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The design and optimization of an emerging pile coupling with application to drilled and PHC pipe cased piles

Hesong Hu[1], Fan Zhao[1,2], Mengxiong Tang[1], Shuo Zhang[1,3], Hang Chen[1,3]*
1. Guangzhou Institute of Building Science Co., Ltd., 510440, Guangzhou, China;
2. Faculty of Land Resource Engineering，Kunming University of Science and Technology，Kunming 650093，China;
3. School of civil engineering, Guangzhou University, 510006, Guangzhou, China.

* Corresponding author: chenhang.thu@gmail.com.

Abstract. To overcome the unstable quality, poor durability of the commonly used welded couplings and structural complexity of existing mechanical couplings, an emerging coupling design with simple structure and higher construction efficiency, with application to the construction of drilling and PHC pipe cased piles, is proposed in this paper. The load bearing capacities of the proposed coupling meets well with the requirements of corresponding standards and specifications. Meanwhile, it makes the connection of the piles strictly sealed, which prevents the grouted cement slurry from permeating into the core of the pipe piles and thereby is beneficial to the maintenance of grouting pressure and quality. Finite element method is employed to reveal its bearing mechanism under different load conditions. The load bearing capacity of the above coupling made of steel with different grades is verified, and its main design parameters are optimized, which can provide theoretical basis for its practical engineering applications.

1. Introduction

Nowadays, jacking and driving methods are the two mainly technologies to install prestressed high-strength concrete pipe piles (PHC piles). The jacking method is merely applicable to the case of soft strata with low piling resistance. Meanwhile, the driving method also leads to excessive noise and possible damage to the pile concrete, especially in the areas of stiff strata. With the continuous accumulation of engineering experience, Tang et al.[1-3] proposed a green and environment-friendly construction method and corresponding equipment to drill hole and install the PHC piles without extra casing materials, namely drilled and PHC pipe cased piles (DPC piles), where the sinking PHC piles synchronously with the drilling operation also act as the casing materials to guarantee the stability of bored hole and avoid the environmental pollution induced by the utilizing of slurry in traditional bored and cast-in-place piles. When the piles reach the desired depth, the gap between the pile and the cylindrical surface of the bored hole is grouted with cement paste to enhance the frictional resistance of the pile foundation. The mentioned new method not only avoids the damages of pile concrete induced by the traditional jacking or driving methods, but also makes it possible to install large diameter PHC piles in the areas of stiff soil/rock strata, which directly widens the application of large diameter PHC piles.

In practice, the welding method is commonly used in the connection of the adjacent piles during the construction of DPC piles. The welding operation always costs a lot of time and the quality of welding depends on the skill level of welders [5]. If the length of the cooling period after the completed welding process is not large enough, the continuous sinking of the piles may lead to the quenching fracture of the high-temperature welds immersed in the underground water. Moreover, such defects in quality are hard to be detected in time [6]. In addition, the high temperature surrounding the welds not only weakens the mechanical properties of the concrete [7]-[9], but also hinders the anti-corrosion treatment of the metal parts. All these mentioned issues will make significant effects on the service life of the PHC piles.

The existing mechanical quick couplings [10] are also not suitable for the construction of DPC piles. For example, many mechanical engagement couplings have the disadvantages of complex structure, high cost, and internal spring easy to be rusted and damaged. Moreover, the mechanical engagement couplings cannot satisfy the sealing demand of DPC piles so that the grouting pressure cannot be guaranteed. Mechanical flanges, fasteners, and threaded couplings should be provided with an enlarged end, which increases the resistance and inconvenience for the installation of the DPC piles.

Therefore, a kind of emerging mechanical coupling, consisting of sleeves, bolts and anchoring bars, is introduced in this paper. The coupling is economical, practical and easy for construction. Meanwhile, it can make the pipe connections strictly sealed, which is beneficial to the accomplishment of the grouting procedure. The finite element method is employed to study the influences of key parameters of the proposed coupling on its mechanical properties, which provides important references for its safety evaluation, as well as the basis to improve its performances.

2. Structures of the quick coupling

In order to adapt to the unique feathers of DPC piles, such as quick construction and high degree of mechanization, a new kind of coupling is designed in this paper to connect the adjacent PHC piles, which mainly relies on the bolts embedded in the concrete pipe pile, steel sleeves and anchoring bars, as shown in Figure 1.

Compared with the traditional welding connection, the steel sleeves increases the bearing capacities of the connection under the axial compressive loads, and avoids severe stress concentration at the traditional welds. The combination of anchoring bars and micro-expansion concrete [11] can increase the tensile strength of the coupling, to make the coupling applicable to the geological conditions with large buoyancy. As it is unfavourable to the in-situ assembly if an integral steel sleeve is employed, three 120-degree arc steel rings, which are hinged successively with fasteners designed to complete the whole loop, are adopted in our invention, as shown in Figure 1(b).

During the construction of DPC piles, the diameter of the bored hole is slightly larger than that of the PHC piles to achieve synchronously sinking of the piles with the bored hole, which also decreases the shaft resistance. Although embedding the pile toe into moderately or slightly weathered rock strata can improve the end bearing capacity of the pile foundation, to make up for the mentioned losses in shaft resistance, the gap between the outer surface of PHC piles and bored hole wall is grouted with cement paste to enhance the adhesion between pile concrete and surrounding strata. Therefore, it is should be guaranteed that the pile connections are strictly sealed so that the flowing cement paste is prevented from permeating into the core of the PHC piles. The sealing performance is the key factor to the maintenance of grouting pressure and the mechanical quality of the grouted interface. As a result, a circumferential bar and corresponding notch are designed on the end of the PHC piles and the steel sleeve, respectively, to achieve the sealing target of the proposed coupling, as shown in Figure 1(c).

On one hand, since the manufacturing accuracy of the PHC piles is inferior to that of the metal products, if the reserve holes on the steel sleeve exactly match the bolts embedded in the PHC piles, it is difficult to achieve accurate installation of the steel sleeve depending on current technology unless large increase in budget is acceptable. In fact, the bolts are uniformly welded on the circular stirrups of the PHC piles. The circumferential position of the bolts can be guaranteed, as the bolts are not far from the end plates of the PHC piles, which is accurately manufactured to locate the rebars. However, due to the deformation of the stirrups during the solidification of the concrete, the position of bolts along the pile length direction cannot be guaranteed. Therefore, the reserve holes on the steel sleeve corresponding to the bolts are designed as kidney-shaped holes. On the other hand, in order to guarantee the success of pile sinking and the integrity of the grouted interface, the radius coordinate of the remote end of the bolts relative to pile axis is smaller than the diameter of the pile (i.e., the bolts are embedded in kidney-shaped holes in the radial direction).

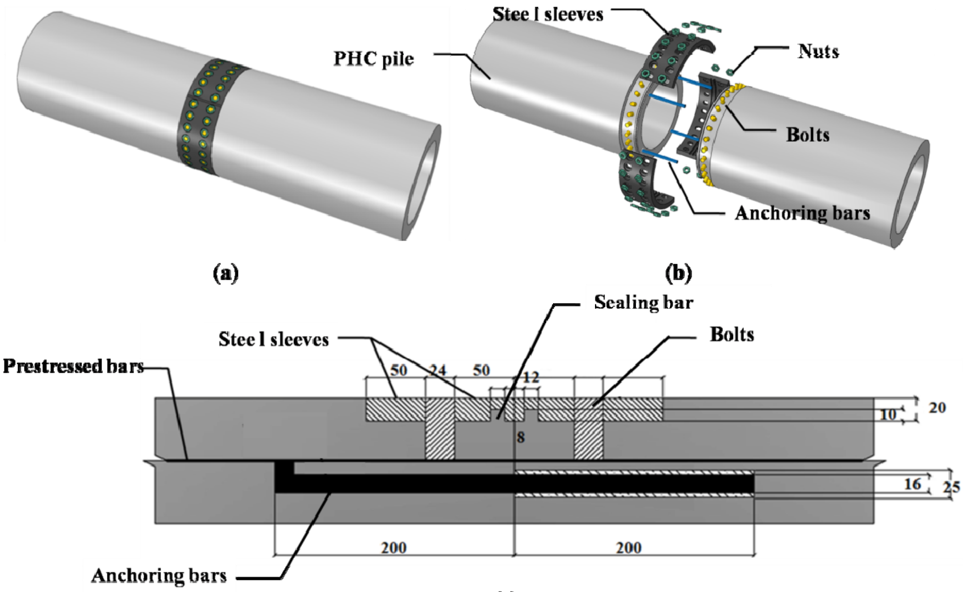

Figure 1. The (a) schematically diagram, (b) assembly structure and (c) profile of the quick coupling for Drilled and Prestressed High-strength Concrete Cased Piles

3. Analysis on the mechanical behaviours of the quick coupling

As elaborated in the Chinese national standard Pretensioned spun concrete piles (GB 13476-2009)[12], the bearing capacity of the coupling joints between the piles should be larger than that of the pile itself. As the strength of the steel components is far larger than that of the concrete, the replacement of the steel coupling to parts of the concrete pile body is beneficial to increase the compressive bearing capacity. Therefore, in this paper, the compressive bearing capacity of the coupling will not be discussed. Although the shear tests is not necessary regarding to the coupling, to make a comprehensive safety evaluation of the proposed structures, detailed verification and analysis are carried out subsequently from the aspects of shear, tensile and bending bearing capacities. The model "PHC800B110" is chosen as an example, the main parameters of which are listed in Table 1.

Table 1. Main parameters of the "PHC800B110"

Parameters	Value
Diameter D (mm)	800
Thickness t (mm)	110
Diameter of the circle where prestressedreinforcements locateD_p (mm)	690
Grade of concrete	C80
Prestressed reinforcement	30φ10.7
Ultimate bending bearing capacity (kN·m)	971
Ultimate tensile bearing capacity (kN)	2700
Ultimate shear bearing capacity (kN)	491

3.1. Shear bearing capacity

When the coupling is under pure shearing loads, the main load bearing components include steel sleeves and anchoring bars. Since the area of the cross-sectional of the anchoring bars is far smaller than that of the steel sleeves, the maximum shear stress in the steel sleeve can be approximated to

$$\tau_{\max} = 2.0\frac{F_{\mathrm{Q}}}{A} \tag{1}$$

where $A=(D^2-d^2)/4$ is the cross-sectional area of the steel sleeve, D and d are the external and inner diameters of the steel sleeve respectively, and F_Q refers to the shear force applied on the cross section.

Considering of "PHC800B110" piles, if the designed steel sleeves made of Q235 mild steel with its inner and external diameters as 760 mm and 800 mm respectively (i.e., thickness of 20 mm), under the load equal to the ultimate shear bearing capacity of the piles, namely F_{Q} =491 kN, the maximum shear stress in the steel sleeves is $\tau_{\max}$=4.12 MPa，which is much lower than the design value of the yield stress τ_c(=120 MPa) of the used material (Q235). Furthermore, rewriting Eq.(1) gives $A>2F_{\mathrm{Q}}/\tau_{\max}$, which indicates that the minimum thicknesses of the steel sleeves for different types of PHC piles, as shown in Table 2. Therefore, the thickness of the steel sleeves can be designed larger than 5 mm, 3 mm, and 2 mm, respectively, when the type of mild steel is Q235, Q460 and Q620, where the effects of pile diameter is ignored to facilitate the industrial manufacturing. Table 3 lists the mechanical properties of the mild steel of different models.

Table2. Minimum thickness of the steel sleeve required by shear capacity

External diameter D (mm)	Thickness t (mm)	Type	F_{Q} (kN)	$\delta_{\min}$ (mm)		
				Q235	Q460	Q620
800	110	A	384	2.556	1.361	1.020
		AB	431	2.870	1.528	1.145
		B	491	3.271	1.741	1.305
		C	551	3.673	1.955	1.465
	130	A	433	2.883	1.535	1.151
		AB	485	3.231	1.720	1.289
		B	553	3.686	1.962	1.470
		C	622	4.148	2.207	1.654
1000	130	A	574	3.056	1.628	1.220
		AB	648	3.451	1.838	1.378
		B	729	3.885	2.068	1.550
		C	785	4.184	2.227	1.669
1200	150	A	783	3.473	1.850	1.387
		AB	880	3.905	2.080	1.559
		B	1017	4.515	2.404	1.802
		C	1096	4.868	2.591	1.942

Table 3. The tensile strength of the mild steel of different type

Grade of steel	Yield strength (MPa)	Ultimate strength (MPa)	Strain corresponding to ultimate strength
Q235	225	370	0.209
Q460	440	550	0.128
Q620	600	750	0.0822

3.2. Tensile and bending bearing capacities

When bending moments and axial tensile loads are applied on the piles, all of the bolts, steel sleeves and anchoring bars share the loads at the same time. The finite element method by ABAQUS [13] is conducted to simulate the mechanical behaviour of the coupling, which is assistant for materials selection and parameter design. All the relevant parameters involved in the simulation are strictly consistent with that given in the Chinese national building standard design atlas Prestressed concrete pipe pile (10G409) [14].

3.2.1. Constitutive models of materials

The concrete damaged plasticity (CDP) model to describe the constitutive relation of the pile concrete can be divided into the following two categories:

(1) The concrete covered by the coupling

The concrete covered by the coupling will experience elastic stage, hardening stage and softening stage when the compressive load is applied. The elastic modulus (E_c) and Poisson ratio (ν_c) during the elastic stage are 38 GPa and 0.2, respectively. The linear hardening model is employed where the lower and upper limitation of the stress-strain relation are the yield strength (σ_{yc}=56.22 MPa) and ultimate strength (σ_c=80.32 MPa, ε_{cp}=6.3×10^{-4}). The residual strength after the softening stage is 4.29 MPa. Under tensile loads, the constitutive model just includes elastic and softening stages, where the elastic modulus is same as that of compression loading and the ultimate tensile stress is 3.14 MPa. The nonassociated potential plastic flow parameters used to define the concrete damaged plasticity of the concrete are shown in Table 4.

Table 4. The main parameters to define the concrete damaged plasticity of the pile

Parameters	Value
The dilation angle measured in the *p–q* plane at high confining pressure	12°
The eccentricity that defines the rate at which the function approaches the asymptote	0.1
The ratio of initial equi-biaxial compressive yield stress to initial uniaxial compressive yield stress	1.16
The ratio of the second stress invariant on the tensile meridian to that on the compressive meridian	0.6667

(2) The concrete outside the covered area by the coupling

To make sure that the system can continuously sustain loads larger than its corresponding (tensile or bending) bearing capacities to further obtain the generalized ultimate strength of the coupling, an elastoplastic model with linear hardening and infinite ultimate strength is adopted as the compressive constitutive relation of the concrete outside the covered area of the coupling, where the elastic modulus in elastic stage and the tangential modulus in plastic stage are the same as these mentioned above. Besides, the constitutive model of the concrete outside the covered area under tensile stress is elastic.

(3) Steel components

The constitutive model of the steel components, including bolts, steel sleeve and anchoring bars, is assumed to be elastoplastic with linear hardening, the key parameters of which are consistent with that given in Chinese national standard.

3.2.2. Contact interactions

The smooth model free of interfacial friction and the hard-contact model (normal displacement is continuous at the interface) in the tangential and the normal directions are used to define the contacts between the steel components and PHC piles as well as the ends of two adjacent PHC piles.

The reinforcement of the pile, the bolt (partly) and the anchoring bars (partly) are embedded in the concrete pile using the "Embed" model. The embedded nodes lies in the corresponding host element of concrete, the translational degrees of freedom of the embedded nodes are constrained to the interpolated values of that of the host element.

3.2.3. The method to apply prestress

The cooling method is employed to apply prestress in the model. The expansion coefficient of the concrete is assumed to be 0, and that of the steel is $\alpha = 1 \times 10^{-5}$℃$^{-1}$. The temperature difference to apply the prestress is determined as

$$\Delta t = \frac{1}{\alpha}\left[\frac{\sigma_{\mathrm{con}} A_{\mathrm{con}}}{E_s A_s} + \frac{\sigma_{\mathrm{con}}}{E_c}\right] \tag{2}$$

where σ_{con} is effective compressive prestress applied in the concrete, A_{con} and A_{s} are the area of the concrete and prestressed bars at the pile section. It is worth special noting that, in order to eliminate the additional stress in the bolts and corresponding reserved holes due to the shrinkage of prestressed bars during the cooling process, the prestress should be applied before the joint is assembled.

3.2.4. Verification of tensile bearing capacity

The ultimate tensile bearing capacity of the "PHC800B110" pile is 2700 kN. The elastic and ultimate bearing capacity of the coupling is defined as the maximum load without any irreversible plastic deformation and fracture in the coupling, respectively. Table 5 gives the elastic and ultimate tensile bearing capacity of the proposed coupling with the steel of different model as well as the corresponding maximum stress in different parts. It is found that only the elastic tensile bearing capacity of the coupling with Q235 steel is smaller than the ultimate tensile bearing capacity of the pile. However, the ultimate tensile bearing capacity of the coupling with Q235 steel is still greater than that of the pile body, which satisfies the clauses in corresponding standards and technical codes. If Q460 or Q620 steel is used, the stress in the coupling is still within the elastic range when the axial tensile force is as large as the ultimate tensile bearing capacity of the pile.

Table 5. Tensile bearing capacity of the proposed coupling made of the steel with different models

Grade of steel	Tensile bearing capacity (kN)	Maximum stress in the steel sleeve (MPa)	Maximum stress in the bolt (MPa)	Maximum stress in the anchoring bars (MPa)
Q235	(E) 1620	80	169	225
	(U) 4860	230	263	370
Q460	(E) 3510	280	400	440
	(U) 8100	445	490	550
Q620	(E) 4320	380	580	600
	(U) 10800	625	730	750

*（E:elastic bearing capacity; U:ultimate bearing capacity）

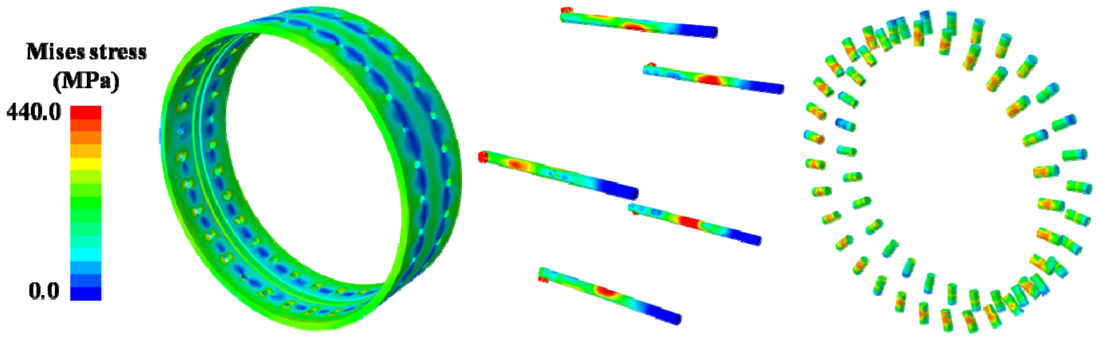

Figure 2. Thestress distribution in the proposed coupling made of Q460 under the ultimate tensile bearing capacity of the PHC piles

Figure 2 shows the distribution of the Von Mises stress in the coupling made of Q460 steel under the ultimate tensile bearing capacity of the PHC piles (i.e., 2700 kN). The stress is evenly distributed in each couple of reserve hole and bolt, where the maximum value is located at the side contrary to the pile contacts due to the extrusion between them. On the other hand, it is also indicated that the maximum stress in the assembled coupling is always located in the weakest middle section of anchoring bars aligned with the contacts between the connected piles. This is because no tensile stress can be achieved between the connected piles and only the steel sleeve and the anchoring bars share the axial tensile load at this section. Although the material properties of the steel sleeve and the anchoring bars are the same, the Poisson's ratio difference between the concrete and steel leads to additional pressure at the side of the anchoring bars remote to the pile axis.

3.2.5. Verification of bending bearing capacity

To obtain the bending bearing capacity, the bending moment is applied within the area of 400 mm from the two simply supported ends of the connected piles. The ultimate flexural bearing capacity of the “PHC800B110” pile is 971 kN m. Table 6 shows the elastic and ultimate bending bearing capacity of the proposed coupling with the steel of different model as well as the corresponding maximum stress in different parts. It is found that only when Q235 or Q460 is used, the elastic bending bearing capacity of the coupling is smaller than the ultimate bending bearing capacity of the PHC piles. Otherwise, the calculated bearing capacity of the couplings are larger than the ultimate bending bearing capacity of the PHC piles.

Table 6. Bending bearing capacity of the proposed coupling made of the steel with different models

Grade of steel	Bending bearing capacity (kN)	Maximum stress in the steel sleeve (MPa)	Maximum stress in the bolt (MPa)	Maximum stress in the anchoring bars (MPa)
Q235	(E) 437.0	90	140	225
	(U) 1456.5	230	252	370
Q460	(E) 776.8	240	380	440
	(U) 2330	448	500	550
Q620	(E) 1165.2	370	580	600
	(U) 3107.2	630	720	750

*（E:elastic bearing capacity; U:ultimate bearing capacity）

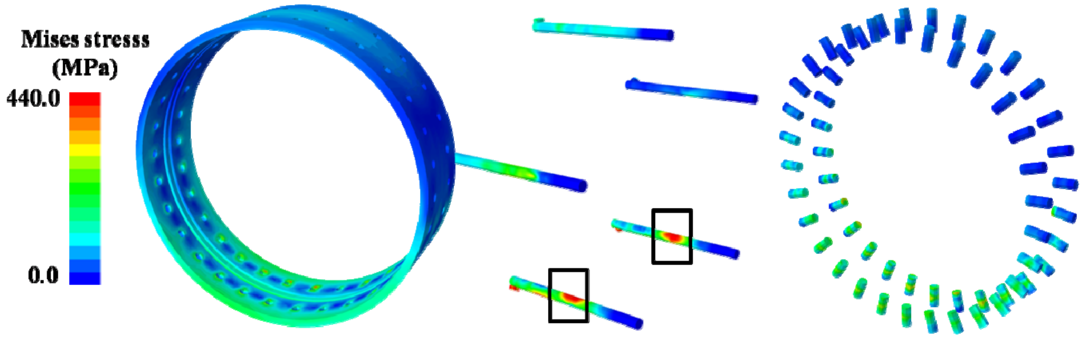

Figure 3. Thestress distribution in the proposed coupling made of Q460 under the ultimate bending bearing capacity of the PHC piles

Figure 3 shows the distribution of the Mises stress in the coupling made of Q460 steel under the ultimate bending bearing capacity of the PHC piles (i.e., 971 kN·m). For the sleeve, obvious stress concentration can be found at the outer edge of the tension half. The stresses here can be mainly divided into two parts: 1) the tensile stress due to bending and 2) the circumferential shear stress due to the difference of Poisson's ratio between the pile concrete and the steel sleeve. As no tensile stress can be generated between the bolts and reserved holes, the maximum stress in the bolts always presents at the side related to compressive contact (i.e., remote side on the tensile half and the proximal side on the compressive half). Under bending load, the maximum stress in the anchoring bars also appears weakest middle section of aligned with the contacts between the connected piles, but the stress of that in the tensile half is much larger than that on the pressure side. This is because, on the compressive half, the contacts between PHC piles can provide compressive stress to balance the moment, while the effective area in the tensile half(i.e., only the anchoring rod and the steel sleeve) is greatly reduced.

3.3 Discussion and optimization

Firstly, the diameter and the number of bolts contribute a lot to the tensile bearing capacity of the coupling, but the shear deformation of the bolts is not economic when compared to the tensile stress in the anchoring bars because the tensile strength of the mild steel is always larger than shear strength. However, the axial stress along the anchoring bars transferred from the solidified micro-expansion concrete cannot be accurately estimated. Therefore, the diameter and the number of bolts could be determined based on the requirement that the axial bearing capacity of coupling should be greater than that of the PHC piles. Meanwhile, due to the thickness limitation of the concrete cover, it is difficult to increase the diameter of the anchor bars. Increasing the number of anchor bars may lead to obvious damage to the pile end concrete with anchoring hole and extra issues related to the assembly precision. On the other hand, based on the theoretical analysis and FEA results of the tensile, shear, and bending bearing capacities, the performance of the coupling under shear and tensile loads is higher than that under bending load. Therefore, the steel sleeve, as the main component of the couplings, is the target to be optimized in this section, where its width and thickness are systematically altered to analyse their effects on the bending bearing capacity of the coupling.

According to the FEA results above, if the Q460 steel is used, the ultimate tensile bearing capacity of the coupling is higher than the ultimate tensile bearing capacity of the PHC piles, and the elastic bending bearing capacity reaches 79.9% of the ultimate tensile bearing capacity of the piles. Meanwhile, the ultimate bending bearing capacity of the coupling is about 2.40 times of that of the piles, which indicates sufficient security. Considering that the compressive and bending deformation is dominant during the service of the pile foundation, the Q460 steel is therefore chosen for subsequent analysis.

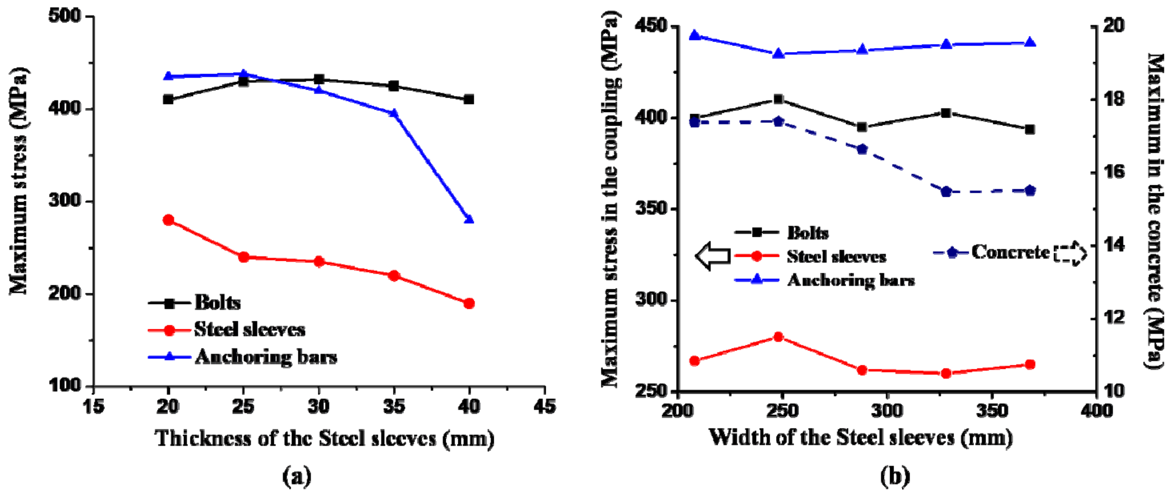

Figure 4. The stress distribution of the proposed coupling (Q460) under the ultimate flexural bearing capacity of the pile

Under the ultimate bending moment of the "PHC800B110" pile (i.e., 971 kN•m), Figs. 4(a) and (b) show the maximum Von Mises stress in the main components of the coupling, including the bolts, the steel sleeve and the anchoring bars versus the thickness and the width of the steel sleeve, respectively. When the width of the steel sleeve keeps constant as 248 mm, the increasing of its thickness and sectional stiffness, the maximum stress in the steel sleeve induced by the bending moment decreases accordingly. In the tension half of the system, the rapid increase of the steel sleeve area, which is efficient to balance the bending moment, is beneficial to reduce the stress level in the anchoring bars. And no significant fluctuation of the stress in the bolts can be found. If the thickness of the steel sleeve exceeds 30 mm, the growing up costs with further increasing of the thickness cannot enhance the bearing performance equivalently. Therefore, the optimal thickness is suggested to range from 25 mm to 30 mm. When the thickness of the steel sleeve is maintained as 20 mm, no obvious change in the maximum stress of the main components of the coupling can be found. However, since the increasing of the sleeve width enlarges the area for load transferring between the concrete and the sleeve, which relieves the stress concentration effect in the surrounding concrete. About 20 discount of the average stress in the concrete covered by the coupling is obtained when the sleeve thickness increases from 208 mm to 328 mm. On the other, when the width of the steel sleeve is large enough, the effective area for load transferring cannot be increased. Therefore, the optimum of the sleeve width ranges from 320 to 360 mm.

4. Conclusions

An emerging coupling with simple structure and higher construction efficiency is designed specially the construction of drilling and PHC pipe cased (DPC) piles in this paper. The load bearing capacity of the proposed coupling meets well with the requirements in corresponding standards and specifications. Meanwhile, it makes the connection between the PHC pile strictly sealed, which is benefit to the grouting of cement slurry. FEA is employed to reveal the underlying mechanism when it is under different load conditions. The load bearing capacity of the above coupling made of steel with different grades is verified, and its main design parameters are optimized, which can provide theoretical basis for its practical engineering applications. Based on the analysis, the following conclusions can be drawn:

(1) The emerging coupling proposed in this paper has higher shear bearing capacity than the bending and tensile bearing capacities. If the thickness of the steel sleeve exceeds 5 mm, the coupling could meet the requirements that its shear bearing capacity is greater than that of the pile body.

(2) The elastic bending and tensile bearing capacity of the coupling made of Q460 is about 2.4 and 0.8 times of the ultimate value of the PHC piles. Moreover, the ultimate tensile bearing capacity of the coupling made of Q460 is larger that of the PHC piles. Therefore, considering that the compressive and bending deformation is dominant during the service of the pile foundation, the Q460 steel is recommended to chosen.

(3) The increasing of sleeve thickness and width within a certain range provides more effective area to balance the bending moment, which can lower the stress in the anchoring bars, and enlarges the area for load transferring between the concrete and the sleeve, which relieves the stress concentration effects in the surrounding concrete. According to the mechanism analysis and corresponding FEA results, the optimal thickness and width of sleeve is suggested to range from 25 mm to 30 mm and from 320 to 360 mm, respectively.

Acknowledgments
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Reference

[1] Tang M 2009 The development and engineering of drilling with PHC pipe cased pile *Guangzhou Arch.* **37(5)** 3-7
[2] Tang M, Qi Y, Zhou Z and Hu H 2013 Comparative study on bending performance between hollow PHC pipe piles and PHC pipe piles with concrete core *Chinese J. Geotech. Eng.* **35(S2)** 1075-1080
[3] Tang M, Chen R 2010 One construction method for drilling with pipe cased piles used as the foundation of buildings and bridges Patent of China, ZL200710029023.5.
[4] Zhang F, Ai L, Zhu K, Tang Y and Zhu J 2010 Experiment of improving crack-resistance of pile concrete *New Build. Mater.* **37(11)** 18-21
[5] Li M, Wang Y, Wang Z and Qiao K 2017 Study on application of stainless steel in constructional engineering and its welding property. *New Build. Mater.* **44(4)** 75-77.
[6] Liu F, Jia L and Li C 2008 The test study on welding joint flexural bearing capacity of prestressed concrete hollow square pile *J. Wuhan Univ. Tech.* **30(5)** 105-108.
[7] Xu W 2004 Production of end plates of prestressed concrete pipe pile *Arch. Door-Window and Metal Build. Mater.* **3** 68-68
[8] Shi M 2004 Development of new bowl-shape connector for PHC piles, *Port & Waterway Eng.* **1** 143-148
[9] Zhang Y and Liang J 2007 Development of new type of U joint applied in spun pile. *Concrete,* **9** 83-85
[10] Shi S Y, Yang H. Type and influence control of connected pile for pre-stressed pipe pile. *Coal Eng.* 2011, **9:** 45-47
[11] Wu Z, Zhang X and Song C 2012 Test research on properties of impervious slight expansive concrete. *New Build. Mater.* **12** 55-57
[12] GB 13476 2009 *Pretensioned spun concrete piles* (Beijing: China Architecture & building Press)
[13] ABAQUS 2009 *Analysis user's manual v.6.9* (Pawtucket, RI: DassaultSystèmes)
[14] 10G409 2010 *Prestressed concrete pipe piles* (Beijing: China Architecture & building Press)