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COMARISON BETWEEN ACI 440 AND FIB 14 DESIGN GUIDELINES IN USING CFRP FOR STRENGTHENING OF A CONCRETE BRIDGE HEADSTOCK

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ABSTRACT: This paper compares and reviews the recommendations and contents of the guide for the design and construction of externally bonded FRP systems for strengthening concrete structures reported by ACI committee 440 and technical report of Externally bonded FRP reinforcement for RC structures (FIB 14) in application of carbon fiber reinforced polymer (CFRP) composites in strengthening of an aging reinforced concrete headstock. The paper also discusses the background, limitations, strengthening for flexure and shear, and other related issues in use of FRP for strengthening of a typical reinforced concrete headstock structure such as durability, de-bonding, strengthening limits, fire and environmental conditions. A case study of strengthening of a bridge headstock using FRP composites is presented as a worked example in order to illustrate and compare the differences between these two design guidelines when used in conjunction with the philosophy of the Austroads (1992) bridge design code.

Keywords: Reinforced concrete bridges, FRP composites, design guidelines, structural analysis, capacity analysis, flexural strengthening, shear strengthening, de-bonding, durability and fire.

1. INTRODUCTION

Rehabilitation and upgrading of the existing civil engineering infrastructure has been of great importance during the last decade. There are number of situations where the structural capacity of a structure in service needs to be increased. These include change of use, new loading criteria, impact and damage and deterioration of material. Hence the aspect of civil engineering infrastructure renewal has received considerable attention over the past few years. Bridge structures are deteriorating at a fast rate, and cost for repair and replacement of deficient bridges are continuously rising. Even when resources are available, extended time is often required for performing needed remedies, causing distribution of traffic and inconvenience to the traveling public. Most of these bridges have older design features that prevent them from accommodating current traffic volumes with modern vehicle sizes and weights. The strengthening or retrofitting of existing concrete structures to resist higher design loads, correct deterioration-related damage, or increased ductility has traditionally been accomplished using conventional materials and construction techniques. Externally bonded steel plates, steel or concrete jackets and external post tensioning are just some of the many traditional techniques available. In the context of the strengthening problem, advanced composites have the potential to prove another promising solution.[1]

FRP composite materials gain their superior characteristics from the component materials used. Their strength comes largely from the fibers, which are usually glass, carbon, or aramid fiber. FRP materials are lightweight, non-corrosive, non-magnetic and exhibit high tensile strength. Additionally, these materials are readily available in several forms ranging from factory made laminates to dry fiber sheets that can be wrapped to conform to the geometry of a structure before adding the polymer resin. The relatively thin profile of cured FRP systems is often desirable in applications where aesthetics or access is a concern. [2]

FRP systems can be used to rehabilitate or restore the strength of a deteriorated structural member, or retrofit or strengthen a sound structural member to resist increased loads due to changes in use of the structure, or address design or construction errors. Due to the characteristics of FRP materials, behavior of FRP strengthened members, and various issues regarding the use of externally bonded reinforcement, specific guidance on the use of these systems is needed. [3]

2. DESIGN GUIDELINES

Since the use of FRP composites for strengthening of reinforced concrete structures is a relatively new technique, the development of design guidelines for externally bonded FRP systems is ongoing in Europe, Japan, Canada and the United States. Within the last ten years, many design guidelines have been published to provide guidance for the selection, design and installation of FRP systems for external strengthening of concrete structures. In Europe, Task Group 9.3 of the international Federation for Structural Concrete published bulletin 14 (FIB 14) on design guidelines for externally bonded FRP reinforcement for reinforced concrete structures [4]. And in the United States, ACI Committee 440 developed a guide for the design and construction of externally bonded systems for strengthening concrete structures [3]. This paper aims to compare and review the recommendations and contents of these guides in the context of the design of n externally bonded FRP system for strengthening of a reinforced concrete bridge headstock.

3. CASE STUDY

The comparison between the two design guidelines is based on a case study of strengthening of a bridge headstock using FRP composites. The bridge discussed in this paper carries a state route of Ipswich-Toowoomba road over Tenthill Creek in Gatton, Queensland, Australia. This three span reinforced concrete, pre-stressed beam structure was built in 1970's. The bridge is 82.15 m long and about 8.6 m wide and is supported by a total of 12 pre-stressed 27.38 m long beams over three spans of 27.38 m. The beams are simply supported on two abutments and two headstocks. A headstock elevation view is shown in Figure 1.

During routine inspection, shear and flexural cracks were observed in the bridge headstock (Figure 1). Concerns about headstock capacity and overall safety of the bridge were heightened by the absence of any documentation containing complete information needed for reliable structural evaluation. Queensland Department of Main Roads (QDMR) elected to rehabilitate the headstock as opposed to replacement or load posing. The objectives of the study discussed in this paper were to compare the two common design guidelines of ACI 440 and FIB 14 and to evaluate effectiveness of the strengthening FRP system for the headstock.

3.1 Structural Analysis of the Headstock

The headstock was analysed as a portal frame considering all necessary design situations and load combinations according to Austroads Bridge standard for ultimate limit state and serviceability limit state. [5]

3.2 Capacity analysis

A typical beam section of the headstock is shown in Figure 2. The positive and negative flexural and shear capacities of the section were calculated in accordance with Australian standard AS 3600 [6]. The nominal areas of steel reinforcing bars, nominal steel yield strength of 400 MPa for longitudinal reinforcement and 240 MPa for shear reinforcement and nominal concrete compressive strength of 20 MPa were used in the section capacity analysis. The degradation due to corrosion of the steel and creep and shrinkage of the concrete were assumed to be negligible although residual strains were considered in developing the strengthening solution.

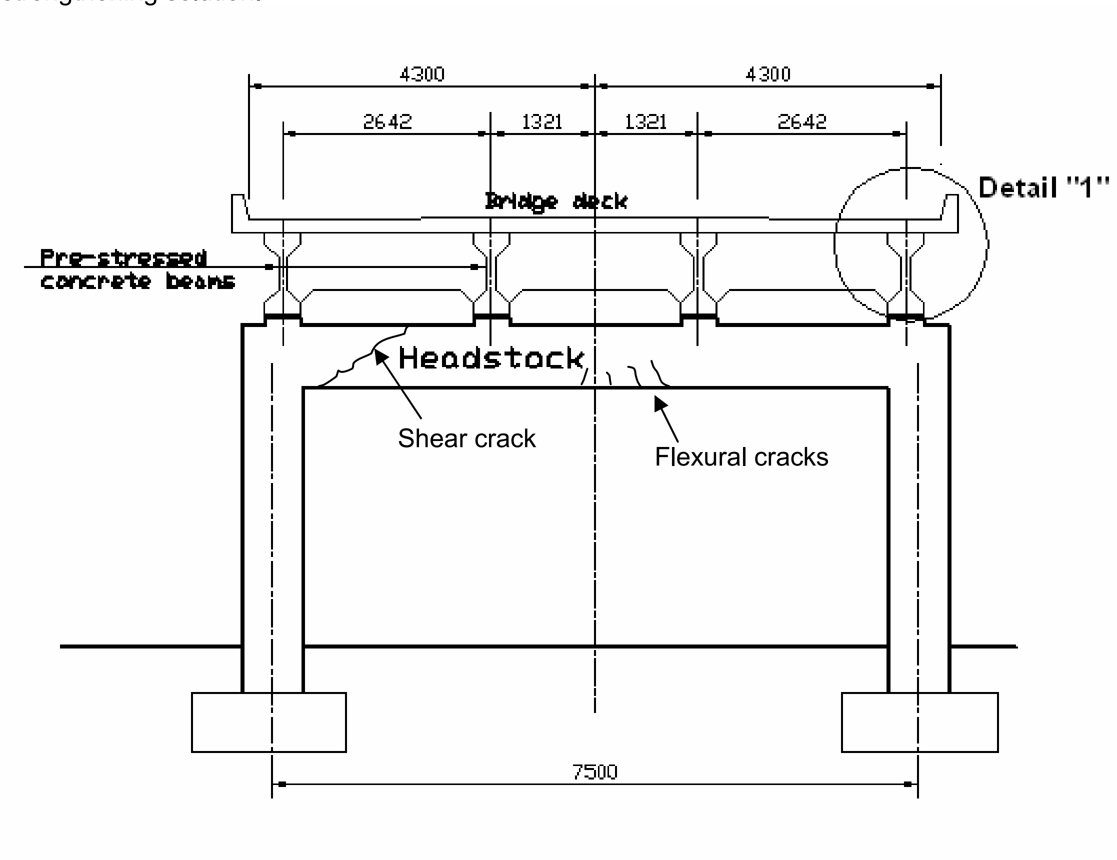

Fig. 1 – A schematic elevation view of the headstock

Although calculated positive, negative bending moments and shear force in serviceability limit state are relatively lower than structural capacity of the headstock, a decision was made to strengthen the headstock for ultimate bending moments and shear and to contain the cracking.

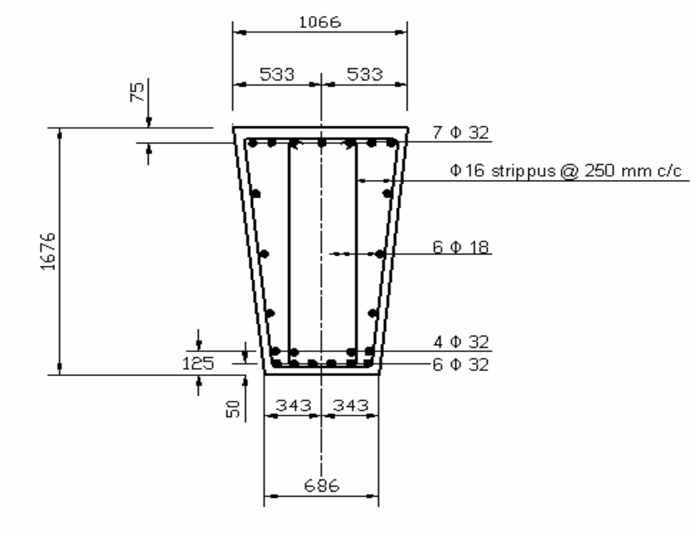

Fig. 2 – Typical beam section of the headstock

4. DESIGN OF FRP STRENGTHENING SYSTEM

It was decided to bond FRP laminates to the tension face of the beam section (bottom fibre) of the headstock with fibres oriented along the length of the member for positive flexural strengthening and use a complete wrapping scheme with fibres oriented along the transverse axis of the beam section for the shear strengthening. After consulting with suppliers of FRP materials in Australia, iIt was decided to use Sika CFRP laminate CarboDur type S for flexural strengthening and Sika CFRP wet lay up type Sika-Wrap-230C [7]. Table 1 shows material properties of proposed systems.

Table 1 – Material properties of FRP systems

TYPE	Tensile Strength (MPa)	Tensile Elastic Modulus (MPa)	Elongation at Break
CarboDur Type S	2800	165000	1.7%
Sika-Wrap-230C	3500	230000	1.5%

5. FLEXURAL STRENGTHENING

In the analysis for the ultimate state in flexure, both codes follow well established procedures using idealised stress-strain curves for concrete, FRP and longitudinal reinforcement.

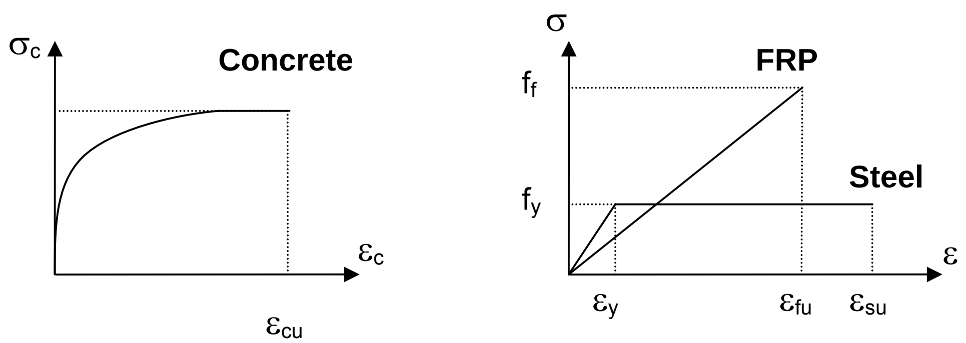

Fig. 3 – Idealised stress-strain curves for constitutive materials at ULS

These curves, along with the following assumptions, form the basis for the ultimate strength ultimate state analysis of concrete element strengthened in flexure. [3, 4]

- Design calculations are based on the actual dimensions, internal reinforcing steel arrangement, and material properties of the existing member being strengthened.
- The strain in reinforcement and concrete are directly proportional to the distance from the neutral axis, that is, a plane section before loading remains plane after loading.
- There is no relative slip between external FRP reinforcement and the concrete
- The shear deformation within the adhesive layer can be neglected since the adhesive layer is very thin with slight variations in its thickness.

The cross section analysis identifies all possible failure modes. Failure of the strengthened element may then occur as a result of various mechanisms as follows:

- Crushing of the concrete in compression before yielding of the reinforcing steel

- Yielding of the steel in tension followed by rupture of the FRP laminates
- Yielding of the steel in tension followed by concrete crushing
- Shear/tension de-lamination of the concrete cover
- De-bonding of the FRP from the concrete substrate

5.1 Design material properties

According to the FIB guideline, the design strength is obtained by dividing the characteristic strength by a partial safety factor. The partial safety factors for concrete (in flexure), γ_{mc}, and steel reinforcement, γ_{ms}, are normally taken as 1.5 and 1.15 respectively. The partial safety factors applied on characteristic strength of FRPs are mainly based on the observed differences in the long-term behaviour of FRP (basically depending on the type of fibres) as well as the application method and on-site working conditions. A partial safety factor for carbon fibre in application type B under difficult on-site working condition, γ_{ms}, of 1.35 is indicated. The design material properties for the headstock according to the FIB guideline are listed in Table 2. [4]

Table 2 – Design material properties complying with the FIB guideline

Material	Design Strength (MPa)	Modulus of Elasticity (MPa)	Allowable strain
Concrete	21/1.5=14	22610*	0.0035
Steel reinforcement	400/1.15=348	200000	0.002
CFRP strips (flexural)	2800/1.35=2047	165000	0.017/1.35=0.0126
CFRP wrapping (shear)	3500/1.35=2593	230000	0.015/1.35=0.0111

* The long term modulus of elasticity of 11305 was used to account for creep of concrete

ACI design guideline suggests that the design ultimate tensile strength should be determined using the environmental reduction factor only for FRP materials. The reduction factors are mainly based on type of fibre and environmental conditions. Similarly it is suggested to reduce the design rupture strain for environmental-exposure conditions. A reduction factor for carbon fibre in aggressive environment, C_E, of 0.85 is indicated. The design material properties for the headstock according to the ACI guideline are listed in Table 3.[3]

Table 3 – Design material properties according to the ACI guideline

Material	Design Strength (MPa)	Modulus of Elasticity (MPa)	Allowable strain
Concrete	21 (β_1=0.91)	22610*	0.003
Steel reinforcement	400	200000	0.002
CFRP strips (flexural)	0.85x2800=2380	165000	0.85x0.017=0.01445
CFRP wrapping (shear)	0.85x3500=2975	230000	0.85x0.015=0.01275

* The long term modulus of elasticity of 11305 was used to account for creep of concrete

5.2 Initial situation

It was noted by both design guidelines that the effect of the initial load prior to strengthening should be considered in the calculation of the theory of elasticity and with the service moment acting on the critical beam section during strengthening. The initial strain distribution of the member may then be evaluated and considered in strengthening calculations. As the service bending moment is typically greater than the cracking moment, the calculation is based on a cracked section. The initial strain distribution of the headstock was calculated based on structural analysis for service loading condition, long-term modulus of elasticity and the cracked section. The same initial strain distribution was used for the design of strengthening scheme using both design guidelines. [3, 4]

5.3 Capacity of the strengthened beam

The cross section analysis indicated that the failure mode of the beam section of the headstock would be the yielding of the longitudinal steel reinforcement followed by concrete crushing, while the FRP is intact. This is the most desirable failure mode, which satisfy the safety requirements in ultimate state for a reinforced concrete section. The design bending moment for the strengthened member was then calculated in accordance with each design guidelines based on well established principles of flexural design of a reinforced concrete beams. The design principles are shown in Figure 4.

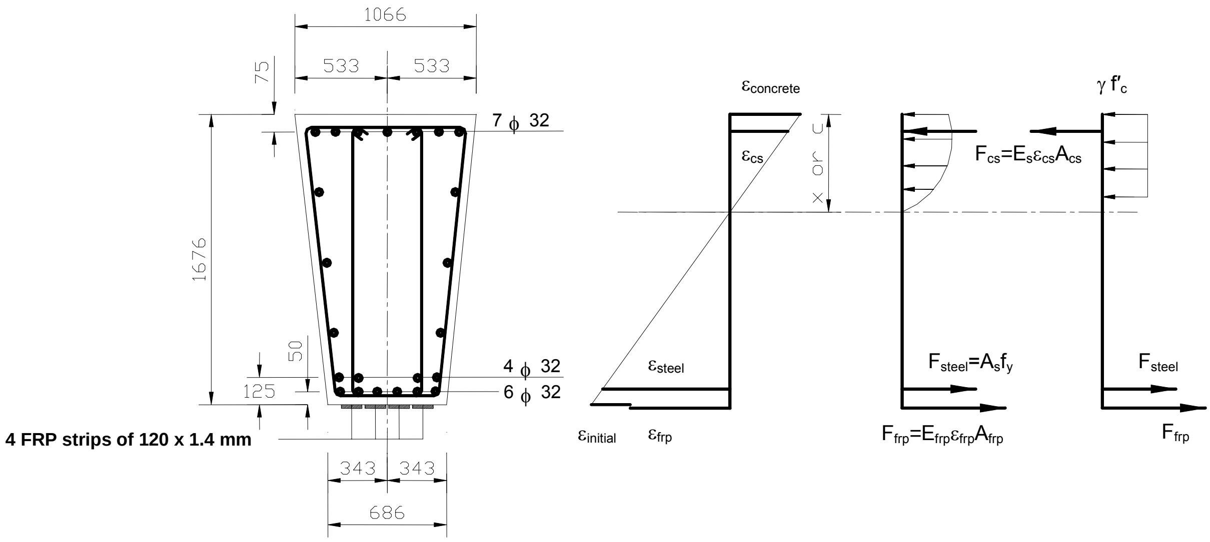

Fig. 4 – Internal strain and stress distributions for the beam cross section of the headstock

The design bending moment capacity was then calculated based on each of the design guideline. The design bending moment capacity of 6720 kN-m and 5854kN-m were calculated for strengthened section based on the FIB and ACI design guidelines respectively. Although both the design guidelines use the same principles to calculate the capacity of the strengthened member, each design guideline introduces different values for ultimate strain of the concrete and the strength reduction and material safety factors. The calculated moment capacities using the two design guidelines indicated that the predicted capacity enhancement based on the ACI guideline is more conservative. This is mainly due to the use of the strength reduction factors (ϕ) required by ACI 318-99 [8] with an additional strength reduction factor of 0.85 applied to the contribution of FRP reinforcement to flexural capacity enhancement.

5.4 Anchorage

Experimental investigations show that the FRP rupture is a rare event and de-lamination of FRP strips is more likely to occur before stress in the FRP reach the ultimate level. De-bonding implies the complete loss of composite action between the concrete and FRP laminates. Bond failure will be a brittle failure and should be prevented. The ACI guideline place a limitation on the strain level in the laminate to prevent de-lamination of FRP from the concrete substrate. [3]

The FIB guideline noted that the following failure modes need to be considered to prevent de-lamination of FRP, depending on the starting point of the de-bonding process.

- De-bonding in an un-cracked anchorage zone
- De-bonding caused at flexural cracks
- De-bonding caused at shear cracks

De-bonding of CFRP strips was checked based on each guideline and the calculations indicated that the strengthening system satisfies the requirements from both guidelines to prevent the de-bonding failure. It appears that the FIB guideline uses a more accurate methodology to check the de-bonding failure, considering all possible failure modes. However the de-bonding failure of CFRP strips in the strengthening of the headstock will be also controlled by applying CFRP wrapping scheme for shear strengthening.

6. SHEAR STRENGTHENING

The design for shear strengthening of a reinforced concrete member in both the guidelines is based on truss model and superposition principle with some considerations for the orthotropic behaviour of the CFRP material. The shear strength of a strengthened member is determined by adding the contribution of the CFRP reinforcing to the contributions from the concrete and shear reinforcement (Equation1).

$$V_{total} = V_{concrete} + V_{Steel} + V_{frp} \qquad (1)$$

where $V_{concrete}, V_{Steel}$ and V_{frp} are the contributions from the concrete, steel and the FRP respectively. Use of CFRP wrapping system increases the design shear capacity of the strengthened member by 310 kN and 323 kN based on the FIB and ACI design guidelines respectively. The results indicated that both the guidelines predict almost the same shear capacity increases using the complete wrapping scheme for strengthening of the headstock. The CFRP shear reinforcement is considered as contact critical shear reinforcement. Hence the ultimate failure does not occur with de-bonding.

7. OTHER DESIGN CONSIDERATIONS AND ENVIRONMENTAL EFFECTS

The ACI guideline suggested imposing reasonable strengthening limits to guard the strengthened member against failure of the FRP strengthening system and collapse of the structure due to fire, vandalism, or other causes. It is recommended that the existing strength of the structure be sufficient to a level of load as described by Equation 2. [3]

$$(\phi R_n)_{existing} \geq (1.2S_{DL} + 0.85S_{LL})_{new} \qquad (2)$$

where S_{DL} is dead load and S_{LL} is live load. It is also recommended that the strength of a structural member with a fire-resistance rating before strengthening should satisfy the conditions of Equation 3. [3]

$$(R_{n\theta})_{existing} \geq S_{DL} + S_{LL} \qquad (3)$$

$(R_{n\theta})_{existing}$ is the nominal resistance of the member at an elevated temperature, which can be determined using the ACI 216R guideline. [9]

Environmental conditions affect the performances of the FRP system. The mechanical properties of FRP systems degrade under exposure to certain environments, such as alkalinity, salt water, chemicals, ultraviolet light, high temperatures, high humidity and freezing and thawing cycles. The ACI guideline account for this degradation using the environmental reduction factor for the design material properties of CFRP as described in section 5.1.

The FIB guideline recommends the accident design verification procedure to prevent failure of the FRP strengthening system and collapse of the structure due to fire, vandalism, or other causes. The existing member is subjected to all relevant accidental load combinations of the strengthened member. The verification is the performance in the ultimate limit state, considering the partial safety factors of 1.0 and considering partial safety coefficients and combination factors using Eurocode 1 (EC1), Part 1 (CEN 1994) [10]. The FIB guideline also recommends that sufficient attention should be paid to the special design aspects, as they can have a considerable influence on the structural safety.

The existing structural strength of the headstock was checked to be sufficient to satisfy the ACI and FIB guidelines requirements in the accidental design situation. The existing structure has not been rated for fire-resistance; hence it was not checked with the requirement of equation 3.

8. CONCLUSIONS

This paper presented a comparison between the recommendations of two design guidelines: ACI 440 and the FIB 14 in design of externally bonded FRP systems to strengthen reinforced concrete beams in flexure and shear. The FRP type used was carbon fiber reinforced polymer (CFRP) in laminate and strip form readily available in Australia. The comparison was presented using a case study of a bridge headstock: Tenthill Creek, Queensland Australia. The following conclusions can be drawn from the design calculations and the comparison.

- Both design guidelines adopt the same principle of design to estimate shear and flexural capacity enhancements of the strengthened member when applied in accordance with the guidelines of the Austroads (1992) Bridge Design code.
- The ACI guideline is more conservative in prediction of flexural capacity enhancement for the strengthened headstock. This is mainly due to the use of an additional strength reduction factor of 0.85 applied to the contribution of FRP reinforcement.
- The FIB guideline uses a more accurate approach to check de-bonding of FRP laminates from the concrete substrate, which covers all possible bond failure modes. Alignment of the design method with Austroads (1992) recommendations will require further work.
- Both design guidelines predicted almost the same shear capacity enhancement for the strengthened member again when used in accordance with the Austroads (1992) code.

In view of above finding, it may be concluded that the use of ACI 440 design guideline may be more appropriate for FRP strengthening applications in Australia. The design concepts and philosophy used by ACI is similar to those adopted by AS3600 (2002). However, in considering the failure of FRP composites in de-bonding and anchorage zones, use of FIB appears to be more appropriate since it systematically covers all possible scenarios. A methodology needs to be developed to align the design procedure with the Austroads (1992) provisions.

9. ACKNOWLEDGEMENT

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