Development of Standardized Nebraska Family of UHPC Decked I-Beams

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Abstract

Nebraska has begun an initiative to develop a standardized bridge superstructure system utilizing ultra-high-performance concrete (UHPC). The initiative was motivated by recently made available structural design guidelines by FHWA and PCI, which followed over 25 years of intensive materials research and non-structural applications. PCI research has demonstrated that UHPC can be produced in available precasting facilities and can result in outstanding tensile properties due to use of steel fibers, and excellent durability due to the application of particle packing theory. Deicing chemicals and freeze-thaw cycles in Nebraska cause deck cracking which requires significant maintenance work. In addition, urban areas require rapid construction to minimize traffic disruption. The newly developed UHPC-DIB was designed to meet both the requirements of accelerated bridge construction (ABC) and deck durability. Due to the relatively high unit cost of the UHPC raw materials, effort was made to minimize the volume of concrete used without sacrificing stiffness. The result is an I beam shape that has a top flange width equal the beam spacing, less an 8 in. (200 mm) longitudinal UHPC closure pour. The web width is 4 in. The bottom flange is house anchorages for PT tendons, in segmental PT option, or pretensioning in full-length pretensioning option. The top flange has a 2.5 in. skin and a total depth of 8 in., with the 5.5 in. difference occupied by thin ribs spaced at 24 in. The family, of three web depths and variable top flange widths, can have simple span up to 200 feet.

Keywords: UHPC Decked I beam, Accelerated Bridge Construction (ABC), segmental post-tensioned.

1. Introduction

Ultra-high-performance concrete (UHPC) is a new class of concrete gaining significant attention in the past 25 years. Most recently, several attempts have been made to convert what had been considered an experimental material for nearly two decades, to main-stay bridge design and construction. This paper addresses an effort by the Nebraska Department of Transportation to introduce a bridge beam shape, designed specifically to take advantage of the outstanding strength and durability properties of UHPC. It is, to our knowledge, the first attempt in the US to introduce a standardized UHPC bridge I-beam in the United States.

The definition of UHPC has varied significantly. In this paper, the definition given by the report on PCI-UHPC (Tadros et al 2021) is adopted. It emphasizes the importance of tensile properties contributed by the steel fibers used in the mix and to a lesser extent the compressive strength. In all cases, UHPC has outstanding durability which makes it attractive for deck applications in snow belt states such as Nebraska.

The product described in this paper is called Standard Nebraska Department of Transportation Ultra High-Performance Concrete Decked I-Beam, simply referred to here as UHPC-DIB. It was born out of previous attempts by others to have an optimized UHPC I beam and to also introduce waffle slab UHPC precast deck panels, which was introduced in Iowa (Moore, 2012) with FHWA's support. Combining the two products into one introduces efficiencies that are unique to the UHPC material. In addition, use of a prestressed decked I-beam has been shown in Washington, Oregon and Alaska to be a very efficient conventional concrete system due to the large top flange which drives the neutral axis of the section up and improves service load stress conditions. More importantly, using a DIB product would qualify the superstructure as an accelerated bridge construction (ABC) system as it does not require deck field forming and cast-in-place concrete curing.

This paper outlines the evolution of the UHPC-DIB and the challenges faces on forming for it and how they were overcome. It proposes a list of standard shapes for possible use in spans up to 200 feet (61 m).

The details of the Belvidere North Bridge, the first bridge designed with this system, will be given. How the forms are adjusted to cover requirements for three other bridges in Nebraska and one bridge anticipated in Ontario, Canada, will be covered.

2. Definition of UHPC Implemented in Nebraska

The state of Nebraska has had interest and an active research program for nearly two decades since its introduction to the US by the Bouygues then parent company of the Omaha headquartered consulting firm HDR, Inc. NDOT has sponsored several UHPC research projects during this period. One of these projects (Mendonca, et al, 2020) resulted in an optimized mix using locally sourced materials and costing a fraction of prebagged commercial materials. The mix was adopted for use in the PCI project that resulted in the refenced PCI-UHPC Report (2021). The properties of the mix met the minimum requirements listed in the materials chapters of the PCI Report. Specifically, compressive strength at service = 17.4 ksi, first crack flexural strength = 1.5 ksi, peak flexural strength = 2.0 ksi, peak-to-first crack ratio > 1.25 to ensure strain hardening behavior, residual flexural strength at specimen deflection of L/150 > 75%, to ensure ductility. All measurements are according to ASTM Standard C1856/C1609. Although heat curing has been shown to have a positive impact on concrete properties, it has not been a requirement for this application, in order to encourage economical usage. As, such, the compressive strength at service may be allowed to be taken at 56 days, with 95% of that value acceptable at 28 days.

Inverse analysis was used to determine the tensile strength of UHPC based on ASTM Standard C1856/C1609 and the limits shown above, for use in flexural strength design. This analysis resulted in a bilinear stress strain diagram with peak stress = 0.75 ksi and ultimate strain beyond which fibers are assumed to lose bond with concrete is 0.005.

Creep and shrinkage properties are assumed in accordance with the PCI-UHPC design recommendations, with one exception. Total ultimate shrinkage is assumed to be limited to 300 micro-strain, controlled by shrinkage reducing admixtures.

3. Design Assumptions

Design allows for use of pretensioning as well as segmental post-tensioning. The design for flexure and shear closely followed the PCI-UHPC report recommendations. However, using the FHWA recommendations would still give essentially the same requirements for level of prestress and shear reinforcement, if any. Essentially, flexure at service is limited by allowable stresses not exceeding the cracking strength. Flexure at ultimate ignores the contribution of fibers and counts solely on the strength provided by the strands. Shear design assumes a total resistance of the concrete matrix and fibers of 1.00 ksi. The modified compression field theory of AASHTO LRFD Bridge Design Specifications is used to determine stirrups if needed, beyond the capacity of the concrete-fiber matrix. In most cases, for simple span design, shear reinforcement has not been found to be required when a web width of 4 inches is used. In some exceptions, when the spans are made continuous of live load, the negative moment zone near the piers requires a limited amount of reinforcing bars.

Other design criteria are similar to those used in conventional concrete design. They do not impact cross section optimization as much as flexure and shear.

It was desired by the NDOT to have a single cross section for both options of prestressing. Also, it was desirable by the precasters to allow them to use the post-tensioned option due to the limited batching capabilities currently in precast plants in the US. Accordingly, effort was made with success to house the post-tensioning anchors within the basic cross section without need for end blocks that are larger than the basic section shape. Only straight (non-draped) post-tensioning is considered, and the anchorage hardware is totally housed in the bottom flange, see Morcous and Tadros 2023.

Recently, the province of Ontario has expressed interest in adopting the Nebraska shape, which indeed had its origins through an earlier design by one of the authors for a bridge in Ontario. The contractor involved in the recent Ontario project, FACCA, Inc., was one of the original sponsors of the PCI Project and an early adopter of UHPC in North America.

4. Proposed Standard UHPC-DIB Dimensions

Figure 1 shows cross section and partial longitudinal section dimensions for the proposed cross section shape for a girder depth of 4ft-6in. Two other sizes are planned to be introduced besides the one shown in Figure 1. These are 72 in. and 90 in. deep girders, expected to cover spans up to 200 feet. The top flange width shown in Figure 1 can vary from 60 to 122 in. depending on the bridge geometry. The properties of the UHPC-DIB-54-122 girder used for design of the Belvidere North Bridge are area (between ribs)= 1,099 in^2 (0.71 m^2), moment of inertia = 454,400 in^4 (0.189 m^4), centroidal depth to bottom fibers = 32.00 in. (0.81 m), and volume (including ribs) = 0.31 yd^3/ft (0.78 m^3/m).

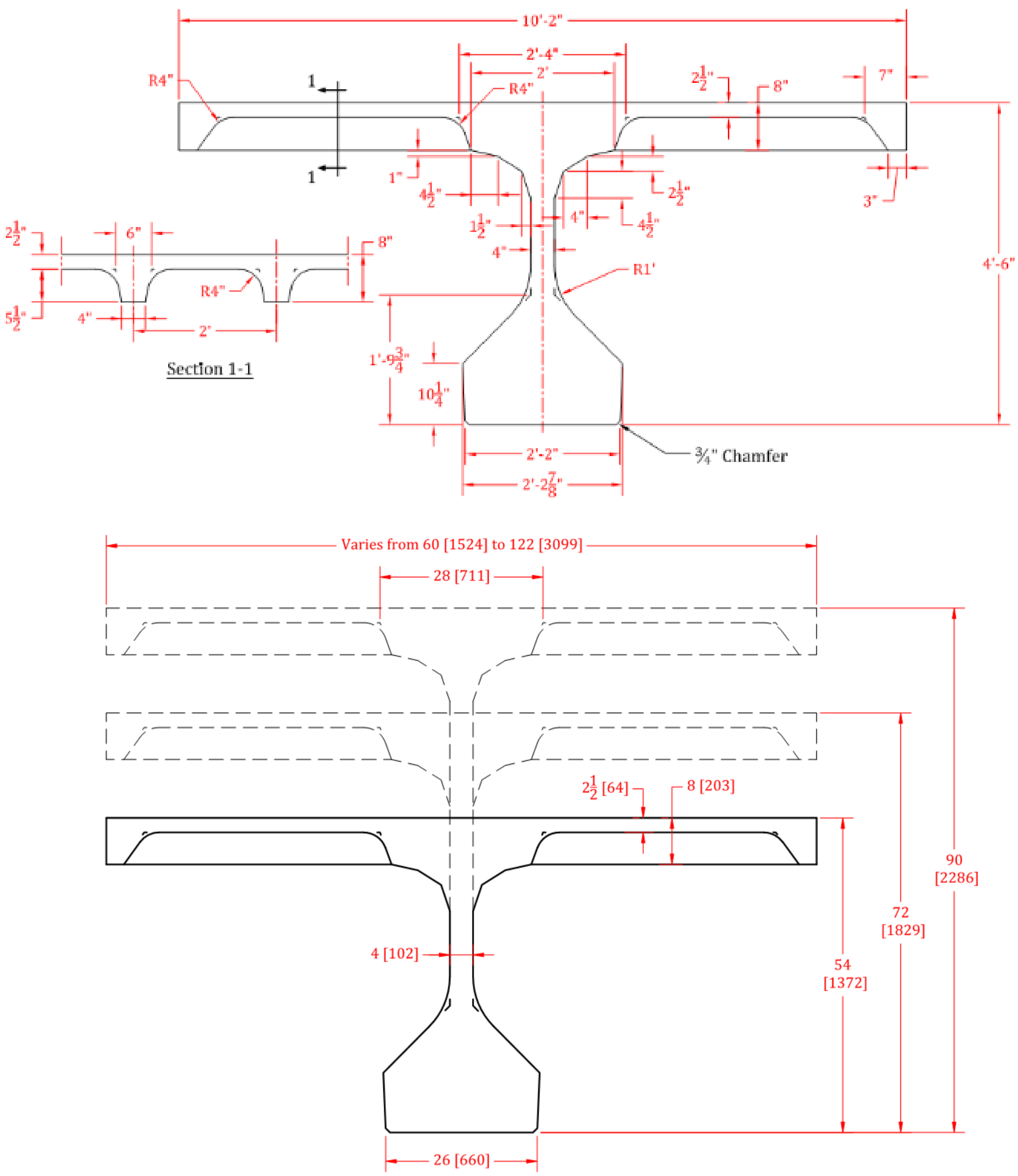

Figure 1- Standard shape of Nebraska's Standard UHPC-DIB-54-122. Also shown is the family of sizes proposed for spans up to 200 ft (61 m).

Figure 2 shows an isometric view of a post-tensioned segment for segmental post-tensioned construction which is expected to be implemented before full span length pretensioned applications. Note shear keys on ends with match casting, and shear keys on sides of top flange for cast-in-place closure pour.

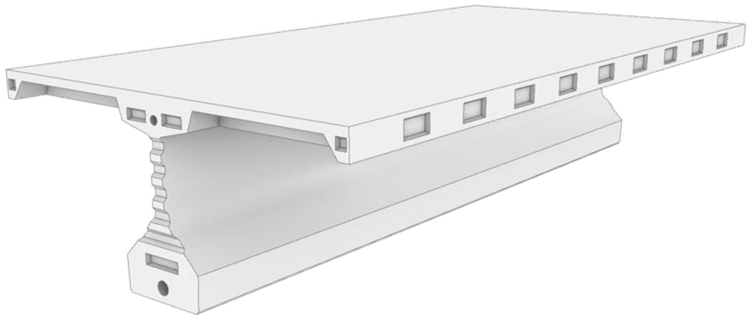

Figure 2- Isometric View of One 20 ft Long Precast Post-Tensioned Segment.

The same basic section as that shown in Figures1and 2 is planned to be used to create the girders for the Nagagamisis Narrows Bridge in Ontario, Canada. The only variation is that the bottom flange is reduced by 4 inches to achieve a depth of 1270 mm (50 in.) as required for that bridge. Because pretensioning is planned, there is adequate space for the strands needed to achieve a span of about 100 ft.

There are several attractive features and several challenges of this highly optimized girder shape. One important feature is minimization of the amount of concrete needed, thus reducing unit cost, and increasing superimposed load capacity. Having a relatively large top flange in lieu of a separate deck not only accelerates construction as only few pieces need to be handled and not deck forming (except for the narrow closure pour) is needed. More importantly, for bridges in cold climates, the deck is much more durable than conventional CIP deck. The shape of the cross section provides just what is needed to have an efficient structural system. The 2.5 in. top flange skin has been shown through testing at North Carolina State University and the University of Nebraska-Lincoln to have a large factor of safety against punching shear. Reinforcement is provided only in the transverse ribs and longitudinal closure pour, greatly reducing the amount of reinforcement needed. The proportions of the cross section create a significant eccentricity of the prestressing force and thus reduce the prestressing needed, compared to standard I-beam.

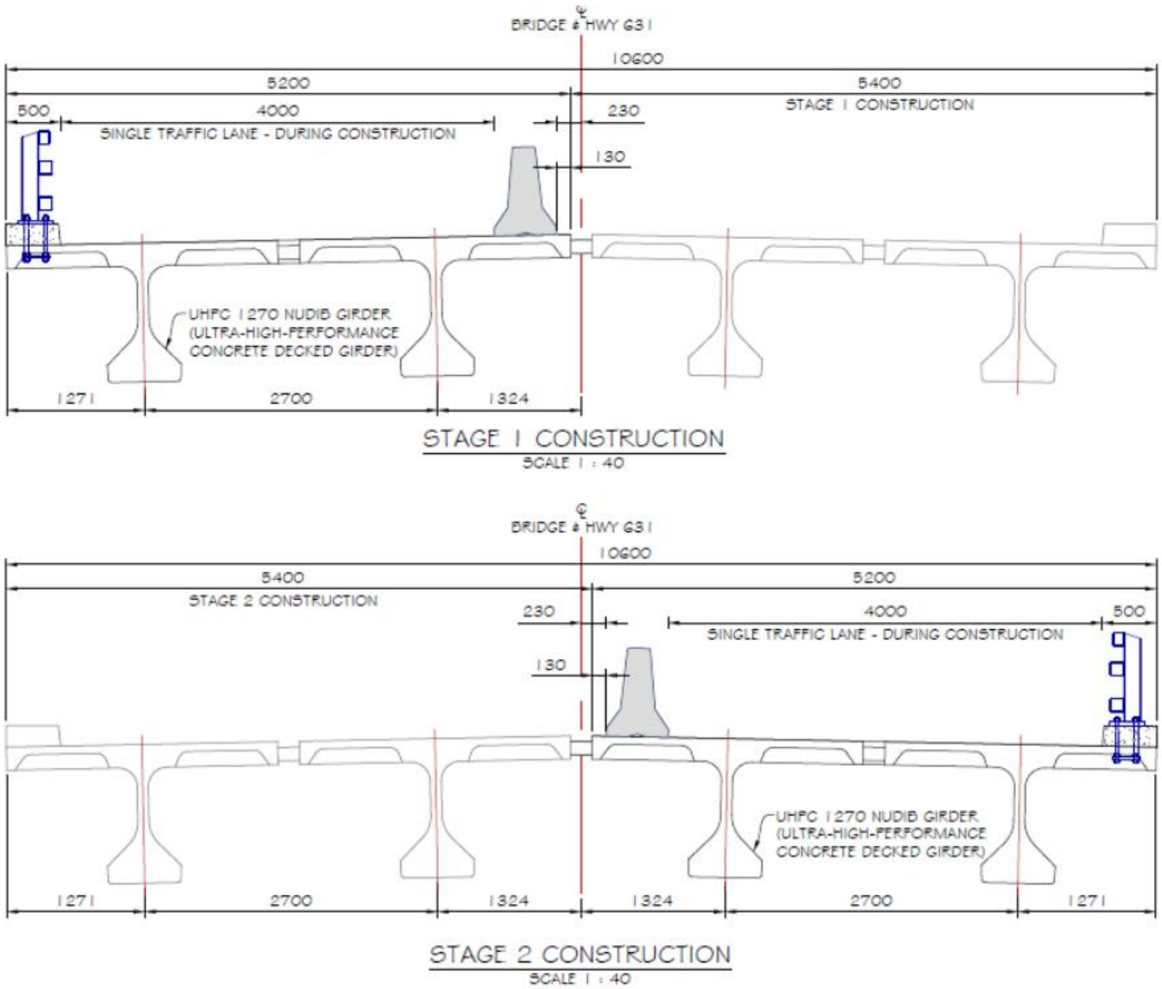

Figure 3-Nagagamisis Narrows Bridge Replacement, Ontario, Canada, Courtesy FACCA Inc.

There are several challenges encountered in this girder type. Skew bridges and bridges that are made continuous over the piers would need to have their ends with a thicker skin to facilitate the space required for extra reinforcement in the end zone. The Belvidere North Bridge, however, was designed to be continuous for LL, and did not require more space for the negative moment continuity steel than already provided in the top bulb of the cross section. Another challenge is keeping the camber within acceptable design limits if no composite topping is desired. For this reason, it is proposed to supply one top unbonded strand tendon, and to allow for field adjustments to achieve the desired camber.

5. Formwork Considerations

A third and most important challenge is stripping the block-out forms between the top flange ribs. It is well known that UHPC exhibits significant early autogenous shrinkage. This effect and the fact that steel pans and concrete can adhere to each other, has promoted the designers to introduce shrinkage compensating admixture, and to engage several formwork companies. As a result, the following forming concept was developed, see Figures. 4 and 5. In Figure 4, the top flange forms are swiveled down after the top concrete gains adequate strength. The top flange forms are expected to swivel with the rib forms and possibly the polyurethane liner (cap) fitted over the steel pans. Before the forms swivel down, the side railing is spaced out to clear the projecting bars.

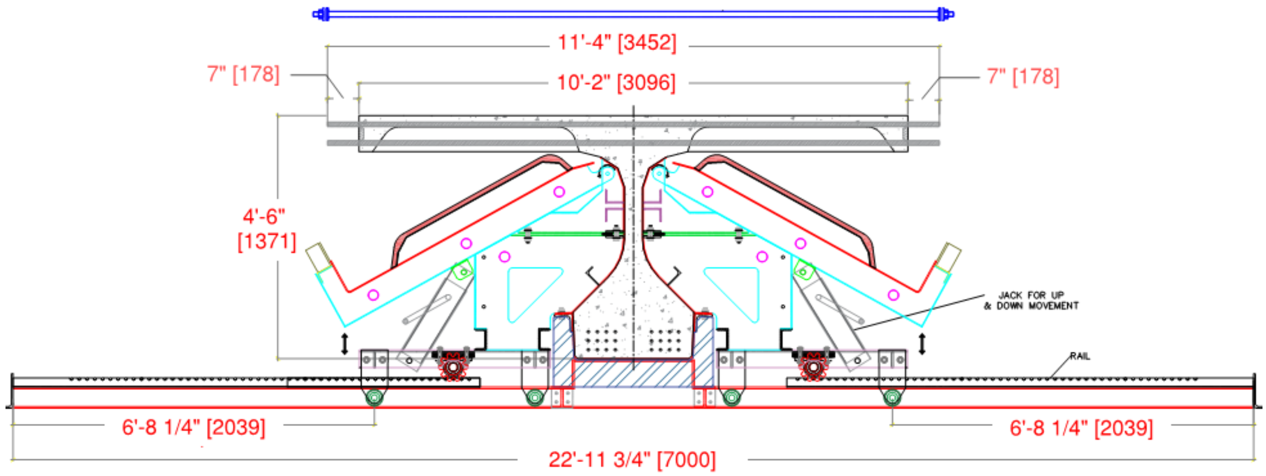

Figure 4- Proposed Forming System

Figure 5 shows the position of the forms when the product is ready to be removed. Note that the plastic caps are shown separated from the steel pans. To allow for multiple use of the pans and the caps, they are required to be well lubricated, and the caps be made of strong polyurethane material with shore hardness of 40 or higher, which is the same material used by precasters to create texture on exterior wall panels. Please note that 300 micro-strain shrinkage is equal to 0.072 in. total shortening, which may be too much for steel but can certainly be accommodated with polyurethane liners.

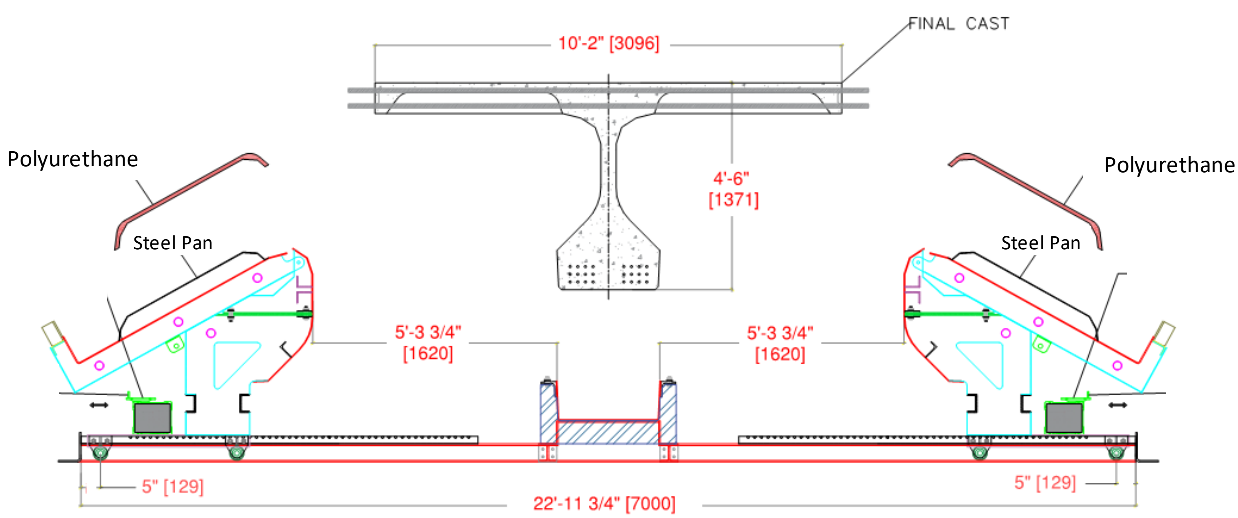

Figure 5- Position of forms to allow for product stripping

The form liner concept has been one of several improvements introduced to the girder geometry and forming design to further optimize the system. One of these improvements is to introduce generous fillets between the top flange ribs and skin. Another one, not shown in the figures, is to allow the steel pans to be made of three segments, each, to allow them to be used for various top flange widths, by removing an replacing the middle segment with another of different length. A third modification was to minimize the bottom width of the edge rib while still allowing for a generous top width to simplify field forming and accommodate top reinforcement details for railing connection where needed. This allows the exterior beam to have the same dimensions as an interior beam.

6. Conclusions

Although decked I-beams have been in use for the last several decades in the northwest region of the US and particularly the state of Alaska, they are gradually finding acceptance is other regions due to their efficiency and rapid construction. The proposed standard shape, created for precast ultra-high performance concrete bridge beams, follows that trend but with UHPC instead of conventional concrete. The proposed system appears to be complicated due to the fact that the top flange is not a solid, but rather a ribbed, slab. However, the complexity is mostly in the forming system which can be amortized over tens of years of usage. The cross section has several advantages including accelerated construction and avoidance of deck forming, and placement of lower quality concrete decks which have been demonstrated to be a source of continuous maintenance and traffic interruption. The optimized system should not be compared to conventional I-beams, but rather with a conventional beam/deck system. Four bridges in Nebraska and one bridge in Ontario, Canada are in various stages of design.

7. References

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