GROUND WATER PRESSURE REDUCTION
FOR ROADBED STABILIZATION

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ABSTRACT

This paper presents a case history of a roadbed stabilization project along a mainline track and controlled siding near Catskill, New York. Chronic instability over a period of several years required frequent slow orders and intense track maintenance on this heavy-tonnage route. Several engineering investigations and exploratory drilling programs were performed over a three-year period beginning in early 1998. Eventually, subsurface conditions were adequately characterized, the cause of the problem was identified and a repair was successfully implemented. The project clearly demonstrates that good definition and characterization of subsurface conditions is always valuable and often mandatory in order to develop cost-effective methods for roadbed stabilization.

INTRODUCTION

Maintenance of roadbed and right-of-way is vital to the railroad industry. Unstable roadbed often requires that the affected track be taken out of service or that a slow order be imposed. The impacts include disruption of service, high maintenance and increased risk to rail traffic, all of which increase operating costs. Cost-effective repairs or treatment techniques are needed to address such problem sites and "stretch" scarce capital and maintenance resources.

Sometimes, the cause(s) of roadbed instability are obvious and can be addressed without the benefit of detailed engineering investigations or designs. In such instances, the causes are typically routine problems such as poor surface drainage, over-steep slopes, or scour along an embankment toe due to flow in an adjacent river or stream. The corresponding repairs often include ditch cleaning, slope flattening and placement of riprap protection. However, when the cause(s) of roadbed instability are not obvious or routine, or when a site is large or complex, engineering investigations and designs are often a pre-requisite to successful treatment or repair.

Between 1998 and 2001, chronic roadbed instability affected CSX Transportation’s mainline and siding on its Riverline Subdivision. Eventually, the site was repaired for a fraction of the cost of conventional structural support systems. This was possible only after adequate site exploration and characterization of subsurface conditions.

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SITE DESCRIPTION

The site consists of a 20-foot high embankment fill at Mile Post QR-106.2 on CSX Transportation’s Riverline Subdivision along the west side of the Hudson River. The fill supports a north-south mainline and controlled siding. Rail traffic is heavy and includes in excess of 25 trains per day.

The embankment fill is situated on gently sloping ground that generally drains eastward toward the Hudson River. The tracks parallel New York State Route 9W located about 100 feet west of the mainline. Immediately west of SR 9W, the ground surface rises sharply forming a bluff about 60 feet above track and roadway level. An active rock quarry is present on the bluff west of the subject site. A cement processing plant is present southeast of the site between the tracks and the Hudson River. Figure 1 shows an aerial view of the overall site.

![Figure 1. Aerial view of site at MP QR-106.2.](image)

Reportedly, the site exhibited some degree of instability for many years, particularly following periods of wet weather. In early 1998, roadbed conditions became chronic forcing the company to repeatedly impose slow orders and perform frequent track surfacing. Problems included severe profile loss, cross-level and eastward alignment as shown in Figures 2 and 3. The mainline was being impacted over a length of about 400 feet; the siding was impacted over a length of about 800 feet.
EXPLORATORY PROGRAMS

Between 1998 and 2001, several engineering investigations of the site were performed. These investigations generally began with a walk-over reconnaissance of the site followed by subsurface drilling, sampling and instrumentation. The visual site reviews detected a few signs of typical slope stability problems such as tension cracks and isolated ground water seeps. However, no obvious signs of major ground movements such as large bulges along the slopes or distorted trees were observed. Consequently, the cause(s) of instability and the actual failure mechanism were not obvious.

The initial investigations in 1998 and early 1999, included a number of geotechnical borings drilled into the built-up portion of the embankment to assess the character and consistency of the fill materials. The borings revealed loose-to-dense, granular fill consisting of cinders, slag and sand. Visual inspection of the site along with a field survey indicated that surface drainage west of the fill was generally poor. Accordingly, a new cross-drain pipe was jacked through the embankment to promote better west-to-east surface drainage. In addition, a series of shallow trench drains were excavated into the shoulders in an attempt to intercept and drain potential ballast pockets within the granular fill embankment. These measures seemed to provide some degree of short-term relief however, significant instability resumed in early 2000 following an extended period of wet weather.

In April 2000, another engineering reconnaissance of the site was performed. Project personnel noted that the previously installed cross-drain had settled and separated near its mid-point and the inlet and outlet ends were raised slightly. In addition, a previous tension crack along the east shoulder had re-developed and enlarged with the trackside portion of the fill settling relative to the east side of the fill (Figure 4). These observations suggested that a deep-seated problem existed and that the built-up fill appeared to be “punching” into the foundation soils. An additional phase of subsurface drilling was recommended for the site.
The recommended additional exploration was performed in June 2000 to assess foundation conditions below the built-up embankment fill. A total of six (6) deep borings were planned, including one on each shoulder, two along a bench-type feature on the east slope, and two about 100 feet east of the tracks beyond the toe of the fill. These borings were advanced to refusal on bedrock at a depth of approximately 50 feet. Below the granular fill interval, each boring generally penetrated firm clay 20 to 30 feet thick, very soft clay 5 to 17 feet thick, and then dense sand and gravel 0.5 to 3 feet thick.

One of the borings along the bench and one boring near the toe of the fill produced artesian conditions after penetrating the deep sand and gravel layer. Two-inch PVC pipe and well screens were placed in these borings. Discharge on the order of 20-30 gallons per minute (gpm) continued for several days. Only one of the wells yielded artesian flow at a time. When the topographically lower boring was installed, it began artesian flow and the previously installed well stopped producing. The change in flow was rapid and indicated that a good hydraulic connection existed within the sand and gravel layer between the boring locations.

**Figure 4.** Tension cracks along east shoulder.

**Figure 5 (right).** Split-spoon sample of soft wet clay from interval above sand and gravel layer.
Following these observations, several additional borings were authorized in an attempt to “chase” the deep, water producing sand and gravel layer and assess possible drainage options. These borings were located on the east side of the fill. Some of the additional borings were drilled in topographically lower ground east of a paved access road below the site. These borings were located 300 to 900 feet east of the tracks. In general, the subsurface profile was consistent across the site however, only a few of these additional borings produced artesian flow. Moreover, the hydraulic response between the borings on either side of the paved access road east of the fill was very slow.

During this phase of site exploration, a single inclinometer casing was installed in one of the non-artesian borings on the east slope of the fill. Subsequent monitoring was generally inconclusive; the data showed only very slight casing deflection within the soft clay interval above the sand and gravel layer. This observation suggested possible lateral deformation of the deep foundation material and the possibility of very slight heave or bulging along the east slope.

An interesting observation by field personnel was made during the drilling program. After the first artesian boring was completed and was consistently yielding 20-30 gpm, the flow rate abruptly dropped by about 50% as train traffic passed across the adjacent fill. Once the train passed, the original flow rate resumed within a few minutes. This observation supported the possibility of a foundation “punching” failure whereby the embankment was being forced into the subgrade and constricting flow within the underlying sand and gravel layer.

At this point in the overall site investigation, several reviews had been performed and four episodes of subsurface exploration had been performed providing a total of 27 geotechnical borings. Slope inclinometer casing had been installed in four of the borings and nine simple standpipe piezometers had been installed. Figure 7 below shows a plan view of the site with the approximate locations of the borings.
In August 2000, an engineering report was issued describing the exploration program and field observations. The discovery of artesian conditions within the deep sand and gravel layer beneath the site resulted in a recommendation for passive subsurface drainage using horizontal drains to lower the potentiometric surface and reduce uplift pressures on the fill.

Subsequent discussions with CSXT personnel and a peer review of the recommendation for horizontal drain installation generally met with favor. However, additional site instrumentation and monitoring were proposed in an attempt to better define the actual failure mechanism and to provide pre-construction data in the event the horizontal drain system did not perform as desired. The additional instrumentation included a series of ground settlement (or heave) monitoring points and installation of four (4) Sondex casings. The Sondex system consisted of an inclinometer casing covered with corrugated plastic pipe sheathing. At selected points along the length of the plastic pipe sheathing, metal sensing rings were installed within the corrugations. The sheathed casing was then installed in a vertical boring and grouted in place. Each Sondex boring was subsequently monitored using an inclinometer probe to check for lateral movement and a Sondex probe to determine the locations of the sensing rings. Movement of the sensing rings would yield data concerning possible ground settlement or heave. Figure 8 shows a site plan with the settlement monitoring points and Sondex instrument locations.
The settlement monument system was monitored from December 2000 to April 2001. Less than 0.5 inch of total movement was detected for any point and no definitive patterns or movement trends were noted. The Sondex borings were monitored from April 2001 to September 2001. Unfortunately, the Sondex data showed little, if any, definitive movement during this period with respect to both lateral and vertical ground deformation. It is possible that the monitoring period was simply too short to detect consistent movements or that the magnitude of movement and equipment sensitivity were incompatible.

FIELD OBSERVATIONS AND ENGINEERING CONSIDERATIONS

As noted previously, project personnel discussed potential treatment or repair alternatives for the site throughout the various phases of site exploration and study. The potential repair schemes were driven by and refined as data on subsurface conditions was generated and additional field observations were made. In general, more common methods for treating the site were applied initially as low-cost attempts to improve conditions. Specifically, the initial site investigation indicated that poor surface drainage and the granular materials comprising the embankment might be contributing to the problem. Accordingly, initial treatments were tailored to improve surface drainage and to prevent or eliminate ballast pockets. During this period, discussions continued concerning the possible need for slurry grout injection to fill voids and improve the strength of the embankment materials.
Following the additional track instability in April 2000, and the observations of cross-drain settlement and tension cracking along the east shoulder, discussions began to consider potential ground improvement schemes such as compaction grouting, jet grouting, stone columns, etc. Other underpinning systems and the possibility of constructing a pile supported land bridge system were also considered given the potential deep problem in this area. In the absence of specific data about deep ground conditions, conceptual schemes and cost estimates for extensive track underpinning or ground modification systems put repair costs well over $1.0 million due to the length of the site and the potential depth to bedrock.

When artesian conditions were revealed during the drilling program performed in mid-2000, project personnel believed that the primary cause of the problem had been identified. Cautiously, project personnel began to focus on ways to address this issue while continuing to explore and monitor the site.

Mapping of the general site area showed a lake (Van Luven Lake) above the site north of the rock quarry and an active cement processing plant south of the site. Project personnel speculated that seepage from the lake might be recharging the sand and gravel layer at the site thus producing the high ground water pressure in the area. Conversely, general grading activities associated with the cement plant may have caused consolidation of the soil overburden below the site thus constricting ground water flow within the sand and gravel layer. Regardless of the actual cause, it was obvious that something had changed with respect to ground water conditions in the area to produce a pressure increase and the corresponding embankment instability. Project personnel began considering options for subsurface drainage to reduce or eliminate the excess pressures. Deep trench-drains, sump pumps, and horizontal drains were among the options considered.

Unfortunately, the slight topographic relief of the area, along with property constraints, would not permit complete gravity drainage via open excavation of trench drains. Well-points and a sump pump system were considered maintenance intensive. Eventually, a system of drilled horizontal drains was designed to lower the potentiometric surface in the area by about 30 feet with the expectation that this would be sufficient to stabilize the site. The drains would be 200-300 feet long with the outlets located east of the embankment. The design was based on an observational approach to treating the site whereby a relatively low cost initial attempt would be made to reduce uplift pressures. Afterwards, observations of track performance and continued instrumentation monitoring could be performed to develop the next level of treatment or repair if the initial phase of work did not perform as desired. Project planning and engineering activities included an Archaeological Investigation of the site, ground water sampling and analytical testing to establish baseline water quality data, and property (construction easement) acquisition. The construction easement was needed to accommodate a drilling pad from which to perform horizontal drilling work.

Figure 9 shows a plan of the horizontal drain system. Figure 10 shows a cross-section of the site and the orientation of the proposed horizontal drains.
Figure 9. Plan of horizontal drain layout.

Figure 10. Site cross-section and the proposed horizontal drainage system.
CONSTRUCTION

Construction of the horizontal drain system was performed from August 30, 2001 to September 21, 2001. A local grading contractor performed site access, drill pad construction and site restoration. A specialty-drilling contractor installed the horizontal drains. The drains consisted of 1.5-inch diameter, schedule 80, slotted, PVC, flush-jointed pipe. The plan called for 14 drains. A total of 10 drains were actually installed in a “fan-like” array for a combined length of about 2,400 linear feet. The reduced number of drains was possible because of the success in encountering the sand and gravel layer and because the drains were producing considerable flow. The drains ranged from 200-300 feet long and were installed at –2 to –3 degrees from horizontal. Figures 11 through 13 show drain construction activities.
During construction, discharge rates from the drains and the water levels in the operable standpipe piezometers were monitored. Figure 14 shows a graph of discharge versus time for several of the drains. As indicated, discharge rates were initially at 30-40 gpm and then subsided as additional drains were installed and as pressure reduction was achieved. Figure 15 shows the drop in the potentiometric level measured in the standpipes as drain installation progressed.

![Figure 14](image1.png)

**Figure 14.** Discharge from selected horizontal drains during construction.

![Figure 15](image2.png)

**Figure 15.** Comparison of ground water levels in selected borings during drain installation.
Following construction, CSXT forces surfaced and aligned the tracks and then gradually returned the area to normal track speed. This process occurred over a 3-4 month period. Since that time, the horizontal drains have been periodically checked for continued discharge and the standpipe piezometers have been monitored. Project personnel report that the drains are functioning properly and that ground water levels remain low. Likewise, no track stability problems have been reported since construction was completed.

Construction of the horizontal drain system cost about $60,000. By comparison, the cost was less than 10% of the estimated cost for structural support or underpinning systems at this site.

CONCLUSIONS

Improving the stability of embankments and slopes using various methods of subsurface drainage is common practice. Consequently, installation of horizontal drains at this site is not considered a new or innovative technology. The lesson to be learned from this case history is that good definition and characterization of subsurface conditions can often result in a very cost-effective solution to the problem. In the absence of such information, considerable resources can be spent treating symptoms rather than the cause.