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高聚物碎石桩加固花岗岩尾泥地基模型试验

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摘要:城市周边的矿坑回填治理是一种新兴的土地开发方式,利用矿区大量淤积的尾泥废料对废弃矿坑进行就地回填和整平形成尾泥地基,是矿区地表生态恢复和土地开发的潜在方案,可降低工程成本并解决尾泥堆积和存放等难题。高聚物碎石桩具有强度高、成型速度快且无需养护等优点,可满足复杂地基处理的需求。因此,本文对高聚物碎石桩加固花岗岩尾泥地基的有效性和可行性开展研究。首先,对花岗岩尾泥进行直剪试验和固结试验,获取其抗剪强度指标及压缩模量和压缩系数;而后,采用大型直剪仪对不同高聚物掺量的高聚物碎石试块进行抗剪强度试验,获取不同高聚物掺量碎石的抗剪强度;最后,设计未加固尾泥地基、碎石桩加固尾泥地基和高聚物碎石桩加固尾泥地基共计9组静载模型试验方案,分析高聚物碎石桩的桩长、桩径和高聚物掺量对尾泥地基加固效果的影响。结果表明:由于高聚物碎石桩桩体刚度更高且桩周具有棘突状的结构,其加固效果明显优于碎石桩。经过高聚物碎石桩加固后的花岗岩尾泥地基承载能力明显提高,相比于碎石桩加固尾泥地基,其最终沉降量减少了56.6%,高聚物碎石桩的加固效果随着桩长、桩径和高聚物掺量的增加而提高;在相同桩长、桩径下,高聚物掺量越大,桩底应力越小。

关键词:高聚物碎石桩;花岗岩尾泥;地基处理;静载模型试验

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中国经济的快速发展和城市边界的扩张导致土地资源紧张,城市周围的矿坑回填整平已成为一种新兴的土地利用方式^[1-2]。尾泥是选矿、开采和生产过程中产生的副产品,它与天然土体有所不同,是花岗岩开采及碎石加工过程中不可避免会产生的工业粉尘经过收集、水处理后形成的一种泥状废料,粒径多小于75 μm;大量的尾泥堆积不仅会对环境造成污染,还占据了大量空间^[3]。因此,将矿场内堆存的尾泥作为矿坑回填材料,可以降低工程成本并解决尾泥堆积问题。湖南省常德市鼎城区于2021年成功对6个石煤矿坑进行了覆土回填,同时利用真空预压固结对稀烂尾泥进行了初步治理^[4]。然而,经过初步治理后的尾泥承载能力相对较低,难以承受建筑荷载,易产生较大沉降。因此,需要对回填的尾泥地基进一步加固处理,从而形成具备基本

使用功能的建设用地,以增加土地的经济效益。

桩体复合地基技术是一种高效的解决地基强度不足和沉降过大等问题的新技术,已得到了广泛的工程应用^[5-6]。其中,碎石桩是目前复合地基中较为常用的地基治理方法之一,具有取材便捷、施工简便、经济性较好等优点^[7-9]。由于碎石桩桩体主要由碎石构成,不具备胶结刚度,如果桩周土体不能提供充分的握裹力,在竖向荷载作用下,碎石桩桩体易发生侧向鼓胀而损坏,进而降低整个复合地基的承载能力^[10-11]。长期以来,为了改善碎石桩的工作性能,提高复合地基承载力,国内外众多学者进行了大量研究。van Impe^[12]提出一种包裹碎石桩的改进方法,即用土工材料对碎石桩进行包裹,使之成为一个整体。赵明华等^[13]通过较大比例的室内模型试验发现,包裹材料能有效减小

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碎石桩的侧向变形,并提高桩体强度,使荷载能较好地传递至桩底。刘汉龙等^[14]提出一种浆固碎石桩,即利用水泥浆液将碎石胶结成坚固的桩体。左威龙等^[15]通过现场试验验证了水泥浆液的注入能明显改善桩和桩周的土体性质。刘汉龙^[16]和温世清^[17]等利用 PLAXIS 3D 计算分析,结果表明浆固碎石桩复合地基加固可以有效地控制软土地基沉降。虽然经过上述多种方式处理后的碎石桩的性质得到了不同程度的改善,但施工过程相对复杂,甚至会影响施工工期。

王复明等^[18]提出将高聚物浆液和碎石相结合,利用高聚物浆液反应将碎石胶结成整体,得到一种新型的高聚物碎石桩;并通过模型试验和有限元扩展分析发现,高聚物碎石桩的承载能力分别随高聚物密度、桩长、桩径的增加而提高。非水反应类高聚物材料是由异氰酸酯和多元醇两种组分按质量比例 1:1 反应生成,具有以下特点^[19]:1) 反应过程中无水参与,适应性好^[20-21];2) 具有较好的抗压性能^[22]和抗拉性能^[23-24];3) 成型较快,不需要养生(最快 10 余秒完成固化,15 min 左右即可达到最终强度的 90%);4) 良好的化学稳定性^[25-26]。高聚物碎石桩既能克服碎石桩适用范围较小的缺陷,又能解决常规水泥碎石桩养护时间较长的问题,因而具备较大的优越性。

王娟等^[27]开展高聚物碎石混合料单轴受压试验,结果表明,高聚物密度对高聚物碎石混合料的变形和强度有显著影响;随着高聚物密度的增加,高聚物混合料表现出更加脆性的特性,且其强度也有所增强。Ai 等^[28]利用数值计算的方法,得出骨料的形状和分布对加载下的高聚物碎石混合料试件的破坏行为有显著影响。Fang 等^[29]通过单轴压缩试验和扫描电镜试验发现,在微观尺度上,高聚物碎石混合料由基体高聚物、碎石和界面过渡区组成。高聚物碎石混合料和高聚物的强度和弹性模量均与基体高聚物的密度呈线性关系。刘勇^[30]进行了高聚物碎石混合料无侧限抗压强度试验和劈裂抗拉试验,结果表明,在碎石质量相同的情况下,高聚物密度较大的高聚物碎石混合料整体的弹性模量较高,劈裂抗拉强度较大。

综上所述,高聚物碎石桩具有强度较高、成型速度快且完工后无需养护等优点,可以满足复杂地基处理的需求,具有良好的应用前景和发展价值。因而该法适用于对尾泥地基快速加固,有利于适应城市用地建设的快速发展。本文采用模型试验研究该方法对花岗岩尾泥地基的加固效果,并分析了桩长、桩径和高聚物掺量对高聚物碎石桩加固花岗岩尾泥地基变形和承载特性的影响,总结探讨了高聚物碎石桩加固花岗岩尾泥地基的有效性和可行性,为废弃尾泥回填矿

坑的地基处理工程实践提供了重要的试验依据和指导。

1 模型试验

1.1 工程背景

本次模型试验针对的工作区位于福建东南沿海丘陵地带,属丘陵地貌。从 20 世纪 70 年代至 2010 年,由于超强度开采,在花岗岩矿山工作区留有凹式采坑百余处,最小的采场不足 100 m²,最大达数万 m²,采场边坡一般高 20~50 m,最高达百余米,坡度多大于 75°,局部近于直立。采坑目前已积水,最深约 8 m。矿山开采不规范造成工作区内地形支离破碎,存在潜在边坡失稳、岩石滑落、崩塌等安全隐患。

研究拟采用现场大量淤积的尾泥直接对大面积、大深度的矿坑进行回填,不仅可以就地消化堆积如山的废弃尾泥,节约巨额的土方外运和弃土场费用,省去大量回填土的采购,治理后还能够形成建设用地,消除污染风险和矿坑安全隐患、地质灾害隐患,创造生态价值、经济价值和社会效益。

1.2 试验材料

1.2.1 试验土体

模型箱内土体由上层 3 cm 厚的细砂垫层和下层 80 cm 厚的花岗岩尾泥组成。模型试验前,先对不同含水率的花岗岩尾泥进行直剪试验和固结试验,获得其抗剪强度指标及压缩模量和压缩系数,试验参数分别见表 1~3。最终,模型试验中的花岗岩尾泥采用的含水率约为 25%,相应的花岗岩尾泥材料参数为黏聚力为 26.7 kPa、内摩擦角为 19.1°,密度为 1.85 g/cm³。

表 1 不同含水率下的花岗岩尾泥抗剪强度参数

Tab. 1 Shear strength parameters of granite tail mud with different water contents

含水率/%	竖向应力/kPa	水平向剪应力/kPa	黏聚力/kPa	内摩擦角/(°)
15	50	46.17	15.2	32.8
	100	78.05		
	200	150.00		
	300	205.10		
20	50	47.92	16.7	32.8
	100	81.07		
	200	147.90		
	300	208.50		
25	50	42.92	26.7	19.1
	100	59.87		
	200	101.60		
	300	127.50		

表 2 不同含水率下的花岗岩尾泥压缩模量

Tab. 2 Compressive modulus of granite tail mud with different water contents

含水率/%	应力/kPa							MPa
	200	300	400	500	600	700	800	
20	10.55	12.66	15.19	18.99	23.74	27.13	29.22	
25	7.60	10.56	12.67	15.84	19.01	21.12	22.36	
30	6.35	9.07	10.88	13.60	17.31	18.13	19.63	
35	5.60	8.28	10.29	13.60	17.31	18.13	19.43	

表 3 不同含水率下的花岗岩尾泥压缩系数

Tab. 3 Compression coefficients of granite tail mud with different water contents

含水率/%	应力/kPa							MPa
	200	300	400	500	600	700	800	
20	0.18	0.150	0.125	0.100	0.080	0.070	0.065	
25	0.25	0.180	0.150	0.120	0.100	0.090	0.085	
30	0.30	0.210	0.175	0.140	0.110	0.105	0.097	
35	0.34	0.230	0.185	0.140	0.110	0.105	0.098	

1.2.2 模型桩体材料

试验模型桩体材料为小碎石和高聚物。碎石取自工程现场,考虑到桩径尺寸,将碎石的最大粒径 D_{max} 控制在 10 mm 以内,并用标准筛对试验碎石进行筛分,选取其中粒径为 5~10 mm 的小碎石作为桩体材料。

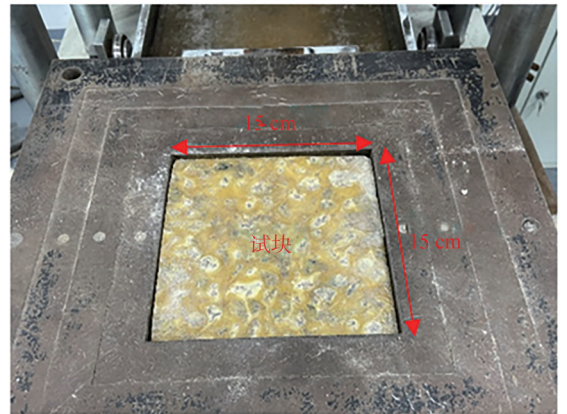
高聚物采用非水反应类双组分聚合物,由多元醇和异氰酸酯两种组分按照质量比 1:1 进行混合,经过一段时间的反应后生成泡沫状固化物。经过多次尝试和调整,将高聚物掺入碎石中进行拌合,形成高聚物碎石混合物料;考虑工程经济性且在拌合过程中发现当高聚物掺量小于 10% 时高聚物不能充分包裹碎石颗粒,高聚物掺量选取 10%、15% 和 20%。

在模型试验开展前,利用 DZJ-300 型大型直剪仪(图 1(a))对不同高聚物掺量的高聚物碎石试块(图 1(b))进行直剪试验,获得不同高聚物掺量的高聚物碎石试块的抗剪强度参数,结果见表 4,为研究高聚物掺量对碎石桩加固效果的敏感性提供试验参数依据。表 4 中,高聚物掺量为 0 表示为纯碎石试件。

由表 4 可知,高聚物加固后碎石试样的抗剪强度显著增大。这是由于高聚物材料本身具有较高的强度,当将高聚物材料注射进碎石时,高聚物固化膨胀填充碎石颗粒间的孔隙,并将碎石颗粒黏结在一起,增大了颗粒重排列的阻力,从而提高了碎石的抗剪强度,增大了碎石的变形模量和刚度,减小了碎石的变形。



(a) DZJ-300型大型直剪仪



(b) 高聚物碎石试块

图 1 高聚物碎石试块大型直剪试验

Fig. 1 Large-scale direct shear test of polymer gravel test block

表 4 不同高聚物掺量下的高聚物碎石试块抗剪强度参数

Tab. 4 Shear strength parameters of the polymer gravel test blocks with different polymer contents

高聚物掺量/%	竖向应力/kPa	水平向剪应力/MPa	抗剪强度参数	
			黏聚力/MPa	内摩擦角/(°)
0	100	0.276	0.147	48.7
	200	0.347		
	300	0.498		
	400	0.604		
10	100	0.695	0.5707	56.7
	200	0.905		
	300	1.054		
	400	1.153		
15	100	1.520	1.4367	47.2
	200	1.682		
	300	1.776		
	400	1.849		
20	100	1.680	1.6169	39.5
	200	1.812		
	300	1.860		
	400	1.939		

1.3 试验方案

通过地基静载模型试验,研究高聚物碎石桩加固花岗岩尾泥地基的刚度和变形特性,揭示其加固机理。参照实际工程中碎石桩的桩体尺寸,几何相似比取 10,设计未加固尾泥地基、碎石桩加固尾泥地基和高聚物碎石桩加固尾泥地基,并分析高聚物碎石桩桩长 L 、桩径 D 、高聚物掺量对花岗岩尾泥地基承载性能的影响。研究共设计 9 组试验,具体方案见表 5。

表 5 模型试验方案
Tab. 5 Model test program

编号	研究对象	参数	研究目标
1	未加固尾泥地基		
2	碎石桩加固尾泥地基	$L=600\text{ mm}, D=80\text{ mm}$	不同加固方式对尾泥地基承载性能的影响
3	高聚物碎石桩加固尾泥地基	$L=600\text{ mm}, D=80\text{ mm}$, 高聚物掺量 15%	
4	高聚物碎石桩加固尾泥地基	$L=500\text{ mm}, D=80\text{ mm}$, 高聚物掺量 15%	
5	高聚物碎石桩加固尾泥地基	$L=700\text{ mm}, D=80\text{ mm}$, 高聚物掺量 15%	碎石桩桩长对尾泥地基承载性能的影响
6	高聚物碎石桩加固尾泥地基	$L=600\text{ mm}, D=60\text{ mm}$, 高聚物掺量 15%	
7	高聚物碎石桩加固尾泥地基	$L=600\text{ mm}, D=100\text{ mm}$, 高聚物掺量 15%	碎石桩桩径对尾泥地基承载性能的影响
8	高聚物碎石桩加固尾泥地基	$L=600\text{ mm}, D=80\text{ mm}$, 高聚物掺量 10%	
9	高聚物碎石桩加固尾泥地基	$L=600\text{ mm}, D=80\text{ mm}$, 高聚物掺量 20%	高聚物掺量对尾泥地基承载性能的影响

1.4 试验布置、装置及仪器

模型试验布置如图 2 所示,试验装置如图 3 所示。试验装置主要包括模型箱和加载装置。模型箱内部尺寸为长 1 000 mm、宽 1 000 mm、高 1 000 mm,侧面的钢化玻璃厚 10 mm。加载装置主要由千斤顶、反力装置和液压传感器组成;反力装置采用压重平台反力装置,具体方法是在模型箱的两侧用两块配重块作为承重墙,将两根横向钢梁架在承重墙上,最后在横向钢梁上放置两块配重块作为压重,为静载试验提供反力。测量项目有:荷载板压力(采用 1 个液压传感器进行测定);荷载板位移(采用两个量程均为 50 mm 的位移传感器进行测量);桩端顶部和底部应力(各采用 1 个土压力盒进行测定);土体竖向应力(沿深度在桩两侧土体设置 10 个土压力盒进行测定)。数据值采用 YY-8 型号岩土力学试验数据采集仪进行采集并记录。

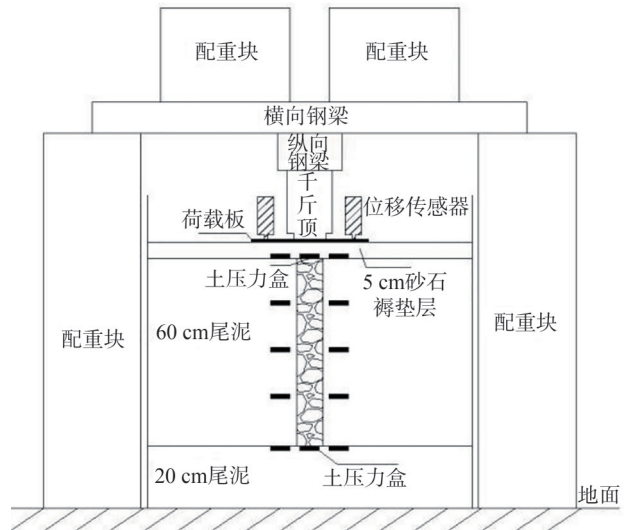


图 2 模型试验布置示意图

Fig. 2 Schematic diagram of test model layout



图 3 模型试验装置

Fig. 3 Model test equipment

1.5 试验步骤

1) 矿坑现场经过脱水处理的尾泥含水率约为 25%。因此,模型试验中将土体含水率控制为 25%。当含水率不足 25% 时,加水配制,含水率以总质量控制,搅拌均匀并用塑料膜完全包裹密置,使该值恒定,以备使用;反之,则在塑料膜上摊铺晾晒,直至含水率降低至 25%。

2) 在模型箱内按试验要求进行分层填土并压实,同时埋入 PVC 管及土压力盒;填土完成后静置 48 h,使尾泥回填土性质更加均匀;拔出 PVC 管,填入高聚物碎石混合料成桩。

3) 将模型表面整平,铺设 30 mm 厚的砂垫层,砂垫层的最大粒径 $D_{\max} < 5\text{ mm}$,在砂垫层上放置荷载板。为了保证所加荷载是垂直的,采用水平尺确保砂垫层和荷载板水平。完成后在荷载板上安放液压千斤顶,最后在荷载板顶面架设位移传感器。

4)仪器设备的连接和调试。模型试验是一个系统工程,在试验开始前要确保千斤顶的液压传感器、土压力盒、位移传感器、数据采集仪等正常工作。

5)加载前先用5 kPa进行预压,再在卸载后将数据采集仪上的力读数和位移读数清零,消除初始值的影响,然后按每级荷载20 kPa进行加载,第1级加载为分级荷载的两倍。

6)加载结束后,重复上述步骤进行下一组试验。

7)整理各项试验数据,分析试验现象与机理,得出结论。

2 试验结果

2.1 成桩效果

成桩效果如图4所示。静载试验结束后将桩体挖出(图4(a)),可以看出高聚物材料经反应后将碎石胶结形成连续性、均匀性较好的桩体,整体呈圆柱体,桩体外观无缺陷、无缩颈等现象,成桩结构完整。高聚物材料在反应过程中会跟桩周土体发生胶结,并且高聚物体积产生膨胀,进而对桩周土体产生挤密作用,增强桩土体共同作用;随着高聚物密度的提高,这种胶结和挤密作用也会随之增强,且随着高聚物密度的增加,高聚物对土体的劈裂作用也将加强,形成更多如图4(b)所示的棘突状结构,从而增大桩和土体间的接触面积,提高尾泥地基承载能力。对比图4(b)左右两根桩可知,高聚物掺量为20%的桩体明显比高聚物掺量为10%的桩体具有更多棘突状的结构。

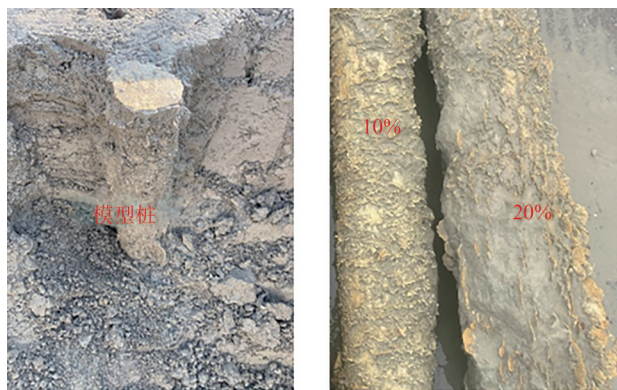


图4 成桩效果

Fig. 4 Effect of pile-forming

2.2 荷载-沉降结果

对静载试验获取的结果进行整理,绘制荷载-沉降曲线如图5、6所示,各工况下的尾泥地基荷载-沉降曲线均为缓变形曲线,无明显拐点。

由图5、6中可以看出:

1)经过高聚物碎石桩和碎石桩加固处理后的尾泥地基承载能力明显提高。尾泥地基的最终沉降量为

132.96 mm,碎石桩加固尾泥地基的最终沉降量为98.62 mm,高聚物碎石桩加固尾泥地基的最终沉降量为42.78 mm。高聚物碎石桩加固尾泥地基的沉降量相比加固前减少了67.8%,相比碎石桩减少了56.6%,可见高聚物碎石桩的加固效果比碎石桩好。

2)在相同桩径和高聚物掺量下,当桩长为500、600和700 mm时,高聚物碎石桩加固尾泥地基最终沉降量分别为51.84、42.78、35.58 mm。可见,随着桩长增加,尾泥地基承载能力逐渐增强;相比于加固前,3种桩长条件下沉降量分别减少61.0%、67.8%和73.2%。桩长为600和700 mm比桩长为500 mm的高聚物碎石桩加固尾泥地基的最终沉降量分别减少17.5%和31.4%。

3)在相同桩长和高聚物掺量下,当桩径为60、80和100 mm时,高聚物碎石桩加固尾泥地基最终沉降量分别为56.24、42.78、38.12 mm;相比于加固前,3种桩径条件下沉降量分别减少57.7%、67.8%和71.3%。可见,随着桩径增加,尾泥地基承载能力得到一定程度的提升。桩径为80和100 mm比桩径为60 mm的高聚物碎石桩加固尾泥地基的最终沉降量分别减少23.9%和32.2%。

4)在桩长和桩径相同的条件下,当高聚物掺量为10%、15%和20%时,高聚物碎石桩加固尾泥地基的最终沉降量分别为44.27、42.78、39.96 mm,相比于加固前,沉降量分别减少了66.7%、67.8%和69.9%。可见,随着高聚物掺量的增加,高聚物材料反应产生更大膨胀,桩体侧面产生更多棘突状结构,桩土间的相互作用增强,使得对应的尾泥地基承载能力也增强。高聚物掺量为15%和20%时比掺量为10%的高聚物碎石桩加固尾泥地基的最终沉降量分别减少3.4%和9.7%。

综上所述,高聚物碎石桩加固效果明显,且桩长、桩径和高聚物掺量的增加都会提升对尾泥地基的加固效果。

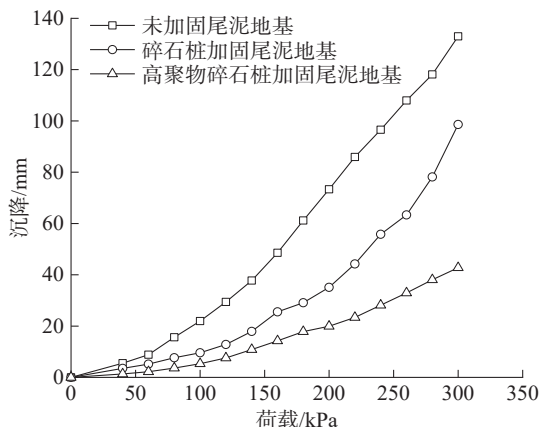


图5 不同加固方式下的尾泥地基荷载-沉降曲线

Fig. 5 Load-settlement curves of tail mud foundation with different reinforcement methods

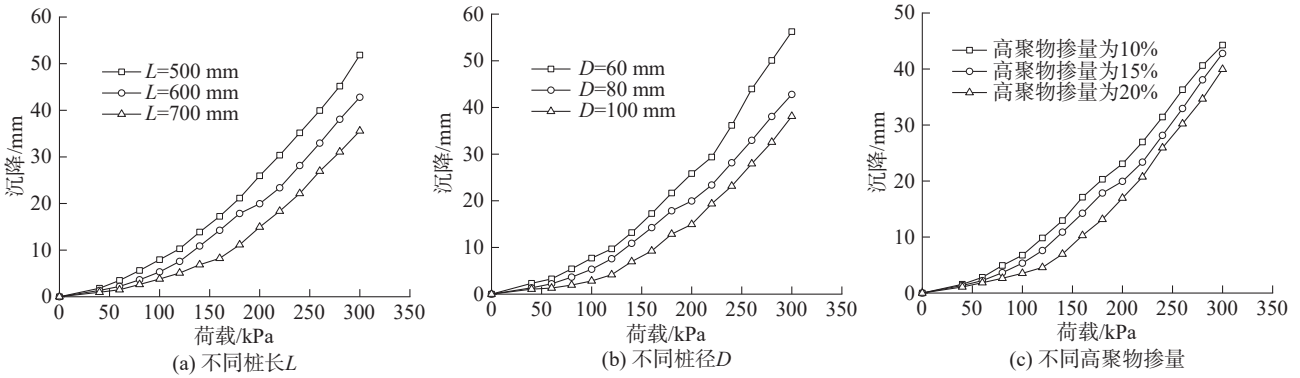


图6 不同桩参数下高聚物碎石桩加固尾泥地基荷载-沉降曲线

Fig. 6 Load-settlement curves of tail mud foundation reinforced by polymer gravel pile with different pile parameters

2.3 桩土应力比

采用土压力盒监测地基模型在各级荷载下的桩应力和土应力,从而得到实测的桩土应力比;绘制不同试验方案下,地基在各级荷载作用下的桩土应力比,结果如图7、8所示。因为桩体刚度远大于尾泥刚度,所以在荷载作用下,桩体将承担更大比例的荷载。

由图7、8可以看出,桩土应力比随施加荷载逐渐增大呈不断增大趋势,桩土应力比数值在1.14~6.69之间。在试验加载初期荷载较小时,高聚物碎石桩加固尾泥地基和碎石桩加固尾泥地基的桩土应力比较小且比值接近。随着荷载逐渐增大,碎石桩的桩土应力比增长趋势变缓,最终仅达到3.32;高聚物碎石桩的桩土应力比呈线性增加,最终达到5.30,比碎石桩大59.6%。

可见,相比于碎石桩,高聚物碎石桩所分担的荷载更多,加固效果更显著。随着桩长和高聚物掺量增

加,桩体的抗压刚度变大,使得桩土应力比增大;随着桩径增加,桩体刚度变大,分担的荷载也增大,但是与桩径呈2次方倍速度增长的桩截面积相比却较小,使得桩土应力比减小。

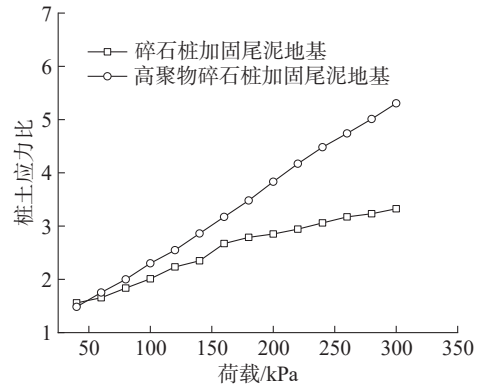


图7 不同加固方式下的尾泥地基荷载-桩土应力比曲线 Fig. 7 Load-pile-soil stress ratio curves of tail mud foundation with different reinforcement methods

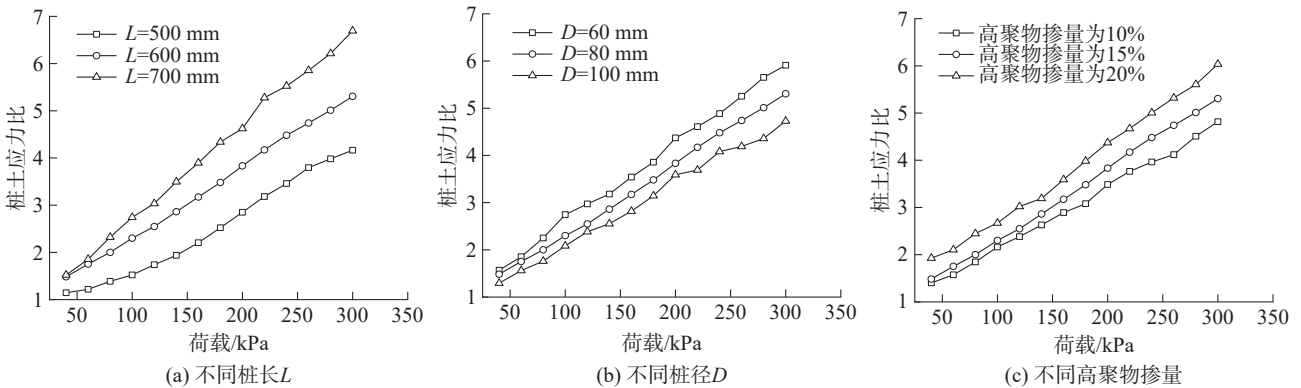


图8 不同桩参数下高聚物碎石桩加固尾泥地基荷载-桩土应力比曲线

Fig. 8 Load-pile-soil stress ratio curves of tail mud foundation reinforced by polymer gravel pile with different pile parameters

2.4 桩底应力

在桩底放置一个土压力盒,监测桩底应力的变化情况。根据得到的结果绘制桩底应力-荷载关系曲线,如图9、10所示。由图9、10可以看出,随着荷载的增加,桩底应力逐渐增大,当荷载达到300 kPa时,高聚物碎石桩和碎石桩的桩底应力分别为121和40 kPa,高聚物

碎石桩的桩底应力比碎石桩增大202.5%。这是由于碎石没有胶结强度,碎石桩在荷载作用下桩体会发生鼓胀变形,使桩顶荷载难以通过桩身有效传递至桩底。而高聚物很好地将碎石桩胶结成整体,提高了桩体质量,使高聚物碎石桩能有效地将荷载传递至桩底,提高了尾泥地基的承载能力。桩顶荷载主要由桩侧阻力和桩

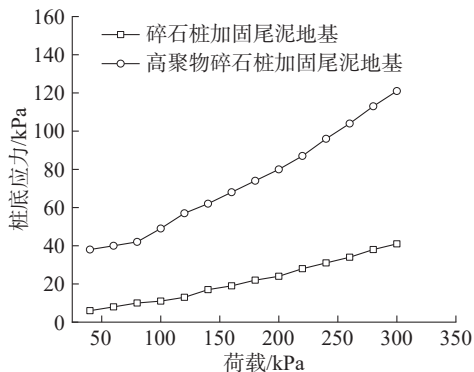
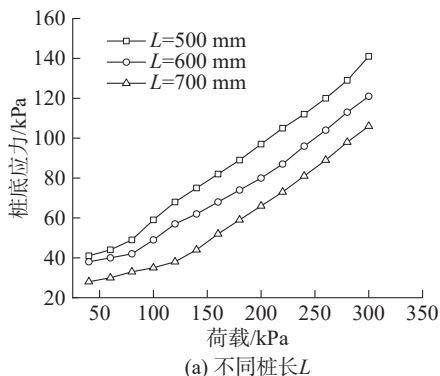
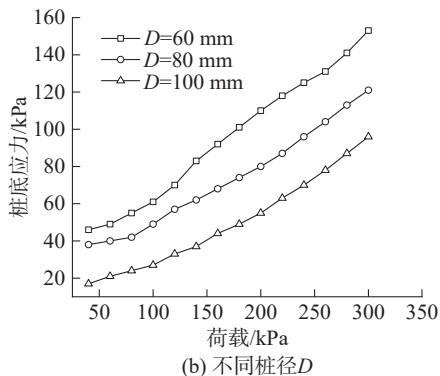


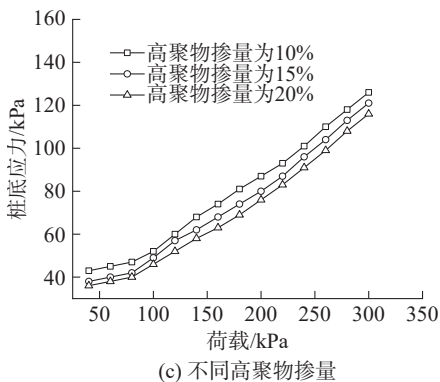
图 9 不同加固方式下的尾泥地基荷载-桩底应力曲线
 Fig. 9 Load-pile end stress curves of tail mud foundation with different reinforcement methods



(a) 不同桩长L



(b) 不同桩径D



(c) 不同高聚物掺量

图 10 不同桩参数下高聚物碎石桩加固尾泥地基荷载-桩底应力曲线
 Fig. 10 Load-pile end stress curves of tail mud foundation reinforced by polymer gravel pile under different pile parameters

底应力承担,在同一荷载作用下,随着桩长、桩径和高聚物掺量的增加,桩底应力逐渐减小。这是因为随着桩长和桩径的增大,桩侧和土体间接接触面积增大,桩侧阻力增大,桩底应力减小。在相同桩长和桩径下,随着高聚物掺量的增加,高聚物材料反应产生更大膨胀,使桩体侧面产生更多棘突状结构,增强了桩土间的相互作用,从而导致桩侧阻力增大,桩底应力减小。

2.5 桩周土体竖向应力

图 11 为当荷载为 300 kPa 时,不同加固方式下的尾泥地基土体竖向应力随模型地基深度的变化曲线。由图 11 可以看出:在地基表层,未加固尾泥地基土体竖向应力最大,碎石桩加固尾泥地基土体竖向应力次之,高聚物碎石桩加固尾泥地基土体竖向应力最小;土体竖向应力主要集中在地基上部,并且均随地基深度的增加快速衰减。在 1/4 深度附近,碎石桩加固尾泥地基的土体竖向应力大于未加固尾泥地基和高聚物碎石桩加固尾泥地基,这是因为碎石桩在荷载作用下桩身发生鼓胀变形,使桩身荷载传递深度有限。高聚物碎石桩加固尾泥地基土体竖向应力在桩底处略有增大,这是因一部分荷载通过桩身传递到桩底引起的,说明高聚物碎石桩相比于碎石桩,能够更为有效地通过桩身将荷载传递至桩底。

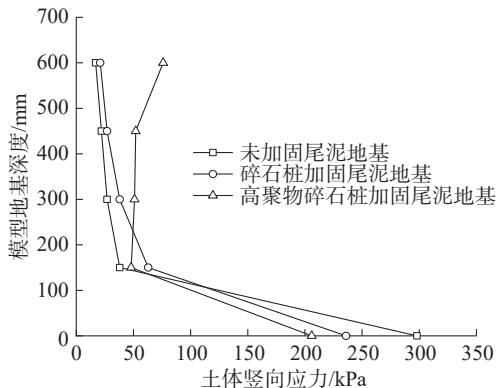


图 11 不同加固方式下土体竖向应力随深度变化曲线
 Fig. 11 Vertical stress of soil versus depth curves under different reinforcement methods

3 结论

高聚物碎石桩是一种利用高聚物将碎石胶结成整体的新型桩。针对废弃尾泥回填矿坑的工程背景,通过模型试验研究了高聚物碎石桩对花岗岩尾泥地基的加固效果,并分析了桩长、桩径和高聚物掺量对加固效果的影响,主要得到以下结论:

1)高聚物碎石桩成桩效果好,整体呈连续、均匀的圆柱体,桩体外观无缺陷、无缩颈等现象,且高聚物材料反应过程中体积膨胀,对桩周土体产生挤密和劈裂作用,并在桩周形成棘突状的结构;同时,随着高聚

物密度的增加,高聚物碎石桩对土体的劈裂作用也会加强,形成更多棘突状的结构,从而增大了桩和土体间的接触面积,提高了尾泥地基承载能力,并减小了地基沉降量和桩底应力。

2)相比于未加固尾泥地基和碎石桩加固尾泥地基,高聚物碎石桩加固尾泥地基的最终沉降量分别减少了67.8%和56.6%。说明高聚物碎石桩能显著增强花岗岩尾泥地基的承载能力,且增强效果比碎石桩好。

3)增加桩长、桩径和高聚物掺量能有效降低尾泥地基的最终沉降量,可以明显提高加固效果。具体而言:桩长从500 mm增加至700 mm,沉降可减小61.0%~73.2%;桩径从60 mm增加至100 mm,沉降可减小57.7%~71.3%;高聚物掺量从10%增加至20%,沉降可减小66.7%~69.9%。

4)相比于碎石桩,高聚物碎石桩的桩土应力比增大了59.6%,桩底应力增大了202.5%。可见,高聚物提高了桩体的刚度,从而有效地将荷载传递至桩底土层;同时说明,在实际工程中将桩底设置在承载能力较好的土层可以进一步提高尾泥地基承载能力。

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Model Test Study on Granite Tail Mud Foundation Reinforced by Polymer Gravel Piles

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Abstract:

Objective The backfill treatment of surrounding mining pits represents an emerging land-use strategy. Utilizing tailings stored in mining areas as backfill material reduces construction costs and addresses the challenge of tail mud disposal. Polymer gravel piles provide high strength, rapid formation, and maintenance-free performance after completion, meeting the requirements of complex foundation treatment. Model tests are conducted to investigate the reinforcement mechanism and bearing characteristics of polymer gravel piles in such foundations to verify the effective-

ness and feasibility of polymer gravel piles in treating granite tail mud foundations. The influence of pile length, pile diameter, and polymer content on reinforcement performance is analyzed. The findings provide a potential technical solution for ecological restoration and land development of mine surfaces.

Methods The materials of the model pile were gravel and polymer, and the maximum particle size of the gravel was controlled within 10 mm. The test gravel was screened with a standard sieve, and gravels with particle sizes of 5–10 mm were selected as the pile material. The polymer was a non-aqueous reaction-type two-component polymer, which was mixed with polyol and isocyanate at a mass ratio of 1:1. After a specific period, a foam-solidified substance was formed. The polymer crushed stone pile was formed by mixing the polymer with gravel. First, direct shear tests and consolidation tests were conducted on granite tail mud with different moisture contents to obtain the shear strength index, compression modulus, and compression coefficient of the granite tailings. Then, shear strength tests were performed on polymer gravel specimens with different polymer contents to determine their shear strength using a DZJ-300 type large direct shear instrument. Then, a total of nine sets of model test schemes for the unreinforced tail mud foundation, the tail mud foundation reinforced by gravel pile, and the tail mud foundation reinforced by polymer gravel pile were designed, and model tests were conducted to study the reinforcement of the granite tail mud foundation using different reinforcement methods. A jack and reaction device were employed to load the foundation model, displacement meters were utilized to record the foundation settlement, and earth pressure boxes were utilized to record the stresses at the pile top, pile bottom, and pile side. The effects of pile length, pile diameter, and polymer content of the polymer gravel piles were analyzed. Finally, the bearing characteristics of the reinforced foundation were tested, including the measurement and analysis of parameters such as settlement and pile bottom stress.

Results and Discussions After the static load test, it was observed from the excavated piles that the polymer material reacted to form a well-connected and uniform pile with good integrity. The overall shape of the pile was cylindrical, and no defects or necking phenomena appeared on the pile surface, indicating a complete pile structure. During the reaction process, the polymer materials bonded with the surrounding soil, and their volume expanded, producing a compaction effect on the soil around the pile. This process formed a jagged structure on the pile surface, which increased the contact area between the pile and soil, improving the bearing capacity of the tail mud foundation. As the density of the polymer increases, the cementation and compaction effects also become stronger. Based on the test results, the final settlement of the tail mud foundation reinforced with the gravel pile was 98.62 mm, whereas the final settlement of the tail mud foundation reinforced with the polymer gravel pile was 42.78 mm. The reinforcement effect of the polymer gravel pile was significantly better than that of the traditional gravel pile, resulting in a 56.6% reduction in final settlement. It was evident that the bearing capacity of the granite tail mud foundation increased substantially after reinforcement with polymer gravel piles. As the load gradually increased, the stress ratio of the gravel pile to soil for the gravel pile exhibited a slower growth trend, eventually reaching 3.32, whereas the stress ratio for the polymer gravel pile showed a linear growth trend, eventually reaching 5.30, which was 59.6% higher than that of the gravel pile. With the increase in load, the pile bottom stress also increased gradually. When the load reached 300 kPa, the pile bottom stress for the polymer gravel pile and the gravel pile were 121 kPa and 40 kPa, respectively. It was evident that the pile bottom stress of the polymer gravel pile increased by 202.5% compared to the gravel pile. From the curve of soil vertical stress variation along the depth of the model foundation, it was observed that the vertical stress in the soil was mainly concentrated in the upper part of the foundation and rapidly attenuated with increasing foundation depth. The length, diameter, and polymer content of the piles have a significant impact on the reinforcement effect. As these parameters increase, the reinforcement effect also improves accordingly. The research findings provided a new technical approach for surface ecological restoration and land development of abandoned mine pits, demonstrating broad application potential.

Conclusions This study confirms the effectiveness of polymer gravel piles in improving granite tail mud foundations through model testing. The results reveal that polymer gravel piles significantly enhance the bearing capacity of the foundation and effectively reduce its settlement. In addition, pile length, pile diameter, and polymer content are critical factors influencing the reinforcement performance. Therefore, in practical engineering applications, appropriate parameter combinations should be selected based on site-specific conditions to achieve optimal reinforcement outcomes.

Key words: polymer gravel piles; granite tail mud; foundation treatment; static load model test

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