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Implication of bridge resilience design and lessons from negative examples

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Abstract

Bridge resilience is a newly proposed bridge design criterion that involves robustness, redundancy and reparability targeting on the rapidity of functionality restoration after suffering extreme actions and long-term durability deterioration. It stipulates a lower probability of reaching the ultimate limit state or strength limit state, which have been only partly involved in bridge design codes around the world. In AASHTO LRFD Bridge Design Specifications and Eurocodes, there are some design principles related to bridge resilience. Yet, it is also necessary to give more requirements for structural ductility and collapse resistance when the actual load exceeds the load combination in the code. This paper focuses on the resilience-based principles for bridge design, and exposes some problematic bridge structural systems and details, such as bridges most likely to overturning, steel bridges with fracture critical members, arch bridges with suspended desk, Morandi cable-stayed bridges, poor details for seismic vulnerability, etc. Whereas overturning is one of the worst anti-resilience scenarios, the resilience design against bridge overturning is highlighted through a detailed discussion including the calculation methods of anti-overturning factor, overturning stability of curved bridges, reasonable disposition of supports, and anti-overturning countermeasures.

Keywords: Bridge resilience, Design principle, Negative example, Bridge overturning

1 Introduction

From the viewpoint of life cycle engineering, a bridge may suffer accidental damage and environmental deterioration during its service life. Extreme events, such as earthquakes, floods, hurricanes, tsunamis and heavily overload, may cause instantaneous damages and result in the collapse or irreparable damage to bridges. In addition, accumulated environmental effects, such as carbonization, chemical erosion, freezing-thawing effect, have potentials to expedite the performance degradation of bridges with the passage of time. If bridges are difficult to be maintained or rehabilitated, safety issues may come on early in its designated service life (Banerjee et al 2019).

In view of numerous bridge failure accidents in recent years worldwide, the concept of resilience looms larger. In order to achieve this goal, it is necessary to take into account all extreme accidents and structural degradation effects in a proper manner during its service life.

The earliest definitions of resilience related to civil engineering can be traced back to several decades ago (Holling 1973; Timmerman 1981). Evolved over time, more comprehensive definition of resilience that accounts for more influential aspects was given by Bruneau et al (2003). Yadlosky and Chavel (2011) adopted a fault tree methodology in the framework to identify potential events that could lead to a bridge failure, where lessons from past bridge failures. Bocchini and Frangopol (2012) systematically introduced the concept of resilience into the infrastructure field. Minaie and Moon (2017) proposed a practical and simplified multistage framework to analyze and assess bridge resilience. Banerjee et al (2019) systematically expounded the research priorities and application prospects of bridge resilience under single-hazard and multi-hazard conditions under the framework of the full-life cycle.

The resilience of bridge structures seems a recently coined concept. Even now, the term "resilience" has not been introduced into the current bridge design codes such as AASHTO, Eurocodes, and Chinese specification. Nonetheless, some requirements related to bridge resilience have been codified into design principles. In AASHTO (2020), it is expressed by a coefficient related to ductility, redundancy, and operational classification. In Eurocode 0 (2005), there are some clauses, such as a structure shall be designed to have adequate: structural resistance, serviceability, and durability; potential damage shall be avoided or limited by appropriate choice of low sensitivity structures. Although there are some deemed-to-satisfy provisions in these codes, it still needs to be further clarified and strengthened. With the aging of bridges all over the world, the problem about the redundancy and reparability of bridges become more and more prominent. This paper focuses on the resilience-based principles for bridge design, and exposes some bridge structural systems and details of lower resilience. Then it can give some enlightenment to engineers.

2 Overall concept of bridge resilience design

2.1 Resilience triangle illustration

Bridge resilience can be regarded as a new comprehensive criterion to ensure the safety of bridges for long-term operation and accidental situations, including robustness, redundancy, and reparability. Robustness refers to the safety of structures in case of accidental action or when the load exceeds the design value, which reflects that bridges have enough strength, ductility and overturning stability. Redundancy refers to the ability for withstanding continuous collapse when bridges are subjected to local failure, which can be understood as the degree of static indeterminacy of bridges and represented as the ability of load redistribution. Reparability refers to the ability of a structure for recovering functions quickly after functional deterioration or disasters, including inspection, reinforcement and maintenance. Among them, the robustness and redundancy are the basis. Only when the two are satisfied can the reparability show its superiority. The characteristics of bridge resilience can be expressed by means of the triangle in a rectangular coordinate system with passage of time as abscissa and functionality as ordinate, as shown in Fig. 1 (Bocchini and Frangopol 2012).

The resilience triangle of bridges reveals the internal relationships among vulnerability, robustness, redundancy, and reparability in Fig. 1. Within the triangle ΔABC, the two sides $\overline{AB}$ and $\overline{AC}$ are vulnerability (Q_0-Q_1) and recovery time (t_2-t_1) respectively. The

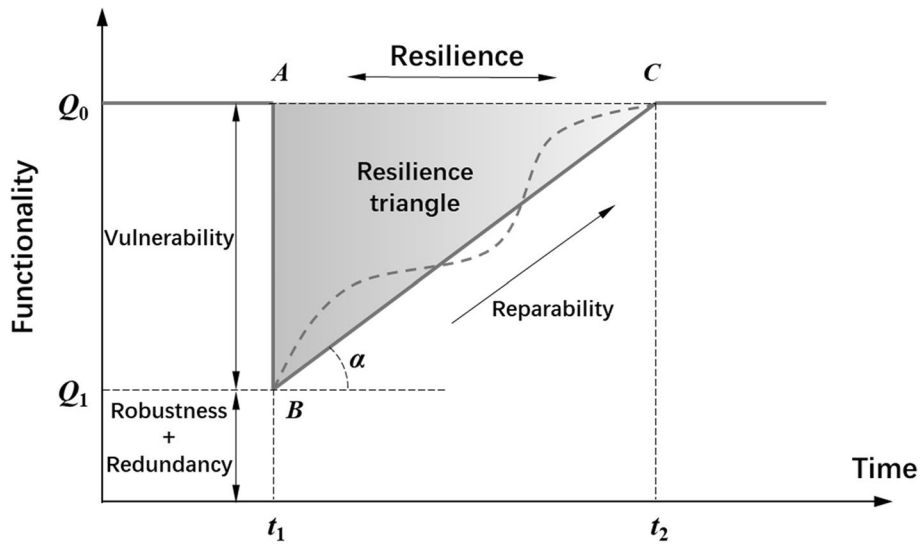

Fig. 1 Resilience triangle of bridge structures

hypotenuse $\overline{BC}$ represents the bridge's reparability, and the angle α represents the speed of bridge recovery, which can be expressed as Eq. (1). The angle α is larger, the bridge reparability is better.

$$\alpha = \arctan \frac{Q_0 - Q_1}{t_2 - t_1} \tag{1}$$

Resilience triangle of a bridge can be expressed by Eq. (2).

$$R_L = \int_{t_1}^{t_2} [Q_0 - Q(t)]\mathrm{dt} \tag{2}$$

where R_L is the area of the resilience triangle, i.e. the bridge resilience loss; t_1 is the time for the extreme event to occur; t_2 is the time for bridge performance to return to initial value; Q_0 is the bridge initial performance; $Q(t)$ is the time function of the recovery path.

It can be seen that the smaller the triangle area is, the better the bridge resilience become. Shortening the recovery time can improve the structural resilience under the same vulnerability, so it requires that the bridge has the ability of being reinforced or repaired rapidly.

2.2 Specific requirements for resilience design

Resilience design is a general concept that targets the life cycle design, which should have specific requirements that are measurable and implementable. Table 1 gives some specific requirements for resilience design and some unfavored occurrences, part of them are reflected in AASHTO LRFD Bridge Design Specifications and Eurocodes. Some negative cases will be introduced in detail in Section 4.

3 Resilience related provisions in existing codes of practice

In order to exam the current understanding of resiliency design, it is necessary to review the general governing equations for limit state design in the two current mainstream codes, AASHTO LRFD Bridge Design Specifications and Eurocodes.

Table 1 Specific requirements for resilience design

Specific requirements	Unfavored occurrences
Ductility	• Brittle failure modes due to shear or bending • Stress concentration
Robustness	• Insufficient strength or ductility of structural members and materials • Inadequate ability of the structure to withstand accidental actions
Redundancy	• Fracture critical member; uplift bearings during operation • Insufficient alternative load paths • Progressive collapse of structures due to local failure
Durability	• Corrosion damage of the suspender • Fatigue failure of the steel bridge deck weld joint
Man-made error	• Human errors are not considered in partial factor design
Overturning	• Insufficient anti-overturning factor for overloading • Improper bridge bearing layout • Too small curve radius
Extreme events	• Collapse upon ship collision • Unseating during earthquake

3.1 Specific requirements for resilience in the AASHTO specifications

In AASHTO LRFD Bridge Design Specifications, each structural component and connection shall satisfy Eq. (3) for each limit state.

$$\sum \eta_i \gamma_i Q_i \leq \phi R_n = R_r \tag{3}$$

in which:

For loads for which a maximum value of γ_i is appropriate:

$$\eta_i = \eta_D \eta_R \eta_I \geq 0.95 \tag{4}$$

For loads for which a minimum value of γ_i is appropriate:

$$\eta_i = \frac{1}{\eta_D \eta_R \eta_I} \leq 1.0 \tag{5}$$

where γ_i is load factor: a statistically based multiplier applied to force effects; ϕ is resistance factor: a statistically based multiplier applied to nominal resistance; η_i is load modifier: a factor relating to ductility, redundancy, and operational classification; η_D is a factor relating to ductility, as specified in Table 2; η_R is a factor relating to redundancy, as specified in Table 2; η_I is a factor relating to operational classification, as specified in Table 2; Q_i is force effect; R_n is nominal resistance; R_r is factored resistance: ϕR_n.

The load factor γ_i for each load comprising a design load combination shall be taken as specified in tables of AASHTO LRFD Bridge Design Specifications. For each load combination, every load that is indicated to be taken into account and that is germane to the component being designed, including all significant effects due to distortion, shall be multiplied by the appropriate load factor. The products shall be summed as specified in Eq. (3) and multiplied by the load modifier η_i.

The resistance factor ϕ shall be taken as specified in Fig. 2. And the value of ϕ associated with net tensile strain ε_t that is between the compression-controlled strain limit ε_{cl} and tension-controlled strain limit ε_{tl} may obtained by a linear interpolation from 0.75 to that for tension-controlled sections.

Table 2 The specific values of η_D, η_R, and η_I

Performance	Factor	Value	
		For the strength limit state	**For all other limit states**
Ductility	η_D	≥ 1.05 for nonductile components and connections = 1.00 for conventional designs and details complying with these Specifications ≥ 0.95 for components and connections for which additional ductility-enhancing measures have been specified beyond those required by these Specifications	= 1.00
Redundancy	η_R	≥ 1.05 for nonredundant members = 1.00 for conventional levels of redundancy ≥ 0.95 for exceptional levels of redundancy beyond girder continuity and a torsionally-closed cross-section	= 1.00
Operational Importance	η_I	≥ 1.05 for critical or essential bridges = 1.00 for typical bridges ≥ 0.95 for relatively less important bridges	= 1.00

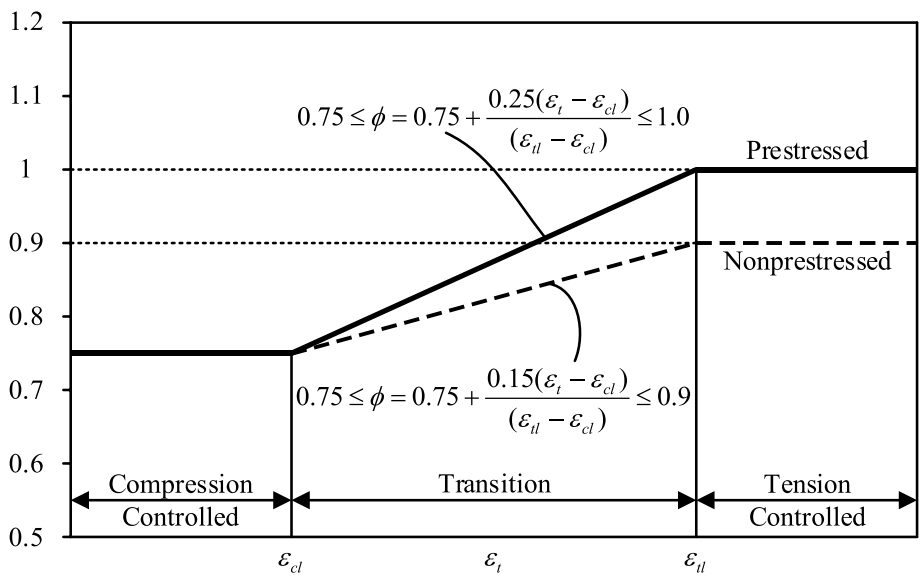

Fig. 2 Variation of φ with net tensile strain ε_t for nonprestressed reinforcement and for prestressing steel

This variation ϕ may be computed for prestressed members such that:

$$0.75 \le \phi = 0.75 + \frac{0.25(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 1.0 \tag{6}$$

and for nonprestressed members such that:

$$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9 \tag{7}$$

where ε_t is net tensile strain in the extreme tension steel at nominal resistance; ε_{cl} is compression-controlled strain limit in the extreme tension steel; ε_{tl} is tension-controlled strain limit in the extreme tension steel.

As mentioned above, the AASHTO Specifications is based on a thorough examination of the values of load modifier η_i, load factor γ_i, and resistance factor ϕ in order to fulfil the requirements of resilience.

3.2 Basic requirements related to resilience in Eurocode

Eurocode requires that a structure shall be designed to have adequate structural resistance, serviceability, and durability. Robustness, as one of the implications of resilience, is defined in Eurocode as: the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause. Eurocode suggests adopting the following approach to ensure sufficient robustness of the structure: designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant strain energy without rupture.

At the same time, some measures used to avoid potential damage are also recommended in Eurocode: 1) avoiding, eliminating or reducing the hazards to which the structure can be subjected; 2) selecting a structural form which has low sensitivity to the hazards considered; 3) selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage; 4) avoiding as far as possible structural systems that can collapse without warning; 5) tying the structural members together. These measures are dedicated to meeting the requirements of redundancy, which is another connotation of resilience.

Durability, as one of the basic requirements related to resilience, is also discussed in detail in Eurocode. In order to achieve an adequately durable structure, the following should be taken into account: 1) the intended or foreseeable use of the structure; 2) the required design criteria; 3) the expected environmental conditions; 4) the composition, properties and performance of the materials and products; 5) the properties of the soil; 6) the choice of the structural system; 7) the shape of members and the structural detailing; 8) the quality of workmanship, and the level of control; 9) the particular protective measures; 10) the intended n1aintenance during the design working life.

In Eurocode, design for limit states (ultimate limit states and serviceability limit states) shall be based on the use of structural and load models for relevant limit states. It shall be verified that no limit state is exceeded when relevant design values for actions, material properties, or product properties, and geometrical data are used in these models. And the basic requirements related to resilience are reflected by the partial factors used for the calculation of the above design values.

When designing the structure, the following equation should be verified:

$$E_d \leq R_d \tag{8}$$

where E_d is the design value of the effect of actions; R_d is the design resistance.

For a specific load case the design values of the effects of actions E_d can be expressed as:

$$E_d = \gamma_{Sd} E\left\{\gamma_{f,i} F_{rep,i}; a_d\right\} \quad i \geq 1 \tag{9}$$

where $F_{rep,\,i}$ is the relevant representative value of action i; $\gamma_{f,\,i}$ is a partial factor for action i which takes account of the possibility of unfavorable deviations of the action values from the representative values; a_d is the design values of the geometrical data; γ_{Sd} is a partial factor taking account of uncertainties.

The design resistance R_d can be expressed in the following form:

$$R_d = \frac{1}{\gamma_{Rd}} R\left\{X_{d,i}; a_d\right\} = \frac{1}{\gamma_{Rd}} R\left\{\eta_i \frac{X_{k,i}}{\gamma_{m,i}}; a_d\right\} \quad i \geq 1 \tag{10}$$

where $X_{d,\,i}$ is the design value of material property i; $X_{k,\,i}$ is the characteristic value of the material property i; η_i is the mean value of the conversion factor taking into account volume and scale effects, effects of moisture and temperature, and any other relevant parameters; $\gamma_{m,\,i}$ is the partial factor for the material property i to take account of the possibility of an unfavorable deviation of a material property i from its characteristic value, the random part of the conversion factor η; γ_{Rd} is a partial factor covering uncertainty in the resistance model, plus geometric deviations if these are not modelled explicitly.

In summary, both the AASHTO Specifications and the Eurocodes highlight the importance of selecting suitable values for the related coefficients in order to achieve the requirements for ductility, robustness, redundancy, durability, etc., required for resilience. However, the current codes are still to be developed and improved. For the load combination value exceeding the code, the structure should still have sufficient ductility and anti-collapse ability by replenishing more executable provisions.

4 Examples of poor bridge resilience

Based on bridge accidents worldwide, some structural systems and details of lower resilience are discussed below.

4.1 Bridges with insufficient overturning stability

Because of the advantage of the reduced land occupation, aesthetics and economy, single-column pier beam bridges are widely used in urban overpass and expressway ramp. However, the forms of single-column piers with single bearing or double bearings of small spacing are particularly unfavourable to the overall anti-overturning stability under the unsymmetrical vehicle passage, and there are no obvious omens before sudden collapse. The next section will discuss in detail overturning issues of single-column pier bridges and curved bridges. In recent years, the instability accidents of single-column pier bridges occur frequently. Figure 3 shows a scenario of the overturning collapse, which occurs at a viaduct intersection in Baotou China in 2007.

Fig. 3 Scenario of a bridge overturning

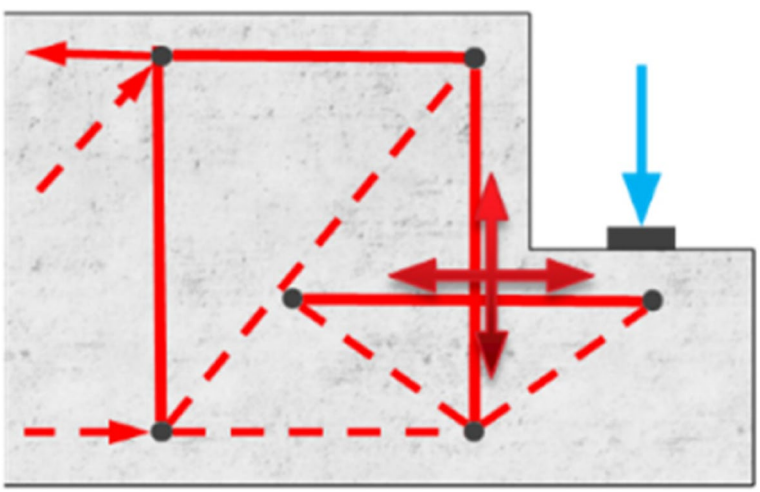

Fig. 4 A strut-and-tie model of a dapped end beam

4.2 Bridges with dapped end beams or drop-in span

Bridges with dapped end beams or drop-in span are short of the entirety, and stress concentration can appear easily in the positions of dapped end beams, where it is easy to occur cracks and difficult to meet the requirement of robustness. Figure 4 shows a strut-and-tie model of a dapped end beam, in which the dotted line is the compression bar and the solid line is the tension bar. In the tension area, a lot of reinforcement is usually designed, but it is still difficult to solve the cracking problem of dapped end beams. From another point of view, the drop-in span is only supported by dapped end beams. If the dapped end beam is damaged, the whole drop-in span will fall down. It indicates that this type of structure has very low redundancy. Figure 5 shows an accident case of a bridge with drop-in span (the Tianzhuangtai Bridge in Liaoning Province).

Fig. 5 The Tianzhuangtai Bridge in Liaoning Province

4.3 Steel bridges with fracture critical members

Such structures are represented by steel truss bridges, and the main reasons for their failure include insufficient bearing capacity of members, instability of members, fatigue brittle fracture of steel, and failure of gusset plates. A typical case is the collapse of the I-35 W Bridge in the United States. Because the bridge has only one point at the bearing position connected to the pier, there is no alternative force transmission path, and the redundancy is insufficient. When a few components are damaged, the entire structure can no longer resist. Therefore, when the U10 gusset plate at the support position is damaged (insufficient robustness), the internal force of the surrounding members is redistributed, and the bridge structure is changed meanwhile, which induces the continuous collapse of the whole bridge. The node design and force transmission path of the I-35 W Bridge in the United States are shown in Fig. 6, and the bridge photo is shown in Fig. 7.

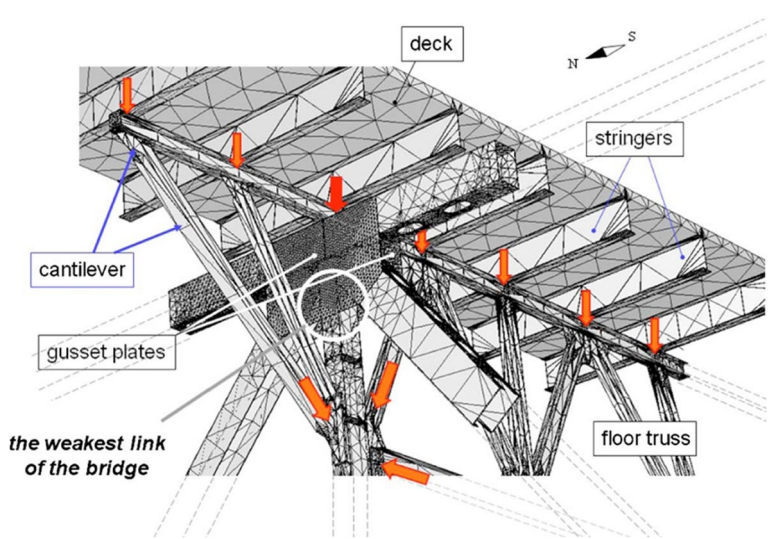

Fig. 6 The force transmission path of I-35w bridge (from Hao (2010)

Fig. 7 The photo of I-35W bridge

Fig. 8 The Xiaonanmen Bridge in Yibin

4.4 Arch bridges with suspended desk

Under the long-term working load, suspenders of an arch bridge with suspended desk are prone to stress corrosion. Compared with long suspenders, short suspenders are more vulnerable to damage. Meanwhile, when the rigid deck is lacked, suspenders may become fracture critical members. Therefore, the type of suspender is prone to sudden fracture, causing the bridge deck to fall. Insufficient robustness is the main reason of the bridge damage. The suspender is difficult to repair after corrosion damage, so it usually needs to be replaced. The destruction site of the Xiaonanmen Bridge in Yibin is shown in Fig. 8.

4.5 Rigid-framed concrete arch

This kind of bridge belongs to high order statically indeterminate structure, and the secondary stresses at joints most likely occur, which causes the concrete to produce

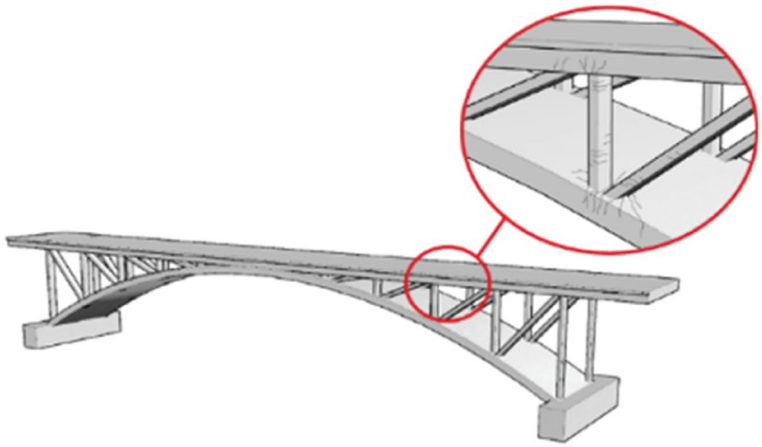

Fig. 9 Crack distribution in the joints

Fig. 10 The photo of the collapse site

interlaced cracks (as shown in Fig. 9). The connection between tie beams and arch leaf is weak and has high vulnerability. The maintenance for the structure needs to spend a lot of money and a long time. Furthermore, it is prone to cause bridge collapse when joints are destroyed.

A typical case is the Huairou Bai river bridge in Beijing, which is a reinforced concrete rigid arch bridge with the span layout of 4×50 m. After several years of operation, the progressive collapse of all the four spans happened when a severely overloaded 6-axle truck weighing 160 t passed through the second span. The bridge collapse site is shown in Fig. 10.

4.6 Bridges with negative support reaction

The auxiliary piers designed at the side span of a cable-stayed bridge can effectively decrease the side span deflection. The cable force change of end anchor cables can be

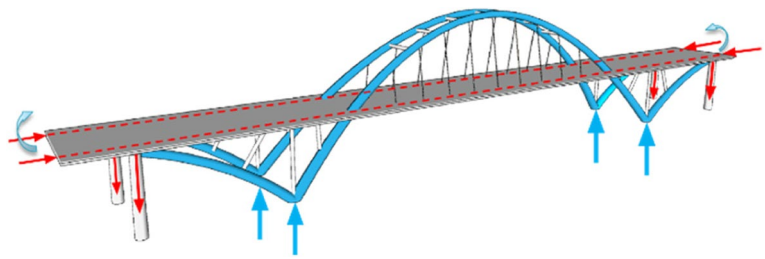

Fig. 11 Force transfer in a three-span half-through arch bridge

greatly decreased under live load, which can greatly improve the bridge overall rigidity. However, the supports of the auxiliary piers of some cable-stayed bridges are designed as tension-compression supports, which are prone to failure, such as the Poyang Lake Bridge, the Jinsha River Bridge, and the Yibin Yangtze River Bridge. For statically indeterminate cable-stayed bridges, the support failure can change the structural system, and cause internal force redistribution, thus affect the normal use of bridges.

The half-through tie arch bridge is a self-balancing system. The side ribs are located on the side pier supports, and the thrust of the arch is balanced by the tie bar tension (Fig. 11). If the structural parameters such as the side-to-mid-span length ratio are not handled properly, the initial pressure of the side pier support will be too small. The side arch ribs may become warped under the vehicle or temperature action load, resulting in negative reaction of the side support, and threatening the safe operation of the bridge.

4.7 Morandi cable-stayed bridges

As an original concrete cable-stayed bridge in the 1960s, the Morandi Bridges have a far-reaching influence in the world. The basic structure of the bridge is shown in Fig. 12. However, with the collapse of the Morandi's Polcevera viaduct in Genoa, Italy (August 14, 2018), bridge engineers have researched in-depth on its causes, such as Morgese et al (2020), Calvi et al (2019). In addition to the external environment, such as sea breeze corrosion, overloading, etc., it is also related to the weaken ductility, redundancy and reparability of the bridge itself. From the present point of view, the Morandi Bridge does not conform to the design principle of resilience. The structural form of double cables makes the cable become a fracture critical member, and the fracture of any cable can lead to the collapse of the whole span.

4.8 Bridges with low seismic ductility

Shear failure (Fig. 13) and bending-shear failure (Fig. 14) often occur in bridges that cannot be properly designed, which are brittle failures and can greatly decrease the ductility of seismic structures. In order to ensure the ductility of the structure and minimize the randomness of earthquake damage, New Zealand scholar Park et al. proposed the principle of capability protection design in seismic design (Park and Ang 1985). At present, more attention has been paid to the design concept of bridge resilience after earthquake.

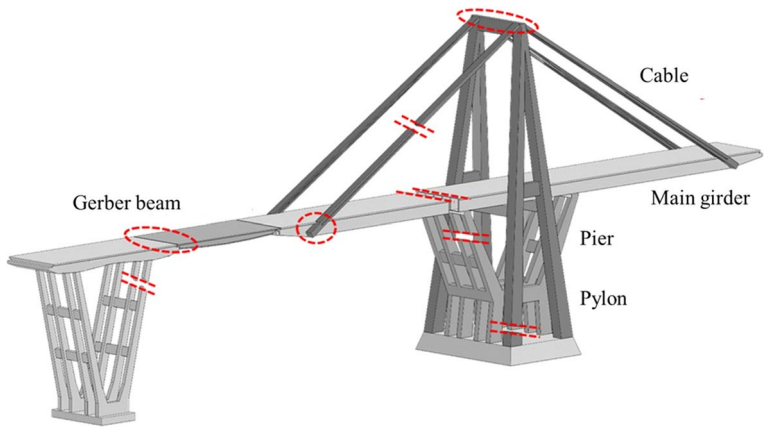

Fig. 12 The basic structure of the Morandi Bridge

Fig. 13 Shear failure

4.9 Tied arch bridges with single row suspenders

The problem is illustrated by the Taiwan Nanfang-ao cross-harbour bridge which is a tied arch bridge. The main span length of the bridge is 140 m and the width of the main beam is 15 m. The main arch is connected to the main beam with a fork at the end. The steel girder bears the horizontal thrust of the self-balancing arch. Completed in June 1998, the whole bridge collapsed in October 2019. The collapse processes are shown in Fig. 15.

It can be seen from Fig. 15 that this accident is a chain collapse problem caused by the local failure of the structure. Suspenders of the arch bridge are in the offshore working environment for a long time, and their stress corrosion is accelerated under vehicle load or other loads. After the failure of the first suspender, the internal forces are redistributed. This results in the increase of the residual suspender stress and the continuous fracture failure of these suspenders. Then the inner force of the arch ring is released, and the arch roof bounces back, and the

Fig. 14 Bending-shear failure

arch foot is contracted. The main beam cannot bear the bending deformation caused by the dead weight, forming plastic hinge in the middle of the span, and finally this leads to the whole collapse.

From the point of view about redundancy and robustness of the resilience design, bridge designers should ensure that even if one of the suspenders fails, the rest can keep in the normal state or at least withstand a progressive collapse. After the collapse of the bridge, the rationality of the bridge type is debatable.

5 Resilience design against bridge overturning

Bridge overturning is one of the worst anti-resilience scenarios, because it may result in catastrophic collapse. After the bridge overturned, looking at the resilience triangle in Fig. 1, the angle α representing the speed of bridge recovery is usually quite

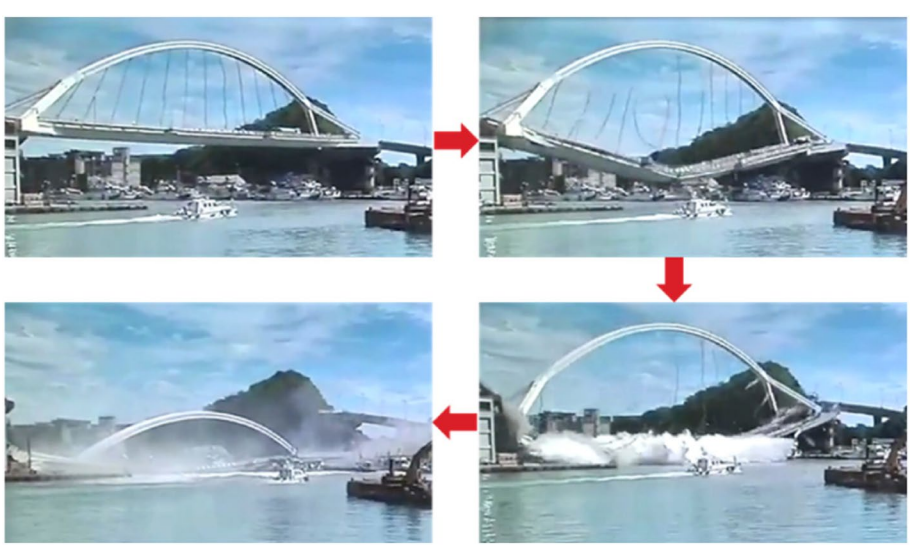

Fig. 15 The collapse processes of Taiwan Nanfang-ao cross-harbour bridge

small, which means that traffic is immediately interrupted and takes a long time to reconstruction.

The bridge types of most likely overturning are as follows: bridge with incorrectly calculated anti-overturning factor, curved bridge with too small curve radius, and bridge with unreasonable bearing settings.

Up to date, the evaluation criteria for bridge overturning is controversial, and the anti-overturning design method is to be developed. The in-depth study of anti-overturning design including anti-overturning factor, geometry of curved bridges, disposition of bearings, etc.

5.1 Calculation methods of anti-overturning factor

The calculation of the anti-overturning factor is an important aspect of bridge anti-overturning design. However, discrepancies in understanding among designers may lead to disparities in calculation results, which may result in inadequate structural robustness. Therefore, it is necessary to first discuss calculation methods of structural anti-overturning factor.

At present, the calculation methods of the anti-overturning factor are mainly divided into two types. The first one is calculated in terms of the deformable body, that is, the overturning problem of the beam is not only a rigid body contact rotation problem, but also contains multiple nonlinear behaviours including volume nonlinearity, material nonlinearity and contact nonlinearity. The second one is calculated in terms of the rigid body, that is, the influence of the stress state change and material deformation of the beam is ignored to facilitate the engineering calculation. In addition, there are two methods of dividing dead load to calculate the anti-overturning factor according to the rigid body: 1) the unified barycentre; 2) the axis of overturning. The two anti-overturning factors are calculated as follows:

(1) Structure seen as a deformable body

There are two definite characteristic states in the overturning process: in the state 1, the unidirectional support of the box girder begins to release from the compression state; in the state 2, the torsion support of the box girder fails completely. The typical failure state is shown in Fig. 16.

The anti-overturning factor is calculated as follows:

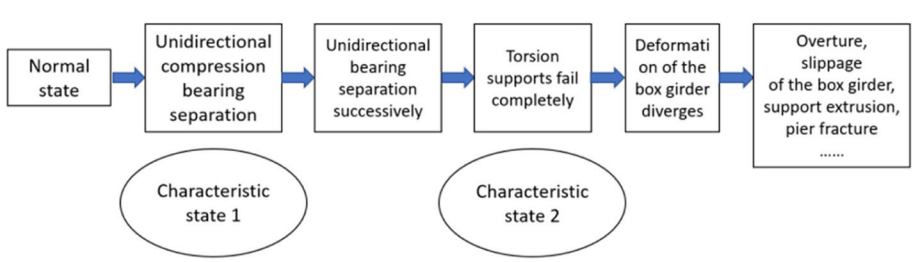

Fig. 16 Typical failure state of the box girder

$$K_{qf} = \frac{\sum S_{\mathrm{bk,i}}}{\sum S_{\mathrm{sk,i}}} = \frac{\sum R_{\mathrm{Gk,i}} l_i}{\sum R_{\mathrm{Qk,i}} l_i} \tag{11}$$

where $\sum S_{\mathrm{bk,\ i}}$ is the design value of the effect stabilizing the superstructure; $\sum S_{\mathrm{sk,\ i}}$ is the design value of the effect destabilizing the superstructure; l_i is the centre distance between the failure support and effective support on the i # pier; $R_{\mathrm{Gk}i}$ is the support reaction force of the ineffective support on the i # pier under permanent actions; $R_{\mathrm{Qk}i}$ is the support reaction force of the ineffective support on the i # pier under variable actions.

The above method abandons the concept of overturning axis and adopts the concept of overturning limit state. However, it needs to establish a fine finite element model and the calculation is more complicated.

(2) Structure seen as a rigid body

$$K_{qf} = \frac{\sum S_{\mathrm{bk,i}}}{\sum S_{\mathrm{sk,i}}} = \frac{\sum M_{\mathrm{K}}}{\sum M_{\mathrm{Q}}} \tag{12}$$

where $\sum M_{\mathrm{K}}$ is the anti-overturning moment; $\sum M_{\mathrm{Q}}$ is the overturning moment.

The following will compare the two methods to calculate the anti-overturning factor.

Algorithm 1: the anti-overturning moment is equal to the product of the weight of the whole section and the distance from the barycenter to the support. The anti-overturning factor expression is shown in the following formula, and the calculation diagram is shown in Fig. 17.

$$K_1 = \frac{\sum M_{\mathrm{K}}}{\sum M_{\mathrm{Q}}} = \frac{\mathrm{W} \times \mathrm{L_W}}{\mathrm{P} \times \mathrm{L_P}} \tag{13}$$

Algorithm 2: the section is separated by the overturning line at the support. The dead load on the side of the live load generates the overturning moment, and the anti-overturning moment is generated on the other side. The anti-overturning factor expression is shown in the following formula, and the calculation diagram is shown in Fig. 18.

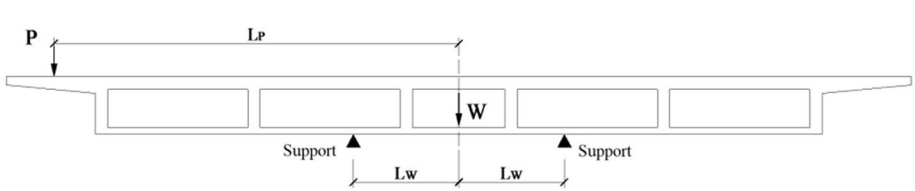

Fig. 17 The diagram of Algorithm 1

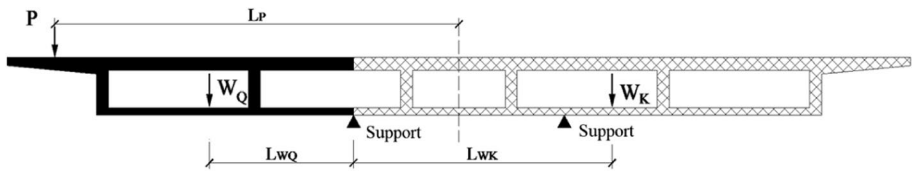

Fig. 18 The diagram of Algorithm 2

$$K_2 = \frac{\sum M_K}{\sum M_Q} = \frac{W_K \times L_{WK}}{P \times L_P + W_Q \times L_{WQ}} \tag{14}$$

In order to make a more obvious comparison between the two algorithms, the following takes a 35 m simply supported linear bridge as an example. The bridge is 38 m wide and the beam is 2.4 m high. The section size is shown in Fig. 19. The value of vehicle load is determined according to JTG D60-2015 (namely General Specifications for Design of Highway Bridges and Culverts), and the calculation results of the two algorithms are shown in Fig. 20.

As can be seen from Fig. 19, the factors obtained by the two algorithms for the same section are quite different. Compared with algorithm 1, the anti-overturning factor obtained by algorithm 2 decreased significantly. For the bridge, the support spacing worth paying attention to is 3.5 m ~ 9.0 m. In this range, algorithm 1 meets the requirements of the specification, and the other does not meet the requirements. However, both algorithms are used in actual bridge design.

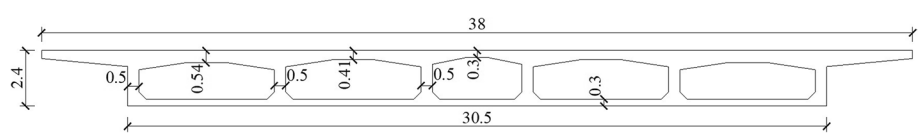

Fig. 19 The section size (unit: m)

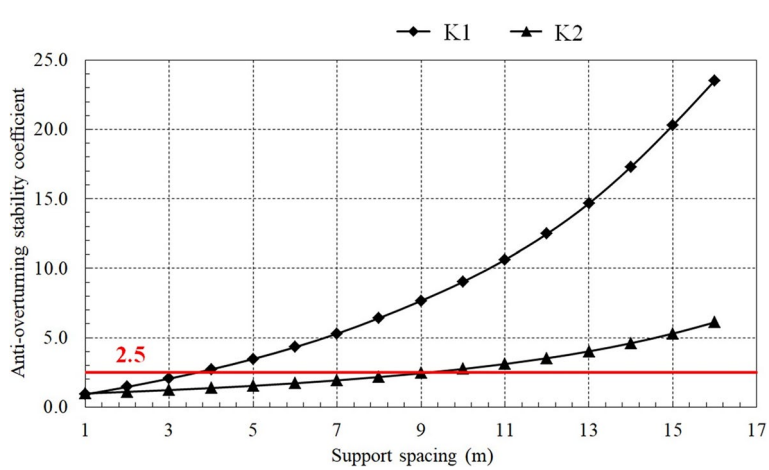

Fig. 20 Comparison of calculation results of two algorithms

5.2 Anti-overturning stability of curved bridges

Different from the linear bridge, the stress state of the curved bridge is more complicated. Due to the influence of curvature, the barycentre of the beam is inclined to the outer side of the centre line, which can generate torque and result in inconsistent side support reaction, or even support separation. In addition, some other factors, such as temperature of beams, super-elevation, cross slope, automobile centrifugal force etc., make the design of the curve beam more complex; more attention should be paid to the resilience design of the curved bridge. The barycentre position of the curved beam under different curvature conditions is shown in Fig. 21. In this figure, R is the outer diameter of the curved beam; r is the inner diameter of the curved beam; B is the beam width; b is the support spacing; θ is half of the central angle; X_c is the distance between the barycentre and the centre; X_{qf} is the distance between the overturning line and the centre; R_e is the distance between the outer support and the centre.

To simplify the calculation, the beam is regarded as a homogeneous plate. When the barycentre falls on the overturning line, the calculation formulas of X_c and X_{qf} are as follows.

$$X_c = \frac{2\left(R^3 - r^3\right)\sin\theta}{3\left(R^2 - r^2\right)\theta} \tag{15}$$

$$X_{qf} = R_e \cos\theta \tag{16}$$

If vehicle load is taken into account, the calculation formula of anti-overturning coefficient is shown in the following formula.

$$K = \frac{W\left(R_e \cos\theta - X_c\right)}{Q\left(X_{qf} - R_e \cos\theta\right)} \le K_{qf} \tag{17}$$

Taking a simply supported curve beam as an example, the basic information: the width is 12 m, support spacing is 4 m, when the barycentre falls on the overturning line, the relationship between r and θ is shown in Fig. 22.

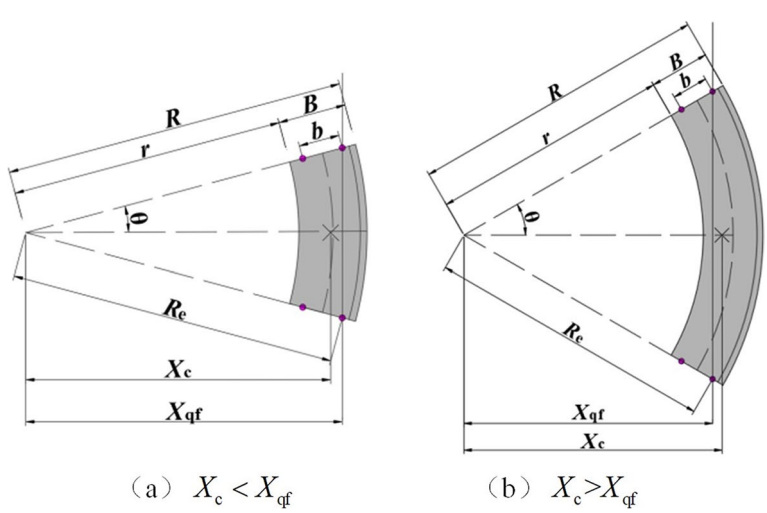

Fig. 21 Barycentre position of curved beam

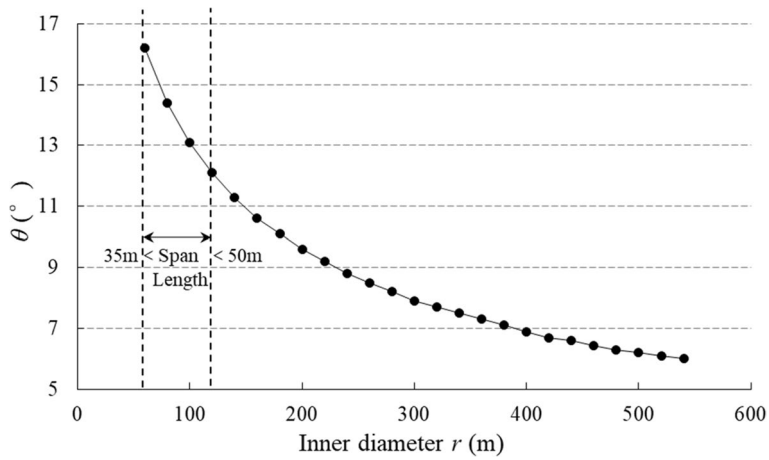

Fig. 22 Relationship between r and θ

As can be seen from the figure above, the area on the upper side of the curve is $X_c > X_{qf}$, while the area on the lower side is $X_c < X_{qf}$. In the actual design, the relationship between r and θ should be ensured to be located on the lower side of the curve. For the bridge, particular attention should be paid to the area between the two dotted lines, that is, 35 m ~ 50 m is the range of the commonly designed span, which is prone to occur designer's design mistakes that the beam barycentre is outside the overturning line.

In the actual situation, the curve beam is also faced with other adverse factors, such as single-column piers, small spacing support, overload, puncture, support disease, lateral wind load, vehicle impact, and earthquake. Therefore, under design principles for bridge resilience, the reasonable design range should be formed to meet the requirements of robustness and redundancy.

5.3 Reasonable disposition of supports

Anti-overturning design mainly focuses attention on the beam itself, and ensures the supports not separation. However, in the support design, the design value of the support reaction is calculated according to the standard value of vertical load, so support collapse may occur before overturning coefficient of the beam reaches 2.5, resulting in the beam overturn. Therefore, the support design should be unified with the anti-overturning design of the beam, but not design them separately.

5.4 Anti-overturning countermeasures

In bridge design, the designer should consider the factors affecting the bridge anti-overturning comprehensively. The correspondence between structural anti-overturning and resilience design criteria is shown in Fig. 23.

In practice, some retrofit options against overturning are shown in Fig. 24, including adding steel tie plate, adding cap beam, adding steel pipe column, and enlarging bridge pier.

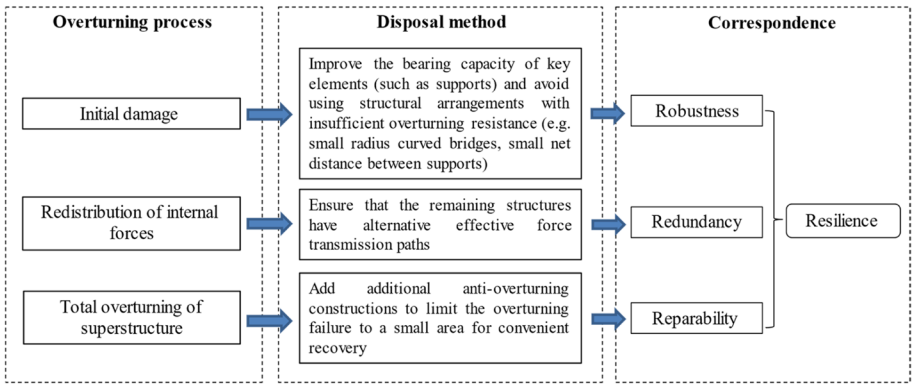

Fig. 23 Correspondence between structural anti-overturning and resilience design criteria

Fig. 24 Retrofit options against overturning

6 Conclusions

(1) Bridge resilience design highlights the ability to adapt to various unforeseen failures and quickly recovery of its loss of functionality, involving robustness, redundancy and reparability. The bridge resilience design should be incorporated into the life-cycle design framework. The bridge resilience triangle is proposed, which indicates that the bridge resiliency can be measured by restoration of sudden functionality degradation with time.

(2) By reviewing the current AASHTO Specifications and the Eurocodes, the specific requirements for resilience, such as ductility, robustness, redundancy, durability, etc., have been partly involved. However, the codes may need still to be improved by provid-

ing more implementable provisions to enhance the bridge ductility and collapse resistance, when exposed to unexpected loading scenarios beyond the code regulations.

(3) In order to calls for constant vigilance, some bridge failure examples with poor resilience design are given and discussed. Whereas overturning is one of the worst anti-resilience scenarios, the anti-overturning design method based on resilience is developed through a detailed discussion including the calculation methods of anti-overturning factor, anti-overturning stability of curved bridges, reasonable disposition of supports, and anti-overturning countermeasures.

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Authors' contributions
EW carried out the literature review, analyzed the bridge resilience against overturning, and prepared the original draft. ZL conceptualized, supervised, reviewed and edited the manuscript. ML and DZ investigated and collected the examples of poor bridge resilience, and helped to draft the manuscript. All authors read and approved the final manuscript.

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Declarations

Competing interests
Zhao Liu is an editorial board member for Advances in Bridge Engineering and was not involved in the editorial review, or the decision to publish, this article. All authors declare that there are no competing interests.

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