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现代隧道技术

冻结裂隙砂岩的力学特性研究

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摘 要 为得出围压、裂隙倾角、冻结温度等因素对砂岩力学性能的影响规律,采用三轴压缩试验方法和理论分析方法,研究了不同围压下、无裂隙0°,15°,30°和45°倾角裂隙砂岩在-5 ℃,-10 ℃和-15 ℃冻结时的强度、内摩擦角和粘聚力变化特征。结果表明:无裂隙和有裂隙砂岩的强度,随冻结温度的降低而非线性增强,随围压的降低而近线性降低,随裂隙倾角的增大而减小;粘聚力随冻结温度的升高而减小,内摩擦角和粘聚力均有随裂隙倾角的增加而先减小后增大的变化趋势;无裂隙,0°,15°倾角裂隙砂岩的内摩擦角均随冻结温度的升高而减小,30°倾角裂隙砂岩的内摩擦角随冻结温度的降低先增大后减小,45°倾角裂隙砂岩的内摩擦角则随冻结温度的降低先减小后增大。

关键词 冻结裂隙砂岩 冻结温度 裂隙倾角 围压 三轴压缩试验
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1 引 言

岩体的力学性质直接影响岩体工程的设计、施工以及后期的运营,特别是对寒区隧道、路基等的影响更甚[1~3]。国内外学者对冻结岩体力学性能进行了大量研究,杨更社等[4,5]以白垩系饱和砂岩为研究对象,研究了冻结岩石三轴试验强度,得出了岩样的粘聚力和内摩擦角等物理力学特性。刘 莹等[6]研究了不同温度下砂岩的物理力学性质,得出了岩石的强度和变形特性。王岳嵩[7]进行了含裂隙砂岩的单向冻结试验,并对试件的温度场进行数值模拟,分析了单向冻结条件下含裂隙砂岩温度分布情况。刘 波等[8]通过数码显微镜观察分析了红砂岩的颗粒、孔洞分布特征,并进行了含冰软弱面的冻结裂隙红砂岩的强度试验。李 宁等[9,10]对冻结裂隙砂岩疲劳现象进行了研究,得出冻结裂隙砂岩样疲劳特性与砂岩的冻结温度和裂隙大小角度等参数有关。刘 慧等[11]研究了裂隙水冻结过程对岩石细观结构的影响规律。还有学者[12~17]进行了砂岩和泥岩力学试验研究,丰富了冻结岩石的试验数据,为工程设计和施工提供了一定的参考,但对含裂隙冻结砂岩的力学性能研究不够深入,还不能满足工程需要。

因此,采用三轴压缩试验和理论分析方法研究围压、温度、裂隙倾角对砂岩力学性能影响规律,以期对寒区的岩质边坡、隧道、公路路基等防护技术提供理论基础。

2 试样制备和试验方案

2.1 试样制备

试验用砂岩试样均按照国家标准进行加工,试样高和直径分别为100 mm和50 mm,砂岩试样如图1所示。为预制砂岩倾角裂隙为0°、15°、30°和45°的试样,在砂岩试样中间位置用锯把砂岩试样按照预制裂隙倾角切割。然后拼合切割后的试样,为使试样侧面平直,上下端面和轴向垂直,用直角尺严格检查。将拼合后无问题的试样用热缩膜包裹成一体,测量此时试样的高度,并将试样轴向拉出,控制位移尺寸为4 mm,形成4 mm的轴向裂缝宽度。将形成裂缝的试样放入清水中,并穿透热缩膜向裂缝中注满清水,静置一段时间后将热缩膜包裹的试样用保鲜膜密封,分组放入3个冷冻柜中,温度依次调节

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图1 加工试样

Fig.1 Processing samples

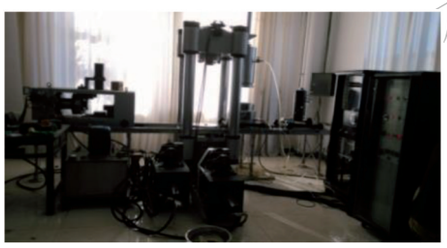


图2 试验机

Fig.2 Testing machine

为-5 ℃,-10 ℃和-15℃,分别冷冻24 h后取出试样,将热缩膜去除,完成不同温度下冻结的不同倾角裂隙砂岩试样的制备。

2.2 试验方案

为得出冻结裂隙砂岩的物理力学性质及围岩、裂隙倾角、冻结温度等因素对其的影响规律,砂岩试样冻结温度选取-5 ℃,-10 ℃和-15 ℃ 3种温度,砂岩试样裂隙倾角选取0°,15°,30°,45°和无裂隙5种类型。试验采用TAW-2000型电液伺服岩石三轴试验仪,如图2所示,试验机的位移加载速率选取0.4 mm/min。隧道埋深一般不超550 m,根据重液公式P=0.013H计算得到试样侧向荷载为7.15 MPa,所以试验选取了4种围压强度,分别为0 MPa,2 MPa,4 MPa和8 MPa。

3 试验结果分析

在围压为0 MPa,2 MPa,4 MPa和8 MPa下,分别测定了无裂隙,0°,15°,30°和45°倾角裂隙砂岩分别在-5 ℃,-10 ℃和-15 ℃冻结时的强度,结果如表1~表5所示。

从试件破坏形态看,无裂隙和0°倾角裂隙冻结砂岩试样有相类似的破坏形态,因此可根据摩尔-库伦准则$\tau=\sigma\tan\varphi+c$计算冻结砂岩的内摩擦角和粘聚力2个力学参数。计算方法如下:先根据围压和强度拟合线性方程,假设拟合方程的斜率为k,直线与竖轴(强度)的截距为b;再根据内摩擦角$\varphi=\arcsin[(k-1)/(k+1)]$,粘聚力$c=b\times(1-\sin\varphi)/(2\cos\varphi)$,计算无裂隙和0°倾角裂隙冻结砂岩内摩擦

表1 冻结无裂隙砂岩强度

Table 1 Strength of frozen fracture-free sandstone

温度/(℃)	极限强度/MPa			
	0	2	4	8
-5	38.02	41.11	45.00	52.01
-10	40.41	45.00	52.32	60.85
-15	44.78	48.77	56.23	69.03

表2 冻结0°倾角裂隙砂岩强度

Table 2 Strength of frozen fractured sandstone with 0° dip angle

温度/(℃)	极限强度/MPa			
	0	2	4	8
-5	18.68	22.59	27.61	37.18
-10	24.58	28.77	37.25	41.86
-15	30.24	35.94	41.05	50.27

表3 冻结15°倾角裂隙砂岩强度

Table 3 Strength of frozen fractured sandstone with 15° dip angle

温度/(℃)	极限强度/MPa			
	0	2	4	8
-5	14.95	18.45	23.34	30.96
-10	16.74	23.42	27.35	35.45
-15	23.26	27.43	31.48	41.65

表4 冻结30°倾角裂隙砂岩强度

Table 4 Strength of frozen fractured sandstone with 30° dip angle

温度/(℃)	极限强度/MPa			
	0	2	4	8
-5	12.58	16.12	20.23	24.83
-10	14.56	18.09	21.56	26.74
-15	18.05	21.58	25.12	30.58

表5 冻结45°倾角裂隙砂岩强度

Table 5 Strength of frozen fractured sandstone with 45° dip angle

温度/(℃)	极限强度/MPa			
	0	2	4	8
-5	11.94	14.77	18.15	22.78
-10	15.03	17.43	21.39	26.28
-15	17.06	19.56	22.34	27.34

角和粘聚力，结果见表6、表7。

由于冻结条件下裂隙砂岩中存在层状冰，而冰的单轴强度较低，可以将其看成软弱结构面，则15°，30°和45°倾角裂隙砂岩也符合摩尔-库伦准则，可计算15°、30°和45°倾角裂隙砂岩内摩擦角和粘聚力，结果见表8~表10。

表6 无裂隙冻结砂岩内摩擦角和粘聚力

Table 6 Internal friction angle and cohesion of fracture-free frozen sandstone

温度/(℃)	拟合方程	内摩擦角/(°)	粘聚力/MPa
−5	$y = 2.47x + 37.85$	25.08	12.04
−10	$y = 2.60x + 40.54$	26.36	12.27
−15	$y = 2.72x + 43.82$	27.52	13.29

表7 0°倾角裂隙冻结砂岩内摩擦角和粘聚力

Table 7 Internal friction angle and cohesion of fractured frozen sandstone with 0° dip angle

温度/(℃)	拟合方程	内摩擦角/(°)	粘聚力/MPa
−5	$y = 2.34x+18.33$	23.64	5.99
−10	$y = 2.42x+25.33$	24.52	8.14
−15	$y = 2.49x+30.68$	25.28	9.72

表8 15°倾角裂隙冻结砂岩内摩擦角和粘聚力

Table 8 Internal friction angle and cohesion of fractured frozen sandstone with 15° dip angle

温度/(℃)	拟合方程	内摩擦角/(°)	粘聚力/MPa
−5	$y= 2.03x+14.83$	19.88	5.20
−10	$y= 2.27x+17.79$	22.95	5.89
−15	$y = 2.30x+22.89$	22.19	7.55

表9 30°倾角裂隙冻结砂岩内摩擦角和粘聚力

Table 9 Internal friction angle and cohesion of fractured frozen sandstone with 30° dip angle

温度/(℃)	拟合方程	内摩擦角/(°)	粘聚力/MPa
−5	$y= 1.36x+12.15$	8.77	5.22
−10	$y = 1.43x+15.01$	10.19	6.28
−15	$y= 1.29x+17.06$	7.47	7.48

表10 45°倾角裂隙冻结砂岩内摩擦角和粘聚力

Table 10 Internal friction angle and cohesion of fractured frozen sandstone with 45° dip angle

温度/(℃)	拟合方程	内摩擦角/(°)	粘聚力/MPa
−5	$y= 1.53x+13.08$	12.12	5.29
−10	$y= 1.51x+14.94$	11.72	6.08
−15	$y= 1.56x+19.37$	12.71	7.75

3.1 冻结温度对裂隙砂岩力学性能影响

由表1可以看出：(1)围压0 MPa时，无裂隙砂岩在−5 ℃，−10 ℃和−15 ℃时的强度分别为38.02 MPa、40.41 MPa和44.78 MPa；(2)围压2 MPa时，无裂隙砂岩在−5 ℃，−10 ℃和−15 ℃时的强度分别为41.11 MPa，45.00 MPa和48.77 MPa；(3)围压4 MPa时，无裂隙砂岩在−5 ℃，−10 ℃和−15 ℃时的强度分别为45.00 MPa，52.32 MPa和56.23 MPa；(4)围压8 MPa时，无裂隙砂岩在−5 ℃，−10 ℃和−15 ℃时的强度分别为52.01 MPa，60.85 MPa和69.03 MPa。从而可得，在相同围压情况下，无裂隙砂岩的强度随冻结温度降低呈非线性增强。

从表2~表5中分析可得：相同围压下，0°，15°，30°和45°倾角裂隙砂岩强度也均随冻结温度降低而非线性增强。

从表6~表10中分析可得：(1)−5 ℃，−10 ℃和−15 ℃下，无裂隙砂岩内摩擦角分别为25.08°，26.36°和27.52°，粘聚力分别为12.04 MPa，12.27 MPa和13.29 MPa；0°倾角裂隙砂岩内摩擦角分别为23.64°，24.52°和25.28°，粘聚力分别为5.99 MPa，8.14 MPa和9.72 MPa；15°倾角裂隙砂岩内摩擦角分别为19.88°，22.95°和22.19°，粘聚力分别为5.20 MPa，5.89 MPa和7.55 MPa；30°倾角裂隙砂岩内摩擦角分别为8.77°，10.19°和7.47°，粘聚力分别为5.22 MPa，6.28 MPa和7.48 MPa；45°倾角裂隙砂岩内摩擦角分别为12.12°，11.72°和12.71°，粘聚力分别为5.29 MPa，6.08 MPa和7.75 MPa。(2)无裂隙，0°，15°倾角裂隙砂岩的粘聚力和内摩擦角均随冻结温度升高而减小；30°和45°倾角裂隙砂岩的粘聚力均随冻结温度升高而减小；30°倾角裂隙砂岩的内摩擦角随冻结温度升高呈先增大后减小的变化趋势，45°倾角裂隙砂岩则呈先减小后增大的变化趋势。

3.2 围压对冻结裂隙砂岩力学性能影响

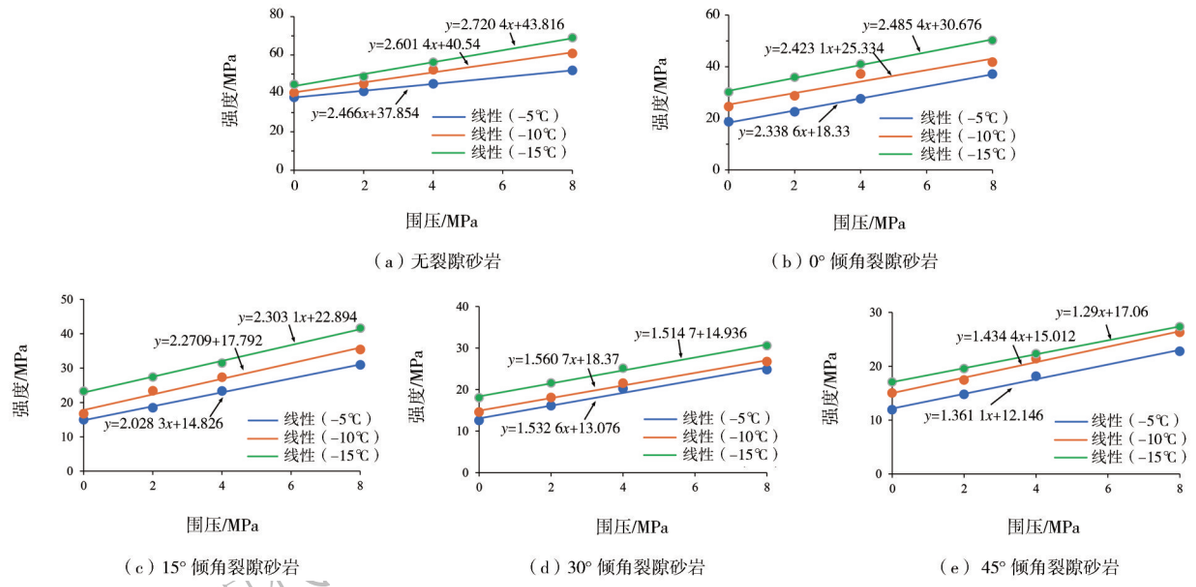
无裂隙和不同倾角裂隙砂岩在−5 ℃，−10 ℃和−15 ℃冻结时的强度与围压关系曲线如图3所示。

从表1~表5和图3中分析可得：砂岩在相同冻结温度情况下，围压对无裂隙和不同倾角裂隙砂岩的强度有相同的影响规律，即砂岩强度均随围压减小而近线性降低。

3.3 裂隙倾角对冻结砂岩力学性能影响

研究了冻结温度为−5 ℃，−10 ℃，−15 ℃和围压为0 MPa，2 MPa，4 MPa、8 MPa下冻结砂岩的强度与裂隙倾角的关系，结果如图4所示。

从表1~表5和图4中可得：砂岩裂隙倾角对强

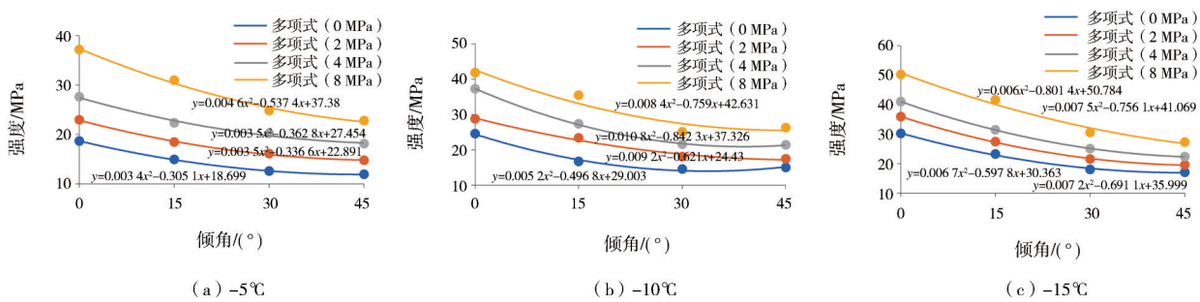


（a）无裂隙砂岩　（b）0° 倾角裂隙砂岩

（c）15° 倾角裂隙砂岩　（d）30° 倾角裂隙砂岩　（e）45° 倾角裂隙砂岩

图3　不同倾角裂隙冻结砂岩强度与围压关系曲线

Fig.3　Relationship curves between strength and confining pressure of fractured frozen sandstone with different dip angles



（a）-5℃　（b）-10℃　（c）-15℃

图4　冻结砂岩强度与裂隙倾角的关系曲线

Fig.4　Relationship curves between strength and dip angle of fractured frozen sandstone

度有较大影响。砂岩在相同围压和冻结温度情况下，0°~45°裂隙倾角内，砂岩强度随裂隙倾角增大而减小，但强度减小幅度随裂隙倾角增大而逐渐降低。

从表6~表10中可得出：砂岩在相同冻结温度下，内摩擦角和粘聚力均有随裂隙倾角增加而先减小后增大的变化趋势。

4　结　论

通过测试-5 ℃，-10 ℃，-15 ℃下冻结无裂隙，0°，15°，30°，45°倾角裂隙砂岩在不同围压下的力学性质，得到了如下结论：

（1）在相同围压条件下，无裂隙和有裂隙砂岩强度均随冻结温度降低而非线性增强。在相同冻结温度条件下，无裂隙和有裂隙砂岩强度均随围压的降低而近线性减小。

（2）相同围压和冻结温度下，砂岩强度随裂隙倾角增大而减小，但强度减小幅度随裂隙倾角增大而逐渐降低。在相同冻结温度下，内摩擦角和粘聚力均有随裂隙倾角增加而先减小后增大的变化趋势。

（3）无裂隙和有裂隙砂的粘聚力均随冻结温度升高而减小；无裂隙、0°、15°倾角裂隙砂岩的内摩擦角均随冻结温度升高而减小；30°倾角裂隙砂岩内摩擦角随冻结温度降低先增后减，45°倾角裂隙砂岩则先减小后增大。

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Study on the Mechanical Properties of Frozen Fractured Sandstone

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Abstract In order to derive the influence patterns of confining pressure, fracture dip angle and freezing temperature on the mechanical properties of the sandstone, this paper uses the triaxial compression test method and theoretical analysis method to study the changing characteristics of the strength, internal friction angle and cohesion of sandstones without fracture and fractured sandstones with dip angles of 0°, 15°, 30° and 45° at frozen temperatures of −5 ℃, −10 ℃ and −15 ℃ under different confining pressures. The results show that the strengths of fracture-free and fractured sandstones increase nonlinearly with decreasing of freezing temperature, decrease nearly linearly with decreasing of confining pressure, and decrease with increasing of fracture dip angle; the cohesions decrease with the increase of the freezing temperature, and both the internal friction angle and cohesion have the tendency to decrease first and then increase with the increase of the fracture dip angle; and the internal friction angles of fracture-free sandstone and fractured sandstone with 0° and 15° dip angles all decrease with the increase of the freezing temperature, the internal friction angle of fractured sandstone with 30° dip angle increases first before decreasing with the decrease of the freezing temperature, and the internal friction angle of fractured sandstone with 45° dip angle decreases first before increasing with the decrease of the freezing temperature.

Keywords Frozen fractured sandstone; Freezing temperature; Fracture dip angle; Confining pressure; Triaxial compression test

（上接第129页）

Study on Damage Behaviors of Tunnel Secondary Lining under Rheological Load

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Abstract The rheological effect of soft and weak surrounding rocks can easily lead to the cracking of tunnel secondary lining, affecting the safety and durability of the structure. In view of this, this paper takes the Dujiashan tunnel on Guangyuan-Gansu Expressway as an example and establishes a numerical calculation model based on Weibull damage distribution to investigate the damage evolution behaviors of the secondary lining under rheological load. The results show that: (1) if the secondary lining is in good contact with the surrounding rock, the lining cracking area is distributed near the arch waist, arch foot, as well as the top and bottom of the arch respectively when the lateral pressure coefficients are λ=0.5,1 and 2; (2) the cavity has a great impact on the lining cracking area and the order of the occurrence of the cracking, that is, the cracking is first induced in the concrete near the cavity, and the concrete at that specific location will all produce quite serious damage; and (3) the influence of the geostress on the damage patterns of the secondary lining with cavities can be divided into two kinds: the impact on the order of the occurrence of the damage location, but not on the final damage pattern of the structure, or the impact on both the order of the damage occurrence and the final damage pattern of the structure.

Keywords Tunnel; Secondary lining; Rheology load; Damage behaviors; Soft and weak surrounding rocks