Journal of World Architecture

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Journal of World Architecture

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- Impacts of architecture on society, economy and environment;
- Analysis of occupants physically and psychologically and the application of new technologies, materials to meet their needs;
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Numerical Simulation Research on Response Characteristics of Grouting Defects of Ground Penetrating Radar for Detection of Grouting Quality behind Tunnel Wall

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Abstract: The effect of grouting behind tunnel wall directly affects the surrounding ground settlement and the stability of tunnel structure, so the grouting quality detection is very necessary. As an efficient and convenient shallow geophysical exploration method, ground-penetrating radar can meet the high-resolution and non-destructive requirements of grouting quality detection behind the tunnel wall, so it is widely used in engineering in recent years. Most of the existing studies have obvious regional pertinence and special geological conditions, and there are few universal studies on the characteristics of the ground penetrating radar reflection image of the grouting defect behind the tunnel wall. In view of this, this paper uses the finite difference time domain method to simulate several grouting defects behind the wall, such as voids, water-bearing anomaly, cracks, and other grouting defects. The simulation results show that the reflection image of the direct wave is characterized by a white band with strong amplitude; the interface between primary support and second lining, primary support, and surrounding rock is also banded; the circular cavity and water anomaly characteristics are all hyperbolic, the difference is that the phase of the lower part of the radar image of the cavity anomaly is 0, and there are only hyperbolic tails on both sides, and the water-bearing anomaly also has obvious hyperbolic characteristics at each interface; the reflected wave characteristics of the rectangular crack are striped and watery and the reflected wave characteristic of rectangular cracks is striped, and the abnormal range of water-bearing cracks on the radar image is larger than that of air. The research results can provide an effective theoretical reference for the engineering application of ground penetrating radar detection of grouting defects behind the tunnel wall.

Keywords: Grouting behind the wall; Quality detection; GPR; Numerical simulation

Publication date: July 2021; ***Online publication:*** July 31, 2021

1. Introduction

With the rapid development of China's social economy, more and more road tunnels are being constructed. However, due to various reasons such as tunnel construction technology, construction environment, supervision efforts, and other reasons, most tunnels will inevitably have geological abnormalities such as cavities, water-bearing anomaly, and cracks behind concrete walls. In engineering, grouting is generally used to reinforce and treat them. Therefore, the detection of grouting quality is very important. Commonly used methods of grouting quality detection include analytical method, detection hole method, and acoustic monitoring method [1-3], but these methods all require excavation and drilling, which requires a large amount of work and is time-consuming and labor-intensive. As an efficient and convenient geophysical detection

method, ground penetrating radar has significant advantages such as fast and non-destructive detection. Numerical simulation method is the main method of geophysical exploration research.

The synthetic exploration records of numerical simulation calculation can be used for the design and evaluation of field observation system, test the advantages and disadvantages of various data processing methods and inversion methods, verify the correctness of interpretation conclusions, and provide effective theoretical guidance for practical engineering application. In view of this, many scholars at home and abroad have carried out a lot of numerical simulation research based on GPR technology. Chunjin Lin, Xiongyao Xie et al.[4-5] used finite difference time domain (FDTD) to carry out numerical experiments on ground penetrating radar detection forward simulation and back-projection imaging, and established the ground penetrating radar detection interpretation criteria for typical lining diseases; Mingzhou Bai et al.[6] studied the typical radar image characteristics of different types of karst caves and soil caves by combining numerical simulation and model test, taking the karst area in the eastern section of Beijing Shanghai high-speed railway as the experimental area, and proposed the method and characteristic standard of using ground penetrating radar method to determine the grouting effect. Furthermore, Huang Hongwei et al. [7-9] conducted simulation experiments on the ground penetrating radar detection target behind the soft soil shield tunnel segments, and found that the age, distribution shape, and composition ratio of the grouting material can be used to detect the GPR profile. The figure has a significant impact, as Zeng Chenchao et al. [10] conducted a ground penetrating radar forward simulation on two typical working conditions of uneven grouting behind the tunnel lining wall and tunnel segment cracks and used the consistency elimination method to process the simulation data, and successfully obtained the radar image characteristics of the two working conditions. However, most of the existing research results have obvious regional pertinence and special geological conditions, and there are few general studies on the characteristics of the ground penetrating radar reflection image characteristics of the grouting defect behind the tunnel wall.

On the basis of summarizing the previous research methods and achievements, this paper designs three kinds of abnormal conditions that may exist after grouting in the tunnel wall, which are cavity defect, water-bearing defect and crack defect. The finite element difference method is used to simulate the three kinds of abnormal conditions, and the GPR response characteristics and laws are summarized, the superiority of GPR in detecting the structural integrity of concealed engineering body is verified, which lays the foundation for the follow-up research and has certain application value.

2. Basic principle of ground penetrating radar

As a geophysical detection method, ground penetrating radar (GPR) [11-13] has been widely used in the engineering field with remarkable effect. The geophysical basis of its detection is the difference of electrical parameters between the target and the surrounding medium (including dielectric constant, conductivity, permeability, etc.). The working principle is that the GPR transmitting antenna emits a high-frequency pulse electromagnetic wave (20MHz-2GHz). When the high-frequency electromagnetic wave propagates downward and meets the interface with a large difference in dielectric constant, it will reflect and refract. We can get the internal structure information of the target by analyzing the scattered wave image. **Figure 1.** is a schematic diagram of the working principle of GPR.

In detecting the quality and defects of concealed engineering structures, ground penetrating radar has a good application effect [14-16]. In this paper, 800 MHz ground penetrating radar is used to finely detect grouting defects behind the tunnel wall, and the radar image characteristics of different defects are summarized. **Figure 2.** shows how the ground penetrating radar detects geological anomalies behind the concrete wall.

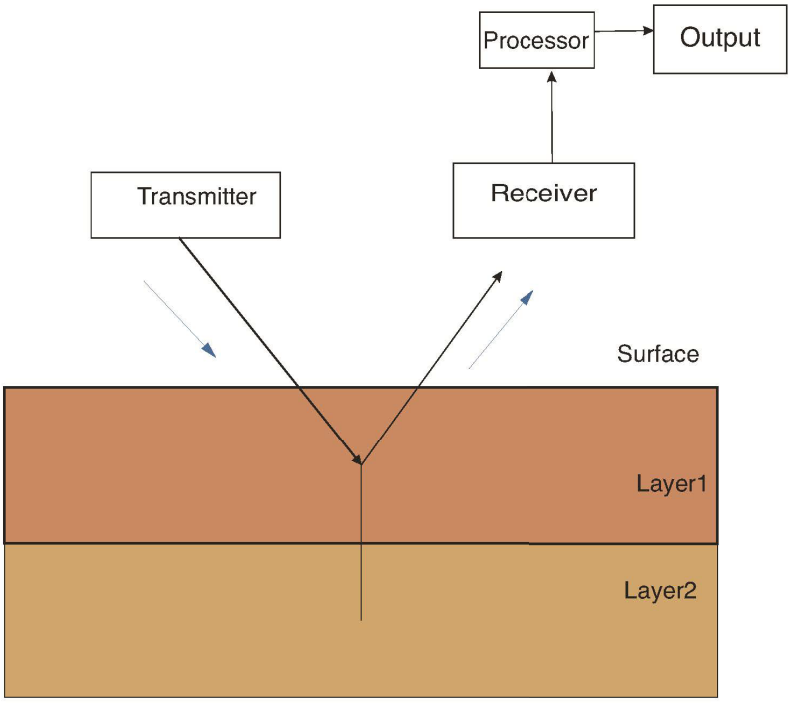

Figure 1. Working principle of GPR

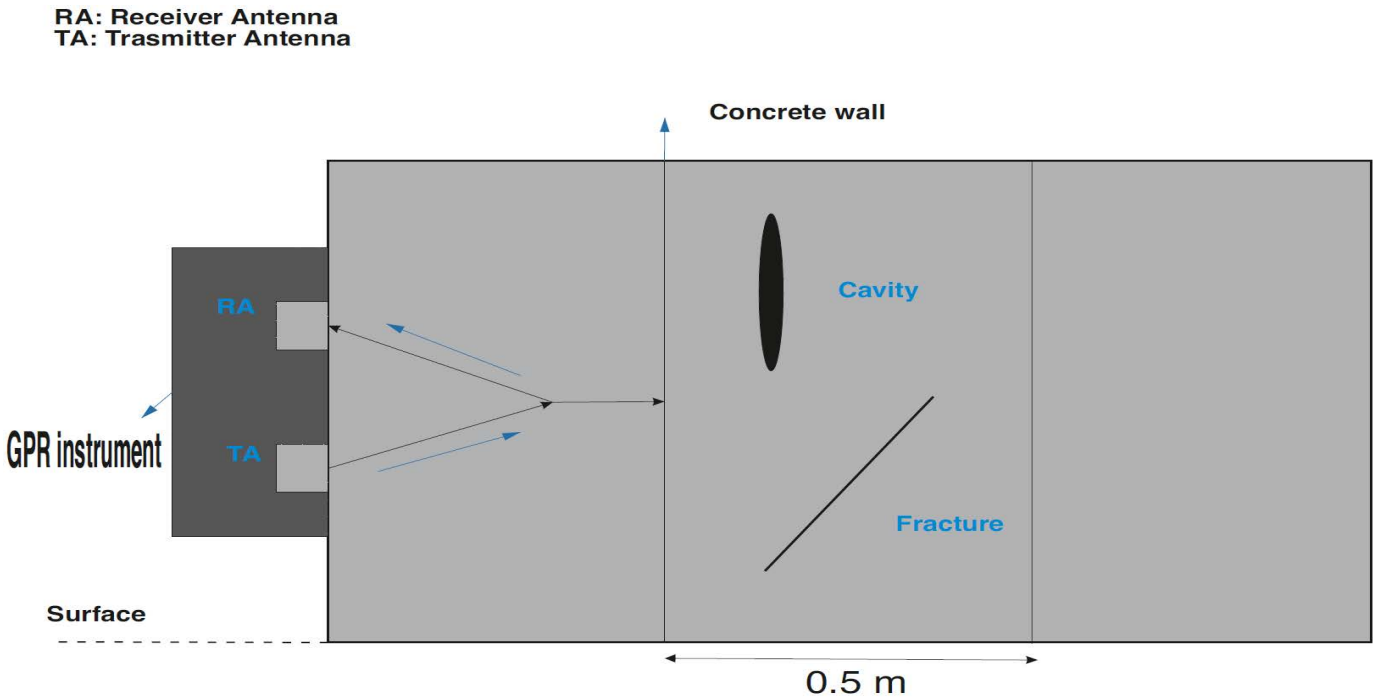

Figure 2. Use of ground penetrating radar method behind concrete wall of tunnel to detect geological abnormalities with high resolution

3. Numerical Simulation

3.1. Numerical simulation method

Ground penetrating radar is a shallow geophysical exploration method that detects the internal structural integrity of concealed engineering body according to the difference of electromagnetic properties of different media by transmitting high-frequency pulse electromagnetic wave and receiving the reflection wave of underground media [17]. According to the theory of electromagnetic wave propagation, the propagation of high-frequency pulse electromagnetic wave emitted by GPR obeys Maxwell equations. Maxwell's equations are composed of electromagnetic induction law and Ampere's circulation law, which

are the basic methods to describe electromagnetic fields.

The difference time domain method [18] is a method based on Maxwell's equations to calculate electromagnetic waves, which was proposed by K.S.Yee [19]. He divided the simulated electromagnetic space into finite grids and solved the electromagnetic field value in the whole simulation space through the electromagnetic field in a single grid.

The basic equation of FDTD method is as follows :-

$$\begin{cases} \frac{\partial H_z}{\partial y} - \frac{\partial H_y}{\partial z} = \varepsilon \frac{\partial E_x}{\partial t} + \partial E_x \\ \frac{\partial H_x}{\partial z} - \frac{\partial H_z}{\partial x} = \varepsilon \frac{\partial E_y}{\partial t} + \partial E_y \\ \frac{\partial H_y}{\partial x} - \frac{\partial H_x}{\partial y} = \varepsilon \frac{\partial E_z}{\partial t} + \partial E_z \\ \frac{\partial H_z}{\partial y} - \frac{\partial E_y}{\partial z} = -\mu \frac{\partial H_x}{\partial t} - \sigma_m H_x \\ \frac{\partial H_x}{\partial z} - \frac{\partial E_z}{\partial x} = -\mu \frac{\partial H_y}{\partial t} - \sigma_m H_y \\ \frac{\partial H_y}{\partial x} - \frac{\partial E_x}{\partial y} = -\mu \frac{\partial H_z}{\partial t} - \sigma_m H_z \end{cases} \quad (1)$$

In the equations, H is the magnetic field intensity (A/m); ε is the dielectric constant; E is the electric field intensity (V/m); σ is the electrical conductivity; μ is the permeability; σm is the magnetic permeability.

The GPR simulation results can be obtained by assigning the value of E and H. The GPR obtains the scanning image of the section by the continuous drag of the antenna. According to the characteristics of the reflected wave signal on the scanning image, the anomaly including cavity, water content, and crack behind the lining and the distribution of steel bars and steel frame are identified. Then, the buried depth and position of different defects and steel bars and steel frames are calculated according to the time-depth conversion formula. Where, h is the depth of the target body; t is the two-way reflection time of target to radar wave.

Calculation formula :-

$$v = \frac{2h}{t} \quad (2)$$

3.2. Forward simulation and analysis

The basic model is a rectangular area of 5m×2m, and the model setting parameters are shown in **Table 1.** The grid step size dx and dy of each model are both 0.01, the time window is set to 18ns. The number of stacks is 480, the center frequency of the antenna is 800MHz, the starting position of the transmitting antenna is (0.10, 1.95), the transmitting and receiving distance is 0.05m, and the antenna moving step size along the horizontal direction is 0.01m.

Table 1. Model setting parameters

Region	Medium	Dielectric constant	Conducti-vity (s/m)	Thickness (m)
Second lining	C35 concrete	9	0.01	0.4
Waterproof board	EVA	3	0.01	0.01
Initial support	C25 concrete	8	0.01	0.25
Surrounding rock	Limestone	20	0.025	1.29
cavity	Air	1	0	0.1
Water-bearing defect	Water	81	0.01	0.1

3.2.1. Circular anomaly model

The orange area at the bottom of the model is limestone, which is used to simulate the surrounding rock of the tunnel; The green area is the primary support, and the blue area is the secondary lining. Its dielectric constant is 8 and 9 respectively. There is a rectangular area in the middle with a dielectric constant of 3 and a thickness of only 0.01, which is used to simulate the waterproof board between the primary support and the secondary lining.

3.2.1.1. Single anomaly model

The anomalies are all located at the same depth of the second lining, and their radius is 0.05m.

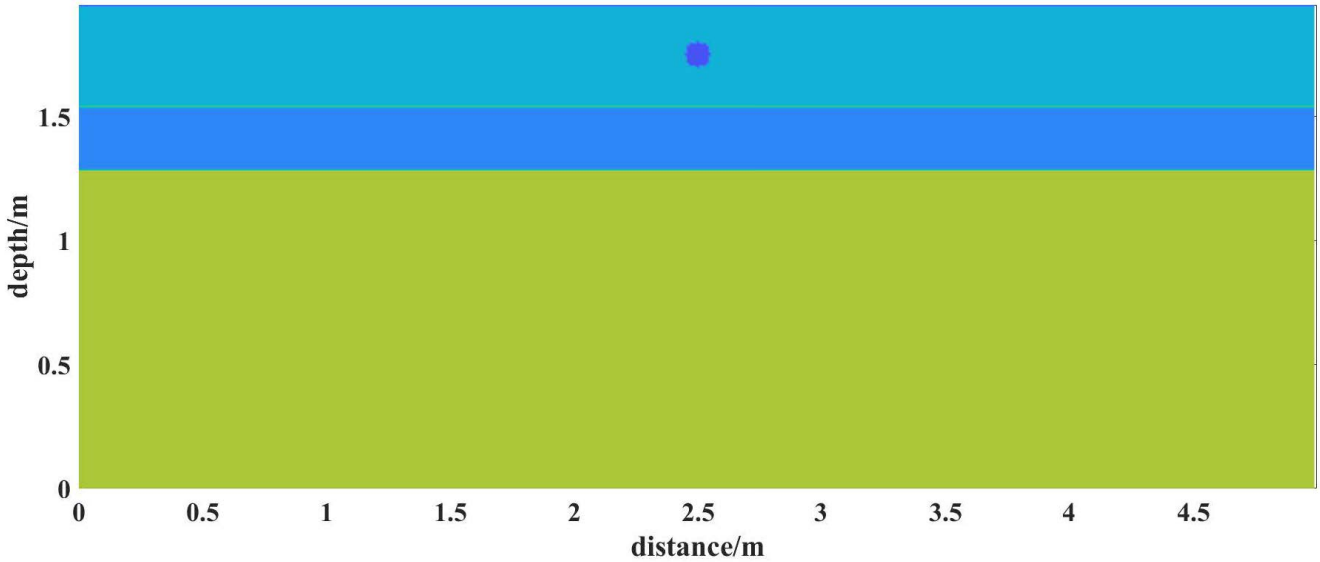

Figure 3. (a) Model

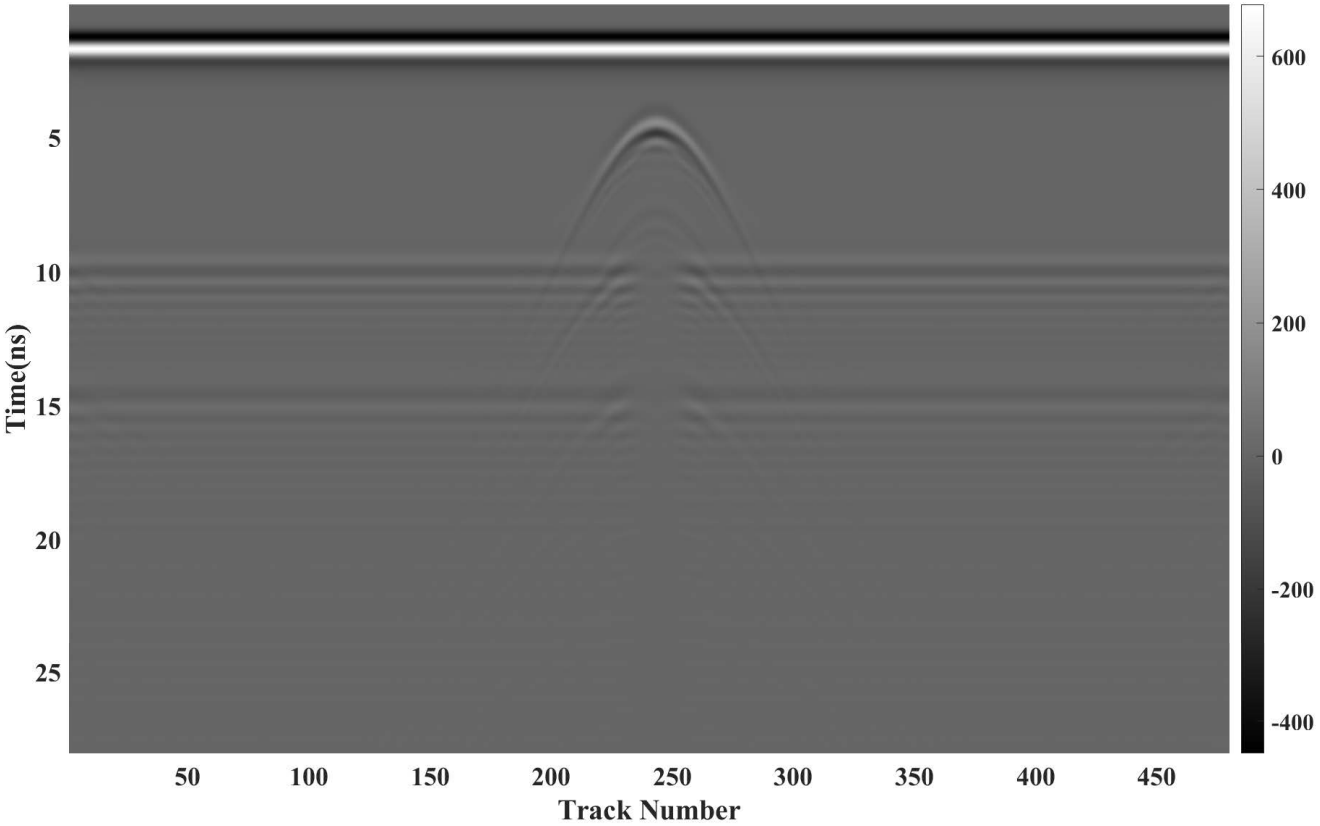

Figure 3. (b) Cavity response record

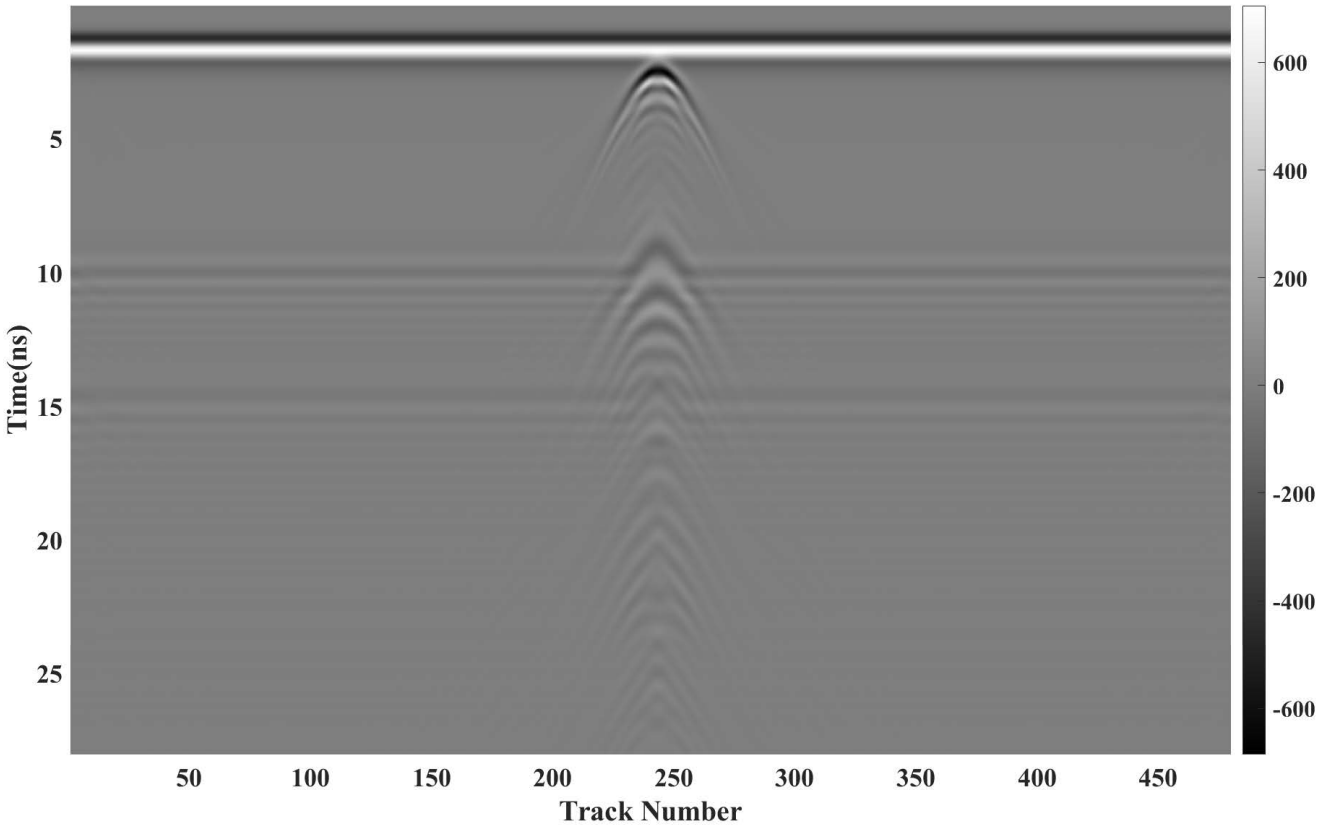

Figure 3. (c) Water-bearing anomaly response record

Figure 3. Forward diagrams of a single abnormal model

It can be seen from **Figure 3. (b)** and **Figure 3. (c)** that when the circular anomalies are located at the same depth, the forward records are basically the same. The white strips in the figure are direct waves, and the cavities and water-bearing anomalies have hyperbolic reflection, as well as the interfaces between the secondary lining and the primary support, and between the primary support and the surrounding rock. The difference is that the phase of cavity anomaly is almost the same at 4ns, and there is no obvious multiple at the bottom, while the phase of water-bearing anomaly is quite different at 4ns, and there is multiple at the bottom. It is also different at the two interfaces. The interface in the hole record, that is, the phase corresponding to the top of the 4ns reflection hyperbola is zero, and only the two ends have hyperbolic tails; while the two interfaces with anomalous water content have the reflection hyperbola is complete, and the top position of the arc is sharper than that at 4ns.

3.2.1.2. Different depth models

The abnormal coordinates from left to right are (1,1.29), (2,1.54), (3,1.75), (4,1.39), which are respectively located at the junction of the primary support and surrounding rock, the junction of the secondary lining and the primary support (i.e. the position of waterproof board), the second lining and the primary support, with radius of 0.05m.

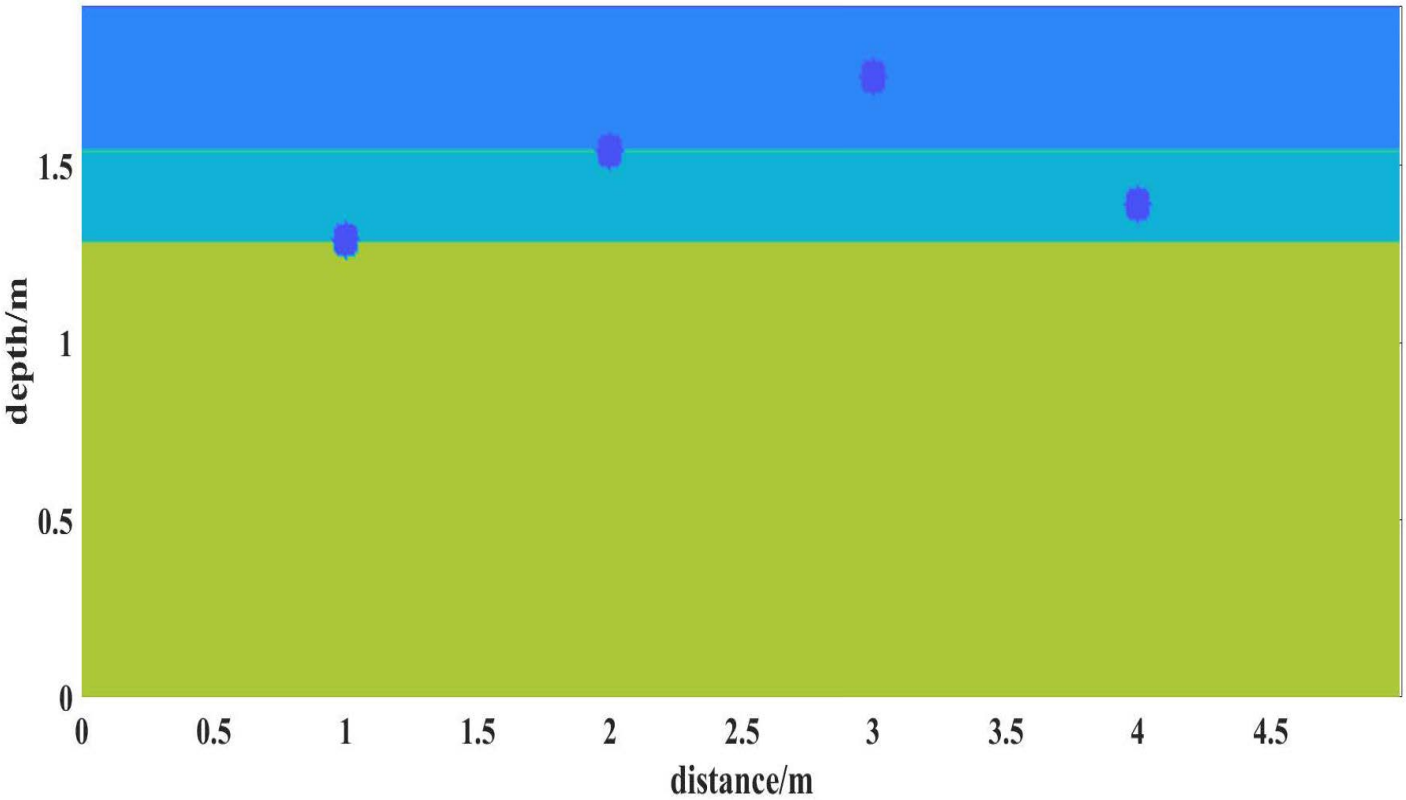

Figure 4. (a) Model

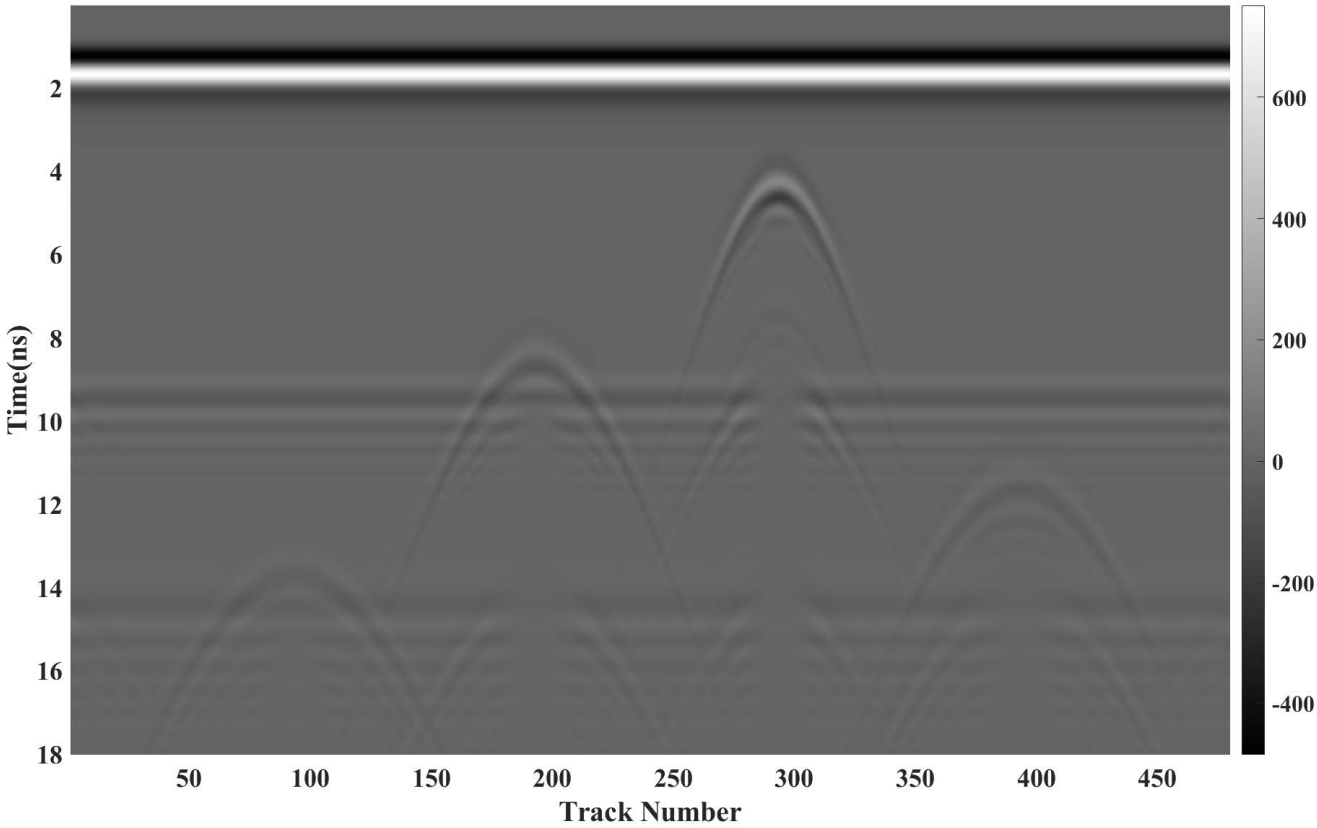

Figure 4. (b) Cavity response record

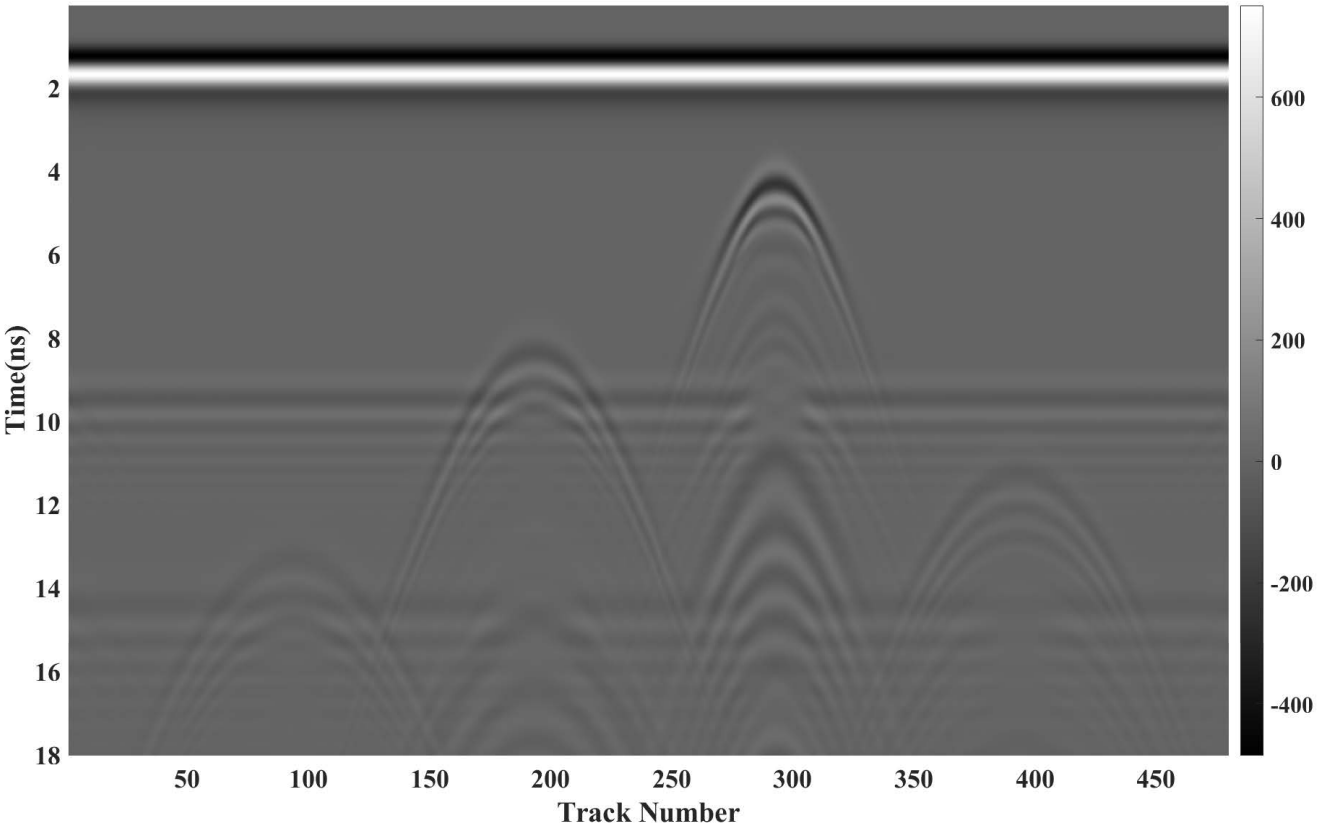

Figure 4. (c)Water-bearing anomaly response record

Figure 4. Forward diagrams of different depth models

It can be seen from **Figure 4. (b)** and **Figure 4. (c)** that when the positions of circular anomalies are the same, the forward records are basically the same. The white strips in the figure are direct waves, and the cavities and water-bearing anomalies have hyperbolic reflection, and there are also interfaces between the secondary lining and the primary support, and the primary support and the surrounding rock. There are first reflected waves at 13ns, 7ns, 4ns, and 11ns in the recording picture, which correspond to the circular anomalies of the model from left to right. The anomaly at the junction of the primary support and the surrounding rock is located below, and there is no interface at the lower part of the anomaly. There is only a group of reflection hyperbolas. The cavity record can only distinguish one hyperbola, while the water-bearing anomaly forward record can distinguish three hyperbolas; There is an interface between the primary support and the surrounding rock under the anomaly at the junction of the primary support and the second lining. There is a group of reflection hyperbolas at the corresponding lower position. The phase of the corresponding position of the cavity record arc top is 0, and the water-bearing anomaly record can roughly distinguish the arc top; There are two groups of hyperbolas at the lower part of the cavity record, and the phase of the corresponding position of the arc top is 0. In the water-bearing anomaly record, the hyperbolas are connected. The phase of the corresponding position of the interface between the primary support and the secondary lining is 0, and there are hyperbolic tails at both ends. The hyperbolic arc top of the corresponding position of the primary support and the surrounding rock is obvious; The anomaly images in the initial branch are roughly the same, and the hyperbolic top at the interface is not obvious.

3.2.1.3. Same depth model

The 15 anomalies of the model are set at a depth of 0.2m, and the horizontal distance is 1.6-4.4m, and the interval is evenly distributed by 0.2m.

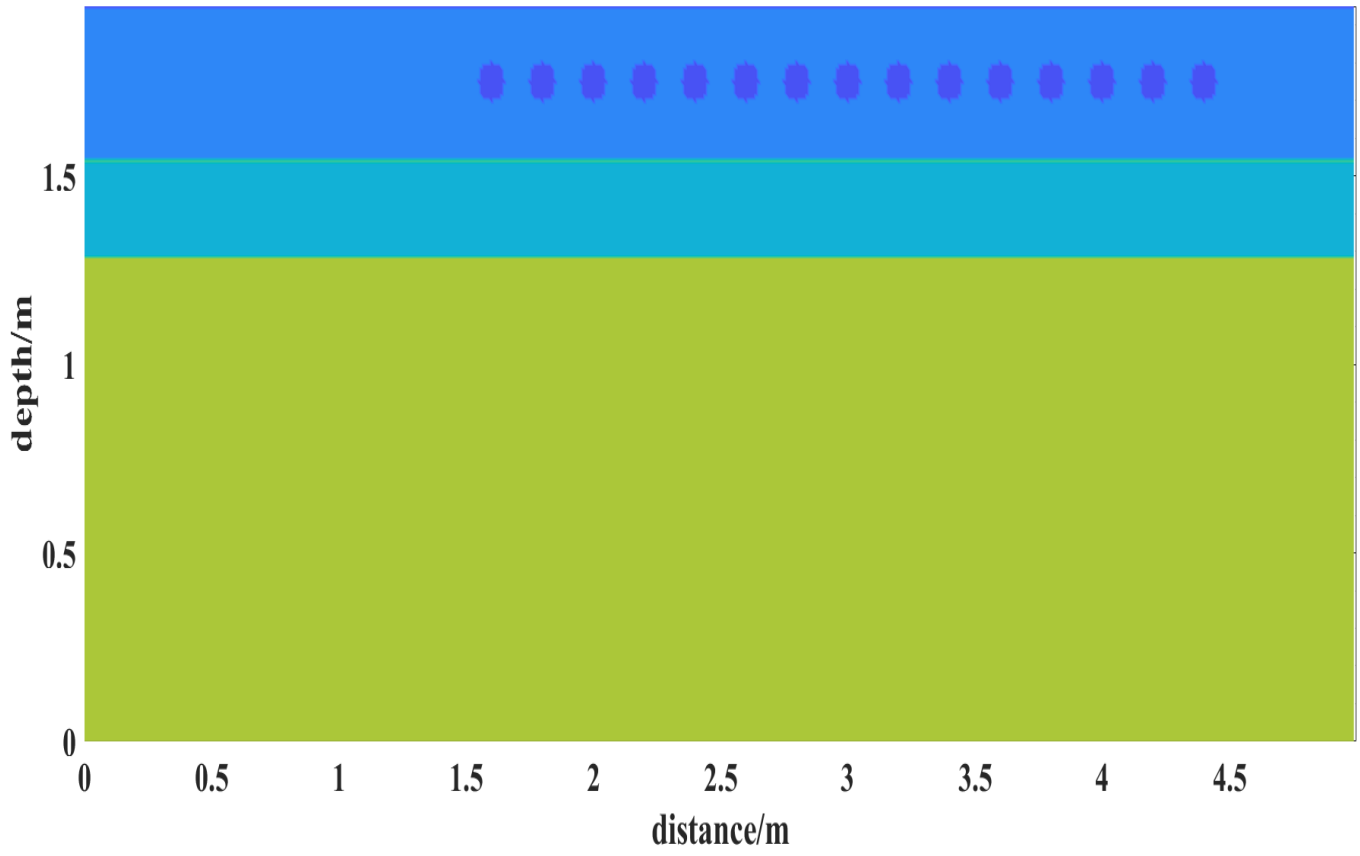

Figure 5. (a) Model

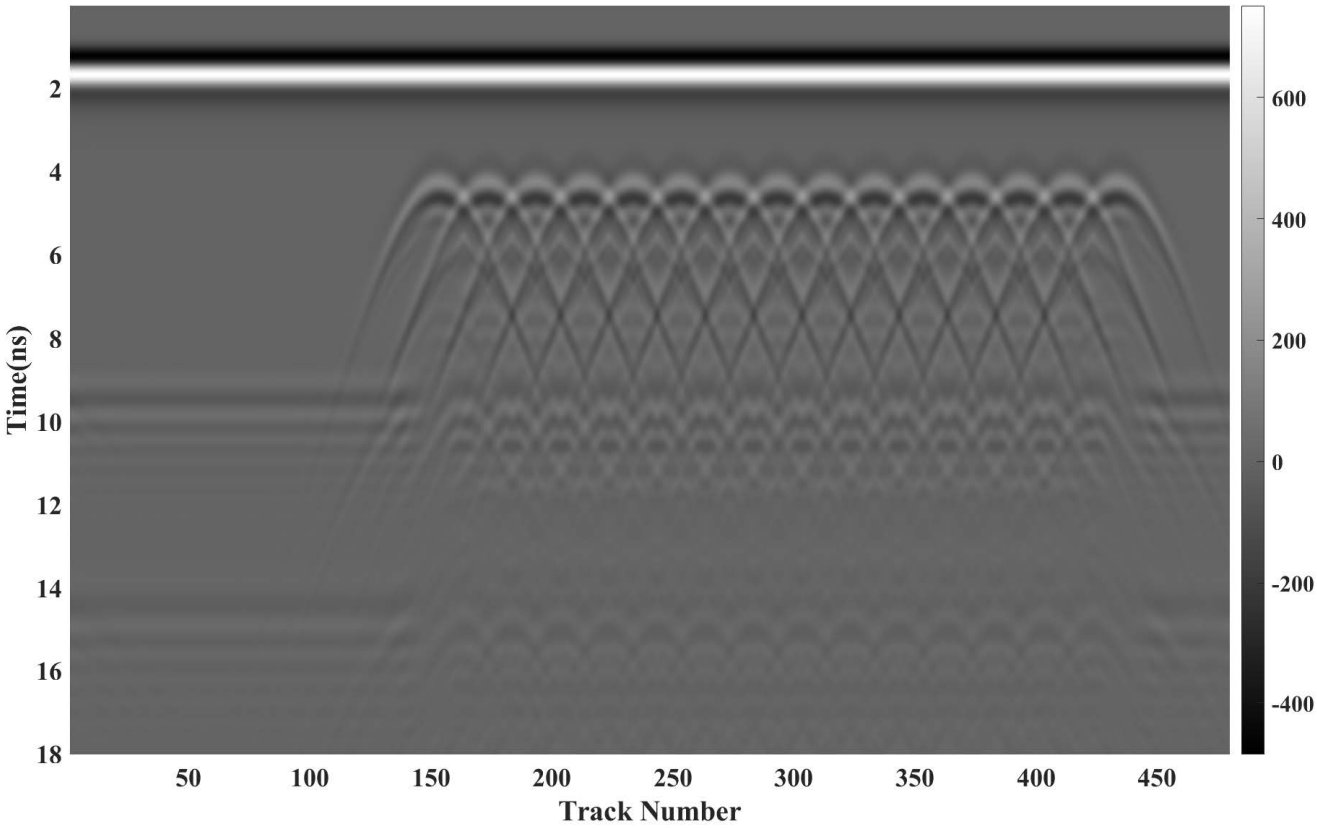

Figure 5. (b) Cavity response record

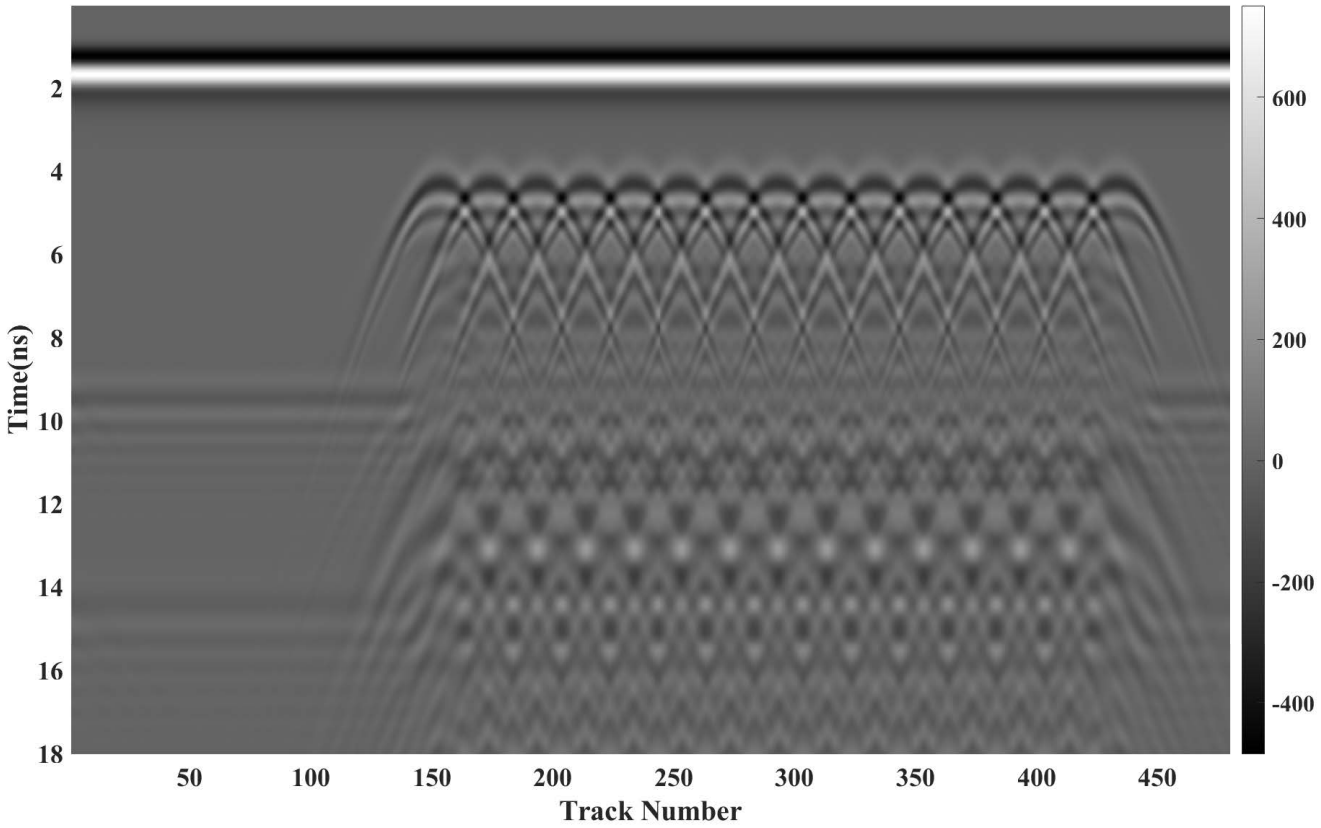

Figure 5. (c)Water-bearing anomaly response record

Figure 5. Forward diagrams of the same depth model

It can be seen from the forward response records of GPR in **Figure 5. (b)** and **Figure 5. (c)** that the water-bearing anomaly is basically the same as the cavity anomaly. The white strips in the figure are direct waves, and the cavities and water-bearing anomalies have hyperbolic reflection, as well as the interfaces between the second lining and the primary support, and between the primary support and the surrounding rock. When the anomaly is located at the same depth and the size is the same, the anomaly area has a strong diffraction effect on the high-frequency electromagnetic wave, and obvious hyperbolas appear at the location of the anomaly, the diffraction hyperbolas generated by the adjacent diffraction points intersect, and high energy points are formed at and below the cavity [20-21]. The position of the upper arc top corresponds to the depth of the anomaly. There is a hyperbola formed by diffraction at the two interfaces. The difference is that below the interface between the primary support and the surrounding rock, the phase of the cavity is more consistent, while the water-bearing anomaly has larger fluctuations and multiple waves are more obvious.

3.2.1.4. Water-bearing anomaly model with different lateral spacing

The model is set as two water-bearing circular anomalies with different horizontal spacing, with a depth of 0.2m and a radius of 0.05m. There are five models in this group, and the horizontal position coordinates and the spacing between the two anomalies are shown in **Table 2.**

Table 2. Coordinates and spacing of water-bearing anomalies

X-coordinate of water- bearing anomaly	X-coordinate of water- bearing anomaly	Spacing (m)
1	4	3
2	3	1
2.3	2.7	0.4
2.4	2.6	0.2
2.45	2.55	0.1

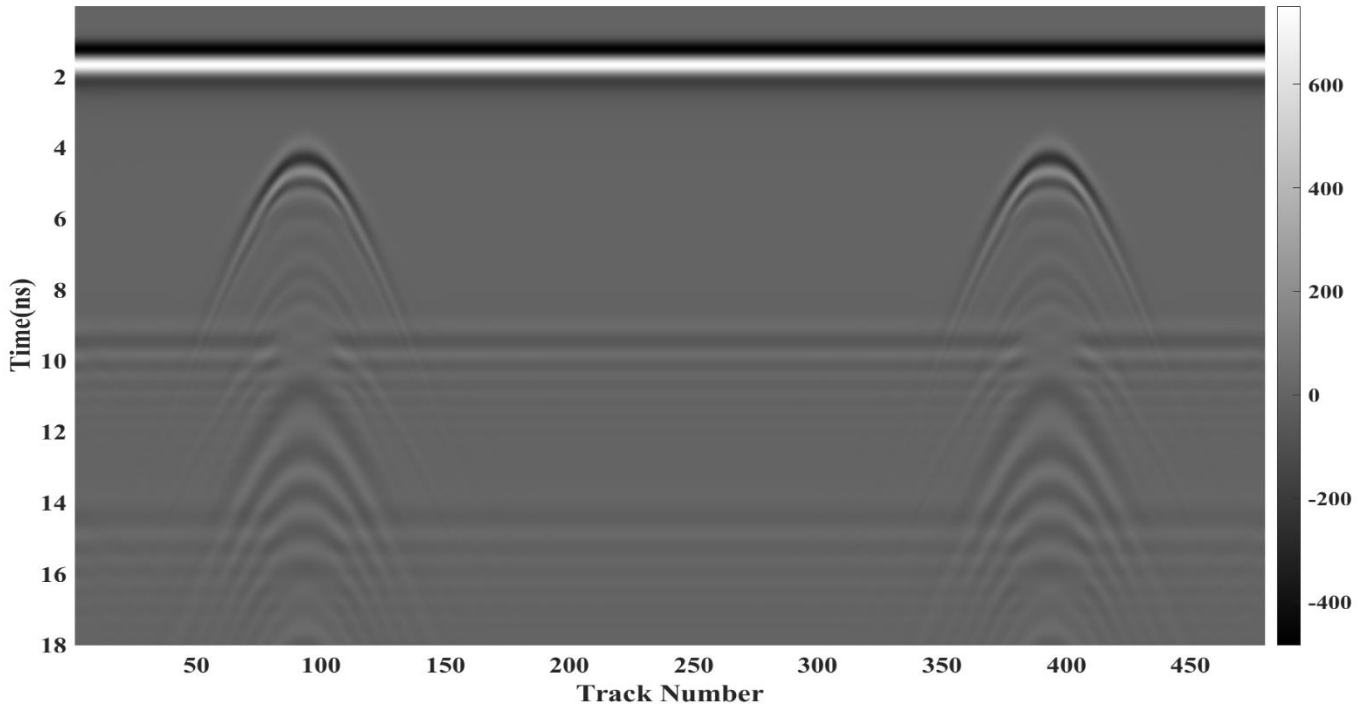

Figure 6. (a) 3m interval

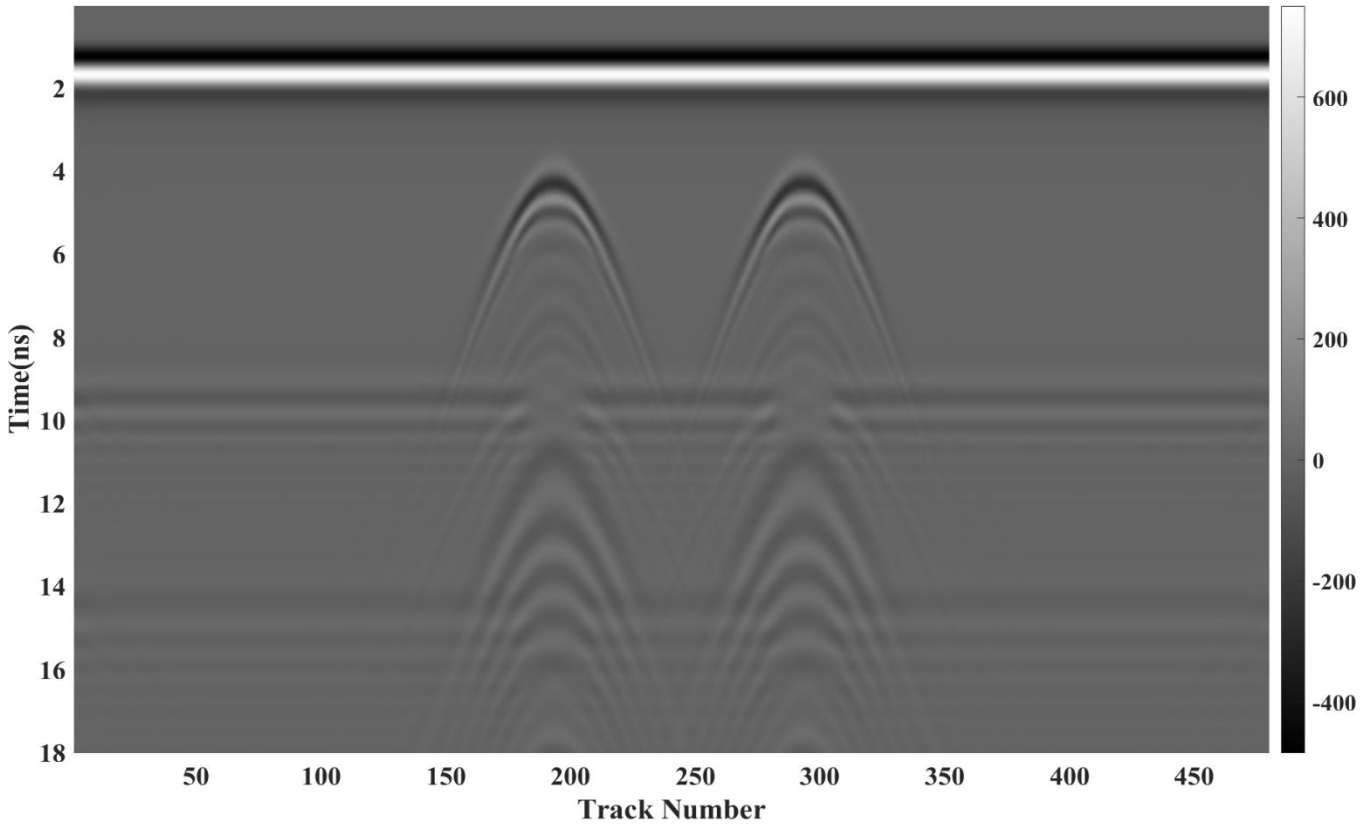

Figure 6. (b) 1m interval

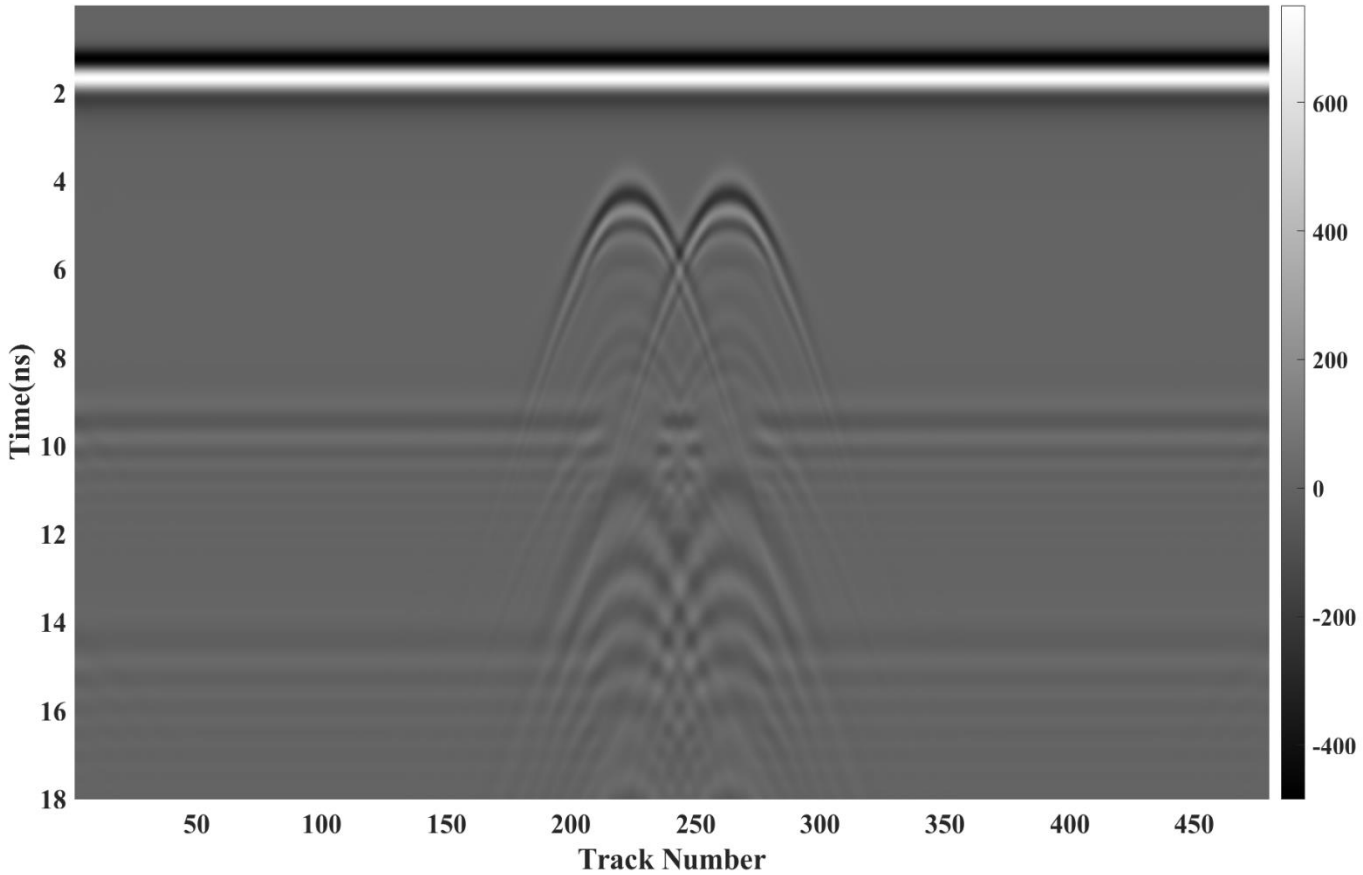

Figure 6. (c) 0.4m interval

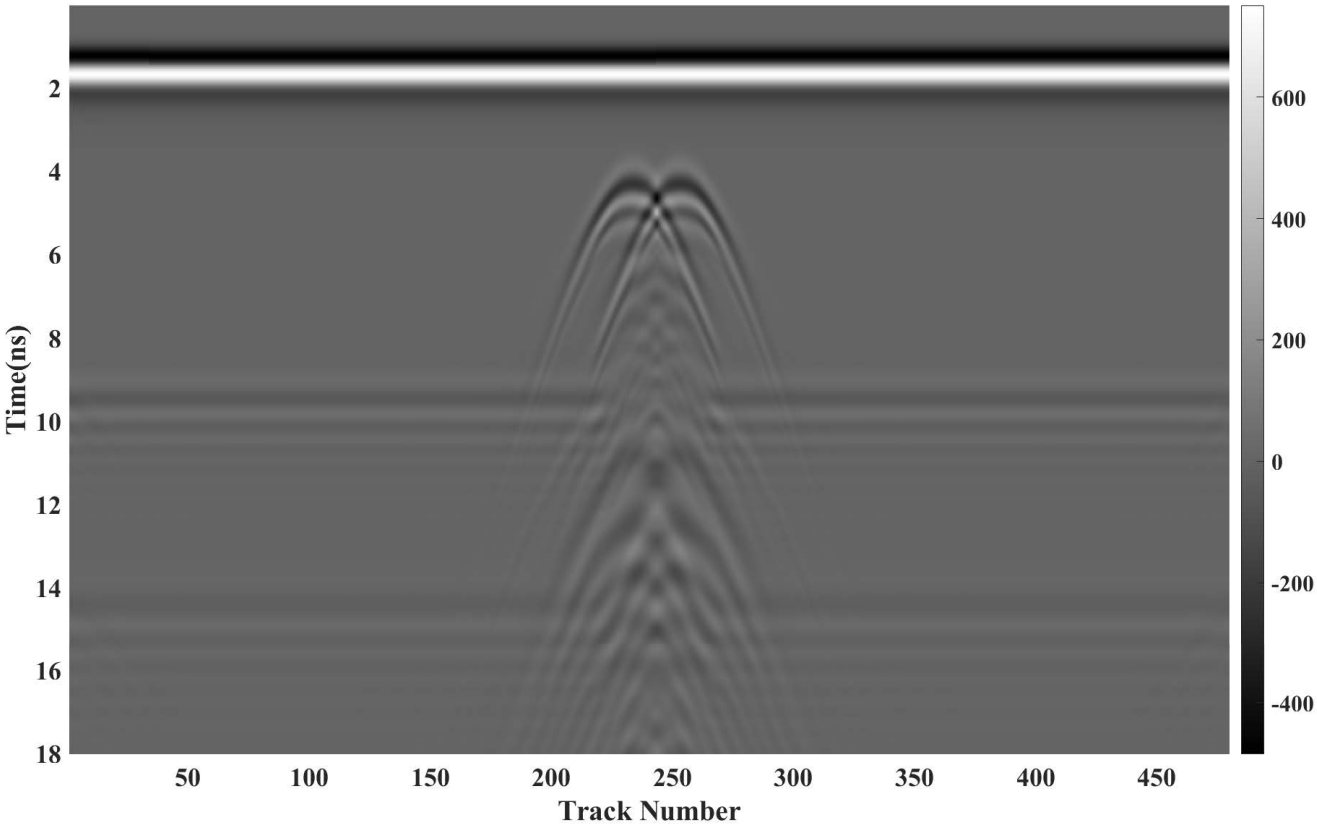

Figure 6. (d) 0.2m interval

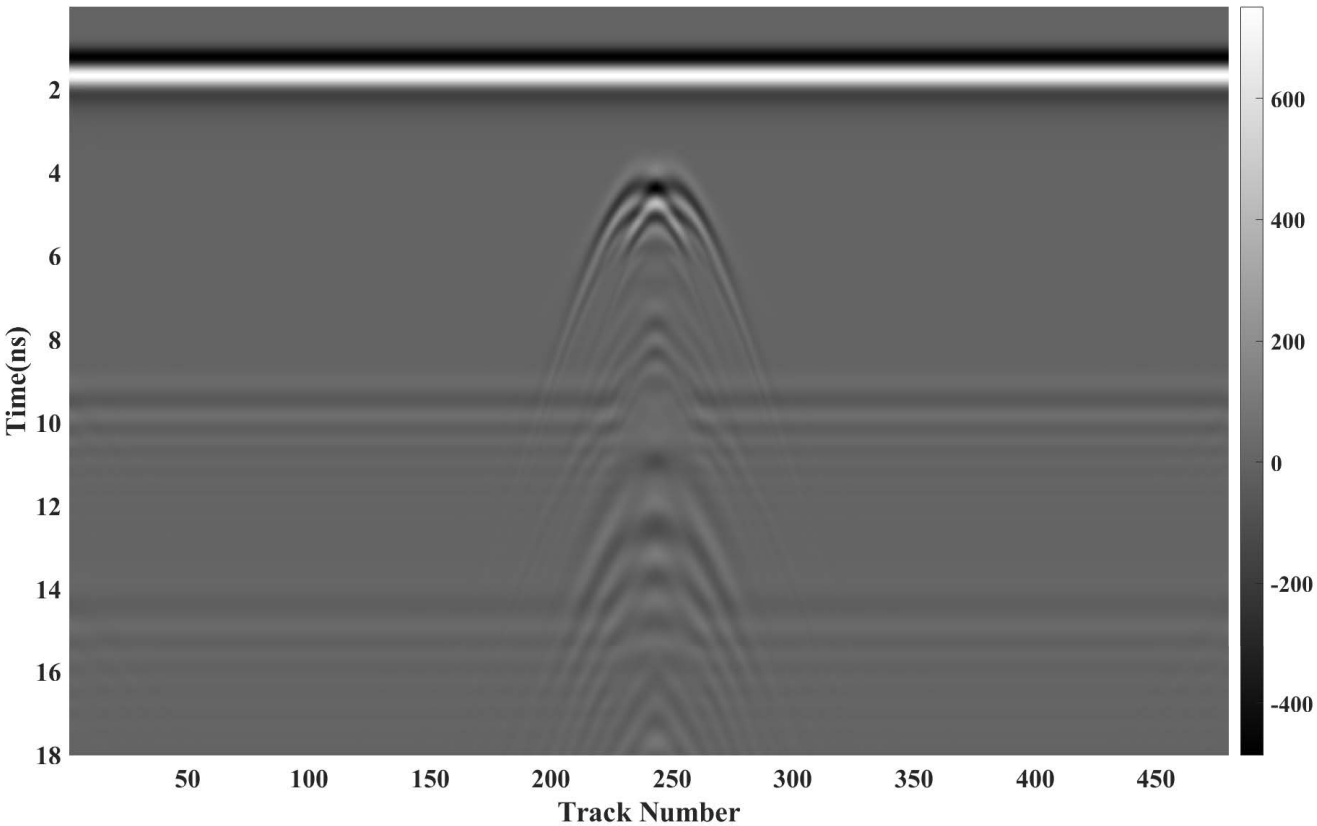

Figure 6. (e) 0.1m interval

Figure 6. Forward diagrams of lateral spacing water-bearing anomaly model

It can be seen from the forward response records of GPR in **Figure 6.** that the white strips in the figure are direct waves, and the water-bearing anomalies at different positions have hyperbolic reflection, as well as the interfaces between the secondary lining and the primary support, and between the primary support and the surrounding rock. When the two anomalies are far apart, the hyperbola is obvious and easy to distinguish, but when the anomaly interval is reduced to 1m, the hyperbola tends to intersect, but it is also easy to distinguish. When the interval is 0.4m, the hyperbola has intersected and can be distinguished, and the two arc tops are easy to distinguish. When the interval is 0.2m, the hyperbola intersects and there is a strong energy point.

3.2.2. Cracks model

The fracture model is set as a rectangle with a thickness of 0.03 and a length of 1 m in the surrounding rock, with air and water inside. Because the anomaly is located in the deep part, the time window size is changed to 28 ns, and the model and record are shown in **Figure 7.**

From the forward response records of GPR in **Figure 7. (b)** and **Figure 7. (c),** it can be seen that the white strips in the figure are direct waves, and the water-bearing anomalies at different positions have hyperbolic reflection, as well as the interfaces between the secondary lining and the primary support, and between the primary support and the surrounding rock. The results show that the electromagnetic wave response characteristics of the cracks are banded, and the approximate time depth is about 16ns. The band filled with water is larger than that filled by air. The band without filling crack is 16-20ns, and the band filled by water lasts to 23ns.

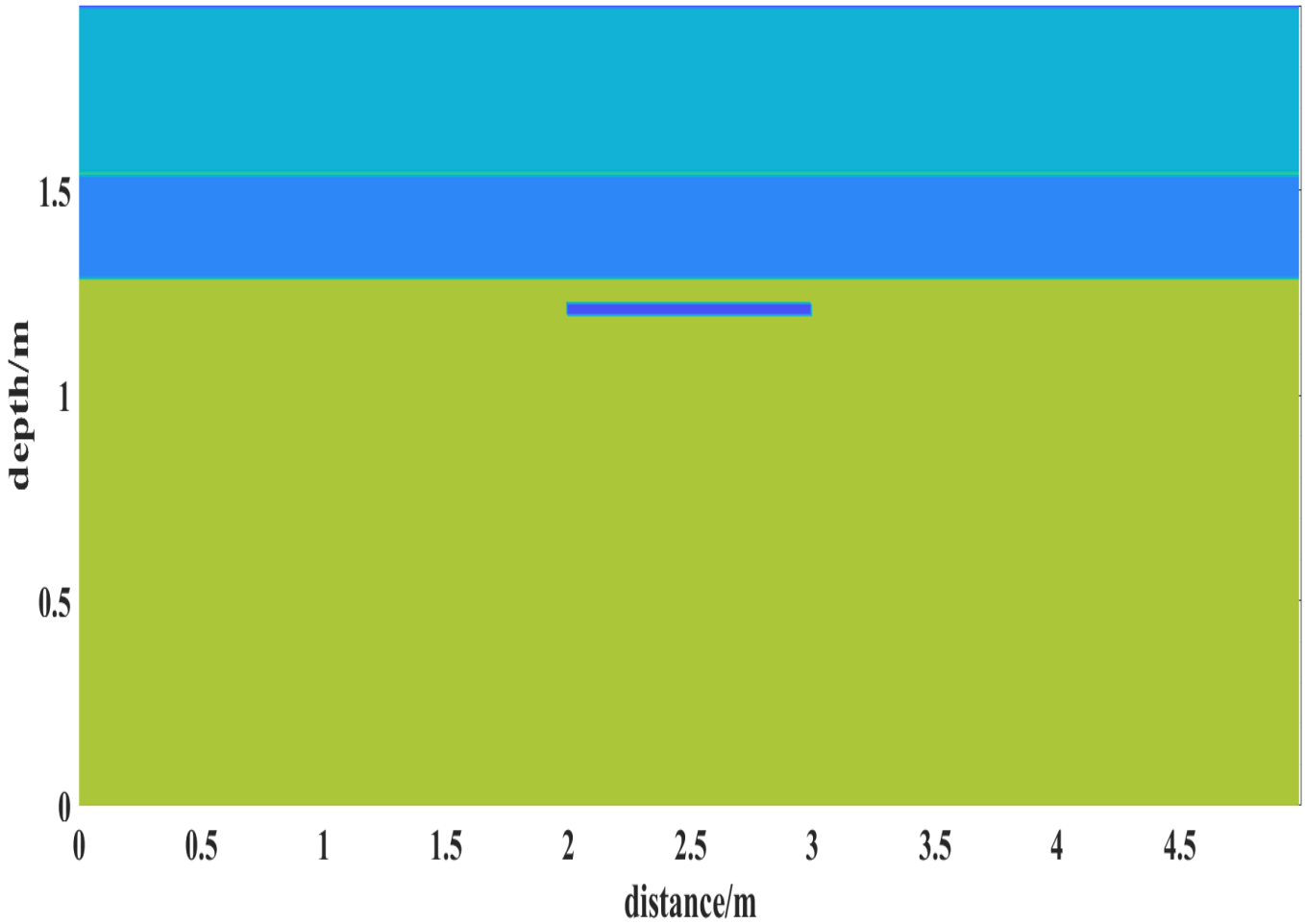

Figure 7. (a) Model

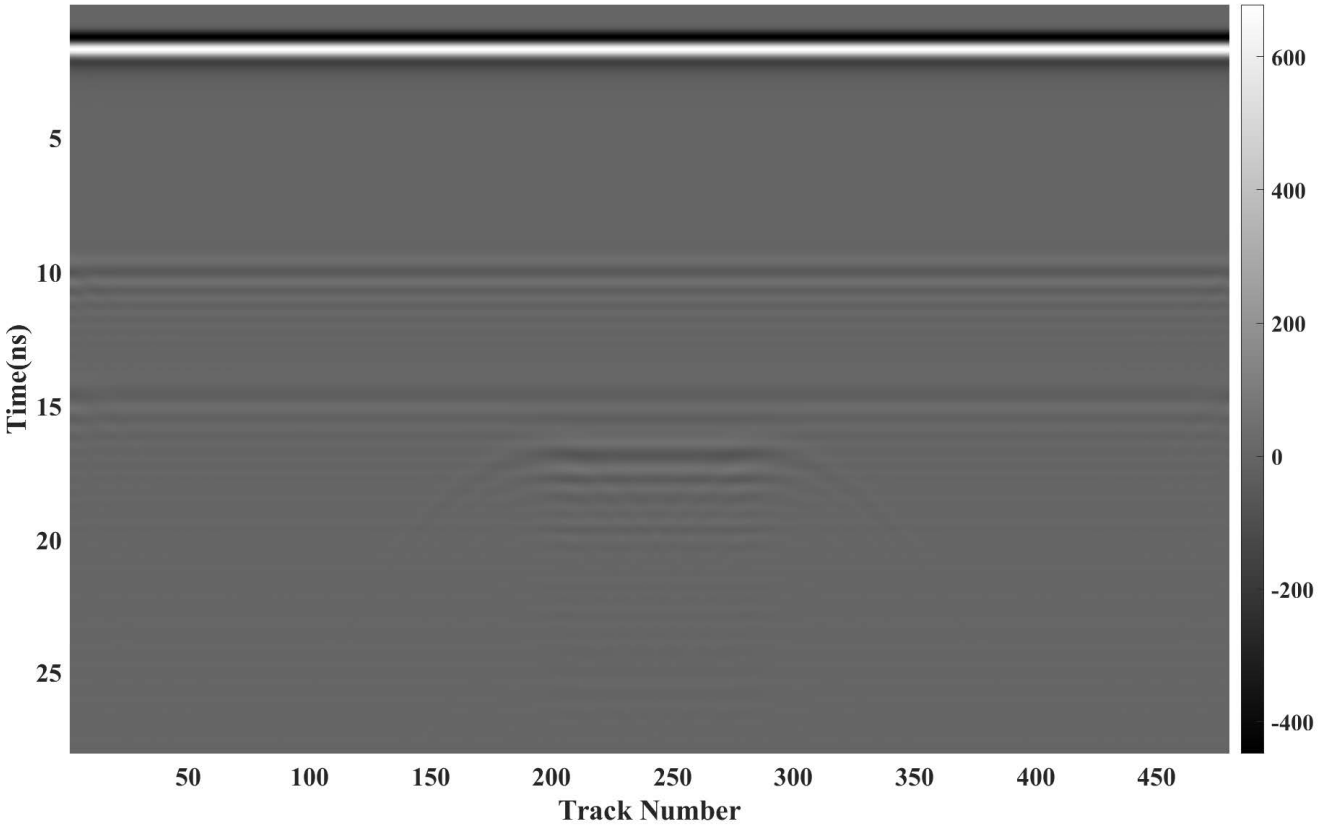

Figure 7. (b) No filling crack model record

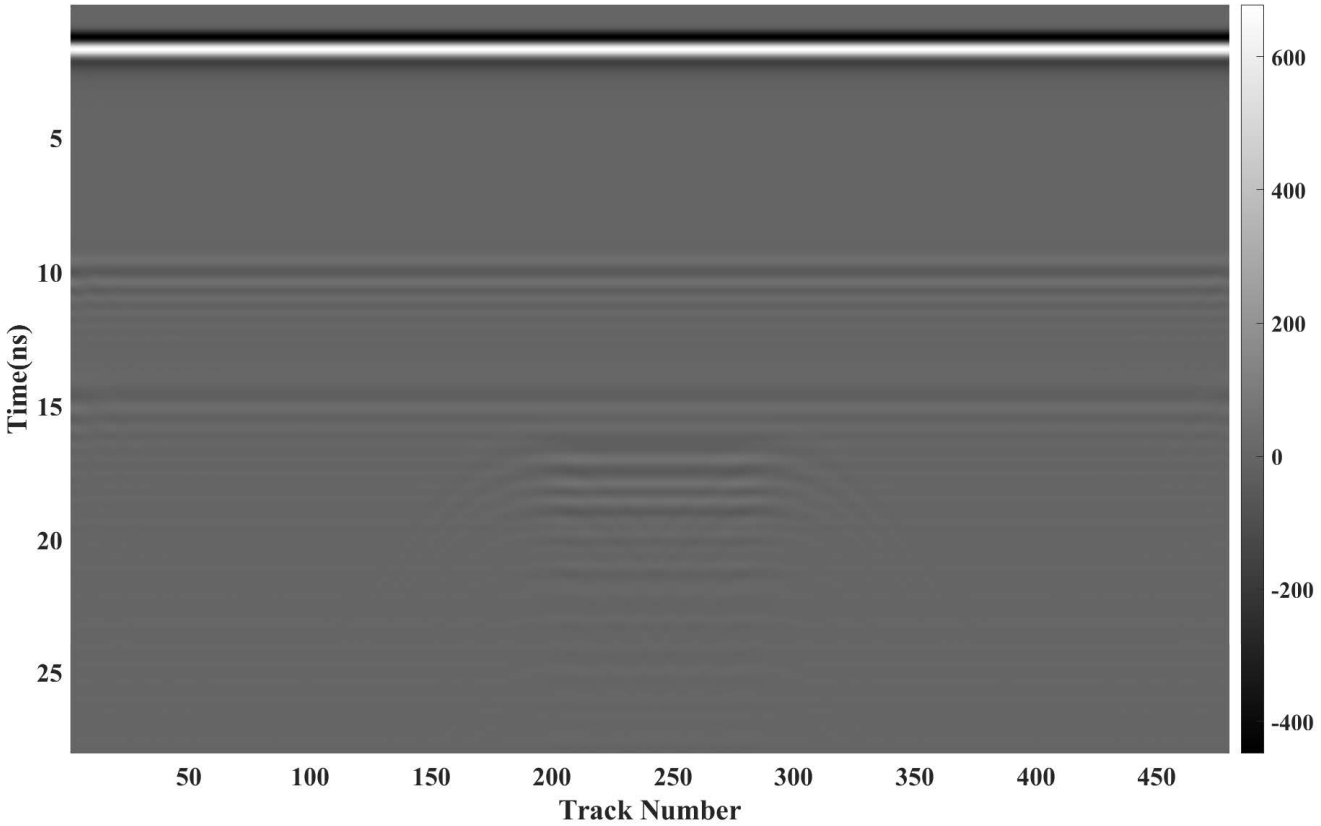

Figure 7. (c) Water-filled crack model record

Figure 7. Forward view of crack model

3.2.3. Composite defect models of cavities and irregular cracks in different media

The model is set as the cavity and water-bearing anomaly in the second lining, and the fracture is filled with air and water. The record is shown in **Figure 8.**

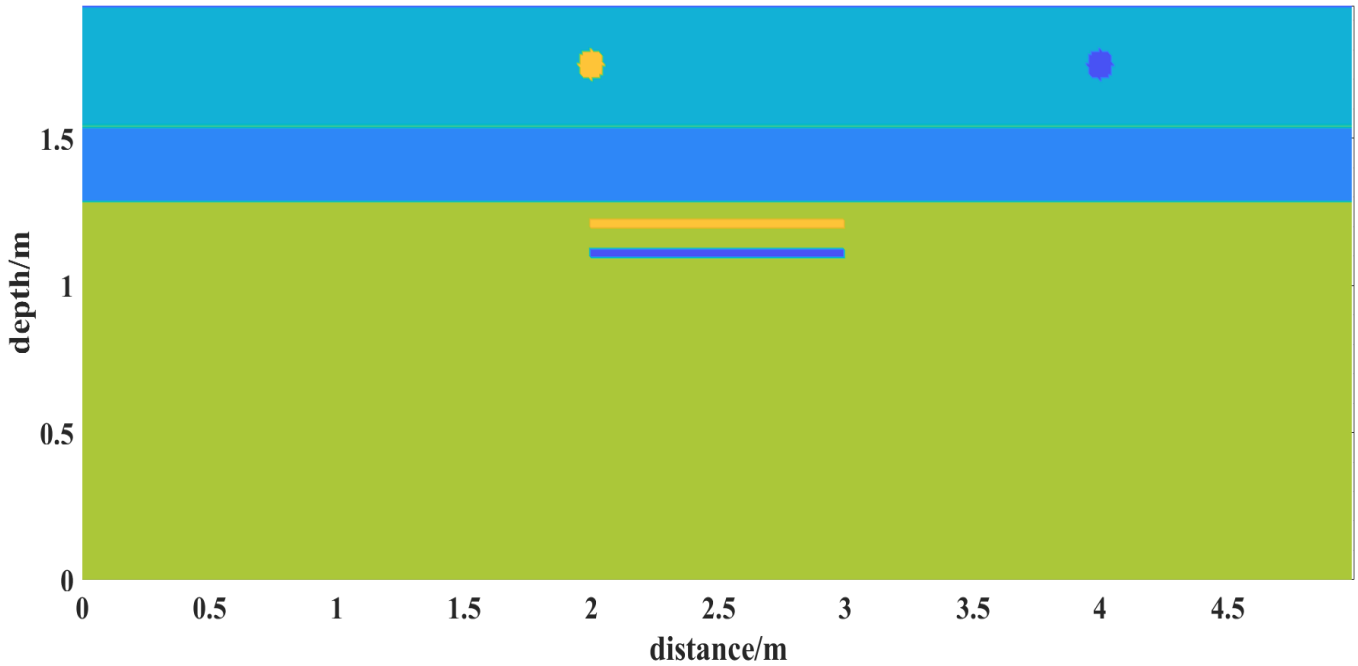

Figure 8. (a) Model

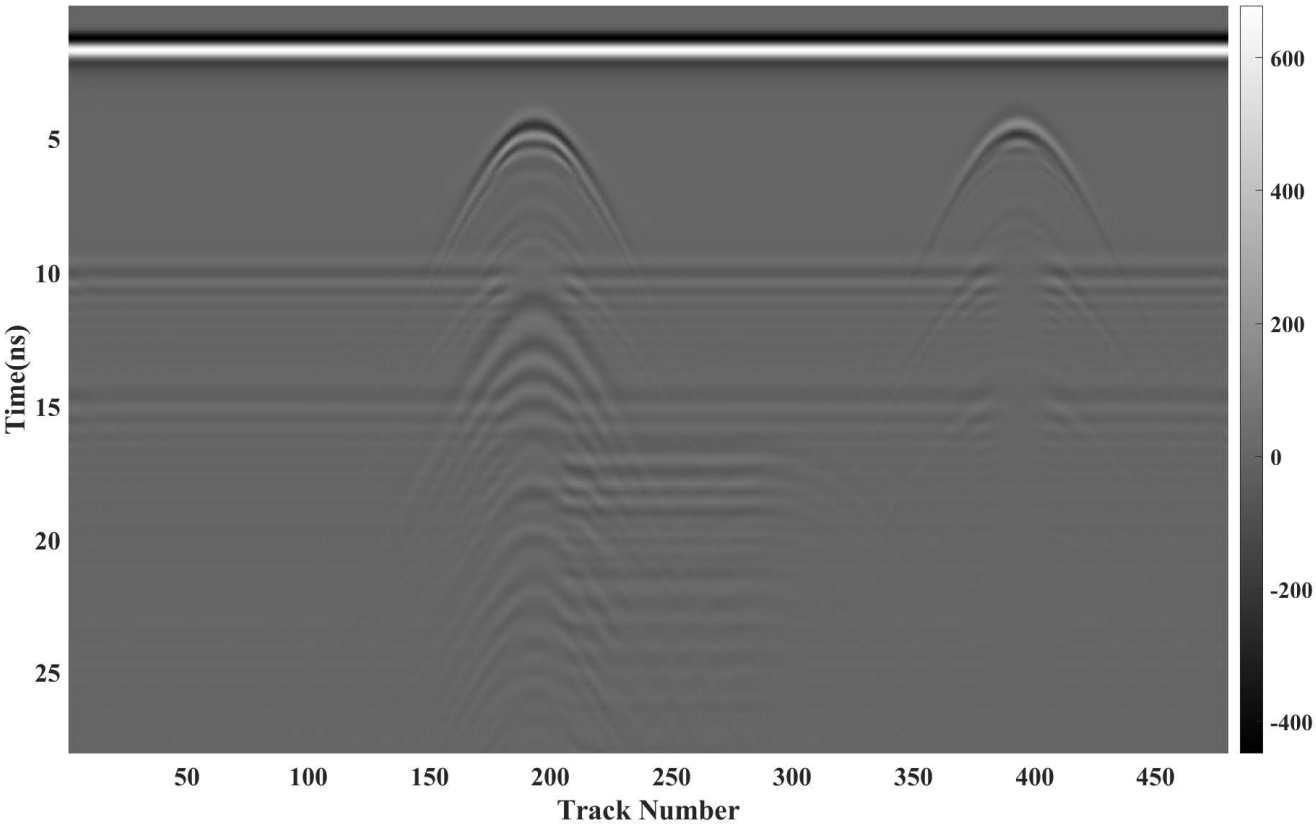

Figure 8. (b) Response record

Figure 8. Forward view of composite model

From the forward response record of the ground penetrating radar in **Figure 8.**, it can be seen that the white bands in the figure are direct waves, and the interface bands between the second lining and the first support, and the first support and the surrounding rock are obvious. There is a hyperbola at positions 10 and 20 at 4ns, and the hyperbola on the left is abundant and obvious, corresponding to the circular water-bearing abnormality. The hyperbola on the right is felt and the phase of the arc crest at the lower part of the interface is 0, only at both ends. The hyperbolic tail branch is the corresponding position of the cavity, and the bands at 10-15 are the response characteristics of the cracks. Because of their mixing, the corresponding positions cannot be distinguished.

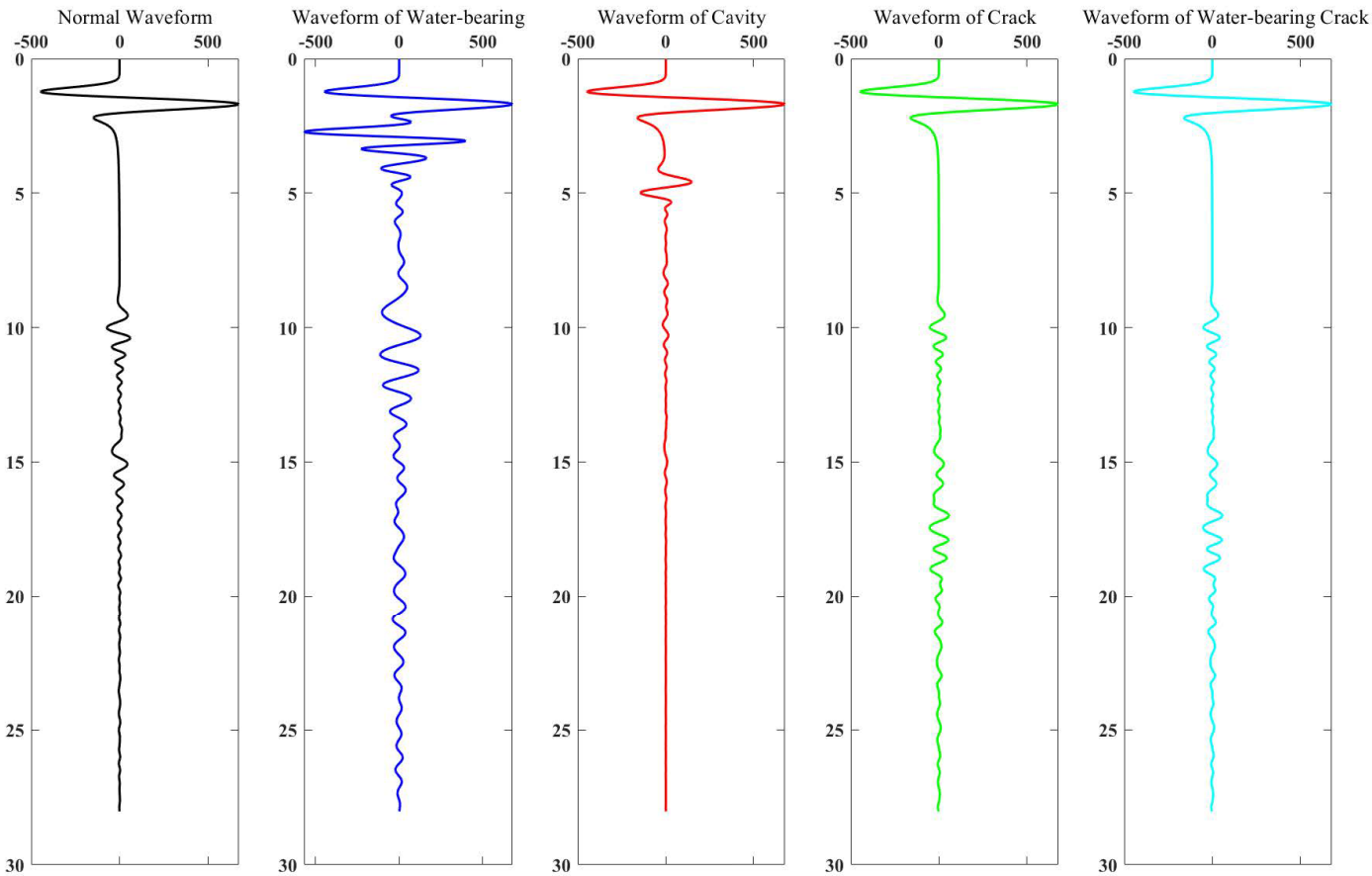

Figure 9. Waveform comparison diagram

It can be seen from the comparison of waveforms in **Figure 9.** that (a) is a normal tunnel, that is, it does not contain any anomalies. Its waveform has a group of peaks and troughs at 2ns, which is a direct wave. At 10ns, it is the interface between the second lining and the primary support, and at 15ns, it is the interface between the primary support and the surrounding rock (b).The figure shows a water-bearing circle in the second lining. The direct wave at 2ns is connected with the water-bearing anomaly at 4ns. The phase value at the interface between the secondary lining and the primary support at 10ns is larger than that in (a). The interface between the primary support and the surrounding rock at 15ns has little influence on the waveform, and the lower part of the waveform also has small amplitude oscillation (c). The figure shows a circular cavity in the second lining. Its waveform has a group of peaks and troughs at 2ns, which is a direct wave. It is easy to distinguish. It can be seen that the cavity does not affect the direct wave and the upper waveform. At 5ns, the cavity is abnormal, but the lower waveform is gentle (d). The waveform of the fracture is similar to that of (a), except that there are peaks and troughs with smaller phase at the crack

location (e). The figure shows the waveform of water-bearing crack, which is the same as that in (d) figure, except that there are small phase peaks and troughs at the crack location.

4. Ground penetrating radar identification standard for grouting defects behind tunnel wall

This paper takes the grouting behind the tunnel wall as the research object, carries out forward simulation, and studies the characteristics of different types of abnormal ground penetrating radar images. Through theoretical analysis, the method and characteristic standard of using ground penetrating radar to detect the tunnel wall and determine the grouting quality are put forward. The radar characteristics of cavities, water-bearing cavities, cracks, and water-bearing cracks are analyzed, and the basic characteristics of ground penetrating radar detection and recognition of grouting behind the tunnel wall are explained. The morphological features of cavities in ground penetrating radar images are usually composed of many hyperbolic strong reflection waves. Based on the ground penetrating radar forward record in Chapter 2, the identification standards for defective ground penetrating radar are summarized as follows in **Table 3.**:

Table 3. Defect identification standard table

Grouting defects	Forward record	Waveform graph
Cavity	Hyperbola, no obvious multiples in the lower part	The peak amplitude is about 200, no influence on the waveform
Water-bearing cavity	Hyperbola with multiple waves in the lower part	The peak amplitude is about 500, which has a greater impact on the waveform
crack	Bands	The peak amplitude is about 100
Water-bearing crack	Bands	The peak amplitude is about 100

According to the characteristics of reflected wave group, phase axis, and waveform of the radar image, the position, and size of interface, cavity, and crack are determined. The direct wave is the wave that directly reaches the receiving antenna along the ground without energy loss, so it is usually the white band with the largest amplitude at the time depth of 2ns; According to the GPR response records, the boundaries between the secondary lining and the primary support, and between the primary support and the surrounding rock are divided into four bands, and their amplitudes are about 100; When the cavity anomaly is located in the second lining, the GPR image is a reflection hyperbola, and the phase at the interface of the corresponding position of the arc top decreases, and there are hyperbolic tail branches on both sides; When there is a water-bearing circular anomaly in the secondary lining, the GPR image is a reflection hyperbola, and the phase at the interface corresponding to the arc top increases, and there is a complete hyperbola at the bottom; When there is a fracture in the surrounding rock, the GPR response record is a short strip with the same length as the fracture, the peak amplitude is about 100, and the duration is 4ns. When there is water in the fracture, the difference between the characteristics and the fracture is that the duration is 8ns.

5. Conclusion

In order to effectively guide the quality detection of tunnel wall grouting based on detection radar, based on the finite difference time domain method, the forward modeling of tunnel wall grouting defects is carried out by using the difference of dielectric constant. Four kinds of abnormal conditions including round cavity,

water-bearing anomaly, rectangular fracture, and water-bearing fracture are simulated.

(1) The forward record of GPR shows that the reflected image of a direct wave is characterized by a white band with strong amplitude; The interface between primary support and secondary lining, primary support and surrounding rock are also banded; The difference is that the phase of the lower part of the radar image of the cavity anomaly is zero, and there are only hyperbolic tail branches on both sides, while the water-bearing anomaly also has obvious hyperbolic features on each interface; The reflection characteristics of rectangular fractures are banded, and the abnormal range of water-bearing fractures in the radar image is larger than that of air. The above abnormal radar reflection record law can lay the foundation for the subsequent explanation.

(2) When two anomalies exist at the same time and the interval is large, the hyperbolas of each anomaly do not affect each other and are easy to identify; however, when the anomaly interval is shortened, the hyperbolas are easy to cross and difficult to distinguish.

Disclosure statement

The author declares no conflict of interest.

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Integrity Design Strategy in Steel Structure Bridge Design

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Abstract: Nowadays, the scope of bridge construction projects in China is becoming wider, which promotes China's economic development to a large extent and also improves China's transportation system. Meanwhile, people also put forward new requirements for the quality of steel structure bridges. However, in actual design, due to the influence of many factors, some problems are inevitable, which will affect the integrity of the design. Therefore, the designer needs to fully grasp the possible design problems, and then take effective measures to improve the integrity of the design scheme, so as to ensure the quality of the steel structure bridge and improves the safety of the steel structure bridge from the fundamentals. This paper mainly focuses on steel structure bridges, analyzed the current status of steel structure bridge types and their selection, and proposes the integrity design strategy of steel structure bridges.

Keywords: Steel structure bridge; Integrity; Design

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1. Introduction

In recent years, the continuous development of China's transportation industry has provided people with convenience in transportation. However, this problem still exists in bridges at this stage. The more significant problem is vehicle overloading, which has certain impact on the safety of bridges. Once the load limit of highway bridges is exceeded, accidents are prone to occur. If a bridge collapses, it will not only cause casualties, but also result in major economic losses. Therefore, higher requirements have been put forward for the design of steel structure bridges, not only to ensure the integrity of the design scheme, but also to increase the load bearing capacity of bridges as much as possible to extend the service life of bridges, achieve better economic benefits from the steel structure bridges and accelerate the development of China's transportation industry.

2. The Current Status of Steel Structure Bridge Types

At this stage, simply supported bridges and continuous bridges has been widely used in China, while steel structure bridges and arch bridges have relatively few applications. There are often shortcomings in the design of steel structure bridges. The main changes are the following:

(1) The problem of impurities. When designing a bridge structure, the selection of materials is very important. When non-metallic materials are used, there are more impurities in them.

(2) Functional problems. When welding steel structures, if the metal crystallization is not handled properly, it will affect the function of the welded part to a large extent, and there will also be corresponding safety hazards.

(3) Damage problems. In the construction of the project, if the welding worker's technical level is low, it is likely to cause welding damage and fatigue cracks.

(4) Details issues. When designing the steel structure of the bridge, due to the complexity of the work, it is necessary to focus on the design of the detailed parts. Failure to consider all aspects will result in the safety hazards of the steel structure.

(5) Environmental issues. Under the influence of environmental factors, the bridge structure will be affected by the external environment. If a serious environmental problem occurs, such as an earthquake, it will directly damage the steel structure of the bridge.

3. Selection of Steel Structure Bridge Types

Under many factors, the damage suffered by different bridge types is quite different. Therefore, when designing highway bridges, it is necessary to select the bridge type reasonably. The following aspects need to be taken notice of: First, Pay close attention to the stiffness and quality of the bridge; secondly, simply supported bridges should be given priority for faults, which can prevent bridges from collapsing; thirdly, arch bridges can be considered if the conditions on both sides of the bridge are more complex, as they have relatively large spans and strong resistance; fourth, on the basis of ensuring the effective connection between the upper and lower structure, the overall balance should also be ensured, and the selected bridge structure should have continuity; fifth, when the steel structure needs designing, the design method of the fuse-type unit needs to be reasonably applied; sixth, under special conditions, if a curved bridge is required, the rigidity of the overall structure should be increased as much as possible at this time, and the connection part and the lower part should be designed specifically .

4. The Integrity Design Strategy of Steel Structure Bridges

4.1. Horizontal Anti-overturning Design

When designing a steel structure bridge, as a designer, you need to fully consider the inclination of the bridge and focus on the anti-overturning design. Especially in the road section with dense vehicles, continuous improvement of the bridge design is required. In order to fully perform the horizontal anti-overturning design work, designers also need to conduct in-depth research on various tasks, conduct a comprehensive inspection of the site, collect relevant data, improve the accuracy of calculation of related parameters, and avoid design errors, which can also render the steel structure bridge with better load bearing capacity and prevent damage to the bridge structure caused by the influence of the layout force. In addition, it is also necessary to reasonably design the stability of the bridge, and the method of sand and gravel filling can be used to improve the force balance.

4.2. Design of welding structure

When designing steel structure bridges, it is necessary to focus on the welded structure in order to ensure the integrity of the design. Due to the continuous development of steel structure, it has been highly valued in the construction industry. In the actual design, it is necessary to pay attention to the structural integrity, and it is also necessary to select compatible materials and technology. According to the actual situation, if the steel structure bridge is damaged, the severity of the damage will continue to expand over time, which will affect the overall safety of the structure. Therefore, the designer must get hold of the details and solve the problem of damage effectively by choosing suitable materials, optimizing the construction process, and fundamentally ensure the integrity of the welded structure. Generally speaking, you can pay attention to the following points: First, it is necessary to ensure the matching of performance in the base materials and welding materials. Second, in the process of welding steel structures, it is necessary to analyze all aspects

of influencing factors. Operators need to focus on the material quality and also need to fulfill the design requirements of the welded structure. Third, strict requirements are put forward for welding quality to ensure that the quality of steel structure bridges can meet the relevant standards and specifications. Welders need to master advanced welding processes to avoid material waste and ensure the overall quality of the bridge structure.

4.3. Stiffening Rib Design

Stiffening ribs refer to a kind of reinforcing item, which can be used in the design of steel structure bridges to improve the stability of components. Generally, U-shaped and plate ribs can be used. In the design of steel structure bridges, stiffening ribs play an important role and designers need to pay close attention to them. Choose suitable stiffening ribs according to the actual conditions of the strip, and also need to accurately calculate relevant data to maximize the accuracy of the data. This allows the design content to be reasonably optimized, thereby ensuring the integrity of the overall design. In addition, when carrying out the designing work, it is also necessary to accurately calculate the internal force of the steel structure bridge. Not only must the appropriate calculation method be applied, but the bridge must also be detailed, divided into several units, and design layer by layer according to the serial number to ensure the integrity of the design.

4.4. Construction Manhole Design

When designing steel structure bridges, it is necessary to get the designing of the manhole construction right, which can provide a good basis for subsequent construction and provide more convenience for construction personnel. As a designer, you need to set the size of the holes on the bridge reasonably to ensure that each hole is allocated at a suitable position. You also need to proofread the manholes design as it is forbidden to have multiple manholes on the same plane, and the arrangement must be staggered. According to past experience, when designing a manhole, it needs to be allocated at a position with a span of 1.5 meters, and it must be allocated at a position with less stress. By reasonably controlling the number and location of manholes, it is helpful to improve the integrity of the steel structure design.

4.5. Anti-corrosion Design

Anti-corrosion design is an important task in steel structure bridges, which directly affects the subsequent use and safety of steel structure bridges. In terms of safety work, it is necessary to get hold of the details. The designer needs to deal with the details of the welded structure and also get the anti-corrosion design right, so that the steel structure bridge can have better anti-corrosion ability, which can extend the service life of the steel structure bridge and realize the economic benefits of bridge projects. When carrying out anti-corrosion design, it is necessary to carry out specific analysis for the environment of the bridge project, select physical or chemical methods for anti-corrosion operations according to the actual situation, and choose suitable anti-corrosion coatings so that steel structure materials can be effectively insulated from air and water, *etc*. to prevent the overall performance of the steel structure from being affected, which not only improves the safety of the bridge structure, but also provided a basis for the service life of the steel structure bridge.

5. Conclusion

In conclusion, with the continuous progress of the times, steel structure bridges have been extensively constructed. For steel structure bridges, in order to achieve economic benefits and promote the development of China's transportation industry, it is necessary to do a good job in design and ensure the integrity of the design, so as to improve the performance of the bridges. When designing steel structure bridges, designers

need to clearly define the design goals. Under the premise of ensuring the service life of the bridge, they also need to increase the load bearing capacity of the bridge as much as possible by selecting suitable materials and optimizing construction technology, *etc*. After analyzing the current status of the steel structure bridge, take effective remedial measures and select a suitable bridge type according to the actual situation of the construction site. These can lay the foundation for the design integrity of the steel structure bridge, so that the construction goal of steel structure bridge can be accomplished efficiently.

Disclosure statement

The author declares no conflict of interest.

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User Experience Survey for Improvement of Australia's University Campus Spatial Safe Design

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Abstract: As the third vital industry supporting Australia's economic development, the education domain and related services are deserved to be emphasized. The domestic university as the major used space of the educational industry is acquiring increasingly government investments. One of the investment aspects is the improvement of safety infrastructure. By providing students and staff a convenient and safe campus experience is an effective approach to produce greater benefits. However, in most Australia universities, the safety issue is still a problem that needs to be enhanced. In this research paper, the Garden City Campus of the Queensland University of Technology is used as the case study. It will be analyzed using the methodologies of observation and survey.

Keywords: Campus safety; Observation and survey; Educational infrastructure

Publication date: July 2021; ***Online publication:*** July 31, 2021

1. Introduction

Recently, Australia's education industry is the major supporting domain for the Australian economy. It is recovering while playing an increasingly vital role in accelerating Australia's economic development and raising its worldwide status [1]. The related educational infrastructure and services contained in this industry are increasingly worthy to be investigated. One of the evaluated aspects is spatial safety design. Particularly in the university, the safety level impacting the users' sensations, behaviors, and decisions, further impacts the education quality, retention rate, and enrollment situation [2]. Appraising the users' engagement situation in the site is an effective approach to explicate the importance of the safe design for education industry development [3]. This research paper selects a typical Australian campus, the Gardens Point Campus of the Queensland University of Technology. Using methodologies of systematic observation to estimate its physical spatial, investigates and analyzes its spatial safe situation, and explores the benefits and drawbacks of the currently spatial safety. Crime Prevention Through Environmental Design (CPTED) is the main theoretical concepts applied in the analysis process.

2. Background

2.1. University campus safety

The crime and violence issues in university campus in the worldwide realm has been appeared for a few decades and particularly in the latter of the last century, the most country including Australia, the crime appears a significant increase [5]. Based on the past of case study about serial crimes and attacks happened in the Australian university campus, until nowadays, the reason that most university campus still exist the safety hazard is that the spatial design of these universities campus lacks the essential protection, prevention and refuges functions to protect users from the threat of unlawful activities [6].

2.2. Relative Theories

Safety theories can help designers to assess site safety comprehensively. Crime Prevention Through Environmental Design is applied in this case. Crime Prevention Through Environmental Design (CPTED) experts from different disciplines, including architects and landscape architects, start to recognize the possibly criminogenic characteristics of the environment [7], for its intended purpose. It serves as an extra expression of ownership. This study endeavor to investigate the validity of theoretical, Crime prevention through environmental design Strategies that to what extent it might be adopted for the design process, and its effectiveness.

3. Methodologies

3.1. Outline of Methodologies Application

Initially, the concentrated aspects of the systemic observation are user behaviors, the use of the phenomenon, and their interactions related to the phenomenon. The researcher will collect related data without any direct involvement with the participants. At the stage of the observation research of this project, observers are required to attend to the target site and record relative phenomena to collect primary sources for the next analyzing stage. The whole progress of the investigation spends five weeks. The observation and survey period plan to spend two weeks and three weeks respectively. In the first two weeks, the researcher will visit the target site every day and record visitors' number and behaviors of the constant spots. The daily visiting time is 8 o'clock in the morning, 12 o'clock in the noon, and 5 o'clock in the early evening.

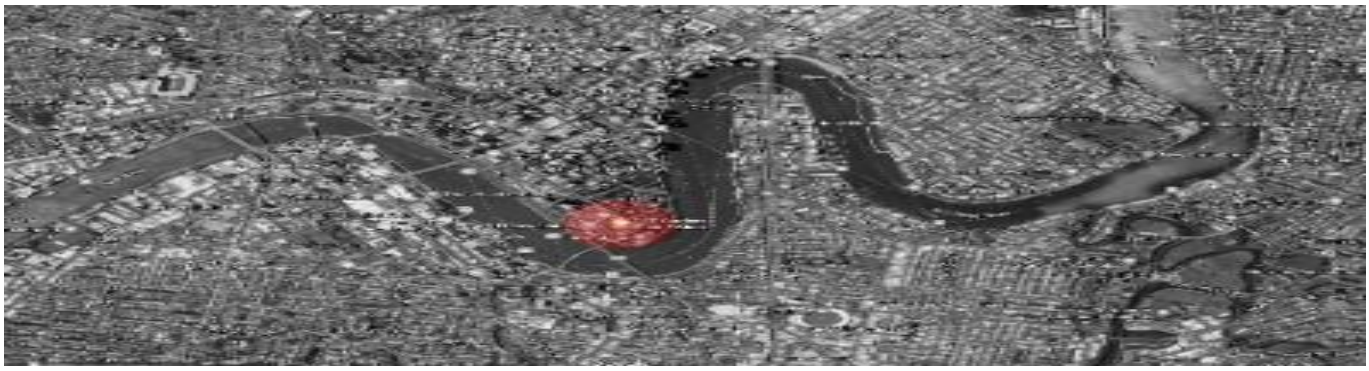

Figure 1. Map for QUT GP campus location, Brisbane, Australia [4]

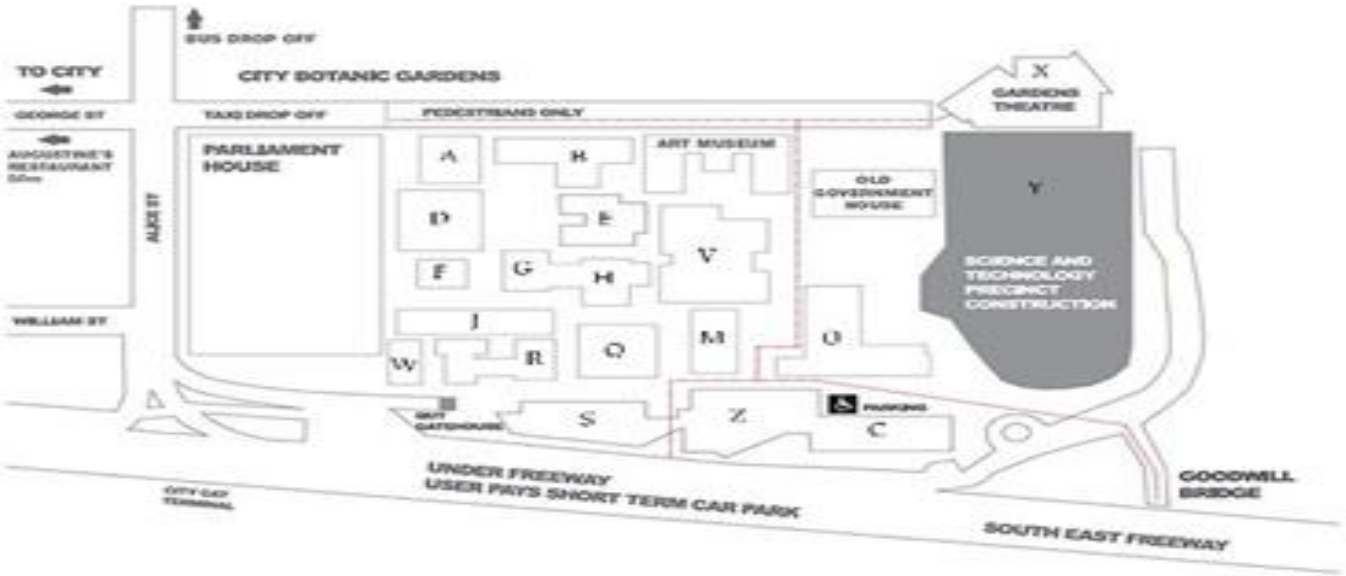

Figure 2. Map for campus observation recording base plan [4]

3.2. Outcomes of observation methodologies

Based on the influenced factors of the safety of space, the campus is divided into three aspects to systematically observe, including the use of quality, legibility and territoriality.

3.2.1. Spatial Use - Quality

The Space Use Quality means the frequency of users' access or stay, and their interaction level with the sites. There are three main types of districts to observe, including campus pathway, buildings access, and campus open space.

Firstly, according to the observation for campus routes, the high use of quality pathways is shown in **Figure 3.** They include the main drive road from the campus entrance, the wide pathways connecting to the main construction, green space and main vehicle-pedestrian shared road (**Figure 3.**). On the other hand, the pathways with low use quality include part secondary routes which are hidden and isolated from the main buildings, such as the V block which is the campus library (**Figure 4.**). The situation of use quality has a little change in three different time period.

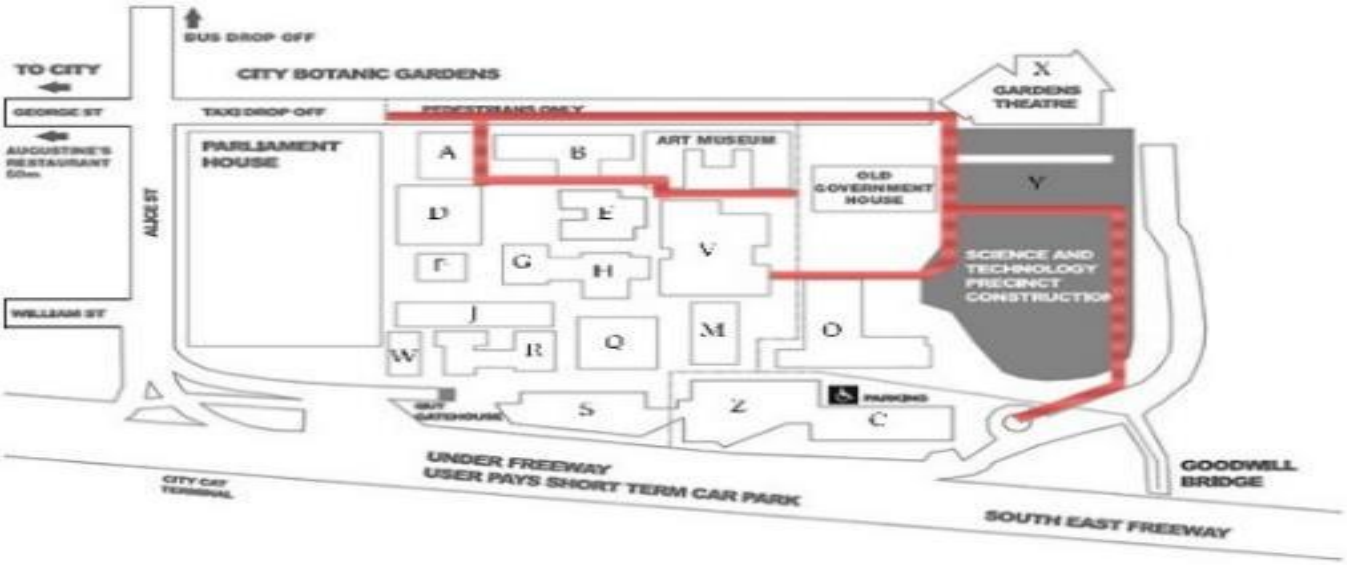

Figure 3. Mapping for high use quality pathway plan [4]

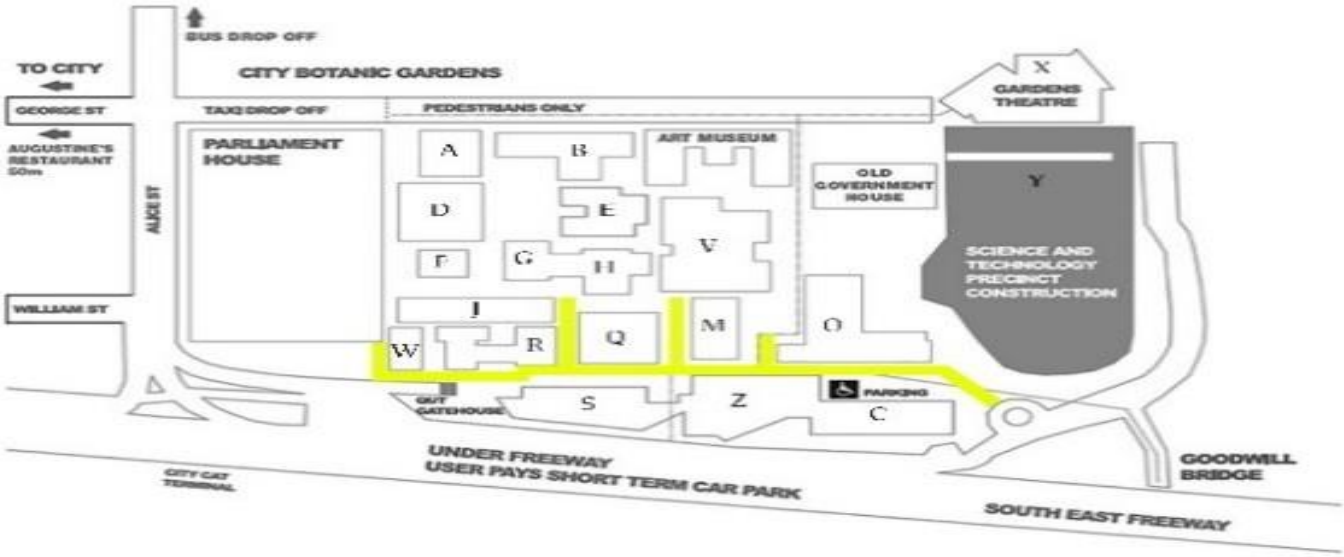

Figure 4. Mapping for low use quality pathway plan [4]

Secondly, building accesses with high use quality are displayed in **Figure 5.** It shows that the buildings locate in the central districts have high accessibility. For instance, the V block, the campus library. However, such as the W, R, O, and C block, which buildings location is at the marginal districts of the entire campus have lower use quality (**Figure 5.**).

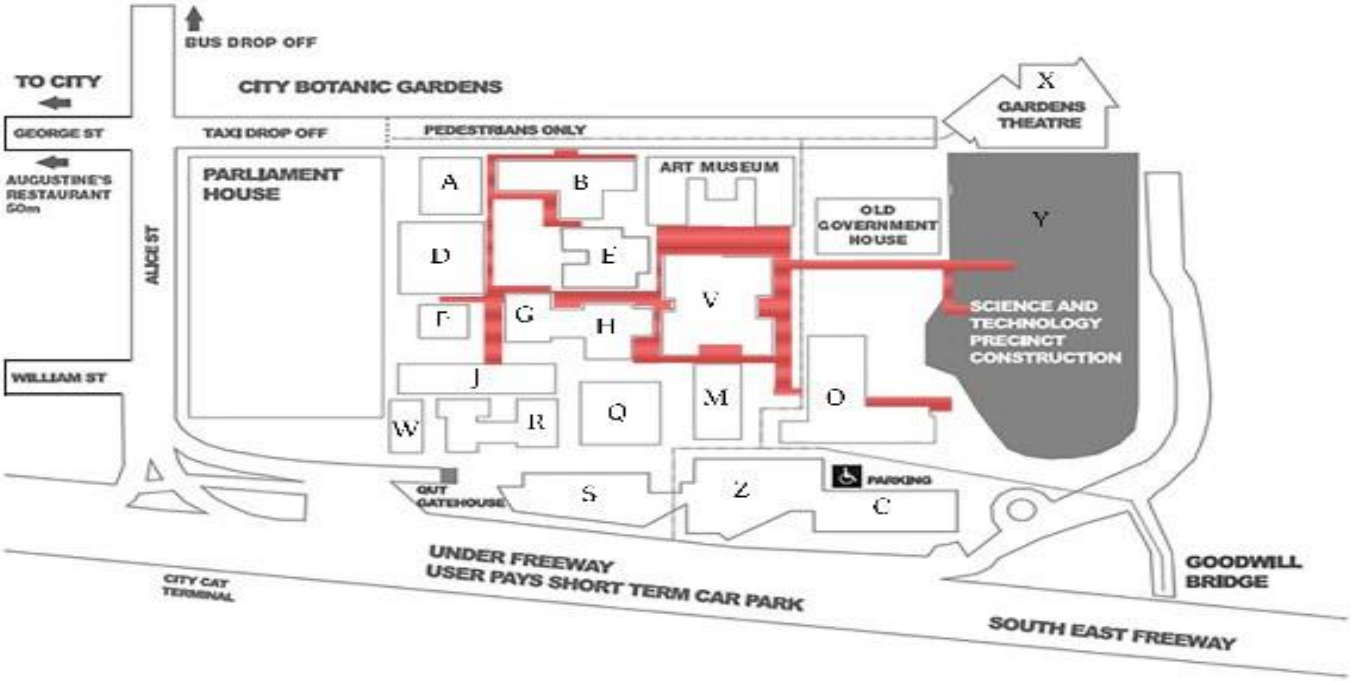

Figure 5. Mapping for high use quality building access plan [4]

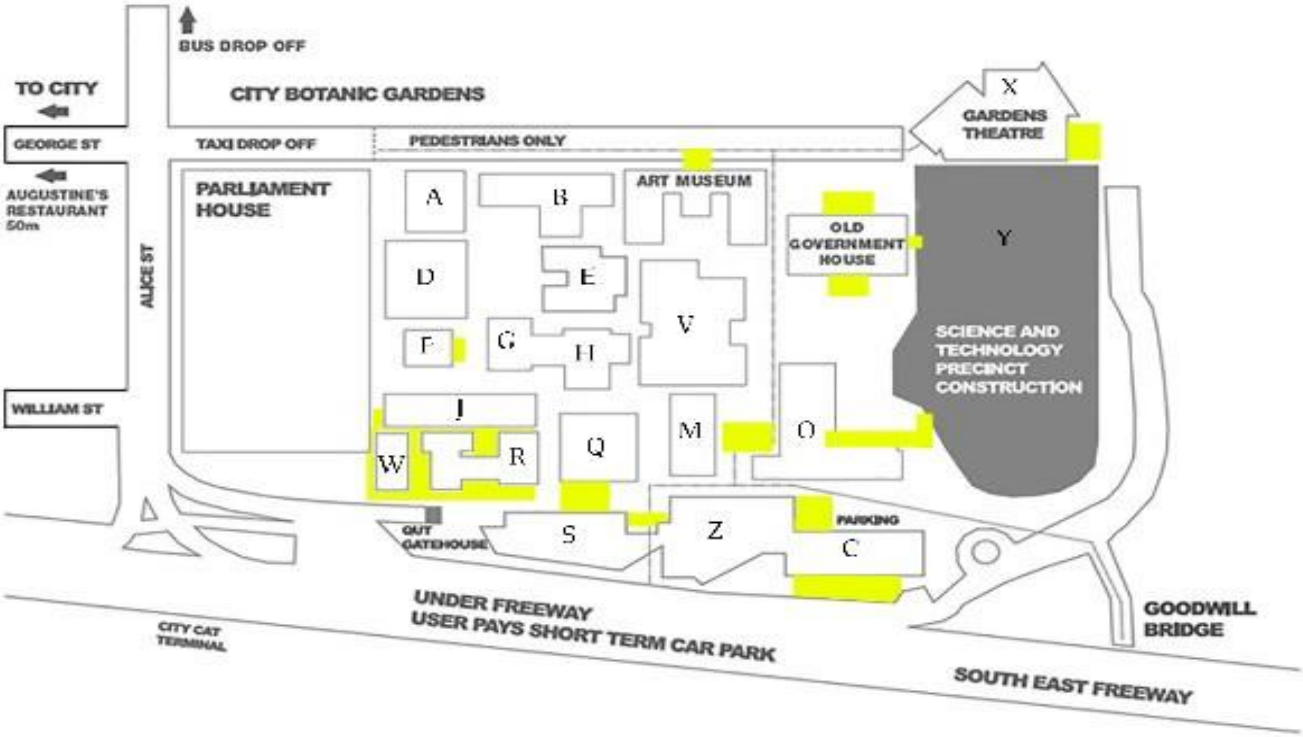

Figure 6. Mapping for low use quality building access plan [4]

The third aspect, campus open space with high use quality is illustrated in **Figure 7.** Most areas exist the features, including the settlement of benches, near sufficient shelters, and provide peaceful open views. For example, the courtyard between the D and E block. In contrast, most open space with low quality is hidden by buildings or flora groups, lack the light and infrastructure, such as the yard in front of the block W and the corner of the block H (**Figure 8.**).

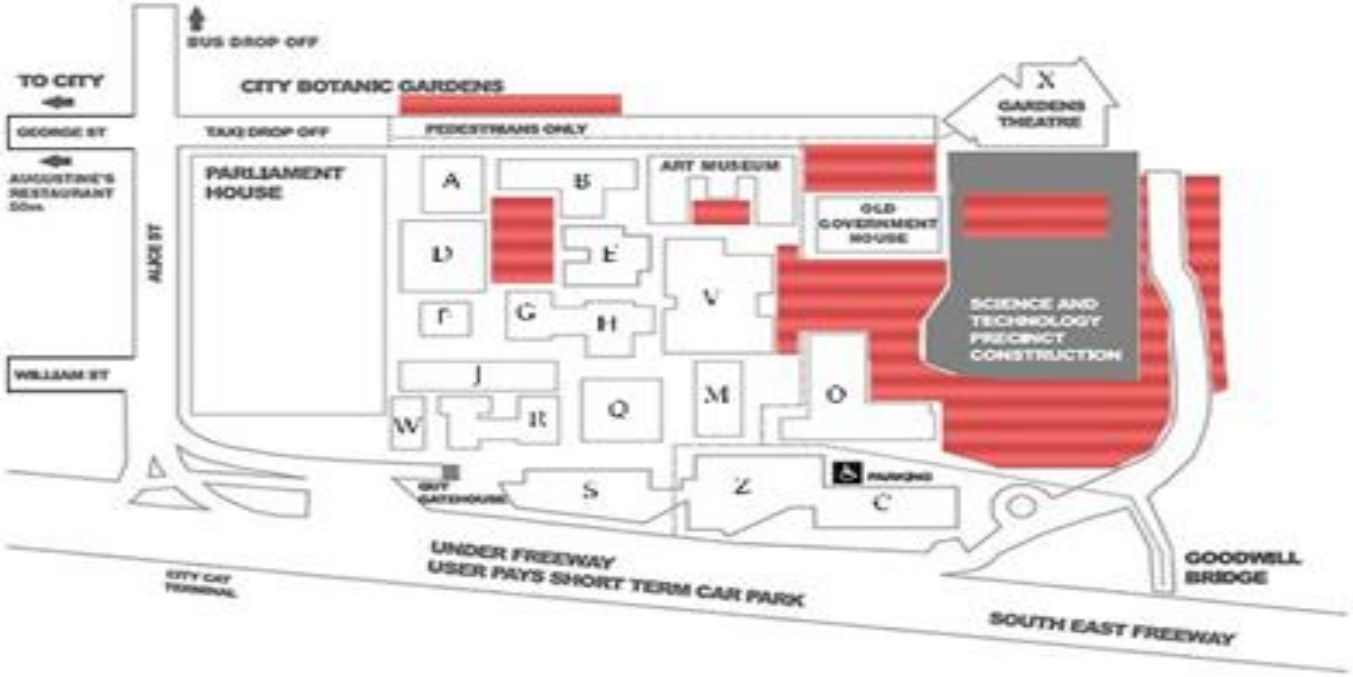

Figure 7. Mapping for high use quality open space plan [4]

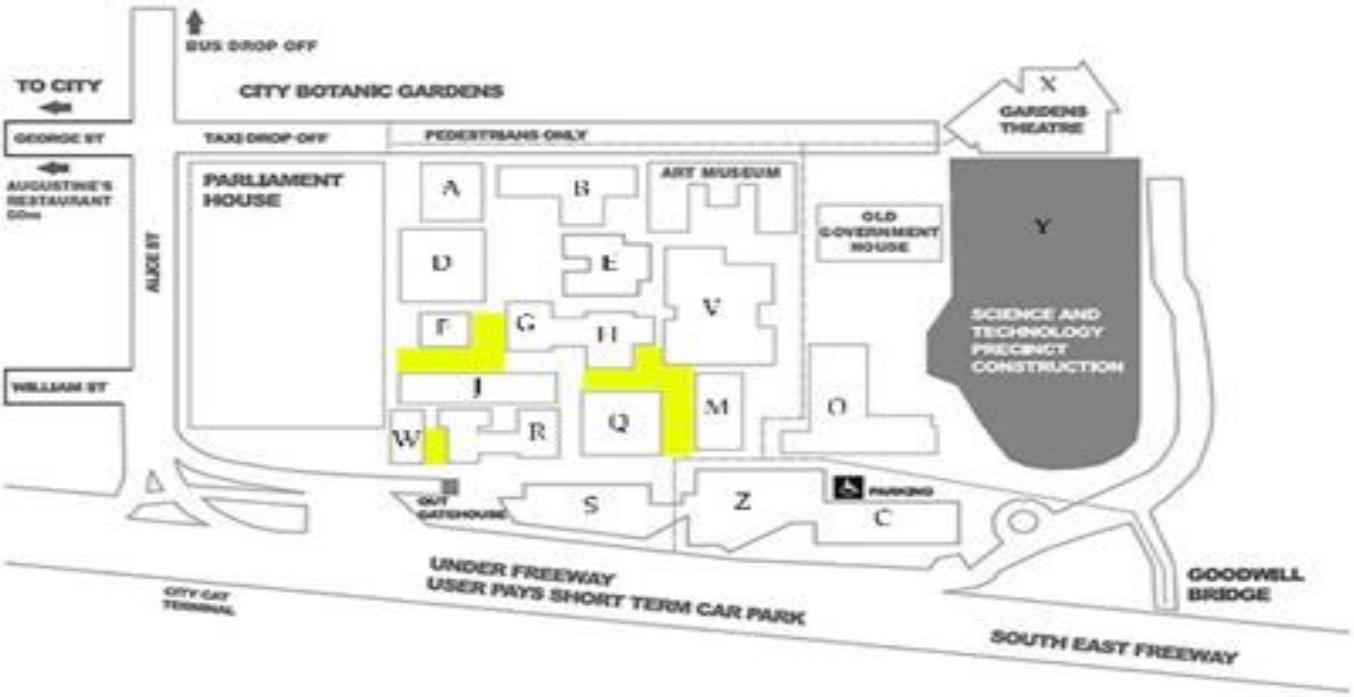

Figure 8. Mapping for low use quality open space plan [4]

3.2.2. Place Legibility

Place legibility is not just to enhance the users' experience, but to increase the place safety. As the

observation for legibility includes three aspects, signage, landmarks and differentiated edges. Firstly, in this campus, there is sufficient and effective signage which can increase the legibility of the site and assist users to find their ways, even cater their needs to seeking for urgent services. For example, most outdoors and indoors signage is at noticeable locations, settling at each intersection and both roadsides. However, some of them lack clear arrows to point the orientation which makes visitors confused (**Figure 9.**)

Figure 9. Campus signage [4]

The second aspect which is beneficial for improving the campus place legibility is the landmark. According to the outcomes of the site observation, there are three main types of landmarks settled on the campus, including art sculpture, graffiti wall and landscape installations (**Figure 10.**). They play a vital role in improving the intellectual legibility of the target place.

Figure 10. Campus landmarks [4]

Thirdly, in the campus, the application of the masonry curb effectively separated different districts, such as the recreation zone and planting-protection zones. Users are easy to recognize safe and unsafe areas (**Figure 11.**).

Figure 11. Campus edges with clear view [4]

3.2.3. Campus Territoriality

There are three main districts which are contracted by the commercial companies, campus food court and two café shops. Each district shows typical identified features of ownership (**Figure 12., Figure13.**). The specific expression is setting obvious logo boards and its public courtyards. It is possible that using the visualization symbols to declare territoriality and ownership of the space, can strengthen the user's trusts and safe sense to this site.

Figure 12. Campus indoor café shop with clear logo to show territoriality [4]

Figure 13. Campus food court with specific territoriality features [4]

4. Findings

Through applying observation and survey methodologies to investigate the Queensland University of Technology, Gardens Point campus, there are three main benefits and four main drawbacks.

The first advantage is that most pathways, building access, and open space in this campus have open and wide views which provide users with a high quality of safety for experiencing. In addition, high legibility with guiding elements, path layouts, landmarks, and differentiated regions are helpful for users to find their way and in the third aspect of the security system, security personnel and camera also support to increase the safety of the site. The fourth one is that the existing territorial zones have high-quality services to provide users with a safer experience.

Nevertheless, four shortcomings also need to pay attention to. The first one is the insufficient lights for some pathways, in the night. It is possible to cause users' insecure sentiments. The second one is the design for territoriality. Some of their design exist drawbacks that cannot provide the users' enough sense of safety. The third one is the identified signage for territoriality. Partly territorial marks which might be unclear for visitor's threat user's safety. Last one, the security system on the campus should be reconnected with the users.

5. Recommendation

Based on the findings from data analysis, there are three design recommendations for the potential problems. They are:

(1) Increasing the numbers of campus lighting, including but not limits the streetlamps and decorative lights, while placing light sources on dark areas to play as the bright signage for the visitors passing the pathways in the night.

(2) Setting territorial signage in more noticeable places and using repeatable marks or color on their infrastructure, can strengthen the users' perceptions for territoriality.

(3) Decreasing the number of blind spots, setting more cameras to monitor and arranging more security personnel to patrol should be useful for keeping the constant watching for hidden space and avoiding accidents happening.

6. Conclusion

In conclusion, based on the background that the increasing crime rate in Australian university campuses, this research selects one typical domestic university as the target site to explore the application of Crime prevention through environmental design principles in the real world. According to the outcomes of the observation, the advantages in terms of use quality, legibility, territoriality, and the security system, demonstrate the improvement of safety. They are sufficiently effective and high-use quality campus space, high legibility for way-findings, and the improving campus security system. At the same time, the relevant drawbacks also display the possible threats for the users which also need to be enhanced and improved in the future, including insufficient lighting, unclear marks for the territoriality zones, as well as the lack of interactions between the campus security system information and the users. Relevant recommendations, including the improvement of campus lighting, territorial marks, and delivery efficiency of security system information to the users, will be beneficial to reduce the Australian university campus criminal rates in the future.

Disclosure statement

The author declares no conflict of interest.

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Influence of Collapsible Loess on Foundation and its Treatment Strategy

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Abstract: The properties of collapsible loess are complex. The self-gravity of overlying soil, self-gravity stress and additional stress act together, which will damage the soil structure and lead to the deformation of the soil structure. Collapsible loess is widely distributed in Northwest and Northeast China. A series of problems caused by its structural characteristics will affect the quality of foundation construction. Therefore, construction enterprises need to deeply study the foundation treatment measures of collapsible loess, so as to avoid the uneven settlement after the construction of collapsible yellow soil foundation. This paper analyzes from the judgment and classification of collapsible loess, studies the impact of collapsible loess on building foundation construction, and explores the specific construction treatment measures of collapsible loess, in order to promote the effective application of foundation construction.

Keywords: Collapsible loess; Foundation construction; Processing strategy; Constructional engineering

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1. Introduction

Collapsible loess is a kind of soil with special properties. When not subjected to the wetting action of water, the collapsible loess has higher strength and smaller compressibility, which can meet the requirements of construction. However, if the collapsible loess is soaked by water, the structure of the soil will suffer serious damage, resulting in the rapid reduction of the strength of the soil, it will inevitably cause a certain impact on the foundation construction of the construction project. Considering the construction characteristics of collapsible loess, construction enterprises should be very careful when carrying out project construction in the collapsible loess area. According to the relevant regulations, comprehensive treatment measures should be taken for the foundation construction to prevent the safety of the building from falling due to the water intrusion. Based on the characteristics of collapsible loess, construction enterprises can study the construction measures to deal with the structural change of collapsible loess in detail, so as to provide safety guarantee for the subsequent operation of buildings.

2. Judgment and classification of collapsible loess

From a geological point of view, the classification of collapsible loess is based on the classification standard of the collapsible coefficient. The symbol of subsidence coefficient δ_s refers to the situation that the collapsible loess layer of unit thickness will collapse under the action of corresponding pressure after being invaded by water. Technicians express it in numerical way, which is the symbol of subsidence coefficient. When technical personnel carry out loess collapsibility detection, if the collapsibility coefficient is greater than 0.015, it can be judged that the soil in the region belongs to collapsible loess. Collapsible loess can be detailed divided self- weight collapsing loess and non self-weight collapsing loess. The collapsible value of self-weight collapsible loess is more than 0.07m, and the measured value of non self-weight collapsible

loess is less than 0.07m. If the construction project cannot keep away from the collapsible loess area, the foundation must be built in the collapsible loess area to complete the construction task, then the technicians should also select the non self-weight collapsing loess area as the location of foundation construction as far as possible, which can relatively optimize the foundation construction quality of buildings. At present, relevant departments in China have formulated the construction code on collapsible loess, that is, the content of "Building Code for Collapsible Loess Areas" (GB50025-2004), in which, the collapsible loess collapsible grade is divided into three grades, the collapsibility of the first stage is in the range of 0.05-0.15; The collapsibility of the second stage is in the range of 0.15-0.35; the collapsibility of the third stage is in greater than 0.35. According to the requirements in the document "Building Code for Collapsible Loess Areas," construction enterprises can accurately determine the level of collapsible loess according to the soil quality measurement in the construction area, and then formulate scientific countermeasures [1].

3. Influence of collapsible loess on foundation

In the collapsible loess paper construction foundation, the main influence is the stability of structure and foundation stability. Because the structural characteristics of collapsible loess are very obvious, belonging to the water physical properties of the cementing material, mainly composed by powder, and the surface of the powder attached to the material, the structural framework of collapsible loess is not stable. Or because of gravity factors, friction factors and other conditions, the larger grain size of the sand in the collapsible loess will continue to rise. After the erosion of water, collapsible loess intermediate workers of the soluble material will gradually soften until finally dissolved, the original stable structure will be destroyed, collapsible loess itself gravity and external pressure will form a joint action, resulting in collapsible loess structure deformation. Externally, the subsidence of collapsible loess will lead to the subsidence of the foundation of the construction project, and the instability of the foundation will affect the overall construction quality of the construction project, and may seriously lead to the collapse of the building [2].

4. Treatment measures of collapsible loess to foundation

4.1. Treatment of collapsible loess foundation in investigation stage

The research on collapsible loess in modern construction industry has sorted out a set of relatively effective treatment methods, which have been applied in Northern Shaanxi and other places. Even in collapsible loess areas, beautiful and safe buildings can be built after effective treatment. Technicians should pay attention to the stability construction of foundation from the survey and design stage in the early stage of construction, Take necessary treatment measures according to the actual situation. Before construction, geological survey must be carried out to provide information reference for subsequent construction drawing design. As a prerequisite for foundation construction, construction enterprises should study the engineering characteristics of buildings according to the geological conditions of foundation engineering, actively make use of the characteristics of foundation soil, carry out effective politics and transformation, put forward practical and feasible schemes, and then evaluate the economy rationality. In daily work, most of the foundation construction is based on theory and practical experience. Therefore, construction enterprises should abide by the construction supervision specifications of the industry, compare a variety of construction schemes to obtain the most appropriate construction plan, so as to achieve the goal of effective investigation [3].

4.2. Treatment of collapsible loess foundation in design stage

The construction of buildings shall be based on accurate geological survey report, draw relevant conclusions according to many information presented in the geological survey report, design construction drawings that meet industry specifications and building engineering needs, select foundation types and

foundation treatment methods that meet construction requirements, and further clarify the foundation structure of foundation part, and draw effective construction drawings. Technical personnel in the process of drawing design, can use the past successful experience, to strengthen the details of the drawing processing. For example, the basement floor of the foundation part of the building adopts the water-resistant plate, which does not follow the raft design idea. The thickness of the floor should be more than 300 mm, and the width of the reinforcement and its cracks may be squeezed due to the reaction force of the foundation, so the design width should be less than 0.2 m. The pile foundation of the foundation part needs to carry out the necessary testing of its own quality and the bearing capacity of a single pile foundation. The technical personnel should accurately describe the measurement method and quantity in the drawings, and not only explain the content of such and such specification document, but also express the detection method accurately and in detail in the drawings. When designing drawings, technicians need to ask more experienced old technicians or department leaders to deal with collapsible loess. Combined with the successful collapsible loess treatment scheme, the successful experience is summarized [4].

4.3. Treatment of collapsible loess foundation in construction stage

The treatment methods of technicians for the construction of collapsible loess foundation include cushion method, lime-soil compaction pile method and dynamic compaction method. These treatment methods are also different according to different construction requirements.

4.3.1. Cushion

Technicians need to remove the collapsible loess under the foundation, and then use plain soil and ash to backfill. Layered rolling can be used to consolidate the coating layer, replace the original part of the collapsible loess, and enhance the stability of the foundation. It can reduce the collapsibility of the collapsible loess, control the deformation index of the foundation, further enhance the load force of the foundation, maintain the stability of the water in the original foundation, and do a good job in the treatment of the whole soil cushion.

4.3.2. Time-soil compaction piles method

This method is more applicable to the collapsible loess area with groundwater content of 14% - 25%. The depth of the pile body is between 5-13m. The technicians use the method of impact and pipe sinking to make the building foundation surface arranged orderly, and then fill the pile holes with lime soil. After layers compacted and squeezed in, the collapsible loess soil structure around the pile body has changed. After the construction, the technicians checked the samples and found that the compaction degree of the soil and the strength of the pile had been significantly improved. However, technicians need to pay special attention to the quality of lime soil cushion, otherwise foundation collapse accidents are easy to occur.

4.3.3. Dynamic compaction method

Dynamic compaction method uses the impact force generated by the free-falling process of gravity hammer to tamp the collapsible loess structure and enhance the strength of collapsible loess. It has been widely used in the treatment of collapsible loess because of its fast treatment speed, low investment and convenient construction method. After being hit by heavy hammer, the collapsibility of collapsible loess itself is reduced. The collapsibility coefficient will also be significantly reduced to achieve the effect of consolidating the foundation.

5. Conclusion

With the development of modern society, the demand for buildings is increasing, and the construction operation will inevitably encounter the problem of collapsible loess. Construction enterprises should have the ability to deal with collapsible loess, summarize the treatment measures of collapsible loess, maintain the stability of foundation engineering construction, and formulate a treatment plan suitable for the situation

of collapsible loess in this area based on the survey results, so as to comprehensively reduce the collapsibility of collapsible loess and achieve the ideal construction goal.

Disclosure statement

The author declares no conflict of interest.

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Analysis on the Layout Evolution and External Space Characteristics of the Red Gate Palace in Taishan

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Abstract: This article organizes and analyzes the historical evolution as well as the current situation of the Red Gate Palace, and probes into the artistic characteristics of the exterior space of the building. The conclusions are as follows: Relying on the unique forest environment of Mount Tai, the Red Gate Palace forms an impression of negative yin-yang where it is overlooking waters and with hills at the back. The guiding space comprising of the climbing route, the archway, and the Feiyun Pavilion which is built on the crossroad plays the role of guidance and suggestion; forming a complete spatial sequence with rhythm. The courtyard space which constitutes the east and west courtyards, in addition to the Little Taishan Temple is reasonable and ingenious in layout along with rich as well as well-proportioned spaces, and pleasant in scale.

Keywords: Red Gate Palace in Taishan; Guiding space; Courtyard space; Space characteristic

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1. Introduction

It is impossible to verify when the Red Gate Palace was founded. It has been repaired repeatedly since the Ming and Qing Dynasties. It is an important part of Taishan Ancient Architecture Group which is the key cultural relics protection unit of the state. Its architecture, tablet inscriptions, and memorial archways in the courtyard are of great historical value for the cultural study of ancient Buddhism, Taoism, as well as extended folk beliefs. As a religious building in a garden, the organic whole formed by the building, courtyard, and surrounding environment has a unique spatial layout.

2. Summary of the current situation

The Red Gate Palace is located at the north end of Hongmen Road, northwest of Wangmu Pool, with Guan Yu Temple at its south, the Middle Stream at its east, and Dacang Ridge at its west (**Figure 1.**). Covering an area of 2,528.2 square meters, it comprises of Yuanjun Hall in the west courtyard (including Little Taishan Temple), Maitreya Hall in the east courtyard, and the Feiyun Pavilion which is built across the climbing route connecting the east and the west courtyards.

The west courtyard of the Red Gate Palace consists of the main gate, Yuanjun Hall, Qiezhi Pavilion, the Heyun Pavilion, and the Floral-pendant Gate (**Figure 2.**). The main gate is the stone memorial archway with an architrave that is titled, "Red Gate Palace," and four words under it, "Yan Zhan Chu Bu." The main entrance has a side entrance to the south and another to the north. The main sections of the west courtyard are the Yuanjun Hall which is comprised of a lifting frame with four pillars and five beams, the west side hall which is the Heyun pavilion, the south hall which is Qiezhi Pavilion, and further down, the incense pool. The Floral-pendant Gate is located between Yuanjun Hall and Heyun Pavilion, which leads to the

northern Zen house and Little Taishan Temple. The Zen house comprises of the main hall and two side halls. To its northwest is the Little Taishan Temple with three columns in its hall.

The east courtyard comprises of the gate, Maitreya Hall, Gengyi Pavilion, South Hall, and other buildings (**Figure 2.**). The gate is westward with a stiff peak made up of four-column canopy. The main building, Maitreya Hall has three broad-faced buildings with nine hard mountain purlins at its front corridor. Gengyi Pavilion, which is a hall-style building is the east side hall of the east courtyard. The South Hall is located at the south of the courtyard, and at the south of the hall is a small courtyard.

Feiyun Pavilion, which connects the east and west courtyards, is a tunnel-like passageway building, which is divided into upper and lower layers. The lower layer is an arched portal whereas the upper part has three main halls. The east and west courtyards as well as the Feiyun Pavilion form a group of semi-enclosed concave space (**Figure 2.**).

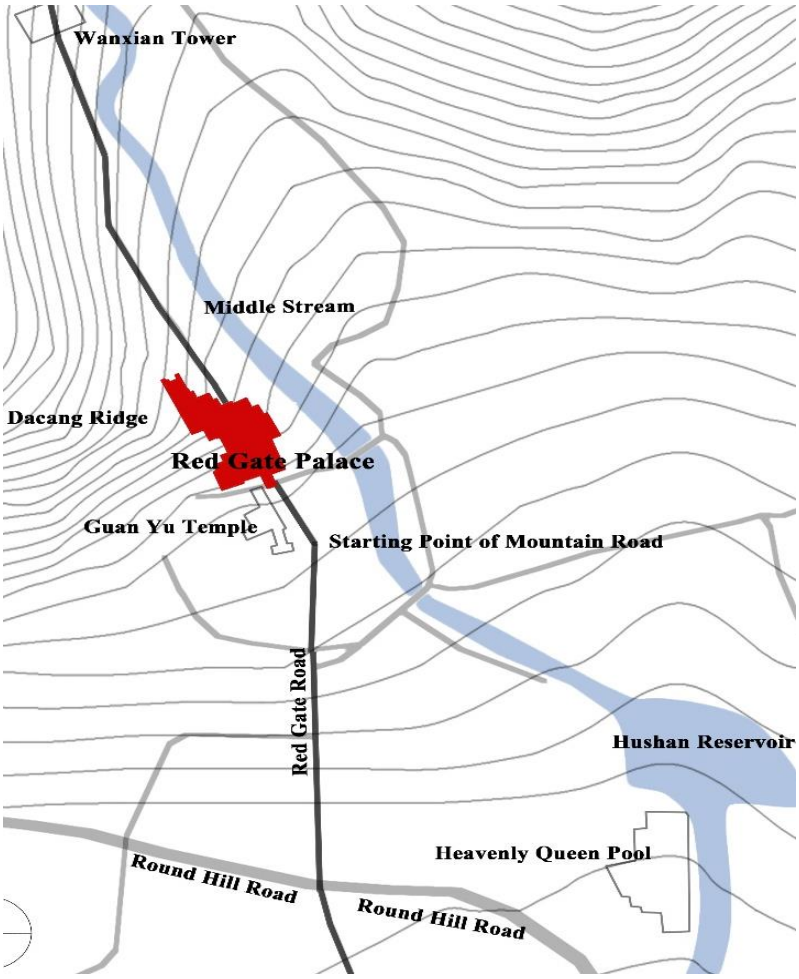


Figure 1. Location map of Red Gate Palace

3. Historical evolution

It is not known when the Red Gate Palace was founded. According to historical records, in the sixth year of Tianqi in the Ming Dynasty (1626), there was an extension construction [1]. In the Wanli Period of the Ming Dynasty (1563-1620), the Gengyi Pavilion (**Figure 3.**) was used for changing clothes when mountaineering [2]. In the Qing Dynasty, Feiyun Pavilion was renamed as Guanyin Pavilion, and the Gengyi Pavilion was renamed as Maitreya Hall. Yuanjun Temple, Red Gate Memorial Archway, and Heyun Pavilion were also built there [3-4] (**Figure 4.**). The Little Taishan Temple in the north of the palace can be seen in many documents during the period of the Republic of China [5] (**Figure 5.**). After the establishment

of the Republic of China, it has been repaired many times. The two integrated, and finally formed the current pattern.

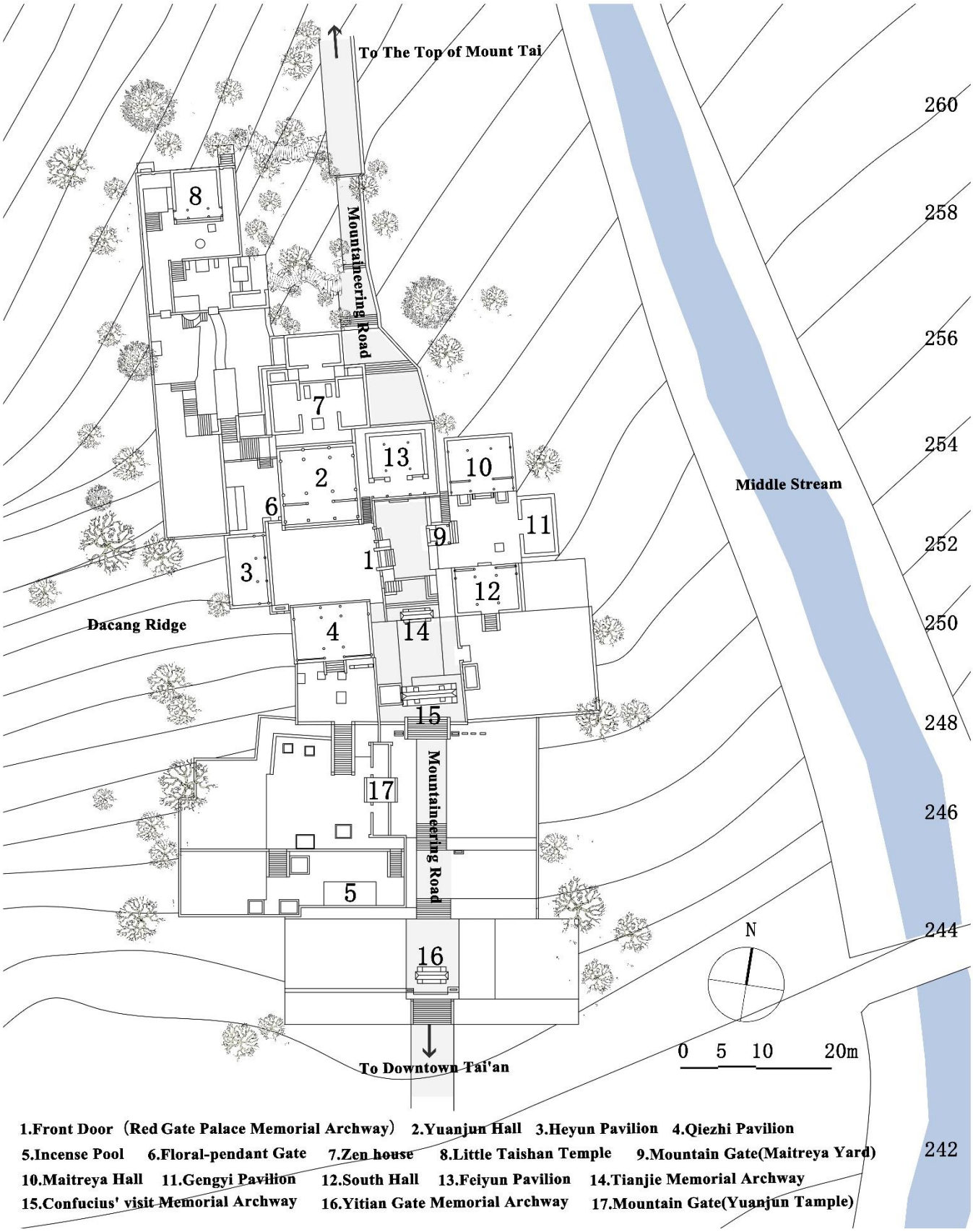


Figure 2. General layout of the Red Gate Palace

4. Site selection and spatial layout

4.1. Site selection

Located at the foot of Mount Tai, the Red Gate Palace is adjacent to the towering Dacang Ridge to the west and the winding Middle Stream to the east, forming an impression of negative yin-yang where it is overlooking waters and with hills at the back. Its surrounding vegetation is luxuriant, with mountains and waters embracing it; being naturally pleasurable.

4.2. Entire allocation

As a group of religious buildings, the Red Gate Palace consists of two parts: the guiding space and the courtyard space [6] (**Figure 6.**). Influenced by the layout of traditional folk houses and temples in the north, the Red Gate Palace is spread out vertically and horizontally in the form of multiple three-hedron courtyards and quadrangle courtyards while the main halls and side halls for the worship of statues are arranged on the axis. Due to its mountainous location, it is different from the strict axial symmetry layout pattern of flat temples. The whole courtyard of the Red Gate Palace emphasizes more on the integration and construction of mountains which reflect the ecological view of Taoism where, "Taoism follows nature." However, some of them are still symmetrical, reflecting the idea of "the notability is in the center." For example, the main buildings of the eastern and western houses, the Maitreya Hall, and Yuanjun Hall are all arranged symmetrically with the climbing route as the central axis (**Figure 7.**) [7-8].

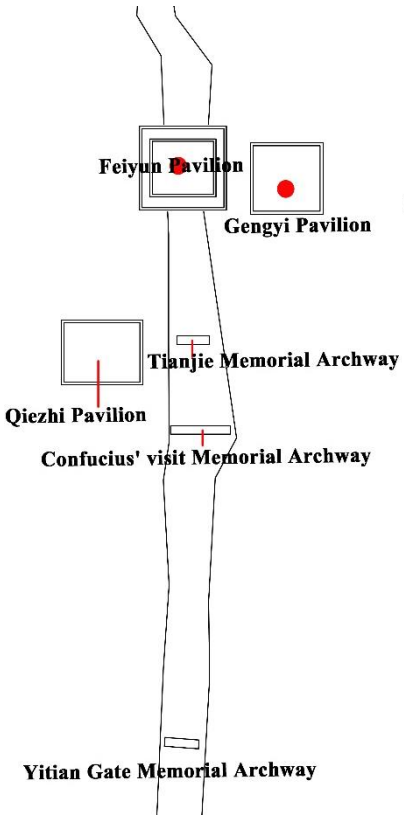
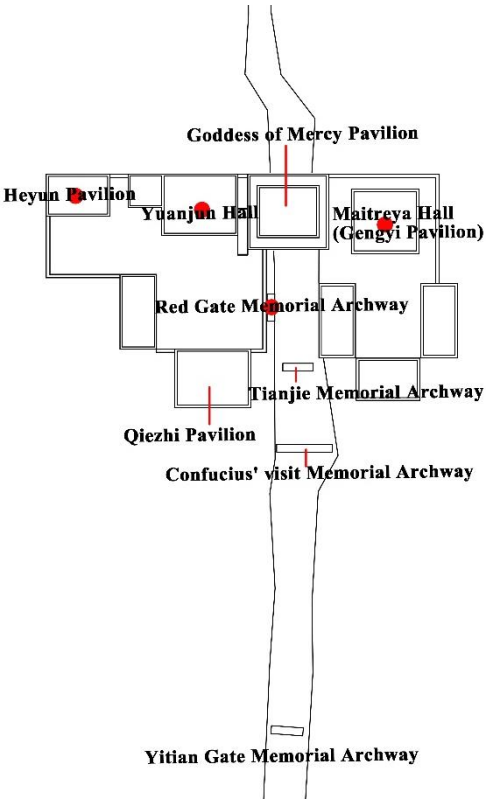


Figure 3.



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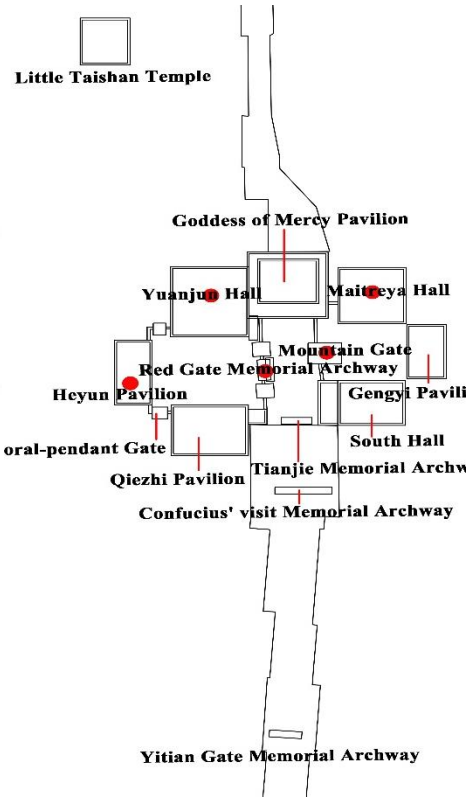

Figure 5.

(1) **Figure 3.** Layout plan of the Red Gate Palace during Wanli Period of Ming Dynasty

(2) **Figure 4.** Layout plan of the Red Gate Palace during Qianlong Period of Qing Dynasty

(3) **Figure 5.** Layout plan of the Red Gate Palace in the early years of the Republic of China

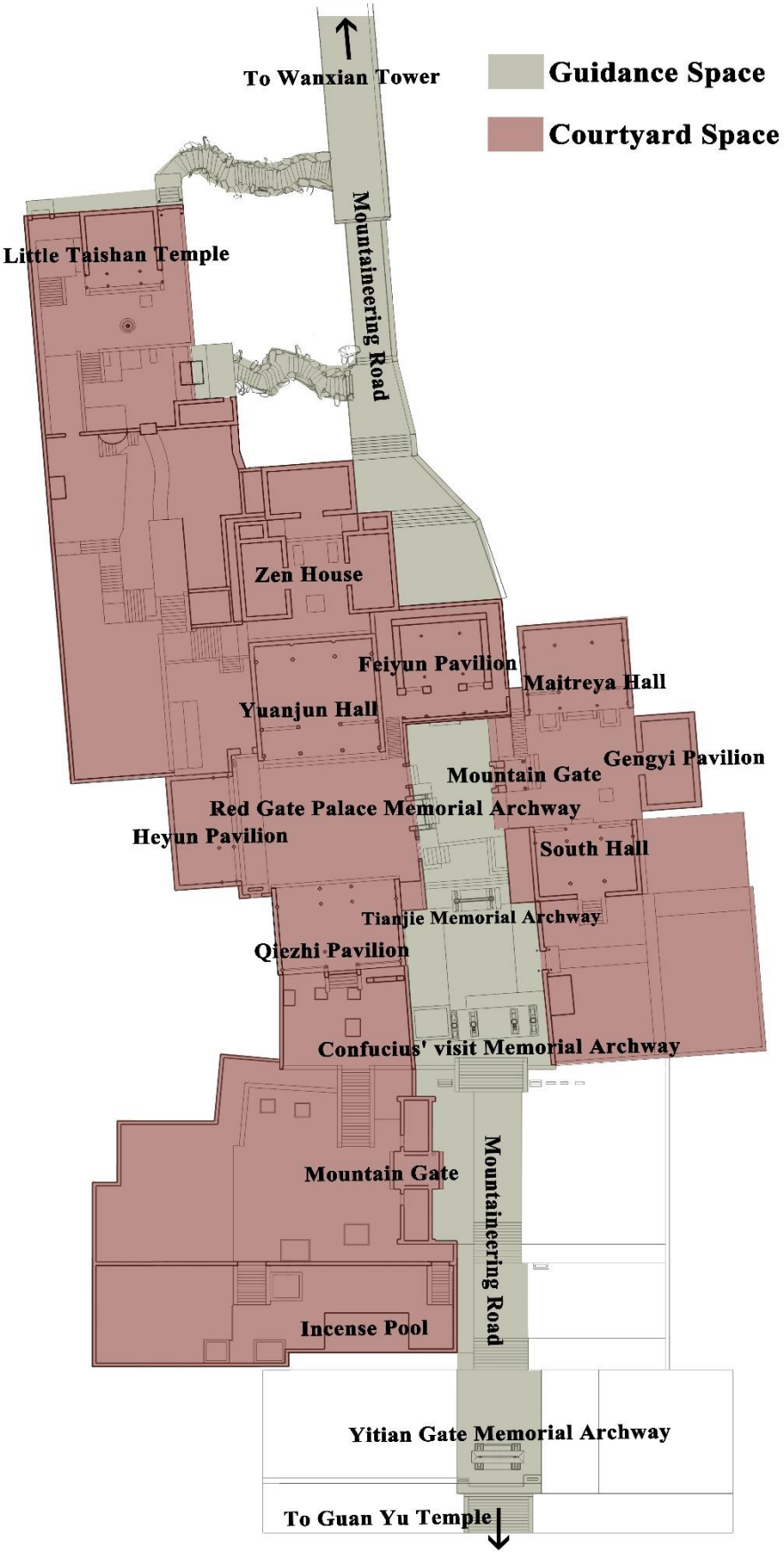

Figure 6. Spatial layout of the Red Gate Palace

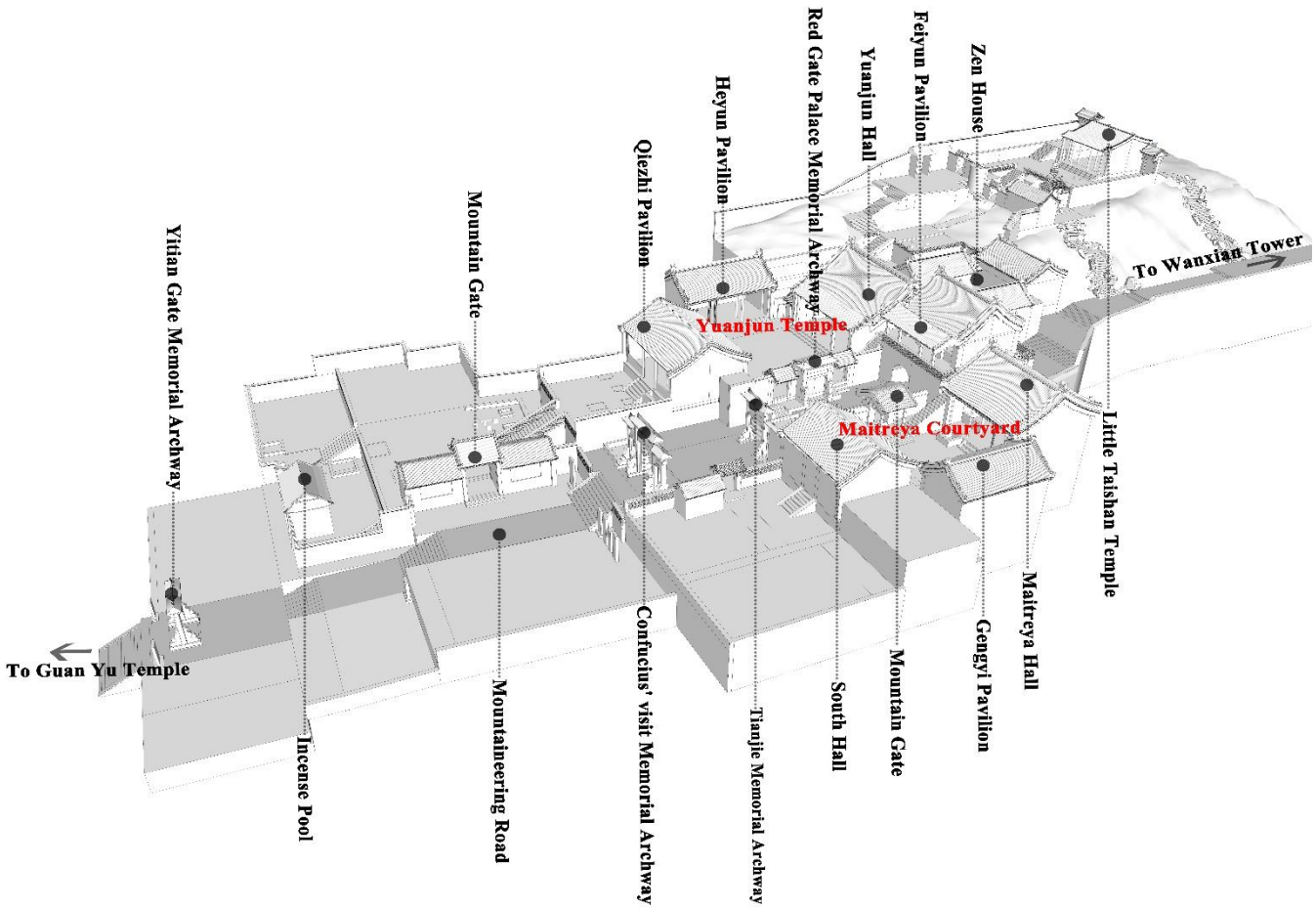

Figure 7. Aerial view of the Red Gate Palace

4.3. Space function and composition

4.3.1. Function and composition of the guiding space

The pilgrimage activities in Red Gate Palace do not begin when the pilgrims enter the courtyard, but gradually unfold in the outer space of the palace [9]. From the steps in front of Yitian Gate to the climbing route in front of the Red Gate Palace, while meeting the basic traffic and sacrificial functions, the space for "pilgrimage" is formed which becomes the beginning of the palace temple sequence. On the one hand, it connects the urban environment with the palace in a natural mountain forest which stimulates the religious emotions in pilgrims. On the other hand, its sacred and solemn atmosphere in addition to climbing its stone steps make pilgrims feel that they are "far from the world." In combination, the outer wall of the building, archway, steles, plants, natural rocks, and the Feiyun Pavilion which is built across the road constitute the guiding space of the Red Gate Palace (**Figure 8.**). The inscriptions on the archway, steles, and the walls of the building expand the spatial information perceived by the pilgrims across the dimensions of time and space.

4.3.2. Function and composition of the courtyard space

The courtyard space mainly meets the religious sacrificial function. Decorated with flowers, trees, and furnishings, it is enclosed by the main building, the wall, as well as the mountain gate. According to its function, it can be divided into sacrificial spaces, working spaces, and sightseeing spaces whereas based on its spatial nature, it can be divided into transitional space, core space, and subsequent space [6] (**Figure 9.**).

The core space is the main sacrificial space which comprises of Yuanjun Hall and its side hall in the west courtyard, as well as Maitreya Hall in the east courtyard with the Feiyun Pavilion connecting them.

The architectural grade and volume are unparalleled in the whole palace. The first courtyard of the west courtyard is a transitional space with multi-layered terraces and pine trees planted in its courtyard which create a solemn and sacred atmosphere, guiding visitors and pilgrims into the main hall. The Zen house and the Little Taishan Temple constitute the subsequent space which is the continuation and supplement of the core space that enriches the spatial level and plays the role of "turning" in order to avoid the abrupt end of the spatial sequence after the climax in eliminating the sense of awkwardness.

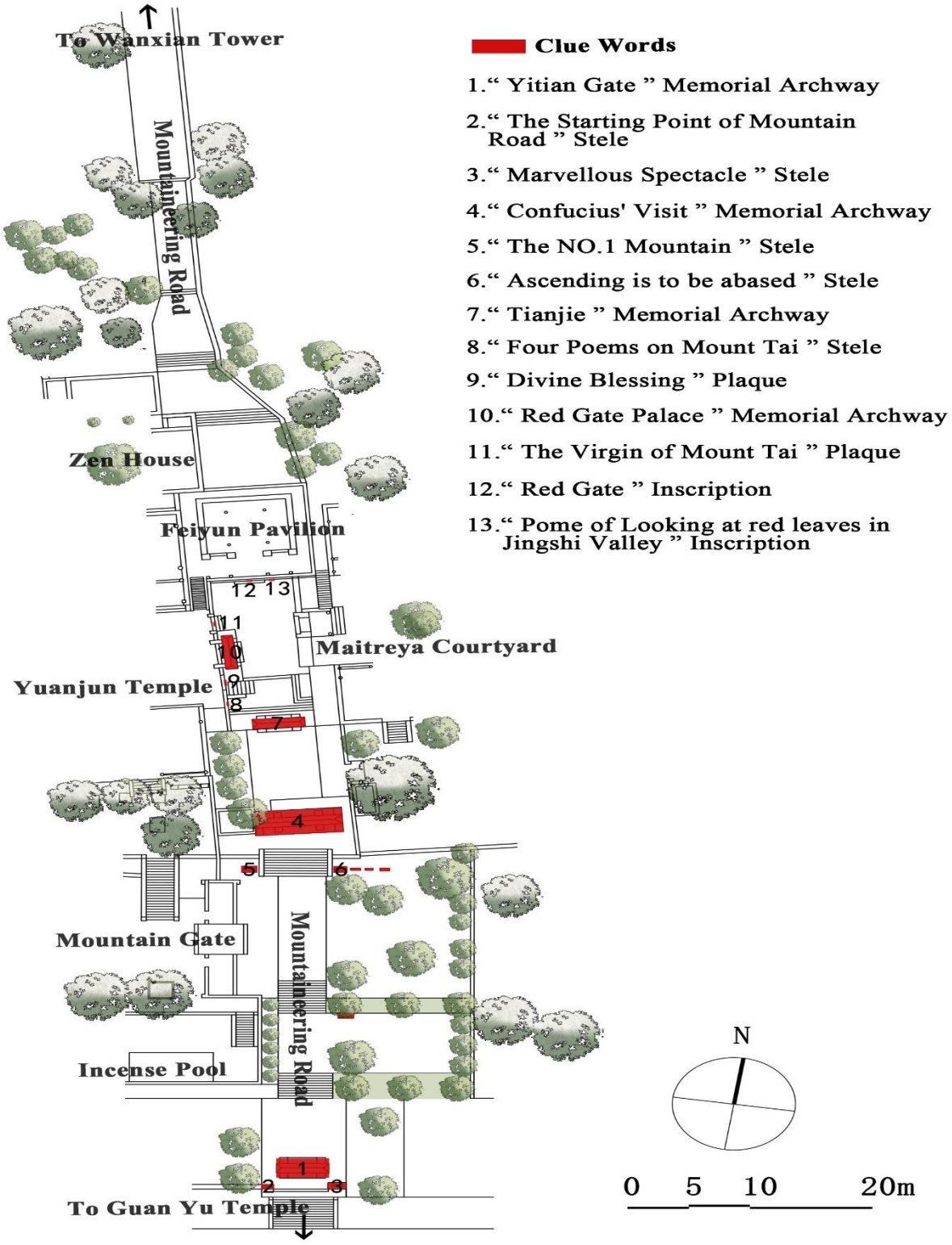

Figure 8. Plan of guiding space

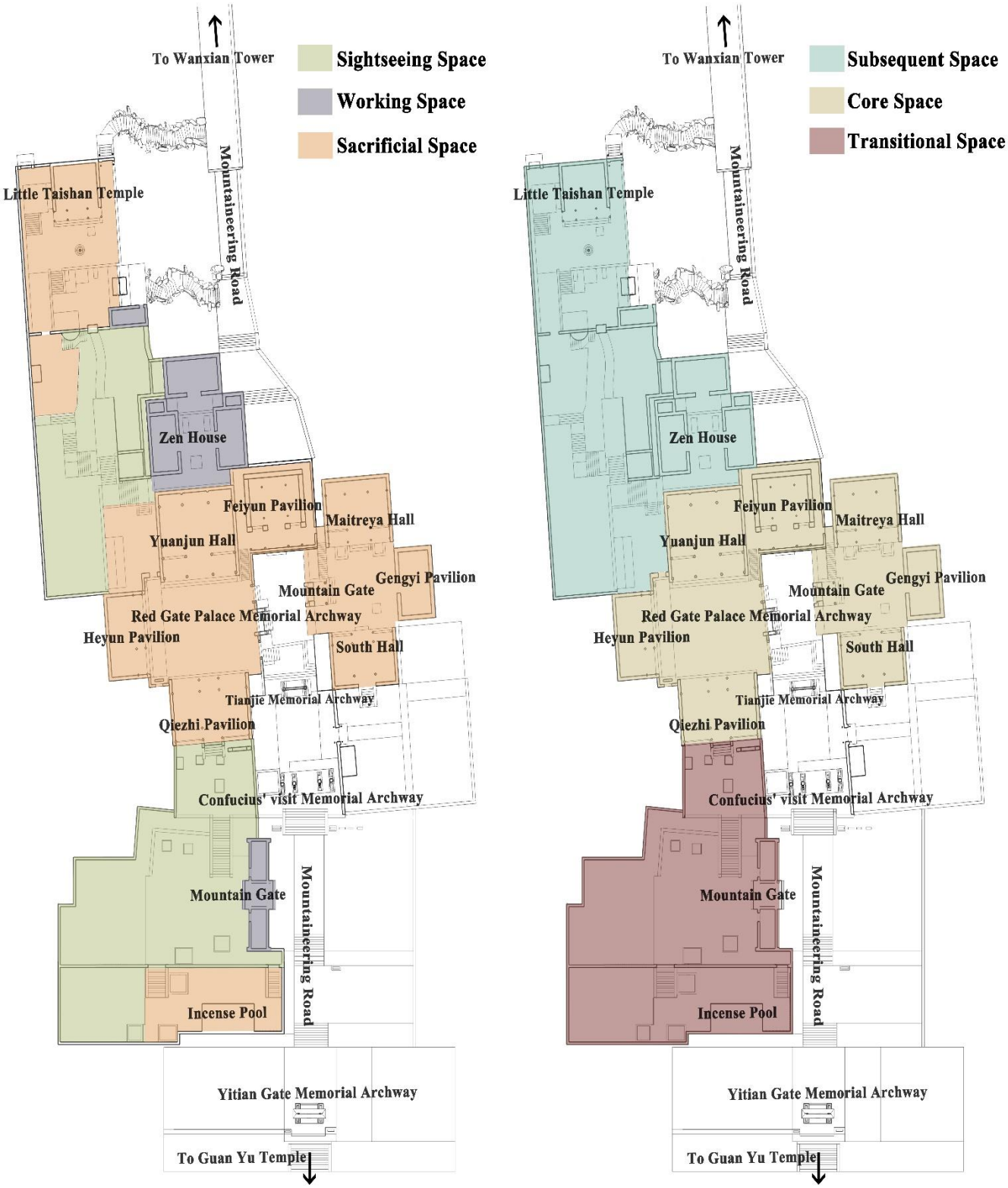

Figure 9. Classification map of the courtyard space of Red Gate Palace

4.4. Spatial scale analysis of the courtyard

Restricted by the rugged mountains, the architectural volume of the Red Gate Palace is not very grand, and the absolute size of the courtyards is very limited, but it still constructs rich space (**Table 1.**).

Table 1. Scales of courtyard space

Types of courtyard space	Name of courtyard	Length (m)	Width (m)	Aspect ratio	Area (m^2)	Plane form	Plane icon
Transitional space	Incense Pool Courtyard	7.50	30.23	0.29	227.30	Rectangle	
	Mountain Gate Courtyard	17.22	25.76	0.71	391.11	Rectangle	
	Feiyun Pavilion	9.00	10.58	0.85	90.71	Square	
Core space	Yuanjun Hall	10.46	14.48	0.72	154.36	Rectangle	
	Feiyun Pavilion	9.56	10.53	0.91	48.17	“回” shape	
	Maitreya Hall	8.90	11.50	0.77	96.91	“凹” shape	
Subsequent space	Zen House	10.48	4.82	2.17	64.42	“工” shape	
	Little Taishan Temple	19.28	12.51	1.54	193.93	“凹” shape	
	West side of Zen House	/	/	/	531.14	Rectangle	

Different from ordinary buildings, the core space of the Red Gate Palace is small in size and limited in space, presenting a special “回” or “凹” shape while the transitional space and subsequent space have relatively wide vision, and their plane forms are mostly square in nature. The length-to-width ratio of each courtyard in the transitional space and the core space is less than 1, which gives people a sense of isolation whereas the length-to-width ratio of the subsequent space courtyard is greater than 1, which has a profound artistic conception. The courtyard of Little Taishan Temple has a length-to-width ratio between 1 and 2 with a compact space and a strong sense of balance [8, 10].

5. Spatial features

5.1. Features of the guiding space

5.1.1. Guidance and suggestion

The building walls on both sides of the middle road enhance the sense of longitudinal extension and form a long, narrow space with strong guiding propensity. The gradually rising steps and building walls also make people feel the change in different positions [11], thus effectively playing the role of guidance and suggestion (**Figure 10.**).

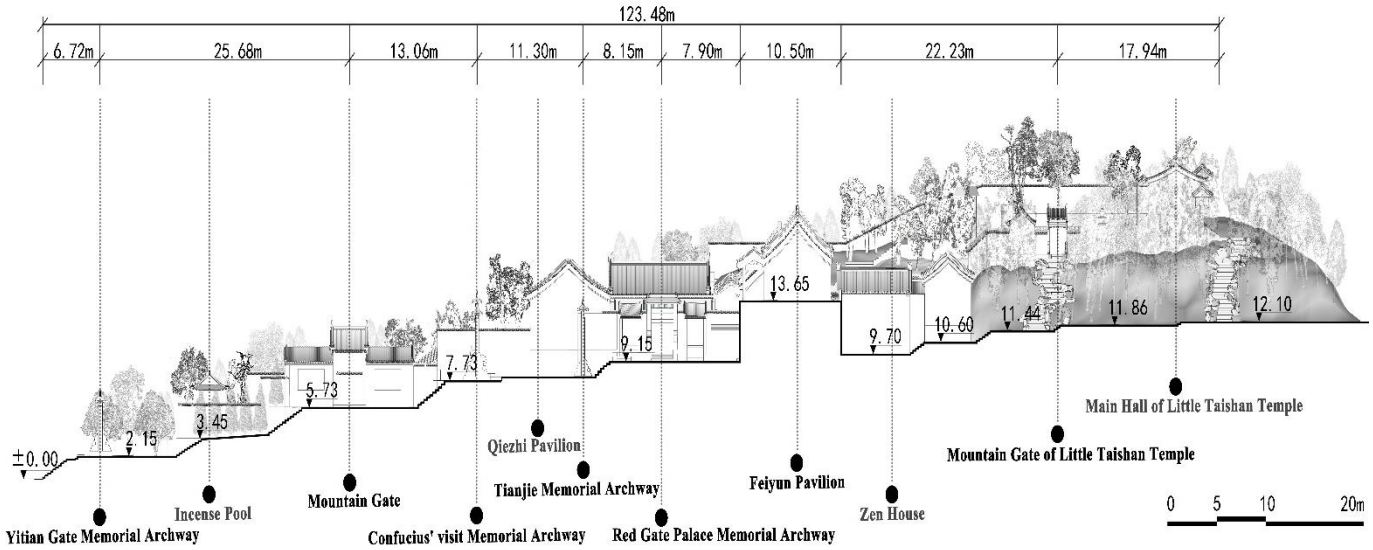

Figure 10. Sections of guiding space

5.1.2. Rhythm and rhyme

A complete rhythm is formed by the elevation of the terrain and the arrangement of the four groups of archways. The distance, volume, shape, and structure of each archway as well as the difference in content exude a certain rhythm. Combining the ups and downs as well as the opening and closing of the level, time, space, and landscape are all coordinated so that the pilgrims and visitors have an experience similar to, "It is like a range when you look at the mountain from the front, but it is like a peak when you look at it sideways" (**Figure 10.**).

5.1.3. Complete spatial sequence

As a space independent of the palace courtyard, the guiding space itself also forms a complete spatial sequence. When the visitors' sight is focused at the center of the road, the memorial archway and buildings built across the road are connected as the focal point of the sight. Yitian Gate square has a wider view which is the initial stage of the spatial sequence. As the tour moves on, the buildings on both sides gradually appear in the line of sight. When arriving at the Confucius' visits and Tianjie Memorial Archway, the space is slightly narrowed. Nearing the Red Gate Palace, both sides of the buildings are closer, and where Feiyun Pavilion stands at the moment, the spatial sequence reaches a climax along with the sentiments of the visitors. After entering the door, the light darkens, the mouth of the cave is suddenly opened, and the visitors' emotions return to calmness. The whole spatial sequence that alternates between light and darkness, as well as between open and bundled adds to the fascination of sightseeing (**Figure 10.**).

5.2. Characteristics of the courtyard space

5.2.1. Reasonable and ingenious overall layout

The layout is compact and reasonable, the space twists and turns, and the steps effectively eliminate the height difference as well as connect the courtyard space. Each courtyard is spread along the axis, but the layout is not strictly symmetrical which not only respects the spatial order of traditional buildings, but it also well reflects the spatial characteristics of mountain buildings [12].

5.2.2. Rich and well-proportioned space

Along the central axis, the buildings alternate from single or double buildings to three or even grouped buildings, and finally return to a single building. On the plane, it is simple – complex – simple, while the external space presents a passive – positive – negative transition. The contrast between the large and the

small, freedom and strict, open and closed courtyard acquires rhythm. The plane composition is clear and simple. The sense of place of the palace can be interpreted through rich spatial sequences [12].

5.2.3. Friendly and pleasant spatial scale

The small area of the core space courtyard marks the main building which is not very large but appears majestic and solemn. The vision of the transition space and the subsequent space is relatively wide which is in contrast with the core space courtyard. This kind of space pattern which conforms to topography, breaks the traditional pattern, and in combination with the volume of the building which is close to the folk house, gives out a friendly and pleasant feeling.

6. Conclusion

The Red Gate Palace, an ancient architectural complex, is to the east of the river, the west of Dacang Ridge, and there are hiking trails in the mountain. Based on these circumstances, it appears to be organic. Relying on the unique natural landscape and cultural environment of Mount Tai, the combination of Yuanjun, in the west and Maitreya, in the east explains the integration and development of Taoism and Buddhism in Mount Tai which carries rich historical and religious cultural values. Through rich spatial transformation and careful line of sight arrangement, the contradiction of the courtyard expansion scale demands and the limitations by mountainous terrain is resolved in which a unique mountain palace landscape is formed.

Disclosure statement

The author declares no conflict of interest.

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Analysis of Key Technologies in the Design of Small and Medium Span Basket Type Steel Box Tied Arch Bridge

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Abstract: In order to ensure the construction quality and safety of small and medium span basket type steel box tied arch bridge, this paper takes a practical project as an example to analyze the key technologies in its design process. It is hoped that this analysis can provide corresponding reference for the design and construction of this kind of arch bridge.

Keywords: Medium and small span; Basket type steel box tie arch bridge; Arch Bridge Design; Key technology

Publication date: July 2021; ***Online publication:*** July 31, 2021

1. Introduction

In the design process of medium and small span basket type steel box tied arch bridge, the most key technology is to analyze and check some important local positions, such as steel structure arch box lifting lug, lower anchor head of suspender, steel-concrete joint section and so on. In this way, the design parameters of each key part can be effectively ensured, so as to ensure the design quality of steel structure tied arch bridge and meet its actual construction and application needs [1].

1.1. Project overview

This paper studies the local design checking calculation of a small and medium-sized span basket type steel box tied arch bridge across the river in Southwest China. The bridge project belongs to the main road of the city, the total length is 106.199m, the design speed is 40km, the road width is 27.2m with two-way four lanes, the structure is the steel box arch bridge, the main span is 74m, the steel box girder material choice is Q345C steel, and the concrete system beam is C50 concrete. This paper mainly analyzes the local key technologies such as arch box lifting lug, anchor head under suspender and steel-concrete joint section.

2. Analysis of key technologies in design

2.1. Local analysis of arch box lug position

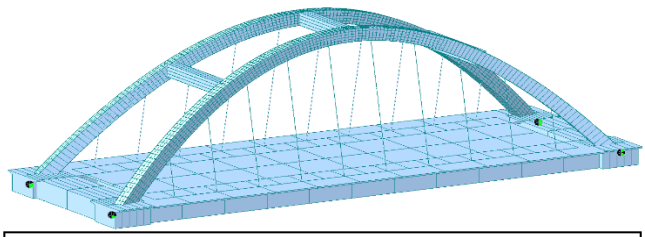


Figure 1. Calculation model of the main beam in this project

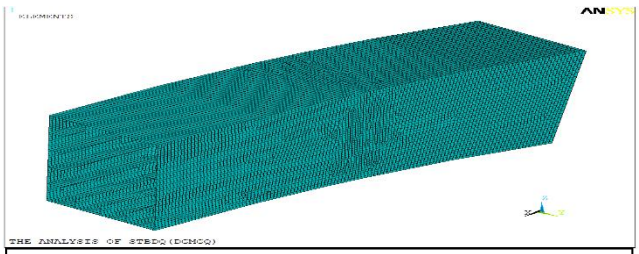
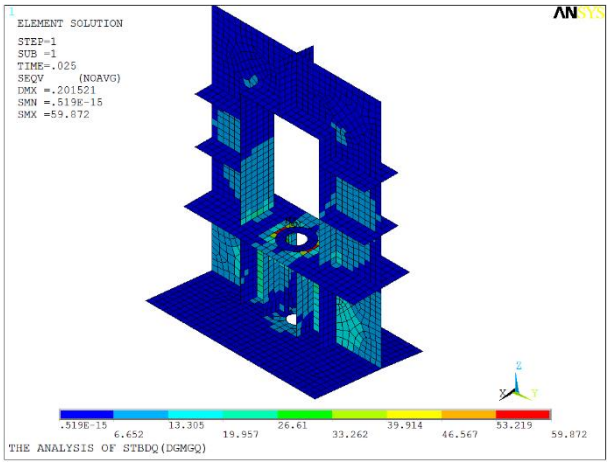


Figure 2. Finite element model of plate and shell in suspender anchorage zone

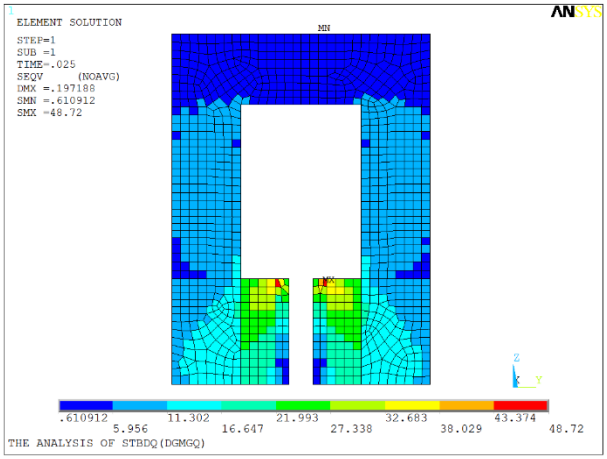
According to the member system calculating analysis **(Figure 1)**, the internal force of the suspender at the vault position is the largest. Under the combined envelope of the limit state of bearing capacity, the maximum internal force of the suspender is 2856kN.

The box arch section of the vault area is selected for local analysis of the anchorage area of the derrick **(Figure 2)**. ANSYS was used to establish the plate and shell finite element model of related segments, SHELL181 elastic-plastic element was used as element type, and the thickness of plate and shell element was given by real constant. The arc length method is used to analyze the ultimate bearing capacity.

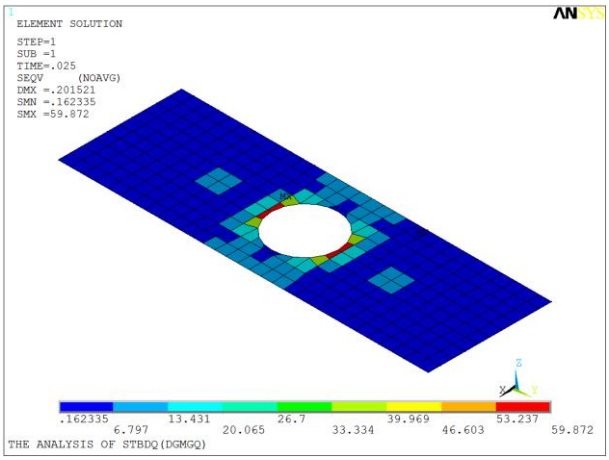
Lifting point displacements and lifting point forces of each sub-step were extracted to form the load displacement curve. When the tangent stiffness of the load displacement curve approached 0, it was the ultimate bearing capacity of the anchorage zone of the suspender. The following is the cloud diagram of equivalent stress in the anchorage zone of (step1) lifting point. **(Figure 3)**



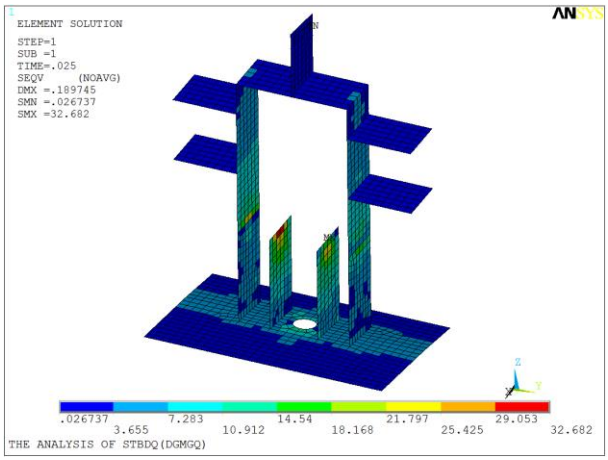
a. Cloud diagram of overall equivalent stress in suspender anchorage zone



b. Equivalent stress cloud diagram of diaphragm N1 and N1A in anchorage zone



c. N2 equivalent stress cloud diagram of anchor bearing plate



d. Cloud diagram of equivalent stress of stiffening plate N3 / N4 / N5 / N6 / N7 / N8

Figure 3. Cloud diagram of equivalent stress in suspender anchorage zone (step1)

The equivalent stress diagram step1 ~ 14 and load displacement curve of suspender anchorage zone can also be obtained as in **Table 1.**

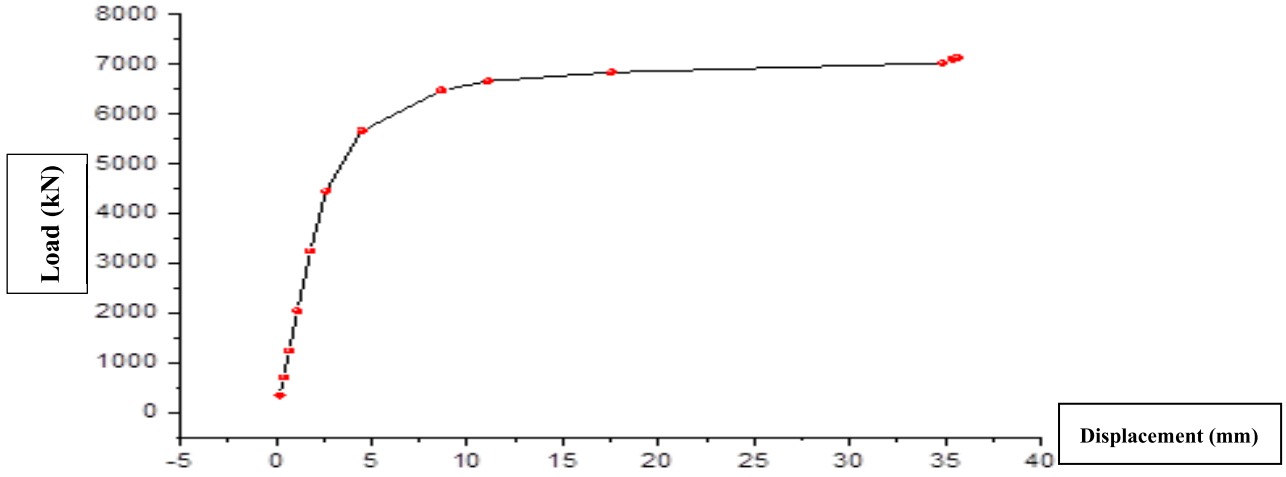


Figure 4. Load displacement curve of the suspender anchorage zone

Table 1. Load displacement table

Step	Load (kN)	Displacement (mm)
1	357	0.2
2	714	0.4
3	1250	0.7
4	2053	1.1
5	3258	1.8
6	4463	2.6
7	5667	4.5
8	6481	8.6
9	6663	11.1
10	6845	17.5
11	7027	34.9
12	7109	35.4
13	7126	35.5
14	7142	35.6

It can be seen from the load displacement curve **(Figure 4)** that when the tangent stiffness tends to 0, the corresponding ultimate bearing capacity is 7027kN, which is greater than the maximum suspender force of 2856kN and the safety factor is 2.46. Meet the requirements.

2.2. Local analysis of anchor head under suspender

The local main girder of the lower anchor head of the suspender is selected for analysis **(Figure 5)**. The model adopts the solid element, and the most unfavorable suspender force of the standard combination envelope is applied to the anchor bearing plate of the lower anchor head in the form of surface pressure load [2].

Below are figures 5-10:

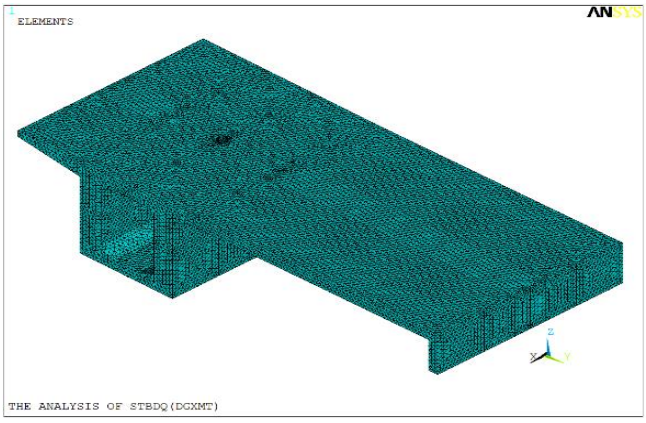


Figure 5. Finite element model for local analysis of lower anchor head of suspender

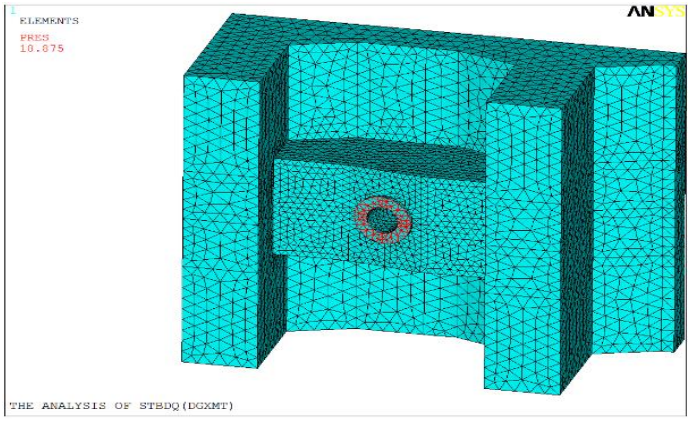


Figure 6. Enlarged finite element model of anchor head under suspender

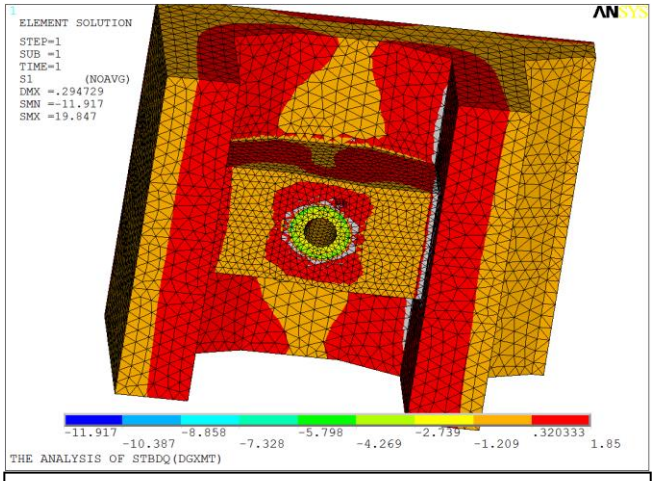


Figure 7. Cloud diagram of principal tensile stress of anchor head under suspender

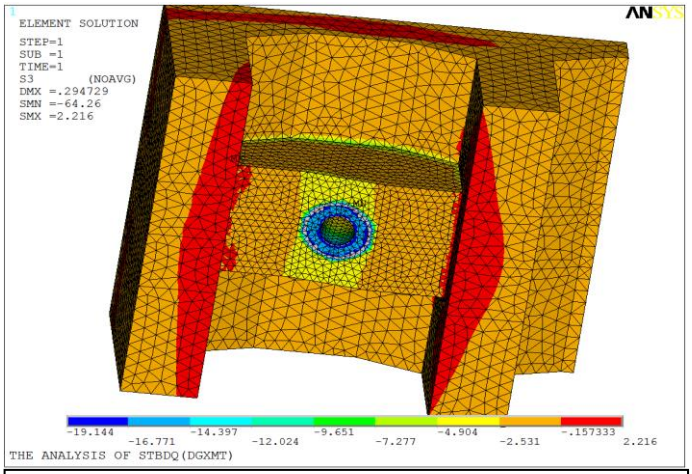


Figure 8. Cloud diagram of principal compressive stress of anchor head under suspender

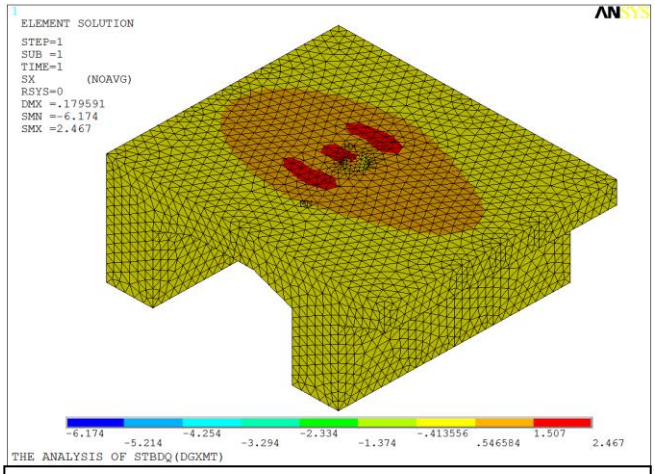


Figure 9. Normal stress of anchor head top plate under suspender along the bridge

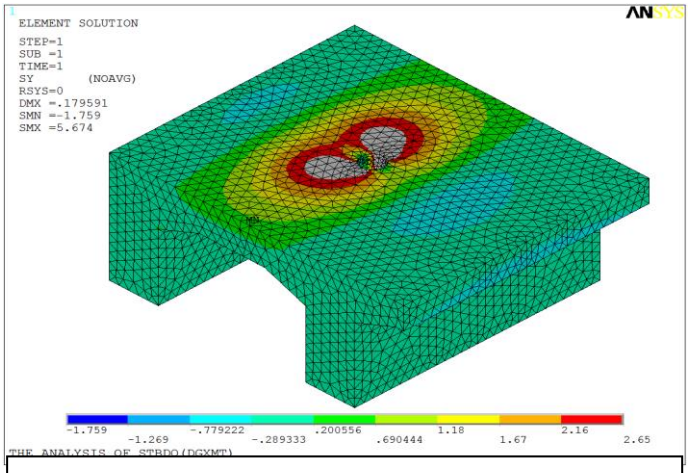


Figure 10. Normal stress of the anchor head top plate under the suspender

According to the calculation, under the most unfavorable suspender force of the standard composite envelope, the main tensile stress in most areas of the anchor head under the suspender is less than the allowable value of 1.855 in the code. The local stress concentration exists in the concrete position of the anchor head **(Figure 6)**, and the local main tensile stress exceeds the limit at the junction of the anchor block and the web. However, considering the contribution of regular reinforcement, the force meets the specification requirements.

According to the calculation, under the most unfavorable suspender force of the standard combination envelope, except for the stress concentration in a small area, the main compressive stress in most areas is less than the allowable value of 19.44mpa, and the checking calculation of the main compressive stress meets the requirements of the specification **(Figure 7)**.

According to the calculation, under the action of the most unfavorable suspender force in the envelope of the standard combination, the maximum normal stress of the top plate along the bridge in the suspender area is 2.46mpa **(Figure 8)**, which occurs only in a small local range, and is less than the allowable value of 1.83mpa in most other areas **(Figure 9)**. Considering that the ordinary steel bars in the top plate participate in the stress, the stress meets the requirements of the code [3].

According to the calculation, under the action of the most unfavorable suspender force of the standard combination envelope of the standard combination, there is a stress concentration phenomenon in the local area of the horizontal bridge of the top plate in the suspender area, and most other areas are less than the allowable value of 1.83 Mpa in the specification **(Figure 10)**. After considering the stress of the regular reinforcement in the top plate, the stress meets the specification requirements.

2.3. Local analysis of steel-concrete joint section

The steel-concrete joint section near the arch foot is selected for local analysis as in **Figure 11**. The finite element model of the steel-concrete joint section near the arch foot is established by ANSYS. The concrete part is simulated by SOLID65 solid element, the steel structure part is simulated by shell63 shell element, and the plate and shell element, prestressed element and solid element in the steel-concrete joint section are mixed and meshed, which are coupled by constraint process.

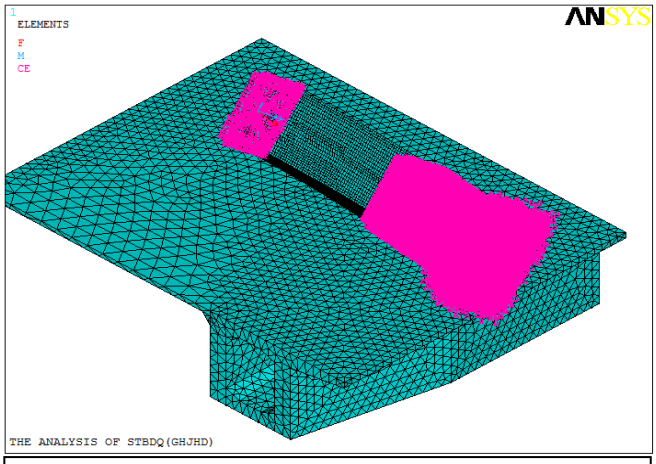


Figure 11. Finite element mesh and constraint equation of steel-concrete section

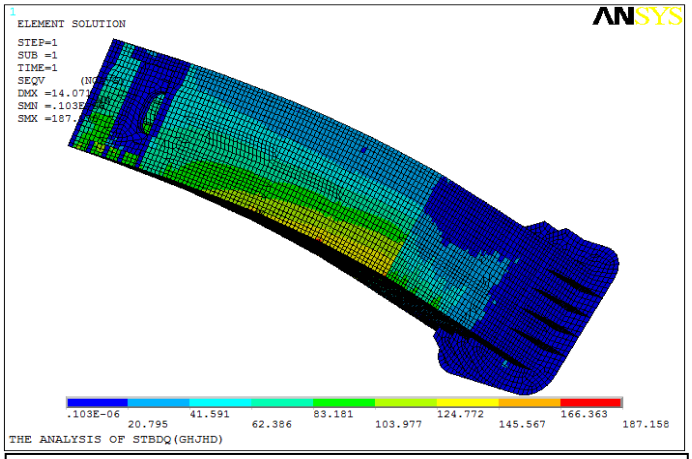


Figure 12. Cloud diagram of equivalent stress of steel structure unit in steel-concrete joint section

Select the most unfavorable load under the standard combination envelope state for loading. As shown in **Figure 12**, under the standard combination envelope, the maximum equivalent stress of the steel structure unit in the steel-concrete joint section is 187.2mpa, which is less than the yield strength 345mpa. The stress of the steel structure unit in the steel-concrete joint section meets the specification requirements [4].

Similarly, in the case of standard combined envelope, other components of steel-concrete joint section are as follows:

(1) The maximum equivalent stress of diaphragm unit is 77.8MPa, which is less than the yield strength of 345MPa. The stress of diaphragm unit in steel-concrete joint section meets the specification requirements.

(2) The maximum equivalent stress of the stiffened steel plate in the steel-concrete joint section is 27.7MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(3) The maximum equivalent stress of the stiffening rib plate unit in is 124.2MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(4) The maximum equivalent stress of the steel plate unit of the top plate of the arch rib steel box is 114.8MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(5) The maximum equivalent stress of the steel plate unit of the bottom plate of the arch rib steel box is 187.2MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(6) The maximum equivalent stress of the steel plate unit of the web of the arch rib steel box is 183.8MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(7) The maximum equivalent stress of arch box stiffener steel plate unit is 179.1mpa, which is less than the yield strength of 345mpa, meeting the requirements of the specification.

(8) The maximum equivalent stress of the base plate steel plate unit in is 40.3MPa, which is less than the yield strength 345MPa, meeting the specification requirements.

(9) In addition to the distortion of local principal tensile stress in the prestressed anchorage zone, the principal tensile stress in most other areas of the steel box is less than the allowable value of 1.855MPa.

(10) In addition to the distortion of local principal compressive stress in the prestressed anchorage area, the principal compressive stress in most other areas of steel box is less than the allowable value of 19.44mpa.

(11) In addition to extend standard of extremely small areas, the normal tensile stress of the concrete of the main beam outside the box arch along the bridge is lower than the allowable value of 1.83MPa. Considering that the reinforcement is involved in the force, the checking calculation of the normal tensile stress of the main girder along the bridge meets the requirements of the code.

(12) The maximum normal tensile stress of the concrete transverse bridge of the main beam outside the box arch is 1.97MPa, slightly greater than the allowable value of 1.83MPa in the code. Considering that the reinforcement is involved in the force, the checking calculation of the normal tensile stress of the main girder along the bridge meets the requirements of the code.

3. Conclusion

To sum up, during the design of small and medium span basket type steel box tied arch bridge, the design unit must make local analysis and checking calculation for key parts such as the upper and lower anchor ends of suspenders and steel-concrete joint sections. Only by ensuring the design requirements of key parts can we effectively ensure the quality and safety of the whole project and meet its actual construction and application needs. This has far-reaching significance for the design, construction and application of small and medium span basket type steel box tied arch bridge.

Disclosure statement

The author declares no conflict of interest.

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2. This result was later contradicted by Becker and Seligman [5].
3. This effect has been widely studied [1–3, 7].

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References

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Note: When referencing an entry from a dictionary or an encyclopedia with no author there is no requirement to include the source in the reference list. In these cases, only cite the title and year of the source in-text. For an authored dictionary/encyclopedia, treat the source as an authored book.

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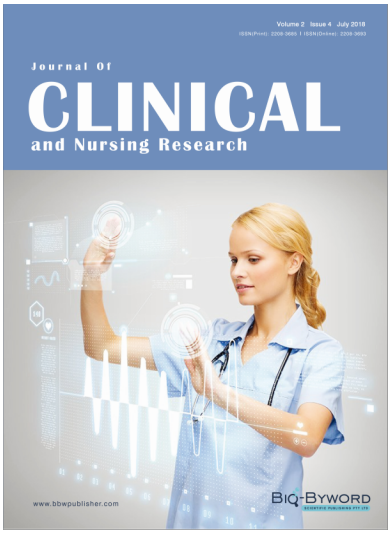

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