GREENWOOD FEASIBILITY STUDY

Preservation of Existing Smelter Heritage Structures

June 2017

DJ&T Engineering Ltd.

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1 Introduction

The City of Greenwood (City) has requested that ISL Engineering and Land Services (ISL) investigate the potential for site development of a decommissioned smelter site in Greenwood, BC. The smelter ruins site holds significant historic value for both the community, and the mining industry in British Columbia. and in Canada. As such, the primary objective of the Project is to develop the smelter ruins site into a historic monument and tourist attraction that can be safely experienced by the general public.

DJ&T Engineering Ltd. (DJ&T) was retained by ISL to undertake a structural assessment of existing brick structures and rock and mortar foundations. The purpose of the structural assessment was to identify site hazards presented by the existing degraded structures and to provide hazard mitigation recommendations for safe public access.

2 Site Visit

DJ&T made a site visit on April 18 2017 with Mr. Sean Anan P. Eng. of ISL, to complete a visual inspection of existing structures of the old Copper Smelter location. The majority of structures identified on site seem to be in various states of disrepair and are obviously structurally deficient.

On April 19 2017, DJ&T met with Roland Heere, P. Eng, and Alberto Martinez, P. Geo., from Metro Testing Laboratories (Metro) to perform the overall evaluation on site and to select the structures considered relevant and safe for Metro's materials testing. Metro assessed the current status of in situ masonry materials found in existing structures and took material samples for laboratory testing of the mechanical properties of masonry elements (mortar and bricks)

3 Description of Structures

The above-ground structures on the old Greenwood Smelter site are comprised of unreinforced brick masonry (URBM) built at the beginning of the 20th century. The existing subterranean structures are mostly built of dry stacked stone blocks, with some showing indication of the use of very weak lime mortar. They appear to have very little to no structural integrity remaining, with many fallen or loose stone units.

Visual inspection was made for the majority of the flume tunnels. Two tunnels, directly tied to the already crumbled base of the main stack flume, were assessed to be at high risk of immediate collapse and were not inspected.

It appears that some minor repairs of the flume wall crowns have been made over time, but they have not been significant enough to upgrade the structures to modern standards.

Only two masonry structures were identified to be economically feasible to stabilize and secure for public use with structural retrofits - the smoke stack and the corner wall of the old smelter office building located on a hillside south east of the stack. The remaining masonry and rock walls found on the old smelter site do not seem to have enough structural integrity to feasibly upgrade and the required remedial works would be cost prohibitive. The majority of the rock walls that form the old foundry flume tunnels should be protected from further deterioration. They may be exposed to viewers, but should be kept inaccessible to the public due to a high risk of falling roof debris.

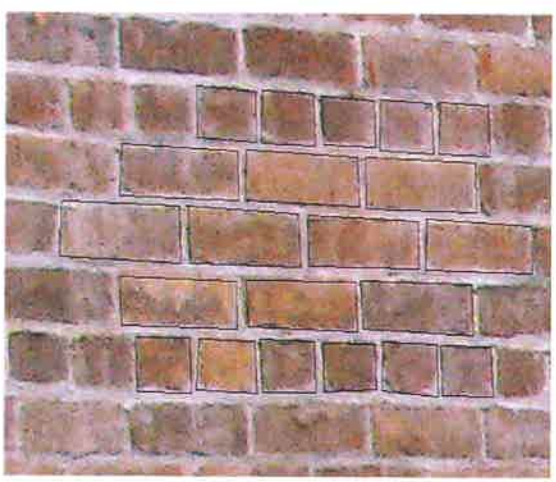


Figure 1 - The brick pattern on the stack wall and the corner building wall is predominantly a 3-layer running bond with stretcher blocks, bounded by single courses of header bricks.

Figure 2 - The only other pattern found on site was for the portion of the partially collapsed flume structure just south of the stack. It shows a common masonry layout with five running bond courses of stretcher blocks laid between single courses of headers

Measured wall thicknesses vary from 13" (building walls) to 48" or more (base of masonry stack wall). The exact number of leafs/wythes in the stack wall could not have been determined without using destructive methods. We have made a reasonable assumption that the central part of the stack wall is comprised of multiple staggered wythes, which would provide a good bearing surface capable of carrying the weight of the structure.

Structural bonding of masonry walls has been accomplished by the combination of overlapping (interlocking) of the masonry units and the adhesion of lime based mortar laid between adjacent wythes of masonry. The mortar bonds the units together and seals the spaces between by compensating for dimensional variations in the units.

The clay brick specimens found on site show slight variation in dimensions, but are generally 8" x 4" x 1 3/4". The tested units indicated relatively high compression capacity.

Walls of the collapsed covered flume structures throughout the site, and the two intact tunnels, were tied near their tops with cast iron bars that provided lateral stability to a shallow arch masonry roof. In some instances, the roof of the flumes was covered with the layer of native soil, likely to improve air tightness.

On the north side of the square stack base there is a 48" wide opening with the deep arch top. There exists a long vertical crack in the arch itself, as well as the masonry wall above it, extending to the interface with the pseudo-cylindrical stack wall. Since the masonry arch was designed to carry compressive forces only, the vertical crack indicates that the massive stack base may have experienced a horizontal movement, most likely at the foundation level (Figure 3 and Figure 4).

Figure 3 - Vertical Crack in Stack Base

Figure 4 - Close-up of the Vertical Crack

Sounding of the masonry walls with the hammer was performed on the inside surface of the stack, identifying a large cavity in the east side of the interior wall. The size of the cavity could not be determined with certainty but was estimated to cover at least 50 square feet, starting from the height of 4 ft above ground and extending as far as could be reached from the ground.

4 Structural Analysis

The masonry stack and the corner wall structures were assessed for both seismic and wind loads as per the latest BC Building Code requirements. Structural analysis was completed with the Finite Element Modeling (FEM) program SAP 2000.

It should be noted that stacks do not explicitly fall under the building code jurisdiction, although there is a portion of the code (Part 4 Commentary) that provides loading schemes suggested for the wind analysis of the stack structures. However, seismic requirements for industrial stacks are not specified, and the actual approach is left for interpretation and rational analysis. The BC Building Code seismic requirements limit the height of unreinforced masonry structures to 15m in height. With almost 40m height above ground, the existing stack does not meet the current code without additional reinforcing.

For this analysis, the stack was treated as an independent tall and slender unreinforced masonry structure. Linear response spectrum analysis was performed for the stack model to determine the governing force demands. The results of this analysis were compared with tested capacities (by Metro) of the existing masonry and mortar material to assess the capacity-demand ratios for the existing structure.

The structural model of the stack was built using thick shell elements based on the average masonry wall thickness of 48 inches. The square shaped base was added to the pseudo cylindrical tube, but the openings on the south and north faces have not been included due to complexities that would arise from modeling of the intricate elements forming the different depth intradoses of the soldier masonry courses framing the tops of openings.

The corner wall of the old office building has also been modeled using thick shell elements for the 16 inches thick masonry made of three wythes.

The foundations for both structures have been assumed to be solid and were neglected during FEM modeling. Also, both models assume an intact and perfect interface between masonry blocks through the lime based mortar. The mortar was modeled using reduced shear capacity of approximately 1.0 MPa for stack and 0.3 MPa for corner walls (as listed in Metro's test results), and based on the FEM analysis, prompted the need for mitigation.

Both wind and seismic loads were based on design information for Greenwood locality from the Design Data Table listed in the 2015 NBC of Canada:

- Basic wind pressure (1/50 year return period): q = 0.40 kPa

- Seismic parameters for general Greenwood area:

Sa(0.2) = 0.27
Sa(0.5) = 0.17
Sa(1.0) = 0.085
Sa(2.0) = 0.049
PGA = 0.14

The graph of the response spectrum was created and used in further seismic analysis with the ductility modifiers of Rd = R0 = 1.0 as specified in the Code for seismic force resisting system comprised of unreinforced masonry structures (Figure 5).

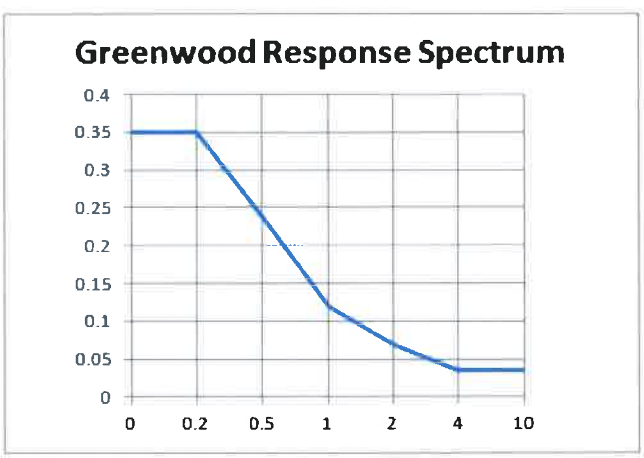


Figure 5 - Response Spectrum for Greenwood

The results of the FEM analysis were in line with the initial expectations based on visual observations: vertical tensile stresses under full seismic load would exceed the compressive stress due to gravity load, thus prompting the need for structural strengthening. The analysis shows that the stack as built would be capable of carrying only 20% of the seismic base shear as per current Code.

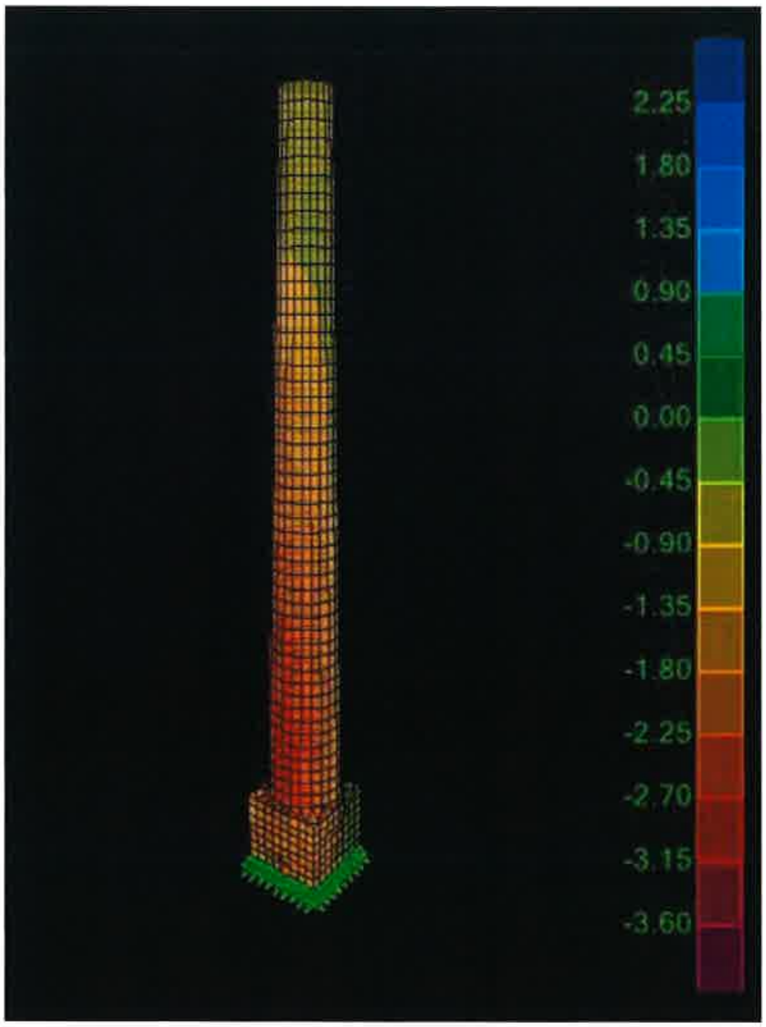


Figure 6 - Stress Results

With regards to wind load, the vertical tensile stresses caused by bending in the stack structure appear to be lower than the compression due to gravity, indicating that wind would not govern stack strengthening.

The corner walls of the old office building have also been analysed under wind and earthquake loading. This structure is significantly lighter and therefore the maximum wind load causes higher horizontal bending in the wall than the seismic load. The model was based on weak and highly deteriorated mortar in the corner wall structure. The results indicate that the wall integrity is highly compromised.

The maximum compressive stress due to horizontal bending of the wall structure under wind load is 1.23 MPa. (Figure 7).

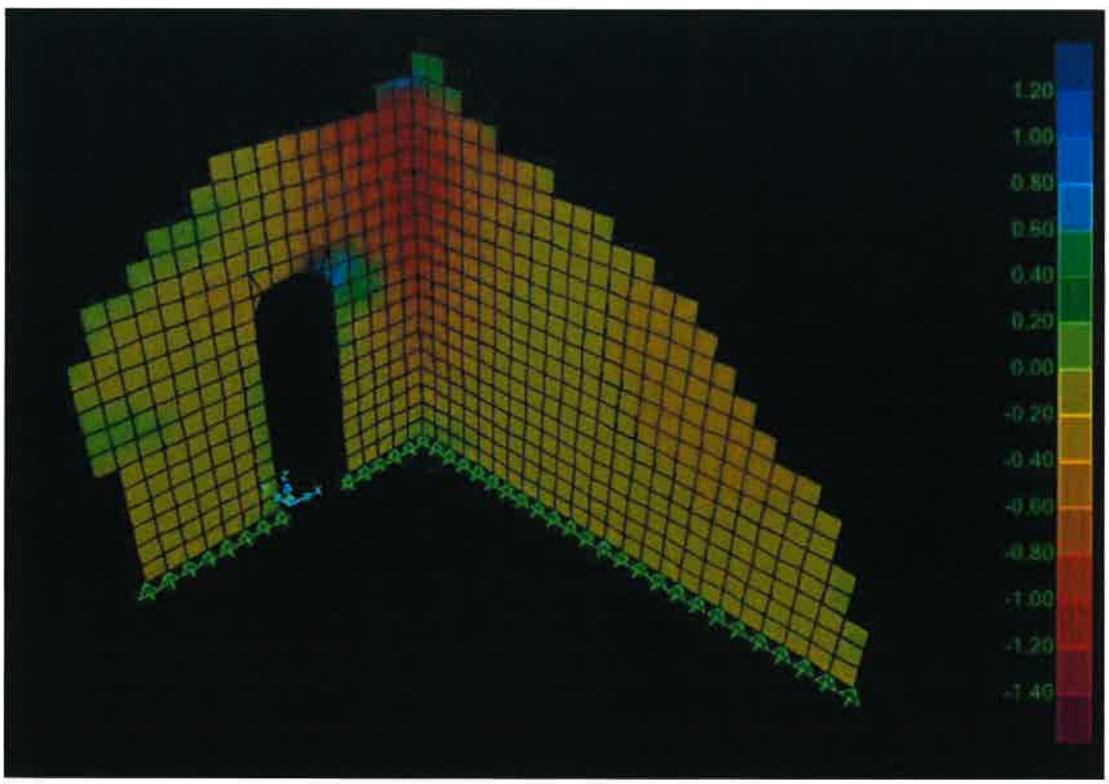


Figure 7 - Compressive Stress in Corner Wall due to Wind

This governing demand exceeds the average shear capacity of the existing mortar joints of 0.3 MPa, and thus we recommend that the corner wall be structurally supported to mitigate against potential collapse.

5 Rehabilitation Options

5.1 Masonry Chimney Structure

Based on results of the FEM analysis, remediation options were considered to stabilize and strengthen the stack structure.

We recommend that imminent retrofit should include at minimum:

- Grout injection of the large void in the east side of the stack wall. Physical and chemical compatibility between the original mortar and injected cement grout material should be determined to avoid deleterious effects from the grouting. The strength of the grout must be adequate but not overly high. However, the critical factor to grouting effectiveness is the achievable bond. The composition, porosity, and mortar grain size of the in-situ mortars and clay bricks in this zone should be established to determine the feasibility of grouting;
- Repairing deteriorated mortar coating and the bricks on the top of the stack crown to mitigate the immediate falling hazard in the event of strong winds;
- Drilling and installation of post-tensioned bars/cables to stabilize the cracked portion of the wall on the north side of the structure. Prior to installation of the PT tendons, the foundation should be exposed and investigated to confirm viability of this option. If the investigation shows that foundation experienced differential movement/settlement on this side of the wall, the retrofit scheme should consider mitigation of the foundation deficiencies at the same time;

We also recommend consideration of additional retrofits, including adding tensile enhancement of the masonry tube wall by installing vertical reinforcing elements in the lower two thirds of the stack structure. One feasible solution could be the installation of carbon fibre reinforced polymer (CFRP) rods instilled into epoxy resin in vertical grooves premade in the wall.

5.2 Corner Wall Structure

The Corner wall structure shows significant deterioration of the masonry mortar. The average demand/capacity ratio is higher than 3.0 for the structure, indicating an imminent danger of collapse under strong winds. We recommend the following approach for remediation:

- The top four courses of both walls should be carefully removed and reinstated using cement based masonry mortar. Since the base fixity of the walls is unknown and presumed nonexistent, the vertical seismic ties/dowels should be installed at every 48 inches center-to-center to enhance wall stability;
- At least one timber support structure should be installed for each wall to provide additional lateral support for the existing wall structure. One possible rehabilitation option is shown on Figure 8. Another possible solution could be structural steel strong back structures (vertical steel posts fixed at the ground level and connected to the walls at prescribed internals).

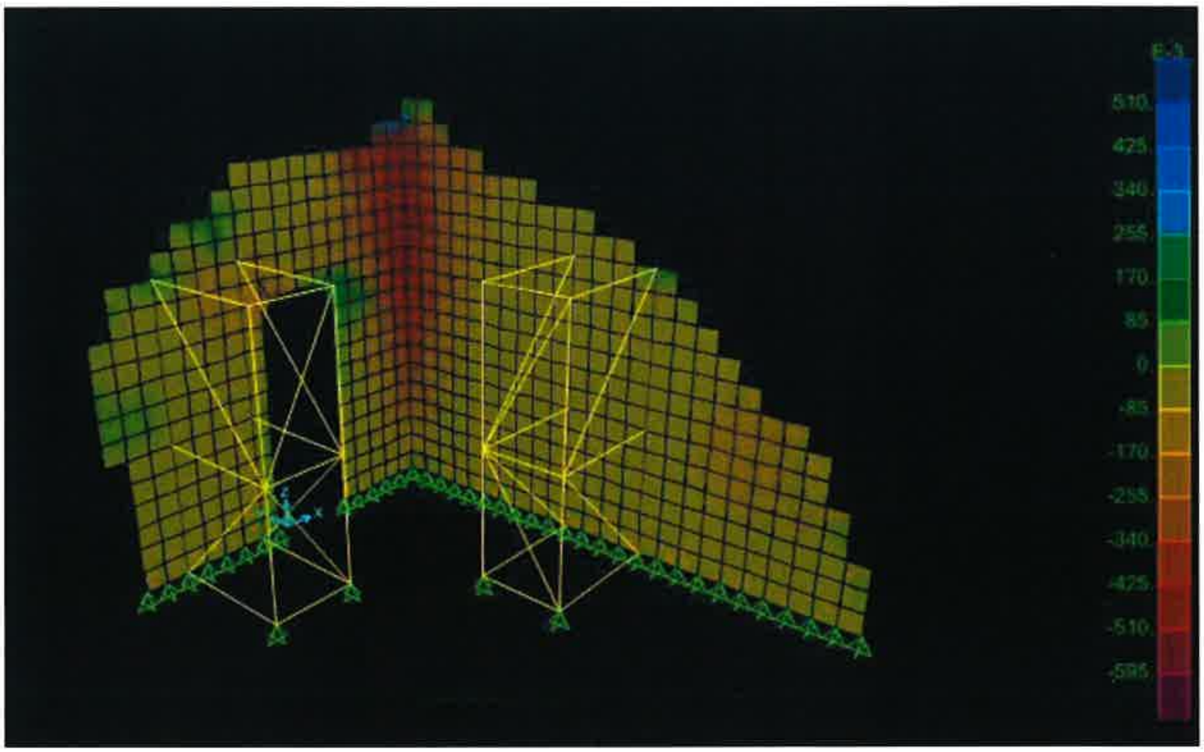


Figure 8 - Possible Rehabilitation Option with Timber Supports

6 Further Recommendations

Other than the two above mentioned structures on this site that in our opinion warrant retrofit to preserve the historical status of the old Smelter site, the majority of retaining rock walls and other subterranean structures that form the old foundry flume tunnels show extreme level of neglect. The nature of those walls, their type of construction and the level of deterioration is such that it would require major undertaking to prevent even a partial collapse of these structures, therefore allowing them to be fully accessed by public.

It is our recommendation that the City of Greenwood consider limiting public access to deteriorated structures on this historic site as well as to start at least immediate minimum retrofit of the old Foundry Stack and remaining Office Building walls in order to preserve them and reduce the risk to public due to potential structural deficiency issues.

The remaining rock walls and tunnels should be protected from further deterioration and depending on available funding, should be locally supported to prevent potentially abrupt collapse. It is our suggestion that most of them should be at least fenced with notes warning public clearly posted to prevent unauthorized access prior to remedial works being done.

All these structures should be referred to a detailed structural evaluation that would determine the exact level of retrofit and/or potential strengthening. Only a detailed evaluation of the existing masonry structures will enable development of a viable program that would provide a proper cost estimates.

We trust you will find this report in order. If you have any questions or require further information, please call the undersigned.

Sincerely,

DJ&T Engineering Ltd.

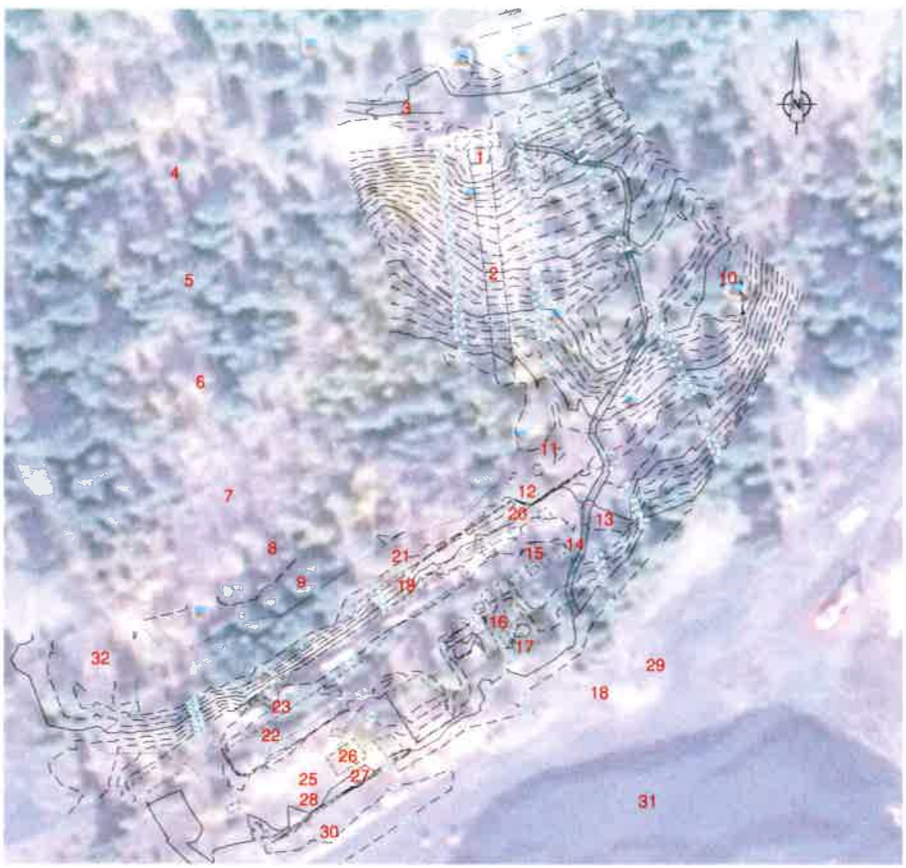
R. VULIC

29-Jun-2017

Ranko Vulic, P. Eng,
Principal Engineer

LEGEND

1 - MAIN STACK
2 - FLUME
3 - SPUR TO MOTHERLOAD MINE
4 - UPPER ORE BINS
5 - SCOOP CAR TRAMWAY
6 - ORE CONVEYOR
7 - SAMPLE MILL
8 - MAIN ORE BINS
9 - STORAGE COKE BINS
10 - BRIQUETTE MILL
11 - POWER HOUSE (PRIMARY TRANSFORMATION
12 - STORES
13 - BLOWER HOUSE (EMERGENCY STEAM POWER)
14 - POWER HOUSE (SECONDARY TRANSFORMATION)
15 - TRANSFORMER
16 - HEATING BOILER
17 - BLAST FURNACE BUILDING
18 - CONVERTER BUILDING
19 - SILICA MILL
20 - CONVERTER DUST CHAMBER
21 - COPPER SHED
22 - WORK SHEDS
23 - CPR SPUR (COPPER TRACK
24 - ASSAY OFFICE
25 - MANAGER'S QUARTERS
26 - TWIN CONVEYOR
27 - BRIQUETTE CONVEYOR
28 - FURNACE FEED TRACK
29 - FURNACE EXHAUST PIPE
30 - SLAG TRACK
31 - SLAG PILE
32 - POND
33 - PUMP HOUSE



PLAN - EXISTING SITE PLAN
SCALE: HORIZ. 1:1000

PRELIMINARY

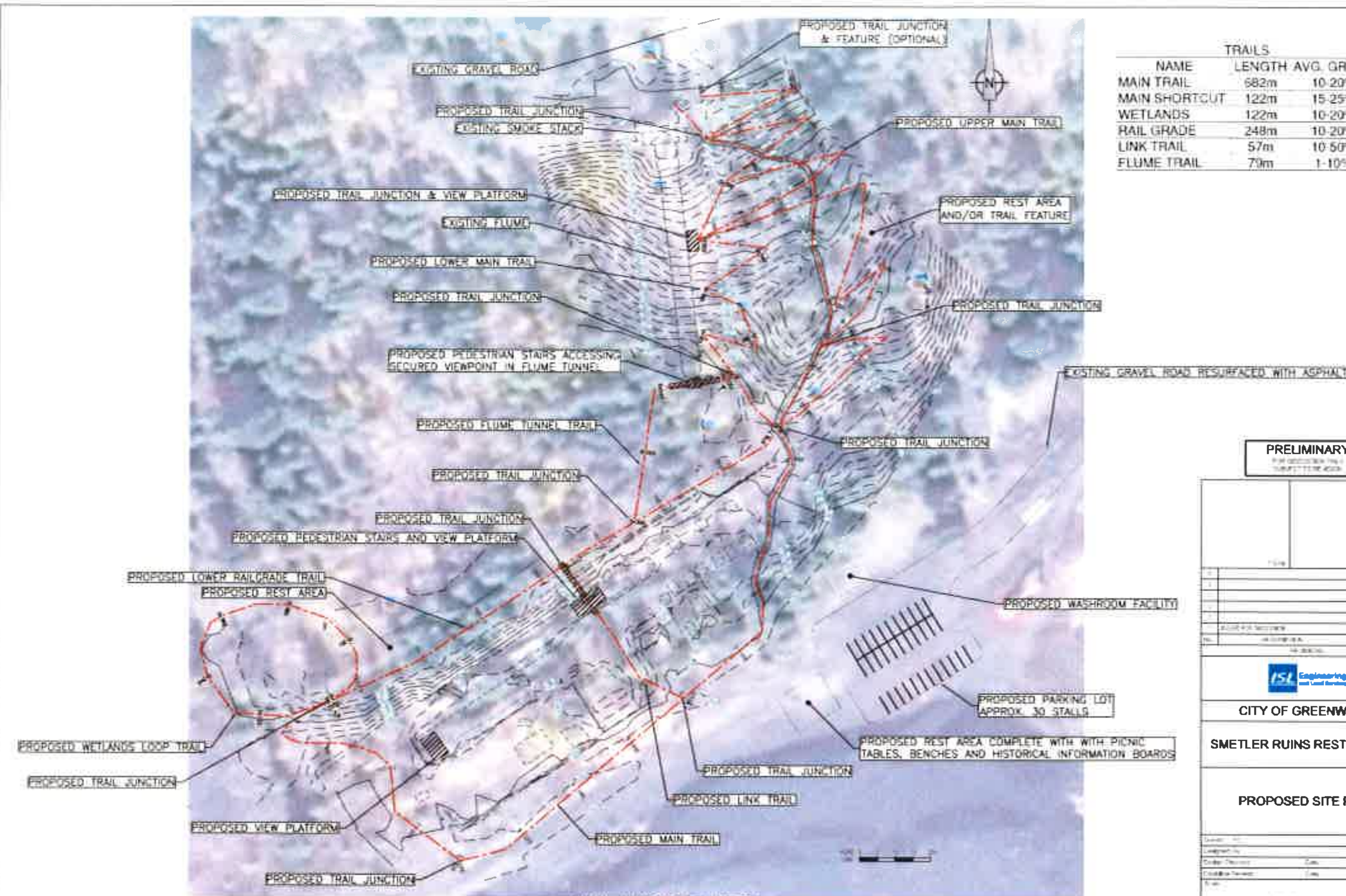
ISL Engineering

CITY OF GREENWOOD

SMETLER RUINS RESTORATION

EXISTING SITE PLAN

31814



TRAILS

NAME	LENGTH	AVG. GRADE
MAIN TRAIL	682m	10-20%
MAIN SHORTCUT	122m	15-25%
WETLANDS	122m	10-20%
RAIL GRADE	248m	10-20%
LINK TRAIL	57m	10-50%
FLUME TRAIL	79m	1-10%

PLAN - PROPOSED SITE PLAN
SCALE: HORIZ. 1:1000

PRELIMINARY

ISL Engineering and Land Services

CITY OF GREENWOOD

SMETLER RUINS RESTORATION

PROPOSED SITE PLAN

31814