford Bridge and Jamaica Bridge projects. The cost savings on the Danby Bridge are difficult to specify since the vuggy rock site conditions precluded shallow foundations and affected the pile design and construction. For the conditions at the site, the IAB design provided savings, but the extent of work required to design a jointed bridge at the site was not pursued since the site conditions were not known during the initial design.

6.0 Conclusions

Four curved steel girder IABs in service in Vermont have been described in this paper. All four of these structures have distinct characteristics that are notable, from the high degree of curvature and superelevation at the Danby Bridge, high skew at the Bradford Bridge, and the two spans and time in service of the Stockbridge Bridge. These structures are very aesthetically pleasing and should provide significantly more durability than an equivalent jointed bridge. Arc span divided by girder radius range from 0.067 to 0.358 radians and girder stiffness varies across the spans, therefore these structures cannot be designed as a straight structure per AASHTO (2017) and required special consideration. All designs were completed by VTrans, using conservative but straightforward, assumptions based on straight IAB design and accounting for vector components for longitudinal and transverse force and displacement components. Designs required bi-axial bending considerations in pile design and displacements, and additional wingwall reinforcement. Most importantly, these curved steel girder IABs were able to be provided at significant cost savings compared to a traditional design, estimated at 30 percent or more, due to the minimally invasive foundations which avoid coffer dams, dewatering requirements and environmental impacts. Evaluation of these bridges at 1 to 12 years of service indicates very good condition, and this was corroborated by bridge inspection reports. Some details that had minor distress were not specific to IABs, including cracking of concrete barriers and associated transverse deck cracks. Specific to IAB construction are minor abutment backwall cracking at the bottom flange of girders and cracking at the wingwall to backwall connection when these were designed as an integral concrete pour. These patterns of cracking are also observed in straight IABs of similar design and are not of concern.

The benefits of IABs are significant and have led to their preference as a design of choice by most state DOTs. However, many states do not construct curved steel girder IABs, or limit their applications, with some Bridge Design Manuals explicitly prohibiting curvature in IAB designs. Based on the VTrans experience, it is recommended that curved girder IABs be considered as an effective option for short to moderate span structures. The bridge analysis, design, and detailing should consider the additional forces and movement effects related to the curved geometry of the bridge, but the VTrans experience shows that this can be accomplished through straightforward procedures and does not require extensive modeling of the structure.

Acknowledgements

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Civil Engineers and Automated Vehicles: A Primer for the Civil Engineering Community

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Abstract

The subject of automated vehicles is omnipresent in current events and in the press. Automated vehicles are nascent in their development and adoption, and yet they are poised to revolutionize the way we conceptualize our transportation infrastructure. A critical goal of this article is to inform colleagues and practitioners why they should care about automated vehicles and their impact on our industry. For the practitioner who does not practice in this area, this article offers a primer on automated vehicles.

Keywords: automated, vehicles, transportation, mobility, infrastructure

1. What Are Automated Vehicles?

First, it is worth mentioning what does not qualify as an automated vehicle. The technologies that constitute automated vehicles do not include connected vehicles, vehicle-to-vehicle infrastructure (V2i), vehicle-to-other infrastructure (V2x), or the multitude of intelligent transportation system (ITS) applications currently in use. The term automated specifically applies to vehicles that act in an automated manner, or without any input from external systems. However, as a practical matter, most experts believe that automated vehicles will not be completely automated, especially in cities, where the benefits of communicating with other vehicles and infrastructure (and even people) can help realize the potential of Vision Zero.

So, what is an automated vehicle? In short, an automated vehicle is a vehicle that is capable of navigating without human input by using sensors and global positioning systems (GPS). The sensors typically include stereo vision cameras, LIDAR (Laser Imaging, Detection and Ranging), and an inertial measuring unit (IMU). The expectation is that a fully automated vehicle can successfully navigate any driving environment without human input.

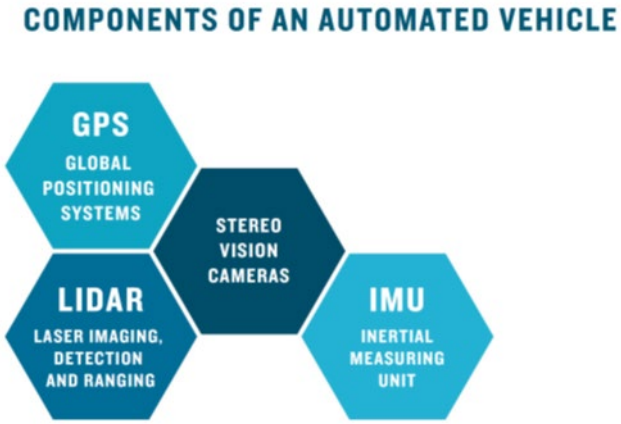
1.1 Levels of Autonomy

According to the Society of Automotive Engineers (SAE), a standard which has been broadly adopted across the industry, there are five levels of autonomy.

A Level 0 automated vehicle is a car with no automation. This is today's no-frills, base purchase automobile. Level 0 vehicles may have cruise control, as long as the cruise control is not dynamic. In other words, if a car has variable cruise control that adjusts to the speed of traffic in front of it, it is not a Level 0 vehicle, it is a Level 1 vehicle.

A Level 1 vehicle is a motor vehicle that has some aspect of dynamic automation. This typically includes steering and speed (acceleration/deceleration). A vehicle that uses a parking assist system or has dynamic cruise control would be a Level 1 vehicle.

A Level 2 vehicle is a motor vehicle that has both steering and acceleration/braking control. A Level 2 vehicle uses automation for both steering and speed control, with the potential for driver intervention. The original Tesla models used Level 2 automation.



A Level 3 vehicle operates under conditional automation. As long as a vehicle is operating within set parameters, it operates under automation. If certain conditions are met, the driver must resume control of the vehicle. For instance, weather conditions might render the sensors of the vehicle unable to perform as designed, and the driver might have to resume control. General Motors and Tesla both make vehicles that are capable of Level 3 automated operation.

A Level 4 vehicle is a fully self-driving vehicle operating through automation in areas where it has been designed to operate. For instance, most of these vehicles have been specifically designed based on the mapping of roadways and other infrastructure, which assist in the guidance of the vehicle. Think of this as the automated vehicle following Google Maps directions. However, if the vehicle is operating outside of map range, it will not be able to operate automatedly. It must be operated by a human in these limited circumstances. This is what defines a Level 4 vehicle versus a Level 5 vehicle.

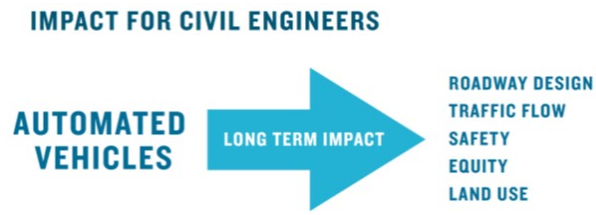
A Level 5 vehicle will operate in automated mode anywhere, under any conditions. This includes extreme weather conditions, off-road conditions, and in areas with no satellite or cell reception to help guide the vehicle. It is expected to operate completely independently of human inputs, other than establishing destinations and waypoints.

SAE FIVE LEVELS OF AUTONOMY

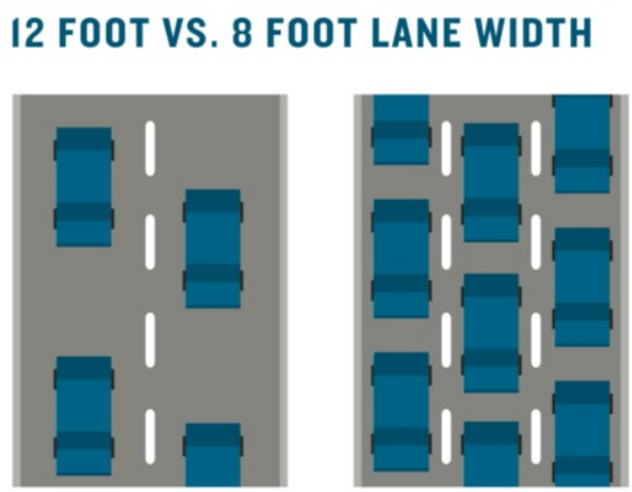
LEVEL	NAME
0	no driving automation
1	driver assistance
2	partial driving automation
3	conditional driving automation
4	high driving automation
5	full driving automation

2. Why Should We Care About Automated Vehicles?

Now that we know what automated vehicle are, why should civil engineers be interested in them? The answer to this question lies in the long-term impacts of automated vehicles on roadway design, traffic flow, safety, equity, land use, and the very existence of the automobile as we know it. Let's pull apart some of the key issues and potential impacts associated with automated vehicles.



Regarding roadway design, the design features related to traffic control, vertical and horizontal geometry, and lane width will all be dramatically changed. Once fully adopted, automated and connected vehicles will no longer need traffic signal indications or sign legends, because the vehicle will be communicating with roadside technology that will tell the vehicle when to slow down, when to stop, and when to obey other traffic requirements. Roadway geometries and lane widths can be dramatically changed because automated vehicles will not be capable of speeding and will not deviate in their lane. Smaller vertical and horizontal curves can be used, and much more narrow lanes – perhaps as narrow as 8 feet in most cities – can be adopted.



Traffic flow will also dramatically change. Automated and connected vehicles are expected to operate at significantly smaller headways – as little as ten feet at highway speed. As stated above, they could operate in lanes as narrow as 8 feet, allowing for an additional travel lane on a highway that currently has three 12-plus foot lanes. The effect of this is to substantially increase vehicle capacity. One research study suggests that the capacity of highways could increase by as much as 100% and freeway travel speeds could increase 20% (Sundquist, 2016). Some prognosticators (including myself) have suggested that this will effectively merge highway and transit modes into one transit system – the so-called "transportation theory of everything."

Perhaps the greatest promise of automated vehicles is safety. Automated and connected vehicles have the potential to put Vision Zero in reach. Vision Zero is the transportation planning and engineering goal of having zero deaths due to motor vehicle

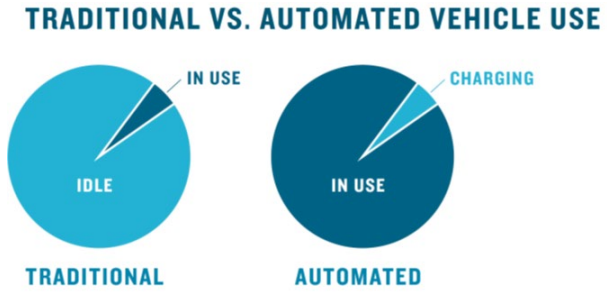
crashes. Vision Zero began in Europe and has become a goal in the United States. It is now a worldwide movement.

By eliminating human error (think distracted driving, impaired driving, fatigued driving, and aggressive driving), and equipping vehicles with sensors that will proactively prevent crashes, it is possible to eliminate virtually all crashes. There will still be occasional crashes due to malfunctions and glitches, but there would be far fewer accidents. In fact, the manufacturers of automated vehicles have already said that they are going to self-insure their vehicles, which portends an ominous outlook for the automobile insurance industry and the auto body repair industry.

There are many pros and cons of the safety aspect of automated vehicles from an economics point of view, but from a safety point of view, automated vehicles are good news. Imagine a world without the 33,000 highway deaths annually in the United States. In addition to the many lives saved, the reduction in lost productivity, and the virtual elimination of property damage, we could possibly eliminate almost all highway incidents from crashes that result in significant congestion on our highways. These would be tremendous benefits for society and the transportation community.

Another big winner in the automated vehicle revolution is disadvantaged populations. The physically disabled, the mentally disabled, the aged and infirm, and children will all benefit. These populations are currently restricted (on some level) from driving. The automated vehicle will level the playing field for these groups and create equity for these populations.

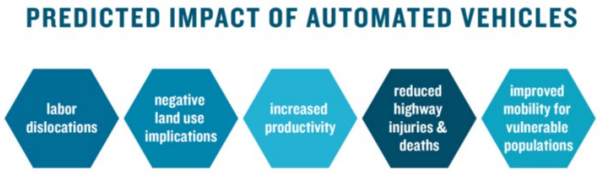
The automated vehicle is also likely to have a very substantial impact on land use. For instance, if your automated vehicle drops you off at your office in the morning, and circulates all day working for Uber, will parking garages in urban settings be needed or economical? Today, most cars in the United States are operated 5% of the time. In an automated vehicle world, vehicles will be in use 95% of the time, and the rest of the time they will be at a charging station. As a result, structured parking garages will become uneconomical and will be redeveloped (either through demolition or adaptive reuse) for higher and better forms of land use, such as housing or office.



In addition, if your automated vehicle can drive you to work three hours from your house, and you can work, rest, or socialize during that drive with no responsibilities for driving, will people be inclined to live further away from their work location? If this occurs, we could induce a whole new generation of sprawl, in the same manner that the interstate highway system induced sprawl and created a whole new world of suburbia. This would be a highly negative and unsustainable land use consequence of automated vehicles.

3. What Are the Long-Term Implications of Automated Vehicles?

Emerging research and debate have suggested that the combination of automated vehicles, the sharing economy, and electric vehicle technology will hasten the end of the internal combustion engine by 2030. This would be a very significant development, as it would have substantial benefits for air quality and climate change. The benefits of eliminating the internal combustion engine in favor of all electric vehicles (powered by renewal energy) would be game-changing.



This confluence of automated vehicles, shared vehicles, and electric vehicles would result in automobiles effectively becoming available on a prescription or a fractional basis. This model is called Mobility as a Service (MaaS) or Shared Electric Automated Mobility (SEAM). Instead of each person owning a car, they will subscribe to a car plan that offers them a set number of hours per week or month. The subscriptions will be offered directly by the car manufacturer, which will maintain the vehicle, insure the vehicle, fuel the vehicle, and store the vehicle (when not operating). This model of car "ownership" is predicted to save every American more than $5,000 per year. It also provides tremendous equity and opportunity for lower income populations in need of mobility options.

On the downside, the MaaS model will radically change the nature and profitability of automobile manufacturers, and will likely bankrupt fossil fuel companies due to stranded extraction and refinery assets. This is to say nothing of the automobile manufacturing and repair workforce, as well as the fossil fuel workforce. However, the model is unavoidable, as some car manufacturers are already offering subscriptions, and the three trends are already converging, with Uber entering the automated and electric vehicle markets.

3. Conclusion

The automated vehicle represents a very serious disruptive force for our entire society. Many positive and negative impacts are predicted, as is the case with any technological disruption. There will be many labor dislocations and there could be very

negative land use implications. There will also be significant benefits to society in terms of increased productivity, reduced highway injuries and deaths, and improved mobility for vulnerable populations.

It is not a matter of "if" automated vehicles happen (or MaaS for that matter). It is a matter of when. Some modelers (such as Tony Seba, see https://www.rethinkx.com/headlines) predict that it will all become mainstream by 2030. Others say it will take 50 years or more. In truth, no-one knows. However, considering it usually takes decades to plan and execute major infrastructure projects, we as a profession must incorporate planning for automated vehicles into our projects today, so that the infrastructure will be ready for a whole new tomorrow.

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