SUBSIDENCE POTENTIAL OVER TWO-SEAM DEVELOPMENTS

BOARDINGHOUSE CREEK AREA BELINA MINES

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SUBSIDENCE POTENTIAL OF BOARDINGHOUSE CREEK AREA  
OVER TWO-SEAM DEVELOPMENTS - BELINA MINES

Dear Mr. Wright:

This Report presents the results of our preliminary rock mechanics study regarding the subsidence potential of the Boardinghouse creek area due to the development of entries and room-and-pillar panels in the Belina Mines.

Empirical theories of subsidence do not provide the basis for estimating surface subsidence due to the development of a two seam mine system beneath a relatively shallow overburden of 200 to 300 feet, such as in the case of the Boardinghouse Creek area for the proposed design layout. Past experience with room-and-pillar mining in the Western U.S. indicates that a "plug" type subsidence may occur under certain geologic conditions, particularly for overburden depths of less than 150 feet. Such a condition is triggered by local roof instability, and its upward extent appears to be governed by the height of the opening into which debris can fall. With bulking of the loosened debris, it seems conservative to assume that the height of the plug should not exceed 10 times the height of the opening. Since full seam recovery in the Belina Mines would result in openings on the order of 20 feet in height, maximum chimney height would not exceed 200 feet. It is recommended, therefore, that until a rock mechanics investigation that more directly addresses the issue of plug-type subsidence can be undertaken, full seam recovery should not be employed in the creek areas where the depth of overburden is less than 200 feet.
A more global form of subsidence occurs when failure zones in the overburden are developed connecting the mine opening to the ground surface. These are usually triggered at rib or pillar abutments adjacent to extensive mined-out panels, and extend to the surface along planes sloping at approximately 20 to 35 degrees from the vertical. The investigation of this type of subsidence requires a relatively detailed structural analysis of the rockmass between the mine and the ground surface. Two dimensional finite element calculations were performed to analyze this type of subsidence potential for the given geologic and geometric parameters. Lower limits of the overburden rock quality were assigned to the finite element models to investigate the "worst case scenario" with respect to the subsidence potential of the creek crossing area.

Based on the results of these analyses, we conclude that there is no subsidence potential due to global overburden failure resulting from the full-seam recovery of the two-seam mine system as proposed. We recommend, however, that unless further study can show adequate protection against plug-type subsidence, bottom coal recovery should not be planned in either seam within the Boardinghouse Creek crossing area where the overburden is less than 200 feet.

Respectfully submitted,

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INTRODUCTION AND SUMMARY

The coal reserve located to the southwest of the Second Right and Third Right panels in the First East Mains is planned to be developed by Valley Camp of Utah. This reserve area, in a triangular form, is bounded by a dike to the north, a regional fault known as the O’Connor Fault to the east and a set of local faults to the west (shaded area in Figure 1). The area will be accessed through tunnels to be driven through the dike from the Third Right entries. Figure 2 is an enlarged map of the area to be developed. Within this triangular block, the overburden depth varies from less than 200 feet to about 600 feet above the top of the upper coal seam. Both the mineable seams, Upper O’Connor and Lower O’Connor, will be mined. According to the drillhole log of DH-73-31-2, the two seams are separated by about 38 feet of interburden. The interburden consists of sandstone and sandy siltstone units.

The mid-section of the development will be beneath a perennial stream, Boardinghouse Creek. Figure 3 shows a layout of the underground workings beneath this creek. The purpose of the present study is to resolve the issue of subsidence potential in this area due to the recovery of coal by a two-seam mining system.

Conventional theories of subsidence based on empirical formulas are not applicable to an underground configuration where the depth of overburden is greater than 1.4 times the room width, for a non-yielding pillar system. For a long-term, progressive deterioration of roof rocks, using a bulking factor of 1.2, about five times the room height may be assumed to eventually cave in. Additionally, an equal height of the roof over the caved area may eventually be disturbed during this long-term process. Thus it would appear conservative to estimate that long term deterioration
will not progress all the way to the ground surface if there is at least 100 feet of cover in the case of top coal mining only, or at least 200 feet in the case of full seam mining. These criteria are based on empirical formulas, and could be relaxed if a rock mechanics investigation specifically addressing the issue of long term subsidence were to prove favorable.

The immediate subsidence potential of the Boardinghouse Creek area, due to over stressing of the overburden, was analyzed using two-dimensional finite element models of the vertical section transverse to the creek (Section A-A in Figure 3). The basic finite element model for these analyses is shown in Figure 4. The section was taken at a location in which there is a depth of 200 feet of overburden from the creek bed to the top of the upper seam. The model extends 600 feet to either side of the creek, and 300 feet below the bottom of the lower seam. Subsidence is taken to be indicated by a zone of yielded rock extending all the way from the mine to the ground surface.

A two-dimensional cross section of the main entry configuration provides a good approximation to the loading in the roof, floor, and barrier pillars, since these structures have very little variation in the out-of-plane direction. The pillars in the room and pillar panels are three-dimensional structures, but the load/deflection properties of these pillars for both top coal and full seam mining has been determined through auxiliary three-dimensional analyses in an earlier report (KCKA, 1989). These load/deflection properties have been incorporated into equivalent homogeneous materials suitable for use in a two-dimensional model. These equivalent properties are represented in three layers of elements, corresponding to Materials 1, 2, and 3, occupying the same position. Material 1 represents the pillars left in place after full seam mining. Material 2 represents the supplementary
stiffness between Material 1 and a pillar resulting from top coal mining alone. Materials 1 and 2 add up to the stiffness of a pillar in a single level room and pillar system. Material 3 is the supplement to intact coal, so that Materials 1, 2, and 3 together represent the layer before any mining takes place. The deactivation of selected elements enables the observation of stress changes resulting from the progressive development of the entire mining system.

The results of these analyses of the proposed development plan indicate no possibility of immediate surface subsidence due to overstressing of the overburden, even when the full height of both seams are recovered. There were no zones of yielded rock indicated in any of the models, even when the rockmass quality of the overburden was assumed to be lower than is actually believed. Based on these results, we conclude that there is no theoretical possibility of subsidence resulting from the two seam mining within the reserve block as planned.

ANALYSES OF SUBSIDENCE POTENTIAL

Four two-dimensional cases have been run to investigate the possibility of subsidence during the development of both seams. The formulation of these two-dimensional models is described in Appendix A. In that maximum disturbance to the overburden occurs at the active face of excavation, due to the maximum discontinuity of support, each case observes conditions when the face of each level of each seam is located directly below Boardinghouse Creek.

Case 1 models top coal mining only in both the upper and lower seams, and consists of 5 load steps. Step 1 consists of intact coal prior to any excavation. Step 2 consists of top coal mining in the upper seam to a point below Boardinghouse Creek, with intact coal occupying the
remainder of the seam. Step 3 corresponds to the completion of top coal mining in the upper seam. Step 4 models top coal mining of the lower seam to a position below Boardinghouse Creek with top coal mining of the upper seam completely developed, and Step 5 models full top coal development in both seams.

Case 2 models full seam development of both the upper and lower seams, and takes place in 9 load steps. One step models intact coal, the next two model top coal development of the upper seam, Steps 4 and 5 model full seam development of the upper seam, and Steps 6 through 9 model the same configurations for the lower seam that Steps 2 through 5 model for the upper seam. Cases 3 and 4 are a repeat of Cases 1 and 2 with the overburden material given a rockmass quality index of "FAIR" (Hoek and Brown, 1980). This constitutes a worst case assessment of the actual conditions to be expected.

The loading of the panel cross section is provided by the weight of the overlying rock, which increases in thickness from the creek bed to either side. The idealized overburden profile employed in the 2-D models is shown in Figure 4. The horizontal component of in-situ stress is thought to be approximately equal to the Poisson’s component due to gravity alone. This allows the side boundaries be rollered to provide confinement under all circumstances.

As mentioned in the previous section, no significant yielding was observed in any of the cases for any of the load steps, even the very conservative cases. These findings are illustrated in the remaining figures, all of which are taken from Case 4, corresponding to double lift mining of both seams with overburden of FAIR quality. This is the most severe of all the cases considered. A more complete presentation of results from all of the cases is provided in Appendices B through E.
Figure 5 shows the distribution in effective stress in the upper seam as a result of top coal mining in the left half of the upper seam. The term "effective stress" is used to denote the pillar load divided by the effective area supported by that pillar (room centerline to room centerline). In the case of the unmined coal in the right half of the seam, effective stress corresponds to the actual stress, since the rooms have not yet been mined. There is a discontinuity in effective stress at the location of the active face. This reflects the fact that the development on the left side of the creek has lowered the stiffness of the support on this side. There is some deflection of the overlying strata in response to this reduction in stiffness of the coal seam, but the stiffness of the overburden prevents it from bearing its full weight on those pillars closest to the active face. The coal on the unmined side of the active face is burdened with the weight of the overburden not carried by the adjacent pillars, resulting in a stress abutment at the face. Away from the face, the stress in both the developed portion and the unmined portion of the seam quickly approach in-situ conditions. It is this discontinuity in support that provides the most likely environment for yielding in the overburden; however, none was observed.

Figure 6 shows a similar plot of effective stress in the upper seam at a time when both top and bottom coal have been mined in the left half of the seam while only top coal has been mined in the right half. Again a discontinuity is induced at the active face, but this discontinuity is smaller than in the case of Figure 5. Again no yielding was observed in the overburden.

Figure 7 shows the profile of effective stress after complete full seam development of the upper seam. With no discontinuity in support stiffness, this plot shows that
there is no discontinuity in effective stress. There is again no yielding in the overburden.

Figure 8 shows a profile of yield factor in the overburden immediately above the upper seam during the load step corresponding to the stress profile of Figure 6. Yield factor is defined as the ratio of maximum shear stress in the material to maximum allowable shear stress at the existing level of mean stress. When the yield factor attains a value of 1, the rock is yielding at that location, and a path of yielded rock connecting the mine with the ground surface is taken to be the criterion for subsidence.

The shape of the profile of yield factor shown in this figure is a result of the variation in depth of overburden and method of loading of the rockmass. Loaded by self weight with rollered boundaries, the rockmass develops a greater vertical stress than horizontal stress, and this difference increases as the depth of cover increases. The actual maximum shear stress is proportional to this difference in vertical and horizontal stress, while the shear capacity for this very low quality rockmass does not increase very much with increasing pressure. This explains why the yield factor in Figure 8 is greater toward the edges of the model, where the depth of overburden is greater. It should also be observed in this figure that the discontinuity in support shown in Figure 6 contributes a nearly imperceptible discontinuity in yield factor.

It is also noteworthy that the yield factor is approaching a value of 1 near the right edge of the model. At this location the depth of overburden is less than 500 feet, and yet the rock is almost yielding. Depths of overburden greater than this exist in other portions of the mine, and this would imply that in these locations, if the rock is really as weak as has been assumed, that the rock would be yielding in-situ. If this were to be the case,
uncontrollable, time-dependent roof sag and floor heave would be taking place in the mine openings at these locations. Since such behavior has not been observed, it is safe to conclude that the overburden is not as weak as has been assumed in this model.

CONCLUSIONS AND RECOMMENDATIONS

There are two main conclusions to be drawn from this set of finite element calculations, all supporting the "Bottom Line" conclusion that there will be no subsidence resulting from the proposed development of the two seam mining system beneath Boardinghouse Creek.

1. The first conclusion is that in all of the analyses performed, no evidence of subsidence, in the form of a continuous path of yielded rock connecting the mine to the ground surface, was observed as a result of developing a two seam mining system beneath Boardinghouse Creek.

3. The second conclusion is that even when the strength of the overburden material was reduced to an unreasonably low level, there was still no evidence of subsidence.

Based on these findings, we conclude that there is no danger of subsidence resulting from the development of the two-seam mining system as proposed. Even when conditions were exaggerated to favor subsidence, no indication of subsidence was observed.

Past experience with room-and-pillar mining in the Western U.S. indicates that a "plug" type subsidence may occur under certain geologic conditions, particularly for overburden depths of less than 150 feet. This criterion is based on empirical formulas, and could be relaxed if a rock mechanics investigation specifically addressing the issue of long term subsidence were to prove favorable. It is recommended, therefore, that until a rock mechanics investigation that
more directly addresses the issue of plug-type subsidence can be undertaken, full seam recovery should not be employed in the creek areas where the depth of overburden is less than 200 feet.

REFERENCES


___, "Ground Control Study for Multiple Seam Mining" Report to Valley Camp of Utah, 1988.


Figure 1. Location of reserves in relation to major geologic structures
Figure 2. Location of Boardinghouse creek area in relation to existing mine workings
Figure 3. Proposed mine layout in vicinity of Boardinghouse creek crossing with study section
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Figure 6. Profile of vertical equivalent stress in the upper seam after complete development of the top coal in the upper seam, and development of the bottom coal in the upper seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure 7. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam. Overburden quality = "FAIR"
Figure 8. Profile of yield factor in the overburden immediately above the upper seam after complete development of the top coal in the upper seam, and development of the bottom coal in the upper seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
APPENDIX A

FINITE ELEMENT MODELING

The basic finite element model used in all of the analyses is that shown in Figure 4, which models the cross section of Boardinghouse Creek shown as Section A-A in Figure 3. The proposed room and pillar grid is shown in Figure 3, from which it can be seen that Section A-A cuts across the panels normal to their axes. It can also be seen that the proposed separation between panels is about the same as the separation between rooms within a room and pillar panel, making it possible to represent the entire region as if it were one uniform room and pillar system.

There are 3 possible stages of panel development:

1. Unmined
2. Top Coal Recovery, in which the room and pillar panel has been developed in the top 10 feet of the seam only
3. Full Seam Recovery, in which the Double Lift mining technique has been used to claim the bottom coal

Within a given panel, the direction of room and pillar development is normal to Section AA, but the direction of panel development is along the section. To cover all possible configurations of developed and undeveloped regions, a worst case is considered in which all of the coal in a given seam on one side of Boardinghouse Creek is developed to a given stage, while all the coal on the other side is at the next lower stage of development. It is presumed that in the vicinity of Boardinghouse Creek, no panel will be developed to Stage 3 before the adjacent panels have been developed to at least Stage 2, and that no development of the lower seam will take place before
development of the upper seam in that area has been completed. Under these assumptions, the analyses performed herein set up the most severe conditions of discontinuity in support of the overburden at the point where the overburden is at its thinnest.

DESCRIPTION OF THE BMINES FINITE ELEMENT CODE

The BMINES computer program provides for the static, two- or three-dimensional, linear or nonlinear analysis of structural and geologic systems. The code was specifically designed for application to mining problems involving the simulation of excavation and construction sequences. This program was originally developed for the US Bureau of Mines by Agbabian Associates (Van Dillen et al., 1981) and was recently modified, without government sponsorship, to run on the IBM PC.

The capabilities of the BMINES code that are of particular interest for the investigation considered herein include primarily its ability to handle very large models. This is made possible by a non-core-resident equation solver that utilizes disk storage to accommodate stiffness matrices far too large to be contained in random access memory. The code also has the ability to model the excavation sequence through the activation and de-activation of elements. This allows one to observe the full stress history associated with a particular mine configuration. The mesh generator is invaluable in assembling large models in a reasonable length of time and with a minimum of error. The material library contains both linear and nonlinear material models, with a variety of plasticity laws and an ability to represent anisotropic, viscoelastic, and viscoplastic materials.

The load options include externally applied tractions and gravity self weight. Options available for other applications include joint or fault interface elements in
both two and three dimensions and a rockbolt element capable of modeling bolt rupture, bond failure, and dowel shearing across a joint interface. The program is limited to small deformation analyses.

MATERIAL MODELS

The finite element model of Figure 4 consists of only 2 material units, coal and overburden. The coal layers must be modeled in such a way that intact coal, top coal room and pillar development, and full seam recovery can all be represented. Three sets of material properties were used to represent these various stages of mine development in the coal layers. The remaining rock consists primarily of sandstone and shale in about 50-50 proportions. This host rock was modeled as a uniform equivalent material having properties that represent a compromise between sandstone and shale. The resulting hybrid material is called "overburden" for the purposes of these analyses.

The technique for modeling the coal layers in their varying stages of development has been developed in an earlier study of the Belina Mine (KCKA, 1989). This earlier study was performed in support of the Double Lift mining method, and consisted of a regional stress analysis of pillar loads and strains during Stage 2 and Stage 3 development. To perform this analysis efficiently, homogeneous materials were developed that duplicate the mechanical response of pillars in both Stage 2 and Stage 3 configurations. The mechanical properties of these pillars were determined by two separate three-dimensional analyses of single pillars in a region where the seam thickness is 22 feet. In one analysis the pillar occupies only the region between 10 and 20 feet above the base of the seam, corresponding to top coal recovery. In the other, the pillar extends from the base of the seam to a height of 20 feet, simulating full seam recovery.
These two models were compressed by prescribed deformations, from which stress/strain characteristics for each type of pillar could be developed. The seam thicknesses in the present analysis are 21 feet for the upper seam and 17 feet for the lower seam. However, because of the time requirements necessary to perform these 3-D pillar calculations, and because pillars of lesser height are stronger than pillars of greater height, it was judged to be consistent with the objectives of "worst case analysis" to incorporate the results of these earlier analyses into the material models of the present analyses.

The stress/strain properties of the full seam pillars were incorporated into Material 1, while Material 2 represents the difference in strength between the top coal pillars and the full seam pillars. Material 3 represents the difference in strength between intact coal and the top coal pillars. Three individual elements, with material sets 1, 2, and 3, respectively, all occupying the same coordinates in the model, are used to model the intact coal. Note that the three material strengths identified above add up to the properties of intact coal. When stages of the analysis corresponding to top coal development are encountered, the element corresponding to Material 3 is de-activated. This leaves Materials 1 and 2 active at the location, which add up to the strength of top coal pillars. At a later time, when full seam recovery is to be modeled at the location, the element corresponding to Material 2 is de-activated, leaving only Material 1, which represents the strength of the full seam pillar.

The overburden hybrid material contains stiffness and strength parameters that represent an average between sandstone and shale used in the earlier analyses at the Belina Mines (KCKA, 1989). The rockmass reduction factors of Hoek and Brown (1980) corresponding to "GOOD" rockmass quality were incorporated into two of the analyses.
(Appendices B and C), representing our judgment of the actual condition of the overburden. Two additional analyses (Appendices D and E) were performed in which the rockmass reduction factors corresponding to "FAIR" quality were utilized in the overburden.
APPENDIX B

ANALYSIS OF SINGLE LEVEL MINING IN BOTH SEAMS WITH OVERBURDEN QUALITY = "GOOD"

The figures contained in this appendix show profiles of stress in both the upper and lower seam at various stages of development for top coal development only. Overburden quality is assumed to be "GOOD" for these results. Diagrams at the top of each figure schematically indicate the configuration of mine development to which the results apply. Also shown are profiles of yield factor in the overburden immediately above the upper coal seam. Yield factor is defined as the ratio of maximum shear stress at a location to the existing shear strength at that location. Consequently, a yield factor of 1 would indicate that the rock is yielding, and a yield factor of less than 1 indicates that the rock has not yet been loaded to its full capacity. A yield factor in excess of 1 is not possible in an ideally plastic model in that plastic flow would modify the stress state back to a yield factor of 1.
Figure B–1. Profile of vertical equivalent stress in the upper seam prior to any mining activity. Overburden Quality = "GOOD".
Figure B-2. Profile of vertical equivalent stress in the upper seam after development of the top coal of the upper seam to a position below Boardinghouse Creek. Overburden Quality = "GOOD".
Figure B-3. Profile of vertical equivalent stress in the upper seam after complete development of the top coal of the upper seam. Overburden quality = "GOOD".
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Figure B–12. Profile of yield factor in overburden above upper seam after complete
development of the top coal in both the upper and lower seams.
Overburden Quality = "GOOD".
APPENDIX C

ANALYSIS OF DOUBLE LEVEL MINING IN BOTH SEAMS WITH OVERBURDEN QUALITY = "GOOD"

The figures contained in this appendix show profiles of stress in both the upper and lower seam at various stages of development for full seam development. Overburden quality is assumed to be "GOOD" for these results. Diagrams at the top of each figure schematically indicate the configuration of mine development to which the results apply. Also shown are profiles of yield factor in the overburden immediately above the upper coal seam. Yield factor is defined as the ratio of maximum shear stress at a location to the existing shear strength at that location. Consequently, a yield factor of 1 would indicate that the rock is yielding, and a yield factor of less than 1 indicates that the rock has not yet been loaded to its full capacity. A yield factor in excess of 1 is not possible in an ideally plastic model in that plastic flow would modify the stress state back to a yield factor of 1.
Figure C-1. Profile of vertical equivalent stress in the upper seam prior to any mining activity. Overburden Quality = "GOOD".
Figure C-2. Profile of vertical equivalent stress in the upper seam after development of the top coal of the upper seam to a position below Boardinghouse Creek. Overburden Quality = "GOOD".
Figure C-3. Profile of vertical equivalent stress in the upper seam after complete development of the top coal of the upper seam. Overburden Quality = "GOOD".
Figure C-4. Profile of vertical equivalent stress in the upper seam after complete development of the top coal in the upper seam, and development of the bottom coal in the upper seam to a position below Boardhouse Creek. Overburden quality = "GOOD".

Overburden configuration in upper seam, mine configuration in lower seam, unmined seam, single level room and pillar development, full seam room and pillar development.
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Figure C-6. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam, and development of the top coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "GOOD".
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Figure C-9. Profile of vertical equivalent stress in the lower seam after complete full seam development of the upper seam, and complete development of the top coal in the lower seam. Overburden Quality = "GOOD".
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Figure C-20. Profile of yield factor in overburden above upper seam after complete full seam development of the upper seam, and complete development of the top coal in the lower seam. Overburden Quality = "GOOD".
Figure C-21. Profile of yield factor in overburden above upper seam after complete full seam development of the upper seam, complete development of the top coal in the lower seam, and development of the bottom coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = “GOOD”.
Figure C-22. Profile of yield factor in overburden above upper seam after complete full seam development of both seams. Overburden Quality = "GOOD".
APPENDIX D

ANALYSIS OF SINGLE LEVEL MINING IN BOTH SEAMS WITH OVERBURDEN
QUALITY = "FAIR"

The figures contained in this appendix show profiles of stress in both the upper and lower seam at various stages of development for top coal development only. Overburden quality is assumed to be "FAIR" for these results. Diagrams at the top of each figure schematically indicate the configuration of mine development to which the results apply. Also shown are profiles of yield factor in the overburden immediately above the upper coal seam. Yield factor is defined as the ratio of maximum shear stress at a location to the existing shear strength at that location. Consequently, a yield factor of 1 would indicate that the rock is yielding, and a yield factor of less than 1 indicates that the rock has not yet been loaded to its full capacity. A yield factor in excess of 1 is not possible in an ideally plastic model in that plastic flow would modify the stress state back to a yield factor of 1.
Figure D-1. Profile of vertical equivalent stress in the upper seam prior to any mining activity. Overburden Quality = "FAIR".
Figure D–3. Profile of vertical equivalent stress in the upper seam after complete development of the top coal of the upper seam. Overburden Quality = "FAIR".
Figure D-5. Profile of vertical equivalent stress in the lower seam after complete development of the top coal in the upper seam, and development of the top coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure D-7. Profile of vertical equivalent stress in the lower seam after complete development of the top coal in both the upper and lower seams. Overburden Quality = "FAIR".
Figure D-8. Profile of yield factor in overburden above upper seam prior to any mining activity. Overburden Quality = "FAIR".
Figure D-10. Profile of yield factor in overburden above upper seam after complete development of the top coal of the upper seam. Overburden Quality = "FAIR".
Figure D-12. Profile of yield factor in overburden above upper seam after complete development of the top coal in both the upper and lower seams. Overburden Quality = "FAIR"
APPENDIX E

ANALYSIS OF DOUBLE LEVEL MINING IN BOTH SEAMS WITH
OVERBURDEN QUALITY = "FAIR"

The figures contained in this appendix show profiles of
stress in both the upper and lower seam at various stages of
development for full seam development. Overburden quality
is assumed to be "FAIR" for these results. Diagrams at the
top of each figure schematically indicate the configuration
of mine development to which the results apply. Also shown
are profiles of yield factor in the overburden immediately
above the upper coal seam. Yield factor is defined as the
ratio of maximum shear stress at a location to the existing
shear strength at that location. Consequently, a yield
factor of 1 would indicate that the rock is yielding, and a
yield factor of less than 1 indicates that the rock has not
yet been loaded to its full capacity. A yield factor in
excess of 1 is not possible in an ideally plastic model in
that plastic flow would modify the stress state back to a
yield factor of 1.
Figure E-1. Profile of vertical equivalent stress in the upper seam prior to any mining activity. Overburden Quality = “FAIR”.
Figure E-2. Profile of vertical equivalent stress in the upper seam after development of the top coal of the upper seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E-3. Profile of vertical equivalent stress in the upper seam after complete development of the top coal of the upper seam. Overburden Quality = "FAIR".
Figure E-5. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam. Overburden Quality = "FAIR".
Figure E-6. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam, and development of the top coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = “FAIR”.
Figure E-7. Profile of vertical equivalent stress in the lower seam after complete full seam development of the upper seam, and development of the top coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E–8. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam, and complete development of the top coal in the lower seam. Overburden Quality = "FAIR".
Figure E-9. Profile of vertical equivalent stress in the lower seam after complete full seam development of the upper seam, and complete development of the top coal in the lower seam. Overburden Quality = "FAIR".
Figure E-10. Profile of vertical equivalent stress in the upper seam after complete full seam development of the upper seam, complete development of the top coal in the lower seam, and development of the bottom coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E-11. Profile of vertical equivalent stress in the lower seam after complete full seam development of the upper seam, complete development of the top coal in the lower seam, and development of the bottom coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E-12. Profile of vertical equivalent stress in the upper seam after complete full seam development of both seams. Overburden Quality = "FAIR".
Figure E-13. Profile of vertical equivalent stress in the lower seam after complete full seam development of both seams. Overburden Quality = "TAIR".
Figure E-14. Profile of yield factor in overburden above upper seam prior to any mining activity. Overburden Quality = "FAIR".
Figure E-19. Profile of yield factor in overburden above upper seam after complete full seam development of the upper seam, and development of the top coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E-20. Profile of yield factor in overburden above upper seam after complete full seam development of the upper seam, and complete development of the top coal in the lower seam. Overburden quality = 'FAIR.'
Figure E-21. Profile of yield factor in overburden above upper seam after complete full seam development of the upper seam, complete development of the top coal in the lower seam, and development of the bottom coal in the lower seam to a position below Boardinghouse Creek. Overburden Quality = "FAIR".
Figure E-22. Profile of yield factor in overburden above upper seam after complete full seam development of both seams. Overburden Quality = "FAIR".
Mr. David B. Miller  
Business Manager  
Lodestar Energy, Inc.  
H.C. 35, Box 370  
Helper, UT 84526

RE: Whisky Creek No. 1 Mine  
ID No. 42-02315  
Ground Control Plan with  
Blasting Addendum

Dear Mr. Miller:

The referenced ground control plan has been reviewed by MSHA personnel and will be filed in the mine file in accordance with 30 CFR 77.1000-1. The plan consisted of a cover letter, dated July 30, 2001, with page 1 and two maps, one titled "Whisky Creek #1 Surface Mining Plan", which shows underground mine workings, and the other titled "Spoil Backfill and Reclamation Details", which shows pit cross-sections; a cover letter, dated September 24, 2001, with pages, 2, 4, and 5, a drawing titled "Whisky Creek", and a three-page blasting addendum with three sheets; and a cover letter, dated October 11, 2001, with page 3 and a map titled "Nearest Structures".

Should a subsequent inspection by an authorized representative of the Secretary determine that the plan does not ensure safe working conditions for all persons, appropriate action will be taken.

If auger, punch or other types of highwall mining systems are to be considered in the future, a ground control amendment will be required. Planning for any highwall mining system should include a safety catch bench, immediately above the work area, in the final pit highwall(s). Additional details should include, but are not limited to, coal web thickness between openings, opening dimensions, seam thickness, safety factor, barrier pillar design, and calculations necessary to ensure prudent mining design.

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DIV OF OIL GAS & MINING
If you have any questions regarding the filing of this plan, please contact Billy Owens at 305-231-5463, extension 145, or Kathleen Kelleher, Delta FO, at 970-874-7637.

Sincerely,

Allyn C. Davis
Acting District Manager

Enclosure
July 30, 2001

District Manager
CMH&S
P.O. Box 25367
Denver, Co 80225-0367

Re: Whisky Creek No. 1 Mine  I.D. No. 42-02315

Dear Sir,

Enclosed for your approval are revised copies of a Ground Control Plan and Blasting plan for the above referenced mine.

If you have any questions, please contact me at (435) 637-9200. Thank you for your cooperation.

Sincerely,  
Rodney Head
Loss Prevention Coordinator

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OCT 26 2001
DIV OF OIL GAS & MINING
September 24, 2001

Mr. Allyn Davis  
District Manager CMH&S  
P.O. Box 25367  
Denver, Co  80225-0367

Re: Whisky Creek No. 1 Mine  I.D. No. 42-02315

Dear Sir,

Enclosed for your approval are revised copies of a Ground Control Plan and Blasting plan for the above referenced mine.

If you have any questions, please contact me at (435) 637-9200. Thank you for your cooperation.

Sincerely,

Rodney Head  
Loss Prevention Coordinator
October 11, 2001

Mr. Allyn Davis  
District Manager  
CMS&H  
P.O. Box 25367  
Denver, CO 80225-0367

RE: Whisky Creek mine I.D. No. 42-02315 Ground Control and Blasting Plans

Dear Mr. Davis:

Enclosed please find replacement pages for the above referenced plans. The Ground Control Plan is revised on page 3 to say that the maximum highwall height between benches will be 60 feet.

The revision to the blasting plan is the Nearest Structure Map that shows the nearest structure as a natural gas line 960 to the west of the mining area. The gas line was inadvertently left off the map.

If you have any questions or require additional information, please free to contact me at 1-877-905-5633, Ext. 15.

Sincerely,

Rodney Head  
Loss Prevention Coordinator

CC: McClane Canyon
ESTABLISHED
GROUND CONTROL PLAN
FOR

DATE    July 27, 2001

LODESTAR ENERGY, INC.
(COMPANY)

WHISKY CREEK MINE NO. 1
(MINE)

SCOFIELD
(LOCATION)

42-02315
(MINE I.D.)

LODESTAR ENERGY, INC.
(OPRATOR'S NAME)

SCOFIELD ROUTE HC 35 BOX 370
HELPER, UTAH 84526
(OPRATOR'S ADDRESS)

435-637-9200
(PHONE)

SIGNED:  
TITLE:  BUSINESS MANAGER

REQUIRED BY TITLE 30 CFR

REVISED:  July 27, 2001
GROUND CONTROL PLAN

Date 7-20-01

Mine WHISKY CREEK MINE NO. 1  ID No. 42-02315
Mine Location SCOFIELD ROUTE, HWY 246  County CARBON
Company LODESTAR ENERGY, INC.
Mailing Address SCOFIELD ROUTE, HC 35 BOX 370 HELPER, UTAH 84526

Type of Mining

[ ] Mountain Top Removal
[X] Contour Mining (See page 3 for detailed description)
[ ] Auger Mining
[ ] Highwall Mining
[ ] Underground Mine Face-up

Additional Information

[X] Blasting Plan
[X] Mine Map
[X] Other

Name of Coal Seams  Elevation of Coal Seams  Seam Height

1. Upper O’Conner  9046.2  20’
2. Lower O’Conner  8944.2  20’

List of Equipment

Loaders 992 or equivalent, 988 or equivalent, 980 or equivalent
Dozers D-11 or equivalent, D-9 or D-10 or the equivalent
Rock drill Ingersol-Rand DML, 5 to 9 inches hole diameter
Overburden trucks 777 or equivalent
Tandem trailer highway trucks for coal haulage
Track Hoe Caterpillar 345 BL

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Description of Mining Method

The method to be utilized if referred to as contour mining. This method develops a pit along the lower seam by removing the rock material that covers the seam. This is accomplished by either moving the material with a bulldozer or by blasting the material and then using a bulldozer. If blasting is used, the maximum drill hole depth is 60 feet, at an angle between 80 to 90 degrees from the horizontal. A bench will be established and then the remaining material will be blasted. The established bench(s), if necessary, will be cleaned with a bulldozer. The maximum highwall height between benches will be 60 feet. The highwalls will be scaled with a bulldozer as the blasted material is removed from the area. When the blasting area moves over the portal areas of the Lower / Upper O’Conner seam the drill operator will locate the coal seam and then raise the bottom of the drill holes 5ft. to 10 ft depending on the type of strata and allow the blasted material to fill the voids left by the underground mining. Also, see page 5 for additional information regarding the underground mine portals. The rock material will be stored adjacent to this area and then will be used to reclaim the final pit area. Once the rock has been removed from a pit location the coal will be removed using road graders or end loaders to clean the rock from the top of the coal and wheel loaders and track hoes will be used to load contractor operated bottom dump tandem highway trucks in the pit.

The upper seam will be mined using the same method except the overburden and interburden will be used to fill the pits behind the coal removal process. No personnel will be allowed directly below an active pit.
GEOLOGIC CROSS SECTION OF  
OVERBURDEN AND INTERBURDEN

Drill Hole 75-30-3

<table>
<thead>
<tr>
<th>Layer</th>
<th>Thickness</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil and alluvium</td>
<td>7&quot;</td>
</tr>
<tr>
<td>Sandstone</td>
<td>12'</td>
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<tr>
<td>Sandy Shale</td>
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<tr>
<td>Shale</td>
<td>12'</td>
</tr>
<tr>
<td>Lower O'Conner</td>
<td>24'</td>
</tr>
</tbody>
</table>

elev. Top – 9,046.2  elev. Bottom – 9,024.2

elev. Top – 8944.2  elev. Bottom – 8,920.2

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DIV OF OIL GAS & MINING

OCT 2001

ACCEPTED/FILED

11/1/2001

OCTOBER 2001
OTHER ADDITIONAL INFORMATION

Additional Precautions

The mine plan has two locations where the highwall and pit will pass over and through the portal areas of the White Oak No. 1 mine (Upper O'Conner seam) and the White Oak No. 2 mine (Lower O'Conner seam). The White Oak No. 1 mine has noncombustible material pushed into the portals while the White Oak No. 2 mine is not sealed at the portals. Internal seals have been constructed a distance of 1000 feet inside the portals. Material will be pushed into the White Oak No. 2 mine portals to prevent access prior to the start of the surface mining.

In mining through the portal areas, an additional bench will be provided over the portals for additional highwall stability. While the active pit is in the area of underground workings, openings will have blasted overburden material pushed into them to prevent access. After coal is removed from the pit, blasted material will be placed over these openings from the adjacent pit. At least 50 ft. of material will be placed over the mine openings when final grades are achieved on the spoil.

The area to be mined contains a spring and two (2) seeps along the upper portion of Whisky Creek. The spring and seeps are active from late May to late June and flow a maximum of 2 gpm. These flows are not expected to degrade the highwall or create spoil instability due to the minimal flows. The Whisky Creek flows only in late April to the end of July during the main snow melt. The area affected by the flow of Whisky Creek will be mined during the winter months. Any flows encountered during mining will be contained in the pit and pumped to one or both of the sedimentation ponds on the mine site, if water depths in the pit prevent coal removal.

There are no bodies of water dammed or held back in either seam to be mined that will have any effect on mining as both seams dip back away from the proposed mining areas.

All equipment, working over underground mine works, will be provided with self contained self rescuers and the operators will be trained in the proper donning procedures. The mine currently does not liberate methane. The active bleeder entries do not liberate methane.

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BLASTING PLAN

INTRODUCTION

The blasting operation will be conducted near the community of Scofield, Carbon County, Utah. The surface mine is a multiple coal seam contour operation. The depth of the overburden between seams ranges from 1' (one foot) to 80' (eighty-feet) and consists of sandstone, sandy shale and shale. There are no known faults or other geological anomalies within the strata of the contour operation. There are no active underground mines within 500' (five hundred feet) of the blasting area, nor any residence within 1,000' (one thousand feet) of the blasting area.

DRILL SIZES, DEPTHS, & PATTERNS

The drill pattern to be used during this blasting operation consists of a square or rectangular pattern. The depth of the blast holes will range from 5' (five feet) to 60' (sixty feet). There may be more than one diameter drill used on the operation, with diameters ranging from 5" (five inches) to 9" (nine inches). The burden and spacing of the blast holes will vary from 10' (ten feet) to 25' (twenty-five feet) depending on the needs of the operation. Please refer to the typical borehole loading profile and shot pattern.

EXPLOSIVES TO BE USED

The explosives to be used during the blasting operation consist of a blasting agent detonated by a primer of high explosives. The blasting agent is a mixture of ammonium nitrate prills and fuel oil (ANFO). The ANFO has a density (specific gravity) of approximately 0.85 to 1.04. The ANFO will be "bulk" loaded into the blast holes and detonated with a 16 oz. (sixteen ounce) cast primer strung on detonating cord attached to a blasting cap. During wet conditions or holes, a blasting agent packaged in water resistant bag will be used.

DELAY PERIODS & DECKING

The ignition and delay system to be used will primarily be non-electric, with electric ignition systems used as appropriate. When the distances to the closest dwelling do not allow detonation of the entire column of explosives within the blast hole, a "down the hole delay system" will be used. The system will use the method of loading the blast holes with smaller and separate explosive charges or "decks". Each deck will be detonated one at a time, and separated by at least 8 ms. (eight milliseconds).

STEMMING

The stemming will consist of drill cuttings and/or gravel. The length of the stemming material will depend on the amount of explosives and the number of decks in each blast hole. Stemming lengths should range from 10' (ten feet) to 12' (twelve feet). However, the amount of stemming will be sufficient to confine and protect the explosives.
MAXIMUM POUNDS PER DELAY

The maximum pounds per delay will not exceed the total amount of explosives as determined by the scale distance factor to nearest dwelling and what would be a safe level for structures in the surrounding area.

Scale Distance Factor refers to the formula: \( W = \frac{(d/f)^2}{2} \). Where \( W \) is the weight of explosives in pounds, \( d \) is the distance to the nearest structure and \( f \) is a constant of 55.

MAXIMUM POUNDS OF LARGEST TOTAL CHARGE

The maximum pounds of the largest total charge proposed on this operation is 40,000 pounds (forty thousand pounds), with the amount of explosives to be detonated per delay limited by what would be a safe level for the nearest control structure.

SEISMOGRAPH MONITORS

The company will not monitor blast with a seismograph. The required scale distance factor will be used to monitor all blasts. If monitoring with a seismograph becomes necessary the blasting seismographs to be used for the monitoring will be the Dallas Instruments, Inc. BR2-3LF Blast Monitor. If the vibration levels cannot be recorded, the necessary scale distance factor will be observed. The seismographs will be located at or near the closest dwellings or within 1000 feet of the blasting area.

CLOSEST STRUCTURE

The closest structure to the blasting area is a Questar Gas 16" pipeline to the west of the blasting area. The closest distance the pipeline comes to the blasting area is 960 feet. See attached map showing the mine plan area where blasting is to occur and the closest distance to the gas pipeline.

PERSONS TRAINED IN USE OF EXPLOSIVES

All persons assigned to work in the blasting area will be trained with regard to the provisions of 30 CFR 77.1300.

BLASTING AND WARNING SIGNS, ACCESS CONTROL, SIGNALS

Blasting signs will read "BLASTING AREA" and be conspicuously placed along the edge of any blasting area that comes within 100 feet of any public road right of way, and at the point where any other road provides access to the blasting area. At all entrances to the mine from public roads or highways, signs conspicuously placed which read "WARNING! EXPLOSIVES IN USE", which clearly list and describe the meaning of audible blast warning and all-clear signals in use and explain the identification of blasting areas where charged holes await firing at the blasting site in the mine area. An authorized company representative will prohibit access to the blasting area by unauthorized persons at least 10 minutes before each detonation. Before each blast is detonated, the

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NOTE:

- THE DRILL OPERATORS SHALL NOT OPERATE THE DRILL EQUIPMENT IN A MANNER WHICH EXPOSES THEM TO DANGEROUS CONDITIONS OR RISKS.
- NO LOADING OR OPERATING MAY OCCUR AT A DISTANCE LESS THAN 50 FEET FROM THE DRILL HEAD OR DRILLING AREA.
- THE DRILL HEAD OR DRILLING AREA MUST BE SECURED TO THE WALL OR OTHER STRUCTURE.
- ALL OPERATORS MUST WEAR PROPER PERSONAL PROTECTIVE EQUIPMENT (PPE) AT ALL TIMES.

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OCT 26 2001

DIV OF OIL GAS & MINING
November 19, 2001

Lodestar Energy Inc.
Mountain Operations
HC 35 Box 370
Helper, UT 84526

TO: DAVE MILLER
FROM: RUTH ANDERSSEN
RE: PRE-BLAST SURVEY QUESTAR GAS PIPELINE
LOCATION: SCOFIELD, UT

PIPELINE RESPONSE TO BURIED EXPLOSIVE DETONATIONS

In 1975 through 1980 a research program was performed by Southwest Research Institute (SwRI) for the Pipeline Research Committee (PRC) for the American Gas Association (A.G.A.).

Many states have a ground motion criteria which limits maximum ground particle velocity to either 1.0 or 2.0 inches/second at the surface. This soil particle velocity criteria evolved from work published by Crandell (1949) for the effects of ground shock on buildings. More recent experimental work investigating the effects of buried charges on buildings such as that of Dovak (1962) in Czechoslovakia and Nicholls, et al. (1971) using data obtained by Thoenen and Windes (1942), Langefors, et al. (1958), and Edwards and Northwood (1960) basically show that threshold soil particle velocity criteria are reasonable when applied to above-ground structures. However, a pipeline is not a building. A steel pipe is a strong structure relative to a building, and when buried, also has a large mass of earth providing additional inertial resistance to any ground shock from a buried detonation. These soil particle velocity criteria have been applied to buried pipelines because: 1) they are simple (even if incorrect), and 2) nothing else existed except for the Battelle equations.

The Battelle equations, McClure, et al. (1964), were developed at the Battelle Memorial Institute under contract for the PRC. These equations are theoretical elasticity solutions based upon Morris' equation (1950) for ground motion, and the assumptions that: 1) a pipeline movement equals exactly that of the surrounding soil, and 2) no diffraction of shock front occurs.

A knowledge of the state of stress caused by blasting is only one of the stress parameters necessary to determine if a buried pipe will yield. Other loading mechanisms also cause a pipe to be stressed. Because of symmetry, circumferential and longitudinal stresses from blasting and other effects are principal stresses. This observation means that an accurate estimate of the elastic state of stress can be
made by superimposition through addition of stresses with their signs considered.

Stress from other causes including longitudinal and circumferential stresses can be caused by:

1) Internal pipe pressurization
2) Thermal expansion or contraction
3) Surcharger or overburden
4) Residual stresses from welding and other installation processes

Strains in a restrained structure (pipelines) will usually be those of the ground surrounding the structure. These may sustain axial strains, and or hoop strains. Axial strains can occur through compressive wave propagation parallel to the long axis. Bending strains can be produced by shear and Rayleigh waves propagating parallel to the long axis. Hoop strains occur when compressive or shear waves propagate perpendicularly the pipe's long axis and cause changes in circumferential stresses.

All of the critical vibration levels for restrained structures are a good deal larger than the traditional 50-mm/s/s (2-in./sec. safe blasting criteria for residential structures. This is a result of the radically different nature of the materials employed in the construction of restrained structures, which for the most part are engineered.

When blasting next to pipelines within 100 feet, gas pressures produced by explosions contribute significantly to the fragmentation process. The large volumes of gas can produce back break of unusually large extent in sedimentary rock with open bedding. As a result, care should be exercised when blasting is employed to isolate large rock blocks. However, the closest point to the blast zone at this site is 954 feet horizontally with a elevation of 9075. The gas line elevation is 9365 with an additional distance of 290 feet vertically. However, when calculating the ground motion from blasting the closest point should always be used for additional safety parameters and in calculating ground motion.

The geology in this area is large horizontal bedding planes of sandstone with thin layers of shale. The shale beds will aid in absorption of more explosives energy than if the geology was homogeneous. These create discontinuities and refractions of the wave trains.

Rock masses are inhomogeneous, ground motion waves travel through strata of different acoustical impedance. Scattering of the ground motion waves, initiated at boundary discontinuities by reflections, lowers the peak vibration levels. Interactions between reflected pulses alter the frequency composition of the wave train. High frequencies are selectively attenuated while some lower frequencies are added to the ground vibrations. On this site however, the closest point of detonation is at 954 feet to the gas pipeline. The presence of joints, fractures, faults, and shear zones in the path of a ground motion wave also act to scatter the peak vibrations. Some of the lateral components of ground motion are lost as the wave crosses a discontinuity. The degree of redirection and dissipation of a ground motion wave is related to the nature and frequency of structural discontinuities in rock.

Seismic wave paths: A seismic waveform represents ground motion at the receiver location as a function of time. It is the product of body waves and surface waves following different travel paths through the earth. Although a seismic waveform may show several distinct arrivals, the easiest and most reliable arrival to identify is that of the P wave (first arrival).

P wave travel paths in the earth are commonly categorized as:

1. The direct wave path
2. The reflected wave path
3. The refracted wave path

The travel time for the direct, refracted, and surface waves increases linearly with distance.
Blast design submitted by WMG has hole diameters of 6.75 inches, hole depth 40 feet and maximum lbs. per delay at 350lbs. Using the scale distance formula, Scale distance then becomes 50.98. Using this measurement a predicted peak particle velocity can be assumed of .3647 ips. on the gas line with the above information. This is assuming that all criteria is met. Using a scaled distance of 50 implies that the probability of producing a peak particle velocity greater than 2 in./sec is very small. At large distances from the blast, the factors of blast design become less critical and the transmitting medium of rock and soil overburden dominates the wave characteristics. Vibration frequency, displacement, and acceleration amplitudes depend strongly on the propagating media. Thick soil overburden and large distances create long-duration, low-frequency wave trains. High frequency wave trains are the most preferred.

In conclusion a scale distance of 50 should provide a safe margin for blasting near the buried pipeline if all parameters submitted from the blasting contractor are accurate.

Respectfully,

Ruth Andersen

RA/laa
Ruth Anderson  
Salt Lake Seismic  
Grantsville, Utah U.S.A. Operations  

Re: Lodestar Energy Seismic Analysis  

Dear Ruth;  
  Congratulations on your success, both locally and internationally, (don't forget your large diameter friends when your borehole needs to increase, especially overseas).  

Diameter 6.75  
Avg. Depth 40'  
Avg Pounds per delay 350  
Cast Boosters Yes (1 pound)  

Sincerely;  

Jeff Wolfe  
Driller
Data requested by Lodestar from Questar to assist in a preblast survey

<table>
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<th>ML104*</th>
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* Note: ML 40 and ML 104 Parallel each other.
Visual Inspection of Questar Gas Pipeline within One-half Mile of Whisky Creek Surface Mine Blasting Area Pre-blast Survey

A pre-blast survey of visually examining the Questar Gas pipeline located within one-half mile of blasting that will be conducted at the Whisky Creek surface mine was conducted on Wednesday, December 19, 2001 between the hours of 10:30 AM and 1:50 PM by David B. Miller - Business Manager and Professional Engineer. Three other individuals accompanied in the survey. Phil Gordon, Raymond Jensen, and Rick Richey. The survey was conducted on snow mobile. The gas pipeline was traveled the entire length north and south and a majority of the distance east and west. The very steep hillside to the south and west was only done in safe accessible sections.

The survey consisted of placing flagging on the gas pipeline sign posts, measuring the snow depth, making observations for irregularities in the snow pack, video taping each site and in both directions from each site, and taking at least three digital photographs of each site. The weather was clear to partly cloudy and the temperature was between 10 to 15 degrees F.

This study will provide a copy of the video tape to be placed in the public room at the Division. The study will include at site by site description with photographs. The survey will be walked in the spring after the snow pack has completely melted.

Site 1: This site is located on the ridge-line where the road from the mine to Boardinghouse Canyon intersects the gas pipeline ROW. The site is located on a flat area of a ridge. This site is along the old 16 " gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. Almost all snow was blown from around this site. This site is undermined by both White Oak #1 and #2 Mines

Looking South

Looking West

Looking North

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Site 2: This site is also located on the ridge-line where the road from the mine to Bordinghouse Canyon runs along the gas pipeline ROW. The site is located on a flat area of a ridge. This site is along the old 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 1.6 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.

Site 3: This site is also located on the ridge-line where the road from the mine to Boardinghouse Canyon runs along the gas pipeline ROW. The site is located on a flat area of a ridge. This site is along the old 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was slightly wind blown and 1.6 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.

Site 4: This site is also located on the ridge-line where the road from the mine to Boardinghouse Canyon turns off the gas pipeline ROW into the trees. The site is located on a slight grade of a ridge. This site is along the old 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was pristine and 2.5 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.
Site 4 (continued)

Looking West  Looking Southwest  Looking North

Site 5: This is located at the top of a steep rise in the gas pipeline ROW. The access is not near the road and is cut through the trees. The site is located on a slight grade. This site is along the old 16 "gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was slightly wind blown and 2 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines

Looking North  Looking East  Looking West

Site 6: This is located at the base of a steep rise in the gas pipeline ROW. The access is approximately 200 feet from the road and is in an open area. The site is located on a slight up-grade. This site is along the old 16 "gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 3.1 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines
Site 6 (continued)

Looking North          Looking Southeast          Looking South

Site 7: This is located near the top of a steep rise in the gas pipeline ROW. The access is approximately 800 feet from the road and is in an open area at the edge of the cut through the trees. The site is located on a steep up-grade. This site is along the old 16 " gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 2.3 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines

Looking West          Looking North          Looking East

Site 8: This is located near the mid-point of a steep rise in the gas pipeline ROW. The access is just off the road to Boardinghouse Canyon and is in an open area at the edge of the cut through the trees. The site is located on a steep up-grade. This site is along the new 24 " gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 2.4 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines
Site 8 (continued)

Looking West  Looking West  Looking Northeast

Looking East  Looking Northeast  Looking Northeast

Site 9: This is located near the base of a steep rise in the gas pipeline ROW. The access is just off the road to access a gas well site near a drainage emptying into Boardinghouse Canyon and is in an open area. The site is located on a slight up-grade. This site is along the gas pipeline ROW for all three lines. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 1.8 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.
Site 10: This is located on a relatively flat area in the gas pipeline ROW. The access is just off the road that runs along the ridge-line. The site is located on a slight side-grade. This site is along the gas pipeline ROW for all three lines. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 1.3 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.

Looking West

Looking South

Looking East

Site 11: This is located on a relatively flat area in the gas pipeline ROW. The access is just off the road that runs along the ridge-line. The site is located on a slight side-grade. This site is along the gas pipeline ROW for all three lines. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 1.2 feet deep at this site. This site was not undermined.

Looking South

Looking East

Looking North

Site 12: This is located on a relatively flat area in the gas pipeline ROW. The access is just off the road that runs along the ridge-line. The site is located on a slight side-grade. This site is along the gas pipeline ROW for all three lines. This is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 0.8 feet deep at this site. This site was not undermined.
Site 14 (continued)

Looking West          Looking Northwest          Looking South

The survey crew then returned to Site 1 to continue along the 16" gas pipeline to the north.

Site 15: This is located 500 feet to the north of Site 1 near the top of a knoll along the gas pipeline ROW. The access is just off the gas line access road that runs along the ridge-line. The site is located on a medium up-grade. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 0.4 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.

Looking West          Looking North           Looking South

Site 16: This is located opposite side of a knoll along the gas pipeline ROW. The access is just off the gas line access road that runs along the ridge-line. The site is located on a slight down-grade. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 0.7 feet deep at this site. This site is undermined by both White Oak #1 and #2 Mines.
Site 16 (continued)

Looking North  Looking East  Looking West

Site 17: This is located on the ridge-line along the gas pipeline ROW. The site is located on level area. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown, drifted and 1.8 feet deep at this site. This site is undermined by White Oak #2 Mine.

Looking West  Looking Northwest  Looking North

Site 18: This is located on the ridge-line along the gas pipeline ROW. The site is located on level area. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown, drifted and 1.6 feet deep at this site. This site was not undermined.
Site 19: This is located on the ridge-line along the gas pipeline ROW. The site is located on level area. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 0.6 feet deep at this site. This site was not undermined.

Site 20: This is located on the ridge-line along the gas pipeline ROW. The site is located on slight down grade area cut out in the aspen trees. This site is along the 16" gas pipeline ROW. This site is shown on the attached map.

Data: There were no noticeable cracks. The snow was wind blown and 1.6 feet deep at this site. This site was not undermined.
Site 20 (continued)

Looking East  Looking North  Looking West

Site 21: This is located on the ridge-line along the gas pipeline ROW. The site is located on slight down grade area cut out in the aspen trees. This site is along the 16" gas pipeline ROW. This site is shown on the attached map. This was the last site.

Data: There were no noticeable cracks. The snow was wind blown and 1.4 feet deep at this site. This site was not undermined.

Looking East  Looking Southeast  Looking South

This report was compiled using video camera, digital camera, level rod for snow depth. Site was walked after photos to determine whether any surface cracks were evident. I certify that all data contained in this report identified the conditions that were present at the time of the survey.

David B. Miller P.E. 7418

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FEB 27 2002
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Evaluation of Underground Mining Under the Questar Pipeline Corridor Within the One-half Mile Distance of Blasting at Whisky Creek Surface Mine
Underground Mining Evaluation

This report is compiled to address concerns of the amount of support and the stability of the pillars remaining under the Questar Gas Pipeline within the one-half mile distance of blasting that will occur during the operation of the Whisky Creek Surface Mine located in the head of the Whisky creek Canyon. This report will identify the corridor of protection on a 35 degree angle of draw to the Upper O’Connor seam workings in the White Oak #1 Mine and to the Lower O’Connor seam workings in the White Oak #2 Mine.

Areas were selected within the corridor to illustrate the percentage of coal left in that area for support of the gas pipeline. The mining in these mines were conducted by three separate mining companies and the record used was from the mine maps that where submitted to the Mine Health and Safety Administration as certified copies of the mining completed. The calculation of the percentage of coal left as support was accomplished by identifying an area and measuring the area to determine the total square feet of coal that was in the area prior to mining. The entries and crosscuts were measured for length and width and an area of removed coal was calculated. The percent recovery was calculated by dividing the recovered area by the total area. The percentage of coal remaining was determined by subtracting the percent recovery from one. Table 1 contains the calculations for the Upper O’Connor Seam Mine #1 and Table 2 contains the calculations for the Upper O’Connor Seam Mine #2.

The tables show that in all instances the percentage of coal left as support was greater than 50 percent. On Table 1 the percent of support coal ranged from 50.16% in Area E to 72.04% in Area C identified on the Map titled Upper O’Connor Seam White Oak #1 Mine Gasline Protection Corridor Stability. Similar areas of support are provided on Table 2. These areas were selected to coincide with the areas in the upper seam. The support coal on Table 2 ranged from 51.25% in Area H2 to 68.45% in Area F2 identified on the Map titled Lower O’Connor Seam White Oak #2 Mine Gasline Protection Corridor Stability.

Please refer to documents already included in the permit for further details see the Ground Control Study For Double Lift Mining Panels; August 1989; Kenneth C. Ko & Associates report in Appendix 523 Labeled “Confidential”. Copies are held at the Division office in Salt Lake City, Utah.

Locations of pillar stability analysis are depicted on both mine maps included in this report. The evaluation of the stability factors analysis was completed by using NIOSH’s ARMPS (Analysis of Retreat Mining Pillar Stability) program. The program was developed to prevent pillar squeezes, massive pillar collapses, and bumps critical to the safe and efficient retreat mining of coal. The program calculates stability factors based on loads applied to, and the load bearing capacities of pillars during development and retreat mining operations. When the ARMPS safety factors fall below 0.75 satisfactory support characteristics are rare and greater than 1.5 the support characteristics are above being acceptable.

The ARMPS program was used on various pillar locations along the corridor using the overburden depth, an entry width of 18 feet, a development height of 9 feet and a total height after bottom coal
Underground Mining Evaluation

removal of 15 feet. The overburden thickness was determined by the difference in elevation from the surface to the coal roof elevation contours shown on each map. The results of this analysis are shown on the sheets of Stability Factors for White Oak #1 Mine and Stability Factors for White Oak #2 Mine. The lowest safety factor of 0.99 after bottom coal mining is at Site 11 in White Oak #2 Mine. This area was mined in third and fourth quarter of 1994. The remaining safety factors are greater than 1 and less than 3.84. The average stability safety factor for all the sites evaluated in Mine # 1 is 2.07 and 1.72 in Mine #2.

The maps also include the presence of faulting in the area. The O’Connor and Connelville Faults are the two large displacements which bound the mining in the Upper and Lower O’Connor seams mine in the White Oak #1 and #2 Mines. There are also four smaller unnamed faults and an igneous dike that cross the pipeline corridor and were contacted and in some instances crossed by the underground mining. These are depicted on both maps included in this report.

The existence of subsidence and the monitoring of locations where mining has been done in areas of less than 200 feet of overburden and in areas where greater than 50 % recovery was practiced. The resulting subsidence resulted in a modification of the permit eliminating the mining of coal by underground methods with less than 200 feet of cover. This eliminated the of mining extensive areas on small pillar centers in shallow cover.

The subsidence monitoring that was completed in 2000 and submitted as part of the Annual Report sent to the Division shows that the subsidence that has occurred had been most evident in areas of greater than 50 % recovery and lower overburden. The subsidence features closest to the gas pipeline corridor are sites B, 25, 26, 50, and 39. The 2000 survey completed by Vicky Miller - Environmental Monitoring Consultant showed that was a crack 1' deep and 50 feet long in 1988 and was determined healed where it blended into the surrounding terrain in 1999. Sites 25 and 26 were identified in 1991 as crack fractures and the 2000 survey shows them as continuing to weather and heal. Site 50 was identified in 1995 as a crack fracture and the 2000 survey shows it as continuing to weather and heal. Site 39 was identified in 1987 and the 2000 survey shows them as continuing to weather and heal. These subsidence site occurred as tension cracks that occurred in the vertical proximity of a remaining barrier at the edge of a panel all and but site 39 occurred on directly stacked workings. Site 39 was not directly stacked and it occurred along the barrier left in the White Oak #1 Mine workings. The results of the 2001 survey will be submitted in the 2001 Annual Report. Subsidence monitoring is an annual requirement of the existing permit.

Contact was made with Mark Mackiewicz - BLM Price to gather any additional information on the requirements placed on the gas pipeline company when encountering geologic hazards such as faults and underground mine workings. Special installation procedures were recommended in this report. The report is attached as Appendix A-1. Also copies of pages A-20, A-21, A-22, B-1, B-2, and B-3 were copied from the internet site at www.qwk-eis.org Volume II of Environmental Impact Statement Questar, Williams, & Kern River Pipeline Project Appendix A- Performance Standards and Supporting Documents to the Performance Standards and Appendix B- Other Issues Evaluated

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Underground Mining Evaluation

Section B.1 Geologic Hazards.

The pre-blast survey completed by Ruth Andersen - Salt Lake Seismic did not include a visual survey of the entire length of buried pipeline within the one-half mile zone of the closest blasting. Lodestar will have a video completed from snowmobile of the pipeline corridor on Wednesday, December 19th. A copy of this video will be provided to the Division to keep in the records room. An additional video will be taken in the spring once snow melt is complete. The areas of the subsidence sites along the corridor will be videoed along with any other anomalous features.

The concern pertaining to blasting adjacent to underground workings is controlled in the MSHA and the State of Utah Coal Mining Rules. The regulations controlling blasting within 500 feet of active underground mine workings are designed to protect the personnel that work in those mines from falling roof fragments. If this blasting would create such a hazard as to destroy the pillar supports within the closest mine areas, then blasting would not be allowed adjacent to active mines. The plan requires special plans providing blasting pattern and blasting load per delay. The scale distance factor has been designed for the protection of all structures within a certain distance from the blast. The blasting contractor has provided data as to the largest amount of explosive to be detonated in the deepest hole to fall within the parameters of the scale distance factor calculations and they are part of the pre-blast survey. Both an approved Blasting Plan and a pre-blast survey have been provided in the permit.
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### Stability Factors for White Oak #1 Mine

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<th>Overburden Depth (ft)</th>
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**Pillar 1**
- 70
**Pillar 5**
- 70

**Loading Condition:**
- DEVELOPMENT

#### ARMPS STABILITY FACTOR

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**Pillar 1**
- 70
**Pillar 5**
- 70

**Loading Condition:**
- DEVELOPMENT

#### ARMPS STABILITY FACTOR

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**Pillar 1**
- 80
**Pillar 5**
- 80

**Loading Condition:**
- DEVELOPMENT

#### ARMPS STABILITY FACTOR

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### Stability Factors for White Oak #1 Mine

**Site 4**

<table>
<thead>
<tr>
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<tr>
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<td>Mining Height (ft) Bottom Total</td>
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</tr>
<tr>
<td></td>
<td>In-situ Coal Strength (psi)</td>
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</tr>
<tr>
<td></td>
<td>Entry Width (ft)</td>
<td>18</td>
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| Overburden Depth (ft) | 0 |
| Crosscut Angle (deg) | 0 |
| Crosscut Spacing (ft) | 0 |

<table>
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<th>100</th>
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<tr>
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<td>Pillar 6</td>
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**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

Development Loading: 3.7
Bottom Coal Removal Loading: 2.46

**Site 5a**

<table>
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<td>Mining Height (ft) Bottom Total</td>
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<tr>
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<td>900</td>
</tr>
<tr>
<td></td>
<td>Entry Width (ft)</td>
<td>18</td>
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</table>

| Overburden Depth (ft) | 560 |
| Crosscut Angle (deg) | 90 |
| Crosscut Spacing (ft) | 105 |

<table>
<thead>
<tr>
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<th>Pillar 3</th>
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<th>Pillar 4</th>
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<tbody>
<tr>
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<td>Pillar 5</td>
<td></td>
<td>Pillar 6</td>
<td></td>
<td>Pillar 7</td>
<td></td>
<td>Pillar 8</td>
<td></td>
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**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

Development Loading: 2.76
Bottom Coal Removal Loading: 1.88

**Site 5b**

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<td>Mining Height (ft) Bottom Total</td>
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</tr>
<tr>
<td></td>
<td>In-situ Coal Strength (psi)</td>
<td>900</td>
</tr>
<tr>
<td></td>
<td>Entry Width (ft)</td>
<td>18</td>
</tr>
</tbody>
</table>

| Overburden Depth (ft) | 560 |
| Crosscut Angle (deg) | 90 |
| Crosscut Spacing (ft) | 60 |

<table>
<thead>
<tr>
<th>Entry Spacing (ft)</th>
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<th>60</th>
<th>Pillar 2</th>
<th>60</th>
<th>Pillar 3</th>
<th>60</th>
<th>Pillar 4</th>
<th>60</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Pillar 5</td>
<td></td>
<td>Pillar 6</td>
<td></td>
<td>Pillar 7</td>
<td></td>
<td>Pillar 8</td>
<td></td>
</tr>
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**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

Development Loading: 1.62
Bottom Coal Removal Loading: 1.15
Stability Factors for White Oak #1 Mine

Name of Mine: Site 5c
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18
Overburden Depth (ft) 410
Crosscut Angle (deg) 90
Crosscut Spacing (ft) 105
Entry Spacing (ft)
Pillar 1 70 Pillar 2 70 Pillar 3 Pillar 4
Pillar 5 Pillar 6 Pillar 7 Pillar 8

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 3.77
Bottom Coal Removal Loading: 2.57

Name of Mine: Site 6a
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18
Overburden Depth (ft) 465
Crosscut Angle (deg) 90
Crosscut Spacing (ft) 80
Entry Spacing (ft)
Pillar 1 80 Pillar 2 80 Pillar 3 80 Pillar 4 80
Pillar 5 80 Pillar 6 80 Pillar 7 Pillar 8

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 3.22
Bottom Coal Removal Loading: 2.2

Name of Mine: Site 6b
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18
Overburden Depth (ft) 465
Crosscut Angle (deg) 90
Crosscut Spacing (ft) 105
Entry Spacing (ft)
Pillar 1 80 Pillar 2 80 Pillar 3 80 Pillar 4 80
Pillar 5 80 Pillar 6 80 Pillar 7 Pillar 8

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 3.84
Bottom Coal Removal Loading: 2.59

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### Stability Factors for White Oak #2 Mine

<table>
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<tr>
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</tr>
<tr>
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</tr>
<tr>
<td>Entry Width (ft)</td>
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</tr>
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<table>
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</thead>
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<td>5</td>
<td>70 Pillar 6</td>
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</table>

**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

- Development Loading: 2.94
- Bottom Coal Removal Loading: 2.01

### Name of Mine: Site 2

| Mining Height (ft) Dev. | 9 |
| Mining Height (ft) Bottom Total | 15 |
| In-situ Coal Strength (psi) | 900 |
| Entry Width (ft) | 18 |
| Overburden Depth (ft) | 495 |
| Crosscut Angle (deg) | 75 |
| Crosscut Spacing (ft) | 70 |

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<tr>
<td>5</td>
<td>70 Pillar 6</td>
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**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

- Development Loading: 2.31
- Bottom Coal Removal Loading: 1.61

### Name of Mine: Site 3

| Mining Height (ft) Dev. | 9 |
| Mining Height (ft) Bottom Total | 15 |
| In-situ Coal Strength (psi) | 900 |
| Entry Width (ft) | 18 |
| Overburden Depth (ft) | 530 |
| Crosscut Angle (deg) | 90 |
| Crosscut Spacing (ft) | 60 |

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<tr>
<th>Pillar</th>
<th>Entry Spacing (ft)</th>
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</thead>
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<tr>
<td>1</td>
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<td>5</td>
<td>60 Pillar 6</td>
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<tr>
<td>9</td>
<td>60 Pillar 8</td>
</tr>
</tbody>
</table>

**Loading Condition:** DEVELOPMENT

**ARMPS STABILITY FACTOR**

- Development Loading: 1.72
- Bottom Coal Removal Loading: 1.22

---

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Stability Factors for White Oak #2 Mine

Name of Mine: Site 4
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18

Entry Spacing (ft)
Pillar 1 60 Pillar 2 60 Pillar 3 60 Pillar 4 60
Pillar 5 60 Pillar 6 60 Pillar 7 60 Pillar 8 60

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 2.19
Bottom Coal Removal Loading: 1.56

Name of Mine: Site 5
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18

Entry Spacing (ft)
Pillar 1 80 Pillar 2 80 Pillar 3 80 Pillar 4 80
Pillar 5 80 Pillar 6 Pillar 7 Pillar 8

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 3.61
Bottom Coal Removal Loading: 2.46

Name of Mine: Site 6
Mining Height (ft) Dev. 9
Mining Height (ft) Bottom Total 15
In-situ Coal Strength (psi) 900
Entry Width (ft) 18

Entry Spacing (ft)
Pillar 1 70 Pillar 2 70 Pillar 3 75 Pillar 4 75
Pillar 5 Pillar 6 Pillar 7 Pillar 8

Loading Condition: DEVELOPMENT

ARMPS STABILITY FACTOR
Development Loading: 1.99
Bottom Coal Removal Loading: 1.35

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### Stability Factors for White Oak #2 Mine

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<tr>
<td>In-situ Coal Strength (psi)</td>
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<td>Entry Spacing (ft)</td>
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</tr>
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<tr>
<td>Pillar 5</td>
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### Stability Factors for White Oak #2 Mine

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<td>Pillar 2</td>
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<td></td>
</tr>
<tr>
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**Loading Condition:**

**ARMPS STABILITY FACTOR**

- Development Loading: 2.14
- Bottom Coal Removal Loading: 1.46

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**Loading Condition:**

**ARMPS STABILITY FACTOR**

- Development Loading: 1.4
- Bottom Coal Removal Loading: 0.99
APPENDIX A

PERFORMANCE STANDARDS AND SUPPORTING DOCUMENTS
TO THE PERFORMANCE STANDARDS
7.10.3 Install and maintain measures to control unauthorized vehicle access to the ROW. These measures may include: signs; fences with locking gates; slash and timber barriers, pipe barriers, or selectively placed boulders; and conifers or other appropriate trees or shrubs planted across the ROW.

7.10.3.1 The measures will meet agency or landowner specifications.

7.11 Fire Prevention and Control

7.11.1 Proponent(s) will prepare and implement a fire prevention and control plan to be approved by the land management agencies. This plan will include required fire fighting equipment, training for construction personnel, assignment of a Fire Guard, and a fire fighter notification plan in the event a fire is accidentally started. The plan will include contingency plans for extreme fire hazard conditions and fire closures.

7.12 Hazardous Materials and Wastes

7.12.1 Proponent(s) will prepare and implement a material storage and spill control and clean-up plan that, at a minimum:

7.12.1.1 Identifies typical fuel, lubricants, and other materials stored or used in the project area and the location, quantity, and method of storage;

7.12.1.2 Describes the preventive and mitigative measures to avoid or minimize impacts of spills of fuel, lubricants, or other materials, especially within any municipal watershed area or within 100 feet of any waterbody or wetland;

7.12.1.3 Requires fueling and lubricating to be done in areas designated for such purposes and specifies measures to avoid or minimize spills when construction equipment (such as pontoon-mounted backhoes and pumps) will be refueled in or within 100 feet of any waterbody or wetland;

7.12.1.4 Identifies emergency notification procedures in the event of a spill;

7.12.1.5 Requires each construction crew to have sufficient supplies of absorbent and barrier materials on-hand to allow the rapid containment and recovery of any spills;

7.12.1.6 Includes procedures for collection and disposal of waste generated during spill cleanup or equipment maintenance;

7.12.1.7 Includes procedures regarding excavation and disposal of any soil or materials contaminated by a spill; and

7.12.1.8 Identifies names and telephone numbers of all state agencies and individuals that will be contacted in the event of a spill.

8 Special Construction Procedures

8.1 Geologic Hazards and Steep Slopes

8.1.1 Fault Crossings. Proponents shall conduct geotechnical studies at all active and potentially active fault crossings as necessary to determine the exact location of the fault or fault zone, and maximum ground displacement that could occur at the pipeline crossing. This information will be used to develop site specific design
measures to minimize the potential for pipeline failure during surface fault rupture. Design measures may include crossing the fault at right angles, special trench and backfill designs, and thicker wall pipe to accommodate the anticipated range of movement. Secondary measures may include block and/or check values. The geotechnical investigative and design report shall be submitted to applicable state and federal agencies for review and approval prior to pipeline construction in the fault crossing.

8.1.2 Unstable Slopes, Landslides, and Potentially Unstable Slope Crossings. The proponent shall conduct geotechnical investigations and provide design recommendations for installation, maintenance, and monitoring of the pipeline ROW in areas of known or suspected landslides, or potentially unstable slopes. Measures may include slope improvements, surface or subsurface drainage improvements, slope buttressing, or local rerouting of the pipeline to avoid unstable areas.

8.1.2.1 The proponents will then use these investigations and designs to develop specific slope stabilization plans to be included in the POD for implementation. The plans will include monitoring and will be subject to agency approval prior to construction.

8.1.2.2 Areas requiring specific plans include Cherry Creek, Gooseberry Creek, Haystack Ridge, Mt. Nebo loop, and Cedar Hill/Water Hollow drainage.

8.1.2.3 A steep slopes stability plan shall be prepared as part of the POD for construction and reclamation on severe slopes. Side-slope construction, such as proposed in the vicinity northwest of Windward Reservoir on the Uinta National Forest also will be addressed in the steep slopes plan.

8.1.3 Liquefaction Hazard Area. For areas identified as having a moderate to high potential for liquefaction, the proponents shall conduct geotechnical investigations to confirm the site-specific risk of liquefaction and provide geotechnical recommendations for design of all pipeline segments or aboveground facilities. The geotechnical investigative and design report shall be submitted to applicable state and federal agencies for review and approval prior to pipeline or facility site construction.

8.1.4 Earthquake-Induced Ground Shaking. All ancillary facilities shall be designed to accommodate earthquake loads specified in the Uniform Building Code, without operational damage.

8.1.5 Collapsible Soils. For areas with suspected collapsible soils, the proponents shall conduct geotechnical investigations to confirm the site-specific risk of local surface subsidence and provide geotechnical recommendations for design of all pipeline segments, or aboveground facilities, located in areas that may contain collapsible soils. The geotechnical investigative and design report shall be submitted to applicable state and federal agencies for review and approval prior to pipeline or facility site construction.

8.1.6 Coal Mine Subsidence. Where the pipeline or ancillary facilities will be located over areas that have been mined, a geotechnical investigation will be performed to define the potential risk of surface subsidence and provide recommendations for design and construction of the pipeline and facilities to mitigate potential damage to the pipeline or facilities from potential mine induced surface deformation. In order to minimize erosion of pipe backfill, recommendations may include special pipe backfill procedures to accommodate potential subsidence-related movement. In addition, any areas potentially subject to subsidence shall be monitored, using ground survey measurements collected on at least an annual basis for a minimum of five (5) years.
to record and report surface subsidence along the pipeline route. Where future mining is planned in the vicinity of the pipeline corridor, a geotechnical investigation shall be performed to determine if there is a risk of damage to the pipeline from mine-induced subsidence. If a risk is identified, segments of the pipeline(s) located over the potential area affected by subsidence shall be relocated to avoid potential subsidence related damage, prior to mining.

8.2 Air Quality

8.2.1 Non-attainment Areas. Control of fugitive dust is required by Utah Air Conservation Rules R307-309. Fugitive Emissions and Fugitive Dust. R307-309 applies to areas designated as non-attainment for particulates less than 10 microns in size (PM_{10}). Utah County, Salt Lake County, and the city of Ogden are designated as non-attainment for PM_{10}. Therefore, measures must be taken to minimize fugitive dust. An owner or operator of a project involving the clearing or leveling of land 0.25 acre or greater in size is required to submit a plan to the executive secretary for the control of fugitive dust.

8.2.2 Attainment Areas. Areas outside of Utah and Salt Lake counties and the city of Ogden are required to comply with R307-205, Fugitive Emissions and Fugitive Dust for attainment areas.

8.3 Water

8.3.1 No blasting shall occur within 500 feet of an identified well or spring. If blasting cannot be avoided, the well or spring yield shall be verified prior to and after construction. If well or spring yield has been reduced, the landowner will be compensated by the proponent(s) for this loss.

8.3.2 Developed Drinking Water Supplies

8.3.2.1 Pipeline construction activities will not take place within 100 feet of any drinking water supply intake, well, or spring without prior approval.

8.3.2.2 Provide written notification to authorities responsible for surface drinking water supplies at least 1 week before entering the watershed.

8.3.2.3 Prior to construction of open-cut water crossings the proponents shall provide a sediment control and monitoring plan that meets the requirements of the clean water and safe drinking water acts.

8.4 Hydrostatic Testing

8.4.1 Perform 100 percent radiographic inspection of all pipeline section welds or hydrotest the pipeline sections before installation under waterbodies or wetlands.

8.4.2 If pumps used for hydrostatic testing are within 100 feet of any waterbody or wetlands, address the operation and refueling of these pumps in the Spill Prevention, Control, and Countermeasures (SPCC) Plan.

8.4.3 The proponents shall notify the owners of water used in hydrostatic testing and receive approval to use such water before appropriating.

8.4.4 Intake Source and Rate

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APPENDIX B

OTHER ISSUES EVALUATED
B.1 Geologic Hazards

The proposed pipelines would cross several types of geologic hazards that would require special engineering design measures to prevent pipeline and ancillary facility damage. These are frequently encountered hazards for which both standard (i.e., Uniform Building Code) and site-specific design solutions are available. The following sections outline the hazards, and the design and monitoring measures that the proponents would perform to protect the pipeline and ancillary facilities.

<table>
<thead>
<tr>
<th>Geologic Hazards Crossed</th>
<th>Questar</th>
<th>Williams</th>
<th>Kern River</th>
</tr>
</thead>
<tbody>
<tr>
<td>Active Fault Zones Crossed (#)</td>
<td>1</td>
<td>3</td>
<td>1</td>
</tr>
<tr>
<td>Soil Liquefaction Risk (Miles)</td>
<td>11.4</td>
<td>20</td>
<td>9.0</td>
</tr>
<tr>
<td>Collapsible Soil Risk (Miles)</td>
<td>8.7</td>
<td>4.7</td>
<td>9.6</td>
</tr>
<tr>
<td>Current Coal Mine Subsidence Risk (Miles)</td>
<td>4.0</td>
<td>4.0</td>
<td>4.0</td>
</tr>
<tr>
<td>Future Potential Coal Mine Subsidence Risk (Miles)</td>
<td>8.0</td>
<td>8.0</td>
<td>8.0</td>
</tr>
</tbody>
</table>

Active Fault Zones and Earthquakes. Large earthquake events (> Magnitude 6) could result in surface displacement (or fault rupture) along active fault traces. All three projects would cross the active Wasatch Fault, which extends along the Wasatch Front between Payson and Nephi. In addition, the Williams pipeline would cross the West Valley and Salt Lake City Segment of the Wasatch Fault. The Wasatch Fault is capable of vertical displacements of 5 to 15 feet (Hecker 1993, 1995; Harty et al. 1997).

Proponents would conduct geotechnical studies at all active and potentially active fault crossings as necessary to determine the exact location of the fault or fault zone, and maximum ground displacement that could occur at the pipeline crossing. Proponents would develop site-specific design measures to minimize the potential for pipeline failure during surface fault movement. Design measures may include crossing the fault at right angles, special trench and backfill designs, and thicker wall pipe to accommodate the anticipated range of movement.

After these design measures are implemented, it is highly unlikely that an earthquake event and potential surface displacement would cause damage to the pipeline. O’Rourke and Palmer (1996) evaluated pipeline performance during 11 major southern California earthquakes (Richter magnitude 5.9 to 7.7) over a 61-year period and concluded that post World War II electric arc welded transmission lines in good repair have not experienced leaks or breaks. USDOT statistics indicate that less than 4 percent of all pipeline releases nationwide were caused by geologic hazard events, including earthquakes, floods, and landslides.

Strong ground shaking from earthquakes can cause damage to aboveground facilities. All aboveground facilities would be designed to the appropriate seismic codes for area where they would be situated. Based on this compliance with these building codes, earthquake damage to these facilities is not expected.

Soil Liquefaction. Liquefaction is the process by which water-saturated sediments lose strength and may fail during strong earthquake induced ground shaking. Liquefaction can result in loss of ground bearing capacity or lateral spreading, both of which could damage pipelines and ancillary facilities. Soil liquefaction hazards are associated with unconsolidated alluvial soils with a high water table. The Juab and Goshen Valleys, and
the West Salt Valley are the primary areas where potential soil liquefaction areas would be crossed by the projects (Hecker, Harty, and Christenson 1988; Anderson et al. 1986, 1994).

For areas identified as having a moderate to high potential for liquefaction, the proponents would conduct geotechnical investigations to confirm the site-specific risk of liquefaction, and provide geotechnical recommendations for design of all pipeline segments, or above ground facilities, located in areas subject to liquefaction.

It is not possible to accurately predict or to design for all possible lateral spreading or rapid earth flow type failures associated with liquefaction. Although a large earthquake capable of causing liquefaction is unlikely to occur during the life of the project, this type of event could trigger lateral spreading or rapid earthflows which could damage the pipeline and aboveground facilities. The most feasible mitigation is to decrease the spacing between block valves in higher soil liquefaction risk areas that are associated with residential and commercial areas to decrease the likelihood and magnitude of a pipeline release. These areas are located between the community of Kearns, Utah, and the northern terminus of the Williams Pipeline.

Collapsible Soils. Hydrocompaction is a process that occurs when certain open-textured soils (called collapsible soils) lose strength through consolidation – typically when these soils become saturated for the first time. This type of subsidence occurs rapidly and commonly results in surface cracks and displacements of several feet or more. Approximately 4.7 miles between Indianola and Nephi are prone to collapse if wet (Goldber 1999). The climate of this route segment is dry, and it is highly unlikely that areas of excavation would be exposed to sufficient precipitation or runoff during the brief period of summer construction to cause soil collapse. After implementation of any site-specific measures to prevent long-term soil collapse (directing surface drainage away from the trench), the risk of damage to the installed pipeline from soil collapse would be very unlikely.

Existing and Future Coal Mine Subsidence. Surface subsidence in areas overlying former underground coal mines could damage pipelines located over mined areas by vertical ground surface displacement over short distances. Approximately 4.2 miles along Segment B west of Mud Creek on the Wasatch Plateau was recently mined by the Skyline Mine. This mining was completed between 1990 and 1998, and included removing portions of two coal seams beneath the pipeline alignment. Up to 18 feet of surface subsidence has occurred along the pipeline corridor; the area of subsidence is estimated to extend up to approximately 400 feet beyond the limits of mining. Subsidence monitoring along this segment indicates that most subsidence occurs within 2 years after mining.

In 2000, Questar relocated its Mainline 41 natural gas pipeline across the Skyline Mine subsidence zone that coincides with the currently proposed alignment by all three proponents. Subsidence monitoring is being conducted for the existing pipeline along this alignment, and would be continued if the QWK projects were constructed. If evidence of subsidence were identified, site-specific stabilization measures would be implemented (backfilling, supports) to ensure continued safe operation of the pipelines.
The proposed QWK projects would cross approximately 8 miles of federal underground coal leases. In the event these leases were mined in the future, the pipelines would likely be relocated to an unmined area until mining subsidence stabilized, and the pipelines could be located back in the original ROW.

Conclusion. Implementation of site-specific geotechnical studies and design measures for earthquake ground shaking, active fault crossings, soil liquefaction, and collapsible soils would result in very low likelihood that damage to pipelines or ancillary facilities would occur over the 50-year life of this project. Although the likelihood of a triggering earthquake event is low, the non-site specific nature of soil liquefaction could require additional pipeline block valves to be placed in residential and commercial areas near Salt Lake City (between Kearns and Bountiful) that are underlain by soils susceptible to liquefaction. Continued monitoring of coal mine subsidence would ensure that new subsidence is immediately identified and measures taken to stabilize the pipeline. If future underground coal mining would cause surface subsidence along the pipeline ROW, the pipelines would be moved to an unmined area.

B.2 Paleontological Resources

Construction of the proposed pipelines on certain geologic formations could disturb or destroy paleontological resources, as a result of direct disturbance, exposure to weathering, or vandalism. Paleontological surveys conducted along the proposed pipeline route identified geological formations and Quaternary informal units, including formations or members classified as areas known to contain significant fossils, particularly vertebrates (BLM Condition Class 1 or 2).

<table>
<thead>
<tr>
<th>Paleontological Resources Crossed</th>
<th>Questar</th>
<th>Williams</th>
<th>Kern River</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geological formations (#)</td>
<td>10</td>
<td>35</td>
<td>13</td>
</tr>
<tr>
<td>Quaternary informal units (#)</td>
<td>6</td>
<td>18</td>
<td>7</td>
</tr>
<tr>
<td>Significant fossil formations or members (#)</td>
<td>8</td>
<td>41</td>
<td>12</td>
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Williams proposed route crosses several formations from which important dinosaur fossils have been excavated. In addition, late Cretaceous and early Tertiary formations contain significant fossil assemblages that would be crossed by the Questar, Williams, and Kern River project ROWs.

No estimates can be made as to the number of fossils that would be encountered or excavated during pipeline construction activities. Prior to construction, paleontological surveys would be conducted for all significant formations except in very steep areas, well vegetated areas, and areas where access was denied. Formations with moderate or unknown potential would be spot-checked. Fossils encountered during construction and identified as important or significant would be collected and deposited in appropriate curation facilities.

Once the pipeline has been constructed, further work on the ROW should not disturb paleontological resources. However, if fresh bedrock is excavated, paleontological resources could be destroyed in units that meet BLM Conditions 1 or 2.

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DIV OF OIL, GAS & MINING
GEOLOGICAL AND GEOTECHNICAL RESPONSES
QUESTAR, WILLIAMS, KERN RIVER
EIS PERFORMANCE STANDARDS
PROPOSED QUESTAR MAINLINE 104
SKYLINE MINE CROSSING
CARBON AND EMERY COUNTIES, UTAH

For: QUESTAR

Kleinfelder File No. 35-8353-04.005
May 21, 2001

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DIV OF OIL GAS & MINING

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A Report Prepared for:

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GEOLOGICAL AND GEOTECHNICAL RESPONSES
QUESTAR, WILLIAMS, KERN RIVER EIS PERFORMANCE STANDARDS
PROPOSED QUESTAR MAINLINE 104
SKYLINE MINE CROSSING
CARBON AND EMERY COUNTIES, UTAH

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May 21, 2001
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## APPENDICES

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2. South Mine Panel Crossing Plan
3. North Mine Panel Crossing Plan
4. Subsidence Plot

### B

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2. Chart B Cumulative Subsidence Along the Southern Portion of Pipeline
3. Chart C Radius of Curvature Induced in Old Pipeline by Subsidence
7. Chart G Surveyed Subsidence Over 4 Left Panel, June 16, 2000
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9. Subsidence Certificate

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CDI Engineering Group Inc, Analysis Letter

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Authorization to Use

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1. INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report has been prepared to outline our review of data prepared by Canyon Fuels LLC for the assessment of surficial subsidence of formerly mined areas over the Skyline Mines in Carbon and Emery Counties, Utah, and to provide recommendations for the proposed Questar ML 104 pipeline construction over the formerly mined areas. The area of concern consists of an approximately four-mile portion of right-of-way (ROW) that is adjacent to the existing Questar ML41 Pipeline. It is also our understanding that Williams Petroleum intends to construct a fuel product pipeline adjacent to the Questar Row.

A previous study addressing the replacement of ML41 over the mined area was prepared by Kleinfelder (File No. 35-8353-01.004) dated July 24, 2000. The recommendations and conclusions from this study are based on the finalization of the previous study. The ML 41 pipeline was rerouted over the mined areas in the Fall of 2000, and has performed adequately in this area since the reroute.

The area of concern is shown on Plate A-1, Site Vicinity Map, and specific mine panel plans are presented on Plate A-2, South Mine Panel Crossing Plan, and Plate A-3, North Mine Panel Crossing Plan.

Canyon fuels and their consultant have monitored surficial subsidence over the mined areas since 1990. Based on their observations, they have concluded that future subsidence on the formerly mined areas will be minimal and not induce distress to buried pipelines on the surface. Our previous July 24, 2000 review and evaluation was conducted because Questar was concerned that subsidence related deformation on the surface above the formerly mined areas may damage the proposed pipeline alignment and disrupt pipeline service.
Our scope of study has consisted of a review of subsidence data tables, subsidence plots, and mining charts prepared by Canyon Fuels LLC, reviews of historic seismic activity data maintained by the University of Utah Seismograph Stations (University of Utah Seismograph Stations, 2000), and reviews of studies from similar mining operations near the Skyline Mine site (Dunrud, 1976; Allgaier, 1982; Allgaier, 1988; Dyni, 1991; Jones, 1994). In addition to the scope of studies listed above, Dr. Greg Schlenker of our office attended an April 18, 2000 presentation on subsidence monitoring at the Salt Lake City Questar Office. The presentation was conducted by Messrs. Mark Bunnell, and Doug Johnson, P.E. of Canyon Fuels, LLC, and Dr. Yoginder Chugh of Southern Illinois University, Carbondale. Dr. Chugh is a consultant for Canyon Fuels, LLC.

The objectives and scope of this study were developed during discussions with Mr. Ted Smith, Design Engineer of Questar Regulated Services Company. The objectives of this study were to provide assistance regarding concerns related to the short- and long-term soil stability over areas mined in the past by Canyon Fuels, LLC. Our scope of work involved providing technical assistance and the review of reports and data provided by Canyon Fuels, LLC and to provide recommendations for pipeline construction over the formerly mined area. In order to provide the appropriate assistance, we developed the following scope of work:

1. An initial office program including a review of existing geological literature, a review of mine subsidence monitoring practices, subsidence data and project design and previous reports.

2. Preparation of this Summary Report.

The scope of services outlined above has been performed in accordance with the performance standards in Appendix A-1, Section 8.1 of the Questar, Williams, Kern River (QWK) Draft Environmental Impact Statement (EIS) that is presently under review. The QWK EIS Performance Standards specify the following actions:
Where the pipeline or ancillary facilities will be located over areas that have been mined, a geotechnical investigation will be performed to define the potential risk of surface subsidence, and provide recommendations for design and construction of the pipeline and facilities to mitigate potential damage to the pipeline or facilities from potential mine induced surface deformation. In order to minimize erosion of pipe backfill, recommendations may include special pipe backfill procedures to accommodate potential subsidence-related movement. In addition, any areas potentially subject to subsidence would be monitored, using ground survey measurements collected on at least an annual basis for a minimum of 5 years, to record and report surface subsidence along the pipeline route. Where future mining is planned in the vicinity of the pipeline corridor, a geotechnical investigation will be performed to determine if there is a risk of damage to the pipeline from mine-induced subsidence. If a risk is identified, segments of the pipeline(s) located over the potential area affected by subsidence will be relocated to avoid potential subsidence related damage, prior to mining.

Authorization to perform this study was provided by Mr. Edward G. Ortega Contract Administrator of Questar who signed Questar Work Authorization No. 09A-4086 dated May 22, 2000. During the course of this study, many of the conclusions and recommendations summarized herein were transmitted verbally to the individuals listed above.

1.2 PROJECT DESCRIPTION

The proposed ML 104 pipeline is to be a 24-inch diameter natural gas pipeline that will extend from the Price Coal Bed Methane Project west of Price to Payson Gate near Payson and continues to Elberta, Utah. As proposed, ML 104 is to be constructed parallel to the existing Questar ML 41 ROW, except where environmental, hazard or space considerations have required variations in the proposed ROW.
2. GENERAL GEOLOGY

2.1 PHYSIOGRAPHY

The proposed ML104 alignment at the Skyline Mine is located in the Middle Rocky Mountain Physiographic Province. The Middle Rocky Mountain province covers parts of Utah, Colorado Wyoming, Idaho and Montana. The geology of the province is an assemblage of sedimentary, igneous, and metamorphic rocks that have been folded, faulted and uplifted. The province is characterized by mountainous terrain with deep canyons, with semi-arid to mesic climatic conditions (Hunt, 1967).

2.2 SURFICIAL GEOLOGY

The surficial geology of the proposed ML104 ROW at Skyline Mine is dominated by nearly horizontal to moderately sloping upper Cretaceous sedimentary rocks ranging from shales and coal beds to medium-grained grained sandstones (Witkind and Weiss, 1991). Quaternary slope-wash colluvium deposits cover most steep sloping areas, and residual soils are found on the more level surfaces. In the vicinity of the mine crossing, the mined coal beds are located roughly 900 and 1000 feet below the surface. No active Holocene-aged faulting has been identified in the vicinity of the realignment (Hecker, 1993).
3. METHODS OF STUDY

3.1 INVESTIGATION

Our investigation consisted of a review of previous reports, presentations and data provided by Canyon Fuels, LLC, a reconnaissance of the proposed alignment to observe subsidence affects on the surface, and the development design and installation procedures to minimize potential pipe damage from future subsidence along the reroute.

3.1.1 Data Review

Subsidence was monitored by Canyon Fuels, LLC using aerial photography, survey and photogrametric methods to calculate changes in surface elevations along the length of the proposed alignment. The photogrametric monitoring was conducted between 1990 and 1999. A plot illustrating the photogrametric monitored subsidence is shown on Plate A-4, Subsidence Plot, and corresponding data along the monitor points are tabulated on Charts A, B and C, which are included in Appendix B.

In addition to the photogrametric monitoring, a ground survey was conducted on the northwest corner of the 4L panel shown on Plate A-2 from 1998 to 1999. The subsidence in this area was monitored using ground survey methods because the 4L panel is the most recently mined area that underlies the proposed alignment. The 1998 to 1999 ground survey data are tabulated on Charts D, E in Appendix B. Charts F, G and H in Appendix B present subsequent ground survey measurements made on May 23, June 16, June 29 and July 9, 2000 respectively.

During the April 18, 2000 presentation Dr. Chugh outlined the following conditions and characteristics of subsidence along the pipeline alignment area based on their subsidence monitoring conducted between 1990 and November of 1999:
1. Two estimates of the angle of draw were made from measured subsidence. These values are 13.5° to 17.0°, respectively.

2. Maximum subsidence associated with the mining of the OU seam (Upper O'Conner seam) varied between 6 to 8 feet.

3. Maximum subsidence occurs in two years or less.

4. After the development of maximum subsidence, typically less than 6 inches of additional subsidence seems to occur.

5. Maximum subsidence associated with the mining both seams (Upper O'Connor and Lower O'Conner seams) varies between 10 to 17 feet.

6. Most subsidence associated with the mining of the lower seam appears to occur in about one year or less.

7. After development of the maximum subsidence associated with the mining of the LOB seam (lower seam), no more than one foot of additional subsidence appears to occur in subsequent years.

8. Almost all of the subsidence movements associated with mining one and/or two seams is complete in two years or less.


Based upon the findings outlined above and the results of the May 23, 2000 subsidence measurements, Dr. Chugh prepared a “Subsidence Certificate” that includes as following excerpts:

...As an independent expert in the field of mining-induced subsidence, and based upon the subsidence data gathered by means of aerial photography and survey level line installed and monitored by Canyon Fuel on the ground in the New ROW, I certify to Questar Pipeline that Canyon Fuel's mining-induced subsidence of the New ROW appears to have stabilized and the New ROW should be structurally safe for the construction and operation of the New Pipeline...
...Notwithstanding issuance of the Subsidence Certificate, Canyon Fuel shall remain responsible for any structural damage to the New Pipeline and associated facilities caused by Canyon Fuel's mining-induced subsidence as determined by Questar Pipeline based on its best engineering judgement. 

3.1.2 Subsidence Area Reconnaissance

On June 7, 2000 Dr. Greg Schlenker of our office visited the proposed pipeline alignment area and made observations along the portions of the ROW that overlie the mined areas. The reconnaissance was conducted to evaluate general surface conditions over the mined areas that may affect pipeline stability or operation.

During the reconnaissance some areas of surface cracking, a locally small sinkhole and a shallow slope failure were observed along the ground survey area on the northwest side of the 4L panel location. The cracking was observed at approximately Station 13.5+ and Station 14.5+ on the ground survey area. The cracking appeared to extend in roughly a north-northwest direction, roughly parallel to the western edge of the 4L panel. The Station 13.5+ crack was observed to be as much as 3 feet in depth, a few inches wide, and was found to extend approximately 85 feet to the north of the proposed alignment. The Station 14.5+ crack was observed to be as much as 2 feet in depth, a foot or more in width, and was found to extend approximately 105 feet across and to the north of the proposed alignment. No more than three inches of vertical displacement was observed across the two cracked areas. In addition to the cracks, at approximately Station 17.5+ on the ground survey area, an approximately ½-foot by 1-foot wide by ½-foot deep sinkhole was observed adjacent to the alignment, and a shallow slope failure was observed on the north side of the alignment at approximately Station 28.0+.

Shallow and/or surficial bedrock was observed between Station 8+ and 10+ along the ground survey area, and between Station 20,000+ and 21,275+ along the aerial survey on the southern end of the Mine Crossing. The length of the Mine Crossing, south of the ground survey area which overlie Panels 3L and 4L, and portions of the Mine Crossing north of the ground survey...
area were also observed evaluate general surface conditions that may affect pipeline stability or operation. The 3L Panel was last mined in 1997, and the areas to the north that underlie the proposed ROW were mined between 1986 and 1996. Other than the observances of shallow and/or surficial bedrock, no issues of concern pertinent to pipeline stability or operation were observed on the proposed ROW south or north of the ground survey area.

3.1.3 Pipeline Design and Installation.

Design and installation procedures were developed to minimize the effects of potential future subsidence, or subsidence related deformation on the proposed pipeline. These procedures are based upon the data provided by Canyon Fuels LLC, observations made in the field and design strategies typically used for crossing earthquake faults (McDonough et al., 1995: Keaton, 1995). In addition to providing recommendations to mitigate subsidence deformation, procedures to mitigate piping and erosion for the Mine Crossing, have also been developed.
4. DISCUSSIONS AND RECOMMENDATIONS

4.1 GENERAL CONCLUSIONS

Based upon the data and information provided by Canyon Fuels LLC, our observations made in the field and the implementation of appropriate design and installation strategies, we believe that the proposed pipeline may be constructed as proposed across the Mine Crossing with minimal effects from subsidence related movements. Although monitoring data indicates subsidence has stabilized, we believe that prudent measures should be taken to minimize the affects of future subsidence that may occur along the proposed alignment.

The conclusion made by Dr. Chugh that 90 percent or more of expected subsidence will occur within the first two years following mining appears to be supported by most, but not all, subsidence measurements. Furthermore, subsidence at similar mining operations in the area (Allgaier, 1988; Dyni, 1991) experienced continued subsidence lasting longer than two years.

The Pipeline Subsidence Profile Plot between the years of 1996 and 1997 show 1.0 to 2.5 feet of subsidence at approximately 14,500 feet to 16,250 south. This section overlies panel 5L Upper and panel 4L Lower. These areas were not mined between 1992 and 1998. The occurrence of this unexplained subsidence suggests that possible seismic activity in the vicinity of the mine may have influenced additional or renewed occurrences of subsidence over areas thought to be stable. However, the largest earthquake in the past 10 years occurred approximately 18 miles east of the site on March 7, 2000. Based on the May and June records on Charts F and G in Appendix B, no additional or renewed subsidence was recorded on the ground survey in response to this 4.2 magnitude earthquake.

Seismic influences on the rate of subsidence, and possible additional or renewed, subsidence are not well understood. The area surrounding the Skyline Mine is subject to periodic weak to moderately strong seismic earthquake ground motions attributable to near-by mine activity and seismogenic sources (University of Utah Seismograph Stations, 2000). Future seismic activity
may influence subsidence over the Mine Crossing, and the newly constructed pipeline should be carefully monitored following strong ground motion generating events

4.1.1 Pipeline Design and Installation.

Design and installation procedures were developed to minimize the effects of potential future subsidence, or subsidence related deformation on the pipeline alignment. These procedures are based upon the data provided by Canyon Fuels LLC, observations made in the field and design strategies typically used for crossing earthquake faults (McDonough et al., 1995; Keaton, 1995). In addition to the subsidence deformation, procedures to mitigate piping and erosion for the rerouted pipeline alignment have also been developed.

The Pipeline Subsidence Profile Plot between the years of 1996 and 1997 show 1.0 to 2.5 feet of subsidence at approximately Stations 14,500 feet to 16,250 south. This section overlies 5L Upper O'Conner panel and 4L Lower O'Conner panel. These areas were not mined between 1992 and 1998. The unexplained subsidence may have occurred in response to seismic loading in the vicinity of the site, or may have been a delayed response to the undermining. In either case, we would consider this amount of movement to be maximum expected unexplained subsidence for the recently mined areas. Based on the unexplained movement we recommend that the pipe over the 4-Left Panel be designed to withstand the following forces:

1. Bending forces-

The steepest transition for observed unexplained movement was 2.5 feet vertical over 260 feet horizontal.

2. Tension forces-

The longest segment for observed unexplained tensional movement was 2.5 feet vertical over 1400 feet horizontal.
3. Point Displacements-

Cracks observed on surface at Station 13.5 and 14.5 ground survey indicate 0 to 3 inches of vertical displacement across crack surfaces.

Based on these forces, Mr. Peter McDonough, Senior Engineer for CDI Engineering Group, Inc. calculated probable stresses on the pipe. A copy of the CDI Analysis Letter is included in Appendix C of this report. Mr. McDonough's calculations are summarized as follows:

Calculations at this location were based on an expected vertical subsidence of 2.5 feet over a distance of 260 horizontal feet. Two methods of calculations were made to determine longitudinal displacement stress. The first was based on the "M. O'Rourke method" (ref. 1, p. 132). The second was based on Questar Pipeline Company standard 1-1-4. The estimate of unrestrained pipeline length was based on the "Newmark-Hall procedure" (ref. 2, pp. 185 -191). The total pipeline stress, for this and all subsequent conditions and locations described in this letter, was based on the Maximum Distortion Energy Theory, as presented in Questar Pipeline Company standard 1-1-4.

I estimate that total longitudinal and bending stress due to subsidence will be between 29,000 and 32,000 psi and the total pipeline stress will be approximately 43,300 psi. Therefore, only elastic deformation is anticipated. Approximately 150 feet of straight pipe is required to totally relieve the calculated axial stress.

Recommendation

A mixed grain dense sand, or lighter material should be placed above the pipeline within the subsidence area and for a distance of 150 feet beyond the expected edge of subsidence.

In conjunction with the forces outlined above, we recommend that special pipe backfill procedures be implemented to accommodate potential subsidence-related movement along the pipe. When placed, the pipe should be centered in the ditch (not laid on the bottom) and surrounded with clean sand backfill. The pipe should be placed in as shallow of a ditch as possible and the sand backfill should extend as close to the ground surface as possible and still satisfy land reclamation requirements. The construction ditch side slopes should be cut at 45 degrees at the points of intersection, and the 45-degree cuts should extend for at least 50 feet.
beyond the points of intersection. The clean sand backfill will allow for subgrade movement to occur without imposing excessive forces on the buried pipe.

Where known surface cracks are located, and over shallow bedrock areas we recommend that construction ditch underlain by a high strength geotextile fabric such as Mirifi 180 N, and have suitable backfill embedment for the placement of the pipe. The fabric should not be folded over the pipe, but rather the ends of the fabric should be pinned to the outsides of the ditch and buried under the reclamation topsoil layer. The geotextile fabric will allow for the movement of water through the pipe backfill and minimize the piping and erosion of the pipe backfill soils.

Monitoring of the pipeline alignment after construction will need to be conducted for at least five years. As minimum, we recommend that the alignment area be monitored with ground survey measurements annually. We also recommend that measurements be made following seismic events 4.0 Richter magnitude or stronger within 20 miles of the alignment area.
5. CLOSURE

5.1 LIMITATIONS

The recommendations contained in this report are based on our field observations, pipeline safety issues, and our understanding of the proposed construction. It is possible that variations in the soil and groundwater conditions could exist between the points observed. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this crossing site which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed construction changes from that described in this report, our firm should also be notified.

This report was prepared in accordance with the generally accepted standard of practice at the time the report was written. No warranty, express or implied, is made.

This report has been prepared for the sole use of the Client. It is the Client's responsibility to see that all parties to the project including the Client's Designer, Contractor, Subcontractors, etc. are made aware of the contents of this report in its entirety. The use of information contained in this report for bidding purposes should be done at contractor's option and risk. Any party other than the client who would like to use this report shall notify Kleinfelder of such intended use by executing the "Application for Authorization to Use" contained in Appendix D. Based on the intended use of the report, Kleinfelder may require additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.
5.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that appropriate additional studies are conducted and an adequate program of tests and observations will be made during the construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.
6. REFERENCES


University of Utah Seismograph Stations, 2000, University of Utah web page, http://www.seis.utah.edu/

APPENDIX 527

Belina Haul Road
Reclamation Plan
Morrison-Knudsen
(1987)
Valley Camp of Utah

Belina Haul Road
Reclamation Plan

January 1987
BELINA HAUL ROAD RECLAMATION PLAN

Prepared For

Valley Camp of Utah, Inc.
Helper, Utah

Prepared By

Morrison-Knudsen Engineers, Inc.
Boise, Idaho

January, 1987
ENGINEERING CERTIFICATION

I, J. Carl Dille, have reviewed Section 2 and 3 and the following Tables, Figures and Exhibits in the Reclaimed Haul Road Reclamation Plan.

Figure 2-1 thru 2-8  Typical Cross Section of Unstable Fill
Figure 2-9            Slope Stability Analysis of Current
Figure 2-10           Slope Stability Analysis of Reclaimed Haul Road
Figure 3-1            Small Drainage Hydraulic Data
Figure 3-2            Bowl Crossing Hydraulic Data
Figure 3-3             Eccles Creek Channel Design
Figure 3-4             Typical Waterbar Detail

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Table 3.2               Watershed Size and Flow Characteristics
Table 3.3               Riprap Design
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2.2 Natural Talus Evaluation  
2.3 Critical Fill Areas  
2.4 Methods and Results of the Slope Stability Analysis  

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3.2 Design Flow  
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SECTION 1.0 - INTRODUCTION

1.1 General

The following reclamation plan is for Valley Camp of Utah's Belina Haul Road which supports their coal mining operation located on Whiskey Creek in Carbon County, Central Utah.

The haul road is constructed on a cut/fill bench having a total road width of thirty-four feet with very steep natural slopes above and below the road. These two facts create several unique problems when considering reclamation of the road.

This reclamation plan addresses the removal of the road surface materials and associated structures and the recontouring of the area to facilitate the return of the disturbed lands to its pre-mining land use of limited rangeland and wildlife habitat.

1.2 Objectives

The objectives of this reclamation plan are to eliminate the permanent road surface and support structures and return the disturbed land to a condition capable of supporting the pre-mine land use of limited rangeland and wildlife habitat. These objectives will be obtained by recontouring the road surface to re-establish a drainage pattern comparable to pre-mining conditions; by replacing the soil medium and re-establishing an effective permanent vegetation cover.

The affective area consists of a strip of land approximately 100 feet wide and 1.5 miles long. Although the right-of-way (ROW) is approximately 100 feet wide, this reclamation plan will only address the road surface (34 feet wide); the outslope areas having

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questionable slope stability and the area to be re-disturbed to re-establish the natural drainages. Figure 1-1 is a typical cross section illustrating the construction of the Belina Haul Road. Approximately ten acres will require recontouring and/or reclamation activities. The majority of the road outslope areas are considered stable as final reclamation, as discussed in Section 2.0, and therefore will not be disturbed. Determination of slope stability is discussed in detail in Section 2.0. The results of a limited geotechnical evaluation concerning the road outslopes and drainage fills are shown in Table 2.1. Only the potentially unstable slopes and their corresponding station location are shown on this table. These station locations were determined from plan and profile sheets showing the general road location and grade. Survey station locations were shown on the map beginning at the mine portal going towards the Eccles Canyon intersection and ending at Station 83+52.

Reclamation activities will be conducted in a manner that will minimize the potential adverse impacts to the air, water, vegetation, wildlife, and general aesthetics of the area. This proposed reclamation plan will establish a permanent, diversified vegetation cover capable of self-regeneration and soil stabilization that will support the post-mining land use of limited rangeland and wildlife habitat.
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
TYPICAL HAUL ROAD CROSS SECTION
Figure 1-1

NATURAL SLOPE
ROAD CUT
ASPHALT
GUARDRAIL
CONCRETE LINED DRAINAGE CHANNEL
ROAD BASE
OUTSLOPE AREA
2.1 General

The geotechnical investigation for the Belina Haul Road was performed in three phases. Phase I was a site evaluation of the natural talus in the local region. Phase II identified the areas considered to be critical. Phase III was the slope stability analysis for typical sections of the road before and after reclamation. Conclusions were then based upon all three phases.

Soils in this region are considered young and primarily consist of weathered rock high in quartz. The Unified Soil Classification System would consider this material as SP since it is gravelly sand which is poorly graded and has very few fines. This type of soil has essentially no cohesion; therefore, it is considered a pure phi (Ø) soil which will control the type of slope failures and geometry of the natural talus slopes.

2.2 Natural Talus Evaluation

Natural talus slopes in the haul road area widely vary between 30 percent to over 100 percent. By considering the friction angles (Ø) of the soil to be constant across the region, the depth to bedrock will control the natural talus slope stability. Shallow soils are stable at greater slopes than deep soils. The length of run also plays a major role in the stability of the slopes. The natural talus in the region was self-stabilizing due to small failures creating a terracing effect across the hillside. The stabilizing of the natural talus slopes is still occurring and numerous natural slope failures
may be seen around the vicinity of the Belina Haul Road. The friction angle of the talus was derived from the geometry of a recent natural slope failure. This failure analysis produced a friction angle equal to 31°. This value is very typical for SP classified soils.

2.3 Critical Fill Areas

Critical fill areas are defined for this discussion as areas which have localized evidence of recent slope failures, slopes which exceed the friction angle of the soil, or slopes that have similar characteristics of recent failures in the region, such as deep soil horizons. These critical areas are listed in Table 2.1:

### TABLE 2.1

**POTENTIAL UNSTABLE SLOPES***

<table>
<thead>
<tr>
<th>AREA</th>
<th>Station Location</th>
<th>Linear Distance</th>
<th>Slope Pre-Construction</th>
<th>Slope Post-Construction</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>21+10 to 22+70</td>
<td>160'</td>
<td>63%</td>
<td>70%</td>
</tr>
<tr>
<td>2</td>
<td>24+06 to 29+34</td>
<td>520'</td>
<td>63%</td>
<td>72%</td>
</tr>
<tr>
<td>3</td>
<td>30+40 to 32+00</td>
<td>160'</td>
<td>55%</td>
<td>68%</td>
</tr>
<tr>
<td>4</td>
<td>37+18 to 44+00</td>
<td>682'</td>
<td>63%</td>
<td>70%</td>
</tr>
<tr>
<td>5</td>
<td>51+17 to 52+75</td>
<td>158'</td>
<td>65%</td>
<td>75%</td>
</tr>
<tr>
<td>6</td>
<td>61+00 to 64+12</td>
<td>312'</td>
<td>70%</td>
<td>80%</td>
</tr>
<tr>
<td>7</td>
<td>73+00 to 75+60</td>
<td>260'</td>
<td>84%</td>
<td>128%</td>
</tr>
<tr>
<td>8</td>
<td>77+18 to 82+46</td>
<td>528'</td>
<td>84%</td>
<td>143%</td>
</tr>
</tbody>
</table>

TOTAL = 2,780 Feet

*Typical Geometries for each one of these reaches are illustrated in Figures 2.1 to 2.8.
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
FINAL CONTOURED SURFACE

Figure 2-1
(NOT TO SCALE)

63% SLOPE (NATURAL)

173% SLOPE (ROAD CUT)

18% (5.5H:1V)

FILL REMOVED

43% SLOPE (2.3H:1V)

63% SLOPE (NATURAL)

70% SLOPE

TYPICAL CROSS SECTION–UNSTABLE FILL
STA 21 + 10 TO 22 + 70
AREA 1
VALLEY HAUL ROAD RECLAMATION

FINAL CONTOURED SURFACE

Figure 2-2
(NOT TO SCALE)

71% SLOPE (NATURAL)

173% SLOPE (ROAD CUT)

19% (5.4H:1V)

47% SLOPE (2.3H:1V)

63% SLOPE (NATURAL)

FILL REMOVED

72% SLOPE

TYPICAL CROSS SECTION-UNSTABLE FILL

STA. 24 + 06 TO 29 + 34

AREA 2
VALLEY CAMP OF UTAH  
BELINA HAUL ROAD RECLAMATION  
FINAL CONTOURED SURFACE  

Figure 2-3
(NOT TO SCALE)

83% SLOPE (NATURAL)  
173% SLOPE (ROAD CUT)  
22% SLOPE (4.3H:1V)  
55% SLOPE (NATURAL)  

FILL REMOVED  
42% SLOPE 2.3H:1V)  

68% SLOPE

TYPICAL CROSS SECTION—UNSTABLE FILL  
STA. 30 + 40 TO 32 + 00

AREA 3
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
FINAL CONTOURED SURFACE

Figure 2-6
(NOT TO SCALE)

100% SLOPE (NATURAL)

173% SLOPE (ROAD CUT)

25% SLOPE (4.3H:1V)

FILL REMOVED

47% SLOPE (2.1H:1V)

70% SLOPE (NATURAL)

80% SLOPE

TYPICAL CROSS SECTION—UNSTABLE FILL
STA.  61+00 TO 64+12
AREA 6
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
FINAL CONTOURED SURFACE

Figure 2-7
(NOT TO SCALE)

125% SLOPE (NATURAL)
143% SLOPE (ROAD CUT)
CLASS I RIPRAP (12" MIN.)
4% BACKSLOPE
EXISTING CONCRETE DITCH
FILL REMOVED
67% SLOPE (1.6H:1V)
84% SLOPE (NATURAL)
128% SLOPE

TYPICAL CROSS SECTION—UNSTABLE FILL
STA. 73 + 00 TO 75 + 60
AREA 7
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
FINAL CONTOURED SURFACE

Figure 2-8
(NOT TO SCALE)

TYPICAL CROSS SECTION—UNSTABLE FILL
STA. 77 + 18 TO 82 + 46
AREA 8
The Bowel Crossing has not been considered to be a critical fill area due to the reclamation plans in this reach. By removing the top portion of the fill, this region does not present a slope stability problem and should remain stable.

2.4 Methods and Results of the Slope Stability Analyses

The slope stability analysis was performed using the computer model STABL 5. Typical geometries before and after reclamation of the Belina Haul Road are illustrated in Figures 2.9 and 2.10. The natural talus slope used for model input was set equal to 53 percent and the talus slope to 68 percent. The reclamation slope was based upon the capability of a backhoe to reach downslope 25 feet. The soil density was assumed to be 100 pounds per cubic foot with a phi value of 31°.

The factor of safety for the talus on the current haul road was estimated to be .904, which is reasonable since the talus' phi value input is 31° and no adjustment was made for the irregular bedrock formation. The factor of safety after reclamation was estimated to be 1.08, which increases the factor of safety from the original haul road geometry by 19 percent. This increases the factor of safety to be greater than most of the natural talus since many of the natural slopes are unstable. Most natural talus slopes in the region have a factor of safety equal to 1.00 for their given geometry. Note that the failure plane configuration produced by the model shows shallow, circular failures which are predominant in this region. The very steep slopes noted in this study were made up of coarser sands and gravels which have considerably higher friction angles than the soil used for the typical section modeling. This non-homogeneity is common in young, shallow soils with some deviation in parent material and weathering exposure.
VALLEY CAMP OF UTAH
BELINA HAUL ROAD CLOSURE
SLOPE STABILITY ANALYSIS
SLOPE GEOMETRY FOR CURRENT HAUL ROAD

Figure 2-9

---

Y-AXIS (FT.)

<table>
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<tr>
<th>0</th>
<th>10</th>
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<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
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<td>0</td>
<td>10</td>
<td>20</td>
<td>30</td>
<td>40</td>
<td>50</td>
<td>60</td>
</tr>
</tbody>
</table>

X-AXIS (FT.)

---

- **Failure Line**: F.S. = .904
- **Talus**
- **Natural Talus**
- **Bed Rock**
VALLEY CAMP OF UTAH
BELINA HAUL ROAD CLOSURE
SLOPE STABILITY ANALYSIS
SLOPE GEOMETRY FOR RECLAIMED ROAD

Figure 2-10

- FAILURE LINE
  F.S. = 1.08

Y-AXIS (FT.)

X-AXIS (FT.)

TALUS FILL
BED ROCK
NATURAL TALUS
TALUS
3.1 General

Surface water runoff was determined for the seven small drainages on the Belina Haul Road using the Soil Conservation Service (SCS) curve number method and the TR-20 Computer Program. Once flows were determined for each of the drainages, typical channels were developed and the velocity was determined so that the riprap sizing could be developed. Also included as part of the surface water design are the water bars to be constructed along the recontoured road.

3.2 Design Flows

The design storm for the seven drainages shown on Exhibit 3-1 was the 100 year, 24 hour, which has a rainfall amount of 3.65 inches. This is based on information developed for the Clear Creek Summit, Utah. Table 3-1 shows the precipitation depths versus return period for the Clear Creek Summit Site. The flows were developed based on a Type II rainfall distribution and are shown in Table 3-1.

The major parameters used in determining the runoff with the TR-20 model are the drainage area, time of concentration and CN. The time of concentration is defined as the time required for water to travel from the most hydraulic point of the watershed to the point of interest. It is computed by adding together the time for various segments of the conveyance system. For the mountainous drainage along the Belina Haul Road the time was estimated following the steps outlined in the SCS TR-55 publication and consist of three parts, sheet flow, shallow concentrated flow and open channel flow. The time of travel for each segment was computed and added together to determine the time of concentration for the drainage.
Estimated precipitation depths for various return periods and durations at Clear Creek, Summit, Utah (from Richardson (1971)).

<table>
<thead>
<tr>
<th>RETURN PERIOD (years)</th>
<th>5 Min</th>
<th>10 Min</th>
<th>15 Min</th>
<th>30 Min</th>
<th>1 Hr</th>
<th>2 Hr</th>
<th>3 Hr</th>
<th>6 Hr</th>
<th>12 Hr</th>
<th>24 Hr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.10</td>
<td>0.16</td>
<td>0.20</td>
<td>0.28</td>
<td>0.35</td>
<td>0.46</td>
<td>0.57</td>
<td>0.84</td>
<td>1.08</td>
<td>1.33</td>
</tr>
<tr>
<td>2</td>
<td>0.12</td>
<td>0.19</td>
<td>0.25</td>
<td>0.34</td>
<td>0.43</td>
<td>0.57</td>
<td>0.70</td>
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<td>1.34</td>
<td>1.65</td>
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<td>5</td>
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<td>0.24</td>
<td>0.31</td>
<td>0.43</td>
<td>0.54</td>
<td>0.72</td>
<td>0.90</td>
<td>1.34</td>
<td>1.73</td>
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<tr>
<td>10</td>
<td>0.19</td>
<td>0.29</td>
<td>0.37</td>
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<td>0.65</td>
<td>0.86</td>
<td>1.06</td>
<td>1.55</td>
<td>1.99</td>
<td>2.45</td>
</tr>
<tr>
<td>25</td>
<td>0.24</td>
<td>0.38</td>
<td>0.48</td>
<td>0.66</td>
<td>0.84</td>
<td>1.08</td>
<td>1.31</td>
<td>1.88</td>
<td>2.39</td>
<td>2.92</td>
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<td>50</td>
<td>0.25</td>
<td>0.38</td>
<td>0.48</td>
<td>0.67</td>
<td>0.85</td>
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<td>1.40</td>
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<td>2.67</td>
<td>3.29</td>
</tr>
<tr>
<td>100</td>
<td>0.27</td>
<td>0.42</td>
<td>0.53</td>
<td>0.73</td>
<td>0.93</td>
<td>1.24</td>
<td>1.54</td>
<td>2.29</td>
<td>2.96</td>
<td>3.65</td>
</tr>
</tbody>
</table>
Runoff curve number (CN) are based on hydrologic soil group, cover type, and antecedent moisten condition of the soil. The soils and vegetation maps from Valley Camps' approved mining permit application (UT-0013) were used to determine the CN value for each of the drainages. The USFS has recently completed classifying their lands and assigning CN values and was contacted to see how values compared. In general the values computed for the haul road agreed quite closely and were slightly higher giving a more conservative estimate of the flow and were judged to be reasonable for forest lands. Table 3-2 below shows the data used to compute the design flows for each of the drainages.

**TABLE 3.2**  
*WATERSHED SIZE AND FLOW CHARACTERISTICS*

<table>
<thead>
<tr>
<th>Watershed #</th>
<th>CN</th>
<th>tc hrs.</th>
<th>Area, AC</th>
<th>Q cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>60</td>
<td>.49</td>
<td>18.8</td>
<td>6.7</td>
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<tr>
<td>2</td>
<td>60</td>
<td>.35</td>
<td>9.6</td>
<td>4.3</td>
</tr>
<tr>
<td>3</td>
<td>60</td>
<td>.74</td>
<td>11.8</td>
<td>3.2</td>
</tr>
<tr>
<td>4 (Bowl Crossing)</td>
<td>60</td>
<td>.71</td>
<td>147.8</td>
<td>44.0</td>
</tr>
<tr>
<td>5</td>
<td>60</td>
<td>.71</td>
<td>14.2</td>
<td>4.0</td>
</tr>
<tr>
<td>6</td>
<td>60</td>
<td>.56</td>
<td>25.0</td>
<td>8.3</td>
</tr>
<tr>
<td>7 (Eccles Creek)</td>
<td>60</td>
<td>1.37</td>
<td>2087.0</td>
<td>378.</td>
</tr>
</tbody>
</table>

3.3 Channel Design

It is proposed in the reclamation plan that the existing culverts be removed and the ephemeral channels reestablished at their original slope and be protected with riprap. Figure 3.1 shows a typical section through the road after regrading and contouring and the various hydraulic data.

The slope will vary from about 15 percent across the road to a maximum of 65-70 percent along the slope face. To replace the culverts on five of the smallest drainages, a small "V" ditch will
be constructed to carry the flow from each of the small drainages through the road sections. These small ditches will be protected with riprap and a typical section is shown on Figure 3-I.

Velocities were computed using the Manning's equation. The channel roughness coefficient, n, was estimated based on values for small mountain streams where the depth of flow is small when compared to the size of bed material. In Open-Channel Hydraulics, Chow suggested n values range from .040 to .070 for small, steep mountain streams with cobbles and boulders. Values were also determined from the paper Two Approaches for Estimation of Manning's n in Mountain Streams, by Weache, et al. at the Wyoming Water Research Center. Based on their method, n is estimated to range from .065 to .085. A value of .06 was used in all of the computation. It was felt that the turbulence would be very high since the average depth of flow would range from .5 feet to 1.0 feet and the riprap size would range from 1.0 feet to 2.0 feet.

While this value is higher than those typically used, (.035 - .045), it is felt justified because the depth of flow is much greater than the stone size and this is not the case for the Belina Haul Road drainages.

3.3.1 Small Drainages

Flows vary from 3.2 to 8.3 cfs for the five smallest drainages. The small "V" ditches were designed based on the maximum flow of 8.3 cfs. This will provide a conservative design and will standardize them making construction easier. The velocity will vary from about 4 feet per second for the 15 percent slope to about 10 ft/sec. on the steeper slope of 70 percent. Details for each of the crossings are shown on Figure 3-1.
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
SMALL DRAINAGE
HYDRAULIC DATA
Figure 3-1

FREEBOARD = 12" MIN.

THICKNESS OF RIPRAP SHOULD EQUAL THE LARGEST STONE SIZE

TYPICAL "V" DITCH
FOR SMALL DRAINAGES

REMOVE EXISTING FILL TO APPROXIMATELY ORIGINAL FLOW LINE AND PROTECT WITH RIPRAP

NATURAL SLOPE

TYPICAL STREAM CROSSING
FOR SMALL DRAINAGES

EXTEND RIPRAP BEYOND FILL SLOPE

NATURAL SLOPE

A  B  C

4:1  2:1  3:1
### VALLEY CAMP OF UTAH
#### BELINA HAUL ROAD RECLAMATION
#### SMALL DRAINAGE
#### HYDRAULIC DATA
#### Figure 3-1a

<table>
<thead>
<tr>
<th>AREA</th>
<th>DRAINAGE</th>
<th>CHANNEL SLOPE</th>
<th>Q in cfs.</th>
<th>*D Depth Ft.</th>
<th>VELOCITY Ft./Sec.</th>
<th>RIPRAP CLASS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6.7</td>
<td>.85</td>
<td>5</td>
<td>I</td>
</tr>
<tr>
<td>A</td>
<td>1</td>
<td>15%</td>
<td>6.7</td>
<td>.85</td>
<td>5</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>15%</td>
<td>4.3</td>
<td>.70</td>
<td>4</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>15%</td>
<td>3.2</td>
<td>.65</td>
<td>4</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>15%</td>
<td>4.0</td>
<td>.70</td>
<td>4</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>15%</td>
<td>8.3</td>
<td>.90</td>
<td>5</td>
<td>I</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6.7</td>
<td>.65</td>
<td>9</td>
<td>II</td>
</tr>
<tr>
<td>B</td>
<td>1</td>
<td>63%</td>
<td>6.7</td>
<td>.65</td>
<td>9</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>63%</td>
<td>4.3</td>
<td>.55</td>
<td>8</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>55%</td>
<td>3.2</td>
<td>.50</td>
<td>7</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>70%</td>
<td>4.0</td>
<td>.50</td>
<td>8</td>
<td>II</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>70%</td>
<td>8.3</td>
<td>.70</td>
<td>10</td>
<td>II</td>
</tr>
<tr>
<td>C</td>
<td>ALL</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>II</td>
</tr>
</tbody>
</table>

*See Figure 3-1 for location*
3.3.2 Bowl Crossing

Design of the channel for the Bowl Crossing drainage (Area 4) was done in a similar manner. The 100 year design flood is estimated at 44 cfs. It is proposed that a small overland flow channel be constructed through the rock fill after the soil fill has been removed, (See Section 4.3.1) which will have a bottom width of four feet. Figure 3-2 shows a typical section through the fill. The existing culvert will remain in place and will carry the smaller flows. The new overland flow channel will carry the flood flows for the more infrequent storms and also if the culvert should become clogged. The velocity in the new channel will vary from 8 ft/sec. across the rock fill where the slope is about 15 percent to 13 ft/sec. down the steeper natural slope. Details of the channel and hydraulic data are shown on Figure 3-2.

The design for Eccles Creek drainage is covered in Section 3.5.

3.4 Riprap Design

Riprap sizing was selected based on the above velocities using USBR Engineering Monograph #25 and FHWA Hydraulic Engineering Circular #11. The $d_{50}$ size is four inches on the flatter slopes (Class I) and is nine inches on the steeper slopes (Class II). Gradation for the different classes of riprap are shown in the Table 3.3.
### TABLE 3.3
RIPRAP DESIGN

<table>
<thead>
<tr>
<th>Class</th>
<th>Max.</th>
<th>(d_{50})</th>
<th>Min.</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>8</td>
<td>4</td>
<td>1 1/2</td>
</tr>
<tr>
<td>II</td>
<td>24</td>
<td>18</td>
<td>6</td>
</tr>
<tr>
<td>III</td>
<td>36</td>
<td>24</td>
<td>8</td>
</tr>
</tbody>
</table>

Riprap should be reasonably well graded from the maximum size down to the minimum. The concrete removed from the project will be used as part of the riprap protection and will be broken so as not to be larger than the \(d_{50}\) size and will not makeup more than 15 percent of the volume. The riprap will extend beyond the toe of the fill slopes a minimum of five feet to provide energy dissipation at the termination of the riprap channels, the energy dissipator will be small mounds of riprap approximately 18"-24" high to help spread the flows out and reduce erosion.

A filter blanket will be constructed and placed between the riprap and the native material. The filter will be constructed of a well-graded gravel with a minimum size of about 3\(1/16\)" up to a maximum required by the riprap class and is shown below in Table 3.4.

### TABLE 3.4
RIPRAP FILTER DESIGN

<table>
<thead>
<tr>
<th>RIPRAP CLASS</th>
<th>MAX, in</th>
<th>MIN, in</th>
<th>THICKNESS, in</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>*</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II</td>
<td>4</td>
<td>3/16</td>
<td>9</td>
</tr>
<tr>
<td>III</td>
<td>6</td>
<td>3/16</td>
<td>9</td>
</tr>
</tbody>
</table>

*Not required; native material acceptable*
3.4.1 Small Drainages

The existing native material appears to be sandy gravelly material based on field inspections. The gradation is estimated to be from 3 inches down to less than 1/8" with a d_{50} size of about 3/8". This material will meet the requirements for a filter material for the Class I riprap, since the d_{15} Riprap/d_{85} Base is less than 5.

3.4.2 Bowl Crossing

Based on field observations and discussions with the mining operation people, it appears that the blast rock in the Bowl crossing fill has a maximum size of 18 inches to 36 inches. This would provide adequate protection based on the above velocities. If, when the soil fill is removed and the channel is constructed, it is determined that the actual blast rock is not large enough, additional riprap protection (Class III) will be provided.

After excavating the soil fill at the Bowl Crossing the base material will be examined to determine if it meets the filter criteria. If it does not, a filter will be constructed meeting the gradation shown in Table 3-4.

3.5 Eccles Creek

The drainage above the Belina Haul in Eccles Creek is the largest with an area of 2,047 acres. The 100 year 24 hour storm is estimated to be about 378 cfs. The channel slope in this area is estimated to be 2 - 2.5 percent. This channel will have a low flow section with a width of 12 feet. The velocity for the 100 year storm will be approximately 6.6 feet per second with depth varying from about 1.5 feet in the floodway to 3.5 feet in the main channel. A Manning's n of .060 was used in computing the flow depth and velocities for Eccles Creek. Based on this velocity and depth of flow, the Class II (24 inch)
Riprap will be required. The channel will be constructed with similar geometry to the recently reconstructed channel below the Belina Haul Road turnout. A typical section through this channel is shown in Figure 3-3. The design of this channel will be similar to the recently completed channel reconstruction just downstream. This will maintain continuity in the channel design. The channel will include several drop structures to maintain a reasonable stream gradient of 2.5 percent or less. These drop structures will be constructed of large rock so that they will maintain a natural appearance. Figure 3-3 shows a plan view of the proposed new alignment after the fill is removed.

The velocity and depth were also computed for the average annual flow to evaluate the effects on fish passage, and are presented in Table 3.3. The average annual flow is estimated to be about 28 cfs. This flow was computed using the USGS report, "Methods for Estimating Peak Discharges and Flood Boundaries of Streams in Utah", WRI 83-4129. In addition to the average annual flow, depths and velocities were computed for several other flows.
HEAVY LINES DENOTE
RECLAIMED CONTOURS

HAUL ROAD FILL
TO BE REMOVED

CONSTRUCT "V" DITCH,
PROTECT WITH CLASS II
RIPRAP

RIPRAP DROP STRUCTURES
CLASS III (TYPICAL 3 PLACES)

ECCLES CREEK PLAN VIEW

APPROX. FLOODWAY WIDTH
25 to 30 ft.

APPROX. 12'

CLASS II RIPRAP PROTECTION
ALONG TOE OF FILL SLOPES

RIPRAP CLASS II
ALONG STREAM BANKS

TYPICAL SECTION THROUGH ECCLES CREEK

VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
ECCLES CREEK CHANNEL DESIGN
PLAN AND SECTION
Figure 3–3
TABLE 3.3
ECCLES CREEK CHANNEL HYDRAULICS

<table>
<thead>
<tr>
<th>Discharge, cfs</th>
<th>Depth, Ft.</th>
<th>Velocity, Ft./Sec.</th>
<th>Channel Slope %</th>
</tr>
</thead>
<tbody>
<tr>
<td>15</td>
<td>.55</td>
<td>2.2</td>
<td>2.0</td>
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<tr>
<td>20</td>
<td>.65</td>
<td>2.5</td>
<td>2.0</td>
</tr>
<tr>
<td>28*</td>
<td>.80</td>
<td>2.8</td>
<td>2.0</td>
</tr>
<tr>
<td>30</td>
<td>.85</td>
<td>2.9</td>
<td>2.0</td>
</tr>
</tbody>
</table>

*Average annual flow
These are within the reported sustainable swimming speed for trout, which is two to six feet per second as reported in Fisheries Handbook, by Milo C. Bell, 1986. These velocities were not related to depth of flow in Milo C. Bell's report.

3.6 Concrete Ditch: Sta 71+00 to 82+00

A riprap ditch will be constructed at the base of the cut slope from about Sta. 71+00 to Sta. 82+00 where the haul road intersects Eccles Creek as shown on Figure 2-7 and Figure 2-8. Class I riprap will be placed over the existing concrete ditch with a minimum depth of about 12 inches. The reclaimed back slope of the road surface will contain the design flows. The last 100 feet of this ditch has a slope of about 35 percent where it drops down into Eccles Creek. This reach will be constructed similar to those in Figure 3-1. The ditch will have side slopes of 2H:1V and be protected with Class II riprap.

3.7 Water Control Bars

Water control bars will be constructed to reduce erosion of the recontoured haulroad. Figure 3-4 shows a typical waterbar. These structures will be spaced approximately 100 feet apart along the road. Waterbars will be placed more frequently if, during the final reclamation work it is determined they would be necessary to control runoff. Class I riprap protection will be included in the construction of the water control bars. The riprap will be placed at the point where the flow breaks over the edge of the old road bed.
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
TYPICAL WATERBAR DETAILS
Figure 3-4
SECTION 4.0 - RECLAMATION PROCEDURES

4.1 Road Surface Removal

Reclamation of the road will begin with the removal of the asphaltic concrete road surface and the Portland cement concrete lining of the water control ditch which is located at the toe of the road cut slope. A portion of the cement concrete ditch (from station 71+00 to station 82+00) will be left in place and backfilled with riprap, as discussed in Section 3.6. After the road surface is reclaimed and the recontoured surface sloped back towards the hill at approximately four percent, this water control structure will convey water to Eccles Creek. Leaving this portion of the concrete ditch in place will minimize infiltration to the fractured rock hillside, thereby lessening the chance of slope failure. This water control structure will be monitored for bond release for the same period as the rest of the reclamation. The cement concrete lining will be rubblized to eliminate any slippage surface when it and the asphaltic concrete and fill material are placed for disposal. The larger pieces of cement concrete will be salvaged and used as riprap if they meet the specifications for riprap discussed in Section 3.4.

The asphaltic concrete will then be broken and will be placed against the toe of the cut slope over the previously placed broken Portland cement concrete. The asphaltic concrete will be piled approximately four feet deep adjacent to the cut slope and graded to ground level seven to eight feet out from the toe of the slope (Figure 4-1). There are approximately 3,500 in place cubic yards of asphaltic concrete to dispose of. To insure a competent fill and prevent piping, the asphaltic concrete will be placed in an engineered manner and compacted. The asphalt will be broken by ripping it with the
VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
TYPICAL CROSS SECTION - STABLE FILL
FINAL CONTOURED SURFACE
Figure 4-1
(NOT TO SCALE)
scarifiers on a motor grader or equivalent machine. The scarifiers are approximately 17-20 inches apart. It is expected, therefore, that their use will create pieces of asphalt less than two-feet square. If necessary a dozer will be walked over the ripped asphalt to further reduce the size. The rubblized asphalt will then be bladed to the toe of the cut slope by a motor grader or equivalent equipment. The asphalt will be compacted in one foot lifts. Once the asphaltic concrete surface has been removed, the gravel road base material will be ripped or disked to eliminate compaction and to promote water infiltration and root penetration.

After the asphalt is placed and compacted it will be covered with soil removed from the out slope fill portions of the road, to a sufficient depth to prevent it from being exposed to the atmosphere. The surface of the replaced soil will be contoured as shown in Figure 4-1 to reestablish a drainage pattern similar to that which was present prior to mining.

4.2 Corrugated Metal Pipe Removal

Seven of the eight corrugated metal pipe (CMP) culverts buried in the Belina haul road will be removed during reclamation. These channels, which include Eccles Creek, will be cleared of fill material, recontoured and riprapped as necessary to prevent excessive erosion. The riprap material will consist of large competent rock and/or broken pieces of cement concrete as discussed in Section 3.4 of this report. The removed CMP will be salvaged if possible, or disposed of in a section of the underground mine workings as detailed in Section 784.13 of Valley Camp's approved Mining and Reclamation Plan Permit Number UTOO13.

The remaining CMP is the large culvert through the fill in the Bowl. As agreed to during a site visit with UDOGM personnel, this CMP will be left in place unplugged. The reconstruction of a channel through the fill will provide a significant overflow safety factor in the unlikely event that the CMP would become dammed or plugged.

0842h/37
4.3 Recontouring

The recontoured areas will be developed by placing soil material excavated from two major fill areas (the Bowl crossing and the Eccles Creek crossing) on the "cut" portions of the road against the cut slopes as buttress fills. Additionally, portions of the road outslope fill areas are considered to be of questionable stability and will therefore also be excavated and placed in the buttress fills. Approximately 30,000 to 35,000 cubic yards of material will be excavated and placed during this recontouring effort. Drainage crossovers will be constructed across this recontoured surface to shorten the slope length and prevent excessive erosion (refer to Section 3.6 for details). These cutouts or crossovers will be riprapped to prevent the development of rills and gullies.

The reclaimed surface of the haul road will in most cases, slope to the outside. In some cases, however, it will slope back to the hill. Approximately the first 1,100 feet of the road, (from station 82+00 to station 71+00 on the CEI, 9/83 drawings) has a very steep outslope (approaching 120 percent) toward Eccles Creek. To keep water off of the face of this area and protect Eccles Creek the recontoured surface will pitch into the hill at approximately four percent (Figure 2.8). Runoff will be collected in a riprapped ditch constructed at the base of the road cut slope and will be conveyed down the hill and released to Eccles Creek approximately at the haul road/creek junction. The design of this ditch is addressed in Section 3.5 of this report.

4.3.1 Bowl Crossing

The largest fill is located near the midpoint of the haul road. It consists of blast rock on the bottom and soil on the top. The soil portion (approximately 15,000 yd³) will be excavated and an overland channel will be developed through the remaining rock. The CMP will be left in place unplugged. This new drainage will be a permanent
structure constructed from competent rock which meets riprap specifications. In addition, energy dissipaters will be utilized, if necessary, to control the flow of water until it reaches the natural drainage channel. Figure 4.2 is a cross section showing the present road surface, fill slopes, and the projected location of the overland flow channel.

4.3.2 Eccles Creek Crossing

The second major fill is located at the intersection of the Belina Haul Road and the Eccles Canyon Road. This fill consists primarily of blast rock from the development of the first section of the haul road and is covered with soil. Again, only the soil portion will be removed. The remaining rock will be used as riprap for the rehabilitation of Eccles Creek, provided it meets riprap specifications. Any unused rock will be disposed of as discussed in Section 784.13 of Valley Camp's approved mining and reclamation plan (UT 0013). The corrugated metal pipe will be removed and disposed of similarly. These activities will allow Eccles Creek to return to its natural channel.

4.3.3 Unstable Fill Slopes

The third area from which backfill material will be obtained is from the portions of the outslope road fills that have been determined to be potentially unstable (Table 2.1). A sufficient quantity of fill will be removed from each of these fill slopes to reduce the potential of the slope failing. To initiate reclamation of these fill slopes, the guard rails will be removed and the support post and metal rails will be salvaged or disposed of.

The excavated material (Figure 4.3) will be removed using a backhoe or a similar machine to reach down the slope to retrieve material. As a result of this operation, the road edge will be cut back toward the
toe of the cut slope ten to fifteen feet. With the removal of this material the final surface will have an approximate slope of 2.5H:1V. The excavated material will be placed on the remaining road surface thereby creating an outslope of approximately 4H:1V.

The quantity of fill material estimated to be removed from the various sources and the estimate of the storage capacity that can be developed from utilizing the road surface is given in Table 4.1.
### TABLE 4.1

VALLEY CAMP OF UTAH
BELINA HAUL ROAD RECLAMATION
SOIL DISPOSAL VOLUMETRICS

**Fill Material To Remove:**

- Eccles Creek Fill  \( 4,000 \text{ yd}^3 \)
- The Bowl Crossing Fill  \( 15,000 \text{ yd}^3 \)
- Haul Road Outslopes  \( 6,000 \text{ yd}^3 \)
- Remaining CMP Removal  \( 1,500 \text{ yd}^3 \)
- Asphaltic Concrete and Broken Cement Concrete  \( 5,000 \text{ yd}^3 \)

**GRAND TOTAL**  \( 31,500 \text{ yd}^3 \)

**Storage Capacity:**

- Haul Road with Stable Outslopes (3,470 feet)  \( 25,000 \text{ yd}^3 \)
- Haul Road with Portion of Outslopes Removed (2,780 feet)  \( 6,000 \text{ yd}^3 \)
- Backslope Section of Road (1,250 feet)  \( 3,000 \text{ yd}^3 \)

**GRAND TOTAL**  \( 34,000 \text{ yd}^3 \)
4.4 Topsoiling

During the construction of the haul road the overlaying topsoil and subsoils were excavated and stockpiled where possible, sidecast or used as fill. During the reclamation of the haul road some of the material which was sidecast and/or used for fill material will be excavated and used to recontour the road. The suitability of this material as a growth medium is evidenced by the vegetation currently growing on it and in fact very similar material has already been approved for use as topsoil at this mine by the Utah Division of Oil Gas and Mining. Prior to using this material as topsoil however, it will be analyzed for pH, texture, electrical conductivity, calcium, magnesium, sodium, organic matter, phosphorous and potassium. Because this material is a mixture of topsoil and subsoil and because no segregated topsoil stockpiles exist at this mine "topsoil" will not be placed on the regraded surface.

4.5 Seed Bed Preparation

The soil removed from the large fills will be replaced using dozers and scrapers. Soil removal from the potentially unstable outslopes will be accomplished using a backhoe or similar equipment. The soil replaced by scrapers and dozers will be scarified to a depth sufficient to allow root penetration whereas the soil placed by the backhoe will not require loosening since it will be subject only to limited packing. The final recontoured surface will then be disked or tracked on the contour prior to seeding.

4.6 Seeding

Seeding will follow the procedures and seed mixes outlined in Valley Camp's approved Mining and Reclamation plan, Permit Number UT 0013.
Areas of the haul road outslopes and cut slopes which will not be disturbed by reclamation activities will be subjected to a statistically valid vegetation survey at the time to determine the adequacy of the existing vegetation when compared to reference areas identified in Mining Permit Number UT-0013. If it is determined necessary, these undisturbed areas will be interseeded or interplanted with shrubs.

4.7 Fertilizing

A chemical analysis will be performed on samples of the soil which will indicate the nutrients and amounts necessary for proper plant growth. Fertilizer will be applied either just prior to or immediately following seeding.

4.8 Mulching

Mulch will be applied at approximately 2,000 pounds per acre, depending on the material of choice, and will follow application of the seed and fertilizer. The mulch will be straw or any of the other commonly used mulch materials. At the time of reclamation, where it is deemed necessary, a tackifying agent or some other means will be used to hold the mulch in place.

4.9 Erosion Control and Maintenance

During reclamation activities, interim erosion control measures such as filter fabric and straw bales will be used to control water flow. Once a drainage channel is established, these interim structures will be removed and the disturbed areas will be seeded, fertilized and mulched. At the conclusion of reclamation activities, runoff will be slowed by the proper placing of straw bales, filter fabric fences, riprap or mulch, in potential
problem areas. If runoff channels develop in excess of nine inches, the most applicable erosion control technique will be selected. For example, small erosion channels will be blocked with a filter fabric fence, a straw bale or some other material to slow the water and allow vegetation to establish.

4.10 Revegetation

The revegetated area will be monitored closely to ensure that a diverse, permanent vegetation cover capable of self-regeneration is developed. Revegetation success of the newly reclaimed haul road areas will be determined by following the techniques developed in Section 817.116 of Valley Camp's approved mining and reclamation plan, Permit Number UT-0013.

4.11 Reclamation Costs

Reclamation costs are summarized by task for the purpose of bonding costs. These cost estimates are made with the knowledge that the efficiency of workers and machinery may far exceed the normal rate because of the very limited work space, and the difficulty in scheduling of crews. The reclamation cost estimates are given in Table 4.2.
**TABLE 4.2**

**RECLAMATION COST ESTIMATES**

Remove Concrete and Asphaltic Concrete:
- Rip Portland Cement Concrete and Breakup Asphaltic Concrete $3,500
- Remove and Place Asphaltic Concrete (40 hrs. @ $87.50/hr.) 2,800
- Compact Asphaltic Concrete 1,600
- Break and Remove Concrete Ditch 6,500

Rip/Scarify Road Base Material (65 acres) (8 hrs. @ $75.00/hr.) 600

Remove corrugated Metal Pipes 8,000

Remove and Dispose Guard Rails, Posts, and Signs 5,800

Remove and Place Fill Material:
- 20,000 yd³ (Intersecting Drainage Fills) 50,000
- 7,000 yd³ (Road Outslope Fills) (80 hrs. @ $100/hr.) 8,000

Recontour Road Surface:
- 10.0 Acres + (80 hrs. @ $100.00/hr. 8,000

Construct Riprap Drainage Channels:
- 8 each (@ 200 feet each) 32,000

Redistribute Topsoil Substitute (10 Acres x 6" Deep):
- 8,100 yd³ + (@$2.50/yd.) 20,000

Seedbed Preparation (Scarification, Disking, Harrowing) 1,000

Fertilizing, Seeding, and Mulching:
- Seed: (10 acres @ 24.0 lbs/acre @ $15.00/P.L.S. lb.) $3,600
- Fertilizer: (10.0 acres @ $425.00/acre) $4,250
- Mulching: (10.0 acres @ $500.000/acre $5,000
- Equipment and Labor: $2,000
- Total 14,850
- Monitoring 1,000

**TOTAL** $163,650

10% Mobilization and Demobilization 16,365
15% Profit and Administration 24,548
Maintenance-10 Acres @ $100.00/ac/yr. 1,000

**TOTAL BONDING COST** $205,563