CONCEPTUAL SPUD APPLICATION

PLANNING & DEVELOPMENT SERVICES
455 Mountain Village Blvd. Suite A
Mountain Village, CO 81435
970-728-1392
970-728-4342 Fax
cd@mtnvillage.org

Revised 1.3.20

The Planning & Development Services Department is here to assist you with your development application pursuant to the Community Development Code (CDC).

This publication outlines the Conceptual SPUD Development Application process of the CDC and also provides the submittal requirements for such development applications.

Contents of the Publication

This publication is intended to address the submittal requirements for a Conceptual SPUD Development Application consistent with the PUD Regulations. The Conceptual SPUD is the first of three required steps to create a SPUD as provided for in the PUD Regulations. Sketch PUD and Final PUD applications are also required in order to receive final SPUD approval from the Town Council. However, it is each applicant's responsibility to review the CDC and any associated regulations to ensure a full understanding of the development application process.

Development Review Process

Conceptual SPUD Development Applications shall be processed as a class 4 application as provided for in the CDC, with a Design Review Board (DRB) recommendation and Town Council approval. After any required worksession with the DRB and/or the Town Council, the class 4 development application process generally consists of the following steps:

Step 1:	**Pre-submittal Meeting with Applicant and Planning Division**
Step 2:	**Applicant Development Application Submittal**
Step 3:	**Planning Division Development Application Completeness Check**
Step 4:	**Planning Division Development Application Referral and Review**
Step 5:	**Planning Division Follow-up Communication**
Step 6:	**Applicant Plan Revisions**
Step 7:	**Planning Division Schedule Review Authority Public Hearing**
Step 8:	**Applicant Public Noticing (Minimum of 30 days prior to hearing)**
Step 9:	**Planning Division Preparation of Staff Report**
Step 10:	**Design Review Board (Recommendation) and Town Council Public Hearings**
Step 11:	**Review Authority Action**
Step 12:	**Planning Division Provides Notice of Action**
Step 13:	**Effective Date of Application Decision and Appeal**
Step 14:	**Length of Validity (12 months)**

Final SPUD Development Applications shall be concurrently processed with any required subdivision application. A separate density transfer application is not required for creating a SPUD. A Conceptual SPUD Application shall concurrently request a rezoning to the PUD Zone District that does not require a separate development application.

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Development Application Submittal Requirements:

The following forms, information and plans will need to be submitted in order to have a complete development application. Situations will occur when all of the listed submittal requirements will not be required and where items not listed as submittal requirements will be required in order for the Town to have sufficient information to fully evaluate the impacts of a development application. The Planning Division is therefore authorized to determine, based on the nature of a development application, whether to waive submittal requirements or require additional submittal requirements.

Submitted (Office Use)	Item No	Submittal Requirements
	1.	**Application Form.** Completed application form (Attached).
	2.	**Fees.** \$2,000 for 32 hours; hourly rate thereafter. The applicant and property owner are responsible for paying all Town fees as set forth in the fee resolution, and are also required by the CDC to pay for Town legal fees, the cost of special studies, and other fees as set forth in the CDC. Such fees are considered a condition precedent to having a complete development application, and shall be paid prior to the Town issuing the final approval.
	3.	**Proof of Ownership.** Copy of current deed or title report on the effected property.
	4.	**Agency Letter.** If application is not submitted by the owner of the property, a letter of agency, signed by the property owner giving permission to a firm or person to submit the requested land use application (Attached). A development application for a PUD may be filed only by the owner(s) of free title to all land to be included within such PUD or other person holding written consent thereto from the owner(s) of all land to be included in such PUD, or any combination thereof. No PUD may be approved without the written consent of the landowner(s) whose property is included in the PUD.
N/A	5.	**HOA Letter.** For development on property that is owned in common by a homeowners association, the development application shall include: **A.** A letter from the Homeowner's Association (HOA) board giving permission for the application (Attached), and, where a vote is required by the HOA governing documents, a copy of the proof of the vote and outcome of such vote. **B.** A copy of the HOA governing documents, including bylaws and declaration.
	6.	**Title Report.** Copy of current title report for the property listing all encumbrances.
	7.	**Development Narrative.** A written narrative of the development application that outlines the request. The narrative should include a summary of how the application meets the key requirements of the CDC, such as the applicable criteria for decision.
	8.	**Existing Condition Plan.** A stamped, monumented land survey prepared by a Colorado registered land surveyor showing existing site and surrounding access (driveway or roadway route, utility route, etc.) conditions drawn at a scale of 1" = 10' to a maximum of 1" = 30' showing the following information:
		A. Lot Size. Lot size needs to be shown.
		B. Existing Lot Lines. Existing platted lot lines need to be shown with distances, bearings and a basis of bearing. Existing property pins or monuments found and the relationship to the established corner also need to be shown.

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		C. **Existing Topography.** Existing topography needs to be shown with two foot contour intervals, including spot elevations at the edge of asphalt along any roadway or driveway frontage for the intended accessway at 25 foot intervals.
		D. **Steep Slopes.** Any slopes that are 30% or greater shall be mapped with a shaded or hatched pattern.
		E. **Wetlands, Ponds, Streams or Drainages (if any).** Wetlands, ponds, streams and drainages need to be shown. Recent wetland delineation by qualified consultant must be surveyed and shown on proposed site plan for United States Army Corps of Engineers approval. If wetlands are located adjacent to the development site, such wetland area also needs to be shown.
		F. **Easements.** Indicated all easements shown on the governing plats and recorded against the property.
		G. **Utilities.** All underground and above ground utilities and pedestals or transformers need to be shown.
		H. **Existing Improvements.** Any existing site improvements need to be shown, such as buildings (including driplines), drainage systems, trails (if part of official Town trail system as shown in the Comprehensive Plan), sidewalks, roadways, driveways, light poles and fences.
		I. **Fire Mitigation/Forestry Management.** A tree survey of all trees with a diameter at breast height of four inches (4") or greater shall be shown to ensure compliance with the fire mitigation and forestry management requirements.
	9.	**Proposed Development Plan.** The following information shall be submitted for the development application:
		A. **Conceptual Site Plan.** A conceptual site plan prepared by a qualified consultant (architect, engineer, planner, etc.) in accordance with the applicable regulations of the CDC (unless a variation is requested pursuant to the PUD Regulations) shall be submitted to show the proposed location of any roads, driveways, buildings, sidewalks, trails, parking areas, amenity areas, plaza areas, or other intended or required development.
		B. **Conceptual Grading Plan.** A conceptual grading plan prepared by a Colorado registered professional engineer showing how the project can meet the CDC roadway and driveway standards, grading and drainage design requirements and pedestrian connections, as applicable, with proposed grading shown with a solid line and spot elevations as needed.
		C. **Conceptual Building Elevations and Floorplans:** Conceptual architectural plans prepared by a Colorado licensed architect designed in accordance with the applicable regulations of the CDC (unless a variation is requested pursuant to the PUD Regulations) including but not limited to building elevations and floorplans with a scale of ¼" = 1' to 1/16" = 1' for larger scale projects.
		D. **Computer Massing Model.** A computer massing model with interactive viewing capability (360 degree rotation, fly by, etc.) showing the proposed buildings and

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		surrounding development to scale so the land uses and the visual impacts of the project can be evaluated pursuant to the CDC Comprehensive Plan project standards.
		E. Conceptual Landscaping Plan. A conceptual landscaping plan in accordance with the Landscaping Regulations shall be designed and prepared by an American Society of Landscape Architecture certified designer or a landscape professional with experience in creating and planting landscape plans in montane and subalpine life zones.
		F. Conceptual Infrastructure Plan. The conceptual PUD development shall include sufficient infrastructure designed by a Colorado registered professional engineer, including but not limited to vehicular and pedestrian access, mass transit connections, parking, traffic circulation, fire access, water, sewer and other utilities. i. **Conceptual Utility Plan.** A conceptual, composite utility plan showing the intended routes for providing water, sewer, electric, cable and telecommunications. ii. **Water and Fireflow.** For PUDs that require the extension of the Town's water system to serve additional lots for development, water supply and fire flow information shall be provided in accordance with the Fire Code. iii. **Evidence of Adequate Water, Sewage Disposal and Utilities.** The applicant shall consult with the director of the Public Works Department, San Miguel Power Association and Source Gas prior to the submission of a development application to include statements from such agencies in the application on the availability of utilities to serve the intended development. iv. **Conceptual Access Plan.** A conceptual access plan providing access to and from the site of the PUD shall be provided, including any needed infrastructure improvements as may be required by the Subdivision Regulations and the Road and Driveway Standards.
N/A	10	**Geotechnical Report.** A geotechnical report prepared by Colorado registered professional engineer or geologist shall be provided for all sites that have never been platted and zoned for development, such as a lot that is zoned for open space that is now intended for development as envisioned in the Mountain Village Comprehensive Plan.
	11	**Proposed Plat.** A draft of the proposed subdivision plat that includes all required plat elements pursuant to the Subdivision Regulations, such as proposed lot lines, easements, rights-of-way, subdivision name, road names, scale (minimum scale is 1" = 20'), north arrow, proposed lot numbering, proposed lot size, title block and legend.
N/A	12	**Practicable Alternatives Analysis:** For development proposing disturbance to wetlands, the general easement or slopes greater than 30%, the Town may require an applicant prepare a practicable alternatives analysis to demonstrate why it is not practicable to avoid such areas.
	13	**Plan Set Sheet Requirements.** All plans sets as set forth in these submittal requirements shall be formatted to have a sheet size of 24" X 36", with cover sheet providing the

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Submitted (Office Use)	Item No	Submittal Requirements
		contact information of all plan consultants, vicinity map, and sheet index; and all sheets showing date of original plan preparation and all revision dates, sheet labels and numbers, borders, title blocks, project name, lot number, address and legends. **A.** All plans submitted by a Colorado licensed architect, surveyor, geologist or interior designer shall be electronically stamped and signed without a locked signature to allow for commenting on the plan sets.
not required at this step	14	**Proposed PUD Development Agreement.** A proposed PUD development agreement setting forth, at a minimum, the proposed permitted uses, density, maximum building height and massing, zoning designations, CDC and Design Regulations variations, PUD community benefits, rezonings, density transfers, subdivisions, requirements for the construction of any public improvements and facilities, timetable and schedule of development, phasing requirements and conditions, any proposed conditions of approval and a statement establishing a vested property right.
	15	**ePlan Submittal.** All development applications shall be submitted pursuant to the ePlans submittal process as outlined in the follow publication: https://townofmountainvillage.com/media/ePlans-Electronic-Submittal-and-Review.pdf

Questions and/or comments on ePlans Process can be directed to cd@mtnvillage.org or call 970-728-1392

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TOWN OF MOUNTAIN VILLAGE FEE REQUIREMENTS ACKNOWLEDGEMENT

The Town of Mountain Village requires specific fees to be paid with a development application including legal and attorney fees associated with processing land development applications, inquiries and review. Please read and acknowledge the below fee requirement which are found at Community Development Code Section 17.4.4. General Provisions Applicable to All Development Application Classes, Section L. Fees.

L. Fees

1. Fee Schedule. The Town Council shall, from time to time, adopt a fee resolution setting forth all development application fees and associated permit fees. Fees for submittals not listed in the fee schedule resolution shall be determined by the Director of Community Development on a case-by-case basis determined by the similarity between the submittal and the development applications listed on the fee schedule together with the estimated number of hours of staff time the review of the submittal will require. No development application shall be processed, nor any development or building permits shall be issued until all outstanding fees or moneys owed by the applicant, lot owner, developer or related entity, as defined by the Municipal Code, to the Town, in any amount for any purpose, including but not limited to any fees, delinquent taxes, required Town licenses, permit fees, court fines, costs, judgments, surcharges, assessments, parking fines or attorney's fees are paid to the Town.

2. Town Attorney Fees. The applicant shall be responsible for all legal fees incurred by the Town in the processing and review of any development application or other submittal, including but not limited to any Town Attorney fees and expenses incurred by the Town in the legal review of a development application together with the legal review of any associated legal documents or issues. Legal expenses so incurred shall be paid for by the applicant prior to the issuance of any permits.

3. Property or Development Inquiries. The Town requires that Town Attorney legal fees and expenses be paid for all development or property inquiries where a legal review is deemed necessary by the Town. The developer or person making the inquiry, whichever the case may be, shall be informed of this obligation and execute a written agreement to pay such legal expenses prior to the Town Attorney conducting any legal review. A deposit may be required by the Director of Community Development prior to the commencement of the legal review.

4. Other Fees. The applicant shall be responsible for all other fees associated with the review of a development application or other submittal conducted by any outside professional consultant, engineer, agency or organization and which are deemed necessary by the Town for a proper review.

5. Recordation Fees. The Community Development Department will record all final plats, development agreements and other legal instruments. The applicant shall be responsible for the fees associated with the recording of all legal instruments.

I have read and acknowledge the fee requirements associated with my application.

MERRIMAC FORT PARTNERS, LLC

DocuSigned by:

E1CE1593BE08449 — 12/23/2021

(signature required) — (date)

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APPLICANT INFORMATION			
Name: MERRIMAC FORT PARTNERS, LLC MERRIMAC FORT PARTNERS, LLC		**E-mail Address:** dale@merrimacventures.com	
Mailing Address: 2434 E Las Olas Blvd		**Phone:** 954-591-6272	
City: Fort Lauderdale	**State:** FL		**Zip Code:** 33301
Mountain Village Business License Number: 000186			
PROPERTY INFORMATION SEE NARRATIVE			
Physical Address:		**Acreage:**	
Zone District:	**Zoning Designations:**	**Density Assigned to the Lot or Site:**	
Legal Description:			
Existing Land Uses:			
Proposed Land Uses:			
OWNER INFORMATION SEE NARRATIVE			
Property Owner:		**E-mail Address:**	
Mailing Address:		**Phone:**	
City:	**State:**		**Zip Code:**
DESCRIPTION OF REQUEST SEE NARRATIVE			

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OWNER/APPLICANT ACKNOWLEDGEMENT OF RESPONSIBILITIES

I,(*Insert property owner name*) TSG SKI & GOLF COMPANY, LLC (the "Owner")

The owner of Lot (*Insert Lot Number*) 67, 69R-2, 71R, OSP-3Y (the "Property") and, if applicable,(*Insert agent's name*) MERRIMAC FORT PARTNERS, LLC

(the "Agent") of (*Insert Agent's company*)

hereby certify that the statements made by the Owner, Agent and their consultants on this application are true and correct to the best of TSG's knowledge. We acknowledge that any misrepresentation of any information on the application submittal may be grounds for denial of the development application or the imposition of penalties and/or fines pursuant to the Community Development Code. We have familiarized ourselves with the rules, regulations and procedures with respect to preparing and filing the development application. We agree to allow access to the proposed development site at all times by members of Town staff, DRB and Town Council. We agree that if this request is approved, it is issued on the representations made in the development application submittal, and any approval or subsequently issued building permit(s) or other type of permit(s) may be revoked without notice if there is a breach of representations or conditions of approval. By signing this acknowledgement, we understand and agree that ~~we are~~ the Applicant is responsible for the completion of all required on-site and off-site improvements as shown and approved on the final plan(s) (including but not limited to: landscaping, paving, lighting, etc.). We further understand that ~~we are~~ the Applicant is responsible for paying Town legal fees and other fees as set forth in the Community Development Code.

12/23/21

Signature of Owner *TSG SKI & GOLF COMPANY, LLC* Date

DocuSigned by:

12/23/2021

Signature of Applicant/Agent *MERRIMAC FORT PARTNERS, LLC* Date

OFFICE USE ONLY	
Fee Paid:	**By:**
	Planner:

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OWNER AGENT AUTHORIZATION FORM

I have reviewed the application and hereby authorize (*Insert agent(s) name*) MERRIMAC FORT PARTNERS, LLC

of (*Insert agent's business name*) ______

to be and to act as my designated representative and represent the development application through all aspects of the development review process with the Town of Mountain Village, with respect to processing the Conceptual SPUD Application re: the Pond lots, through the Town's process.

TSG SKI & GOLF ~~COMPANY~~, LLC

(Signature)

12-23-21
(Date)

~~Chad Horning~~ M.C. Horning, Jr., Manager
(Printed name)

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OWNER/APPLICANT ACKNOWLEDGEMENT OF RESPONSIBILITIES

I,(*Insert property owner name*) CO LOT 161C-R MOUNTAIN VILLAGE, LLC (the "Owner")

The owner of Lot (*Insert Lot Number*) 161C-R (the "Property") and, if applicable,(*Insert agent's name*) MERRIMAC FORT PARTNERS, LLC

(the "Agent") of (*Insert Agent's company*)____________________

hereby certify that the statements made by the Owner, Agent and their consultants on this application are true and correct. We acknowledge that any misrepresentation of any information on the application submittal may be grounds for denial of the development application or the imposition of penalties and/or fines pursuant to the Community Development Code. We have familiarized ourselves with the rules, regulations and procedures with respect to preparing and filing the development application. We agree to allow access to the proposed development site at all times by members of Town staff, DRB and Town Council. We agree that if this request is approved, it is issued on the representations made in the development application submittal, and any approval or subsequently issued building permit(s) or other type of permit(s) may be revoked without notice if there is a breach of representations or conditions of approval. By signing this acknowledgement, we understand and agree that we are responsible for the completion of all required on-site and off-site improvements as shown and approved on the final plan(s) (including but not limited to: landscaping, paving, lighting, etc.). We further understand that we are responsible for paying Town legal fees and other fees as set forth in the Community Development Code.

DocuSigned by:
64800748ADDA4C9 — 12/19/2021

Signature of Owner *CO LOT 161C-R MOUNTAIN VILLAGE, LLC* Date

DocuSigned by:
E1CE1593BE00449 — 12/23/2021

Signature of Applicant/Agent *MERRIMAC FORT PARTNERS, LLC* Date

OFFICE USE ONLY	
Fee Paid:	**By:**
	Planner:

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OWNER AGENT AUTHORIZATION FORM

I have reviewed the application and hereby authorize (*Insert agent(s) name*) ______________________

MERRIMAC FORT PARTNERS, LLC

of (*Insert agent's business name*) ______________________

to be and to act as my designated representative and represent the development application through all aspects of the development review process with the Town of Mountain Village.

CO LOT 161C-R MOUNTAIN VILLAGE, LLC

DocuSigned by:

J R Royer

64800746ADDA4C9

(Signature)

12/19/2021

(Date)

James R. Royer, Manager

(Printed name)

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NOT APPLICABLE

HOA APPROVAL LETTER

I, *(Insert name of HOA president)* ______________________________

the HOA president of the property legally described as *(Insert legal description from condominium map, townhouse plat or other common interest community)* ______________________________

provide this letter as written approval of the plans dated *(Insert date on plan set)* ______________

that have been submitted to the Town of Mountain Village Planning & Development Services Department for the proposed improvements to be completed at the address noted above. I understand that the proposed improvements include: *(Insert description of development improvements below)*

______________________________ (Signature)

______________________________ (Date)

______________________________ (Title)

NARRATIVE

CONCEPTUAL SPUD REVIEW
LOTS 161CR, 67, 69R-2, 71R and OS-3Y

JOINT TOWN COUNCIL AND DESIGN REVIEW BOARD REVIEW

APPLICANT: MERRIMAC FORT PARTNERS, LLC

SUBJECT PROPERTY: LOT 161C-R

LOTS 67, 69R-2, 71R, OS-3Y

CURRENT ZONE DISTRICT: VILLAGE CENTER

CURRENT OPEN SPACE CLASSIFICATION: VILLAGE CENTER OPEN SPACE

CURRENT OWNERSHIP: LOT 161C-R: CO LOT 161C-R MOUNTAIN VILLAGE, LLC

LOTS 67, 69R-2, 71R, OS-3Y: TSG SKI & GOLF COMPANY, LLC

AGENCY ATHORIZATION:

CO LOT 161C-R MOUNTAIN VILLAGE, LLC AGENCY AUTHORIZATION CONTAINED IN APPLICATION FORM

TSG SKI & GOLF COMPANY, LLC AGENCY AUTHORIZATION CONTAINED IN APPLICATION FORM

TITLE COMMITMENTS:

LOT 161C-R ATTACHED HERETO AS ***EXHIBIT A***

LOTS 67, 69R-2, 71R and OS-3Y ATTACHED HERETO AS ***EXHIBIT B***

SUMMARY OF PROJECT AND DEVELOPMENT TEAM

DEVELOPER BACKGROUND

Merrimac Fort Partners, LLC (MFP) is currently under contract to purchase lot 161C-R from CO Lot 161CR Mountain Village, LLC and Lots 67, 69R-2, 71R and OS-3Y from TSG Ski & Golf Company, LLC. MFP is a joint venture between Merrimac Ventures, led by Managing Partner Dev Motwani, and Fort Partners, led by entrepreneur Nadim Ashi. Merrimac and Fort are partners on the Four Seasons Fort Lauderdale project and both have extensive track records of highly successful real estate and hospitality development, including the Four Seasons Surf Club, to date one of the most successful Four Seasons properties. Fort also owns the Four Seasons Palm Beach, the Four Seasons Brickell and is working on other Four Seasons projects internationally. Nadim, an accomplished skier, has been traveling to Telluride annually for the past 30 years with his family. Merrimac Ventures is an extremely active real estate development company, specializing in prime resort, mixed use and multi-family development. Merrimac is currently involved in over $3 billion in real estate development projects, including the 27-acre Miami World Center, one of the largest urban core developments in the United States.

ARCHITECTS

Olson Kundig: Design Architect

Philosophy & Principles
Since the firm's founding more than five decades ago, Olson Kundig has created a body of work that unites culture, nature, art and architecture. We create deliberate and evocative buildings that serve as bridges between people and their environments. We believe the design of great places begins by asking the right questions about a project's context and seeking a balance between the rational and the poetic.

Our ability to create appropriate and high-performance designs in varied cultures and climates across the globe stems from our contextual approach. We believe that all designs should be informed from the very start by research about a site's history, culture, climate and other environmental factors. Through this contextual research, buildings can be integrated thoughtfully with their surroundings, whether urban or rural. In our work, exterior and interior architecture work together cohesively, harmonizing with and taking inspiration from natural features of the site, as well as built and cultural histories.

For us, connecting to place often means collaborating with local craftspeople and artists. These partners help tell the story of the surrounding personal and cultural contexts of our buildings. We frequently work with local fabricators to develop specific building elements, and merge art and architecture to create a seamless spatial experience. The resulting designs possess a quiet, dramatic elegance that is born of collaboration and that inspires with its authenticity.

Mountain Architecture
Olson Kundig has decades of experience designing projects in extreme climates around the world. Our roots in mountain architecture trace to Tom's youth skiing and climbing, then to his formal architectural training and practice in Alaska and Switzerland. We have a deep appreciation for the mountains and that appreciation manifests in how we design, creating spaces that allow you to seek refuge from the cold, connect to the landscape around you and gain prospect views.

Our architecture seeks to highlight the unique qualities of each place. With a long history of working in Telluride we are familiar with its unique Western aesthetic and deeply rooted local community. We understand the opportunities and challenges of designing in Telluride, both from a community and technical standpoint, and will bring a new perspective to redefine and expand on the architecture of the Mountain Village Core.

OZ Architecture: Architect of Record

At OZ Architecture, we create the spaces and places where life happens. With roots from 1964, we value a pioneering spirit of innovation, an attitude of openness, collaboration and community stewardship. Across geographies, disciplines and project types, we design environments that endure time and precede trends. Places that push the boundaries to enhance the human experience and shape the built environment for the better.

PROJECT VISION

MFP is submitting this Conceptual SPUD Application for consideration to construct a five-star luxury branded resort and residences, with associated amenities, attracting an upscale family-oriented clientele, while providing additional services and amenities to the community. The project will consist of at least 50 traditional Hotel Rooms, branded Hotel Residences and Private Residences, a spa and fitness center, meeting facilities, après ski and restaurants. Furthermore, the Project will contain a wetlands riparian corridor walking trail, connecting the Gondola Plaza to the Village Pond Plazas, a publicly accessible plaza adjacent to the Gondola Plaza and an additional stairwell connection from the Project to Gondola Plaza. Rather than maximizing site coverage and density and overwhelming the site, the buildings have been carefully located to respect neighboring properties, create open space, view corridors and public areas. The intent is for the buildings to blend into the hillside more naturally. A five-star luxury hotel/resort brand or "flag" will operate and manage the resort and residences in accordance with the goals of the Town's Comprehensive Plan.

HOTEL AND HOTEL RESIDENCES

The Hotel and Hotel Residences are located adjacent to the Village Pond and behind the Le Chamonix and Heritage Plaza complexes. The Hotel and Hotel Residences consist of a lower, horizontal massing with the façade broken up into two masses: (i) the base and (ii) the upper volume that is further subdivided in plan at the shift in massing North and South. The top Hotel Residence penthouses will be set back so as to minimize their visual impact from the ground.

The base will be made of a substantial material, stone or cultured stone, as per the Design Regulations and will be more solid and weighted than the upper volume. The base will hold all public facing functions of restaurants, meeting rooms and the spa, and will provide much needed energy and activity to the Village Pond and associated plazas.

The upper volume, which will hold the Hotel and Hotel Residences, will be comprised of a frame that will be made of a more refined material that will be lighter in color and echoes the neighboring building's stucco facades. Screens and balconies will be incorporated into this mass to provide a layered and varied interplay of light and shadow both at night and day.

PRIVATE RESIDENCES

Further up the site, the Private Residences are broken up into two buildings to create separation which will minimize the massing and enable view corridors for neighboring properties. Much like the Hotel and Hotel Residences, the façade is broken up into two masses, the base and the upper volumes with the penthouses set back to minimize visual impact from the ground. The base will be the same material as the Hotel and Hotel Residences, creating a consistent material language that stitches the site and Project together. Much like the Hotel and Hotel Residences, the base will hold all public facing functions of lobby and amenity spaces.

LOBBY

Connecting the two separate program elements will be a single-story Lobby which will serve as a grand arrival point and provide circulation and connection among the Project components. The Lobby will be the jewel box of the Project and will have a distinct architectural expression. It will provide the port cochere for the Project and connect out into the auto-court on one side, while providing a dramatic backdrop and view towards the ski slopes as guests arrive.

INTEGRATED PARCEL FOR THE PROJECT.

In order to develop the Project and provide a high-quality luxury branded resort and experience, it is necessary to replat Lot 161CR with Lots 67, 69R-2, 71R, OS-3Y into one integrated parcel, Lot 161C-RR, consistent with the Town's SPUD Regulations and Comprehensive Plan.

This Application includes a request to incorporate approximately 0.487 acres of Village Center Open Space (OS-3BR2 and OS-3XRR) owned by the Town of Mountain Village into the replatted development parcel Lot 161C-RR, in order to provide sufficient land area in the vicinity of the wetlands and the Gorrono Creek riparian corridor to achieve the goals and public benefits set forth in the Town's Comprehensive Plan for Parcel D (Lots 67, 69R-2, 71R, OS-3Y) and Parcel F (Lot 161C-R) to create a public walking trail that emphasizes the natural features of the wetlands, Gorrono Creek and associated riparian corridors connecting the Village Pond and Heritage Plaza.

A summary of the current lots, parcels, their acreage, density and zoning is set forth in Table 1.

The Conceptual SPUD Plans submitted in this Application provide conceptual internal layout and configuration of the individual units, however, the exact unit counts and internal configurations will continue to be refined as the SPUD Plans progress through the SPUD process. We have included Table 2 as an example of proposed density unit counts and types for the replatted integrated Lot 161C-RR, however, the unit counts and types remain subject to change and further refinement as this SPUD Application moves through the Town process; provided, however, the Applicant shall provide at least 50 "traditional" Hotel Rooms, which will not be individually condominiumized and will remain under common ownership. Additionally, Applicant shall provide at least 35 branded hotel residences (70 lodge units) which shall be restricted to short term occupancy.

TABLE 1 ***CURRENT LOTS, PARCELS, ACREAGE AND DENSITY***

LOT/PARCEL	ZONING	ACREAGE	CONDOMINIUM UNITS	HOTEL EFFICIENCY UNITS	EMPLOYEE APARTMENT UNITS
161C-R	Village Center	2.84	33	2	
67	Village Center	0.12	14		
69R-2	Village Center	0.23	12		
71R	Village Center	0.17	9		1
OS-3Y	Village Center Open Space	0.587			
OS-3XRR	Village Center Open Space	2.726			
OS-3BR2	Village Center Open Space	1.969			
Total Current Density Units			**68 Units**	**2 Units**	**1 Unit**
Total Current Density Population (211 Persons)			**204 Persons (3 persons per unit)**	**4 Persons (2 persons per unit)**	**3 Persons (3 persons per unit)**

TABLE 2 CONCEPTUAL PROPOSED DENSITY

Project Units	Efficiency Lodge	Lodge Units	Lodge Units	Condominium Units
50 traditional Hotel Room	50 units			
37 Hotel Residences with lock-off units		74 units		
9 Hotel Residences without lock offs			9 units	
31 Private Residences				31 units
Density Population (180.25 persons) 50 Efficiency Lodge Units 83 Lodge Units 31 Condominium Units	**25 persons (0.50 persons per unit)**	**55.5 persons (0.75 persons per unit)**	**6.75 persons (0.75 persons per unit)**	**93 persons (3 persons per unit)**

SPUD APPLICATION COMPONENTS

1. REZONE AND DENSITY TRANSFER. The CDC and the Comp Plan require that parcels included within a SPUD Application be rezoned to the PUD Zone District. A separate Rezone and Density Transfer Application is not required. This Application includes a rezone of the parcels replatted into new Lot 161C-RR (discussed below) from the Village Center Zone District to the PUD Zone District. In addition, this Application proposes to rezone portions of Village Center Open Space to the PUD Zone District and to rezone and transfer both the number and types of density units allocated to the replatted Lot 161C-RR to and from the Town of Mountain Village Density Bank. Table 2 above sets forth conceptual density unit counts and types for the replatted integrated Lot 161C-RR, however, the units counts and types remain subject to change and further refinement as this SPUD Application moves through the Town process; provided, however, the Applicant shall provide at least 50 "traditional" Hotel Rooms. The final density unit counts and types will be achieved by a combination of rezoning of density allocated to the currently platted parcels, transfer of density from the Town's Density Bank to Lot 161C-RR and transfer of density from the currently platted lots to the Town's density bank. The density rezone and transfers will be detailed in the Sketch SPUD Application.

2. **SUBDIVISION/REPLAT.**

A. Replat Lot 161CR, Lot 67, Lot 69R-2 and OS-3Y into one integrated platted lot to be designated as lot 161C-RR.

B. Request replat of approximately 0.424 acres of OSP-3XRR and 0.063 acres of OS-3BR, zoned as Village Center Open Space and owned by the Town of Mountain, into proposed replatted Lot 161C-RR in order to provide sufficient area to create a public walking trail connecting Heritage and Village Pond Plazas and enhancement of the Gorrono Creek riparian corridor in accordance with the Comp Plan.

C. Lots 67, 69R-2 and 71 are designated as "Building Footprint Lots" under the CDC. The CDC and Comp Plan recognize the unique classification of Village Center Open Space under the 1999 San Miguel County Settlement Agreement and the 2012 Open Space Agreement between the Town and San Miguel County and does not require "replacement open space" be provided in connection with the rezoning and replatting of Village Center Open Space. CDC Section 17.3.4(H)(6)(a) allows an increase in the area of Building Footprint Lots by 25% as a matter of right. CDC Section 17.3.4(H)(6)(b) allows an increase in the area of Building Footprint Lots by more than 25% in connection with a PUD application.

D. A Subdivision Application will be submitted in connection with Sketch PUD Application to be processed concurrently with the SPUD Application.

2. **<u>DESIGN REVIEW</u>**. The SPUD Regulations do not require a separate Design Review Application be submitted with a SPUD application, rather Design Review of the SPUD shall be processed concurrently with the SPUD application components.

<u>TOWN OF MOUNTAIN VILLAGE COMPREHENSIVE PLAN</u>

In June 2011, the Town of Mountain Village adopted the "Mountain Village Comprehensive Plan" ("**<u>Comp Plan</u>**"). The Comp Plan is an advisory document that sets forth the *Mountain Village Vision* and a way to achieve the visions through principles, policies and actions. The Comp Plan is "intended to direct – the present and future- physical, social and economic development that occurs within the town and define the public interest and the public policy base for making good decisions."

In accordance with Colorado law, the Comp Plan is advisory and does not have the force and effect of law. While the Comp Plan itself does not have the force and effect of law, the Comp Plan specifically envisions that the Comp Plan can become part of the Town's laws by amendments to the Town's land use regulations. In 2013, the Town adopted the Community Development Code ("**<u>CDC</u>**"), which includes a requirement that certain land use applications must be in "general conformance" with the Comp Plan. As stated in the Comp Plan, when evaluating "general conformance" Town Council and DRB should "evaluate an application against the entirety of the goals, policies and actions contained in the Comp Plan and need not require compliance with every provision contained in the Comp Plan".

<u>MOUNTAIN VILLAGE CENTER SUBAREA</u>

The parcels included in this SPUD Application are located within the Mountain Village Center Subarea as depicted in the Comp Plan. The Village Center Subarea is intended to be the center of tourist accommodations, activity. The key policies, principles and goals incorporated into the Village Center Subarea are focused primarily on the development of hotbeds, flagship hotels and enhancing pedestrian connections throughout the Village Center. While not defined in the Comp Plan, the CDC defines "Hotbed Development" as development that provides lodging/accommodation type units that are available on a nightly basis for short-term rentals and which may be composed of Lodge Units, Efficiency Lodge Units and Hotel Units.

DEVELOPMENT TABLE

The Comp Plan includes a Development Table (Table 7) that intends to further the goal of providing hotbed development and sets forth various parameters for consideration for designated parcels. Per the Comp Plan, "the Development Table is not intended to set in stone the maximum building height or target density, and the applicant or developer may propose either a different density and/or a different height provided such density and height "fits" on the site per the applicable criteria for decision making for each required development review application."

In evaluating the Development Table for this SPUD Application, MFP strived to design a project that provides a flagship hotbed development that enhances the economic vibrancy of the Village Center, incorporates the components necessary for a high-quality luxury branded resort, while balancing the physical constraints of the site and respecting and complementing neighboring properties.

The Applicant interprets the target densities for Parcel D and Parcel F in the Development Table as maximum limits. The Applicant has spent a significant amount of time discussing the project layout and unit mix with flagship hotel brands and has proposed a unit mix and project design and layout for this specific property that meets the demanding standards of 5-star luxury hotel brands and meets the primary goal of the Village Center Subarea to provide a flagship hotel/resort. While this Application does not approach the maximum quantity of units envisioned by the Development Table, it does strike a balance between quantity and quality, with quality as the determinative factor in accordance with flagship brand standards.

PUBLIC BENEFITS TABLE

The Comp Plan includes a Public Benefits Table (Table 6) that sets forth proposals that emerged from the then sitting Town Council's review of the Comp Plan, but specifically contemplates that future Town Councils may change the proposed public benefits during a specific development review process. The Comp Plan envisions that provisions will be made for the proposed public benefits in connection with a PUD application for a Village Center Subarea Plan parcel listed in the Public Benefits table in connection with the evaluation of the application's "general conformance" with the Comp Plan.

The following table addresses the specific Public Benefits listed in the Comp Plan Public Benefits Table (Table 6) applicable to the parcels included in this SPUD Application (Parcel D and Parcel F) and establishes that the Application is in "general conformance" with the Public Benefits provisions of the Comp Plan.

PUBLIC BENEFIT TABLE ITEM #	APPLICANT'S RESPONSE
12. The owner of Parcel F 161-CR in the Mountain Village Center Subarea provides utility, vehicular access, and other needed infrastructure easement through Parcel F 161-CR toParcel G Gondola Station.	***Investigations and studies were conducted which determined that it was not feasible to provide vehicular access to Parcel G through Parcel F.*** ***In order to attract a 5- star luxury hotel/resort brand, the project site must be self-contained and free from disruption from other properties.***
13. TSG to provide utility, vehicular access and other needed infrastructure easement through Parcel D Pond Lots and ParcelG Gondola Station to Parcel F Lot 161-CR to facilitate vehicularaccess at a lower grade, with the goal of keeping the Gondola Plaza at one level grade as it is extended into Parcel F Lot 161-CR.	***Parcel D and Parcel F are proposed to be replatted into one integrated parcel, which facilitates vehicular access and continuity of the grade between the Gondola plaza and the project's plaza areas.***
14. TSG to provide utility, vehicular access and other needed infrastructure easement through Parcel D Pond Lots to Parcel E Le Chamonix to facilitate vehicular access to ParcelE Le Chamonix.	***It is necessary to replat Parcel D, Parcel F and adjacent open space into one integrated parcel in order to provide a site that is able to be developed to the standards required by 5-star luxury hotel/resort brands. It would not be feasible to incorporate vehicular access to Le Chamonix from Mountain Village Boulevard.***
15. Parcel F Lot 161-CR owner evaluates the technical feasibilityof establishing a public loading dock and trash collection facility. If a public loading dock and trash collection facilityis feasible, as determined by the town, Parcel F Lot 161-CR owner shall construct such facility and provide necessary delivery/access easements to and from the town's plaza areas.	***The standards required by 5-star luxury hotel/resort brands would not allow the incorporation of this type of facility into the project as it would negatively impact the standards and quality of experience demanded by luxury brands.*** ***The project includes a trash compactor which provides a benefit to the community by reducing the number of trips through the Village Center to service the project trash removal requirements.*** ***The project incorporates two parking spaces in the underground parking garage which will be conveyed to the Town. The parking spaces will be located near the gondola plaza and will provide parking for Town staff to access and service the gondola terminal.***

17. Provision of an enhanced riparian area along the west side ofParcel D Pond Lots and Parcel E Le Chamonix, and the east side of Parcel D Pond Lots with additional riparian planting,a footpath, benches and water features, with such stream-lined to the pond to prevent groundwater encroachment in Mountain Village Center. Create more natural creek drainageand a bridge north of Centrum at pond outlet.	***The project incorporates a public walking trail that extends from Heritage Plaza through the site to the Village Pond. The proposed trail and trail improvements, including a bridge, respect and compliment the natural riparian corridor and provide a unique public pedestrian experience within the Village Center. The trail integrates this unique riparian corridor into a unique connection between Heritage and Village Pond plazas. The trail includes a spur that departs the main trail between the Le Chamonix and Heritage buildings providing an additional pedestrian connection to the plaza. The Applicant will evaluate the feasibility of lining Goronno Creek in the Sketch SPUD Review.***

SITE SPECIFIC POLICIES.

The Comp Plan provides that development applications that require "general conformance" with the Comp Plan to address site-specific policies for designated parcels. This SPUD Application includes Village Center Subarea Parcel D and Parcel F. The following tables address the site-specific goals for each of Parcel D and Parcel F and establishes that the Application is in "general conformance" with the applicable site-specific policies of the Comp Plan.

PARCEL D (Lots 67, 69R-2, 71R, OS-3Y) SITE SPECIFIC POLICIES

SITE SPECIFIC POLICY
a. Encourage the owner of Parcel D Pond Lots to participate in good faith with the owners of the Parcel E Le Chamonix, Parcel F Lot 161-CR and Parcel G Gondola Station to develop the parcels together pursuant to an integratedand coordinated development plan with the goal of creating a large flagship hotel site utilizing the entirety of Parcel D Pond Lots. Parcel E Le Chamonix, Parcel F 161-CR and Parcel G Gondola Station consistent with the overall development and uses identified inthe Development Table. It is anticipated that the affected parcel owners could achieve the desired coordination by various means, including, without limitation: (1) a replat combining Parcel D Pond Lots, Parcel E Le Chamonix, Parcel F 161-CR and Parcel G Gondola Station to accommodate the entire project; (2) development of separate structures on each parcel in line with the development identified for each Parcel as noted in the Development Table, which development pods could be phased and would be tied together to address necessary and appropriate integrated operation and management requirements, as well as vehicular and pedestrian access,utility extensions, parking, mechanical facilities, loading docks, back of the house space, and similar areas not dedicated to residential or commercial uses and activities (common space). Costs and expenses for designing, constructing and operating common spaces would be fairly allocated between the parcels. The town will cooperate and assist the parcel owners in attempts to createa PUD or development agreement for Parcel D Pond Lots, Parcel E LeChamonix, Parcel F 161-CR and Parcel G Gondola Station that lays the foundation for a flagship hotel and for the mutually beneficial, combined and coordinated development of these parcels consistent with the policies of the Comprehensive Plan, which may involve the use of an independent third-party facilitator with extensiveexperience in land development and asset evaluation to facilitate the creation of a coordinated development plan for Parcel D Pond Lots, Parcel E Le Chamonix, Parcel F 161-CR and Parcel G Gondola Station. ***RESPONSE: The Application complies with this policy by proposing to replat Parcel D, Parcel F and adjacent open space into one integrated parcel in order to provide a coordinated development plan that meets the standards required for the development of a 5-star luxury flagship hotel/resort. The Applicant is under contract to purchase both Parcel D and Parcel F which will enable the seamless incorporation of the separate parcels into one integrated development parcel.***
SITE SPECIFIC POLICY

b. Determine if exchange land should be provided for any town-owned Mountain Village Center open space that is included in a development plan. ***RESPONSE: The Applicant requests the inclusion of approximately 0.487 acres of Village Center Open Space owned by the Town. The boundaries for Parcel D, as depicted on the Village Center Subarea Map in the Comp Plan, specifically includes this open space and is discussed in further detail under Site Specific Policy (C) below.***
SITE SPECIFIC POLICY c. Only allow for a rezoning of Mountain Village Center open space within Parcel D Pond Lots and conveyance of such open space from the town to the developer of Parcel D Pond Lots if such property provides a coordinated development plan through a PUD or development agreement with Parcel E Le Chamonix, Parcel F Lot 161-CR and Parcel G Gondola Station. ***RESPONSE: The Applicant is proposing a coordinated development plan that includes the entirely of Parcel D and Parcel F. Parcel D includes Village Center Open Space OS-3Y owned by TSG Ski & Golf, LLC and portions of Village Center Open Space OS-3XX owned by the Town. Village Center Open Space is not included within the acreage requirements for Open Space under the 1999 County Settlement Agreement and accordingly does not require the provision of replacement open space. Incorporation of the designated portions of OS-3XX AND OS-3BR2 owned by the Town will allow the developer to fully integrate the desired public trail connection between Heritage and Village Pond plazas and to enhance the Goronno Creek riparian corridor in accordance with Public Benefit #17 discussed above. Rezoning of Village Center Open Space is authorized under CDC Section 17.4.3(H).***
SITE SPECIFIC POLICY **d.** Determine if the current parking garage entry for Westermere can be legally and structurally used to access the parking for Parcel D Pond Lots, Parcel E Le Chamonix, Parcel F Lot 161-CR and Parcel G Gondola Station; consider positive and negative impacts of such access. ***RESPONSE: The Applicant explored this site-specific policy, however, due to the physical constraints of the Westemere parking garage it is not feasible to access the Project through this entry point. Common access would negatively impact the Westemere project and would not provide an arrival point that meets the standards of a 5-star luxury hotel brand.***
SITE SPECIFIC POLICY **c.** Determine the best alignment for Gorrono Creek through Parcel D Pond Lots to the pond and design a significantly enhanced landscaped, riparian corridor with a small crushed-gravel pedestrian trail and appropriate amenities, such as lighting and benches. Line Gorrono Creek through the site to minimize water intrusion into the surrounding parking garages and convey water below Village Creek. **RESPONSE: See Public Benefit #17 discussion above**. The Applicant will evaluate the proposal to line Gorrono Creek in connection with the Sketch SPUD Application.
SITE SPECIFIC POLICY d. Expand the pond, to the maximum extent possible, to create a recreational and landscaped amenity in Conference Center Plaza and provide a significantly improved amenity. Explore a boardwalk or plaza surface around the pond, the installation of a smalldock, and other pond recreational activities. Line the pond to prevent groundwater intrusion. Design the pond to retain a high-water quality and prevent foul water to the extentpractical. ***RESPONSE: The developer proposes to work with the Town to improve the Village Pond and associated plazas by contributing design services and financial contributions towards these public improvements.***
SITE SPECIFIC POLICY e. Create an open drainage swale with a more natural channel from the pond outlet to its current open channel, with a five foot wide pedestrian bridge and an landscape feature that lets the public interact with this creek area.

RESPONSE: See Public Benefit #17 discussion above
SITE SPECIFIC POLICY **f.** Explore the creation of a deck area next to the pond for restaurant and entertainment use. ***RESPONSE: The Project includes a spa near the Village Pond which will be open to the public and incorporates improvements and landscaping along the eastern edge of the Village Pond. Both the spa and the walking trail will provide much needed vibrancy, activity and vitalization of the Village Pond plazas.***
SITE SPECIFIC POLICY **g**. Design the building on Parcel D Pond Lots to be integrated into the existing, unfinished wall on Westermere to the extent allowed by town codes and legal agreements. **RESPONSE: The landscaping for the Project is intended to provide integration with the Westermere building.**

PARCEL F (Lot 161C-R) SITE SPECIFIC POLICIES

SITE SPECIFIC POLICY **a.** Site Specific Policy (a) are identical for both Parcel D and Parcel F. ***RESPONSE: The Application complies with this policy by proposing to replat Parcel D, Parcel F and adjacent open space into one integrated parcel in order to provide a coordinated development plan that meets the standards required for the development of a 5-star luxury flagship hotel/resort. The Applicant is under contract to purchase both Parcel D and Parcel F which will enable the seamless incorporation of the separate parcels into one integrated development parcel and common ownership.***
SITE SPECIFIC POLICY **b.** Determine the best alignment for Gorrono Creek through Parcel F Lot 161-CR to the pond and design a significantly enhanced landscaped, riparian corridor with a small crushed-gravel pedestrian trail and appropriate amenities, such as lighting and benches. Line Gorrono Creek through the site to minimize water intrusion into the surrounding parking garages and convey water below Village Creek. **RESPONSE: See Public Benefit #17 discussion above**.
SITE SPECIFIC POLICY **c**. Strive to keep the Gondola Plaza at the same level as it extends onto the new plaza onto Parcel F Lot 161-CR. Providing access from Parcel D Pond Lots to Parcel F Lot 161-CR by an underground garage may better enable this desired level plaza grade. ***RESPONSE: The replatting of Parcel D and Parcel F into one integrated development parcel enables the construction of an underground garage to serve the project. The grades of the plazas within the Project adjacent to Gondola Plaza are at a similar grade to the Gondola Plaza.***
SITE SPECIFIC POLICY **d.** Continue to provide parking and access for the Ridge project as required by legal agreements. ***RESPONSE: The Project has incorporated all parking and access facilities for the Ridge project as required under the 2019 Settlement Agreement that encumbers Lot 161C-R.***
SITE SPECIFIC POLICY

e. Provide the town ownership of any public areas on Gondola Plaza that extend out onto Parcel F Lot 161-CR through a condominium subdivision.

The Application proposes to provide publicly accessible plazas adjacent to Gondola Plaza as designated in the SPUD Conceptual Plans. The Gondola Plaza is owed by TSG Ski & Golf, LLC. The Town and TMVOA are the beneficiaries of an easement on Gondola Plaza. The developer proposes to provide an easement to the Town on the designated public plazas within the Project, which would be granted by the owners' association for the Project.

SITE SPECIFIC POLICY

f. Provide an easement for a town loading dock and trash facility to serve Mountain Village Center that also provides for multiple points of access to the plaza areas by a coordinated development plan with Parcel D Pond Lots, Parcel E Le Chamonix and Parcel G Gondola Station.

RESPONSE: It is not possible to incorporate this type of facility in the Project. These facilities would generate significant levels of activity and disruption during all hours of the day. It would not be possible to engage a 5-star luxury flagship brand if this type of facility was required to be included within the Project.

SITE SPECIFIC POLICY

g. Strive to provide a significant viewshed for Lot 97 across Parcel F-1 to the extent practical. Development should consider protecting Parcel F-1 from development.

RESPONSE. The Conceptual SPUD Plans demonstrate the efforts to provide viewsheds for Lot 97. No vertical improvements are proposed for Parcel F1. This was primarily accomplished by creating two separate buildings which provide strategic separation between the buildings in order to preserve Lot 97's view corridor. Additionally, we met with the owner of Lot 97 and consulted with him throughout design to preserve his views. In order to accomplish this goal and meet the other requirements of the Project required by a luxury flagship hotel brand it is necessary to increase the height of each private residence building so the footprints of the buildings do not intrude into Parcel F1.

SITE SPECIFIC POLICY

h. Provide any parking and access and other facilities for the Ridge project as may be required by legal agreements.

RESPONSE: The Project has incorporated all parking and access facilities for the Ridge project as required under the 2019 Settlement Agreement that encumbers Lot 161C-R.

SPUD CRITERIA AND STANDARDS.

In addition to achieving "general conformance" with the Comp Plan, the CDC sets forth specific criteria and standards for SPUD applications. These criteria and standards have been incorporated into the Conceptual SPUD Plans submitted with this Application and are discussed in further detail below. These criteria and standards will be addressed in further details as the Conceptual SPUD Plans are refined through the SPUD Process.

CDC SECTION 17.4.12.E CRITERIA FOR DECISION

G. Criteria for Decision

The following criteria shall be met for the review authority to approve a rezoning to the PUD Zone District, along with the associated PUD development agreement:

1. The proposed PUD is in general conformity with the policies, principles and standards set forth in the Comprehensive Plan;

Response: The PUD generally conforms with the policies, principles and standards set forth in the Comprehensive Plan as discussed in detail above.

2. The proposed PUD is consistent with the underlying zone district and zoning designations on the site or to be applied to the site unless the PUD is proposing a variation to such standards;

Response: The parcels included in this SPUD Application are located in the Village Center Zone District. This Application complies with the Village Center District standards, except as specifically identified in the requests for variances and/or variations discussed in further detail below.

3. The development proposed for the PUD represents a creative approach to the development, use of land and related facilities to produce a better development than would otherwise be possible and will provide amenities for residents of the PUD and the public in general;

Response: The replatting of Parcel D and Parcel F into one integrated parcel provides sufficient land area to allow the developer to provide a development plan and project that meets the demanding standards of 5-star luxury hotel brands. The increase in land area allows the project components to be disbursed on the site and provides amenities for the PUD residents and additional amenities that are available for use by both the PUD residents as general public such as a spa, restaurants and plaza areas. While the CDC allows for 100% lot coverage, the developer creatively used height to disburse the buildings on the site to preserve major view corridors and to create light and space as opposed to a single monolithic slab structure. The proposed project utilizes height where it is required to preserve significant open space, allowing for extensive open areas on the site. Furthermore, the developer is utilizing a creative approach to the plaza area between the buildings, using a landscaping approach which will bring the fauna and terrain of the surrounding mountain cascading through the plaza, combining rock, water and plant life to create an amazing mountain oasis.

4. The proposed PUD is consistent with and furthers the PUD purposes and intent;

Response: Further detail to be provided in the Sketch SPUD application pursuant to 17.4.12.D.1(b)

5. The PUD meets the PUD general standards;

Response: The project is consistent with the General Standards set forth in CDC Section 17.4.12.I. All fee title owners of the contiguous real property included in the application have provided written consents. The density for the project is greater than 10 units. Density will be transferred from Density Bank Certificates #38 and #42. Landscaping and public spaces are included in the project and create an attractive and welcoming environment for the project, as well as surrounding properties and the Village Center. The project will include sufficient infrastructure to serve the project. In addition, enhanced pedestrian walkways and access through the Village Center plazas are integrated into the project. The project will not be phased.

6. The PUD provides adequate community benefits;

Response: Please see the detailed discussion regarding community public benefits below.

7. Adequate public facilities and services are or will be available to serve the intended land uses;

Response: Adequacy of public facilities and services have been verified with the Town and utility providers.

8. The proposed PUD shall not create vehicular or pedestrian circulation hazards or cause parking, trash or service delivery congestion; and

Response: The proposed PUD dramatically improves pedestrian circulation, creating a wetlands walking trail to connect the Gondola Plaza to the Village Pond Plaza. Additionally, it provides a and additional stair connection to the Gondola Plaza to ease pedestrian traffic up the existing stairs to the Gondola Plaza from Heritage Plaza. Lastly, trash and service deliveries will be made to the far northern corner of the project and will be fully enclosed and will include an internal trash compactor. Vehicular traffic to the project is routed off of Mountain Village Blvd and queued internal to the property.

9. The proposed PUD meets all applicable Town regulations and standards unless a PUD is proposing a variation to such standards.

Response: The PUD is consistent with the Town's regulations and standards but is seeking the variances and variations identified in this narrative.

<u>**CDC SECTION 17.4.12.H COMPREHENSIVE PLAN**</u>

H. Comprehensive Plan Project Standards

Each <u>**final**</u> SPUD or MPUD plan shall include specific criteria and requirements to satisfy the following Comprehensive Plan project standards:

1. Visual impacts shall be minimized and mitigated to the extent practical, while also providing the targeted density identified in each subarea plan development table. It is understood that visual impacts will occur with development.

Response: Developer has made every effort to minimize visual impacts. This project will be an iconic architectural structure; however, the west building is comparable in mass and scale to the neighboring properties in the Village Center, allowing for a smooth transition between structures. Furthermore, Developer has studied the visual impact of the site from Heritage Plaza and designed in a way to minimize the views of the project. Lastly, the Private Residences buildings have been recessed from the lot lines to provide spacing from the neighbors and to improve the view corridors. Rather than maximizing density, the developer has designed a project that will minimize visual impact while accomplishing appropriate density necessary for a 5-star luxury hotel brand to be developed.

2. Appropriate scale and mass that fits the site(s) under review shall be provided.

Response: See response to #1

3. Environmental and geotechnical impacts shall be avoided, minimized and mitigated, to the extent practical, consistent with the Comprehensive Plan, while also providing the target density identified in each subarea plan development table.

Response: Developer has engaged geotechnical and environmental experts who are intimately familiar with the Town of Mountain Village and the subject sites. Developer will actually be improving the existing wetlands as part of its plan.

4. Site-specific issues such as, but not limited to the location of trash facilities, grease trap cleanouts, restaurant vents and access points shall be addressed to the satisfaction of the Town.

Response: Further detail to be provided in the Sketch SPUD application pursuant to 17.4.12.D.1(b). Trash facilities are located at the far northern end of the main structure and internal to the building and will include a trash compactor.

5. The skier experience shall not be adversely affected, and any ski run width reductions or grade changes shall be within industry standards.

Response: The project will have no adverse impact on ski runs.

<u>CDC SECTION 17.4.12.G PUD COMMUNITY BENEFITS</u>

G. PUD Community Benefits

1. One or more of the following community benefits shall be provided in determining whether any of the CDC requirements should be varied or if the rezoning to the PUD Zone District and concurrent (for SPUD) or subsequent (for MPUD) rezoning, subdivision, or density transfer request should be granted for a PUD:

a. Development of, or a contribution to, the development of public benefits or public improvements, or the attainment of principles, policies or actions envisioned in the Comprehensive Plan (unless prohibited under number 2 below), such as benefits identified in the public benefit table.

RESPONSE:

The SPUD Regulations require SPUD applications to provide adequate “community benefits.”

Community Benefits are defined in the CDC as follows:

“The dedications, conveyances, public improvements, exactions and conditions required to ensure that the impacts of a development project are adequately mitigated. Community benefits include, without limitation: additional affordable or employee housing; conveyance of land or easements for public purposes; construction and/or land, material or financial contribution to the construction of public facilities, such as public parking and transportation facilities, pedestrian improvements, streetscape improvements, lighting, public cultural facilities, parks, conference centers, public buildings and features; and other public facilities determined by the Town Council to meet the requirement for community benefit as set forth in the PUD Regulations."

The Comp Plan includes a Public Benefits Table (Table 6) that sets forth specific Public Benefits desired for Parcel D and Parcel F. The Public Benefits Table has been discussed in detail above.

In addition to the Public Benefits discussed above, this SPUD Application provides the following Community Benefits that support the rezoning, subdivision, density transfers, variances and variations requested in this Application:

A. Publicly accessible plaza areas connecting to the public Gondola Plaza and provision of additional amenities for skier and public use, including a proposed restaurant and seating areas. The plaza will be extensively planted to maintain the natural landscape as it flows through the site.

B. Enhancement of and incorporation of the existing wetlands into a lush, wetlands walking trail 6 feet in width connecting the Pond/Convention Center Plazas to Heritage Plaza and the Gondola Plaza.

C. A fixed financial contribution to the Town for revitalization of and improvements to the Village Pond area and adjacent plazas, including pedestrian circulation around the western edge of the Pond, allowing for more intensive improvements and plantings on the eastern edge and connecting the wetlands walking trail from the Pond/Convention Center Plaza to Heritage/Gondola Plaza.

C. Improvements to alleyway between Tracks and the Gondola station, creating a more pedestrian friendly connection between Heritage Plaza, the wetlands trail and a stairwell access to the Gondola Plaza and station.

D. Conveyance of two deeded parking spaces within the project's underground parking garage to the Town to be used by Town staff in connection with gondola operations.

E. A fixed financial contribution to the Town for Employee Housing to be determine in connection with processing of this SPUD Application and adoption of the Town's pending employee housing regulations.

G. Construction of 36 dedicated parking spaces for owners within The Ridge at Telluride development.

H. Construction of a loading/unloading zone for the owners within The Ridge at Telluride development.

I. Construction of an additional stair access to the Gondola Plaza to facilitate new pedestrian circulation routes through the Project, to and from the Village Pond Plazas and to facilitate access from the parking spaces provided for the owners within the

Ridge at Telluride. This additional stair access will reduce pedestrian and skier congestion on the sole existing stair access to the Gondola Plaza.

G. Construction of a trash compacting facility within the project which will reduce the number of trips over Mountain Village Boulevard by large trash removal trucks and equipment.

H. Incorporation of snowmelt within the Project's plaza areas and the roofs of the buildings in order to minimize the amount of snow shedding and snow removal from the project and reduce the number of trips over Mountain Village Boulevard by large trucks and snow removal equipment.

VARIANCES REQUESTED

A. **Building Height Limits (CDC 17.3.11 and 17.3.12)**

For the Village Center, the CDC limits the maximum building height to 60' and the maximum average building heights to 48'. However, the Mountain Village Comprehensive Plan, last edited on February 15, 2018, establishes the target max building height to 78.5' for the Pond Lots and 95.5' for Lot 161C-R. The proposed development currently exceeds the limitations set forth in the CDC but falls within the target values stated in the MVCP. The Developer has intentionally placed buildings on the site so as to maximize view corridors and open space, while minimizing the impact to neighbors and the views from Heritage Plaza.

B. **Condominium-Hotel Regulations (CDC 17.6.3)**

Waiver of the Condominium-Hotel Regulations.

The Project will consist of Hotel Rooms and Hotel Residences which will be operated by a 5-star luxury hotel brand operator and will be managed in accordance with the standards and criteria required by the flagship operator.

The Hotel Rooms will be restricted from being individually condominiumized and will remain as one block of Hotel Rooms, which will remain in common ownership and will carry the short-term rental restrictions in accordance with the definition of Efficiency Lodge Units. The Hotel Residences will be a mix of Lodge Units including lock-off units.

CDC AND DESIGN REGULATION WAIVERS AND VARIATIONS

The Conceptual SPUD plans are in general conformance with the specific design regulations in the CDC; provided, however, that since this Application is currently at the Conceptual SPUD Review stage, the SPUD plans are conceptual and will be further refined as this Application moves through the SPUD process.

Building Design (CDC 17.5.6)

The building design generally complies with CDC 17.5.6, exemplifying a simplified form, grounded base, and materiality that reflects the surrounding architectural and natural language. Variations are requested for the following design elements:

Roof design (CDC 17.5.6 C1) - Request to go with a simplified and clean shed roof design in lieu of emphasized sloped planes, varied ridgelines, and vertical offsets.

Roof Material (CDC 17.5.6 C3) – Roof material may also be requested as a variation. Consideration will be given to the visibility of the roof from the ski hill, and to adjacent roofing materials. Material selection will be presented to the DRB in the Sketch SPUD application pursuant to 17.4.12.D.1(b)

Glazing Variance (CCDC 17.5.6.G) - Request to exceed the 40% maximum window area of the total building facade. The building will include wood screening elements (see elevations and renderings) that will emphasize a relationship of solid and void that is appropriate to the contextual architecture and building typology.

Decks and Balconies Variance (CCDC 17.5.6.I) – The building design utilizes semi-continuous balconies which are variegated in scale and rhythm by screening wood elements. These balconies emphasize views and solar exposure per CDC guidelines.

Lighting regulations (CDC 17.5.12)

The proposed development intends to comply with the Lighting regulations. Including, as noted, a separate variation for Section 17.1.11(E)(5), Section 17.5.12(A) and the Lighting Design Requirements provided at Section 17.5.12(F) during the building-specific design review process.

Parking regulations (CDC 17.5.8)

Parking will be addressed in greater detail as part of the Sketch SPUD Application. A total of 137 parking spaces will be required based on the following requirements from CDC 17.5.8 Table 5-2:

- 31 Condominiums at a 1.0 ratio = 31 spaces
- 50 Hotel Rooms(Efficiency Lodge and Lodge Units) at a 0.5 ratio = 25 spaces
- 46 Residences (83 Lodge Units) at a 0.5 ratio = 42 spaces
- 6,024 Restaurant Space (high intensity) @ 1 space/500 SF = 13 spaces
- 6,829 Spa/Pool/Fitness (low intensity) @ 1 space/1,000 SF = 7 spaces
- ***Total required = 118 spaces***
- Additional 36 Ridge Parking Spaces (not required for the proposed project, but required under the Settlement Agreement)
- Additional 2 parking spaces for the Town per public benefits above
- Total of 156 parking spaces

The current design submittal includes:

- 75 Spaces provided for condominium units
- 80 Spaces provided for Hotel/Lodge Units and Commercial parking

- 36 Spaces provided for Ridge Residents
- 2 spaces provided for the Town
- 58 Spaces provided for Hotel Operations
- Total provided = 251 spaces

Tandem parking spaces, where indicated in plan, shall be either valet parked or organized in the manner described in CDC 17.5.8 C.7, subject to review authority authorization as noted.

Density (CDC 17.3.7 and 17.3.8)

Discussed in further detail above and subject to change and refinement as the SPUD Application moves through the SPUD process and the SPUD Plans are refined.

Workforce Housing (CDC 17.3.9)

As the Town of Mountain Village is in the process of revising its workforce housing code, it is impossible to identify the plan for this component at this time. Under the current code, there is one workforce housing unit assigned to lot 71-R to be constructed in the project. Given the constraints of the program, MFP will need to fulfill workforce housing offsite through mitigation and will work with the Town to develop a plan to address this issue.

Maximum Lot Coverage (CDC 17.3.13)

There is no lot coverage limit for the Village Center Zone District due to the high-density nature of this zone in the Comprehensive Plan.

General Easement Setbacks (CDC 17.3.13)

A 16' general easement exists along the property line of Lot 161C-R at Lots 97, 98, 100, 101, and Tract OS-3U. The remainder of Lot 161C-R and all Pond Lots indicate 0' lot lines. The 16' general easement along the boundary of Lot 161C-R that is will be replatted into Lot 161C-RR will be vacated.

Building Siting Design (CDC 17.5.5)

The proposed development intends to comply with the Building Siting Design standards. At grade walls will have a rhythm of solid and glazing that will create vertical proportions throughout that reinforces overall building compositions and architectural languages.

Lower-level walls will be of a different material in scale and color to differentiate between residential spaces above.

Grading and Drainage Design (CDC 17.5.7)

The proposed development intends to comply with the Grading and Drainage Design standards.

Landscaping regulations (CDC 17.5.9)

The proposed development intends to comply with the landscaping regulations.

Trash, recycling and storage areas (CDC 17.5.10)

The proposed development intends to comply with the Trash, recycling and storage areas design standards. Trash and recycling will be part of the loading dock/service area, located on the north end of the hotel, and will contain a trash compactor within the building, accessed via an overhead door.

Sign regulations (CDC 17.5.13)

The proposed development intends to comply with the sign regulations and will be detailed in the Sketch SPUD Application.

Commercial, ground level and plaza area design regulations (CDC 17.5.15)

The Commercial frontages will be articulated with covered canopies to lower the scale of these taller floors to a more human scale. Entries will be clearly defined with site elements, lighting, and architectural features that clearly invite guests and patrons in. Restaurant and Commercial spaces will include large sliding walls that connect interior and exterior spaces to blur the line of indoor and outdoor extending the scale of plaza spaces in the summer and shoulder seasons.

The Lower levels of the project will be constructed out of a distinct material which will differentiate their uses from the upper floors. The canopies, lighting, landscape elements, and large sliding walls will further distinguish the retail and commercial storefronts from the hotel, hotel residences and private residences above.

Utilities (CDC 17.5.11)

Existing utilities that currently run through the site will be rerouted around the proposed building footprint with exception of the water line, which will be routed through the parking garage.

SITE CIRCULATION AND PUBLIC ACCESS

A site circulation diagram is attached to illustrate the proposed circulation within the Project. The following narrative describes the preliminary site circulation intent:

The site circulation has been divided into 3 categories – General public, Amenity patrons (paying public to the hotel), and Shared Private - hotel guests and residents. The general public will be limited to the perimeter of the Project, primarily along the west and south sides. A public trail (6' wide) has been provided along the west side that connects through to adjacent community amenities of Conference Plaza to northwest, Heritage Plaza to west, Ski Beach and beyond to southwest, and Gondola Plaza to the south. Gorrono Creek will be improved to create an aesthetic amenity for all who travel or view this corridor while also maintaining (and improving if necessary) its functionality.

Within the Project, there are two levels of access. Along the eastern side it is primarily private for the residents who will be contained within the two resident buildings. The western building will be primarily hotel-oriented (however it will also contain some private residences) so will cater to both hotel guests but also paying public patrons of the hotel that may patronize the lobby, two restaurants (Chalet Suisse and Apres Ski Restaurant) and Spa amenities. The central garden space (highlighted in green) will be shared private and for the hotel guests and residents. Access to this area will be via a key card. All proposed hot tubs and fire pits, with the exception of the fire pit to the north of Gondola Plaza (for general public) and the private hot tubs and fire pits adjacent to the resident buildings, will be for amenity patrons.

On the southwest corner of the hotel, a concierge will be provided for hotel guests and residents to facilitate outdoor-oriented equipment.

All vehicular arrivals to the Project will be via the auto-court on the north side with valet parking for residents, hotel guests and amenity patrons. Some residents may desire to self-park which will be permitted with elevators and stairs available for them to circulate to lobby spaces.

Elevators and stairs within the lobby spaces of the western hotel building and eastern resident tower buildings will facilitate vertical circulation to the various outdoor amenity spaces when at grade passage is not possible.

Any proposed outdoor landscape lighting associated with the site circulation or amenity spaces will be safety related (e.g., at steps, ramps, egress doors, etc.) only and dark-sky compliant.

All proposed exterior walking surfaces will be slip-resistant and ADA accessible where required.

REZONE AND DENSITY TRANSFER CRITERIA AND STANDARDS (CDC 17.4.9 AND 17.4.10)

The Sketch SPUD Application will address these criteria and standards in detail.

SUBDIVISION CRITERIA AND STANDARDS (CDC 17.4.13)

The Sketch SPUD Application will address these criteria and standards in detail.

VESTED PROPERTY RIGHTS CRITERIA AND STANDARDS (CDC 17.4.17)

The Sketch SPUD Application will address these criteria and standards in detail.

Conceptual Renderings:

*These are conceptual renderings which are subject to further change and modification.

SURVEYOR'S CERTIFICATE:

To LAND TITLE GUARANTEE COMPANY, RAMESH ACQUISITIONS, LLC, A FLORIDA LIMITED LIABILITY COMPANY, TSG ASSET HOLDINGS, LLC, A DELAWARE LIMITED LIABILITY COMPANY AND TSG SKI AND GOLF, LLC, A DELAWARE LIMITED LIABILITY COMPANY:

This is to certify that this map or plat and the survey on which it is based were made in accordance with the 2021 Minimum Standard Detail Requirements for ALTA/NSPS Land Title Surveys, jointly established and adopted by ALTA and NSPS, and includes Items 1,2,3,4,5,6(b),8,9,11,13,16,18,19, and 20 ($1,000,000) of Table A. The field work was completed on August 02, 2021.

Date: ____________

COLORADO REGISTERED DAVID R. BULSON 37662

Digitally signed by David Bulson
Date: 2022.01.14 11:50:14 -07'00'

David R. Bulson PLS 37662

LEGAL DESCRIPTION:

LOT 67, TELLURIDE MOUNTAIN VILLAGE, FILING 1, ACCORDING TO THE PLAT RECORDED MARCH 9, 1984 IN PLAT BOOK 1 AT PAGE 476, COUNTY OF SAN MIGUEL, STATE OF COLORADO.

LOT 69R–2, TELLURIDE MOUNTAIN VILLAGE, FILING 1, ACCORDING TO THE REPLAT OF LOT 69R–1 AND LOT 69R–2 RECORDED SEPTEMBER 5, 1991 IN PLAT BOOK 1 AT PAGE 1164, COUNTY OF SAN MIGUEL, STATE OF COLORADO.

LOT 71R, TELLURIDE MOUNTAIN VILLAGE, FILING 1, ACCORDING TO THE REPLAT AND RE–ZONING OF LOT 71R OF REPLAT NO. 3 RECORDED DECEMBER 2, 1991 IN PLAT BOOK 1 AT PAGE 1208, COUNTY OF SAN MIGUEL, STATE OF COLORADO.

TRACT OS–3Y, TOWN OF MOUNTAIN VILLAGE, ACCORDING TO THE REPLAT OF TRACT OS–3, OS–3B, OS–3C & OS–3E RECORDED JULY 14, 2004 IN PLAT BOOK 1 AT PAGE 3325, COUNTY OF SAN MIGUEL, STATE OF COLORADO.

LOT 161C–R, TOWN OF MOUNTAIN VILLAGE, ACCORDING TO THE PLAT RECORDED APRIL 2, 1999 IN PLAT BOOK 1 AT PAGE 2529, COUNTY OF SAN MIGUEL, STATE OF COLORADO.

NOTES:

1. Easement research and property description according to Land Title Guarantee Company, Order Number ABS86011705, dated June 10, 2021 at 5:00 P.M. as to Lot 67, Lot 69R–2, Lot 71, and Tract OS–3Y

Easement research and property description according to Land Title Guarantee Company, Order Number ABS86011452, dated April 2, 2021 at 5:00 P.M. as to Lot 161C–R

2. The Land does not lie within a Special Flood Hazard Area as defined by the Federal Emergency Management Agency ("FEMA"). According to the Flood Insurance Rate Maps for San Miguel County, Colorado, Community Panel 08113C0287D, dated 09/30/1992 this property lies in Zone X, areas determined to be outside of the 500 year flood plain.

3. BASIS OF BEARINGS. The bearing along the western boundary of Lot 161C–R, was assumed to be S08°03'05"W according to the plat recorded April 2, 1999 in Plat Book 1 at page 2529, County of San Miguel, State of Colorado. The end points of said western boundary are as monumented and described hereon.

4. Lineal units represented hereon are shown in U.S. Survey Feet or a decimal portion thereof.

5. This survey is valid only if a print has original seal and signature of surveyor.

6. Any person who knowingly removes, alters, or defaces any public land survey monument and/or boundary monument or accessory, commits a class two (2) misdemeanor pursuant to C.R.S. 18–4–508.

7. The word certify as used hereon means an expression of professional opinion regarding the facts of this survey and does not constitute a warranty or guarantee, expressed or implied.

8. This survey is prepared for the exclusive use of the party or parties indicated within the surveyor's statement. Said statement does not extend to any unnamed person or parties without an express statement by the surveyor naming said entities.

9. According to Colorado law, you must commence any legal action based upon any defect in this survey within three years after you first discover such defect. In no event may any action based upon any defect in this survey be commenced more than ten years from the date of the certification shown hereon.

10. There is no evidence of this lot being use as a solid waste dump, sump, or sanitary land fill.

11. There is no evidence of earth moving or building construction within recent months on these lots.

12. There is no observable evidence of recent street or sidewalk construction or repairs.

13. Utilities shown hereon are according to best available records and site specific locates. The surveyor makes no assurance as to the accuracy or completeness of the information. Prior to any construction or site disturbance, the contractor is required to call the Utility Location Center of Colorado (*811) for a site specific Utility locate.

TITLE COMMITMENT NOTES:

Land Title Guarantee Company, Order Number ABS86011705, dated June 10, 2021 at 5:00 P.M. as to Lot 67, Lot 69R–2, Lot 71, and Tract OS–3Y "Pond Lots"

Schedule B–2 (TITLE EXCEPTION RESPONSE/CLARIFICATION)

1. Site inspection and Survey performed by Bulson Surveying conditions shown hereon.
2. There are portion of the surveyed property being used for paid and permitted parking although there were no Easements, liens or encumbrances, or claims thereof, not shown by the Public Records brought to the attention of this Surveyor during the course of this Survey.
3. Site inspection and Survey performed by Bulson Surveying conditions are as shown hereon
4. Not survey related.
5. Not survey related.
6. Not survey related.
7. (a) Based upon a search of the USBLM public records, there are no unpatented mining claims affecting the subject property. (b,c) The patent from the United States of America number 131878 dated May 23, 1910 subjects the Land to "any vested and accrued water rights for mining, agricultural, manufacturing, or other purposes, and rights to ditches and reservoirs used in connection with such water rights, as may be recognized and acknowledged by the local customs, laws, and decisions of courts, and also subject to the right of the proprietor of a vein or lode to extract and remove his ore therefrom, should the same be found to penetrate or intersect the premises hereby granted, as provided by law. And there is reserved from the lands hereby granted, a right of way thereon for ditches or canals constructed by the authority of the United States". There is no evidence of ditches or water storage structures located within the area of this survey.

8. The are portions of the Pond Lots being used for public access and permit parking. There have been no off–record lease or tenancy agreements brought to the attention of Bulson Surveying during the course of preparing this ALTA/NSPS Survey

9. The Plats noted within this exception pertain to the Town of Mountain Village as a whole and were not acknowledged or approved by the owner of the Subject Property at the time. It is the opinion of the Surveyor that they do not create any easements relevant to the property. It is beyond the scope of this survey to determine whether there are Conditions, Covenants, Restrictions or Notes contained within these Plats that affect the Property.

10. Restrictive Covenants for the Mountain Village noted within this exception are blanket in nature and affect the Pond Lots.

11. The Tap Fee Assignment and Assumption Agreement recorded March 8, 1999 under reception No. 324840 affects Lot 69R2 and Lot 71R and is blanket in nature.

12. The Underground Parking Amendment recorded July 21, 1989 in Book 455 at page 550 references a Lot 152, Telluride Mountain Village and does not appear to affect the property being surveyed, but the document speaks for itself as to its relevance to the subject property

13. The Facilities, Water Rights and Easements noted within this exception affect the Pond Lots and are blanket in nature.

14. The Town of Mountain Village Employee Housing Restrictions noted within this exception affect Lot 71R and is blanket in nature.

15. All easements noted on the Plats cited in this exception are shown and labeled on this ALTA/NSPS Survey, with the exception of easements that have been altered or eliminated by subsequent plats or other legal instruments. Revised easements are shown according to the locations cited in the most current documentation.

16. The Telluride Company reserved the rights to minerals and oil, gas, and other hydrocarbons located on, in, or under Lot 67 according to the deed recorded November 5, 1993 in Book 411 at Page 903 and located on, in, or under Lot 69R–2 and Lot 71R according to the deed recorded March 8, 1999 under Reception Number 324838. There is no visible evidence of mining activity on the subject property

17. According to the Warranty Deed recorded at Book 520, page 23 relating to Lot 67, there were reservations number 13 and 14 which noted a limitation on the used allowed on Lot 67. It is unclear as to the relevance of this reservation and the document speaks for itself.

According to the Warranty Deed recorded at Reception 324838 and relating to Lot 69R–2 and Lot 71R, there were reservations numbers 10–15 which noted a limitation on the used allowed on the Lot 71R. It is unclear as to the relevance of this reservation and the document speaks for itself.

18. According to the Agreement recorded at Book 431, page 544 and relating to Lot 67 and Lot 71R There are restrictions on Lot 71R which limit what may be constructed on Lot 71R. The location of the Public Walkway noted within the agreement is generally shown hereon although the precise location is unclear

19. The Right–of–Way Easement noted within this exception is blanket in nature and affects Tract OS–3Y

20. The Promissory Note recorded in Book 474 at pages 66–67 is blanket in nature and affects Lot 69R–2

21. The Resolution recorded in Book 482 at page 171 is blanket in nature and affects Lot 69R–2

22. The Resolution recorded in Book 485 at page 259 is blanket in nature and affects Lot 71R

23. The Resolution recorded at reception numbers 318369 and 318449 are blanket in nature and affect Lot 71R

24. The Utility Easement Agreement noted within this exception is blanket in nature and affects Tract OS–3Y

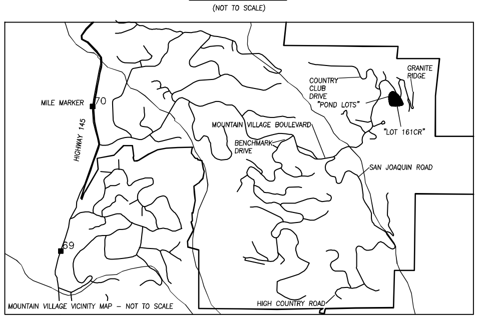
25. The San Miguel Power Association Notice cited within this exception is blanket in nature and affects the Pond Lots

26. The Easement Agreement noted within this exception is blanket in nature and affects Tract OS–3Y

27. The Mountain Village Openspace list noted within this exception is blanket in nature and affects Tract OS–3Y

VICINITY MAP

(NOT TO SCALE)



EXISTING DENSITY

(ACCORDING TO 08/26/2021 RECORDS OF TOWN OF MOUNTAIN VILLAGE)

Lot Number	Zoning Designation	Units	Person Equivalent Density
67	CONDO	14	42
69R2	CONDO	12	36
71R	CONDO	9	27
71R	EMP APT	1	3
161CR	CONDO	33	99
161CR	HOTEL EFF	2	4

TITLE COMMITMENT NOTES:

Land Title Guarantee Company, Order Number ABS86011452 dated April 02, 2021 at 5:00 P.M. as to Lot 161C–R, Town of Mountain Village "Lot 161C–R"

Schedule B–2 (TITLE EXCEPTION RESPONSE/CLARIFICATION)

1. Site inspection and Survey performed by Bulson Surveying conditions shown hereon.
2. There are portion of Lot 161C–R being used for paid and permitted parking although there were no Easements, liens or encumbrances, or claims thereof, not shown by the Public Records brought to the attention of this Surveyor during the course of this Survey.
3. Site inspection and Survey performed by Bulson Surveying conditions are as shown hereon
4. Not survey related.
5. Not survey related.
6. Not survey related.
7. (a) Based upon a search of the USBLM public records, there are no unpatented mining claims affecting the subject property. (b,c) The patent from the United States of America number 131878 dated May 23, 1910 subjects the Land to "any vested and accrued water rights for mining, agricultural, manufacturing, or other purposes, and rights to ditches and reservoirs used in connection with such water rights, as may be recognized and acknowledged by the local customs, laws, and decisions of courts, and also subject to the right of the proprietor of a vein or lode to extract and remove his ore therefrom, should the same be found to penetrate or intersect the premises hereby granted, as provided by law. And there is reserved from the lands hereby granted, a right of way thereon for ditches or canals constructed by the authority of the United States". There is no evidence of ditches or water storage structures located within the area of this survey.

8. The are portions of the Lot 161C–R being used for public access and permit parking. There have been no off–record lease or tenancy agreements brought to the attention of Bulson Surveying during the course of preparing this ALTA/NSPS Survey

9. The United States Patent recorded June 08, 1918, in Book 99 at page 142 reserved a right for ditches and canals constructed by the Authority of the United States. There is a ditch located on the western side of Lot 161C–R as depicted hereon. It is beyond the scope of this Survey to determine if it was constructed under the Authority of the United States.

10. The Plats noted within this exception pertain to the Town of Mountain Village as a whole and were not acknowledged or approved by the owner of the Subject Property at the time. It is the opinion of the Surveyor that they do not create any easements relevant to the property. It is beyond the scope of this survey to determine whether there are Conditions, Covenants, Restrictions or Notes contained within these Plats that affect the Property.

11. Restrictive Covenants for the Mountain Village noted within this exception are blanket in nature and affect the Lot 161C–R.

12. The Water and Sewer Tap Fee notice and agreements noted within this exception do not make specific mention of Lot 161C–R. It is beyond the scope of this survey to how these notices and agreements affect the Property.

13. The Underground Parking Amendment recorded July 21, 1989 in Book 455 at page 550 references a Lot 152, Telluride Mountain Village and does not appear to affect the property being surveyed, but the document speaks for itself as to its relevance to the subject property

14. The Right–of–Way Easement noted within this exception cites a blanket easement over Tract OS3, a portion of which has been included within Lot 161C–R, pursuant to the plat recorded according to the plat recorded April 2, 1999 in plat Book 1 at page 2529. The portion of Lot 161C–R which is subject to this easement is noted hereon

15. The Facilities, Water Rights and Easements noted within this exception affect Lot 161C–R and are blanket in nature.

16. There is a 16' General Easement along the perimeter of Lot 161C–R as indicated hereon. The Agreements noted within this exception relate to this area on Lot 161C–R and affect what may occur within this area of the Lot.

17. The Town of Mountain Village Employee Housing Restrictions noted within this exception affect Lot 161C–R and are blanket in nature.

18. All easements noted on the Plats cited in this exception are shown and labeled on this ALTA/NSPS Survey, with the exception of easements that have been altered or eliminated by subsequent plats or other legal instruments. Revised easements are shown according to the locations cited in the most current documentation.

19. The deeds listed within this exception cite restrictions on future development of portions of Lot 161C–R. These restrictions are noted within each of the deeds and they pertain to previously platted lots which were combined to create Lot 161C–R as depicted on plat recorded April 2, 1999 in plat Book 1 at page 2529. This survey depicts the areas of each of the previous lots and indicates the original deed conveyance for each sub–parcel of Lot 161C–R.

20. The Termination of Title Exceptions listed within this exception remove restrictions on future development of portions of Lot 161C–R which were originally imposed by the deeds cited in Exception 19. These Termination of Title Exceptions pertain to previously platted lots which were combined to create Lot 161C–R as depicted on plat recorded April 2, 1999 in plat Book 1 at page 2529. This survey depicts the areas of each of the previous lots and indicates the original deed conveyance for each sub–parcel of Lot 161C–R.

21. The San Miguel Power Association Notice cited within this exception is blanket in nature and affects Lot 161C–R

22. The Resolution recorded under reception number 325408 is blanket in nature and affects Lot 161C–R

23. This exception notes a deed restriction pertaining to wetland areas. A delineation was performed by Terra Firm, Chris Hazen during July of 2021 and is depicted hereon.

24. This exception notes a 16' General Easement along a portion of the northern boundary of Lot 161C–R as depicted hereon

25. This exception has been intentionally deleted

26. This exception has been intentionally deleted

27. The Station Mountain Village Covenant contains a defined "Covenant Area" which allows for the future removal of a portion of an existing wall along the Gondola Station. This Covenant Area is along the southern boundary of Lot 161C–R and is depicted hereon.

28. The Resolutions cited in this exception pertain to allowable development density associated with Lot 161C–R. They are blanket in nature and affect the entire property.

29. The Communication Line easement is not located within Lot 161C–R, nor does it appear to benefit Lot 161C–R

30. The Density Assignment and Transfer cited within this exception does not contain reference to Lot 161C–R and it is unclear whether this density has been assigned to a specific property.

31. The Density Assignment and Transfer cited within this exception does not contain reference to Lot 161C–R and it is unclear whether this density has been assigned to a specific property.

32. The Shoring Easement Agreement noted within this exception allows for the placement of shoring along a portion of the southwestern property line of Lot 161C–R at the location as depicted hereon

33. The Density Assignment and Transfer cited within this exception conveys density previously assigned to Lot 161C–R to other property within the Mountain Village.

34. The Easement noted within this exception is not located within Lot 161C–R, nor does it appear to benefit Lot 161C–R

35. The Density Assignment and Transfer cited within this exception conveys density previously assigned to Lot 161C–R to other property within the Mountain Village.

36. A portion of Lot 161C–R is being used for a surface graveled parking lot as depicted hereon. This exception cites a Conditional Use Permit associated with this parking lot.

37. The Density Assignment and Transfer cited within this exception conveys density previously assigned to Lot 161C–R to other property within the Mountain Village.

38. The Settlement Agreement and Mutual Release cited in this exception is blanket in nature and affects Lot 161C–R

39. The Resolution noted within this exception is blanket in nature and affects Lot 161C–R

40. The Memorandum of Reservation cited within this exception burdens the future development of Lot 161C–R but does contain any defined location and is therefore not depicted.

41. The Memorandum of Reservation cited within this exception burdens the future development of Lot 161C–R but does contain any defined location and is therefore not depicted

42. Bill of Sale cited within this exception conveys density previously assigned to Lot 161C–R to other property within the Mountain Village.

43–50 The Memorandum of Reservation cited within these exceptions burden the future development of Lot 161C–R but do not contain any defined locations and are therefore not depicted

SHEET INDEX:

1. Certifications/Notes/Density
2. Lot Dimensions/Recorded Easements
3. Topography and Existing Improvements

Ramesh Acquisitions, LLC
2434 E. Las Olas Blvd.
Fort Lauderdale FL 33301

Lots 67, 69R-2, 71R, 161C-R and Tract OS-3Y
Town of Mountain VIlage
San Miguel County, Colorado

DATE : August 27, 2021

PROJECT NUMBER
21057

BULSON SURVEYING

SHEET 1

ALTA/NSPS Land Title Survey

CURVE TABLE					
CURVE	LENGTH	RADIUS	DELTA ANG.	CHORD	CHORD BEARING
C6	100.29'	117.50'	48°54'14"	97.27'	S 43°39'39" E
C5	184.04'	532.81'	19°47'28"	183.13'	N 29°06'16" W

Line Table		
Line #	Direction	Length
L147	N 74°12'32" W	47.74'
L148	N 74°12'32" W	51.51'

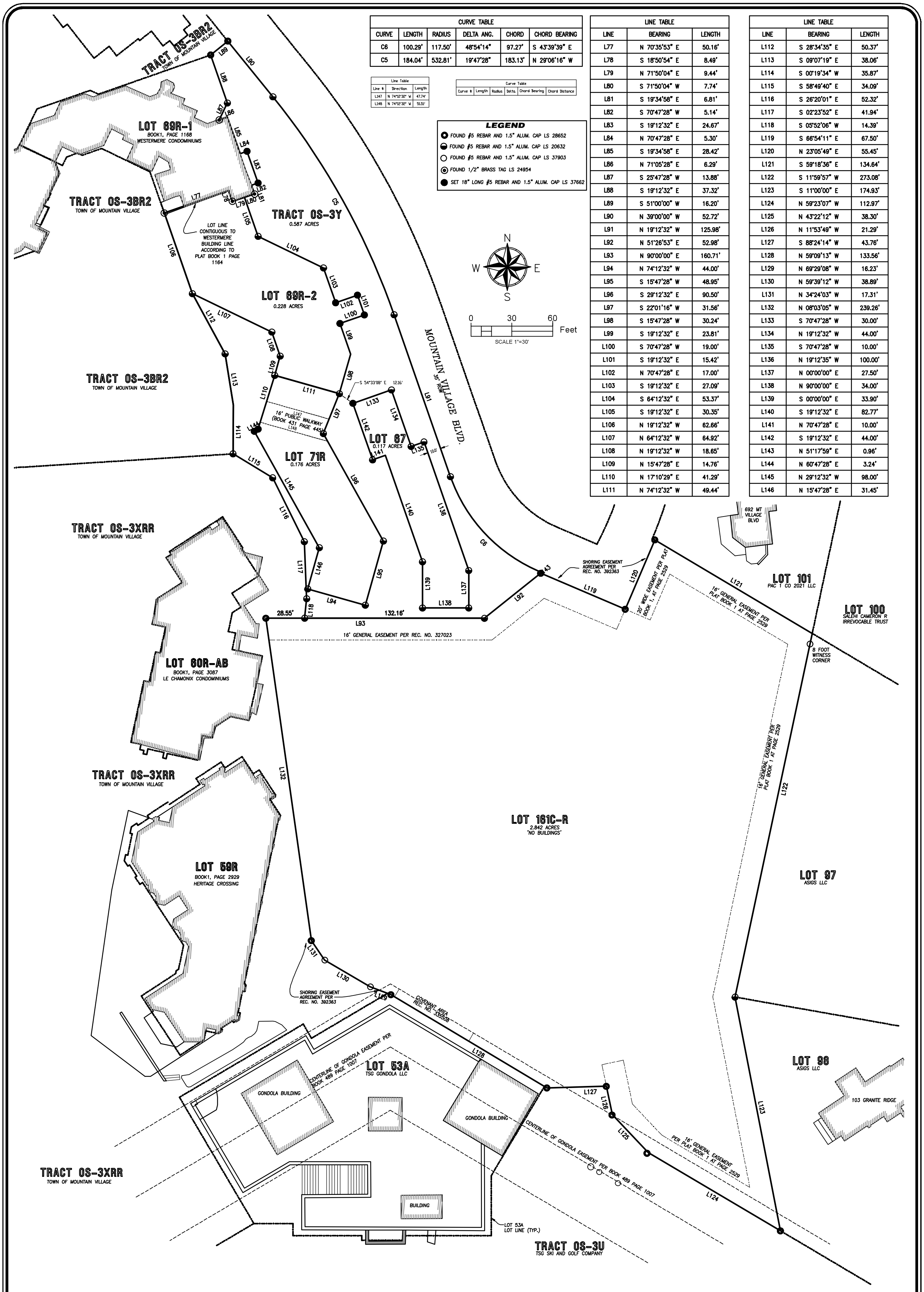
Curve Table					
Curve #	Length	Radius	Delta	Chord Bearing	Chord Distance

LINE TABLE		
LINE	BEARING	LENGTH
L77	N 70°35'53" E	50.16'
L78	S 18°50'54" E	8.49'
L79	N 71°50'04" E	9.44'
L80	S 71°50'04" W	7.74'
L81	S 19°34'58" E	6.81'
L82	S 70°47'28" W	5.14'
L83	S 19°12'32" E	24.67'
L84	N 70°47'28" E	5.30'
L85	S 19°34'58" E	28.42'
L86	N 71°05'28" E	6.29'
L87	S 25°47'28" W	13.88'
L88	S 19°12'32" E	37.32'
L89	S 51°00'00" W	16.20'
L90	N 39°00'00" W	52.72'
L91	N 19°12'32" W	125.98'
L92	N 51°26'53" E	52.98'
L93	N 90°00'00" E	160.71'
L94	N 74°12'32" W	44.00'
L95	S 15°47'28" W	48.95'
L96	S 29°12'32" E	90.50'
L97	S 22°01'16" W	31.56'
L98	S 15°47'28" W	30.24'
L99	S 19°12'32" E	23.81'
L100	S 70°47'28" W	19.00'
L101	S 19°12'32" E	15.42'
L102	N 70°47'28" E	17.00'
L103	S 19°12'32" E	27.09'
L104	S 64°12'32" E	53.37'
L105	S 19°12'32" E	30.35'
L106	N 19°12'32" W	62.66'
L107	N 64°12'32" W	64.92'
L108	N 19°12'32" W	18.65'
L109	N 15°47'28" E	14.76'
L110	N 17°10'29" E	41.29'
L111	N 74°12'32" W	49.44'

LINE TABLE		
LINE	BEARING	LENGTH
L112	S 28°34'35" E	50.37'
L113	S 09°07'19" E	38.06'
L114	S 00°19'34" W	35.87'
L115	S 58°49'40" E	34.09'
L116	S 26°20'01" E	52.32'
L117	S 02°23'52" E	41.94'
L118	S 05°52'06" W	14.39'
L119	S 66°54'11" E	67.50'
L120	N 23°05'49" E	55.45'
L121	S 59°18'36" E	134.64'
L122	S 11°59'57" W	273.08'
L123	S 11°00'00" E	174.93'
L124	N 59°23'07" W	112.97'
L125	N 43°22'12" W	38.30'
L126	N 11°53'49" W	21.29'
L127	S 88°24'14" W	43.76'
L128	N 59°09'13" W	133.56'
L129	N 69°29'08" W	16.23'
L130	N 59°39'12" W	38.89'
L131	N 34°24'03" W	17.31'
L132	N 08°03'05" W	239.26'
L133	S 70°47'28" W	30.00'
L134	N 19°12'32" W	44.00'
L135	S 70°47'28" W	10.00'
L136	N 19°12'35" W	100.00'
L137	N 00°00'00" E	27.50'
L138	N 90°00'00" E	34.00'
L139	S 00°00'00" E	33.90'
L140	S 19°12'32" E	82.77'
L141	N 70°47'28" E	10.00'
L142	S 19°12'32" E	44.00'
L143	N 51°17'59" E	0.96'
L144	N 60°47'28" E	3.24'
L145	N 29°12'32" W	98.00'
L146	N 15°47'28" E	31.45'

LEGEND

- FOUND #5 REBAR AND 1.5" ALUM. CAP LS 28652
- FOUND #5 REBAR AND 1.5" ALUM. CAP LS 20632
- FOUND #5 REBAR AND 1.5" ALUM. CAP LS 37903
- FOUND 1/2" BRASS TAG LS 24954
- SET 18" LONG #5 REBAR AND 1.5" ALUM. CAP LS 37662



Ramesh Acquisitions, LLC
2434 E. Las Olas Blvd.
Fort Lauderdale FL 33301

Lots 67, 69R-2, 71R, 161C-R and Tract OS-3Y
Town of Mountain Village
San Miguel County, Colorado

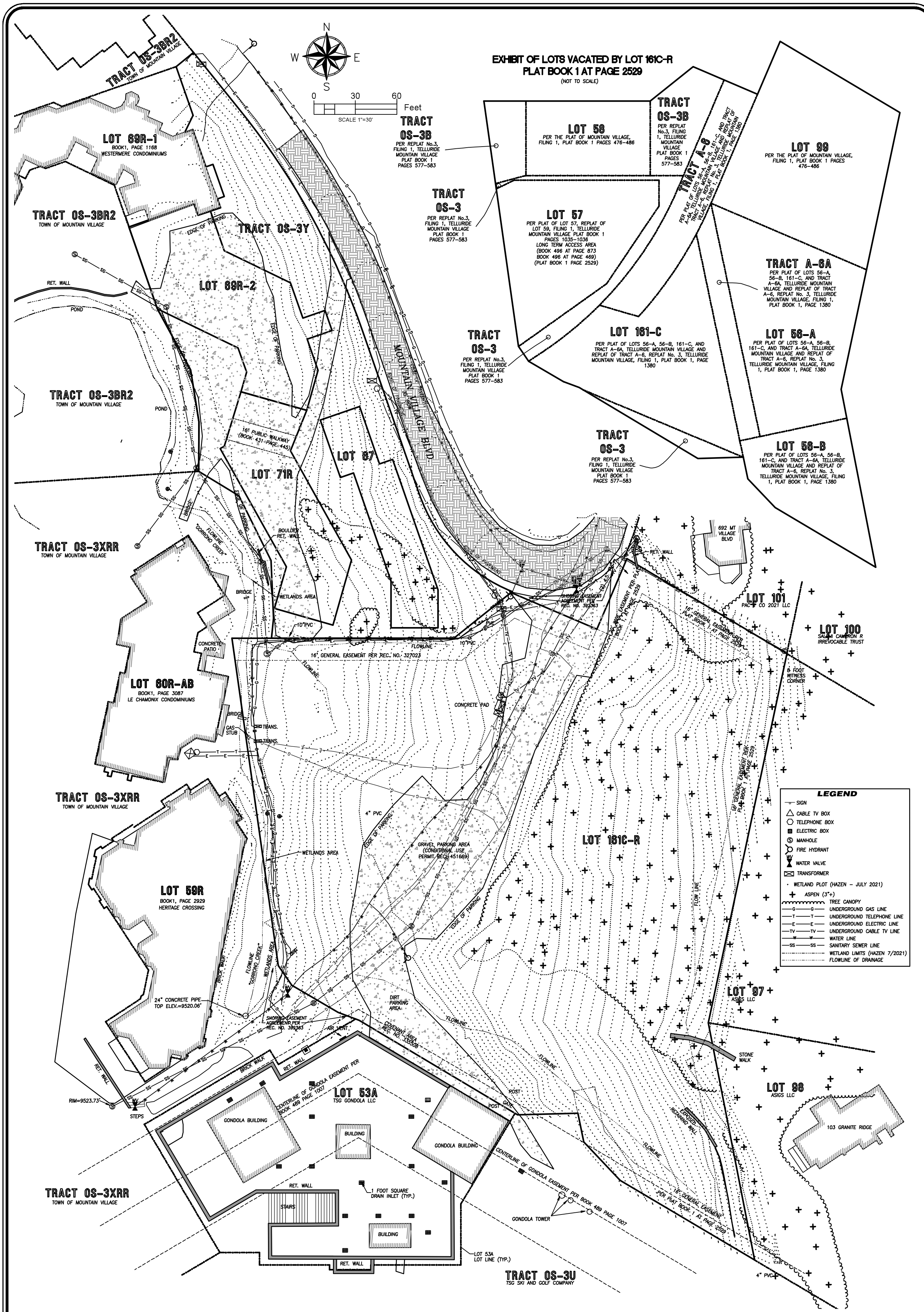
DATE : August 27, 2021

PROJECT NUMBER
21057

BULSON SURVEYING

SHEET 2

ALTA/NSPS Land Title Survey



Ramesh Acquisitions, LLC
2434 E. Las Olas Blvd.
Fort Lauderdale FL 33301

Lots 67, 69R-2, 71R, 161C-R and Tract OS-3Y
Town of Mountain Vllage
San Miguel County, Colorado

DATE : August 27, 2021

PROJECT NUMBER
21057

BULSON SURVEYING

SHEET 3

ALTA/NSPS Land Title Survey

CONCEPTUAL REPLAT

Legend

- UTILITY AND PUBLIC ACCESS CORRIDOR (TO BE CREATED)
- STRUCTURAL ENCROACHMENT EASEMENT (TO BE CREATED)
- UTILITY EASEMENT (TO BE CREATED)
- PROPERTY LINE TO BE VACATED
- PROPERTY LINE TO BE CREATED
- EXISTING PROPERTY LINE TO REMAIN
- ADJOINING PROPERTY LINE TO REMAIN

0 50 100 Feet

SCALE 1" = 50'

Ramesh Holdings, LLC TBD Mountain Village Boulevard Town of Mountain Village, State of Colorado, 81435	**Proposed Lot 161CRR** Town of Mountain Village, Colorado
December 18, 2021	BULSON SURVEYING
PROJECT NUMBER 21057	

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COVER SHEET

S0.00

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ARCHITECTURE
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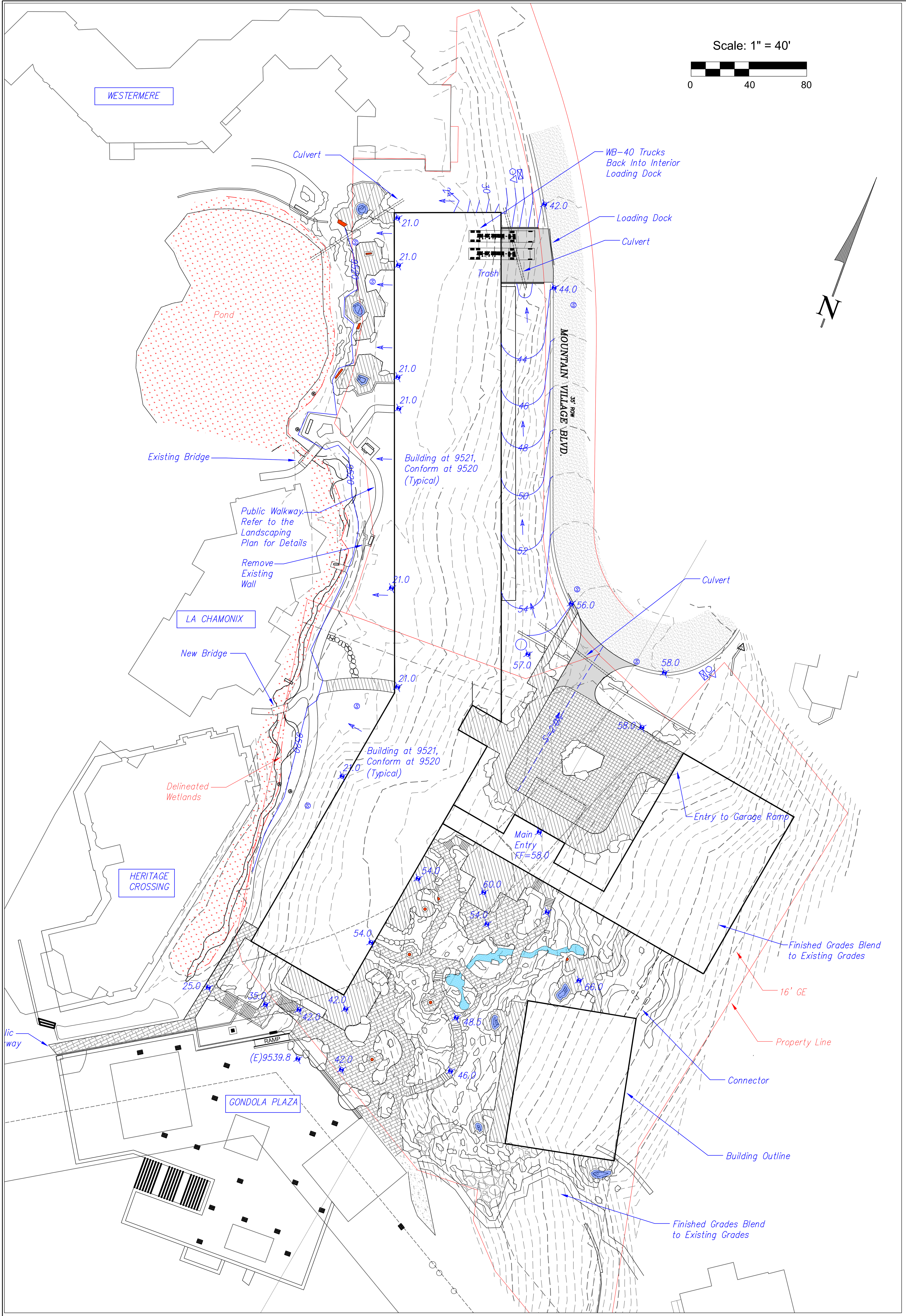
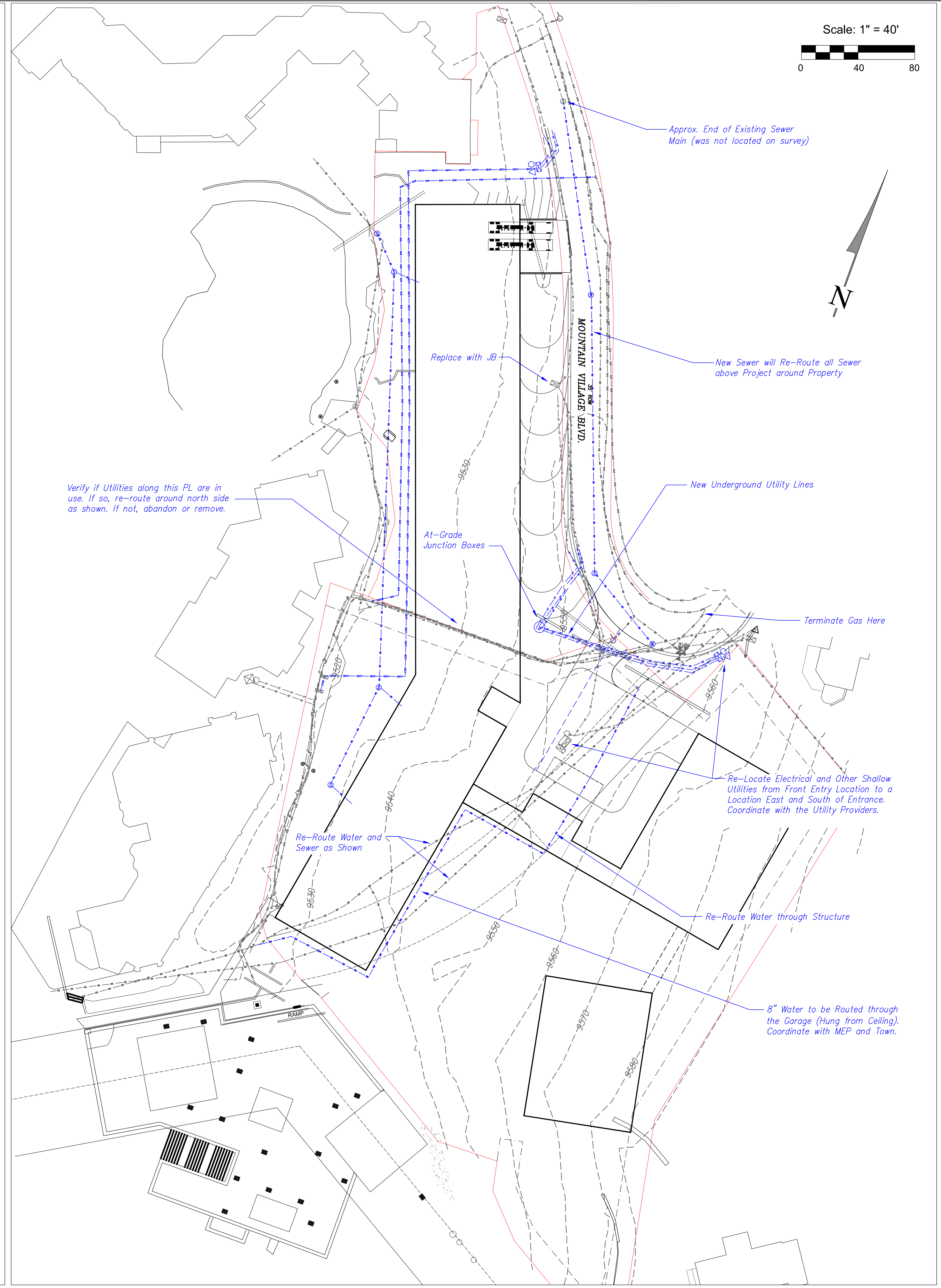
INTERIOR DESIGN



Site Grading



Utility Plan

Uncompahgre Engineering, LLC

P.O. Box 3945
Telluride, CO 81435
970-729-0683

SUBMISSIONS:

SUBMITTAL	2022-01-05

Lot 161 CR
Mtn. Village, CO

NOT FOR CONSTRUCTION

CONTRACTOR TO REVIEW AND COMPARE ALL CHAPTERS AND INTERDISCIPLINARY DRAWINGS AND REPORT ANY DISCREPANCIES TO THE ARCHITECT PRIOR TO ANY FIELD WORK BEING DONE IN ACCORDANCE WITH AIA DOCUMENT A201

Combined Conceptual Site Grading and Utility Plan

C1

DESIGNWORKSHOP

LOT 161 CR
TELLURIDE MOUNTAIN VILLAGE, CO

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CONCEPTUAL SITE PLAN

L1.01

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CONCEPTUAL LANDSCAPING PLAN

L1.02

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CONCEPTUAL SITE CIRCULATION PLAN

L1.03

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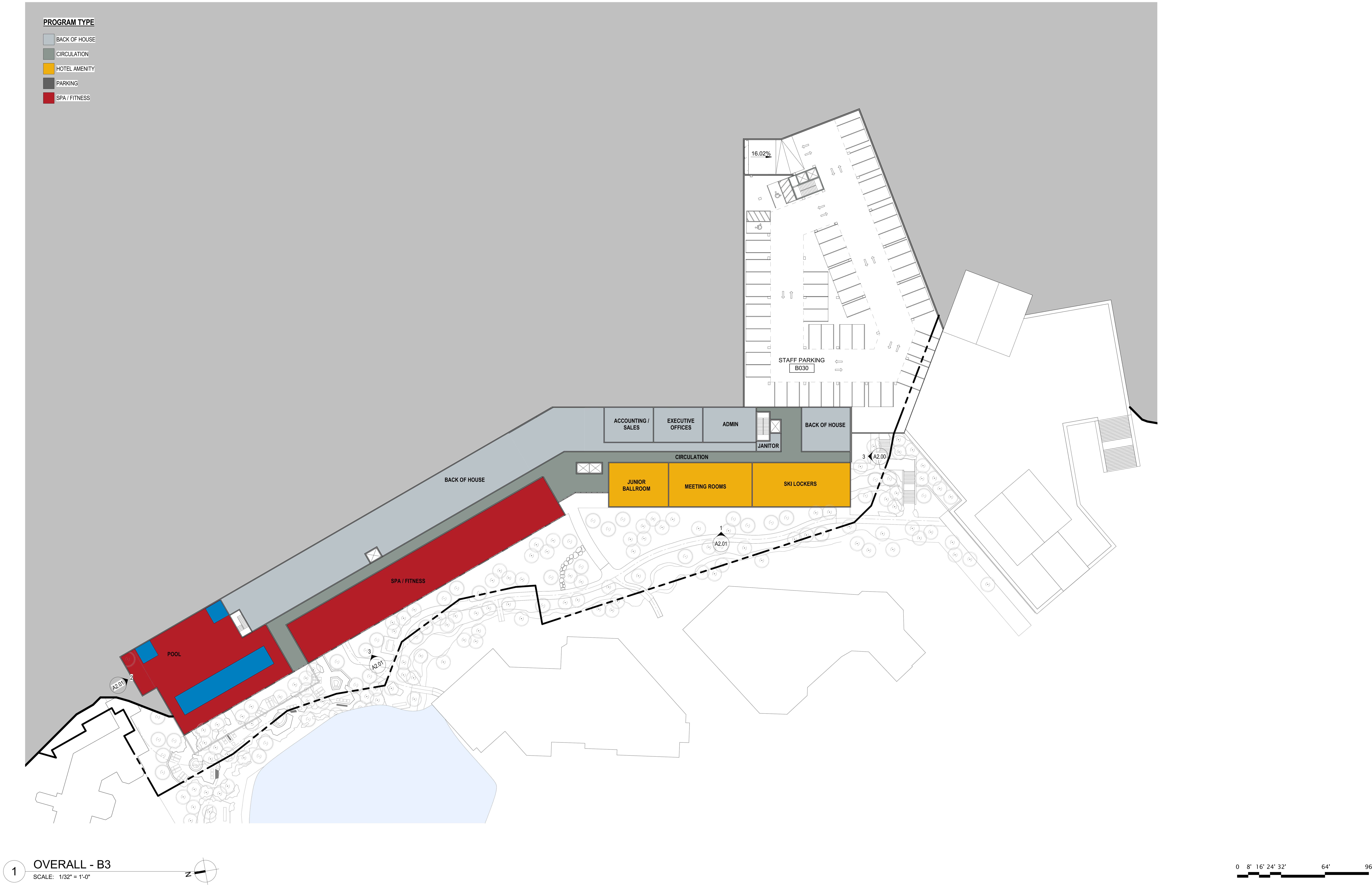

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OVERALL PLAN - B3

A1.11

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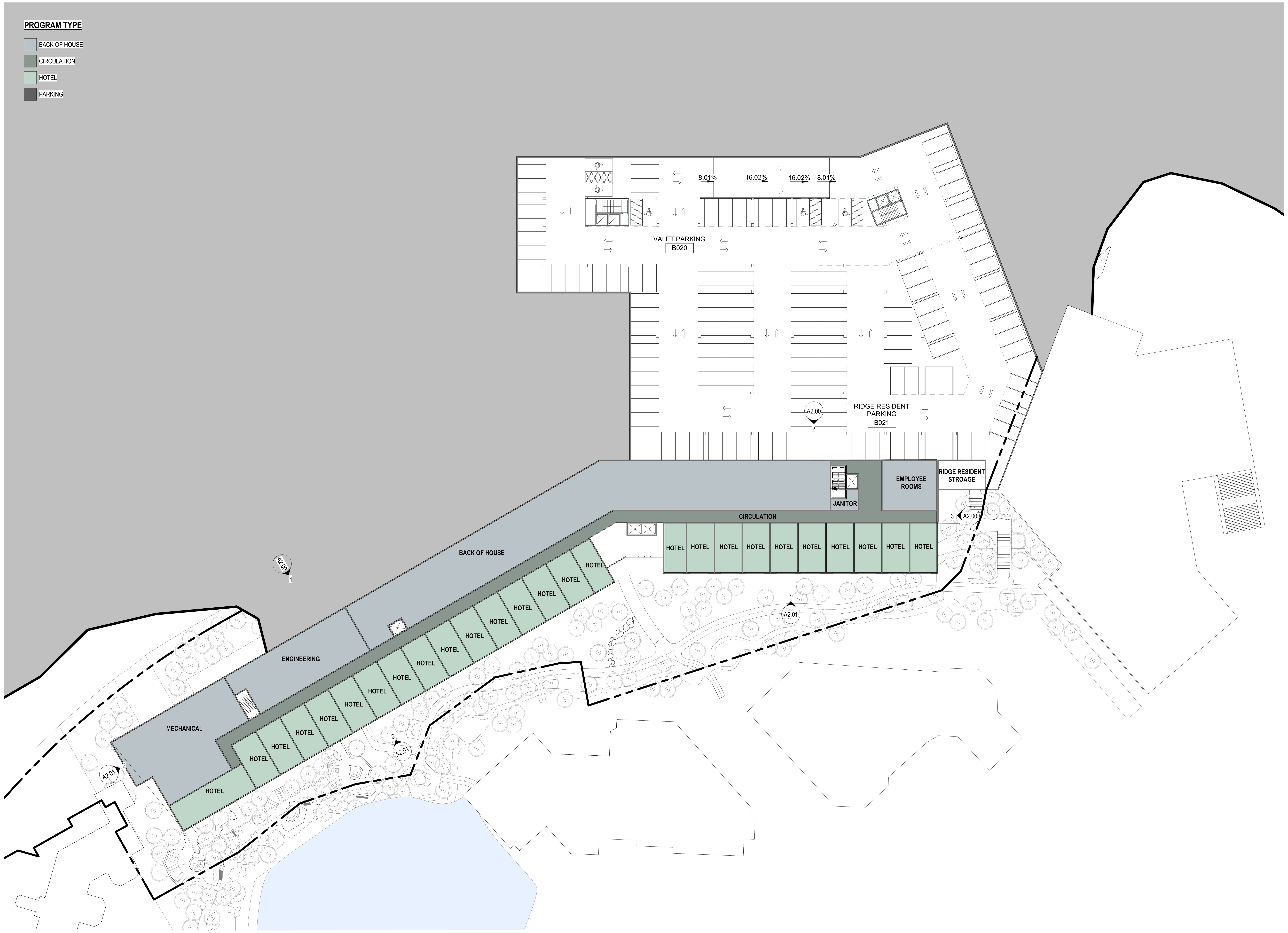

1 OVERALL - B2
SCALE: 1/32" = 1'-0"

0 8' 16' 24' 32' 64' 96'

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OVERALL PLAN - B2

A1.12

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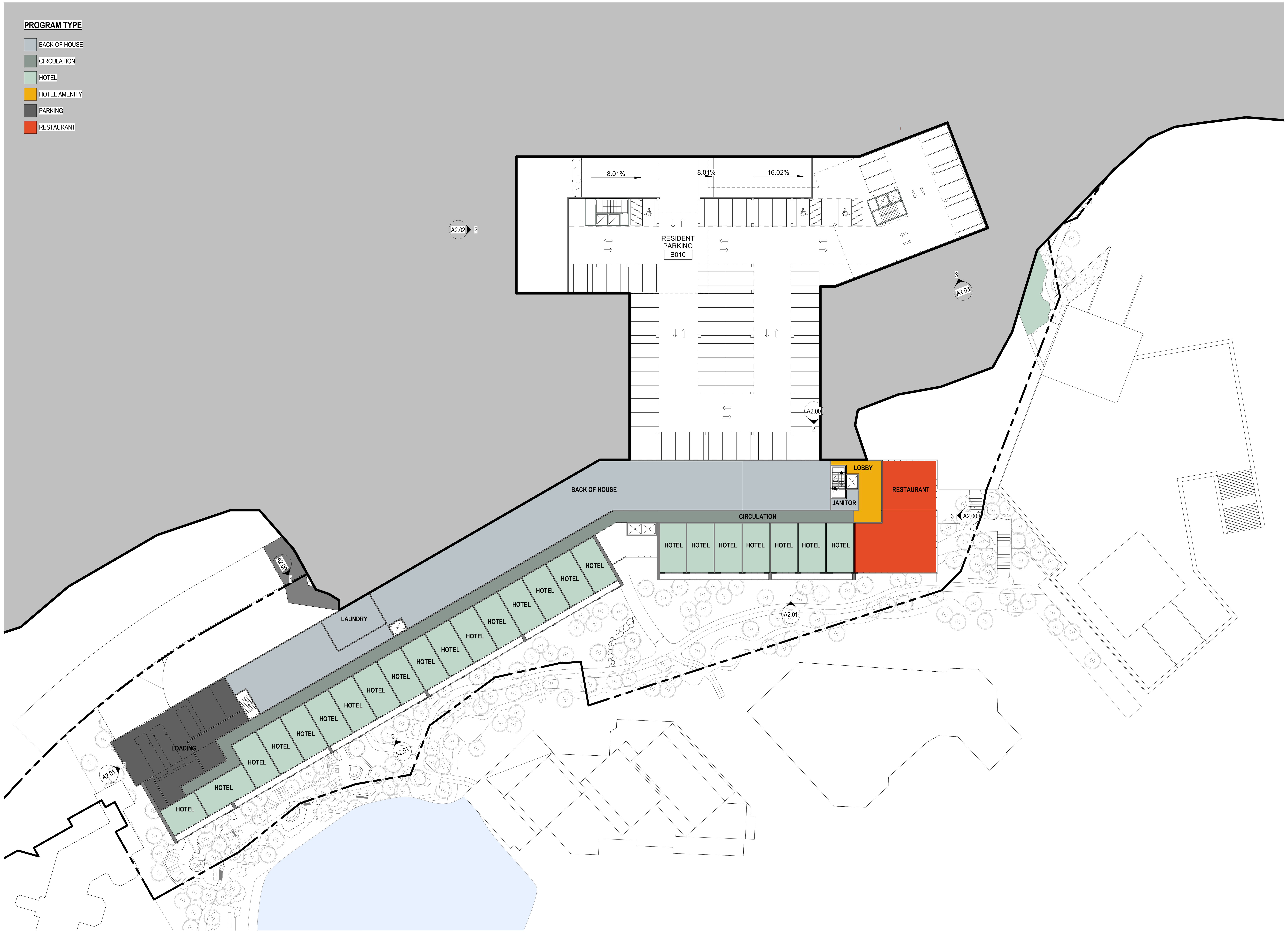

1 OVERALL - B1
SCALE: 1/32" = 1'-0"

0 8' 16' 24' 32' 64' 96'

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OVERALL PLAN - B1

A1.13

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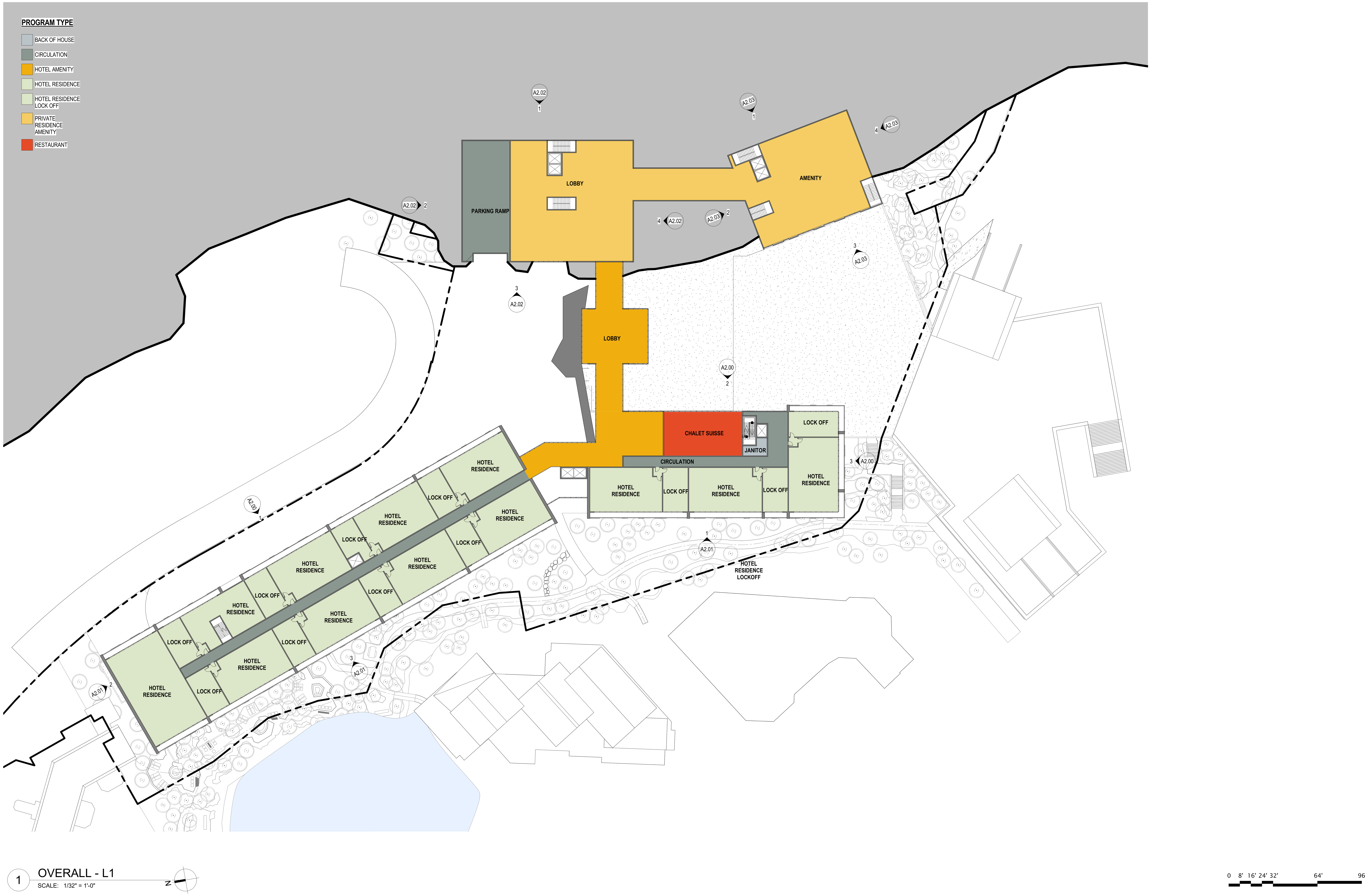

OVERALL PLAN - LEVEL 1

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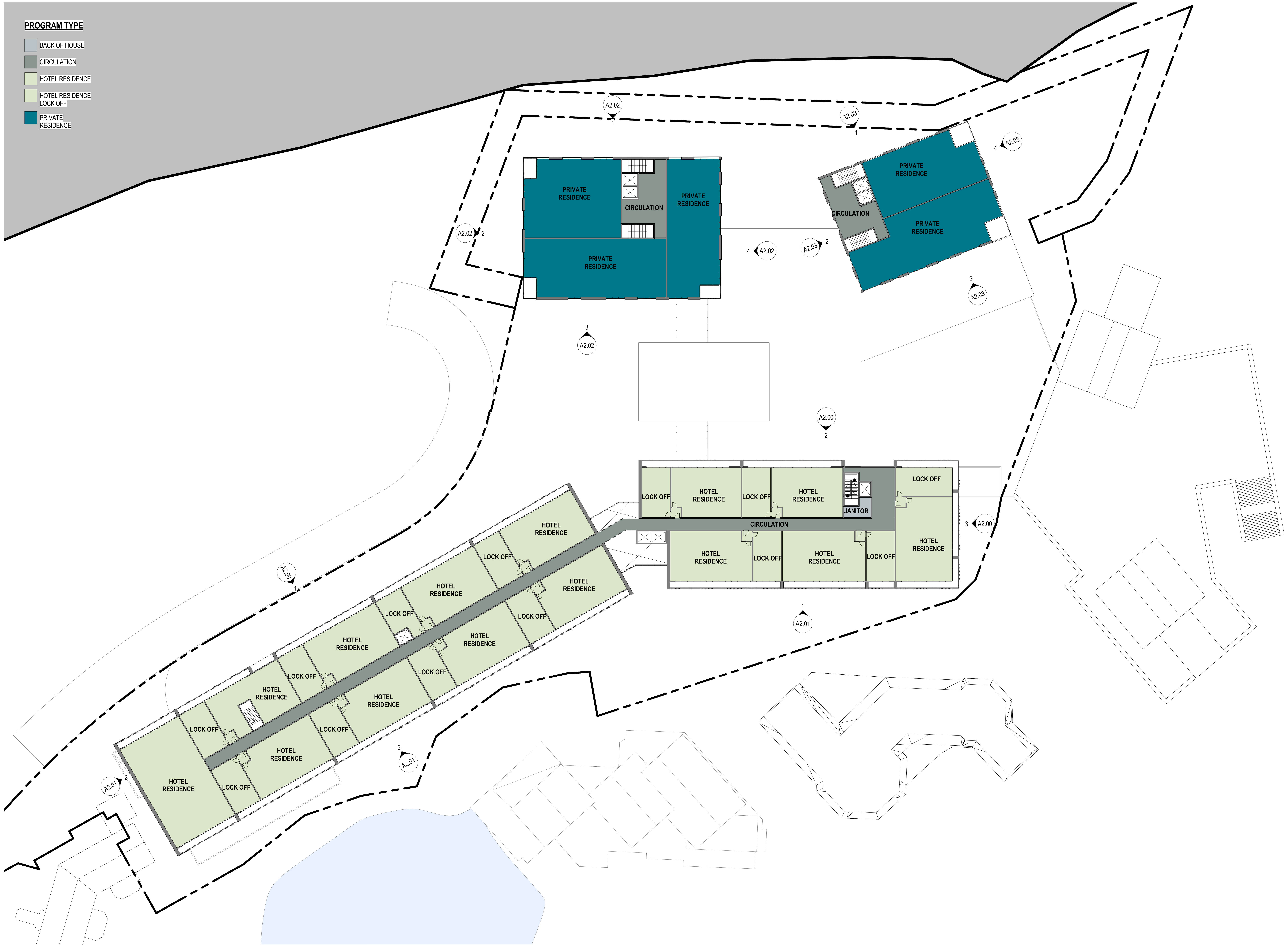
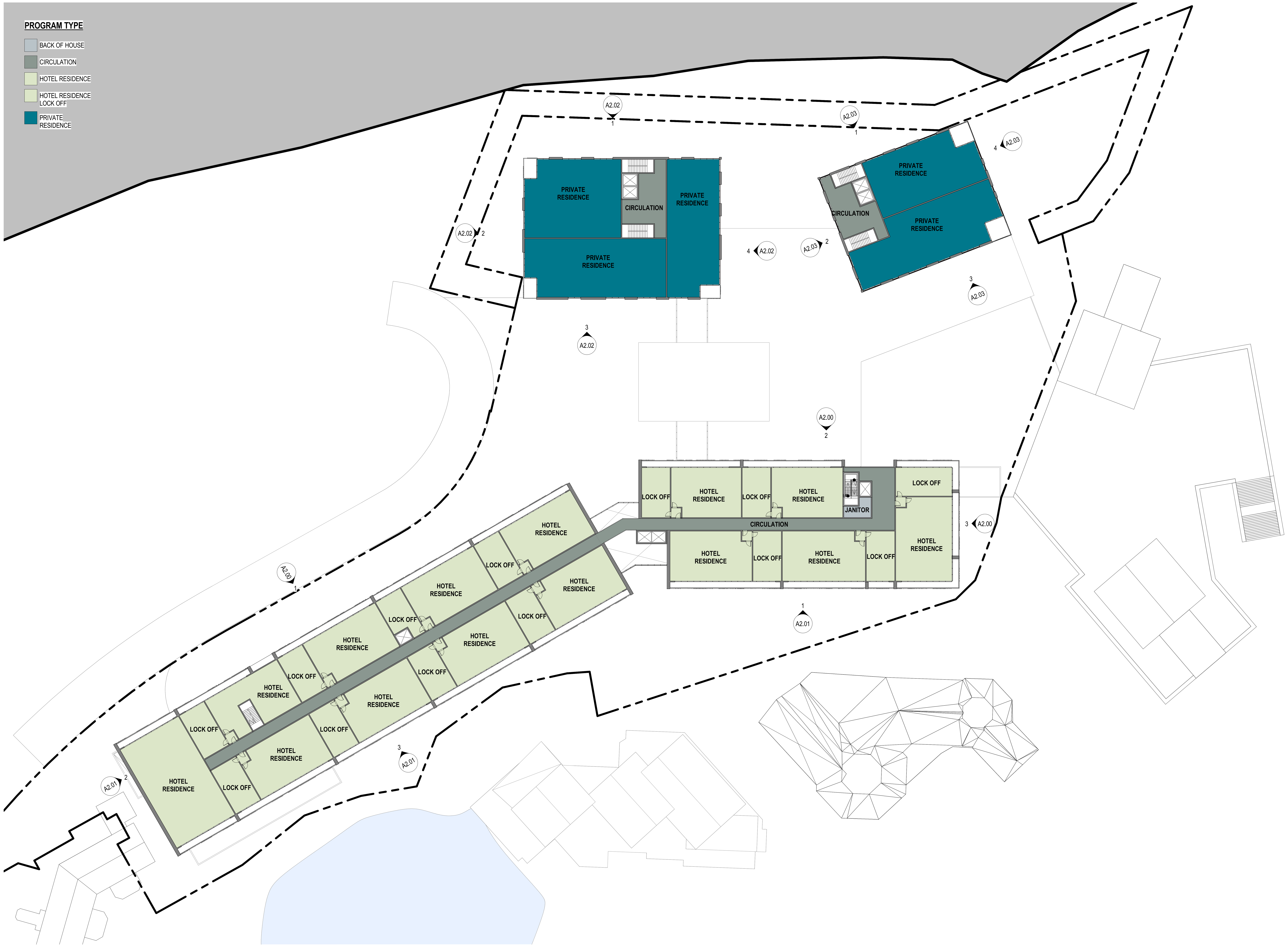


1 OVERALL - L2
SCALE: 1/32" = 1'-0"

0 8' 16' 24' 32' 64' 96'



1 OVERALL - L3
SCALE: 1/32" = 1'-0"

0 8' 16' 24' 32' 64' 96'

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OVERALL PLAN - LEVEL 3

A1.16

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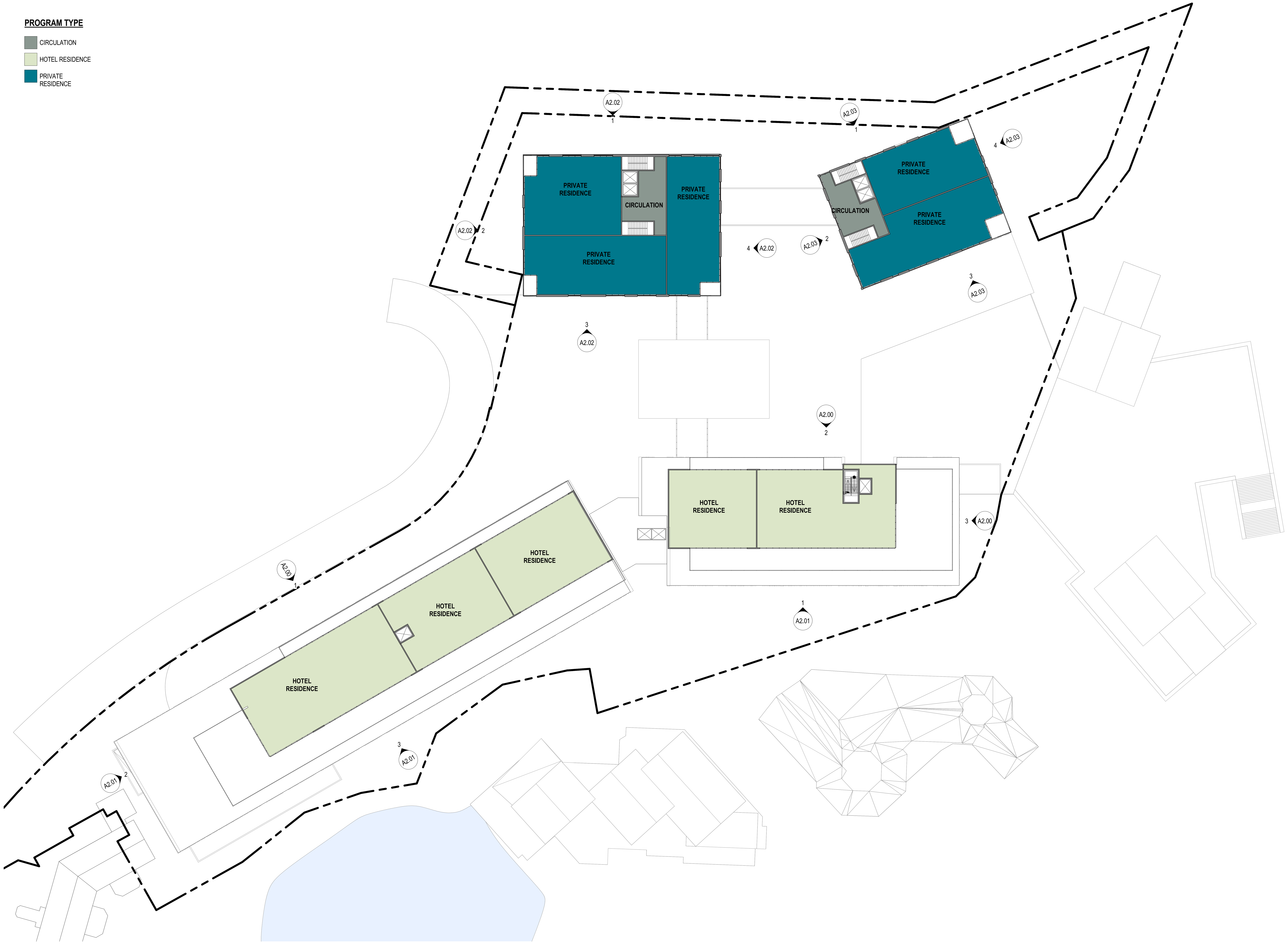

1 OVERALL - L4
SCALE: 1/32" = 1'-0"

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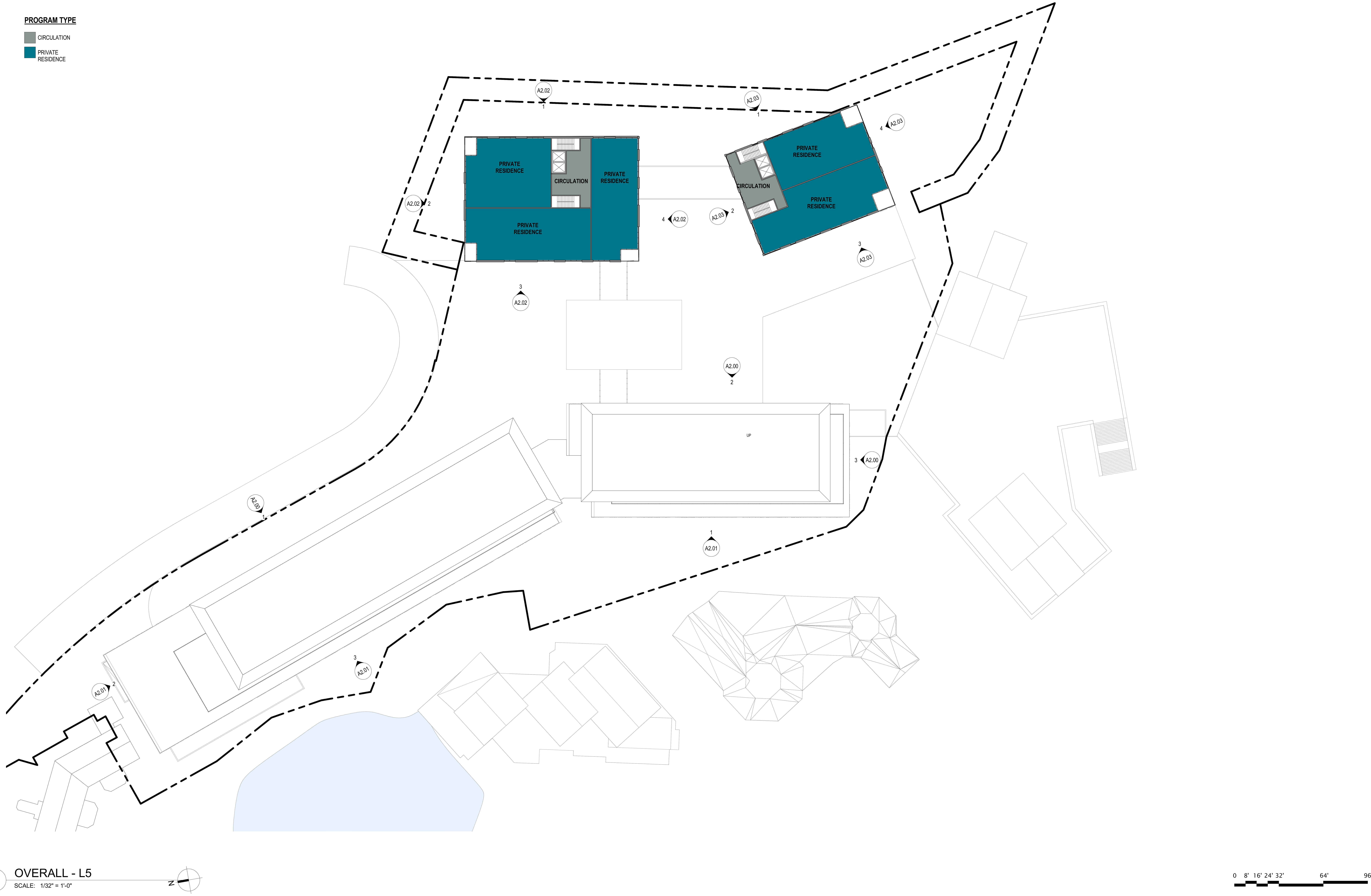

1 OVERALL - L5
SCALE: 1/32" = 1'-0"

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OVERALL PLAN - LEVEL 5

A1.18

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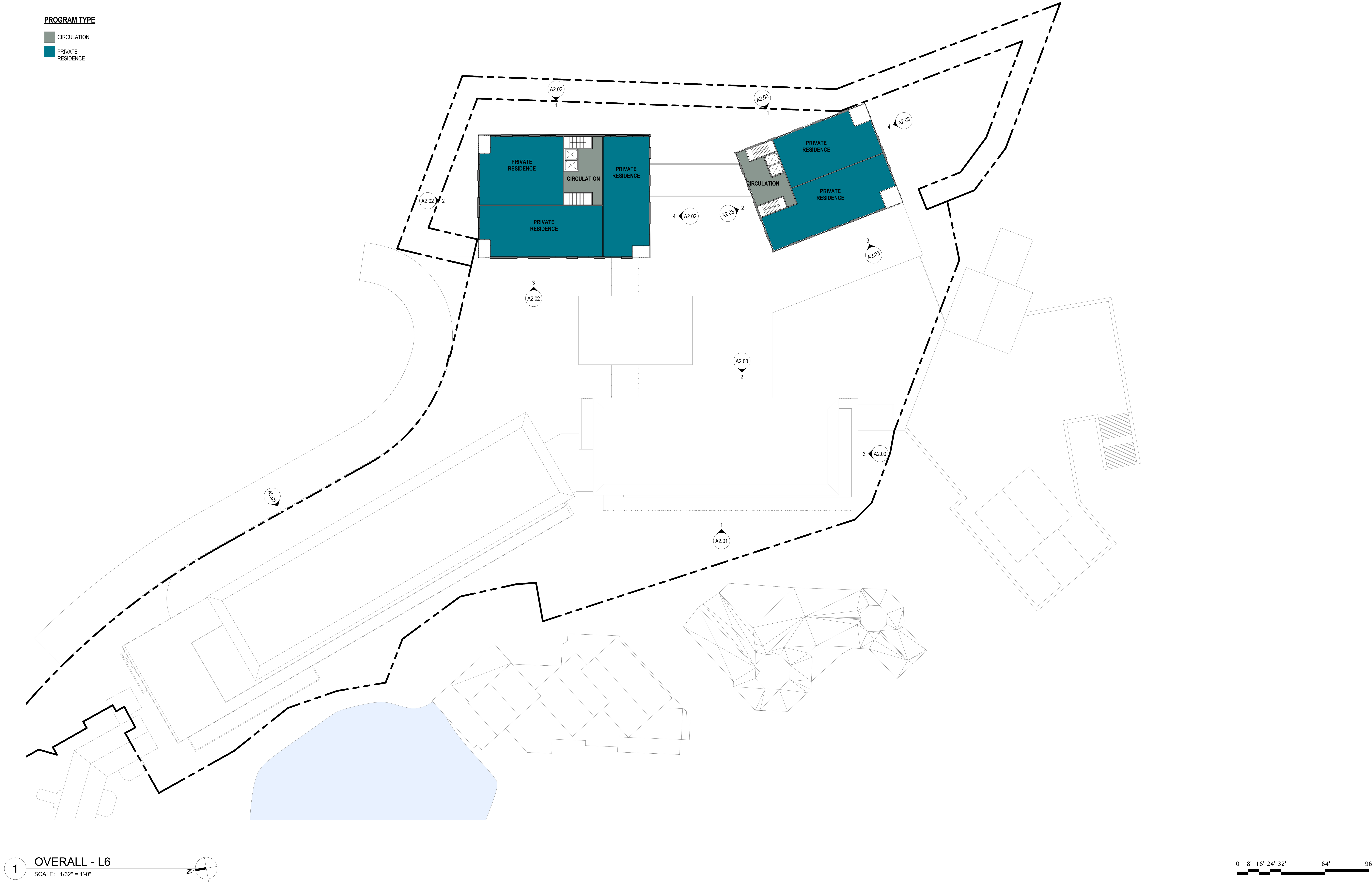

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OVERALL PLAN - LEVEL 6

A1.19

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1 OVERALL - L7
SCALE: 1/32" = 1'-0"

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OVERALL PLAN - LEVEL 7

A1.20

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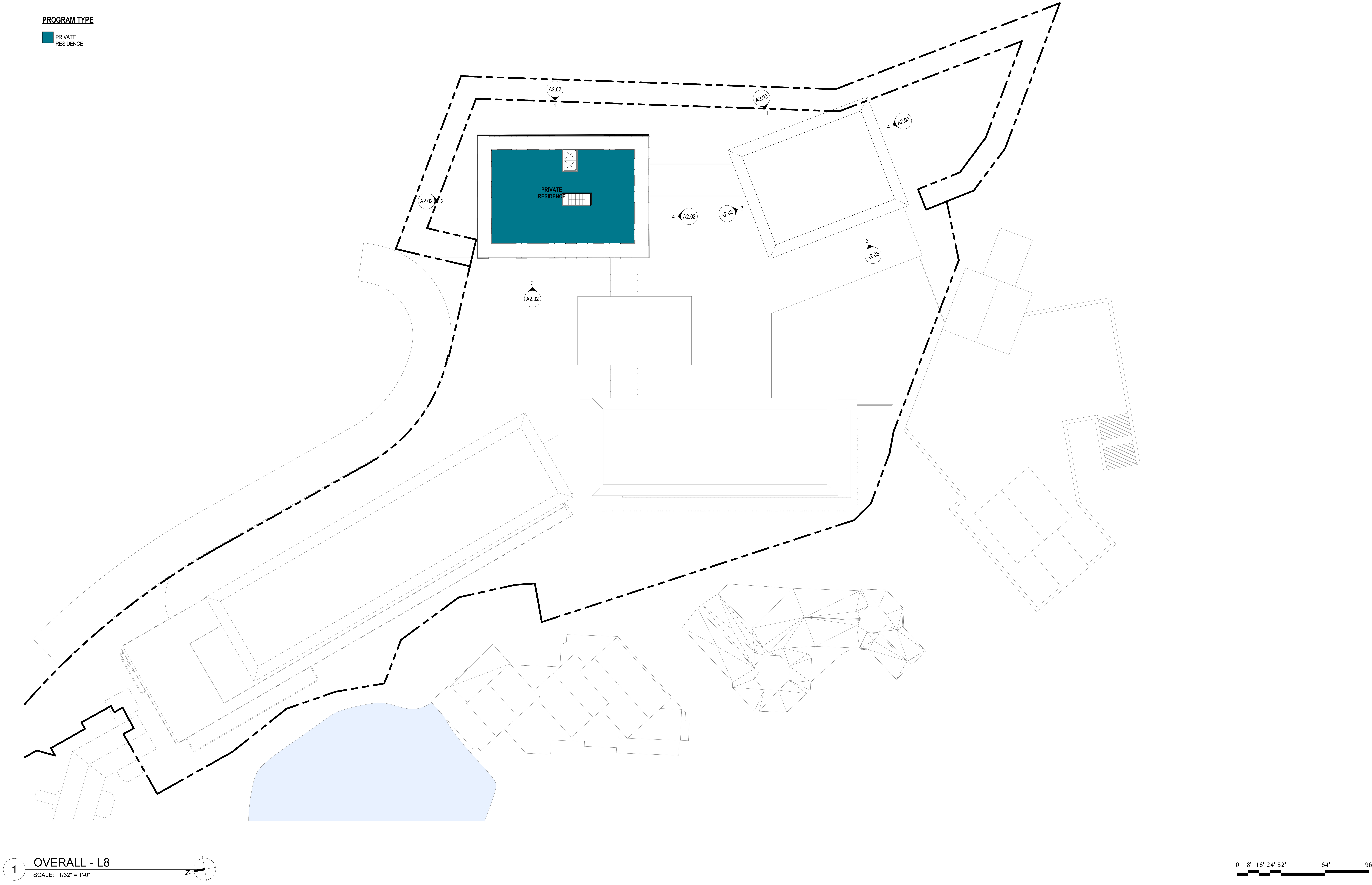

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OVERALL PLAN - LEVEL 8

A1.21

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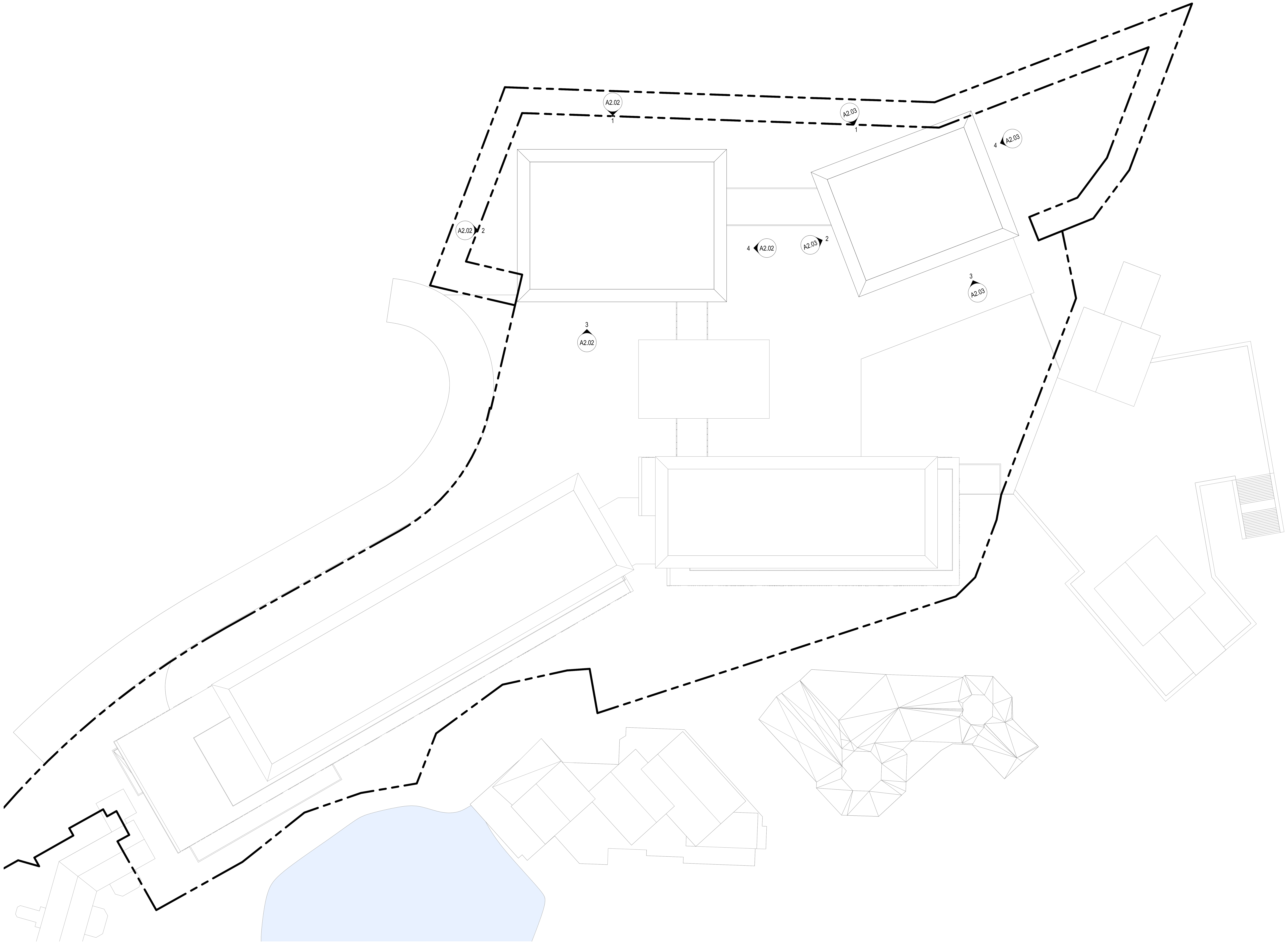

1 OVERALL - ROOF PLAN
SCALE: 1/32" = 1'-0"

0 8' 16' 24' 32' 64' 96'

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OVERALL PLAN - ROOF PLAN

A1.22

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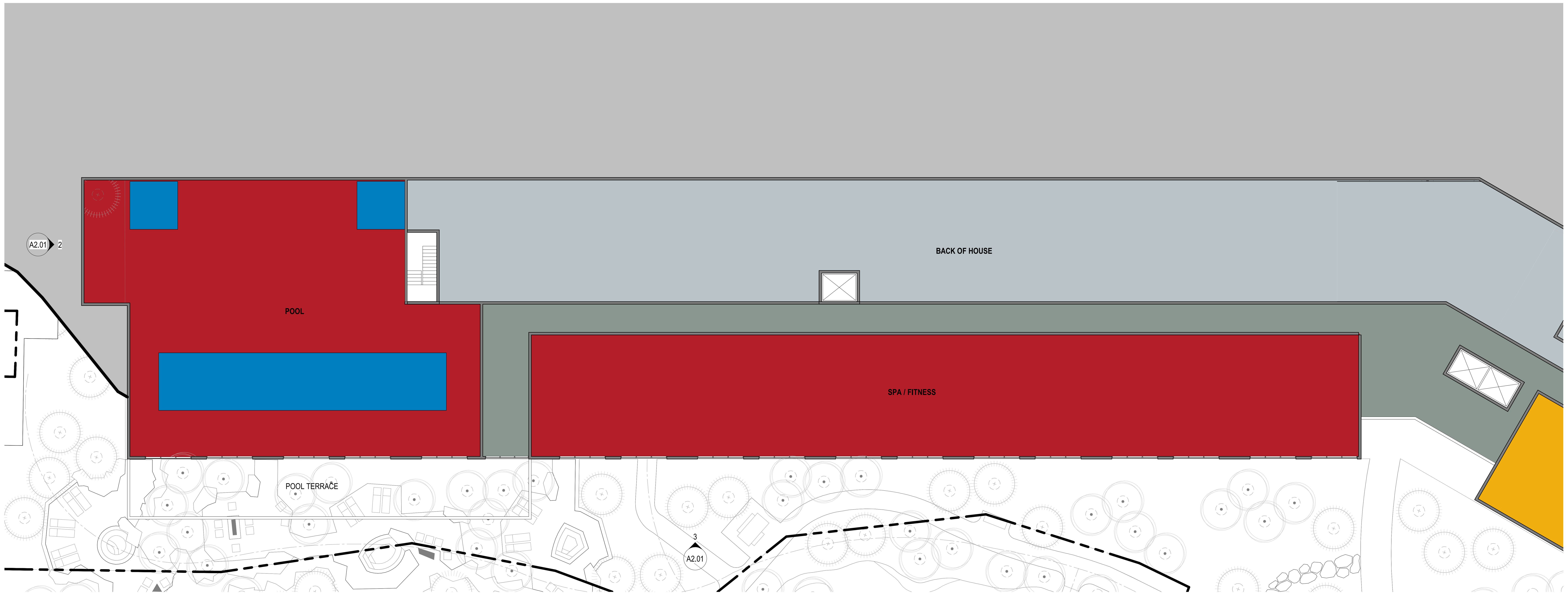
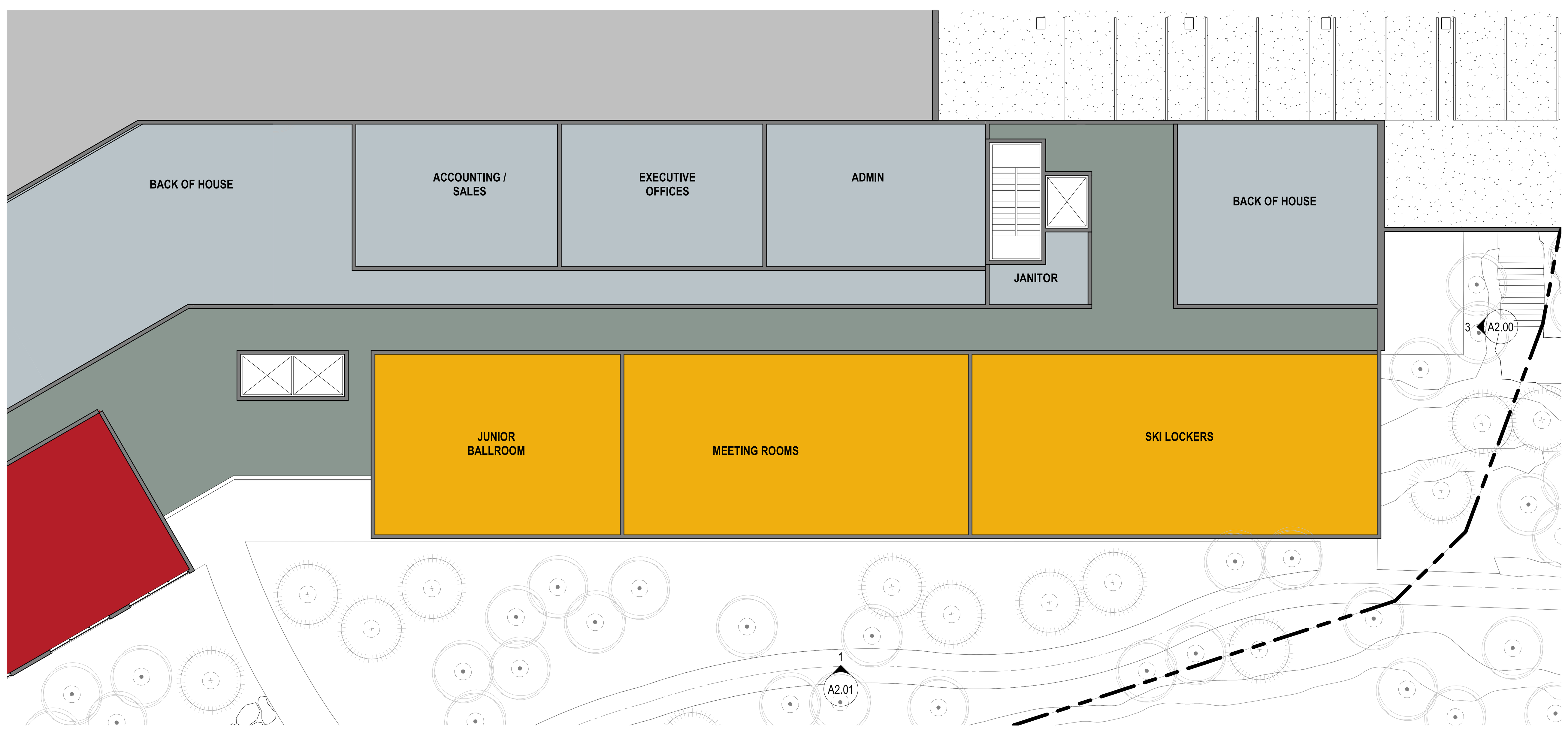
INTERIOR DESIGN



1 HOTEL - B3 - NORTH

SCALE: 1/16" = 1'-0"



2 HOTEL - B3 - SOUTH

SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - B3

A1.31_H

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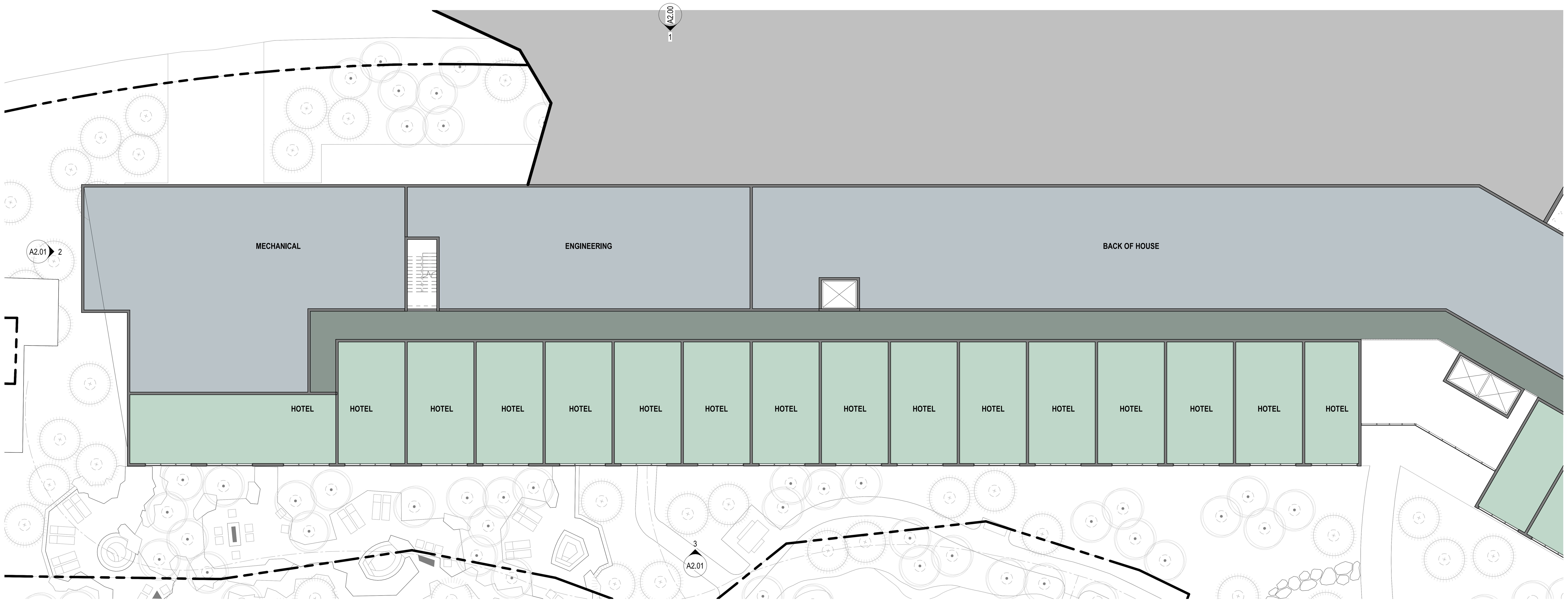
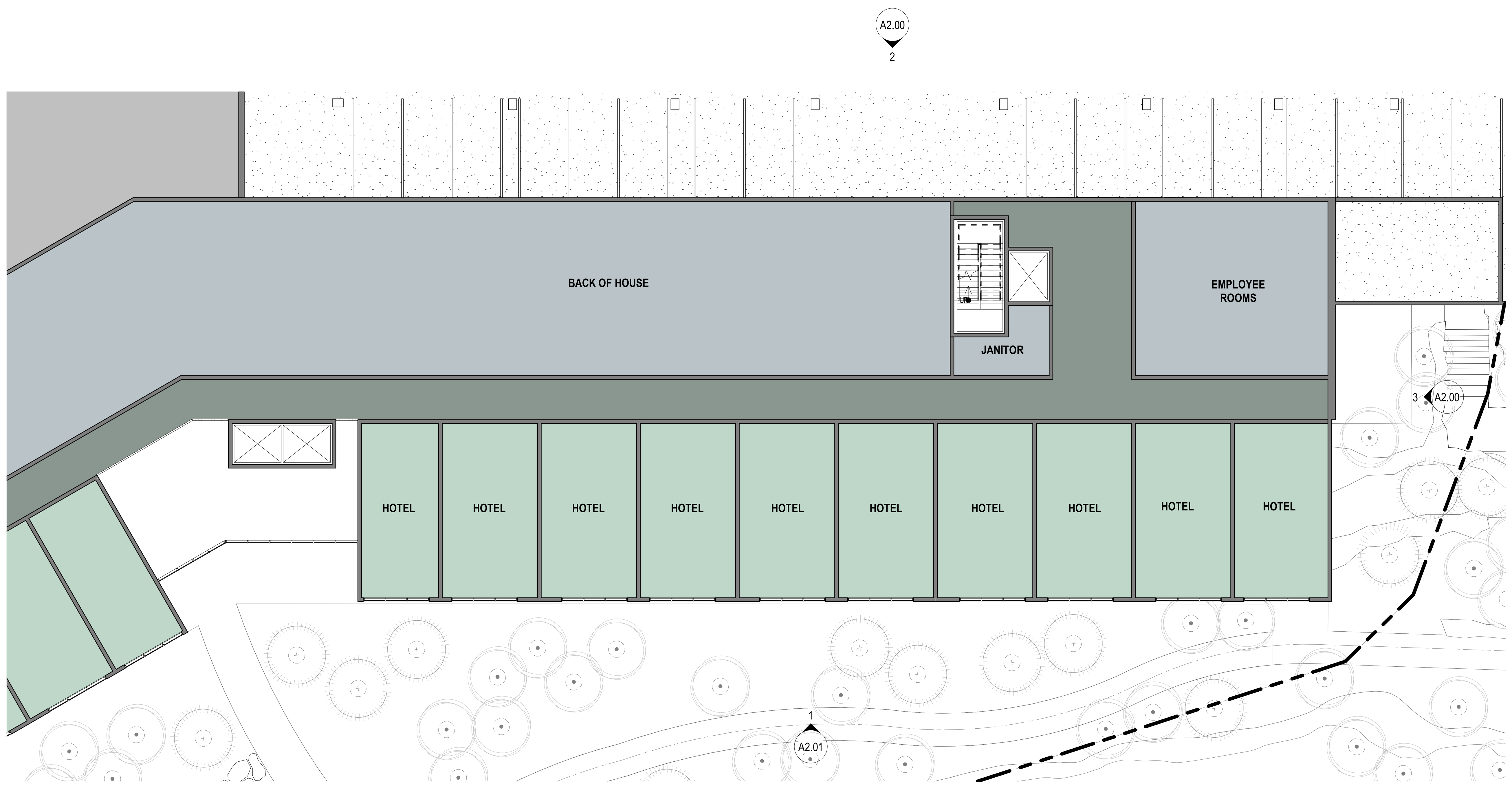
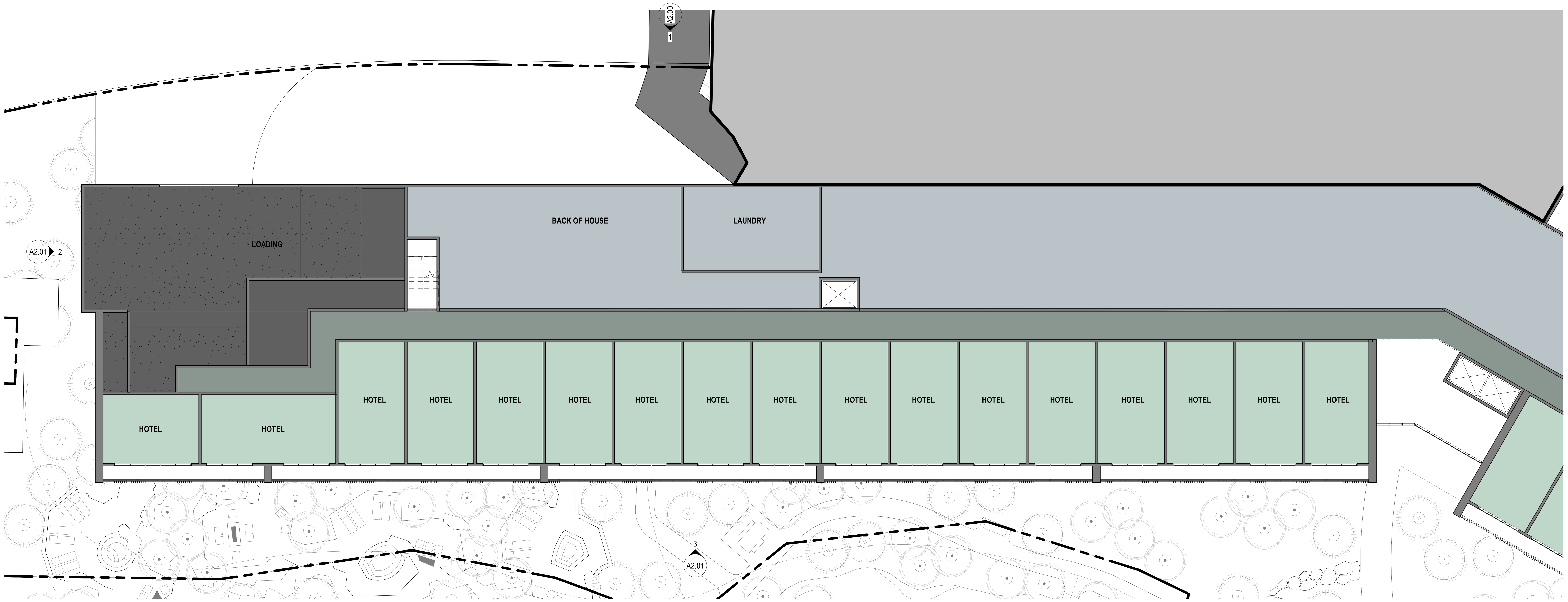


1 HOTEL - B2 - NORTH
SCALE: 1/16" = 1'-0"

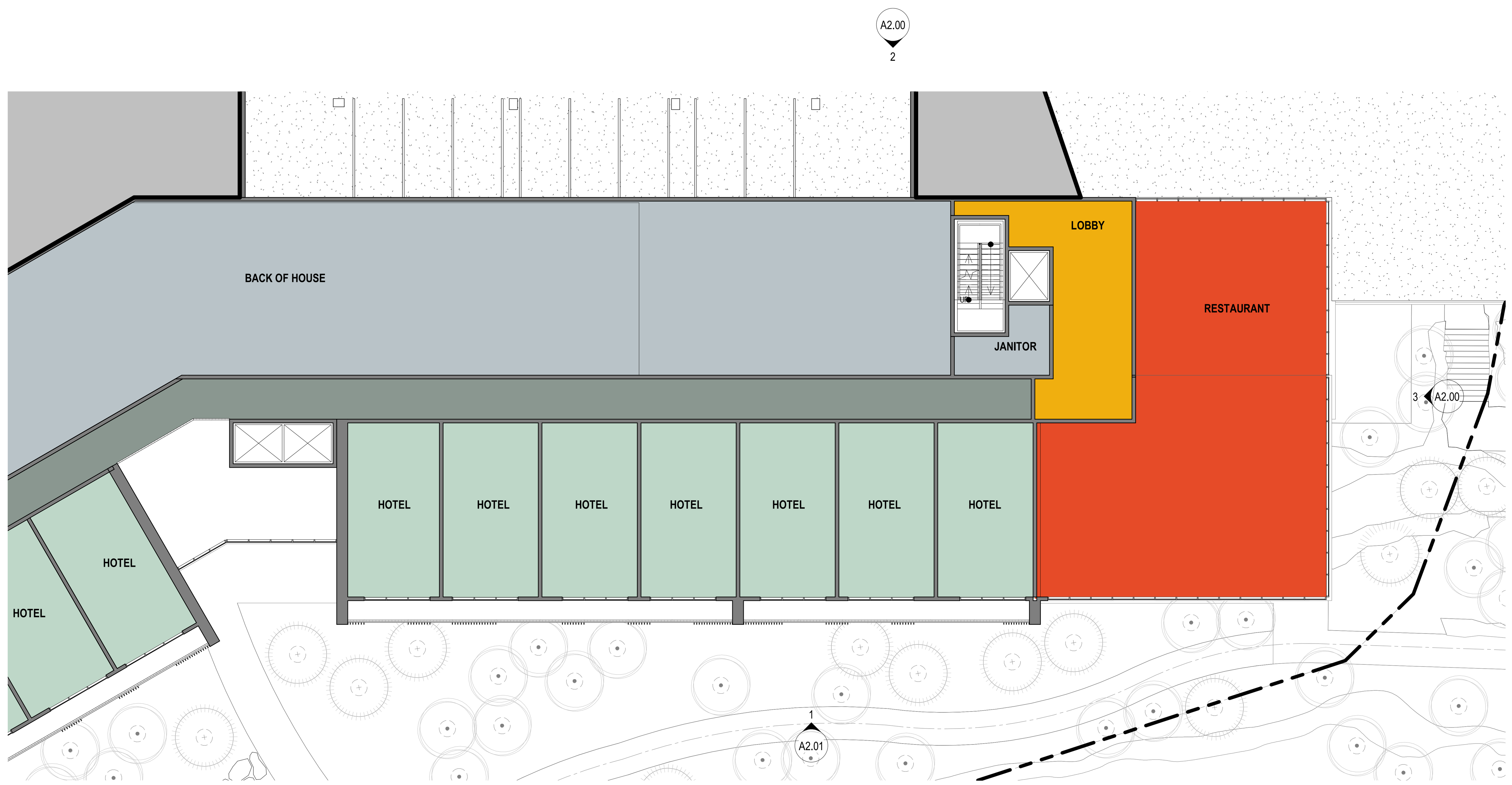


2 HOTEL - B2 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'



1 HOTEL - B1 - NORTH
SCALE: 1/16" = 1'-0"



2 HOTEL - B1 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - B1

A1.33_H

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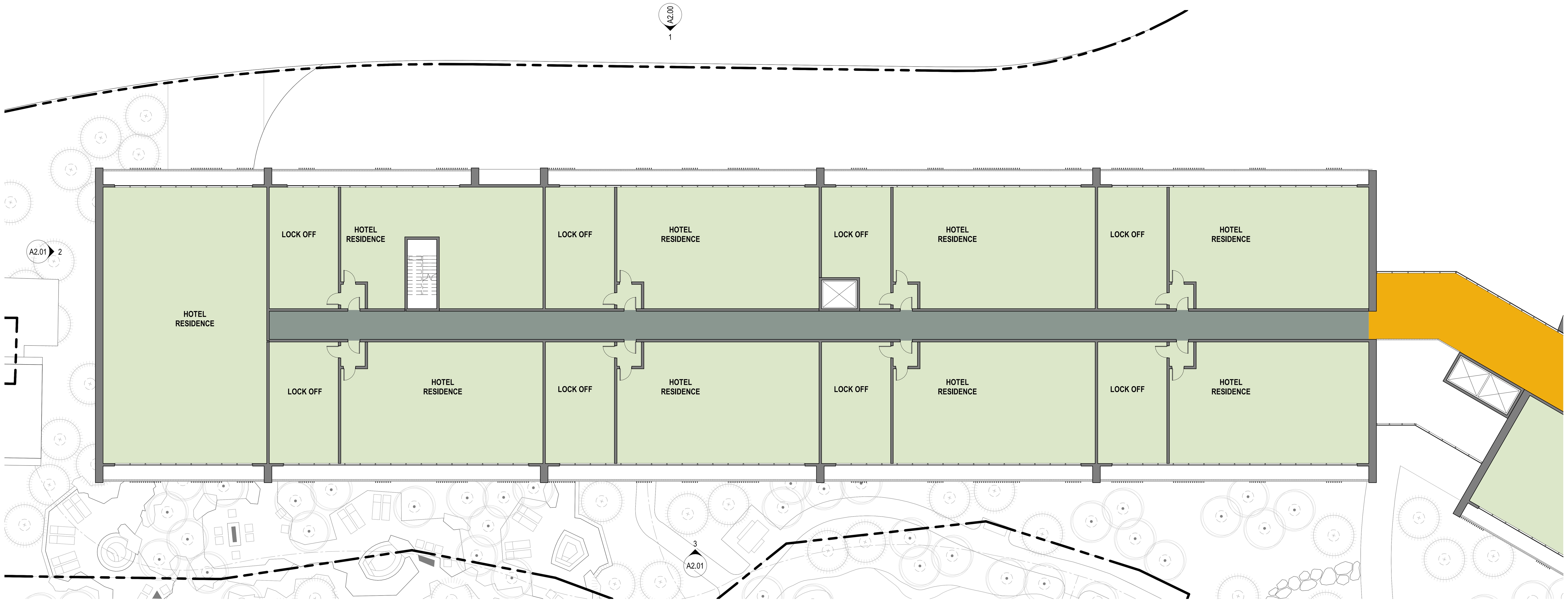
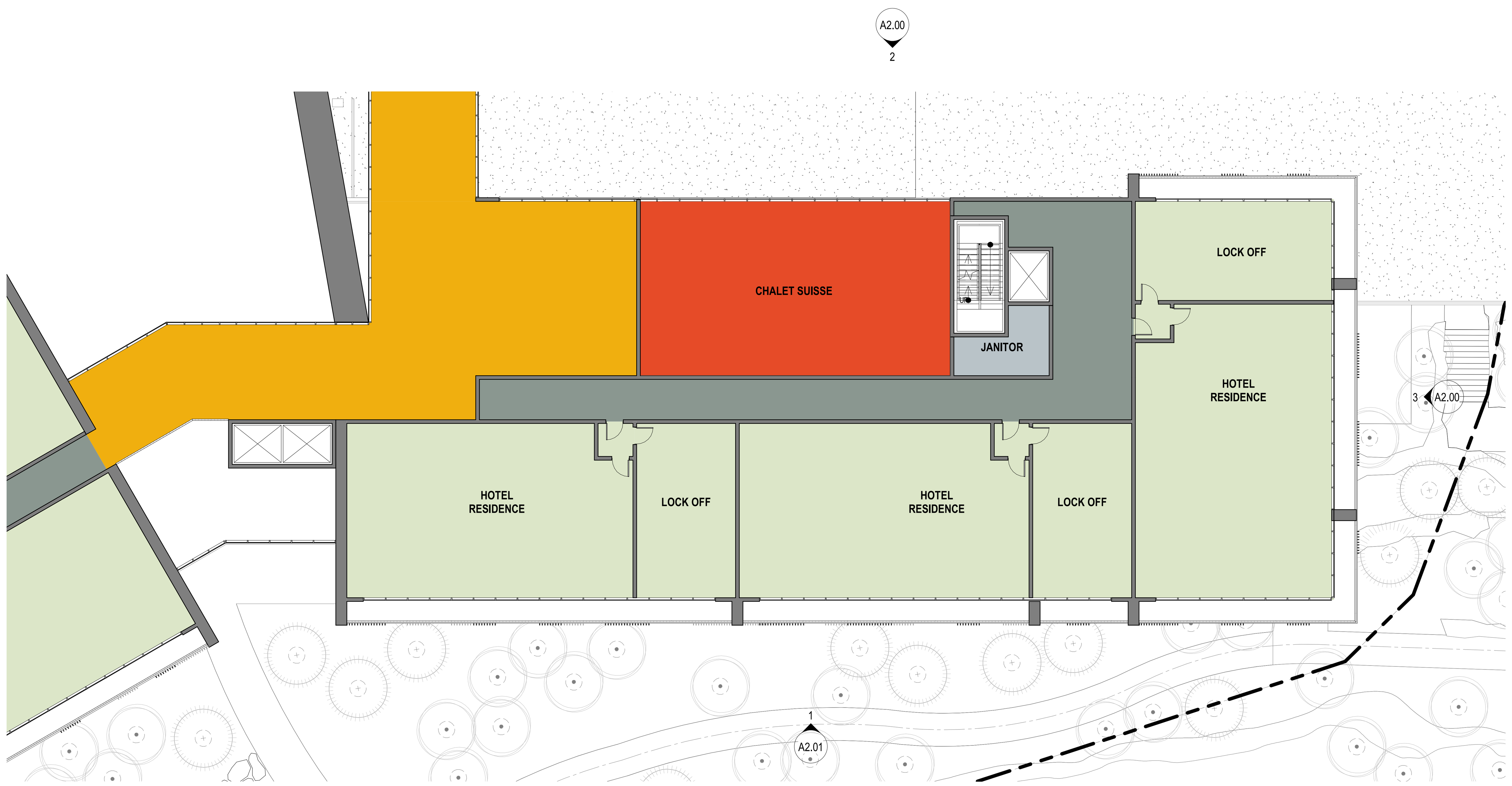


1 HOTEL - L1 - NORTH
SCALE: 1/16" = 1'-0"



2 HOTEL - L1 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - LEVEL 1

A1.34_H

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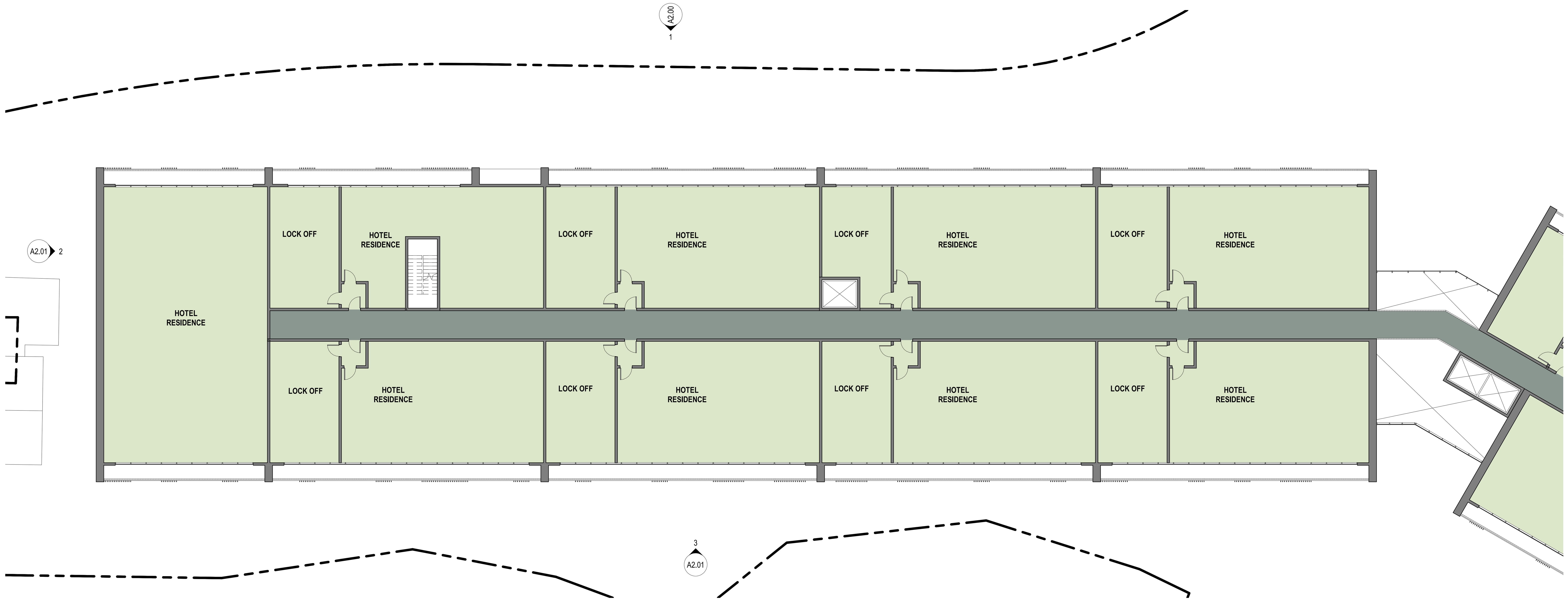
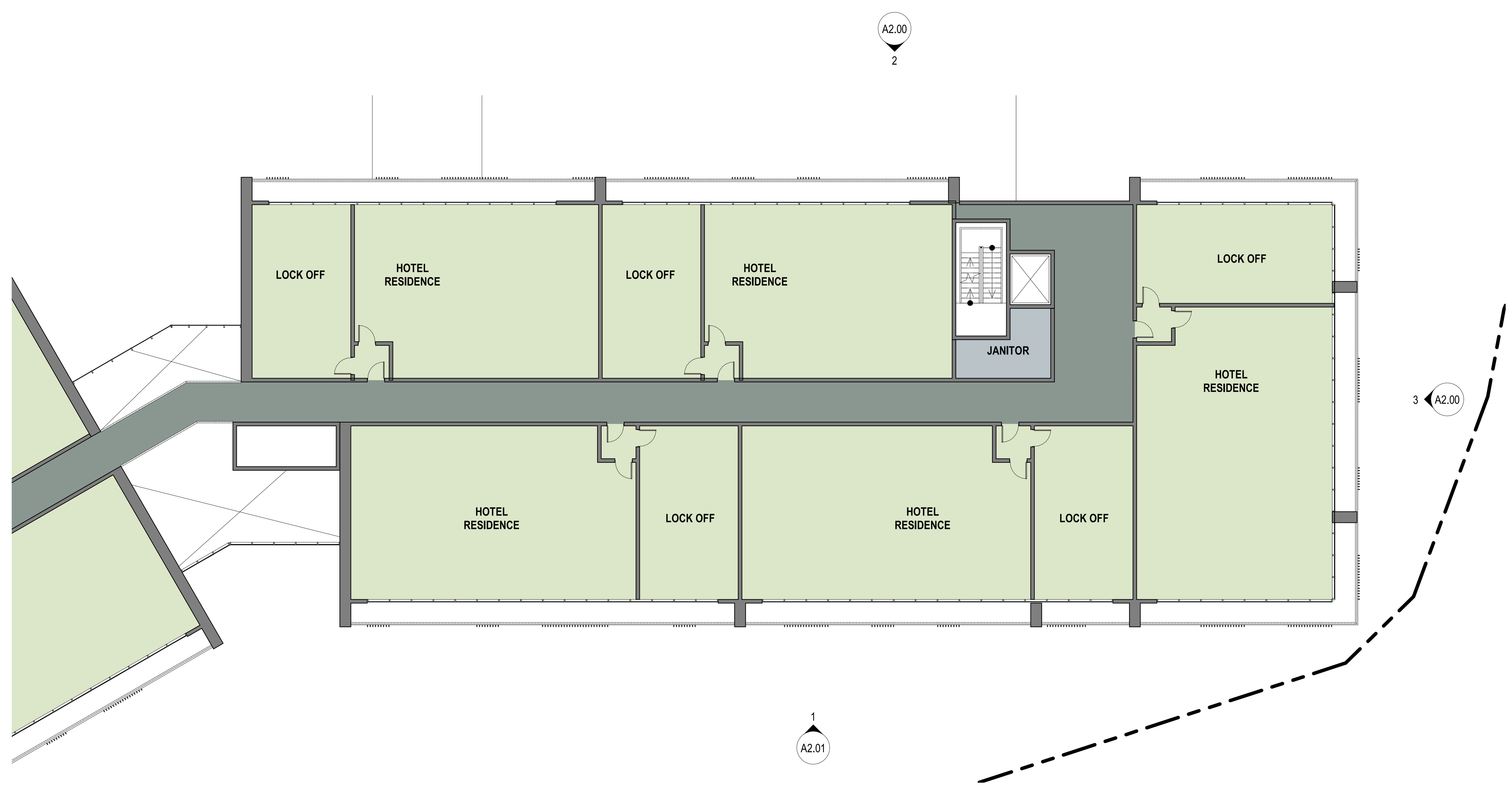


1 HOTEL - L2 - NORTH
SCALE: 1/16" = 1'-0"



2 HOTEL - L2 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - LEVEL 2

A1.35_H

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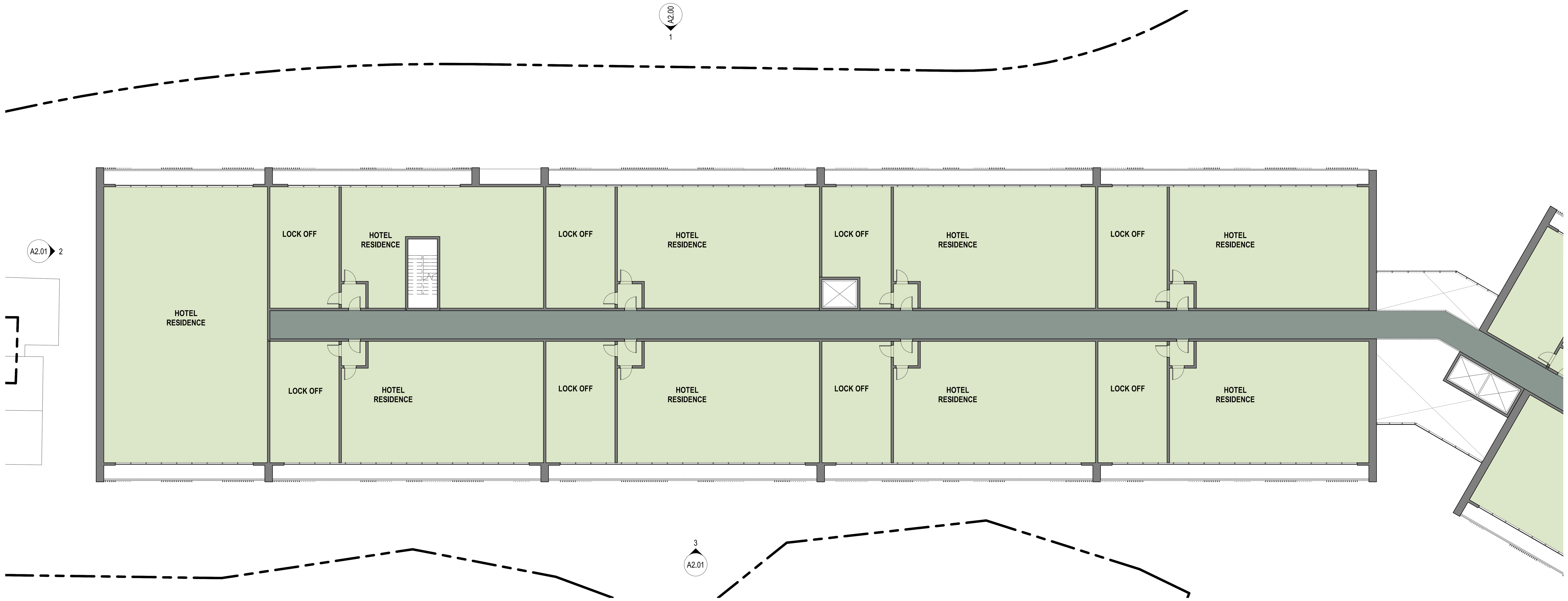
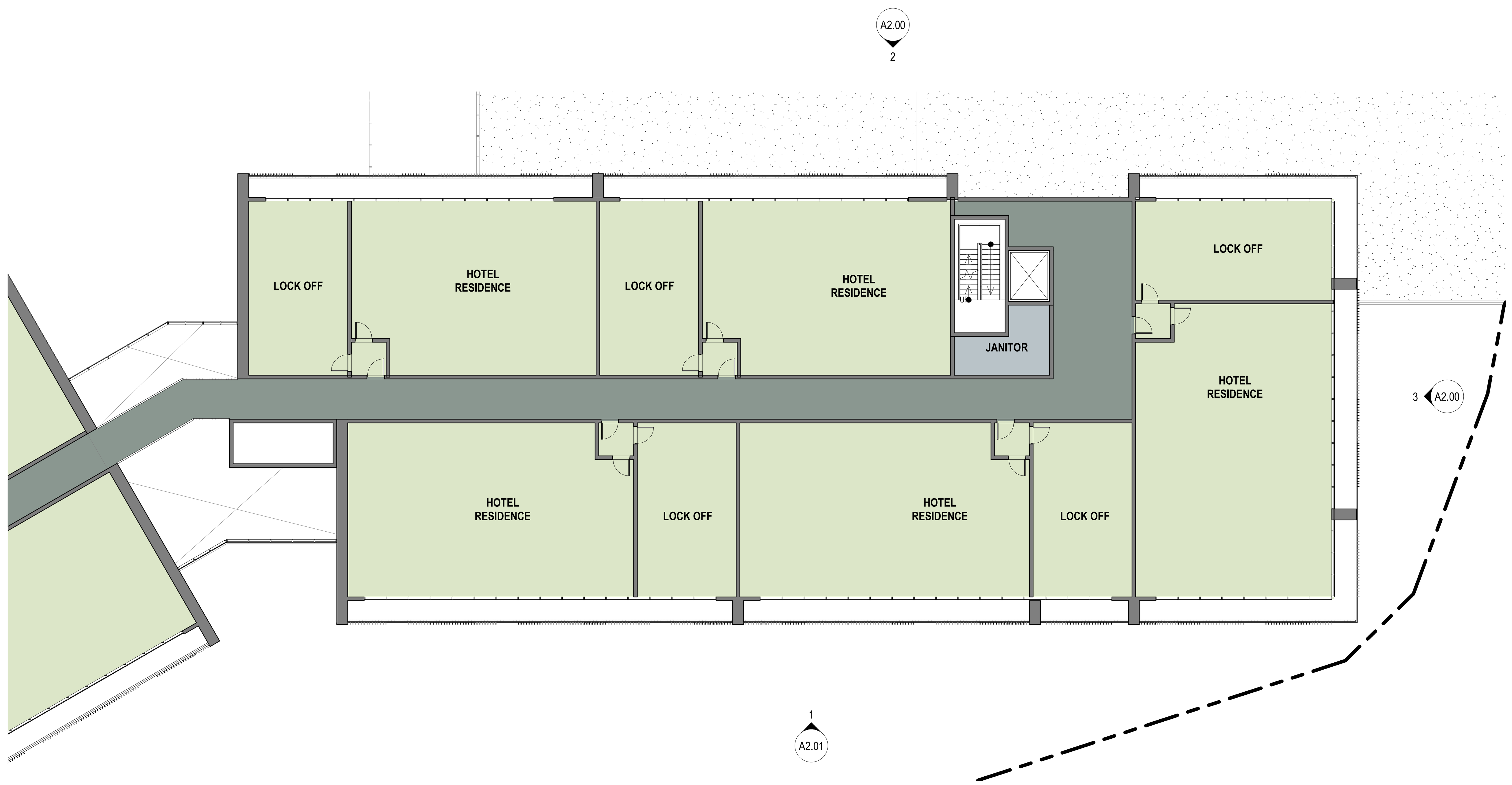
INTERIOR DESIGN



1 HOTEL - L3 - NORTH
SCALE: 1/16" = 1'-0"



2 HOTEL - L3 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - LEVEL 3

A1.36_H

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ARCHITECTURE
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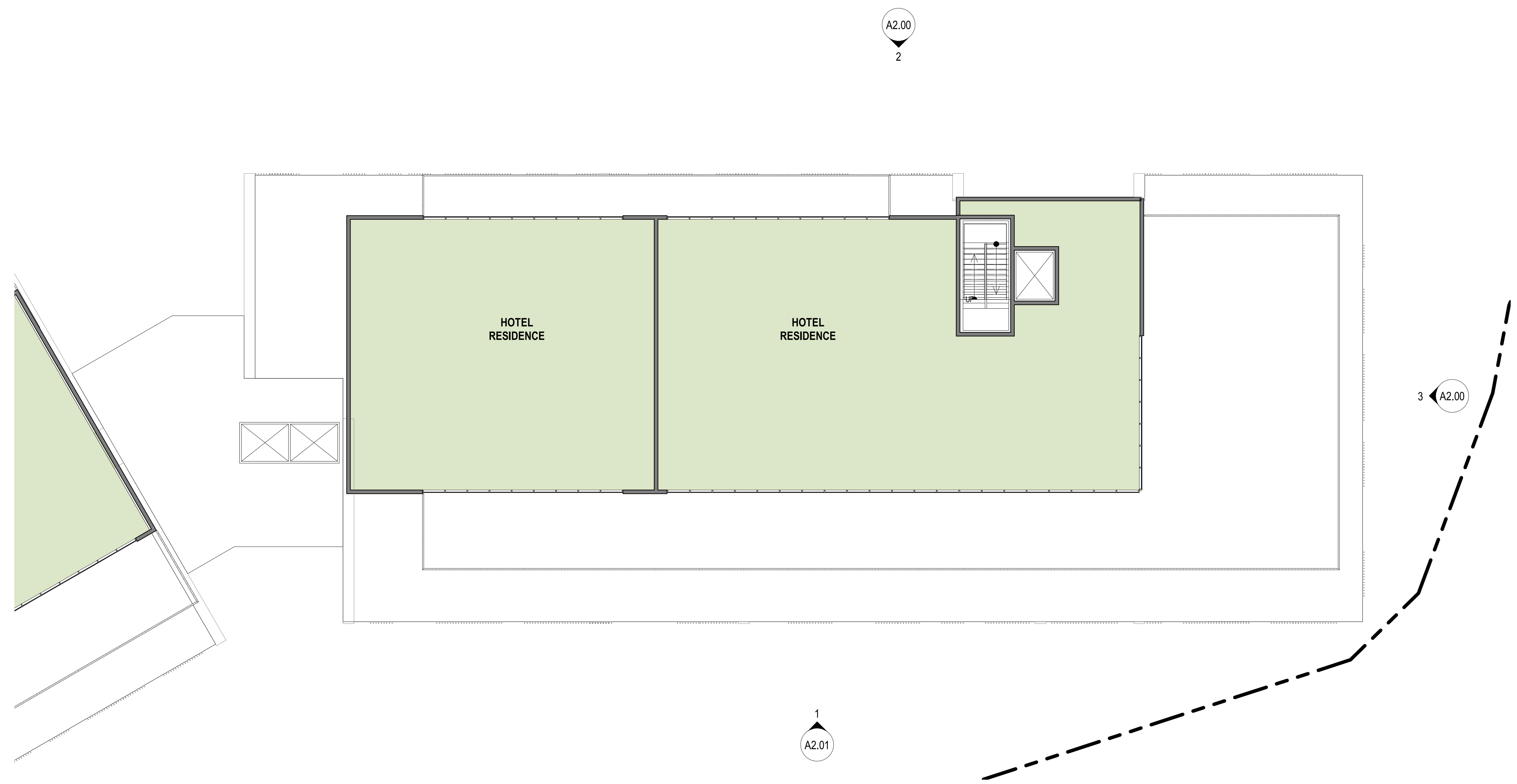
1 HOTEL - L4 - NORTH
SCALE: 1/16" = 1'-0"



2 HOTEL - L4 - SOUTH
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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HOTEL - FLOOR PLAN - LEVEL 4

A1.37_H

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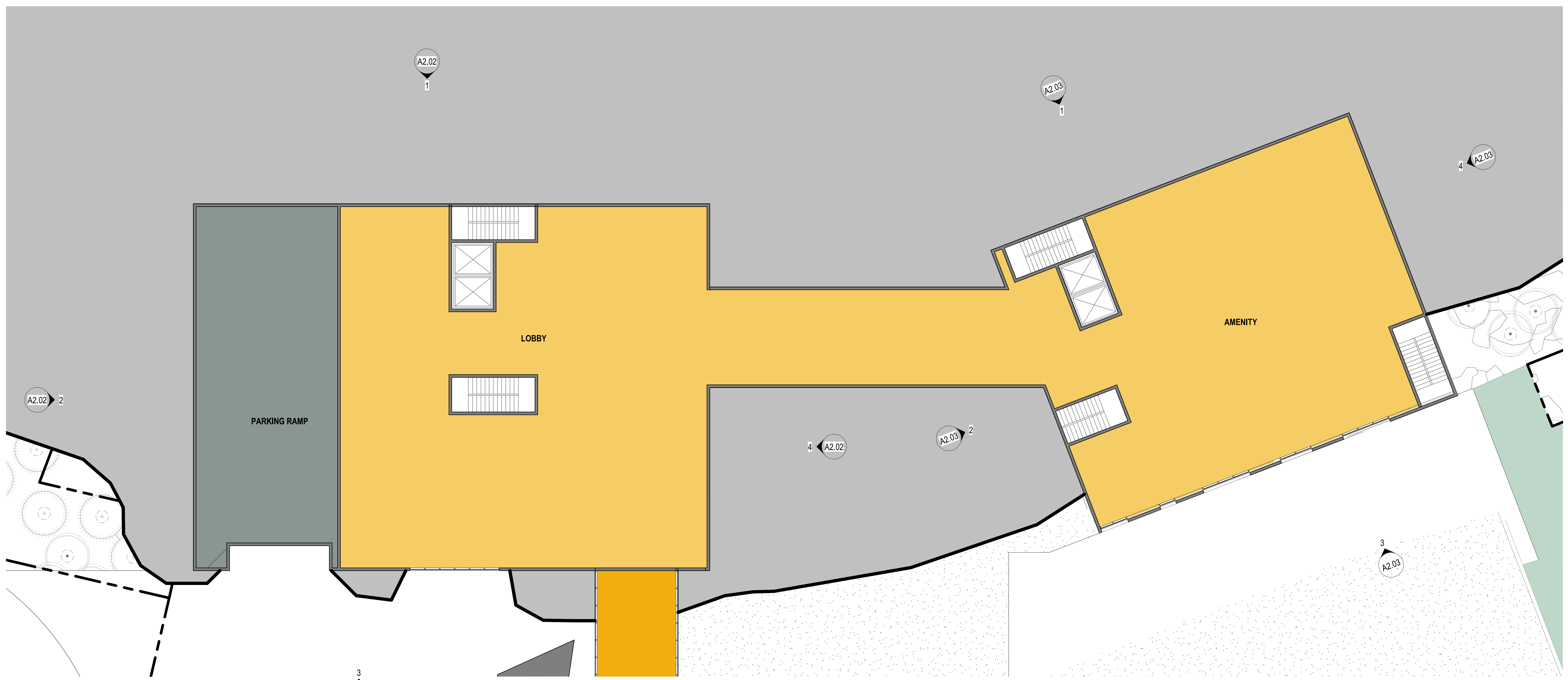

1 PR - L1
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

1 PR - L2
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

1 PR - L3
SCALE: 1/16" = 1'-0"

0 4' 8' 12' 16' 32' 48'

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PRIVATE RESIDENCES - LEVEL 3

A1.43_T

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PRIVATE RESIDENCES - LEVEL 4

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PRIVATE RESIDENCES - LEVEL 5

A1.45_T

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1 PR - L6
SCALE: 1/16" = 1'-0"

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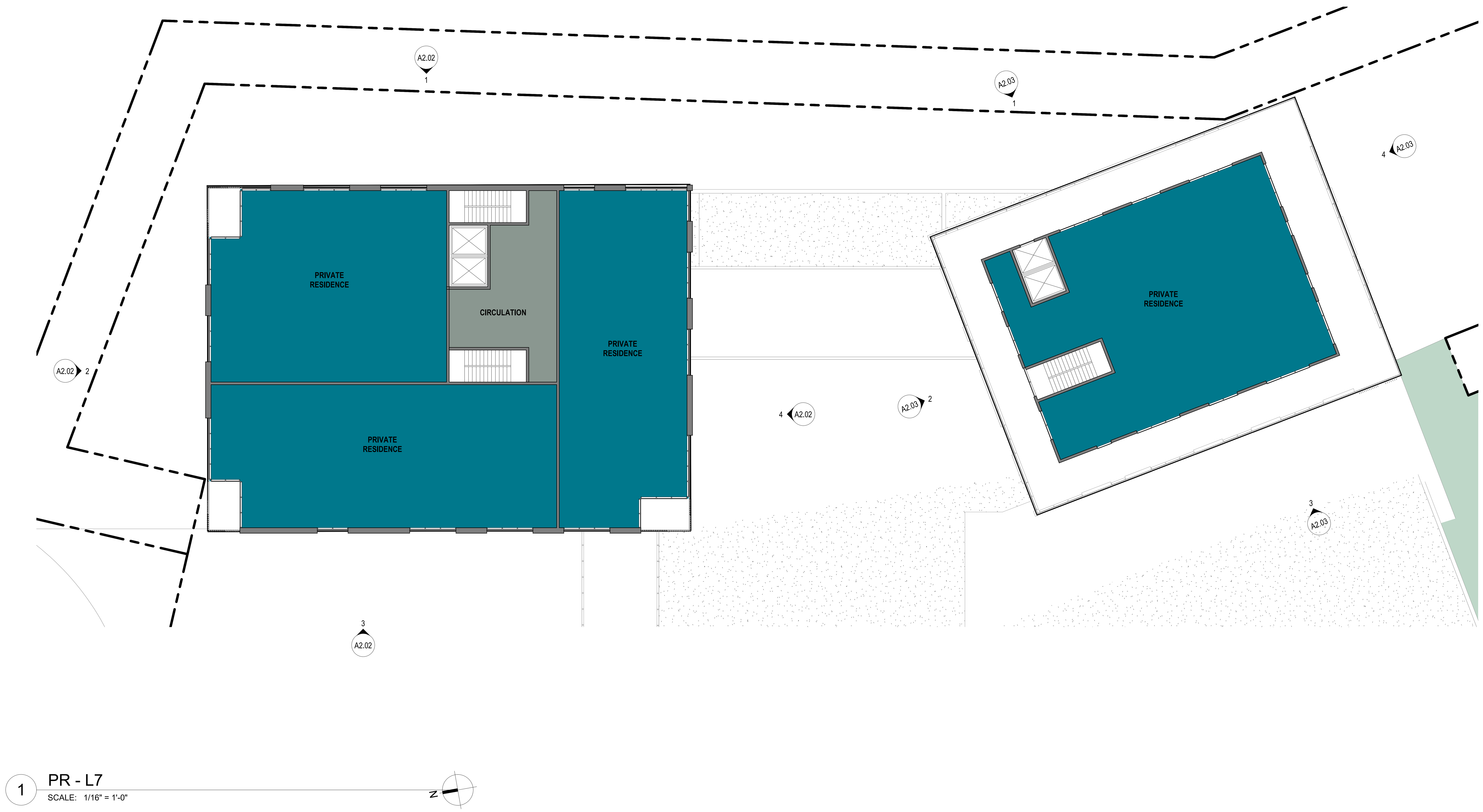

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PRIVATE RESIDENCES - LEVEL 7

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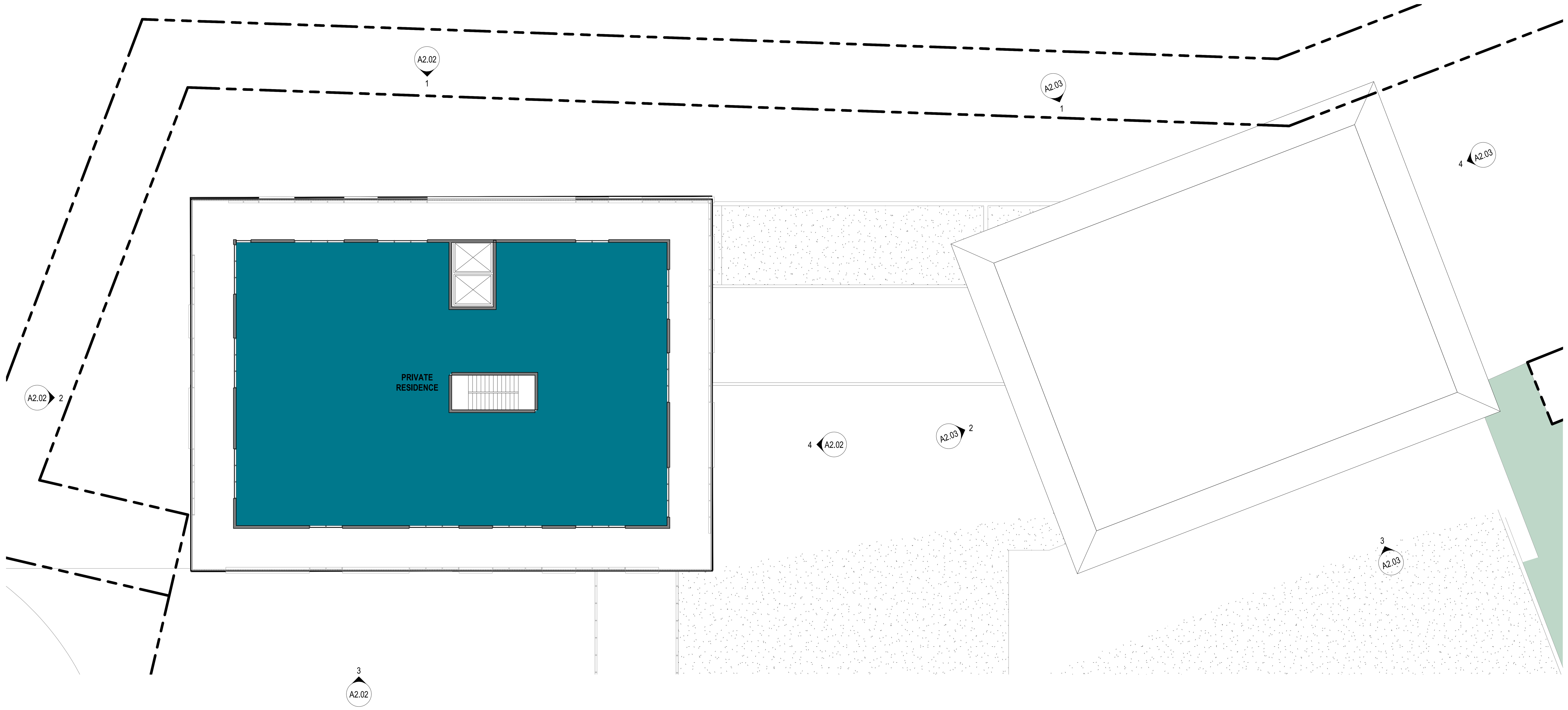

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PRIVATE RESIDENCES - LEVEL 8

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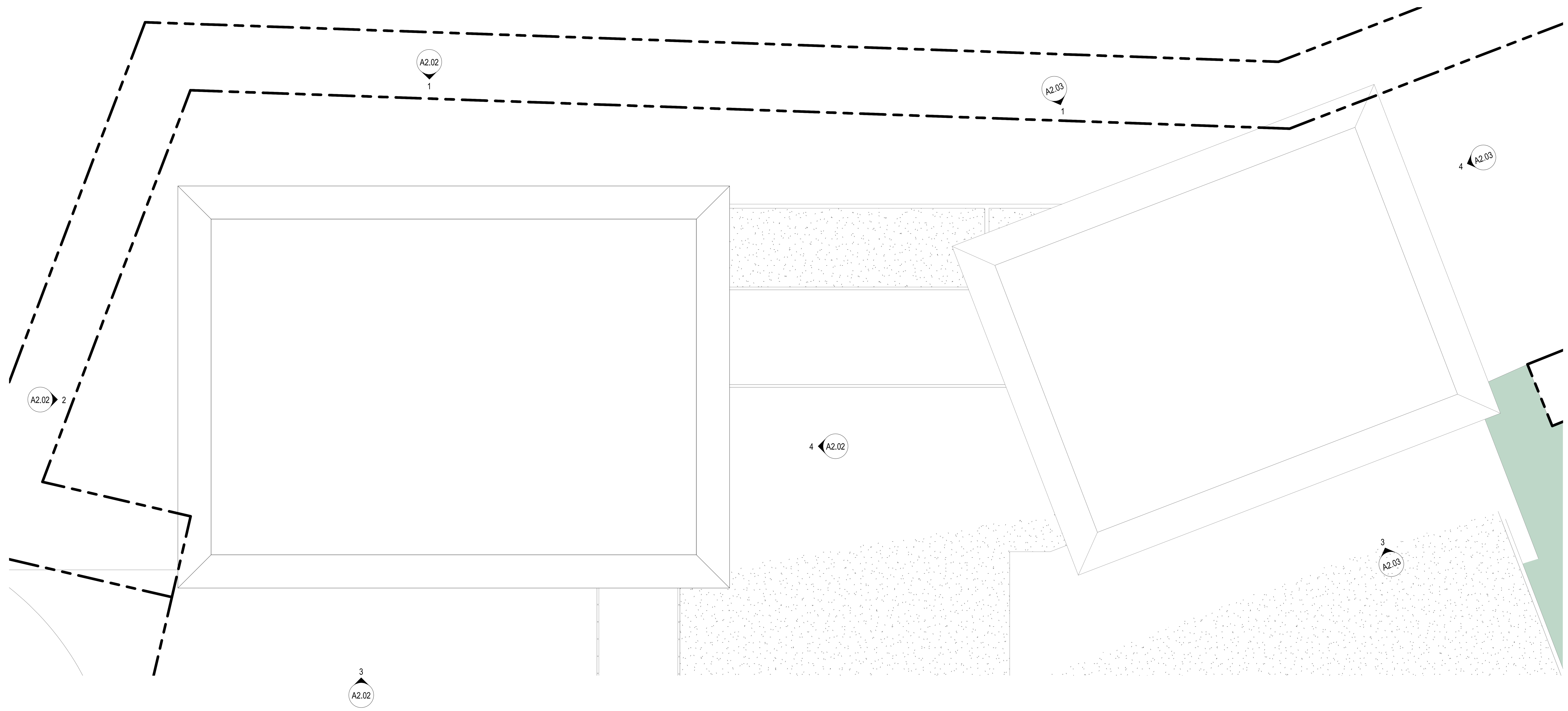

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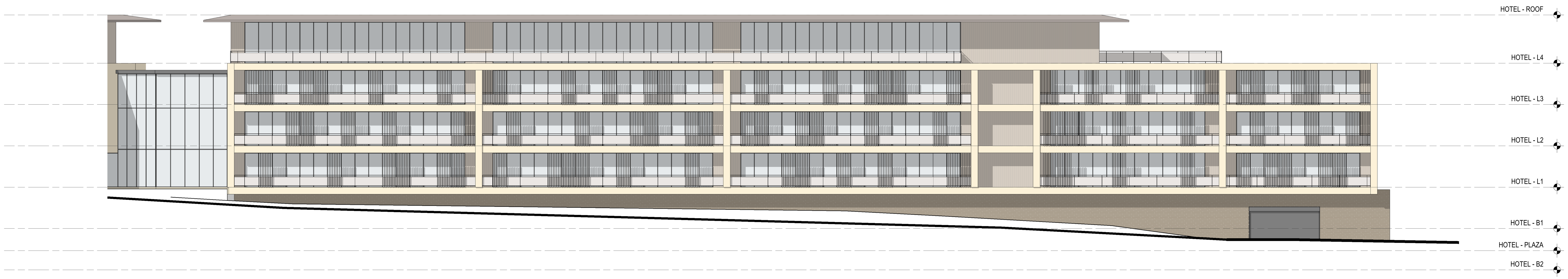

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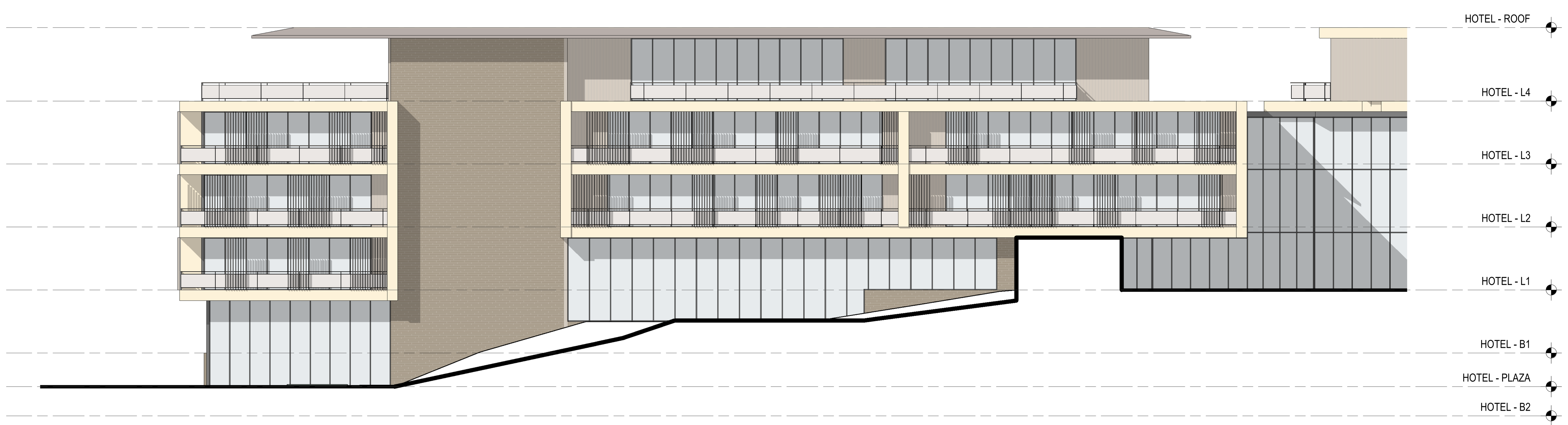

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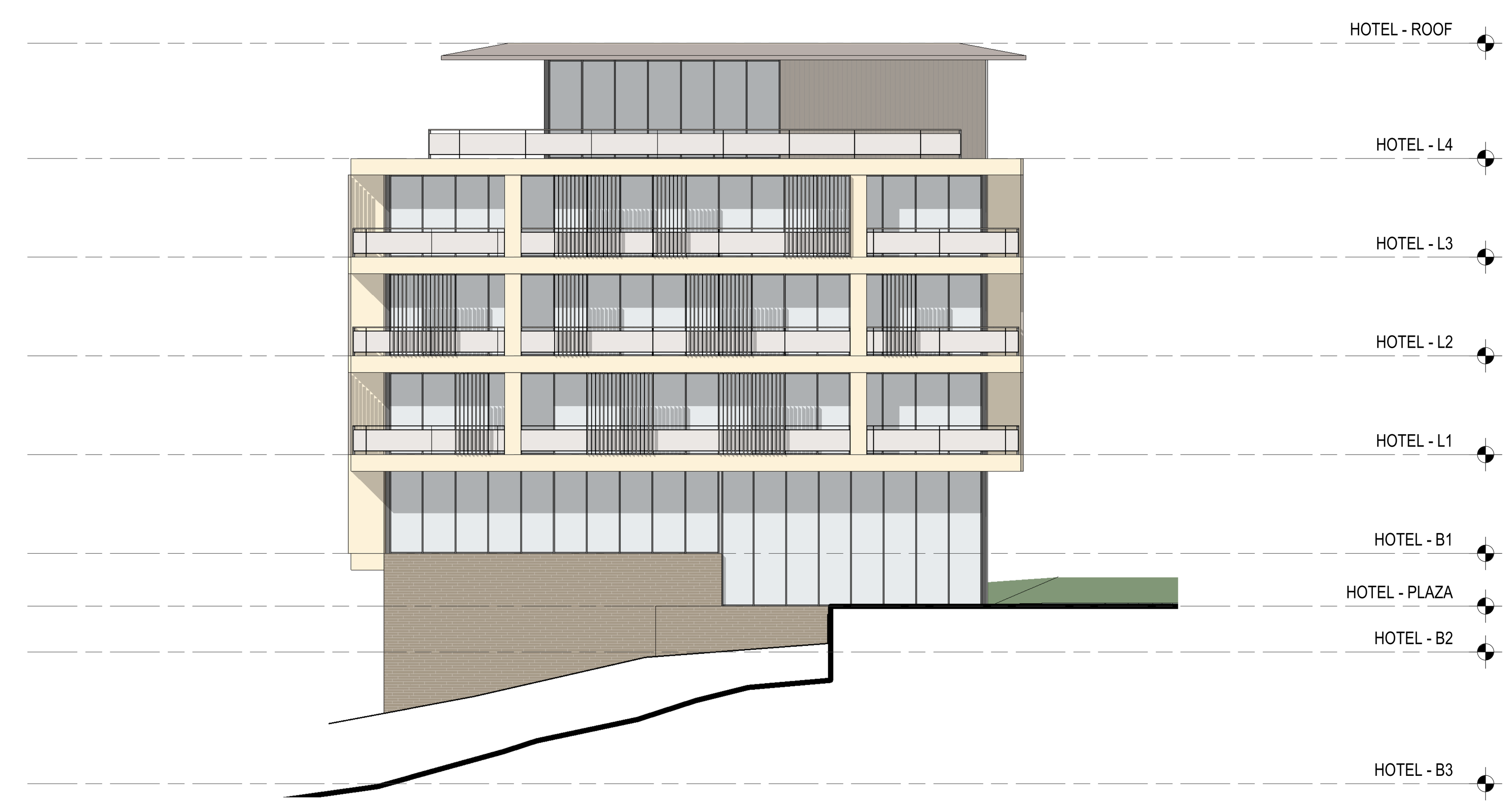

3 ELEVATION - HOTEL S - SOUTH
SCALE: 1/16" = 1'-0"

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EXTERIOR ELEVATIONS

A2.00

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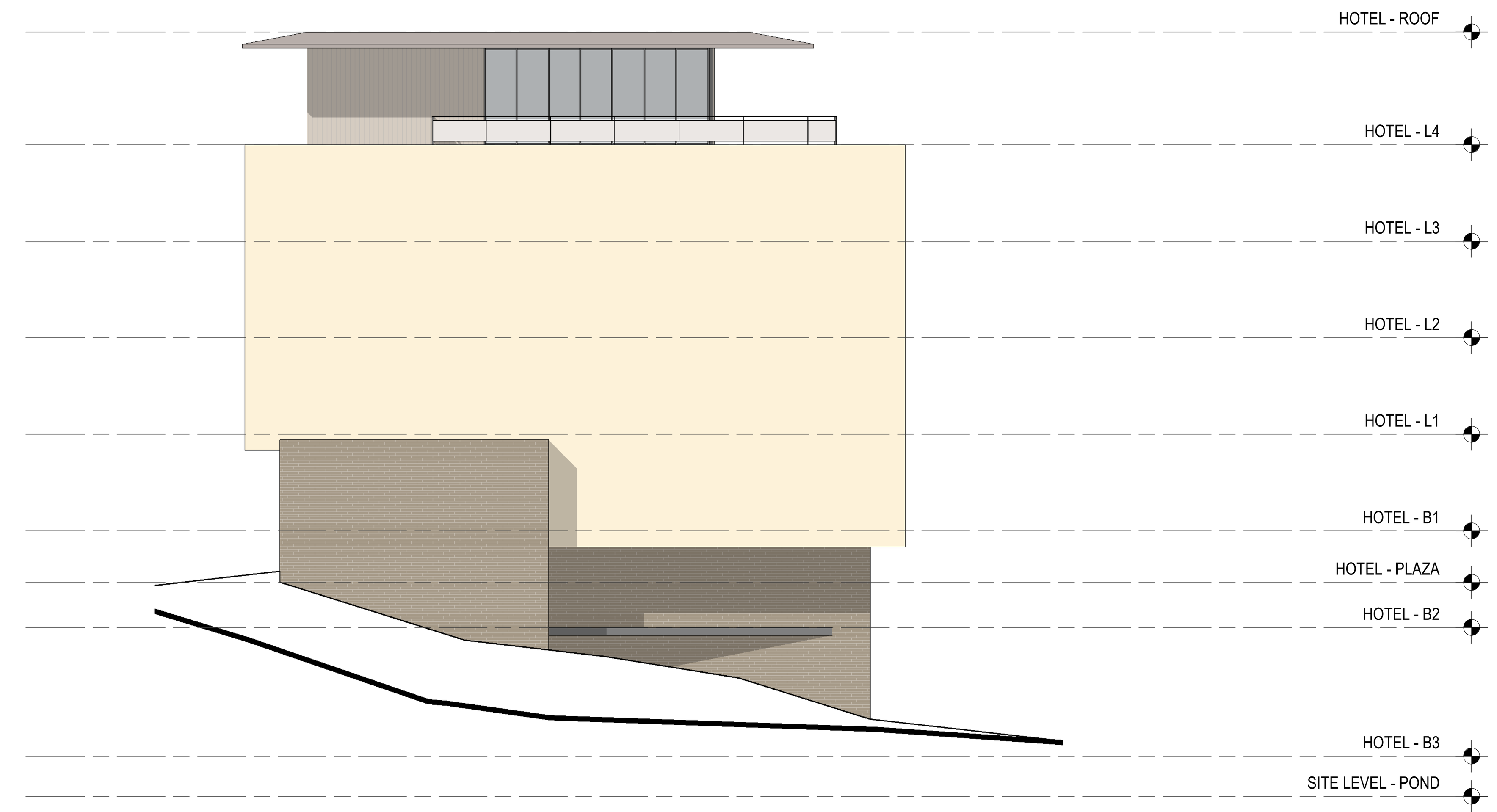
1 ELEVATION - HOTEL S - WEST
SCALE: 1/16" = 1'-0"



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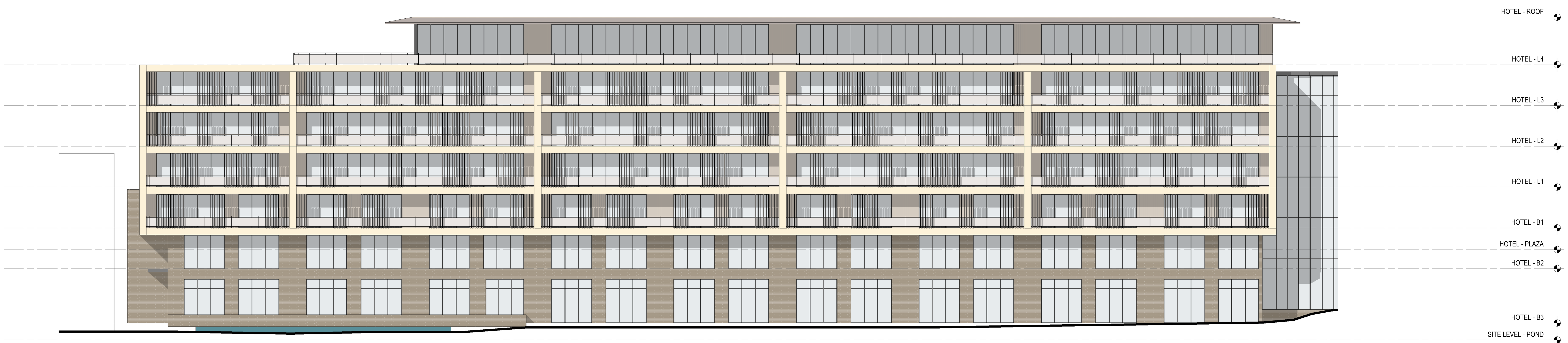

3 ELEVATION - HOTEL N - WEST
SCALE: 1/16" = 1'-0"

EXTERIOR ELEVATIONS

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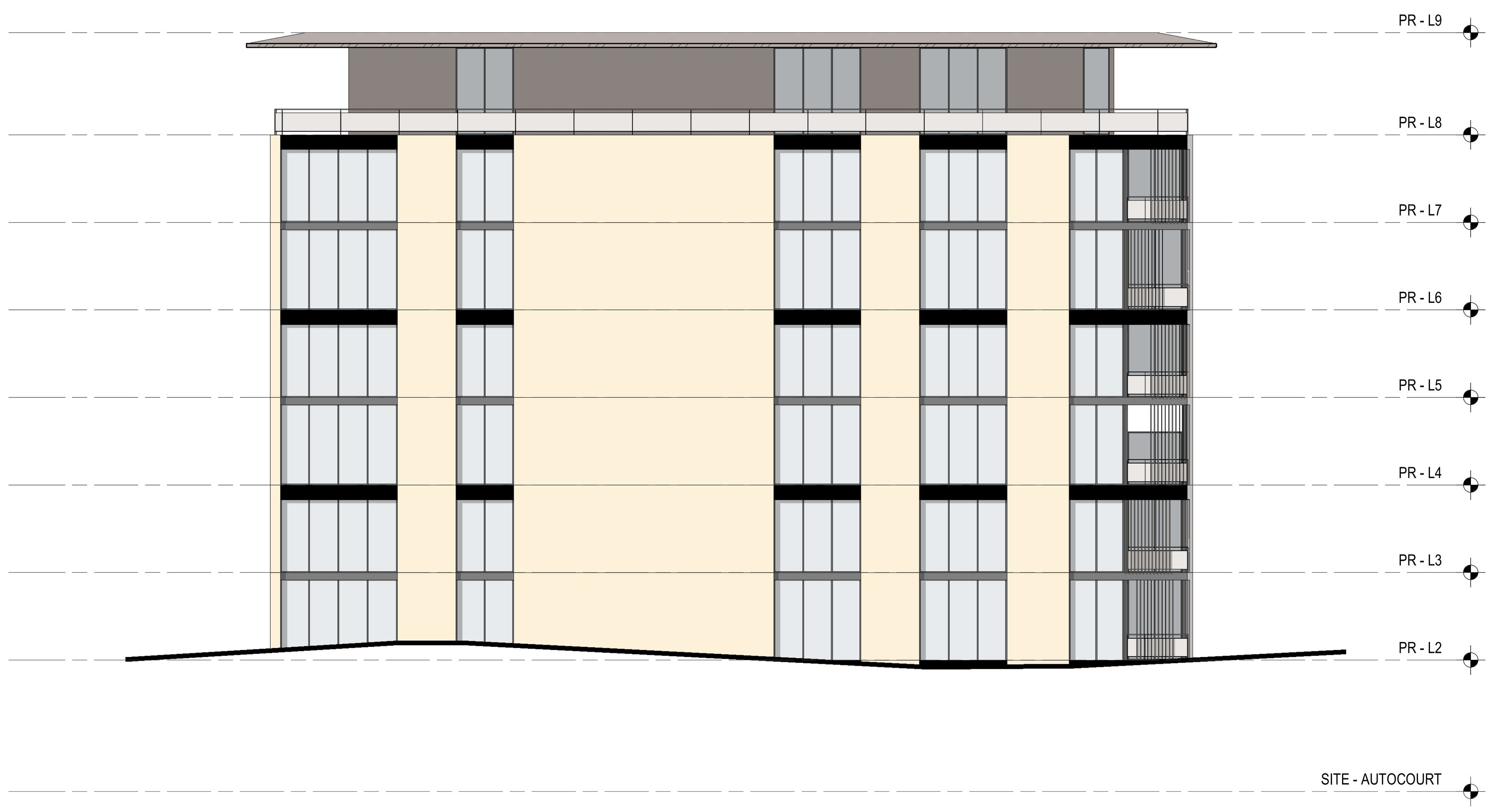

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2 ELEVATION - PRIVATE RESIDENCE N - NORTH
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3 ELEVATION - PRIVATE RESIDENCE N - WEST
SCALE: 1/16" = 1'-0"

4 ELEVATION - PRIVATE RESIDENCE N - SOUTH
SCALE: 1/16" = 1'-0"

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EXTERIOR ELEVATIONS

A2.02

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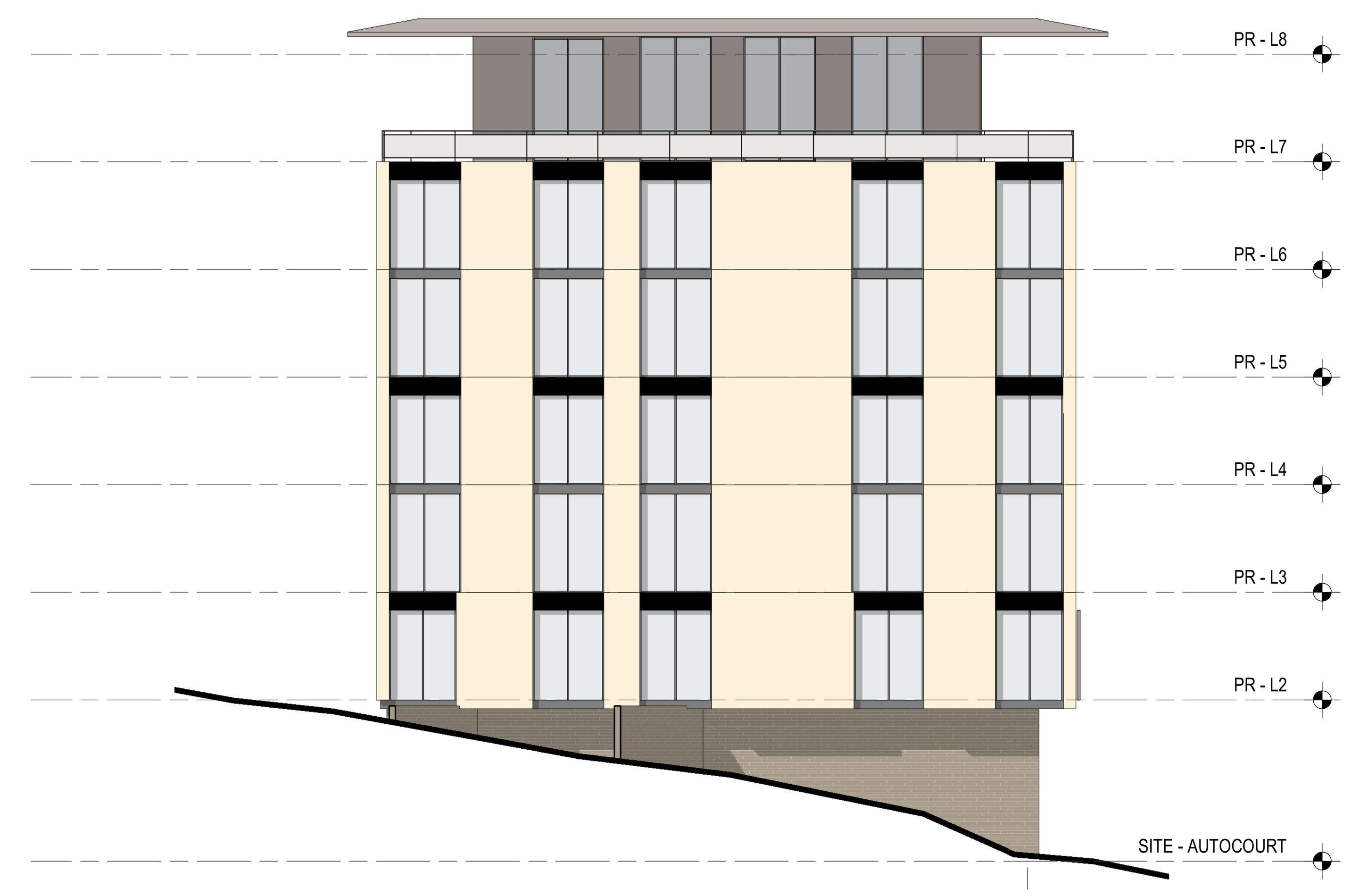
1 ELEVATION - PRIVATE RESIDENCE S - EAST
SCALE: 1/16" = 1'-0"



2 ELEVATION - PRIVATE RESIDENCE S - NORTH
SCALE: 1/16" = 1'-0"

3 ELEVATION - PRIVATE RESIDENCE S - WEST
SCALE: 1/16" = 1'-0"

4 ELEVATION - PRIVATE RESIDENCE S - SOUTH
SCALE: 1/16" = 1'-0"

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EXTERIOR ELEVATIONS

A2.03

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1 Site Plan - Height Analysis - Existing Grades

SCALE: 1" = 30'-0"

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Height Analysis - Existing Grades

A3.00

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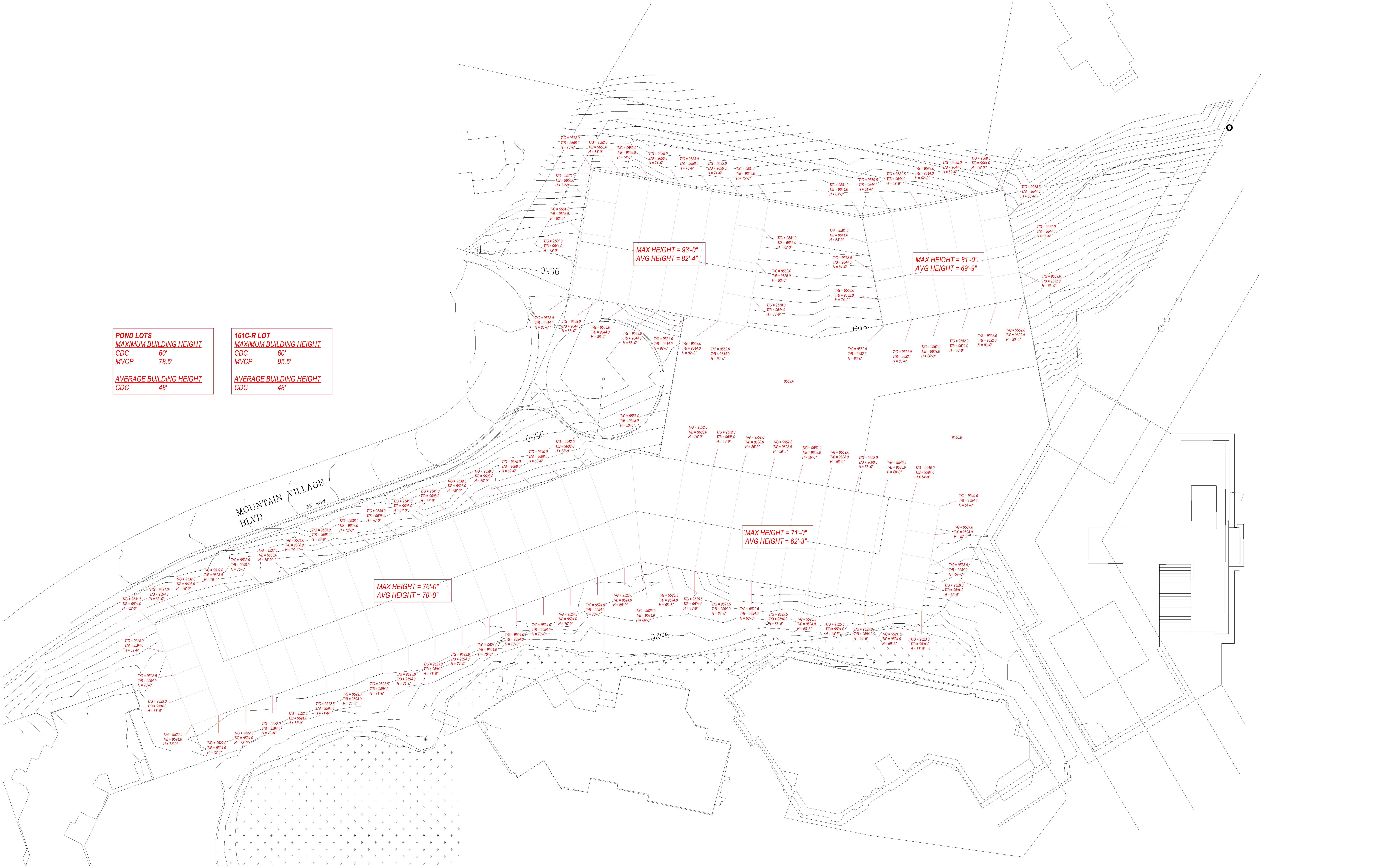

1 Site Plan - Height Analysis - Proposed Grades

SCALE: 1" = 30'-0"

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Height Analysis - Proposed Grades

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A3.01

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GEOTECHNICAL ENGINEERING INVESTIGATION FOR SHORING AND FOUNDATIONS

LOT 161C-R
MOUNTAIN VILLAGE, COLORADO

MFG Project No. 181308

***Prepared for*:**

MR 1.81 LLC
1155 Connecticut Avenue, NW
7th Floor
Washington, DC 20036
Attn: Mr. Bill Krokowski

February, 2007

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1.0 INTRODUCTION

This report presents the results of our geotechnical investigation for excavation shoring and foundations at Lot 161C-R, Mountain Village, Colorado. Planned development includes construction of a new multilevel residential resort complex with recreational, retail and parking facilities at the lower levels. The purpose of our investigation was to assess the geological and subsurface geotechnical conditions at the site in order to evaluate the feasibility of shoring at the site, and to provide shoring and foundation design recommendations for the proposed construction. The report was prepared from review of published geological documentation, field reconnaissance and investigation, engineering analysis of field and laboratory data, and from our experience with similar projects. Our report includes descriptions of the site geology, subsurface geotechnical conditions found in sixteen exploratory borings, and our opinions and recommendations for design and construction of the excavation shoring, foundation, and drainage and dewatering systems. The results of our investigation indicate that complex geological conditions exist at the site which will have considerable impacts on the planned construction- particularly with respect to excavation shoring.

2.0 SUMMARY OF CONCLUSIONS

A summary of our findings and conclusions is presented below. Detailed recommendations for shoring and foundation design and construction are presented in the report.

1. Complex geological conditions identified at the site include landslides, potentially unstable natural slopes, unstable cut slopes, soil creep, groundwater, and steeply dipping bedrock. These conditions will require special consideration during excavation, and also in the design of shoring, foundation, dewatering and drainage systems.
2. The primary risks to the development of Lot 161C-R are associated with the effects of groundwater and steeply dipping bedrock on excavation shoring construction. Portions of the landslide could be reactivated in response to seismic activity or development-induced rises in groundwater levels.
3. Subsurface conditions were investigated by drilling 16 exploratory borings across the site to depths ranging from 19 to 100 feet below the existing ground surface. Borings along the eastern excavation alignment were equipped with inclinometers to permit monitoring of slope movement. Standpipe piezometers were installed in 7 of the borings and vibrating wire piezometers were installed with one of the inclinometers to evaluate groundwater conditions across the site.
4. Subsurface conditions identified in our borings consisted of 2 to 41 feet of sandy clay, clayey sand, and silty sand overburden and landslide deposits overlying interbedded shale, sandstone, and siltstone bedrock. The bedrock dips into the east side of the excavation at angles ranging from 18 to 36 degrees. Laboratory tests indicate that the site soils and bedrock possess very low to non-expansive swelling characteristics. The bedrock exhibits high to very high bearing characteristics.
5. Groundwater levels were measured on November 1-3, 2006 at depths ranging from 9.4 feet to 36.2 feet below the ground surface. Artesian conditions were encountered within a coal-bearing zone which will be daylighted along the majority of the east excavation face. Water quality tests were conducted on samples of groundwater collected from the site. Groundwater conditions will impact both shoring construction and long-term drainage considerations. Preconstruction dewatering is recommended to ensure that groundwater does not adversely affect shoring construction and also to ensure that the design groundwater conditions are maintained.
6. Inclinometer readings indicate that no slope movement beyond the precision of the instrumentation occurred between August 30 and November 20, 2006.
7. The results of our analyses show that tieback shoring for the planned excavation at this site is feasible using reasonable anchor capacities and lengths. If the confined coal-bearing zone is not dewatered prior to shoring construction, artesian flow will complicate installation of anchors and facing. Preconstruction dewatering and permanent dewatering issues are discussed.
8. Tieback and/or soil nail shoring with reinforced shotcrete facing, drilled tangent soldier pile walls with tiebacks or internal bracing, and H-piles and lagging set in drilled concrete piles are considered appropriate for the site conditions. The viability of each option and its classification as temporary or permanent will be determined by the acquisition of permanent shoring easements. Geotechnical recommendations are provided for each system.
9. Protection of the gondola building will require temporary shoring and/or underpinning. Settlement conditions are discussed, and geotechnical recommendations for micropile underpinning are provided.
10. The unstable slope at the south side of the lot will require special consideration to protect stakeholder interests. The risks to the planned construction posed by the existing tieback slope-retaining structure remain to be evaluated. Special shoring and construction considerations will be required for this area.

11. Creeping soils on the north side of the complex and sliding soils on the south side will continue to move downhill after construction of the development is complete. Soil and rock anchors and exterior structural and/or architectural features such as outdoor patio dining, landscaping, utilities, subsurface drainage structures, and other improvements or appurtenances constructed in these areas will be affected, and should be considered during planning.

12. Blasting may be required to achieve planned elevations. Complex geological conditions will require special considerations for blasting. Preliminary costs can be estimated assuming that approximately 56,000 bank (in-place) cubic yards (cy) of rock will require blasting. For hauling of the blast rubble, we recommend an expansion of 25%, or 70,000 cy.

13. Development of the Lot 161C-R complex will not dewater the wetlands on the west side of the lot. Drainage into the excavation face is expected to be high initially, but will dissipate as subsurface storage is drained. Preliminary cost studies can be performed assuming long term design drainage flows of 1 gpm per lineal foot of excavation face. Shoring drainage recommendations are provided. The permanent shoring dewatering system should be designed by a registered professional engineer.

14. Drilled straight-shaft concrete pier foundations are recommended for the structure. We anticipate lengths will be on the order of 20 feet for the east half of the building envelope. Pier lengths of 30 to 50 feet are anticipated along the majority of the western property line as the bedrock surface dips below the excavated G2 level floor elevation, with several pier lengths exceeding 60 feet in the vicinity of boring MFG-7.

15. Concrete slab-on-grade floors are acceptable for portions of the excavation floor where bedrock is exposed. Placement of slabs-on-grade on the unconsolidated landslide deposit soils at the west side of the site would result in excessive differential settlement. Consequently, these soils are unacceptable for support of floor slabs. Structural floor slabs are recommended in these areas. Below-slab drainage recommendations are provided in the report.

16. If certainty that the groundwater levels and artesian pressures can be controlled and maintained for the service life of the structure cannot be achieved, the only alternative is to redesign the shoring to handle the full artesian pressures for the design life of the development. Thus, the propensity of the upper and lower groundwater systems to drain by gravity must be evaluated prior to construction of the first levels of shoring so that modifications, if necessary, can be made before shoring construction activities commence. Additional piezometers are recommended uphill of the lot to monitor groundwater levels in the upper system and artesian pressures in the confined zone as construction progresses.

17. We recommend a slope monitoring program be implemented to track ground movements at critical locations. A recommended slope monitoring schedule is provided.

18. Inclinometer MFG-S2I should be replaced by an additional inclinometer located at the extreme northeast corner of the property. The locations of the remaining inclinometers should be surveyed to confirm that they are located far enough from the shoring that they will not be damaged by construction; if any of the existing inclinometers lie on or within the shoring alignment, they too should be replaced with additional inclinometers.

19. The preliminary shoring design to date has been substantially completed from a draft version of this report which was issued in November, 2006. The shoring engineer should review the final geotechnical report to verify that the final geotechnical design recommendations have been complied with.

20. The range of anticipated deflections calculated by the shoring engineer should be conveyed to the Town building department so that they are aware that such deflections are expected and that deflections will be monitored.

3.0 SITE CONDITIONS

Lot 161C-R is located in the NW¼, NE¼, Township 42N, Range 9W (New Mexico Principal Meridian),San Miguel County, at One Gondola Place in the Town of Mountain Village, Colorado (Figure 3-1). Access to the lot is from the north via Mountain Village Boulevard. The building envelope extends to within several feet of the irregular lot boundaries on nearly every side. The lot is bordered on the west by the La Chamonix and Heritage Crossing hotel/condominium/retail developments, and on the south by the Mountain Village/Telluride Gondola base. Residential structures occupy Lot 97 above the southeast corner of the lot and Lot 101 above the northeast corner of the lot. Other adjacent lots on the east and north are undeveloped. Utilities run along the unpaved temporary service road which passes through the site from north to south.

The lot is situated along the western toe of Coonskin Mountain, a northwest-trending ridge which separates Mountain Village from the San Miguel River valley and the Town of Telluride. As shown on Figure 3-2, site topography is characterized by moderate to steep mountainside terrain to the east, and comparatively flatter grades to the west. The high point (9605 feet) and low point (9516 feet) of the lot are located at the northeast and northwest corners, respectively, for a total topographic relief on the order of 89 feet. Natural grades across the lot range from 24% to 43%. Grades at the south-central portion of the lot were artificially steepened during construction of the gondola base. Gentler grades between the entrance at the north center of the lot and the gondola base at the southwest corner were created by cutting from the north and filling in the west-central area, creating steeper artificial slopes along the majority of the west side.

Site drainage is generally to the west. An artificial wetland drainage along the west property line collects runoff from the property and also water collected from south of the gondola and carries it off site to the north. The undisturbed mountainside is vegetated with thick stands of mature aspen and native shrubs and grasses, with sporadic spruce and fir trees. No rock outcrops or incised surface drainage features were observed within the lot boundaries.

Slight to moderate “pistol-butting” of younger aspen tree trunks on the natural slopes across the site indicates slope creep has occurred and is likely active in these areas. Evidence of active slope failure was only observed at the southeast corner of the lot between the gondola base and the residence on Lot 97. A tieback stabilization structure approaching 100 feet long was constructed 8 to 10 feet west of and parallel to the southernmost portion of the east property line adjacent to Lot 97. The soils below the northern end of the stabilizing structure have slipped down the slope leaving a maximum 3-foot scarp between the structure and the downhill soil surface. These and other slope stability issues are discussed in detail in later sections of the report.

4.0 PROPOSED CONSTRUCTION

The complex will consist of hotel and condominium units combined with retail and indoor recreational space. Project plans by James Watt, Architect (Project No. 0623, July 14, 2006) indicate four base levels (G2, G1, Plaza, and Lobby), with a total of 324 parking spaces. Three residential towers- two on the north side and one on the southeast corner- will each have seven stories above the lobby level, plus a mezzanine, for a total of eleven levels. Columns, shear walls, and foundation walls will be constructed of cast-in-place reinforced concrete, and floor slabs will be post-tensioned. Typical column service loads within the tower footprints will be on the order of 1,800± kips, with substantially lower loads outside the tower footprints.

The lowest level (G2) will have a finished floor elevation of 9518.5 feet, which will nearly meet existing grades along the west side. Due to the steep mountainside to the east, grades along the east side will not be reached until the 4th floor or 5th floor (7th or 8th level). This configuration will require maximum cuts up to 82 feet plus foundation excavations along the east side of the complex, tapering down to less than 10 feet along the west side. Slightly deeper cuts will be required on the west side for small mechanical and pump rooms.

Shoring will not be required along the west side as finished floor grades are very nearly at finished exterior grades. Excavation around the north, east, and south sides of the building envelope will require temporary or permanent shoring of over 900 lineal feet of excavation face. Permanent excavation shoring is planned for the east excavation face provided that permanent long term easements can be acquired from the owners of bordering properties to the east. In areas of permanent shoring, only the lowermost portions (bottom two levels or less) of the shoring will be in contact with the shoring; the shoring will otherwise slope upward and away from the structure at anticipated maximum slopes of 0.1:1 (horizontal:vertical). In several locations, such as along the west side of the north wall and the south and south-central portions of the east wall, the structure will be in contact with the wall via struts or braces.

Due to the anticipated difficulties with acquisition of permanent shoring easements along portions of the north and south sides of the complex, temporary shoring is planned along these areas, which will be replaced with internal structural bracing. Planned foundation grades immediately adjacent to the gondola base station complex at the southwest corner will be only several feet below and away from the existing foundations. Natural stone facing is planned for some of the exposed shoring faces.

5.0 FIELD INVESTIGATION

Subsurface conditions at the site were investigated by drilling a total of 16 borings within or immediately outside the building envelope at the approximate locations shown on Figure 5-1. The borings were located to provide characterization of subsurface conditions across the building envelope; particular emphasis was placed on the west side to evaluate the influence of wetlands along that side, and on the east side to characterize subsurface conditions along the alignment of the tallest shoring. Ground surface elevations at the boring locations were estimated from topographic mapping provided by Foley Associates, Inc. (Final DRB Existing Conditions. October 3, 2006).

The field investigation was performed in multiple phases. Nine borings (MFG-1 through MFG-9) were drilled in the western and central portions of the building envelope, and five (MFG-S1 through MFG-S5IB) were drilled along the eastern property line to assess shoring requirements. MFG-WQ was drilled near the entrance to collect groundwater samples for analytical testing. Inclinometers were installed in borings MFG-S2I through MFG-S5I ('S' indicating shoring and 'I' indicating an inclinometer was constructed in the boring). These borings were located a few feet outside the building envelope so that inclinometer readings could continue during and after construction. Expansion of the building envelope to the south after MFG-S5I was installed caused it to be within the building envelope, so another inclinometer was installed at location MFG-S5I(B). Slope Indicator vibrating wire piezometers were installed in MFG-S3I at depths of 21.8 feet and 62.8 feet; the calibration reports for both piezometers are included as Appendix C. Drilling dates, methods, and instrumentation are summarized on Table 5-1. Baseline inclinometer readings were taken on August 30, 2006, and the first readings were taken on November 20, 2006. The inclinometer readings are discussed in a later section of the report, and indicate that no movement beyond the precision of the instrumentation occurred between August 30 and November 20, 2006.

Borings MFG-S1 and MFG-S5I were drilled by Spectrum Exploration, Inc. of Colorado Springs, Colorado using a track-mounted Boart Longyear DB-540 drilling rig, and the remainder of the borings were drilled by D.A. Smith Drilling Company of Grand Junction, Colorado using a track-mounted Deidrich D50 drill rig. Samples were obtained in the overburden soils and weathered shales with a 2.5-inch outside diameter California-type sampler driven into the soils with blows of a 140-pound hammer falling 30 inches. Standard Penetration Tests (SPTs) were performed in the harder rock just beneath the weathered shales using a 2.0-inch outside diameter split barrel. MFG-S2I was continuously cored in the upper 18.5' to allow for observation of the contact between overburden soil and the bedrock surface. In borings MFG-S2I through MFG S5I(B), the rock below the overburden soils was wet-cored using either HX or HQ coring equipment.

A field engineer from MFG was present during drilling to oversee the logging of soils encountered in the borings and to collect soil and rock samples. Graphic logs of the subsurface conditions identified in the borings, including soil and rock types, samples collected, field tests performed, sample recovery, rock quality designation (RQD), instrumentation, and groundwater conditions are presented in Appendix A.

Table 5-1 Summary of Boring Information

Boring ID	Date Drilled	Drilling Method	Instrumentation
MFG-1	3/7/06	6” Solid Stem Auger	¾” PVC Piezometer (31 ft)
MFG-2	11/2/06	6” Solid Stem Auger	-
MFG-3	3/7/06	6” Solid Stem Auger	¾” PVC Piezometer (28’)
MFG-4	11/1/06	6” Solid Stem Auger	-
MFG-5	3/7/06	6” Solid Stem Auger	¾” PVC Piezometer (46’)
MFG-6	11/2/06	6” Solid Stem Auger	¾” PVC Piezometer (30’)
MFG-7	11/1/06	6” Solid Stem Auger	2” PVC Well/Piezometer (30.5’)
MFG-8	11/1/06	6” Solid Stem Auger	-
MFG-9	3/7/06	6” Solid Stem Auger	¾” PVC Piezometer (19’)
MFG-S1	6/13-14/06	6.5” Hollow Stem Auger (soil) 4.25” HQ Core (rock)	2” PVC Well/Piezometer (60’)
MFG-S2I	7/19/06	8.25” Hollow Stem Auger (soil) 3.78” HX Core (rock)	2.75” PVC Inclinometer (57’)
MFG-S3I	7/14/06	8.25” Hollow Stem Auger (soil) 3.78” HX Core (rock)	2.75” PVC Inclinometer (91’) Vibrating Wire Piezometers (21.8’, 62.8’)
MFG-S4I	7/17-18/06	8.25” Hollow Stem Auger (soil) 3.78” HX Core (rock)	2.75” PVC Inclinometer (67’)
MFG-S5I	6/15-19/06	6.5” Hollow Stem Auger (soil) 4.25” HQ Core (rock)	2.75” PVC Inclinometer (23’)
MFG-S5I(B)	11/2/06	6” Solid Stem Auger	2.75” PVC Inclinometer (30’)

6.0 LABORATORY INVESTIGATION

Geotechnical laboratory testing for the shoring investigation was performed by Smith Geotechnical Engineering Consultants, Inc. of Fort Collins, Colorado and Advanced Terra Testing, Inc. of Golden, Colorado. Laboratory testing was performed on selected California samples and rock core samples obtained during the drilling. Tests included water content, dry density, Atterberg limits, gradations, unconfined compressive strength, triaxial compressive strength, direct shear (consolidated-drained and loaded rock), triaxial shear (consolidated-undrained with pore pressure measurements), sulfates, and consolidation-swell tests. The results of the laboratory tests are summarized on Table 6-1, and the complete laboratory report is attached as Appendix B.

(Note: Boring MFG-9 was originally designated as MFG-10, but was subsequently changed. Consequently, the laboratory test results reported in Appendix B.1 for MFG-9 are designated as MFG-10.)

Table 6-1 Summary of Laboratory Test Results

										Shear Testing				
Borehole No.	**Sample Depth (ft)**	**Sample Type [1]**	**Soil Type**	**Water Content (%)**	**Wet Density (pcf)**	**Dry Density (pcf)**	**Passing #200 Sieve (%)**	**Atterberg Limits LL/PL/PI (%)**	**Soil Sulfates (ppm)**	**Direct Shear c/φ (Residual) (psf/deg)**	**Triaxial Shear,[9] c/φ (psf/deg)**	**Compressive Strength (psi)**	**Inundation Pressure (psf)**	**Percent Swell (%)**
MFG-1	5	CA	Clayey Sand	7.4	138.2	128.7	39	20/14/6					50	0.0
	10	CA	Sandy Clay	6.7	128.6	120.5	68						1,000	-0.4
	10	CA	Sandy Clay	8.6	122.0	112.4		33/17/16				24.07[8]		
	15	CA	Sandy Clay				56	25/18/7					2,000	0.3
MFG-2	25	CA	Shale										500	0.0
	30	CA	Shale	9.3	143.6	131.4		22/18/4				23.35[8]		
MFG-4	30	CA	Shale	15.8	135.9	117.4		26/19/7	279			28.94[8]		
MFG-5	10	CA	Fill: Sandy Clay	16.0	114.1	102.6	58	28/13/15				8.02[8]		
	25	CA	W. Shale	10.5	134.8	122.0	25	20/NP					3,000	-1.4
	35	CA	Shale	10.8	138.0	124.6	33	22/NP				8.63[8]		
MFG-6	10	CA	Shale	14.1	121.5	106.5							500	-0.1
	30	CA	Shale	5.7	139.6	132.1		20/16/4					2,500	0.0
MFG-7	5	CA	Sandy Clay	15.2	126.3	109.6	52	27/13/4					500	0.2
	15	CA	Sandy Clay						28					
	40	CA	Sandy Silt	5.3	146.7	139.3		16/13/3				8.36[8]		
MFG-9	10	CA	Silty Sand	12.6	140.1	124.4	37	18/NP				8.15[8]		

Table 6-1 Summary of Laboratory Test Results (Cont'd)

Borehole No.	Sample Depth (ft)	Sample Type [1]	Soil Type	Water Content (%)	Wet Density (pcf)	Dry Density (pcf)	Passing #200 Sieve (%)	Atterberg Limits LL/PL/PI (%)	Soil Sulfates (ppm)	Shear Testing: Direct Shear c/φ (Residual) (psf/deg)	Shear Testing: Triaxial Shear,[9] c/φ (psf/deg)	Compressive Strength (psi)	Inundation Pressure (psf)	Percent Swell (%)
MFG-S1	4	CA	Fill: Sandy Clay	12.0	118.9	106.2			92.0					
	14	CA	Sandy Clay	13.4	125.6	110.8	54[3]	25/15/10					1,000	-0.44
	19	CA	Sandy Clay	13.7[13]	133.3[13]	117.2[13]					469/27.5			
	24	CA	Broken Shale	12.7	132.1	117.2								
	29	CA	W. Shale	9.7[11]	130.5[11]	119.0[11]				296/33.1[4]				
	34	CA	W. Shale	8.4[12]	132.2[12]	121.9[12]						16.35[8]		
	45	Bag	Shale					22/14/8						
	36.5-37	Core	Shale		155.1[14]					490/42.0[7]				
	50-51	Core	Siltstone	1.2	160.9	159.0						10,080[6]		
	55.5-57	Core	Turbated SS	1.1	160.1	158.4						100[5] 12,950 200[5] 16,030 500[5] 14,630		
MFG-S2I	5	CA	Sandy Silt	13.8[12]	113.8[12]	100.0[12]						4.87[8]		
	18	Bag	W. Shale in Clay				55[3]	32/17/15	360.0					
	19-19.5	Core	Turbated SS	0.6	160.0	159.0						13,620[6]		
	19.5-21	Core	Turbated SS	0.7	161.9	160.8						100[5] 18,020 200[5] 21,760 500[5] 26,110		
	42.5-43.5	Core	Shale		157.7[14]					2,534/63.4[7]				

Table 6-1 Summary of Laboratory Test Results (Cont'd)

Borehole No.	Sample Depth (ft)	Sample Type [1]	Soil Type	Water Content (%)	Wet Density (pcf)	Dry Density (pcf)	Passing #200 Sieve (%)	Atterberg Limits LL/PL/PI (%)	Soil Sulfates (ppm)	Shear Testing: Direct Shear c/φ (Residual) (psf/deg)	Shear Testing: Triaxial Shear,[9] c/φ (psf/deg)	Compressive Strength (psi)		Inundation Pressure (psf)	Percent Swell (%)
MFG-S3I	10	CA	Silty Clay	13.1	128.4	113.6								1,000	-0.7
	15	CA	Clayey Sand	14.4[13]	136.7[13]	119.5[13]	40[3]	28/16/12			1,502/22.7				
	20	CA	W. Shale	10.7	146.2	132.1									
	24-25	Core	Turbated SS		163.2[14]					1,886/66.1[7]					
	43-43.5	Core	Shale	1.0	157.5	155.9						6,460[6]			
	56.5-57.5 & 59-59.5	Core	Shale	1.6	163.5	160.9						100[5] 200[5] 500[5]	11,350 7,840 19,380		
	57-58	Core	Coal		93.3[14]					7,416/33.7					
	74-74.5	Core	Bedded SS	0.7	157.9	156.8						17,080[5]			
MFG-S4I	10-12	Bag	Silty Clay				60[2]	22/13/9							
	25	CA	W. Shale	15.4[12]	135.4[12]	117.4[12]						15.24[6]			
	56.5-57	Core	Shale	1.7	156.7	154.1						6,080[6]			
	73-75	Core	Int. SS/Shale	0.5	160.7	159.9						100[5] 200[5]	13,370 14,630		

Table 6-1 Summary of Laboratory Test Results (Cont'd)

Borehole No.	Sample Depth (ft)	Sample Type [1]	Soil Type	Water Content (%)	Wet Density (pcf)	Dry Density (pcf)	Passing #200 Sieve (%)	Atterberg Limits LL/PL/PI (%)	Soil Sulfates (ppm)	Shear Testing: Direct Shear c/φ (Residual) (psf/deg)	Shear Testing: Triaxial Shear,[9] c/φ (psf/deg)	Compressive Strength (psi)	Inundation Pressure (psf)	Percent Swell (%)
MFG-S5I	4	CA	Sandy Clay	17.3	133.7	114.0								
	7	CA	W. Shale	18.5	131.0	110.5	62[3]	41/18/23		0/56.8[4]		5.86[8]	700	0.6
	9	CA	Shale	9.5	111.7	102.0								
	16.5-17 & 21.5-22.5	Core	Shale	1.4	162.2	159.9						100[5] 6,380 200[5] 6,800 500[5] 11,490		
	34.5-35	Core	Turbated SS	0.7	136.4[10]				575.0					
	38-40	Core	Siltstone	0.9	159.4	158.0						16,320[5]		
	37.0-40.0	Core	Siltstone	1.0	158.9	157.4						100[5] 17,240 200[5] 5,200 500[5] 14,660		
	45.5-46	Core	Massive SS	0.3	159.5	159.0						27,980[6]		
	74-75	Core	Shale		154.8[14]					3,816/39.0[7]				
	79-79.5	Core	Turbated SS	0.4	165.8	165.1						17,930[6]		
MFG-S5I(B)	5.5	CA	Shale	7.2	122.1	113.9							500	-0.3

NOTES:
1. CA = Undisturbed California Sample, Bag = grab sample, Core = sample taken from the core
2. Results given as the percentage by weight passing the 200 sieve,
3. Results given as the percentage by weight passing the 200 sieve, full grain-size distribution included in Appendix B.
4. Direct Shear (ASTM D3080)
5. Triaxial Compressive Strength (ASTM D7012 Method A)
6. Unconfined Compressive Strength (ASTM D7012 Method C)
7. Direct Shear (ASTM D5607)
8. Unconfined Compressive Strength (ASTM D2166)
9. Triaxial Shear – Consolidated, Undrained with pore pressure measurements (ASTM D4767)
10. Paraffin coated Density (ASTM D4531-B)
11. Data from Direct Shear Test
12. Data from Unconfined Compressive Strength Test
13. Data from Triaxial Compression Test
14. Data from Direct Shear Test

7.0 SUBSURFACE CONDITIONS

Subsurface conditions identified in our borings consisted of 2 to 41 feet of sandy clay, clayey sand, and silty sand overburden deposits overlying interbedded shale, sandstone, and siltstone bedrock. Groundwater levels were measured across the site at depths ranging from 9.4 feet to 36.2 feet below the ground surface. Swell/consolidation tests performed on 12 samples of the soils and bedrock exhibited volume changes ranging from -1.4 percent (hydroconsolidation) to 0.6 percent swell when wetted under loads ranging from 50 psf to 3,000 psf, indicating the site soils and bedrock have very low to non-swelling expansive characteristics. The bedrock exhibits high to very high bearing characteristics.

7.1 Man-Placed Fill

Man placed fill materials were identified over the overburden soils in borings MFG-5 and MFG-S1. The fill materials in MFG-5 were placed along the west-central portion of the site in an end-dump fashion to create access and parking for the gondola station. Topography indicates the fill is as deep as approximately 15± feet. The fill in MFG-S1 is approximately 4 feet deep, and appears to have been constructed by berming soils at the entryway at Mountain Village Boulevard. The fills appear to have been constructed of native on-site materials, and consequently have geotechnical properties similar to those of the overburden materials. The fill materials are considered unsuitable for support of foundations or floor slabs. Planned excavation elevations indicate that all of the fill materials will be removed during mass excavation for the development.

7.2 Overburden Soils

The overburden soils consist predominantly of very sandy, silty clay and clayey, silty sand with variable quantities of fine shale and sandstone gravels. Thin, discontinuous pockets of sandy silt were identified in several borings. These soils generally varied in depth across the site from 7 feet to 25 feet below ground surface, with deeper deposits (41 feet) found at boring MFG-7. The overburden deposits are shallower at the south end of the lot where the soils were thinned for construction of the gondola base station. The overburden soils are primarily colluvial (slope wash) and landslide deposits of the Silver Mountain Landslide Complex (discussed in detail in later sections), and are considered unsuitable for support of foundations or floor slabs.

The consistency of the unconsolidated overburden soils is highly variable, ranging from soft to very stiff. In-place water contents of the samples ranged from 5.3 to 17.3 percent, and dry densities ranged from 100.0 to 139.3 pcf. Liquid limits ranged from 16 to 33, and plasticity indices ranged from 0 (non plastic) to 16. The overburden soils had between 37 and 68 percent passing the No. 200 sieve. Tests on 3 samples of the overburden soils and fill indicate water-soluble sulfate concentrations ranging from 28 to 360 parts per million (ppm).

Six samples of the overburden soils were selected for swell-consolidation testing. The samples exhibited volume change ranging from -0.7 percent (hydroconsolidation) to 0.3 percent swell when inundated under normal loads ranging from 500 psf to 2,000 psf. These results indicate that the overburden soils have very low to non-expansive swelling characteristics. One sample of sandy clay and one sample of clayey sand were tested in triaxial shear tests. The cohesion was determined to be 469 psf with a friction angle of 27.5 degrees (peak) for the sandy clay, and 1502 psf and 22.7 degrees (peak) for the clayey sand. Results of the unconfined compression tests indicated unconfined compressive strengths ranging from 701.2 psf for a sample of sandy silt to 3,466.1 psf for a sample of sandy clay.

7.3 Bedrock

7.3.1 Weathered Shale

Weathered shale was encountered beneath the overburden soils in approximately half of the borings. The thickness of the weathered shale varies from 5 to 20 feet, with the thickness decreasing with rising elevation to the east. In-place water contents of the samples ranged from 8.4 to 18.5 percent, and dry densities ranged from 110.5 to 132.1 pcf. The grain size distribution for three samples of weathered shale indicated 25 to 62 percent of the material passing the #200 sieve; it is likely that some of the shale particles did not break down all the way to their full constituent particle sizes, thus biasing the gradations toward coarser grain size distributions. Full grain size distributions are included in Appendix B. Two samples of the weathered shale had liquid limits of 20 and 41, with corresponding plasticity indices of 0 (non-plastic) and 23, respectively. Of two samples of the weathered shale selected for swell/consolidation testing, one swelled 0.6 percent under an inundation pressure of 700 psf, and the other exhibited 1.4 percent hydroconsolidation under an inundation pressure of 3,000 psf. The weathered shale is expected to have very low to non-expansive swelling characteristics. The unconfined compressive strength of the weathered shale ranged from 5.86 psi for a highly weathered sample from MFG-S5I to 15.24 and 16.35 psi for samples taken from MFG-4SI and MFG-S1, respectively.

The Rock Quality Designation (RQD) was calculated as the sum of core specimens with a length greater than or equal to 5 inches (twice the core diameter) in a given run divided by the run length. The RQD for one core interval of the weathered shale from boring MFG-S4I was calculated to be 90 percent.

7.3.2 Shale and Coal

Shale bedrock was encountered in all borings drilled at the site. The fine-grained sedimentary rock varied laterally and stratigraphically from laminated, fissile shale to massive mudstone. In places the shale and mudstone was interbedded with sandstone, siltstone, and coal. Some of the shale contained rip-up clasts of siltstone, shale, mudstone and sandstone. Several of the borings encountered thin (less than 5mm) laminae of very low grade coal and/or carbonaceous material within the shale. A coal seam on the order of 12 to 18 inches thick was identified at depths ranging from 52 feet in MFG-S2I to 72 feet in MFG-S5I. The coal seam is storing and transporting groundwater under confined (pressurized) conditions. Groundwater rose to the surface when the coal seam was penetrated during drilling operations causing low artesian flows (less than 1-5 gallons per minute). The coal seam will impact construction and long term dewatering for the development, as discussed in later sections.

An Atterberg limits test on one shale sample indicated low plasticity with a liquid limit of 22 and a plasticity index of 8. Direct shear testing for five shale samples taken from 7 feet, 29.5 feet, 36.5 feet, 42.5 feet, and 74 feet was conducted; cohesion for these samples ranged from 0 psf to 3,816 psf, and friction angles ranged from 33.1 to 63.4 degrees (residual). In a direct shear test, a sample of coal exhibited a cohesion of 7,416 psf and a friction angle of 33.7 degrees (residual). Peak friction angles for these 6 tests ranged from 43.3 degrees to 84.4 degrees.

Swell testing of shale samples from nearby sites has shown moderate to high swelling characteristics in thin, widely spaced and discontinuous strata. Although higher swelling beds may exist in the subsurface at Lot 161C-R, MFG considers the shales at the Lot 161C-R site to have predominantly very low to non-expansive swelling characteristics. RQDs of the shale obtained during drilling of MFG-S1 through MFG-S5I are as follows: MFG-S1: 0 – 36.6 percent; MFG-S2I: 43 – 81.2 percent; MFG-S3I: 70 – 94.4 percent; MFG-S4I: 46 – 87.5 percent; MFG-S5I: 40.8 – 85.4 percent. RQDs of the interbedded sandstone and shale were determined from core samples from MFG-S3I and MFG-S5I. RQDs ranged from 38.8 to 82 percent at MFG-S3I and from 37.5 to 84.2 percent at MFG-S5I.

In some cases the carbonaceous beds were indistinguishable from the shale beds unless split with a hammer. In other cases, the beds were fractured during the coring operation. Where the thin carbonaceous layers split, in some cases we observed degrees of coalification including dull luster and poor cleavage. In several isolated instances, these surfaces contained features which resemble slickensides. Small, localized forms of slickensides in coal deposits are common, and are thought to be related to minor movement of the coal during compression of the coal layers from overlying sediments. The age of these features is unknown, but the origin is likely related to movement during Tertiary mountain building events or compression of the coal layers during lithification. The discontinuous and irregular nature of these features suggests they do not represent continuous, large scale, or significant movement planes.

7.3.3 Sandstone

Sandstone was encountered in borings MFG-S1 through MFG-S5I. The sandstone encountered varied from laminated to massive, and was fine- to medium-grained.. Some of the sandstone contained rip-up clasts of shale, siltstone, mudstone and sandstone. In general, sandstone approximately 10 to 25 feet thick underlies the overburden soils and/or weathered shales.

A test on a core sample from this sandstone indicated a compressive strength of approximately 13,000 psi. Beneath the sandstone are thinner beds of interbedded sandstone, shale, and siltstone with compressive strengths ranging from 10,000 to 17,000 psi, as determined during laboratory testing. Tests on samples from a sandstone bed underlying the coal in MFG-S3I at 74 feet and MFG-S5I at 79 feet indicated a compressive strength of approximately 17,000 psi. A direct shear test on one sample of the sandstone core indicated a cohesion of 1186 psf and a friction angle of 66.1 degrees (residual), and a peak friction angle of 89.2 degrees.

RQDs of the sandstone obtained during drilling of MFG-S1 through MFG-S5I are as follows: MFG-S1: 88.3 percent; MFG-S2I: 75.6 – 86.7 percent; MFG-S3I: 21 – 89.6 percent; MFG-S4I: 49 – 97 percent; MFG-S5I: 23.3 – 94.2 percent.

7.3.4 Siltstone

Siltstone was encountered in borings MFG-S1, MFG-S2I, and MFG-S3I. In general, the siltstone was highly turbated and contained rip-up clasts of sandstone and shale. RQDs of the siltstone obtained during drilling are as follows: MFG-S1: 77.5 percent; and MFG-S3I: 61 percent. Siltstone obtained from boring MFG-S5I at a depth of 38 feet had a compressive strength of 16,320psi.

7.4 Groundwater

Groundwater was encountered in all of the borings except MFG-9 and MFG-S5I-B, and will have a significant impact on the planned development. Groundwater levels and elevations in all borings are summarized in Table 7-1. Results of analytical water quality tests are presented in future sections of the report.

The shallower borings across the central and western portions of the site (MFG-1 through MFG-8 and MFG-S1) encountered groundwater during drilling either in the shale or just above the overburden/shale contact at depths of 20 to 41 feet below the ground surface. Groundwater levels in most of these borings rose on the order of 5 to 20 feet by the time drilling was complete, indicating slightly confined conditions within the upper shale strata. This is a common condition in areas with steeply dipping sedimentary bedrock when water becomes trapped in the tilted beds of more permeable strata. Groundwater levels were measured again on November 1-3, 2006, and ranged in depth from 9.4 feet (MFG-3) to 36.2 feet (MFG-S1). Intermediate groundwater measurements showed little fluctuation after the initial rise and subsequent stabilization.

In addition to the shallow surficial groundwater measured in MFG-1 through MFG-8 and MFG-S1, we encountered a deeper groundwater system in borings MFG-S2I through MFG-S5I. This system is confined within coal-bearing strata as previously described. Confining pressures were sufficient to bring artesian flow to the ground surface when the system was penetrated with boring equipment. Artesian flows were measured or estimated to range from 0.4 to 5 gallons per minute (gpm). Observations during drilling and inspection of continuous core samples indicate that the primary coal seam has a local thickness up to 18 inches, but that it is not continuous across the site. Flow rates in MFG-S3I during drilling initiated approximately 5 feet before the coal seam was penetrated, suggesting that although the coal seam is the primary conduit, water is also conducted under artesian pressures through fractures immediately above and below the coal seam. Groundwater is also anticipated in thin fissures along bedding planes within the carbonaceous shale at other elevations in the profile.

We believe the surficial and confined groundwater systems are not strongly hydrologically connected. In boring MFG-S3I, one vibrating wire piezometer was installed at a depth of 21.8 feet to measure the upper groundwater system, and another was installed at 62.8 feet to measure the confined system. Piezometer readings showed the piezometric surface in the upper system to be at a depth of 14.1 feet, which is consistent with our concept of groundwater flowing in the overburden and upper shale strata. In contrast, the lower piezometer measured the piezometric surface to be 19.1 feet above the ground surface, confirming the confined condition. The coal seam is estimated to be approximately 120 feet or more below the ground surface at the western property line. These observations indicate that the two groundwater systems are hydrologically independent.

The groundwater levels we measured were observed during a comparatively dry time of the year and in a period where annual precipitation has been below the 100-year average for the past 6 years. Groundwater levels should be expected to fluctuate seasonally, and will rise during wetter seasons and in wetter years. The implications of the local groundwater conditions on the planned development, and recommendations for the control of surface and subsurface drainage, are provided in later sections of this report.

Table 7-1 Summary of Groundwater Levels

Boring ID	Boring Depth (ft bgs)	TOB Elev. (ft)	During Drilling Date	During Drilling GWL_{DD} (ft bgs)	During Drilling GWE_{DD} (ft)	After Drilling GWL_0 (ft bgs)	After Drilling GWE_0 (ft)	Subsequent Date	Subsequent GWL (ft bgs)	Subsequent GWE (ft)
MFG-1	31	9520	3/7/06	19.0	9501.0	17.3	9502.7		16.8	9506.7
MFG-2	39	9543	11/2/06	30.0	9513.0	24.9	9518.1	11/3/06	24.3	9518.7
MFG-3	28	9560	3/7/06	20.0	9540.0	8.7	9551.3	11/2/06	9.4	9552.8
MFG-4	35	9567	11/1/06	33.5	9533.5	21.5	9545.5	11/3/06	21.1	9545.9
MFG-5	46	9543	3/7/06	39.0	9504.0	33.7	9509.3	11/1/06	33.0	9514.3
MFG-6	30	9524	11/2/06	21.5	9502.5	16.4	9507.6	11/3/06	15.9	9508.1
MFG-7	50	9523	11/1/06	41.0	9482.0	20.3	9502.7	11/3/06	25.4	9497.6
MFG-8	37	9562	11/1/06	27.0	9535.0	15.4	9546.6	11/3/06	14.7	9547.3
MFG-9	19	9538	3/7/06	dry	--	dry	--	3/30/06	> 19	> 9519
MFG-S1	60	9557	6/13/06	35.0	9522.0	38.0	9519.0		36.2	9520.8
MFG-S2I	65	9578	7/19/06	51.0	9527.0	artesian				
MFG-S3I[1]	100	9593	7/14/06	56.0	9537.0	14.1	9578.9	11/20/06	13.4	9579.6
MFG-S3I[2]	100	9593	7/14/06	56.0	9537.0	-19.1	9612.1	11/20/06	-19.6	9612.6
MFG-S4I	75	9586	7/17/06	63.0	9523.0	artesian				
MFG-S5I	95	9578	6/15/06	72.0	9506.0	artesian				
MFG-S5I(B)	30	9581	11/2/06	DRY	--	dry	--			

GWL_{DD} = Groundwater Level During Drilling

GWE_{DD} = Groundwater Elevation During Drilling

GWL_0 = Groundwater Level Immediately After Drilling

GWE_0 = Groundwater Elevation Immediately After Drilling

[1] Vibrating wire piezometer installed at 21.8 feet below ground surface, measured 8/30/06

[2] Vibrating wire piezometer installed at 62.8 feet below ground surface, measured 8/30/06

8.0 GEOLOGICAL CONDITIONS

MFG's geologic evaluation of the site included review of available published geological documentation in addition to field and laboratory investigations. A geological reconnaissance of the site was performed on June 15 and 16, 2006 by Tom Chapel, CPG, PE, MFG's project geological engineer. Rock core samples were inspected by Mr. Chapel, Eileen Dornfest, PG, CEG, MFG's project engineering geologist, and Mark Abshire, MS, PE, MFG's project manager. The general geology of the region is presented, followed by discussion of the site-specific geological conditions. The section concludes with discussion on geologic hazards associated with the site geology and planned development.

8.1 General Geology

In general, the site is located in the San Juan Mountains of the Southern Rocky Mountain physiographic province. The geology of the Telluride area is complex, and is generally composed of Permian to Tertiary sedimentary rocks overlain by Tertiary volcanics. Intense structural activity and mineralization occurred in areas immediately east of the subject site (Burbank and Luedke, 1966). During the Pleistocene, glaciers carved steep-walled valleys through the mountains, resulting in localized glacial deposits and oversteepened valley walls subject to slope instability. The site is located near the northern edge of a 14-square mile landslide deposit known as the Silver Mountain Landslide (Lincoln DeVore, 1979), a deposit reportedly up to 300-m thick in some areas (Luedke and Burbank, 1977).

8.2 Local Site Geology

Our investigation indicates that the irregular eastern boundary of the landslide deposit is located less than 350 feet northeast of the east boundary of Lot 161C-R. Quartzitic sandstones and carbonaceous shales of the Cretaceous Dakota Formation form the erosion-resistant ridge north of the site that separates Mountain Village from the San Miguel River valley and the Town of Telluride. These sedimentary units will be the primary stratigraphic units exposed in the planned excavation.

Burbank and Luedke (1966) indicate a strike of N21W and a dip of 30 degrees southwest several hundred feet northeast of the site. Additional geologic reconnaissance by MFG indicates a general strike of N24W and a dip of 34 degrees southwest. Field strike and dip measurements on a sandstone outcrop adjacent to the gondola averaged N20W with a dip of 18 degrees southwest. Bedding planes observed in a roadcut approximately 350 feet east of the subject site were observed to be variable, but had a general strike of N15W to N24W and dipped 30 degrees to the southwest. In one location the dip was measured at 40 degrees. Strike measurements are reported with reference to true north. Bedding planes in core samples were also variable, but had maximum inclinations from the horizontal of 5 to about 30 degrees. The inclination of bedding observed in the core samples was generally less near the northern portion of the subject site. Correlations between borehole logs indicated an average dip of 19 to 24 degrees towards the southwest.

Using graphical procedures described in Compton (1962), apparent dips were corrected to approximate true dips assuming a strike of N24W. Using the orientation of anticipated excavation sidewalls relative to the sediment strike, an anticipated maximum bedding inclination at the excavation was calculated to be 36 degrees. These measurements were assumed to be sufficiently accurate and conservative considering the variability of the site geology.

The carbonaceous shales encountered in our borings and observed in outcrops to the east are interbedded with well-cemented lenticular to massive sandstone and thin, discontinuous lenses of poor-quality, low grade coal. The coal and carbonaceous layers provide conduits for water transmitted from higher topographic areas to the east, and result in small, localized and comparatively weaker zones in the subsurface.

8.3 Geologic Hazards

Potential geologic hazards mapped within the Telluride Quadrangle include flood plains, alluvial fans, rockfall areas, landslide deposits, potentially unstable natural slopes, unstable cut slopes, soil creep, and groundwater (Luedke and Burbank, 1977). Of these, our geological investigation indicated that landslides, potentially unstable natural slopes, unstable cut slopes, soil creep, and groundwater apply to the Lot 161C-R site; we did not identify rockfall areas, alluvial fans, or floodplains in the immediate vicinity of Lot 161C-R. Potential geologic hazards associated with seismic events are also discussed. Each of the identified hazards is addressed individually below.

8.3.1 Landslides

As discussed previously, the Silver Mountain Landslide complex is a soil deposit that resulted from large scale landsliding that occurred during the geologic past. Mapping by Luedke and Burbank (1966, 1976, and 1977) indicates the subject site is approximately 200 feet southwest of the northeastern lateral extent of the Silver Mountain Landslide. Our geologic mapping and field observations confirmed outcrops of intact Dakota Sandstone in roadcuts about 350 feet northeast of the east property line of Lot 161C-R. Our estimate of the surface geology and approximate landslide boundary are shown on Figures 6-1 through 6-3.

Observations and geologic mapping up-slope of Lot 161C-R did not reveal evidence of landsliding such as the irregular hummocky terrain which was apparent down-slope of the mapped slide boundary and observed outcrops. Additional observations near and east of the ridge did not disclose evidence of landsliding in that area, and no major scarps were observed near the mapped landslide boundary. These observations indicate that the slide mass is thin along its northeastern lateral extents and thickens toward the west, which is toward the center of the mapped slide deposit.

These observations suggest that the slide mass is thin within the Lot 161C-R property boundaries, rather than very thick, or deep-seated, as is likely the case in downslope areas to the west and south. This condition was confirmed by our exploratory borings, which indicate a landslide deposit thickness generally less than 20 feet thick along the eastern edge of the proposed excavation and increasing to 30 feet or more at the western edge. MFG's geotechnical investigations on nearby lots to the southwest indicate that the thickness of the deposit increases dramatically a short distance to the southwest of the building envelope. Further discussion on the future stability of the landslide and risk evaluation are provided at the end of this section.

8.3.2 Potentially Unstable Natural Slopes

The site is not located within areas delineated by Luedke and Burbank (1977) to have potentially unstable natural slopes. However, indications of soil creep and development-induced slope instability were observed at the site, as discussed in following paragraphs.

8.3.3 Unstable Cut Slopes

Lincoln-DeVore (1979) identified the instability of artificial slopes as one of the most critical geologic hazards associated with the landslide complex. Excavation slopes for development of Lot 161C-R fall into this category. This hazard will be created when excavations penetrate the slide deposit and underlying weaker sedimentary layers. The soil and bedrock layers on the east and north sides of the excavation will dip into the excavation. Anchoring and support for blocks and wedges of sedimentary bedrock must be provided artificially to counterbalance the removal of supporting sediments on the downslope sides of the cut. This condition will be exacerbated by the comparatively lower strength coal and carbonaceous shale beds, and by water flowing both in the shallow subsurface along the bedding planes and by the confined groundwater in the coal seam and associated strata identified in our borings.

Previous excavation for construction of the gondola base at the south side of the lot resulted in the removal of all but 7 to 10 feet of the landslide deposit in the area between the gondola and Lot 97, and an oversteepened cut on the order of 12 feet high immediately south of Lot 161C-R. A tieback slope-stabilizing structure approximately 100 feet long was constructed on Lot 161C-R immediately south of and parallel to the property line below Lot 97. The soils below the northern 30 feet of the tieback structure have pulled away from the down-slope face, exposing the structure and leaving a scarp approaching 3 feet high. This observation combined with linear slip and ridge features indicate that the upper portions of the soil profile below the tieback structure in this area are actively sliding.

Because the tieback structure lies within the building envelope of the planned structure, its presence and/or removal will impact the development at Lot 161C-R. In order to evaluate the risks this condition presents to the planned development, 2 inclinometers were installed within the slope in this area (MFG-S5I and MFG-S5I(B)), and MFG is actively researching the historical events associated with the unstable slope. Although we have no more information at this time, these observations indicate that the local slopes are susceptible to shallow excavation-induced destabilization.

The primary result of the site conditions summarized above is that shoring along the north and east sides will have to retain significant lateral loads. Excavation shoring analysis and design performed by an experienced registered professional engineer in consideration of the site-specific geological conditions and surrounding development, combined with installation by an experienced contractor, will mitigate potentially unstable cut slopes. Stabilization of the excavation and shoring analysis methods are discussed in more detail in later sections of this report.

8.3.4 Soil Creep

Soil creep is the downslope movement of the surficial soils and uppermost strata of the weathered bedrock under the effects of gravity and surface drainage, and is most prevalent on steep slopes in regions of shallow bedrock. Slight to moderate "pistol-butting" of younger aspen tree trunks on the natural slopes across the site indicates slope creep has occurred and is likely active in these areas. We believe the planned excavation on Lot 161C-R will remove this potential hazard in all areas except for those areas north and south of the planned structures. Creeping soils in these areas will be unaffected (i.e. unstabilized) by the planned development and should be expected to continue. Soil creep rates are difficult to predict; downhill creep on the order of several inches per year would not be unusual, with higher creep magnitudes and rates occurring during wetter periods. Improvements outside the primary building envelope in these areas, including shoring anchors, utilities, patios, paved paths and landscaping, should be designed in anticipation of ongoing soil creep.

8.3.5 Poor Foundation Conditions

The movement and deposition of soils by landslide activity can result in a comparatively soft and poorly consolidated soil deposit, which may present poor foundation conditions for developments planned on such soils. Luedke and Burbank (1976) indicate that the landslide deposit materials are generally poor foundation materials. Lincoln-DeVore (1979) reported that bearing capacity problems can be associated with the unconsolidated landslide debris, and that developments in the undifferentiated slope failure complex materials should be preceded by a detailed geotechnical investigation. Additionally, Lincoln-DeVore (1979) reports that expansive characteristics may exist in some of the fine-grained soils within the landslide deposit. Strata of the Mancos Shale formation and soils derived from it are frequently shown to possess variable shrinking and swelling characteristics upon wetting and drying.

MFG's site-specific geotechnical investigation included drilling 16 borings across the site and collecting representative samples of each material for laboratory testing. Swell/consolidation tests performed on 12 samples of the soils and bedrock exhibited volume changes ranging from -1.4 percent (hydroconsolidation) to 0.6 percent swell when wetted under loads ranging from 50 psf to 3,000 psf. Swell testing of shale samples from nearby sites has shown moderate to high swelling characteristics in thin, widely spaced and discontinuous strata. MFG considers the site soils and bedrock to have predominantly low to non-expansive swelling characteristics. Although higher swelling strata may exist in the subsurface at Lot 161C-R, they are not expected to be of sufficient thickness or distribution so as to impact shoring, foundation, or slab-on-grade performance.

Poor foundation conditions in the form of undifferentiated landslide deposits were identified across the building envelope during our subsurface investigation. The deposits consist of clays with variable amounts of sand, silt and broken shale, overlying intact shale at depths which increase toward the west. The planned excavation will remove all of these deposits except for those along the approximate western one-fifth of the building envelope, where the dipping shale surface drops below the lowest structure level. The impacts of this condition on excavation shoring in affected areas are discussed in later sections.

8.3.6 Groundwater

As described in the SUBSURFACE CONDITIONS section, groundwater was encountered in 13 of 15 borings across the site, and will significantly impact the planned development. Two primary groundwater systems were identified during our field investigation. The surficial system comprises stormwater and snowmelt which is trapped by the shallow sedimentary bedrock and flows within the more permeable upper bedrock strata until it reaches broader shallow subsurface storage at the toe of Coonskin Mountain. The steep hillside configuration results in slight pressurization of the subsurface water; this condition was evidenced by rises in the water levels of 5.1 to 15.6 feet measured in the borings almost immediately after the bedrock surface was reached. A confined system was also identified within a coal-bearing stratum at depth, which brought artesian flow to the surface when the stratum was penetrated with drilling equipment. Flow rates at the ground surface in borings MFG-S2I through MFG-S5I ranged from 0.4 to 5 gallons per minute (gpm) prior to grouting the inclinometers in these borings. Both groundwater systems will affect shoring installation and long term drainage considerations for the development. Further discussion and recommendations for the control of surface and subsurface and drainage are provided in the subsequent sections of this report.

8.3.7 Seismic Considerations

For a seismically inactive state, we believe that designing the structure according to the seismic criteria outlined in this report will protect the structure for design ground accelerations of the magnitudes typically used in this region. However, it is important to understand that these criteria do not address the response of the local landslide complex to the anticipated ground accelerations.

Although the Telluride region is in an area considered to have low seismic activity, our investigation indicates that the seismic stability of the Silver Mountain Landslide Complex, including the Lot 161C-R site, has not been evaluated. Consequently, the risks associated with reactivation of the landslide complex in response to seismic activity cannot be quantified at this time. It is probable that none of the existing development on the Silver Mountain Landslide Complex has been designed in consideration of reactivation of the landslide in response to seismic activity. Luedke and Burbank (1977) reported that "Vibrations related to earthquakes are an ever present possibility but are uncommon in the Telluride area. Only three earthquakes of probable intensity V or larger (Modified Mercalli scale), as well as several small tremors, have been recorded for the entire San Juan Mountains region during the past 100 years, but all were felt in the Telluride area."

8.3.8 Future Stability of the Regional Landslide Complex

Our investigation indicates that the global stability of the Silver Mountain Landslide Complex, including the Lot 161C-R site, has not been evaluated. It is uncertain if the slide mass is presently active; however, no evidence of large scale landsliding has been reported in recent site history. The risks associated with reactivation of the landslide mass in the vicinity of the site are considered to be very low in a seismically inactive state, but are strongly dependent on the stability of the slide mass downhill of the site. Increasing groundwater levels that result from development on landslide deposits have been known to reactivate the slide mass. Reactivation can progressively migrate in an uphill direction toward the scarp of the slide. We are aware of cases where golf course developments in geological conditions similar to those in Mountain Village have reactivated the slide upon which the development was constructed. Reactivation can begin soon after development is complete, or it can take many years to begin. If the slide downhill of Lot 161C-R were to reactivate and migrate to the site, the western portion of the complex where the structure will lie over existing landslide deposits could be impacted.

It is important to realize that the magnitude of the response of the landslide mass to rising groundwater or seismic activity cannot be even generally estimated with the information presently available. Quantifying the response of the landslide complex to seismic activity would require a formal engineering geology investigation of the entire Silver Mountain complex. Because the response of the complex in the vicinity of Lot 161C-R would be dependant on the response of the complex both above and below it, evaluation of the complex only in the immediate vicinity of the site would not yield reliable results. The scale of such an investigation would necessarily be very involved and costly, and we do not expect that these costs would be born by any single entity. If the owner is interested in the scale of investigation described above for the purposes of more clearly defining the level of risk for the development in the event of seismic activity, MFG is well qualified to perform such an investigation, and we would be please to submit a proposal for this work if requested.

8.4 Conclusions

In our opinion, the primary risks to the development of Lot 161C-R are associated with the effects of groundwater and steeply dipping bedrock on excavation shoring construction, and with reactivation of portions of the landslide in response to rising groundwater levels and seismic activity. The recommendations provided in this report are expected to mitigate the effects of the geologic hazards identified, with the exception of the potential for reactivation of all or portions of the Silver Mountain Landslide Complex as discussed. Risks should be evaluated by the owner in consideration of the level of investigation that has been performed to date.

9.0 GEOTECHNICAL ANALYSES

Temporary shoring is predominantly planned for the north and south sides of the complex, where difficulty is anticipated in acquiring permanent shoring easements from adjacent property owners; geological conditions in these areas are favorable for internally braced excavation cuts. However, complex geological conditions at the site along the eastern edge of the building envelope preclude internal bracing of excavations; permanent shoring is planned for this area.

The scope of our investigation included using the results of field and laboratory investigations and review of published geological documentation to develop design geotechnical parameters for shoring design. These parameters were then used to evaluate the feasibility of shoring for the proposed excavation. Both the global stability and the local stability (internal shoring-level) were assessed. The results of our geotechnical analyses are presented in detail below.

9.1 Design Geotechnical Parameters

The predominant subsurface material types identified in our field and laboratory investigations included overburden soils, weathered shale, shale, interbedded sandstone/shale, sandstone, and coal. Bedrock strike and dip were determined from geologic mapping and measured during site-specific geologic reconnaissance. The geologic cross sections presented in Figures 9-1, 9-2, and 9-3 were developed using the results of our field and laboratory investigations. Strength properties for each of the material types were developed from direct shear, triaxial shear, and compressive strength tests, as appropriate for the individual material type. Strength test results are presented on Table 6-1. The design material strength, geological, groundwater, and seismic parameters are presented individually in the following sections.

9.1.1 Design Material Strength Parameters

Design material strength parameters can be issued in several forms, depending on the method of analysis employed. The shoring design consultant requested design material strength parameters in Mohr-Coulomb format. The Mohr-Coulomb material strength parameters summarized in Table 9-1 were developed from the results of our field and laboratory investigations and our experience with similar geologic conditions. Graphs showing the range of laboratory test data and the design material strength envelope for each material are included in Appendix D.

Because the strata represented are rock and not soil, they do not fit the Mohr-Coulomb model in a rigorous sense. For example, the cohesion reported represents an "apparent cohesion" that is used to account for properties such as cementation of the bedrock. However, the strength properties must also consider weaknesses such as bedding planes and geologic factors that are not accounted for in the Mohr-Coulomb analysis.

While the design material strength parameters presented are conservative, they are unfactored, and represent material strengths that we feel can be reasonably relied upon in design analyses for a safety factor of 1.0.

Table 9-1 Design Mohr-Coulomb Material Strength Parameters[1]

Material Type	Total Unit Weight (pcf)	Compressive Strength (psi)	Cohesion (psf)	Friction Angle (deg)
Overburden Soil	127	5	100	20
Weathered Shale	136	15	375	22
Shale	158	6,000	500	39
Interbedded Sandstone/Shale	159	10,000	500	39
Sandstone	158	15,000	1,500	60
Coal	93	1,000	500	33

[1] The design parameters presented are unfactored.

9.1.2 Design Geological Parameters

Design geological parameters include bedrock strike and dip orientation, and any other conditions within the local geology that are determined to be pertinent to slope stability analysis and excavation shoring design. The results of our field investigation and geological reconnaissance indicate the bedrock strike and dip orientation for the Lot 161C-R site summarized in Table 9-2.

Table 9-2 Design Geological Parameters

Bedrock Strike	N20W to N24W
Bedrock Dip	Maximum 36° downward to southwest

As described in the GEOLOGICAL CONDITIONS section, very small, localized and isolated slickenside-like features were observed within the more highly carbonized strata of the bedrock. Due to the bedrock dip angle and the semi-friable texture of the rock in which these surfaces were identified, we were unable to perform laboratory strength tests on these surfaces. Due to the size, discontinuous distribution and wide spacing of these features, we do not believe they indicate local slope movement or present a risk of large scale excavation instability. These surfaces introduce the potential for blocks of rock to loosen in the excavation face, and recommended that shoring facing be designed to support blocks with a nominal dimension of 10 feet cubed. The completed field and laboratory investigations indicate that the potential size of loosened blocks in the excavation will be much smaller than originally anticipated. We do not expect this condition to have any significant impact on the design or performance of the shoring systems.

9.1.3 Design Groundwater Parameters

Groundwater conditions at the site were described in the SUBSURFACE CONDITIONS and GEOLOGICAL CONDITIONS sections. Two piezometric surfaces should be used for the design condition. The conditions reported are for long term design conditions, and presume that the surficial upper groundwater system will remain active and also that the confined system will be effectively drained. It is important to clarify that by 'drained' we do not mean 'emptied'. Once the initial groundwater storage in the confined system is drained off and the artesian pressure is diminished, flow through the coal-bearing zone will be intercepted by the shoring drainage system, thus preventing rebuilding of the confined pressures.

If the coal seam is not dewatered prior to excavation and shoring, the shoring above the coal seam will have to be designed in consideration of the temporary artesian pressures that will exist until the seam is breached and dewatered. As reported in previous sections, the piezometric surface of the coal seam is at an elevation of 19.6 feet above the ground elevation at boring MFG-S3I, which corresponds to an elevation of 9612.6 feet. In the upper groundwater system, the design condition will be that groundwater is flowing in a zone which is 5 feet above and 5 feet below the overburden/shale contact; this condition assumes that groundwater will perch upon the lower permeability bedrock surface, and that the upper 5 feet of the bedrock is highly weathered and may also be transporting groundwater. In the lower groundwater system, the design condition will be that groundwater is flowing in a zone 5 feet above and 5 feet below the coal seam; this condition assumes that the 5 feet of material above and below the coal-bearing strata may also be transporting groundwater. Design groundwater parameters are presented in Table 9-3.

In order to ensure that the long term design condition will be met, it is imperative to ensure that the confined system will be effectively dewatered to remove the artesian pressures, and that these pressures will not recharge. We recommend additional piezometers be installed uphill of the east shoring wall in order to allow monitoring of groundwater levels during shoring construction. Piezometers should be installed prior to commencement of preconstruction dewatering in order to evaluate the response of the confined system to dewatering. If it is determined that the artesian pressures are not easily relieved, or that there is not sufficient confidence that the coal-bearing strata will not drain easily under gravity flow alone, the shoring will have to be designed to include the artesian pressures for the long term design condition. Recommended locations of the additional piezometers are presented in the SLOPE MONITORING PROGRAM section.

Table 9-3 Design Groundwater Parameters

Groundwater System	Design Condition	Affected Shoring Sections	Design Groundwater Parameter
Upper System	Temporary and Permanent	All sections where base of shoring is below the overburden/shale contact or less than 5 feet above the overburden/shale contact	Groundwater is flowing in a 10-foot thick zone 5 feet above and 5 feet below the overburden/shale contact surface
Confined System	Temporary	All sections where base of shoring is below the coal seam or less than 5 feet above the coal seam	Groundwater is flowing in a 10-foot thick zone 5 feet above and 5 feet below the coal seam. The piezometric surface is at elevation 9612 feet to model the artesian condition.
Confined System	Permanent	All sections where base of shoring is below the coal seam or less than 5 feet above the coal seam	Groundwater is flowing in a 10-foot thick zone 5 feet above and 5 feet below the coal seam.

9.1.4 Design Seismic Parameters

The results of our field and laboratory investigations indicate that the subsurface profile falls within Site Class C based on IBC 2003 seismic criteria. This classification results in a maximum considered earthquake spectral response acceleration for short periods, S_{MS}, of 54% g and for a 1-second period, S_{M1}, of 17% g. Published USGS documentation shows a peak ground acceleration of 0.217g for the Telluride region for a 2% probability of exceedance in 50 years. These values of spectral response and peak ground accelerations are typically reduced by some specified factor, depending upon which design methodology is employed by the design engineer.

9.2 Global Stability Analysis

The complex geologic conditions at the Lot 161C-R site initially raised serious concerns about the global stability of the site in response to the planned development. The 'global' stability of a site refers to stability on a scale which is typically larger than the specific area being directly impacted- such as an excavation. For the site-specific geology at Lot 161C-R, the global stability potentially includes any uphill areas which could be affected by excavation at the site; considering the presence of the mapped landslide boundary uphill of the site, the limits of global stability extend up to and beyond the ridge of Coonskin Mountain to the north and east.

Challenging site-specific geological conditions include the location of the site very near to and within the mapped boundary of the Silver Mountain Landslide, combined with sedimentary beds dipping steeply into the excavation. The chief concern for the development initially was reactivation of affected portions of the landslide by the deep excavations planned. It was originally unknown if the slide surface passed deep beneath the lowest planned excavations or if it would be daylighted in the excavation face.

As previously discussed in the GEOLOGICAL CONDITIONS section, our investigation indicates that because the site is located so close to the edge of the landslide boundary, only thin slide deposits remain within the site boundaries. The results of our geological reconnaissance and our field and laboratory investigations show that the slide plane is within approximately 20 feet of the ground surface at the eastern property line, which coincides with the areas of the deepest planned excavation. This condition is favorable for the excavation shoring because only a comparatively thin mass of landslide deposit at the surface will have to be retained; this is in contrast to a much thicker and deeper slide plane within the bedrock itself, which may have proved physically or economically infeasible to stabilize. In conclusion, the results of our investigation indicate that global stability of the site is sound, which reduces the slope stability issue to one of local shoring stability.

9.3 Shoring Feasibility Analysis

MFG performed stability analyses to evaluate the feasibility of shoring for the complex geological conditions at the Lot 161C-R site. Analyses were performed using the computer program SLOPE/W (GEOSLOPE, International, 2004). SLOPE/W solves limit equilibrium slope stability problems by several different methods. Spencer's method was chosen for this study because it considers both force equilibrium and moment equilibrium. Spencer's method and SLOPE/W are widely used in the engineering community.

In order to evaluate the feasibility of shoring at the site, two sections were selected which approximate the worst case shoring conditions at the site- a combination of the highest excavation faces which are also nearly parallel to the bedrock strike. Slope stability analyses were conducted for cross sections A-A' and B-B' as presented on Figures 9-1 through 9-3. Conditions modeled included short term, pre-dewatering conditions and long term drained conditions for both the static and pseudostatic (i.e. seismic) conditions. Our analysis was performed using the parameters presented in Table 9-1 and the Hoek-Brown material strength model for rock masses. The Hoek-Brown model is a shear strength function for rock masses which accounts for geologic parameters in addition to the uniaxial compressive strength of the rock. The stabilization system analyzed consisted of a shotcrete facing with anchors grouted into bedrock beyond the coal seam. The design parameters utilized in our analyses are presented in Table 9-4.

Table 9-4 Shoring Feasibility Analysis Parameters

Parameter	Minimum Value
Reinforcement Angle	15º
Reinforcement Spacing (Vertical and Horizontal)	4 ft
Anchor Length	100 feet
Bond Skin Friction	17,280[1] psf
Bond Safety Factor	2
Anchor Ultimate Tensile Capacity	1,500 kips
Tensile Safety Factor	1.4
Shear Capacity of the steel (lbs)	75% of Ultimate Capacity

[1] Computed based on a bond stress of 120 psi between grout and “soft shale”.

The scope of our analysis was to evaluate the feasibility of permanent shoring for the planned excavation. Consequently, the stabilization configuration was simplified, and iterative analyses were performed until a uniform combination of anchor capacity, length and spacing were achieved which indicated a minimum safety factor of 1.8 for the static case. No attempt was made to further refine the model for the purpose of optimizing design parameters or determining shoring stability at shorter excavation sections; this process is left to the shoring design engineer.

In general, the scenarios evaluated for each cross section included dips of both 26 degrees and 36 degrees to cover the reported range of dip at the Lot 161C-R site. We conservatively assumed vertical shoring faces, although angles of 1:10 (horizontal:vertical) are more likely. The static condition was evaluated for both cross sections, and the pseudostatic (seismic) condition was evaluated at Cross section B-B’. For the pseudostatic condition, a seismic coefficient of 0.17g (78% of the peak ground acceleration) was used. Additionally, the elevated groundwater condition which will occur if the coal seam is not dewatered prior to excavation was analyzed; as previously discussed, this temporary condition will not exist after the coal seam is drained. Slip surfaces were evaluated using circular (grid and radius), block-specified, fully specified, and Auto Locate modes in Slope/W. The various scenarios analyzed are presented in Table 9-5.

Table 9-5 Shoring Scenarios Analyzed in SLOPE/W

SLOPE/W Scenario	Cross Section	Condition	Angle of Dip	Piezometric Surface(s)
1	A-A’	Static Conditions	26	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
2	B-B’	Static Conditions	26	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
3	B-B’	Pseudostatic Conditions (0.17g)	26	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
4	A-A’	Static	36	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
5	B-B’	Static	36	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
6	B-B’	Pseudostatic Conditions (0.17g)	36	Water Table between Overburden Soils and Top Sandstone layer and in Coal Layer
7	A-A’	Static	36	Artesian Condition in the Coal
8	B-B’	Static	36	Artesian Condition in the Coal

In scenarios 1, 2, and 3 cross sections A-A’ and B-B’ were analyzed with the bedding planes inclined at an angle of 26° with the horizontal. The cross sections were evaluated with two water table (piezometric surface) conditions: One water table just below the overburden soils, and the other water table one foot above the coal seam lying between the shale layers. Scenario 3 analyzes the pseudostatic condition for cross section B-B’. The failure surface for Scenario 3 is shown in Figure 9-4. The minimum factor of safety for the critical slip surface in this case was 1.8, with the slip surface being a shallow circular slip fully within the overburden soil.

Scenarios 4, 5, and 6 analyzed cross sections A-A’ and B-B’ with the bedding planes inclined at an angle of 36° from horizontal and with the same water table conditions as scenarios 1-3. The minimum safety factors calculated for the critical slip surfaces were greater than 2.0 for each of the three cases.

In scenarios 7 and 8, the artesian condition in the coal was analyzed to simulate full artesian pressures. The elevation of the top of boring MFG-3SI was approximately 9,593 feet. Based on piezometer readings, an additional 20 feet of total head was added, for a piezometric surface elevation of 9,513 feet. This piezometric surface was applied only to the coal seam zone in cross sections A-A' and B-B'. Only the static case was analyzed, and in both cases the factor of safety was above 2.0. Figure 9-5 shows the outcome of the scenario with the artesian condition for section A-A'.

The results of our analyses show that shoring for the planned excavation at this site is feasible using reasonable anchor capacities and lengths. Structurally, it is not necessary to dewater the coal seam prior to excavating to that level, provided the design accounts for this temporary condition. However, if the coal seam is not dewatered prior to shoring construction, artesian flow will complicate installation of anchors and facing. These issues are discussed further in following sections.

9.4 Inclinometer Readings

MFG installed inclinometers in borings MFG-S2I, MFG-S3I, MFG-S4I, MFG-S5I, and MFG-S5I(B). The inclinometers provide a means of monitoring the slope in which they are installed for movement. Inclinometers can detect movement of the slope at any depth within the inclinometer casing. The depths of individual inclinometers and dates readings have been taken are summarized on Table 9-6. The first reading for inclinometer MFG-S5I(B) has not yet been taken.

The results of the inclinometer readings are presented in Appendix E. The precision of the instruments is ±0.3 inches. The data indicates that no movement beyond the precision of the instrumentation occurred between August 30 and November 20, 2006. Inclinometers will continue to be read according to the schedule suggested in the SLOPE MONITORING PROGRAM section.

Table 9-6 Summary of Inclinometer Installations

Boring ID	Inclinometer Depth (ft)	Date Installed	Baseline Reading	First Reading
MFG-S2I	57	7/19/06	8/30/06	11/20/06
MFG-S3I	91	7/14/06	8/30/06	11/20/06
MFG-S4I	67	7/18/06	8/30/06	11/20/06
MFG-S5I	23	6/19/06	6/28/06 8/30/06	11/20/06
MFG-S5I(B)	30	11/2/06	11/20/06	--

10.0 SHORING RECOMMENDATIONS

The lowest level (G2) will have a finished floor elevation of 9518.5 feet, which will nearly meet existing grades along the west side. Slightly deeper cuts will be required on the west side for small mechanical and pump rooms. Except for excavation adjacent to the gondola complex, shoring will not be required along the west side as finished floor grades are very nearly at finished exterior grades. Underpinning will be required for impacted sections of the gondola complex foundations. Due to the steep mountainside to the east, grades along the east side will not be reached until the 4th floor or 5th floor (7th or 8th level). This configuration will require maximum cuts approaching 82 feet plus foundation excavation depths along the east side of the complex, tapering down to less than 10 feet along the west side. Excavation around the north, east, and south sides of the building envelope will require temporary or permanent shoring of over 900 lineal feet of excavation face. Active groundwater conditions will be encountered during shoring. Shoring drainage considerations are discussed in later sections of the report.

The following sections present our geotechnical recommendations for temporary and permanent shoring systems for the planned excavation at Lot 161C-R. The recommendations are based on the results of our geological reconnaissance, field and laboratory investigations, geotechnical analyses, our understanding of the proposed construction, and our experience with similar projects. The recommendations presented are based on the proposed construction as currently planned; revision of the planned construction could affect our recommendations. If plans change from the assumptions presented herein, we should be contacted to review our recommendations and determine if revisions are needed. The review of plans and specifications for this site is an integral part of the conclusions and recommendations provided in this report. We recommend that project plans and specifications such as the grading plan and shoring plan be reviewed by MFG to verify compatibility with our recommendations.

10.1 Applicable Shoring Systems

Permanent shoring is desired for all shoring conditions. However, temporary shoring may be more practical for sections where excavation heights can be practicably braced internally by the structure. Furthermore, temporary shoring will be required where permanent shoring easements beyond the property lines of affected sections cannot be acquired. Temporary shoring will not be feasible for the difficult geological conditions (i.e. deep excavations, steeply dipping bedrock, groundwater) along the eastern side of the property as the lateral loads required for stability will be too high to be carried by the structure; permanent shoring is planned for these areas provided that permanent long term easements can be acquired from the owners of bordering properties to the east.

Shoring systems discussed to date include soil/rock nailing with reinforced shotcrete facing, drilled tangent soldier pile walls with tiebacks or internal bracing, and H-piles and lagging set in drilled concrete piles. All of these alternatives are considered technically appropriate for the site conditions. However, the viability of each option and its classification as temporary or permanent will be governed by the acquisition of permanent shoring easements. Considerations of permanent shoring easements will be discussed first, followed by recommendations for the individual shoring systems identified above.

10.1.1 Permanent Shoring Easement Requirements

Permanent shoring easements will be required where shoring anchorage zones will extend onto adjacent properties. Legal descriptions will be required defining the lateral and vertical extents that will ensure permanent protection of the shoring anchorage system. In order to protect the uppermost shoring anchors from damage during future development over shoring anchorage zones, the legal descriptions of permanent shoring easements must define a lower threshold elevation below which excavation for improvements above the shored zone is prohibited. Permanent shoring easements should also contain an exclusion of deep foundation systems for future development over shoring anchorage zones.

10.1.2 Soil/Rock Nailing and Tieback Anchors

Soil and/or rock nailing and tieback anchors can be used for both temporary and permanent shoring systems. The design material strengths presented in the GEOTECHNICAL ANALYSES section should be used for design of nails and anchors. The entire anchor bond zone should be beyond (stratigraphically below) the coal seam for all anchors. Design bond strengths in the anchorage zones should be established by the experienced shoring design engineer based on the material strength properties presented in Table 9-1. Considering the importance of the structure from a life/property risk perspective, we recommend a minimum safety factor of 1.5 for the global (external) stability of the permanent shoring.

As discussed in previous sections, groundwater is actively flowing through the upper portions of the bedrock strata, and four of our deeper borings (MFG-S2I through MFG-S5I) encountered an artesian groundwater system within a coal seam at depth, which brought flow to the ground surface. Excavation plans and boring elevations indicate that the coal seam will be daylighted in the northern three-quarters of the east excavation wall and for a short wrap toward the west in the easternmost end of the north excavation wall (See Figure 9-3). Initial artesian flows as high as 5 gpm were measured, which subsequently dropped to 1 to 2 gpm immediately prior to grouting up to a day or so later. Groundwater flowing through the upper bedrock strata and artesian flow in the coal seam will complicate installation of anchors that pass through these strata, and may compromise the integrity of grouting in the anchorage zone beyond the coal seam. Additionally, artesian flow from the coal seam where it is daylighted in the face of the excavation may complicate or inhibit installation of shotcrete facing in these areas. Pre-excavation dewatering of the coal seam would alleviate these conditions. Shoring drainage and dewatering considerations are discussed in later sections of the report.

10.1.3 Drilled Pile Walls

Drilled tangent soldier pile walls (tangent pile walls) and drilled piles with vertical H-piles and lagging are appropriate systems for shoring sections where permanent anchorage easements on adjacent properties cannot be acquired. Tangent pile walls can be designed as permanently cantilevered structures, or they can be supported temporarily with tiebacks or internal bracing. Drilled piles with vertical H-piles and timber lagging can also be supported temporarily with tiebacks or internal bracing. Temporary support loads for both systems are eventually replaced by permanent internal foundation bracing. Timber components of lagging systems should not be incorporated in permanent structures due to long term deterioration and strength degradation.

The drilled piles should be founded in competent bedrock to mobilize the lateral forces required for excavation stability. Recommendations for the design and construction of drilled piles for tangent pile walls and/or vertical H-piles and lagging systems are presented below. These recommendations were developed from analysis of field and laboratory data and our professional experience. The structural engineer should also consider design and construction details which may impose additional design and installation requirements.

Drilled Pile Recommendations

1. Piles should have a minimum embedment of 10 feet into competent bedrock below the planned excavation floor.
2. Piles should be reinforced for the full length of the pile. Reinforcement steel should be designed by the structural engineer to resist the design lateral loads.
3. Pile excavations should be clean prior to placing concrete. Concrete should be placed immediately after the holes are drilled, cleaned and inspected. Ground water is not anticipated to be a problem; however, concrete should not be placed in any hole containing greater than 3 inches of free water. If drilling problems are encountered, MFG should be contacted to discuss alternatives.
4. If casing is used, concrete used in cased piles should have sufficient slump to fill the excavations and not hang on the sides of the casing during extraction of the casing. We recommend a slump in the range of 5 to 7 inches if casing is used.
5. Installation of drilled piles should be observed on a full-time basis by a representative of our firm to identify the proper depth and construction techniques.
6. Care should be taken to avoid ‘mushrooming’ at the tops of the piles.
7. Excavation drainage behind the facing should be provided as outlined in the following sections of the report.

Several methods are available to analyze laterally loaded piles. With a pile length to width ratio of 7 or greater, we believe the method of analysis developed by Matlock and Reese is most appropriate. The method is an iterative procedure using applied lateral load, moment, vertical load and pier size to develop deflection and moment versus depth curves. The computer program LPILE developed by Reese can be used to calculate deflections for the various pier sizes and loading conditions anticipated by the structural engineer. Moment versus depth curves are developed from these analyses to aid the structural engineer in the selection of pier dimensions. Suggested criteria for LPILE analysis are presented in Table 10-1.

Table 10-1 Summary of Geotechnical Properties for Lateral Pile Design

Material Type	LPILE Soil Type	Modulus of Subgrade Reaction, K (pci)	Strain Factor (ε_{50})
Overburden and Weathered Shale	Soft Clay	500	0.005
Shale	Stiff Clay w/ free water	1,000	0.004
Sandstone	Strong Rock	2,000	0.004

The ε_{50} represents the strain corresponding to 50 percent of the maximum principle stress difference. "K" is the modulus of subgrade reaction used by the program to generate the slope of the initial portion of the "P-Y Curves."

10.2 Shoring Deflection

Lateral deflection of the shoring face into the excavation is expected during shoring construction as the shoring system and the soils and rock within the anchor zone equilibrate to the new slope configuration. FHWA (1999) reports that the lateral deflection of anchored shoring systems in stiff clays such as the overburden soils at Lot 161C-R average approximately 2 percent of the shored height (H) with a maximum of approximately 0.5% H. Corresponding vertical deflections (settlements) average 0.15 percent H with a maximum of approximately 0.5% H. Lateral and vertical deflections in anchored systems may be slightly higher in the unconsolidated colluvial and landslide deposits that overlie the bedrock at the site, but are expected to be considerably lower in the bedrock. Lateral movements for non-anchored temporary shoring are commonly estimated to be 1% H at the top of the shoring. Based on the planned excavation and the subsurface conditions identified in our borings, we estimate maximum lateral deflections will be less than approximately 2 inches for the anchored permanent shoring systems, and less than approximately 4 inches for the temporary soldier pile systems. Architectural details should provide a minimum clearance between the structure and the finished shoring face equal to 1 percent of the shored wall height to account for lateral deflections.

Lateral and vertical shoring deflections will likely be differential as the retained and anchorage materials and the shoring types transition across the shoring face. The shoring engineer should verify the estimated deflections previously presented, and should account for the anticipated differential deflections in the shoring face. Construction joints should be provided as necessary to control stresses induced by differential deflections.

A program for monitoring the vertical and lateral deflection of the shoring is presented in a later section. The range of anticipated deflections calculated by the shoring engineer should be conveyed to the Town building department so that they are aware that such deflections are expected and that deflections will be monitored.

10.3 Protection of Adjacent Structures

10.3.1 Northeast Side of Gondola Building

Planned foundation levels adjacent to the southeast side of the gondola complex are on the order of 10 feet below and 6.5 feet away from those of the gondola complex. This configuration will require temporary shoring of the excavation between the two structures. Temporary shoring will both stabilize the foundation soils supporting the gondola footings, and it will also prevent sliding of the steep temporary excavation face, which could cause loss of support to the soils supporting the interior slab at the daycare center and subsequent slab settlement along this edge. We envision that temporary shoring in this area would consist of soil nailing the excavation face. Temporary shoring design in this area should include review of the gondola building structural and foundation plans and soils report. The shoring engineer should calculate anticipated foundation settlements for the gondola building based on existing foundation loads and the shoring system to ensure that they are within tolerable limits. Shoring should be designed according to the recommendations previously provided.

10.3.2 Northwest Side of Gondola Building

Plans also indicate that regrading of the sidewalk area at the northwest face of the gondola building to match the new Lot 161C-R building elevations will require removal of up to 3 feet or more of the foundation wall backfill in this area. Removal of the soils outside the existing foundation wall will reduce frost depth protection, which introduces a risk of frost heave along affected areas. Soil removal will also remove lateral pressures that are balancing those from the backfill inside the foundation wall, which are in turn supporting the interior floor slab. If these lateral forces are not immediately replaced, there is a risk of outward movement of the foundation wall. Both of these conditions could not only have detrimental effects on the structure and interior framing, but could also result in settlement of the interior floor slab along affected portions of this wall. Underpinning will be required to provide continued stabilization of the existing foundation system along affected portions of the gondola building.

Boring MFG-9 encountered refusal at a depth of 19 feet (el. 9519) in two separate attempts. Top-of-foundation elevations of the gondola complex along the Lot 161C-R property line are reported to be on the order of 9519 feet. Bedrock was encountered in boring MFG-7 at a depth of 41 feet (el. 9482'), or approximately 36 feet below the bearing elevation for the gondola building foundation. Bedrock is exposed at the ground surface just southeast of the gondola terminal at an elevation of approximately 9540 feet; thus, the bedrock gets closer to the surface (i.e. landslide deposit gets thinner) closer to the hillside. Based on these observations, it is our opinion that micropiles drilled into bedrock would provide the most cost-effective solution for the likely subsurface conditions in the area to be shored.

In order to design an underpinning system for impacted portions of the gondola complex, it will also be necessary to review the structural and foundation plans to assess the foundation loads in these areas. Depending on the results of the structural review, in addition to the underpinning it may be necessary to nail the existing foundation wall at some prescribed interval, masking the nail heads with stone facing.

Recommendations for micropile foundation underpinning systems are provided below. The micropile recommendation is made under the assumption that most of the post-construction settlement of the gondola building has already occurred. If this assumption is incorrect, underpinning the structure with micropiles would result in differential settlement as the unpinned portions of the building continue to settle. The structural engineer for the gondola building should be consulted to assess the historical performance of the structure and to evaluate the risks of differential settlement associated with underpinning the structure. The geotechnical report for the gondola complex would also provide valuable information about foundation conditions in this area. MFG should be given a copy of the report for review to evaluate settlement issues and to make a final determination of the appropriateness of micropiles for underpinning the gondola building.

Micropile Underpinning Recommendations

1. Micropiles should have a minimum penetration of 10 feet into hard bedrock.
2. Micropile capacities should be calculated assuming a micropile diameter of 3½ inches and a side friction of 2,500 psf for portions of the pile in hard bedrock.
3. Because the engineering properties of the fill or clay soils above the bedrock are unknown, frictional capacities in these materials should be neglected. Micropile capacity from end bearing will be small, and should be neglected.
4. Micropiles should be designed to resist uplift forces due to seismic and/or wind loads. Uplift forces can be resisted using 100 percent of the recommended frictional capacity values.
5. Micropiles should be constructed with minimum 40-20 (outside diameter-hole diameter, in millimeters) rods.
6. Underpinning brackets and jacks should be compatible with the micropile system used, and should be pre-approved by the structural engineer prior to installation.

7. Micropiles should have a center-to-center spacing of at least 6 pile diameters or they should be designed as a group. If it is necessary to have piles in close proximity, we can provide criteria for design of groups.
8. Micropiles should be constructed with freshly mixed grout having a minimum 28-day unconfined compressive strength of 3,500 psi.
9. Pile caps for isolated interior or exterior columns should extend a minimum of 18 inches or 48 inches, respectively, below the finished ground surface for lateral stability and frost protection.
10. If excessively high grout takes or voids which are large enough that they cannot be filled are encountered, the structural engineer should be consulted to advise alternate micropile locations.
11. Micropile foundation installation should be observed by a licensed geotechnical engineer to confirm the proper length and bearing materials, and to check the contractor's installation procedures.
12. One test pile should be constructed for each underpinned wall at a maximum spacing of 75 feet to verify that design capacities and pile settlements are within project tolerances.
13. We estimate settlement of piles designed and constructed according to the above criteria to be less than ½ inch. Differential settlement between piers can equal total settlements.

10.3.3 Structures on Lots 97, 98, 100, and 101

Shoring design should include consideration of the existing residence on Lot 97 and future residential construction on Lots 98, 100, and 101. MFG provided hypothetical future residential foundation design parameters for Lot 97 to the shoring design engineer. These parameters could also be used for Lots 100 and 101.

10.4 Active Slope Failure Below Lot 97

Evidence of active slope instability exists at the southeast corner of the lot between the gondola terminal complex and the residence on Lot 97. A concrete tieback stabilization structure approaching 100 feet long was constructed 8 to 10 feet west of and parallel to the southernmost portion of the east property line adjacent to Lot 97. The soils below the northern end of the stabilizing structure have slipped down the slope leaving a maximum 3-foot vertical separation between the structure and the downhill soil surface.

As the stabilizing structure appears to have been constructed to mitigate the effects of slope movement on the Lot 97 residence, MFG attempted to acquire all much information available describing the events that led to the slope failure. We learned that excavation for the gondola terminal complex on the southwest side of the site in the early 1990’s allegedly destabilized the slope directly above the gondola and directly below the residence on Lot 97. We have been unable to obtain documentation describing the affects the unstable slope may have had on the residence. Personnel from Foley Associates, Inc. indicated that they were contracted to monitor the slope during the events leading to construction of the tieback structure, and provided MFG with plans for two different tieback stabilizing plans for the slope. The existing tieback structure is not consistent with either of the plans. Both plans reference a geotechnical report by Lambert and Associates. We contacted Lambert and they confirmed that they conducted a geotechnical investigation for the slope, but would not release the report without permission from the owner. The report and stabilization plans were performed for The Telluride Company, which no longer exists, and ownership of the gondola lot (Lot 53A) has been transferred several times. Considerable efforts by both MFG and the Lot 161C-R development team have been unsuccessful at locating the authority who can give permission to release the report, but efforts continue.

Development plans for Lot 161C-R indicate that the tieback structure is within or very near the building envelope for the south building. This condition will require removal of portions of the tieback structure in order to accommodate the new structure. The restraining forces of the tieback wall will have to be replaced in order to prevent slope movement. The design of the replacement structure and the sequencing of demolition and reconstruction will be critical to maintain slope stability. It is imperative that MFG review the geotechnical report describing events associated with the slope failure prior to assessing the impacts of the structure on development plans for Lot 161C-R.

Besides the conditions related to the tieback structure, the slope failure at the southeast corner will have other impacts on the planned development. Plans indicate that the permanent shoring along the east property line will extend all the way to the south end of the south building. The ground immediately underlying the footprint of the southernmost end of the south building is known to be in active failure. In order to ensure that foundation piers for portions of the south building that extend southward over the G2-through-lobby-level do not pass through the sliding soil mass, it will be necessary to remove all slide materials below the footprint and replace them with densely compacted imported structural materials prior to drilling foundation piers for this portion of the structure. The unstable soils could be retained from below; however, the unstable soils are not suitable for support of the floor slabs of the new structure, so structural floors would be required if these soils are not removed. Based on the depth to bedrock identified in borings at this location, we anticipate that most of the slide materials beneath the footprint will be removed during excavation for the structure, although thin layers of the slide mass may remain along the west side. Retaining walls benched (and possibly pinned) into the bedrock will be required below impacted portions of the south building to support the replaced soils. Retaining walls in this area should be designed by a registered professional engineer in consideration of the site-specific geological conditions. Concrete cantilever walls or mechanically stabilized earth (MSE) walls are appropriate for these conditions.

MFG will continue investigating the events associated with the slope failure, and will issue the results and our recommendations for shoring and construction in this area at a later time.

10.5 Effects of Soil Creep and Slope Failure on Shoring and Exterior Improvements

As discussed in previous sections, creeping soils will remain outside the north side of the primary building envelope, and actively sliding soils will remain outside the south side of the building envelope. These unstable soils will continue to move downhill after construction of the development is complete. Soil movements will typically be very slow for both creeping and sliding soils, although either could experience accelerated movement after periods of heavy runoff or precipitation.

In addition to shoring design, exterior structural and/or architectural features such as outdoor patio dining, landscaping, utilities, subsurface drainage structures, and other improvements or appurtenances constructed in these areas will also be affected. The impacts ongoing soil movement outside these areas of the complex must be considered during planning and design.

The shoring along the north and south sides should be designed in consideration of the active slope conditions in these areas. If soil or rock anchors are planned for the temporary or permanent shoring at the north and south sides of the project, the upper layers of nails or anchors would extend outward through the actively moving soils. This condition could transfer slope-movement-induced forces directly to the structure. Consequently, soil or rock anchors should be avoided for temporary or permanent shoring systems in these areas.

Foundation walls in these areas can be designed to support lateral earth pressures internally. As the "shored apparent dip" is nearly flat and the unstable soil to be retained is relatively thin (< 10') in these areas, lateral forces on shoring walls are expected to be manageable. If soil anchors are unavoidable in these areas, it should be expected that the heads of these anchors will likely rotate slightly in response to downhill moving or creeping soils in which the anchors are imbedded, and sections of the shoring may be pulled away from the structure. The impacts of moving soils will be more pronounced at the south side, where the potential slope movements associated with the past landslide are greater. Shoring should be isolated from the structure. If mechanical connections between the primary structure and the shoring are unavoidable in areas where soil or rock anchors are also unavoidable, these connections should be designed in consideration of the head rotation or pullout that could occur in response to soil movement behind the shoring. MFG is available to assist the design team if more accurate identification of the locations of movement-susceptible soils is required for shoring design.

10.6 Lateral Earth Pressures

The lateral loads on most of the shoring systems will be determined by formal slope stability analyses. However, some of the lower temporary shoring systems may require conventional lateral earth pressure parameters for design. The design earth pressure is dependent upon the rigidity or constraint of the retaining system, the retained soil type, surcharge loads, loads from adjacent structures, the slope of the backslope surface, and drainage conditions behind the shoring.

Active earth pressures should be used when shoring is free to rotate slightly about its base without causing damage. At-rest earth pressures should be used where there is no tolerance for rotation of the top of the shoring system. We recommend shoring of the overburden soils and weathered shales be designed to resist lateral earth pressure calculated using an equivalent fluid density of 60 pcf for active conditions and 85 pcf for at-rest conditions. The design lateral earth pressures reported above do not include allowances for hydrostatic pressures, or for surcharges induced by traffic or snow loading. We recommend drains be installed behind shoring to prevent the buildup of hydrostatic pressures. Shoring drainage systems are discussed in later sections of the report.

10.7 Excavation

We anticipate that the upper 18 to 35 feet of overburden soil and weathered bedrock can be excavated with conventional heavy excavation equipment. Below these depths, the rippability of the harder bedrock strata will become difficult to marginal; blasting may be required to break harder rock or to hasten excavation where ripping becomes tortuous.

Considering the landslide history of the Mountain Village region, the local strike and dip of the bedrock, local groundwater conditions, the known slope instability at the south side of the lot, and the presence of adjacent structures, dynamic loads which would result from blasting activities could present significant slope stability risks to the project. These risks should be evaluated by an experienced, registered professional blasting engineer in consideration of the site-specific geological challenges, and conveyed to the Owner and its representatives. Blasting should only be performed where absolutely necessary, and when conventional means of mass excavation have been proven unsuccessful or uneconomical.

The blasting program should be designed and overseen by the blasting engineer in consideration of the site-specific geological challenges to mitigate the potential risks. The shoring engineer should coordinate with the blasting engineer to incorporate the anticipated blast-induced seismic loads into the shoring stability analysis and design. Blasting should be performed by an experienced and competent blasting contractor. Blast-induced vibrations should be monitored to ensure allowable limits are not exceeded. Blasting charges should be designed to minimize overblasting and fugitive dynamic loads; excavation of the final face should be accomplished without blasting wherever possible to minimize blast-induced reduction in the integrity of the rock mass quality within the shored wall.

We expect areas which will require blasting to achieve Level G2 elevations will be approximately limited to the east side of a line connecting borings MFG-3 and MFG-9. We estimate that as much as 56,000 bank (in-place) cubic yards (cy) of rock may require blasting. For hauling of the blast rubble, we recommend an expansion of 25%, for a total of up to an estimated 70,000 cy to be hauled. This estimate is based on the depth to refusal encountered in our borings, laboratory test results, and empirical estimates of rippability. The estimate conservatively assumes that all rock below the first strata requiring blasting must also be blasted; it is likely that intermittent lower strata will be rippable. The estimate also conservatively assumes that excavation will proceed to elevation 9518.5 within the full shored perimeter. Blasting of the harder sandstone strata is expected to produce large boulders; secondary blasting, pneumatic hammering, or chemical expansion breaking of oversized boulders should be anticipated.

Soft, wet areas may be encountered during excavation of the clays and softer shale strata. Trafficability in these areas can be improved by placing several inches of gravel.

10.8 Concrete, Shotcrete and Grout

Concrete, shotcrete and grout which comes into contact with the site soils and bedrock can be subject to sulfate attack. Water-soluble sulfate concentrations in samples of the overburden clay, weathered shale, and sandstone ranged from 28 ppm to 575 ppm. According to the American Concrete Institute (ACI), water-soluble sulfate concentrations of 0 to 150 ppm present negligible risk of sulfate attack, and concentrations in the range of 150 to 1,500 ppm present a moderate risk. For these sulfate levels, ACI indicates that moderate sulfate resistance can be achieved by using Type II cement with a water to cement ratio of 0.5 or less. ACI also indicates concrete in moderate exposure environments should have a minimum compressive strength of 4000 psi. We recommend these measures at a minimum for this project.

11.0 DRAINAGE AND DEWATERING RECOMMENDATIONS

Groundwater conditions at the site were previously described in the SUBSURFACE CONDITIONS and GEOLOGICAL CONDITIONS sections of the report. To summarize, two primary groundwater systems were identified during our field investigation. The upper system comprises stormwater and snowmelt which is trapped by the shallow, sedimentary bedrock and flows within the upper 30 to 40 feet of the bedrock until it reaches broader, shallow subsurface storage at the toe of Coonskin Mountain. Stabilized groundwater levels across the site range from 14.7 to 36.2 feet below the existing ground surface; groundwater elevations range from 9497.6 to 9547.3 feet, or 20.9 feet below to 28.8 feet above the level G2 finished floor elevation. A confined system was also identified within a coal-bearing stratum at depth, which brought artesian flow to the surface when the stratum was penetrated with drilling equipment. The groundwater level in the confined system was measured in boring MFG-S3I to be over 19 feet above the ground surface at that location, corresponding to an elevation of 9612.6 feet. Both groundwater systems will affect shoring installation and long term drainage considerations for the development. Preconstruction dewatering is recommended to ensure that groundwater does not adversely affect shoring construction and also to ensure that the design groundwater conditions are maintained.

MFG conducted a flow test in one of the borings and measured or estimated artesian flows in 4 others. These data are presented in the following sections. The results of our field investigation and flow test were used to develop drainage and dewatering recommendations for shoring and excavation activities. MFG also collected groundwater samples for analytical testing for use by others to evaluate the drainage, treatment, and discharge permitting requirements for the Lot 161C-R development.

There is uncertainty associated with the reliability of both the preconstruction dewatering system and the permanent shoring dewatering system in ensuring that artesian pressures do not redevelop in the coal seam within the shored geomaterials uphill of the site. The design assumption for long term slope stability is that the artesian pressures are completely relieved (initially by the preconstruction dewatering and ultimately by the permanent shoring dewatering system). The shoring engineer ran analyses that indicated that if the artesian pressures only drop to half of their current values during dewatering (or similarly, if they redevelop to this level after pre-construction dewatering stops), slope stability safety factors drop below 1.0. Thus, it is imperative to have confidence that this condition will persist for the service life of the structure. The primary factor that will control the artesian pressures is the propensity for the water-bearing zones to drain by gravity. If certainty that the artesian pressures can be controlled and maintained for the service life of the structure cannot be achieved, the only alternative is to redesign the shoring to handle the full artesian pressures for the design life of the development. The propensity of the upper and lower systems to drain by gravity must be evaluated prior to construction of the first levels of shoring so that modifications can be made before shoring construction activities commence, if necessary.

The most practical means of being able to monitor the effectiveness of the dewatering and drainage efforts is to install additional piezometers uphill of the lot to monitor groundwater levels in the upper system and artesian pressures in the confined zone as construction progresses. This will provide us the ability to monitor the pressures in the coal seam during dewatering and shoring construction to ensure that they drop to design levels, and will also permit us to monitor them for a period afterwards to ensure that artesian pressures are not regenerated.

11.1 Flow Quantity Estimates

11.1.1 Surficial Groundwater System

A flow test was conducted in boring MFG-S1. The 2-inch PVC well in this boring was completed to a depth of 60 feet, and the annulus packed with 10/20 silica sand all the way to the ground surface. The subsurface profile at this location consists of 25 feet of overburden clay over layered shale, siltstone and sandstone bedrock strata. The groundwater level since drilling has fluctuated between 35.0 and 38.4 feet, which is always well below the overburden clays. The results of the flow test in boring MFG-S1 are summarized in a memorandum which is included as Appendix F.

Because the well was sand-packed for it's entire depth, the results of the flow test reflect the average flow through the more permeable strata in the upper 35 feet of the bedrock at this location; this zone is within the 'surficial' groundwater system previously described, and flow test results are considered to be representative of this system. In summary, 4.5 hours of pumping at an average rate of 1 gallon per minute (gpm) lowered the water level in the well 10.0 feet. The water level had not stabilized at the time the test was stopped. The simplified interpretation of this data is that the local groundwater levels and the average permeability of the upper 35 feet of the bedrock at this location were not sufficient to keep up with a 1 gpm pump rate, causing the measured drop in the groundwater level. In reality, the majority of this water is probably flowing through only a few of the fractures and more permeable strata in the 35-foot interval tested.

This information is useful in estimating dewatering flows from excavations in areas with similar subsurface profiles. The 4.25" diameter HQ core used in boring MFG-S1 from 35 to 60 feet results in an interior boring surface area of 66.8 square feet. Assuming an equilibrium pumping rate of 1gpm gives an estimated average drainage rate of 1 gpm for each 66.8 square feet of excavation face, or 0.015 gpm per square foot of exposed excavation face in the upper 35 feet of bedrock. Although flows through the rock are expected to diminish as the degree of weathering decreases with depth (with the exception of the coal seam), it is conservative to assume this value for all rock exposed in the excavation. As an example, using this value and the boring log from MFG-S2I (near the deepest excavation) gives an estimated 0.62 gpm of flow per lineal foot of excavation face from the bedrock in this vicinity. Lower excavation heights where less bedrock is exposed will yield lower drainage estimations.

11.1.2 Confined Groundwater System

Initial artesian flow rates from the confined system were measured or estimated in four of our borings between the time the boring was drilled and the time the inclinometers were set in grout, which effectively sealed off the artesian flow. Flow rates were measured in MFG-S2I, MFG-S3I, and MFG-S4I to be 0.4 gpm, 5 gpm, and 2.2 gpm, respectively, and were estimated at 1gpm for MFG-S5I.

Our interpretation of the local geology indicates that the subsurface area of the confined zone uphill of Lot 161C-R is of limited extent. Assuming a bedrock dip of 26 degrees, the coal seam would daylight approximately at or before the ridgeline of Coonskin Mountain to the northeast. Our interpretation of the subsurface extents of the coal seam which could potentially influence development at Lot 161C-R is presented on Figure 11-1. Assuming the confined zone is continuous to its theoretical daylight plane, we estimate the zone has a subsurface catchment and storage area of approximately 9.2 acres. For a dip angle of 36 degrees the storage area is even smaller.

The limited subsurface area of the coal seam has several implications which are very important to the dewatering and drainage issues for Lot 161C-R. First, the limited subsurface area equates to an equally limited storage volume, which translates to a limited initial dewatering volume. Secondly, site data indicate that only a portion of the storage area upgradient of the site is actually storing groundwater.

Our mapping of the theoretical daylight limit of the confined zone indicates that a continuous coal seam could potentially store water to an elevation of approximately 9,950 feet. A piezometric surface (groundwater level) at this elevation would create a pressure head at Lot 151C-R of over 400 feet of water. Overburden ground pressures of this magnitude (over 25,000 psf) would be insufficient to confine the water below the level at which the coal seam exists beneath Lot 161C-R, resulting in seeps, springs, and probably small geysers. In actuality, initial artesian flows less than 5 gpm were measured, and the pressure head measured in boring MFG-S3I indicates a pressure head of only 82 feet (19 feet above the ground surface), which is equivalent to a groundwater surface elevation of just over 9612 feet at this location.

11.1.3 Wetlands

Groundwater issues have been historically persistent in the Mountain Village area. Construction of the core village area in the early 1980's began with filling in natural wetlands. Wetlands existed in the immediate vicinity of the western extents of Lot 161C-R, and extended for several hundred yards to the west, north, and south. The wetlands that exist today along the western property line are remnants of the original wetlands, although their overall hydrology has changed considerably by local development.

Groundwater was measured on November 2 and 3, 2006 in MFG-1, MFG-S6 and MFG-S7 at elevations of 9503.2, 9508.1, and 9497.6 feet, respectively. These levels are 8.9 to 22.4 feet below the estimated wetlands flowline elevations of 9517 to 9520 feet. Additionally, the coal seam is estimated to dip to 120 feet or more below the ground surface at the western property line, indicating that it is not a direct contributor to the wetland system. Discussion with contractors involved with construction of the adjacent gondola and Heritage Crossing complexes indicated that extensive subsurface drainage systems were incorporated in those developments.

These observations combined with the results of our investigation indicate that although the wetlands may have originally been charged by both subsurface water and surface runoff, development in the village core has essentially eliminated the subsurface component, leaving the wetland along the western property line of Lot 161C-R as a surface water feature. In consideration of the planned construction elevations, development of the Lot 161C-R complex will not dewater the wetlands.

11.1.4 Design Dewatering Volumes

Although the interpretations and preliminary analyses presented above are highly simplified, they beneficially illustrate that the subsurface storage in the coal seam is of limited volume. Consequently, initial drainage volumes are expected to be high, but will dissipate quickly as the coal seam is drained. This condition was demonstrated in MFG-S3I, where initial artesian flows of 5 gpm dissipated to 2 gpm in less than 24 hours. Similarly, water stored in the fractures and higher-permeability strata of the upper bedrock surface are also of limited volume. As the coal seam has a comparatively higher permeability than the strata above and below, it will continue to conduct water which is collected in its upgradient subsurface catchment indefinitely. Long term flows in both groundwater systems are expected to be significantly lower than those measured under stored conditions.

Groundwater along the west side of Lot 161C-R is well below the lowest planned excavations, and is not expected to contribute to site dewatering issues. Groundwater is also not expected to rise from below into the excavation floor, although some seepage from upgradient can be expected at the north and south edges of the excavation where water-transmitting strata are exposed in the excavation floor; this seepage is expected to diminish with time as the seams become dewatered. Groundwater is also anticipated in thin fissures along bedding planes within the deeper carbonaceous shale strata; we expect these features will manifest as small seeps when daylighted by excavation, but they are not expected to contribute significantly to the dewatering volumes. Consequently, the only groundwater sources expected to significantly contribute to the dewatering volumes are the surficial and confined systems previously described.

Adding the drainage volumes previously calculated for the surficial and confined systems yields a total initial maximum of approximately 5.6 gpm per lineal foot of excavation face at the highest portions of the excavation. Lower excavation heights where less bedrock is exposed will yield lower drainage estimations. Using these measurements and observations, Tetra Tech (2007) estimated design dewatering volumes of 300 to 400 gpm for the short term, and 30 to 40 gpm for long term.

11.2 Water Quality Sampling and Testing

In order to evaluate the impacts water quality will have on temporary and permanent dewatering discharge for the project, water samples were collected from the upper and confined groundwater systems for analytical water quality testing. An additional boring (MFG-WQ) was drilled during the period of November 28 through December 3, 2006 in order to collect water samples from the artesian groundwater system, and groundwater samples from the upper system were also collected from the previously drilled piezometer in boring MFG-S1.

Spectrum Exploration Inc. drilled MFG-WQ using a Simco 2800 drill. A casing advance air/water drill system with a 0.4-inch outside diameter shoe was used from the ground surface to the bedrock surface. Bedrock was encountered at 20.5 feet below ground surface (bgs) and was wet cored (HQ3 - 2.4-inch diameter) to a total depth of 82 feet bgs. Lost circulation (no cuttings/returns to surface) persisted throughout drilling which resulted in significant water loss to the formation during drilling. At the 72-foot and 77-foot bgs drilling breaks, there was water to the top of the core rods but no measurable discharge. A graphic log of the boring is presented in Appendix A.

The borehole was completed as a piezometer. The piezometer was constructed with 2-inch schedule 40 PVC slotted screen and casing. Fifteen feet of 0.010-inch slotted casing with a bottom cap was installed from approximately 81 feet bgs to 65 feet bgs. Filter pack was not installed. A rubber boot/seal was installed at 58 feet bgs and bentonite pellets were installed above the boot to five feet bgs. An 8-inch diameter 5-foot long PVC surface casing was installed and capped with a flush-mount well cap. The 2-inch PVC was capped with a 2-inch PVC compression fitting, a ¼ -inch stainless steel valve, and topped with ¾-inch threaded PVC. The top of the 2-inch PVC is at approximately 5 feet bgs, the valve is at approximately 4 feet bgs, and the top of the ¾-inch threaded PVC is approximately 2 feet bgs. A peristaltic pump was used to pump the water out of the ¾-inch PVC above the valve, and fiberglass insulation was installed in the surface casing.

Artesian flow was did not reach the surface until the piezometer was partially completed. Once the PVC was installed and partially sealed with bentonite pellets, 4 to 4.3 gallons per minute (gpm) discharge was measured (approximately 3 foot stickup). The discharge was measured again after sample collection but before pressure measurement (after approximately 21.5 hours of unrestricted discharge) at 4.6 gpm with a 1.5 foot stickup.

The piezometer was then sealed and the pressure allowed to stabilize for approximately 2 hours. The pressure stabilized at 13 psi with a 1.5 foot stickup; this pressure is equivalent to a piezometric surface which is 31.5 feet above the ground surface at the boring location, or an elevation of approximately 9589.5 feet.

Groundwater samples were collected from both MFG-WQ and MFG-S1. MFG-S1 was sampled on December 2, 2006 when MFG-WQ had been advance to 77 feet bgs, and approximately 42 hours since drilling had stopped due to mechanical breakdown of the drill rig. The static water level in MFG-S1 was at 37 feet below top of PVC or 37.37 feet bgs. S-1 was purged with a RediFlo2 pump installed to the bottom of the well. Field parameters were monitored throughout purging. The well was purged dry three times (a total of 45 gallons) before sample collection. The pumping rate was then reduced to 100-200 mL/minute for sample collection. MFG-WQ was sampled after approximately 21.5 hours of unrestricted discharge.

Recommendations for analytical tests to be performed and interpretation of the laboratory test results were provided by Tetra Tech RMC, Inc. The recommended field and analytical laboratory tests included cations and anions, RCRA/Priority Pollutant Metals (total and dissolved), total dissolved solids (TDS), pH (field), electrical conductance (field), Tritium content, and semi-volatiles. The analytical laboratory testing was performed by ACZ Laboratories, Inc., with Tritium testing conducted by University of Miami, The Rosentiel School Tritium Laboratory. The field parameter measurements for both wells are presented in Table 11-1. Analytical laboratory results are presented in Appendix G.

Table 11-1 Summary of Field Water Quality Measurements

Well ID	Temperature (°C)	(pH)	Turbidity (NTU)	Conductivity (µS/cm)	Eh (mV)
MFG-S1 (Upper System)	7.3	6.69	136	525	223
MFG-WQ (Confined System)	5.5	6.30	0.54	472	259

11.3 Temporary and Permanent Shoring Drainage

Both groundwater systems will influence the excavation shoring at Lot 161C-R. The temporary shoring drainage will intercept and divert overland flow from above the shoring, and will also intercept and divert seepage from the excavation face to control excess water during shoring construction. The permanent shoring drainage system will intercept and divert both surface flow and seepage behind the shoring for the service life of the development. Weep holes through the shotcrete facing should be included above critical drainage paths (e.g. bedrock surface, coal seam, excavation floor) to provide redundant dewatering relief in the event of failure of the permanent drainage system behind the shoring face. The drainage system should consider freezing temperatures to minimize restriction or blockage of the system from ice buildup. A conceptual permanent shoring drainage detail is presented in Figure 11-2. The permanent shoring drainage system should be designed by a registered professional engineer.

11.4 Preconstruction Dewatering

Groundwater conditions at the site will have several important impacts on shoring installation. First, depending on the shoring design, installation of the first layers of shoring anchors may breach the coal seam. As a result, the initial artesian flows of the magnitudes measured during our field investigation will impact both anchor and facing installation well before the coal seam is daylighted by excavation. Additionally, hauling of wet, heavy soils and shales should be anticipated within the upper 50 or more feet of the profile until drier shales and sandstone are reached. The shoring engineer should review this report to assess the impacts the site groundwater conditions will have on installation and project costs.

Preconstruction dewatering of the site would mitigate both the installation and the hauling issues, and should be evaluated in preliminary costing studies. An additional and potentially significant advantage of preconstruction dewatering is that the temporary drainage system would only need to be designed for the lower long-term flows. The wells would discharge to collection pipes and ultimately to a predetermined outlet point.

11.5 Discharge Permitting

Depending on the destination, discharge from preconstruction dewatering, shoring installation, and long term drainage may be subject to the water quality standards and discharge permitting regulations of the Water Quality Control Division of the Colorado State Department of Public Health and Environment. The necessity for storage and/or water treatment prior to discharge as a part of the final subsurface drainage system design will be significantly impacted by the quality of discharge water. Tetra Tech, Inc. has been retained by the Owner to evaluate water quality issues, to assess discharge and permitting requirements, and to design the subsurface and permanent shoring drainage system for the development.

11.6 Surface Drainage.

Carefully planned and maintained surface drainage practices are essential to the satisfactory performance of shoring systems. Surface drainage should be designed by a professional engineer to ensure that flow from uphill of the shoring is directed away from the shoring, effecting rapid and complete drainage away from the shoring.

12.0 FOUNDATION RECOMMENDATIONS

Subsurface conditions at anticipated foundation elevations across the majority of the building envelope consist predominantly of shale bedrock. Figure 12-1 shows our interpreted contours of the surface of the bedrock across the site. Figure 12-2 presents an interpretive map of the excavated floor at level G2, showing that the bedrock surface drops off sharply below elevation 9518.5 along the west side of the building envelope.

Excavation in steeply dipping sedimentary bedrock results in excavation floor conditions that can change rapidly and dramatically in the direction of the bedrock dip as different bedrock strata are exposed. These conditions result in highly variable bearing capacity and settlement characteristics across the building footprint, which present considerable risks of differential settlement to shallow foundations constructed upon them. Additionally, as discussed in previous sections, the overburden soils which will remain beneath the western side of the building envelope are unsuitable for support of foundations or floor slabs. In our opinion, drilled straight-shaft cast-in-place concrete piers are the most appropriate foundation system for the site. The hard to very hard shale is capable of supporting heavy loads with little deflection. For design of the drilled pier foundations we recommend the following criteria.

12.1 Drilled Straight-Shaft Pier Design Recommendations

1. Piers should have a minimum length of 20 feet and a minimum penetration of 15 feet into hard, competent bedrock, as determined by the geotechnical engineer. We recommend piers be drilled with a large, heavy-duty drill rig (Williams LDH or equivalent) to facilitate the required bedrock penetration.
2. Piers should be designed for a maximum allowable end-bearing pressure of 125 ksf based on pier cross-sectional area. For service load capacity, an allowable skin friction of 10 ksf can be used for portions of the pier that penetrate hard, competent bedrock, ignoring the first 3 feet of pier below grade beams or pier caps, where applicable. The portion of any pier in the overburden clays and/or weathered bedrock should be ignored when calculating frictional capacity.
3. Uplift forces can be resisted using 100 percent of the recommended service load capacity skin friction values plus deadload pressures. Deadload pressures should be designed to be as high as practicable.
4. Actively moving soils exist outside the north and south sides of the building envelope, as discussed in previous sections of the report. Foundations in these unstable soils should be avoided. Where foundation loads which are outside the main building footprint in these areas are unavoidable, such as those used to support ancillary structures like entryway canopies, foundations will need to be evaluated on an individual basis to account for anticipated foundations movements. MFG should be consulted for design recommendations in these areas.
5. We estimate the downward vertical movement of drilled piers designed for the above criteria will be less than approximately 3/4 inch settlement plus loading compression. Differential settlement between piers can equal total settlements.
6. The results of our field and laboratory investigations indicate that the subsurface profile falls within Site Class C based on IBC 2003 seismic criteria. This classification results in a maximum considered

earthquake spectral response acceleration for short periods, S_{MS}, of 54% g and for a 1-second period, S_{M1}, of 17% g. Additional geophysical testing, such as downhole shear wave velocity testing, would be required to confirm the C classification.

7. All piers should be reinforced for the full length of the pier. Reinforcement should extend into grade beams, pier caps, or foundation walls.
8. There should be a continuous void beneath all grade beams, pier caps and foundation walls, between the piers, to concentrate the deadload of the structure on the piers. We recommend a minimum 4-inch void. Voids should be constructed using a moisture-degradable void-forming material such as Sure Void™.
9. Piers should have a center-to-center spacing of at least 3 pier diameters, or they should be designed as a group. If it is necessary to have piers in close proximity, we can provide criteria for design of groups.
10. A minimum pier diameter of 18 inches is recommended. The quantity and size of column reinforcement, or the size of base plates, may dictate the most convenient size of drilled piers. Economy can be achieved by varying the depth of penetration and limiting the number of pier sizes.
11. Pier holes should be cleaned prior to placing concrete. Groundwater is prevalent across the site, and will impact pier construction. It may be possible to construct most piers using a drill-and-pour method. If excessive water develops (more than about 3 inches at time of placement), the pier holes will require pumping and underwater concrete placement. Concrete should not be placed by free-fall methods if more than 3 inches of water is present.
12. We generally do not recommend casing be used in the portion of bedrock where skin friction must be developed. We do not anticipate problems with caving bedrock, although caving overburden soils and landslide deposits may occur along the western side of the project. If caving conditions develop, it may be necessary to case to below the problem zones. If it is necessary to case into the planned side friction zone, the pier length should be increased by the same depth.
13. Concrete should be ready and placed in the pier holes immediately after the holes are drilled, cleaned and inspected, and reinforcing steel set. For cased piers, or tremie placement, we recommend the use of high slump concrete (6 inches ± 1 inch) at the point of placement. Higher slump may be necessary for pumped concrete to achieve the recommended placement slump. The concrete should be designed for the specified strength at the higher slump. At least 5 feet of concrete should be maintained above the groundwater level prior to (and during) casing removal.
14. If free-fall concrete placement is possible, we recommend concrete placement with a reversed chute, or a hopper with a discharge hose, to direct concrete vertically downward to minimize contact with reinforcing steel.
15. Drilled pier foundation installation should be observed by our representative to confirm the piers are bottomed in the proper bearing material and check the contractor's installation procedures.

Piers should be designed to resist lateral loads. Several methods are available to analyze laterally loaded piers. With a pier length-to-width ratio of 7 or greater, we believe the method of analysis developed by Matlock and Reese is most appropriate. The method is an iterative procedure using applied lateral load, moment, vertical load and pier size to develop deflection and moment versus depth curves. The computer program LPILE developed by Reese can be used to calculate deflections for the various pier sizes and loading conditions anticipated by the structural engineer. Moment versus depth curves are developed from these analyses to aid the structural engineer in the selection of pier dimensions. Suggested criteria for LPILE analysis are presented in Table 12-1.

Table 12-1 Soil Input Data For "LPILE"

Input Parameter	**Overburden Soils and Weathered Shale**	**Shale**	**Sandstone, Siltstone**
Density (pci)	0.073	0.091	0.091
Cohesion, C (psi)	100	500	500
Friction Angle, φ Degree	20	39	39
ε_{50} (in/in)	0.005	0.004	0.004
K (pci)	500	1,000	2,000

The ε_{50} represents the strain corresponding to 50 percent of the maximum principle stress difference. "K" is the modulus of subgrade reaction used by the program to generate the slope of the initial portion of the "P-Y Curves."

Other procedures using beam on elastic foundation analysis methods require input of a horizontal modulus of subgrade reaction (K_h). For purposes of design, we believe the hard bedrock can be assigned a uniform value equal to:

$$K_h = 300/d \text{ (tons/ft}^3\text{)}$$

and the clay or fill overburden soils to have a value equal to:

$$K_h = (30)/d \text{ (tons/ft}^3\text{)}$$

where d = pier diameter (ft).

12.2 Foundation Wall Backfill

Foundation backfill will be required between the shoring and the structure foundation along portions of the north side of the building. Backfill heights will approach 40 feet at the entry drive, but will be less in other areas. The excavated bedrock materials are not suitable for foundation backfill. The overburden clays are acceptable for use as foundation backfill; however, they possess a higher propensity for settlement. Consequently, they should not be used in areas where slabs-on-grade will be placed over foundation backfill, such as at critical entryways. The top 2 to 3 feet of backfill in unpaved areas should be clayey soils to reduce water infiltration. Imported structural fill is recommended for areas where slabs-on-grade, pavement, or appurtenant structures will be placed on foundation backfill, such as at the entry drive. In such critical areas, the width of the structural backfill zone should be no less than twice the height of backfill (backfill should extend ½ the backfill height on both sides of the critical area.

Imported structural fill should consist of silty or clayey sands containing 3 to 30 percent fines (passing the No. 200 sieve) with a liquid limit less than 30 and a plasticity index less than 15 (CDOT Class 6 road base meets these criteria). Imported structural fill should be approved by MFG for use as backfill prior to delivery to the site.

We recommend foundation wall backfill be moisture-conditioned to ±2 percent of optimum water content; moisture should be added and the soils thoroughly mixed to provide uniform distribution of water throughout the fill. Fill should be placed in lifts with a maximum loose thickness of 8 inches. Backfill which will not support pavement or flatwork should be compacted to at least 90 percent of ASTM D 698 maximum dry density. Backfill which will support pavement or flatwork should be compacted to at least 95 percent of ASTM D 698 maximum dry density.

Our experience indicates backfill will settle even if properly compacted. Utility pedestals or other appurtenant structures should be located outside backfill zones. Slabs or pavements at critical entryways to the structure can be supported on haunches.

12.3 Foundation Drainage

Groundwater seepage from the excavation walls will persist for the service life of the structure. A permanent dewatering system is planned behind the shoring, which will control the majority of water. Although we do not anticipate groundwater to contribute significantly to lateral foundation pressures, exterior perimeter foundation drainage is recommended to ensure removal of any water that might bypass the shoring drainage system from the edges or from below.

A conceptual sketch of an exterior perimeter drain is presented as Figure 12-3. The foundation drain should outlet to daylight, a sump pit, or to the municipal storm drainage system. Municipal building officials should be contacted to coordinate the latter as approval and permits may be required. If the outlets for the foundation drain do not terminate in a heated space, they should be designed in consideration of freezing winter temperatures to ensure that they are not blocked by ice formation and accumulation.

12.4 Lateral Earth Pressures

Maximum backfill heights of 40 feet are anticipated. Below-grade walls which retain earth should be designed to resist lateral earth pressures. The design earth pressure is dependent upon the rigidity or constraint of the foundation wall, the backfill material type, surcharge loads, loads from adjacent structures, the slope of the backfill surface, and drainage conditions behind the walls.

We recommend below-grade foundation walls be designed to resist lateral earth pressure calculated using the equivalent fluid density for this site listed in Table 12-2. Active earth pressures should be used when walls are free to rotate slightly without causing damage. At-rest earth pressures should be used where there is no tolerance for rotation of the top of the wall. The design lateral earth pressures reported above do not include allowances for hydrostatic pressure on walls, or surcharges induced by traffic or snow loading. Exterior perimeter foundation drains will prevent the buildup of hydrostatic pressures outside foundation walls.

Table 12-2 Equivalent Fluid Densities for Site Soils

Backfill Material	Active	At Rest
On-Site Clays	60	85
Excavated Bedrock	Not recommended	Not Recommended
Imported Structural Fill (e.g. CDOT Class 6)	35	55

13.0 FLOOR SYSTEMS

The majority of the floor of the building footprint at the G2 level (elevation 9518.5) will consist of shale and sandstone bedrock. As shown on Figure 12-2, the bedrock surface dips sharply beneath this level along the western side of the site, leaving overburden soils beneath the building footprint in this area. The overburden soils consist of unconsolidated colluvial and landslide deposits, and are unsuitable for support of floor slabs. These materials approach depths of 10 to 40 feet along the western property line; thus, the anticipated removal volumes and shallow groundwater conditions are expected to make removal of these materials economically unviable. Structural concrete slabs are recommended for areas where overburden materials remain beneath floors.

Swell/consolidation tests performed on 6 samples of the bedrock indicate they have very low to non-swelling expansive characteristics. Although thin, isolated bedrock strata may possess higher swelling potential, we believe the pervasive groundwater conditions at the site will mitigate any heaving problems that might exist. Consequently, we believe the risks to slab-on-grade floors associated with heaving bedrock are very low for this site, and consider slab-on-grade floors appropriate. Visual observation of the excavation floor for potentially expansive materials will provide further reduction of the risks of detrimental differential slab heave.

The recommended shoring and foundation drainage systems will control the majority of water that could reach interior floor areas. However, the local geological and groundwater conditions create the potential for seepage through bedrock joints and fractures anywhere within the building footprint, although the volumes of such seepage are expected to be very low and easily controlled with underslab drainage gravel. We recommend slabs be placed on a 6-inch layer of free-draining rock (e.g. ¾" screened rock) both to provide a capillary break and also to allow for drainage beneath the slab. The subgrade should drain at a minimum slope of 1% to a drain system, including collector pipes or sump pits, where water can be removed by pumping. A conceptual sketch of a below-slab drain is presented as Figure 12-3.

Interior slab-on-grade floors and exterior flatwork should be designed and constructed in accordance with the following criteria.

1. Slabs should be separated from exterior walls and interior bearing members with a slip joint which allows for free vertical movement of slabs.

2. Where slab bearing partitions are necessary, a slip joint (or float) allowing at least 1½ inches of free vertical slab movement should be provided. If the float is constructed at the top of partitions, the connections between slab-supported partitions and foundation-supported walls should be detailed to accommodate differential movement. The owner must maintain these joints.

3. Underslab plumbing should be eliminated where possible. Where such plumbing is unavoidable, it should be thoroughly pressure-tested for leaks before slabs are constructed and should be provided with flexible couplings. We do not recommend directing roof drains below slab-on-grade floors.

4. Plumbing and utilities which pass through slabs should be isolated from the slabs. Heating and air conditioning systems supported by the slabs should be provided with flexible connections. For this design, we recommend conservatively assuming 1½ inches of slab movement for exterior slabs and parking levels.

5. Except for the case of structural slabs at sensitive entryways, exterior slabs should be isolated from the foundations. These slabs should be well-reinforced to function as independent units.

6. Frequent control joints should be provided to reduce problems associated with shrinkage and curling in accordance with recommendations of the American Concrete Institute. For the lower level slabs we advocate an additional control joint about 3 feet inside walls.

7. The underslab drainage system should be interfaced with the shoring and foundation drainage systems, and should be designed by a registered professional engineer.

14.0 SLOPE MONITORING PROGRAM

We recommend a slope monitoring program be implemented to track ground movements at critical locations. The program should include continued readings of the existing slope inclinometers and the installation of additional inclinometers and survey monuments at critical locations. The existing inclinometers will show movement of the soils immediately behind the shoring facing as the shoring system and shored soils equilibrate, but will not give an indication of the slope movement outside the influence of the shoring. Additional inclinometers should be installed uphill and beyond the zone of influence of the shoring system. Figure 14-1 presents recommended locations of additional inclinometers and survey monuments. In addition to survey monuments behind the shoring walls, monuments should also be established on the shoring face.

Monitoring should include precision surveying of the northing, easting, and elevation of all survey monuments and inclinometer flush-mount covers, and regular periodic reading of the inclinometers. The results of each set of readings should be summarized and discussed in a letter report. We recommend weekly readings during excavation and shoring construction, continuing for 4 weeks after shoring construction is complete. Inclinometers and monuments should be read monthly during construction of the of the structure, continuing for 12 months after construction is complete, and should be read annually for 5 years after construction is complete.

In order to protect the interests of MR 1.81 LLC, MFG recommends a thorough assessment of the existing structures on all sides of the proposed structure be performed by a licensed structural engineer. The assessment should include photographic documentation and surveying, if warranted, to accurately document the pre-construction structural conditions and elevations of the existing structures.

NOTE: The building envelope was modified after inclinometers MFG-S2I through MFG-S5I were installed. Consequently, MFG-S2I and MFG-S5I will eventually be destroyed by excavation. MFG-S5I was replaced by MFG-S5I(B). Inclinometer MFG-S2I should be replaced by an additional inclinometer located at the extreme northeast corner of the property. The locations of the remaining inclinometers should be surveyed to confirm that they are located far enough from the shoring that they will not be damaged by construction; if any of the existing inclinometers lies on or within the shoring alignment, they too should be replaced with additional inclinometers. Efforts should be made during shoring construction to prevent disturbing the existing inclinometers along the eastern property line.

15.0 QUALITY ASSURANCE/QUALITY CONTROL PROGRAM

The review of plans and specifications for this project and geotechnical engineering inspections are an integral part of the conclusions and recommendations provided in this report. We recommend that project plans and specifications, such as the grading, shoring, and foundation plans, be reviewed by MFG to verify compatibility with our recommendations. If we are not retained to review the project plans and specifications and perform geotechnical engineering inspections, MFG's cannot assume responsibility for potential claims that may arise from inconsistencies with our geotechnical recommendations or subsurface conditions that vary from those anticipated from our investigation.

Regular inspections and testing should be performed to ensure that the subsurface conditions encountered in the field are consistent with those anticipated from our investigation, and that construction materials and techniques meet project specifications. The QA/QC consultant should be contracted by and report directly to the Owner, and should be independent of the general contractor. Inspections and testing will include, at a minimum, the following:

1. Geotechnical engineering inspections by MFG of each excavated level of shoring during excavation and prior to the application of shotcrete facing to verify the shored materials are as anticipated and that the permanent shoring drainage system is installed as designed;
2. Inspection and materials testing of shoring structural materials, construction methodologies and workmanship;
3. Verification that anchor load tests meet the design requirements;
4. Observation of pier drilling to ensure that piers are founded in proper bearing strata, that minimum lengths and penetrations are achieved, that groundwater issues are properly addressed, and that the steel reinforcement meets design specifications; and
5. Testing of concrete for drilled piers, foundation walls and slabs-on-grade.

16.0 LIMITATIONS

Our borings were located to obtain a reasonably accurate characterization of subsurface conditions which will affect shoring and foundation performance for the planned development of Lot 161C-R. Variations in the subsurface conditions not indicated by our borings are always possible. Actual subsurface conditions should be verified by a representative of our firm during construction.

We believe this investigation was conducted in a manner consistent with the level of care and skill ordinarily used by geotechnical engineers practicing in this area at this time. No warranty, expressed or implied, is made.

If we can be of further service in discussing the contents of this report, or in the analyses of the influence of the subsurface conditions on the design of the structures, please do not hesitate to call us.

Sincerely,

MFG, INC.

Mark S. Abshire, MS, PE
Senior Geotechnical Engineer

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APPENDIX A

LOGS OF EXPLORATORY BORINGS

APPENDIX B

LABORATORY TEST RESULTS

Appendix B.1
Smith Geotechnical Engineering Consultants, Inc.
Project No. 2006.038T
June 6, 2006

Appendix B.2
Smith Geotechnical Engineering Consultants, Inc.
Project No. 2006.080T
January 3, 2007

Appendix B.3
Advanced Terra Testing, Inc.
Job No. 2540-22

Appendix B.1

Smith Geotechnical Engineering Consultants, Inc.
Project No. 2006.038T
June 6, 2006

Appendix B.2

Smith Geotechnical Engineering Consultants, Inc.
Project No. 2006.080T
January 3, 2007

Appendix B.3

Advanced Terra Testing, Inc.
Job No. 2540-22

APPENDIX C

VIBRATING WIRE PIEZOMETER CALIBRATION REPORTS

APPENDIX D

DESIGN MATERIAL STRENGTH ENVELOPES

Appendix D.1
Design Shear Strength Parameters

Appendix D.2
Design Compressive Strength Parameters

Appendix D.1

Design Shear Strength Parameters

Appendix D.2

Design Compressive Strength Parameters

APPENDIX E

INCLINOMETER READINGS THROUGH NOVEMBER 20, 2006

APPENDIX F

RESULTS OF MFG-S1 FLOW TEST

APPENDIX G

WATER QUALITY ANALYTICAL TEST RESULTS

Appendix G.1
ACZ Laboratories, Inc.
Analytical Report: Project ID l60239
January 8, 2007

Appendix G.2
University of Miami
The Rosentiel School: Tritium Laboratory
Data Release #07-007
Job # 2291
January 4, 2007

Appendix G.1

ACZ Laboratories, Inc.
Analytical Report: Project ID l60239
January 8, 2007

Appendix G.2

University of Miami
The Rosentiel School: Tritium Laboratory
Data Release #07-007
Job # 2291
January 4, 2007

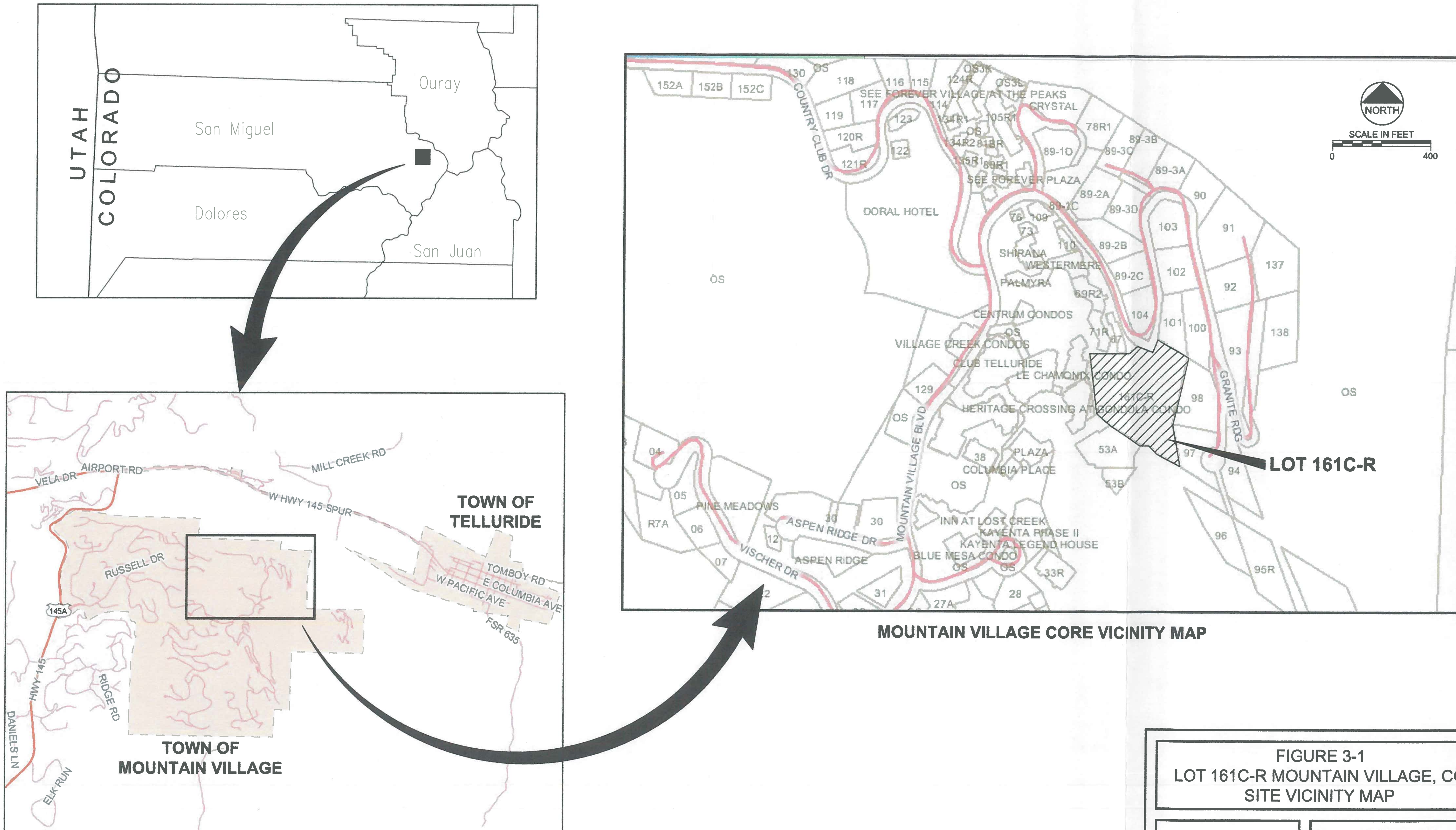

FIGURE 3-1
LOT 161C-R MOUNTAIN VILLAGE, CO
SITE VICINITY MAP

MFG, Inc.
consulting scientists and engineers

Date:	OCTOBER 2006
Project:	181308
File:	VICINITY-01.DWG

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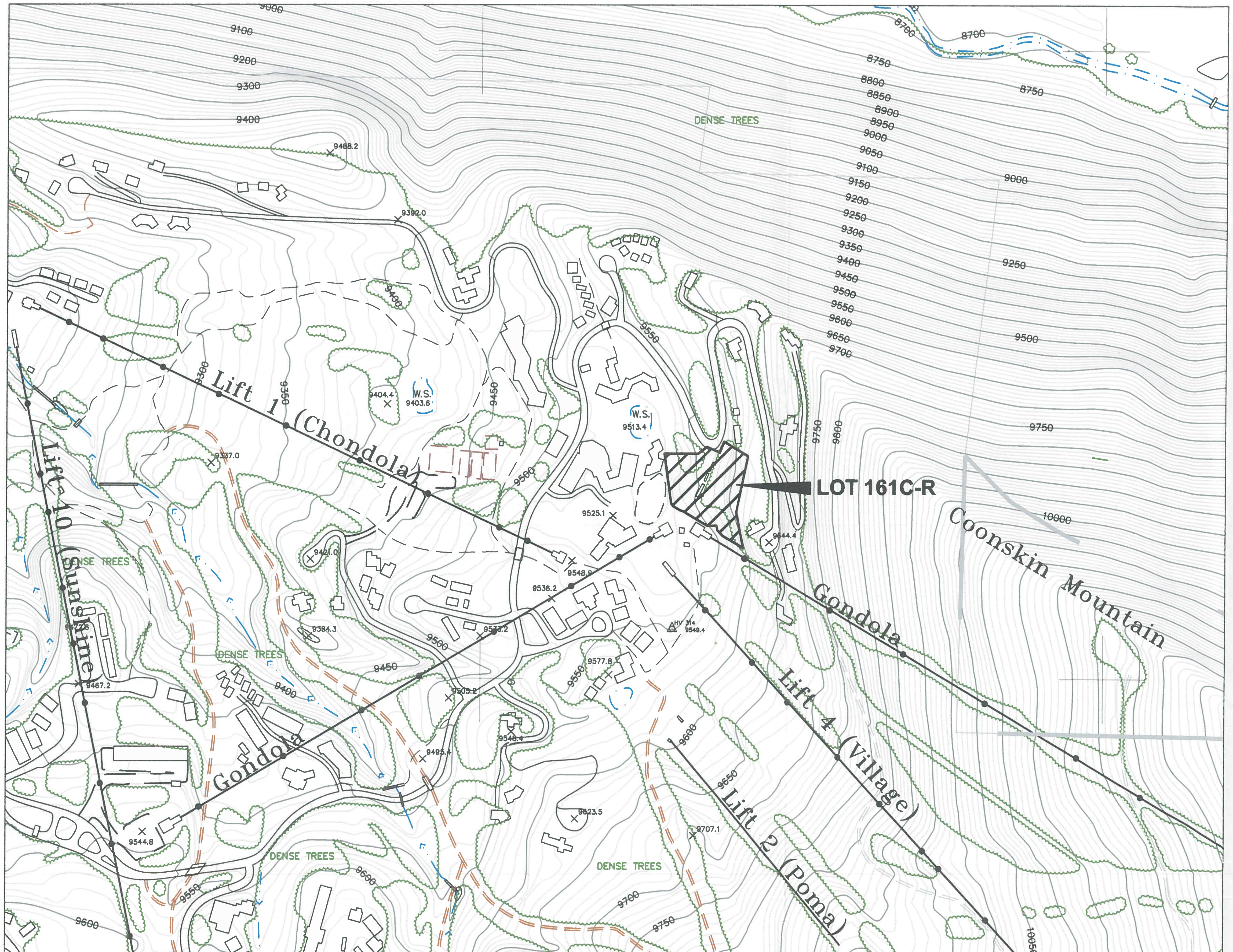
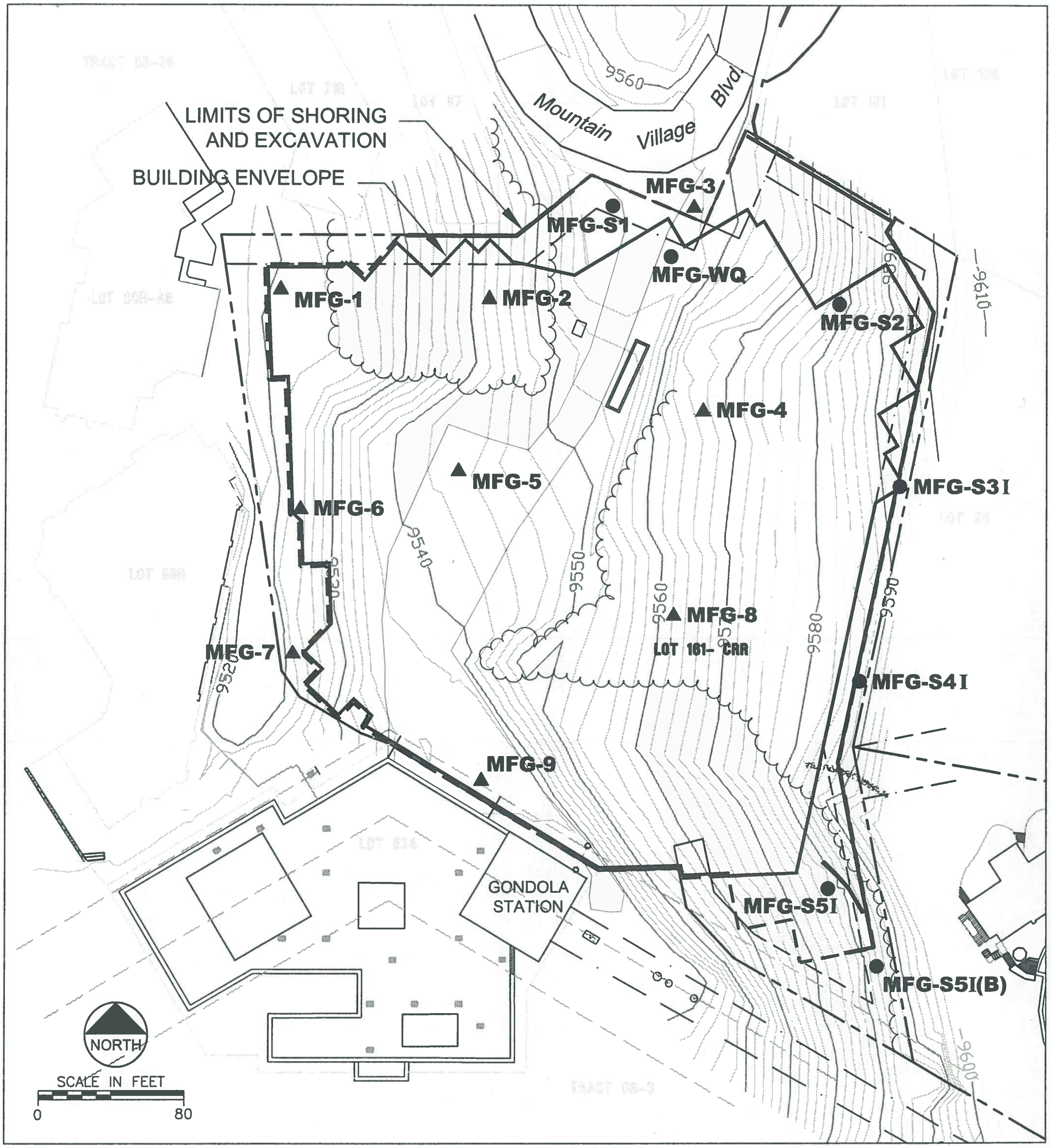


FIGURE 3-2
LOT 161C-R LOCAL TOPOGRAPHY

MFG, Inc.
consulting scientists and engineers

Date:	NOVEMBER 2006
Project:	181308
File:	TOPO-LOCAL.DWG

E:\181308\TOPO-LOCAL.dwg



LEGEND

● SHORING INVESTIGATION BORING LOCATION

▲ FOUNDATION BORING LOCATION

NOTES:

1. AN "I" AFTER THE BOREHOLE DESIGNATION INDICATES THAT AN INCLINOMETER WAS INSTALLED IN THAT BORING.
2. CONTOURS ARE FROM TOPOGRAPHIC SURVEY PREPARED BY FOLEY ASSOCIATES, INC. (FINAL DRB EXISTING CONDITIONS, OCTOBER 3, 2006).

LOT 161C–R
MOUNTAIN VILLAGE, CO

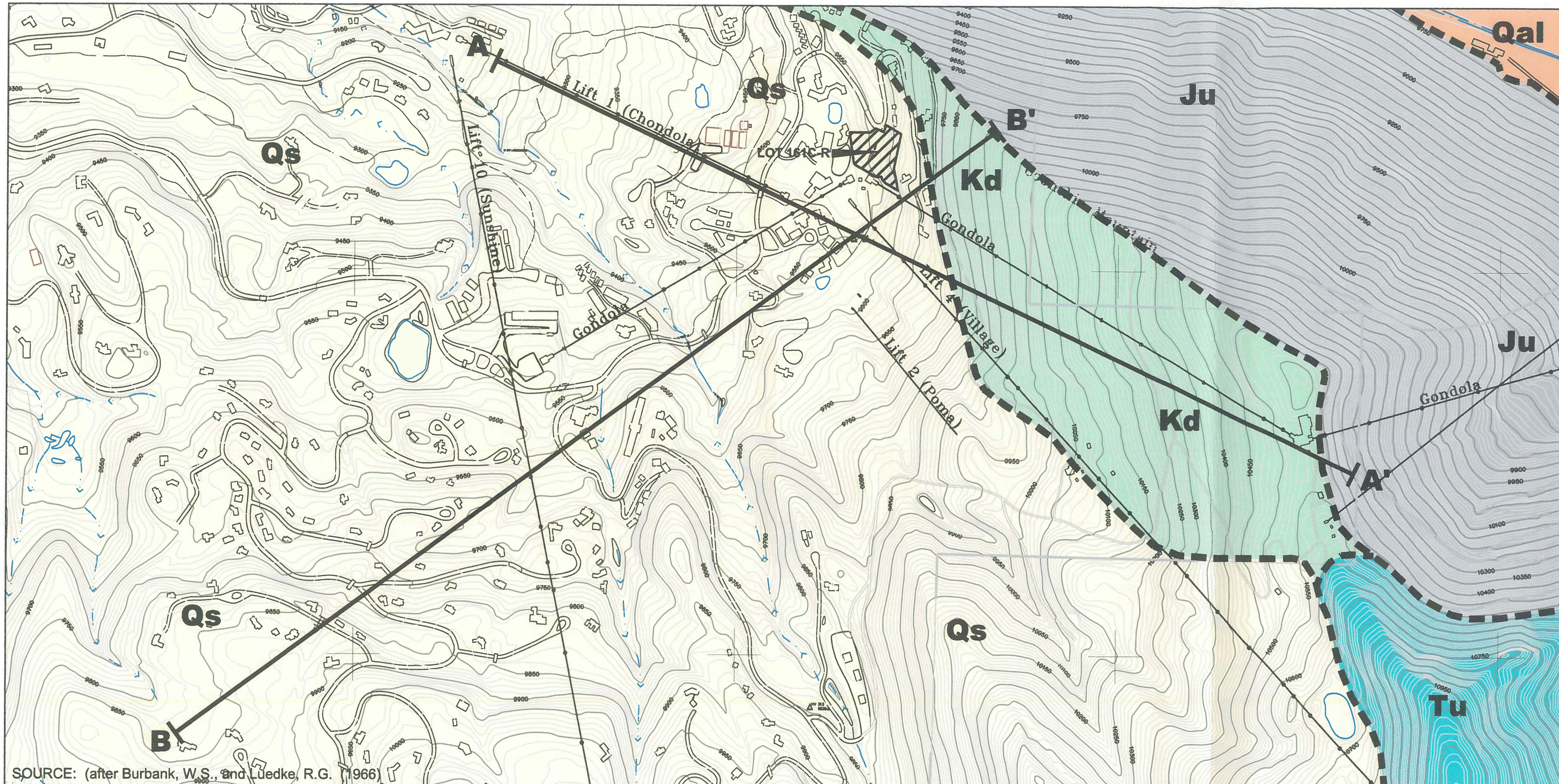
FIGURE 5–1

BORING LOCATION MAP

PROJECT: 181308	DATE: NOVEMBER 2006
REV:	BY: TGB CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

E: \181308\BH-L...02.dwg



SOURCE: (after Burbank, W.S., and Luedke, R.G. 1966)

LEGEND

Qal	QUATERNARY ALLUVIUM
Qs	LANDSLIDE DEPOSITS: SANDY TO SILTY CLAY WITH SAND, GRAVEL AND CLAYSTONE FRAGMENTS
Tu	TERTIARY UNITS, UNDIFFERENTIATED
Kd	DAKOTA SANDSTONE: LOCALLY GRAY QUARTZITIC SANDSTONE WITH GRAY TO BLACK CARBONACEOUS SHALE AND MINOR, THIN, DISCONTINUOUS LOW GRADE COAL
Ju	JURASSIC UNITS, UNDIFFERENTIATED

CROSS SECTION LOCATION

GEOLOGICAL CONTACT

NORTH

SCALE IN FEET

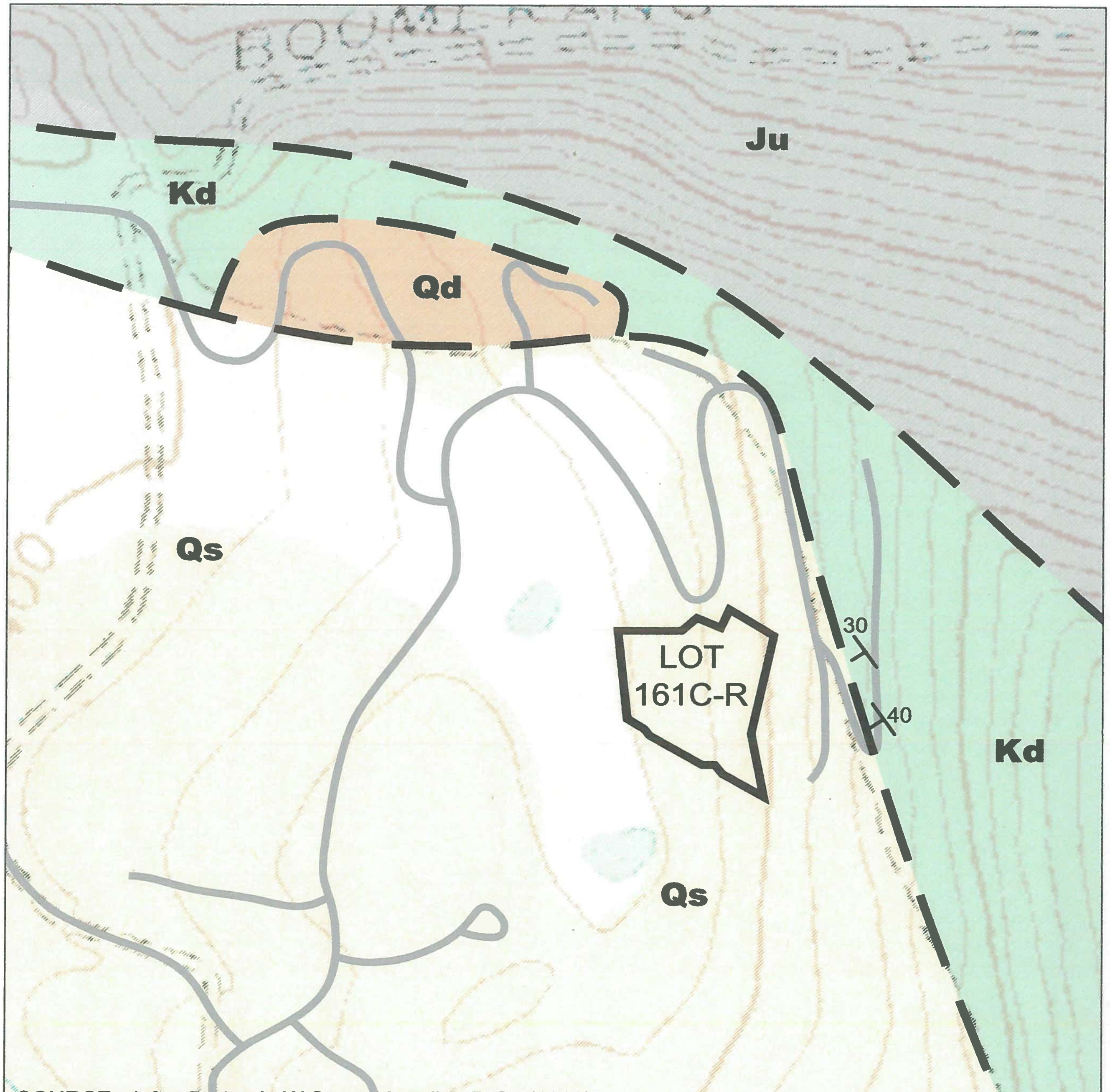
0 800

FIGURE 6-1
GEOLOGY AND TOPOGRAPHY

MFG, Inc.
consulting scientists and engineers

Date: NOVEMBER 2006
Project: 181308
File: GEOLOGY-TOPO.DWG

E: 0181308\geology-topo.dwg



SOURCE: (after Burbank, W.S., and Luedke, R.G. (1966)

SCALE IN FEET
0 400
NORTH

LEGEND

Qs	LANDSLIDE DEPOSITS: SANDY TO SILTY CLAY WITH SAND, GRAVEL AND CLAYSTONE FRAGMENTS
Qd	GLACIAL DRIFT: UNCONSOLIDATED AND UNSORTED BOULDER TO SILT SIZED ROCK FRAGMENTS IN A CLAY MATRIX
Kd	DAKOTA SANDSTONE: LOCALLY GRAY QUARTZITIC SANDSTONE WITH GRAY TO BLACK CARBONACEOUS SHALE AND MINOR, THIN, DISCONTINUOUS LOW GRADE COAL
Ju	JURASSIC UNITS, UNDIFFERENTIATED
┬	INDICATES BEDROCK DIP AND STRIKE DIRECTION OBTAINED IN FIELD BY MFG

LOT 161C-R
MOUNTAIN VILLAGE, CO
FIGURE 6-2

SITE GEOLOGY

PROJECT: 181308	DATE: NOVEMBER 2006
REV:	BY: TGB / CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

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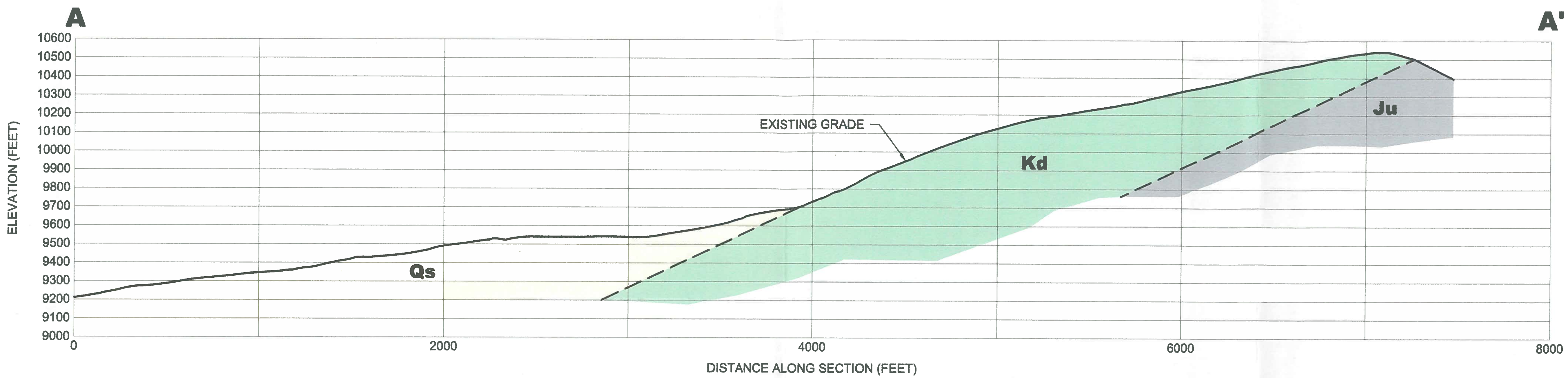
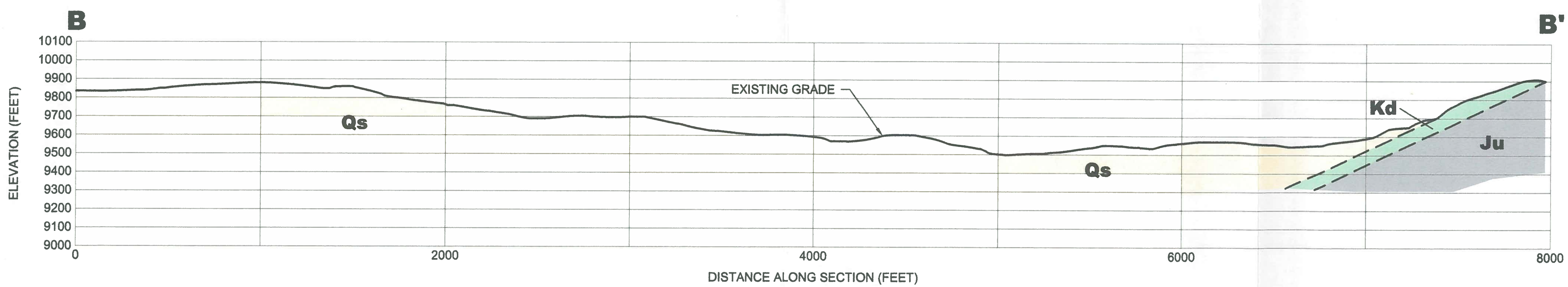




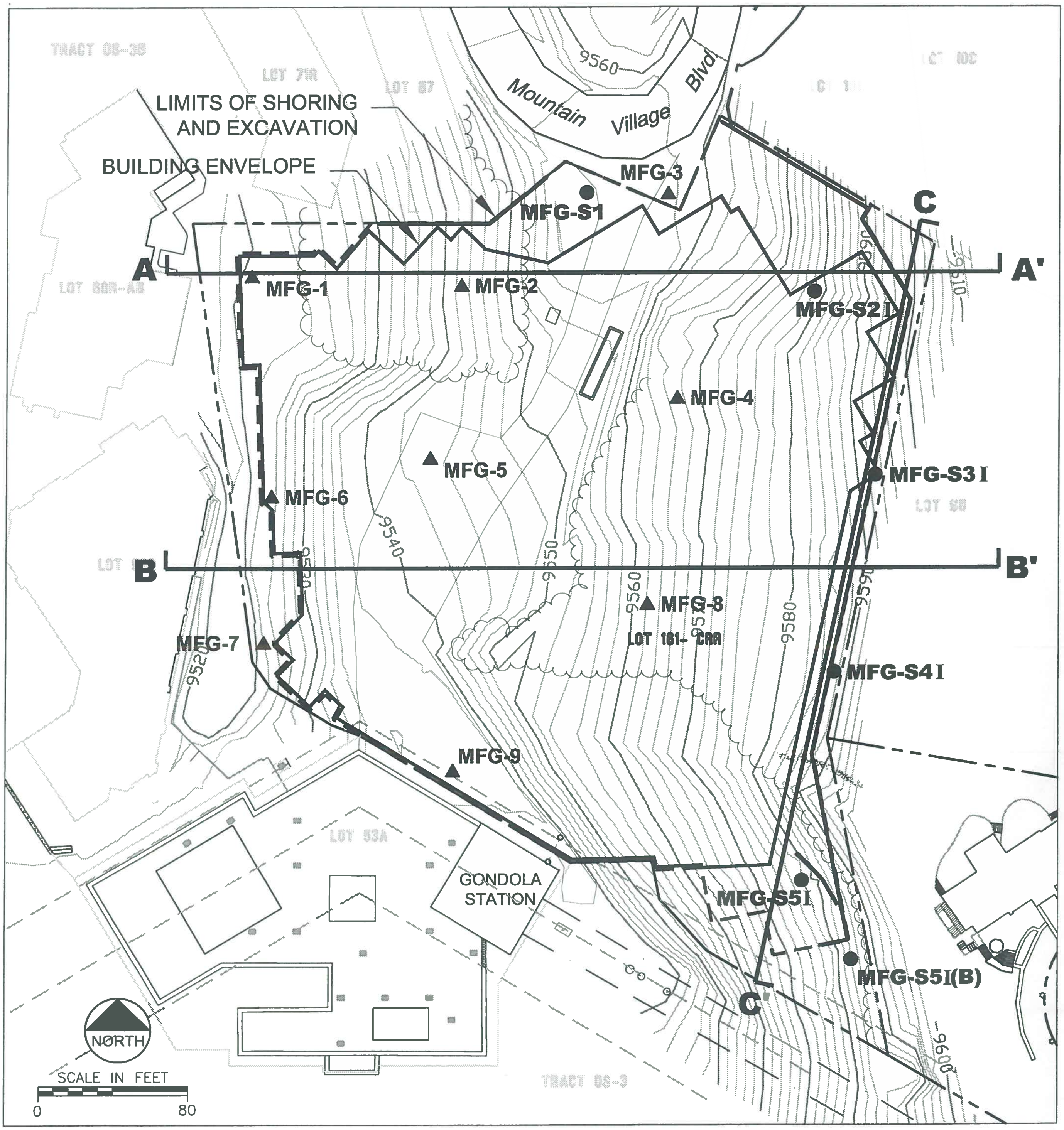
LEGEND

Qs	LANDSLIDE DEPOSITS: SANDY TO SILTY CLAY WITH SAND, GRAVEL AND CLAYSTONE FRAGMENTS
Kd	DAKOTA SANDSTONE: LOCALLY GRAY QUARTZITIC SANDSTONE WITH GRAY TO BLACK CARBONACEOUS SHALE AND MINOR, THIN, DISCONTINUOUS LOW GRADE COAL
Ju	JURASSIC UNITS, UNDIFFERENTIATED

FIGURE 6-3
GEOLOGIC CROSS SECTIONS

MFG, Inc.
consulting scientists and engineers

Date: NOVEMBER 2006
Project: 181308
File: GEOLOGY-TOPO.DWG



LEGEND

MFG-S1● LOCATION OF EXPLORATORY BORING

1. AN "I" AFTER THE BOREHOLE DESIGNATION INDICATES THAT AN INCLINOMETER WAS INSTALLED IN THAT BORING.
2. CONTOURS ARE FROM TOPOGRAPHIC SURVEY PREPARED BY FOLEY ASSOCIATES, INC. (FINAL DRB EXISTING CONDITIONS, OCTOBER 3, 2006).

LOT 161C–R MOUNTAIN VILLAGE, CO	
FIGURE 9–1	
CROSS SECTION LOCATION MAP	
PROJECT: 181308	DATE: NOVEMBER 2006
REV:	BY: TGB	CHECKED: MSA
MFG, Inc. *consulting scientists and engineers*	

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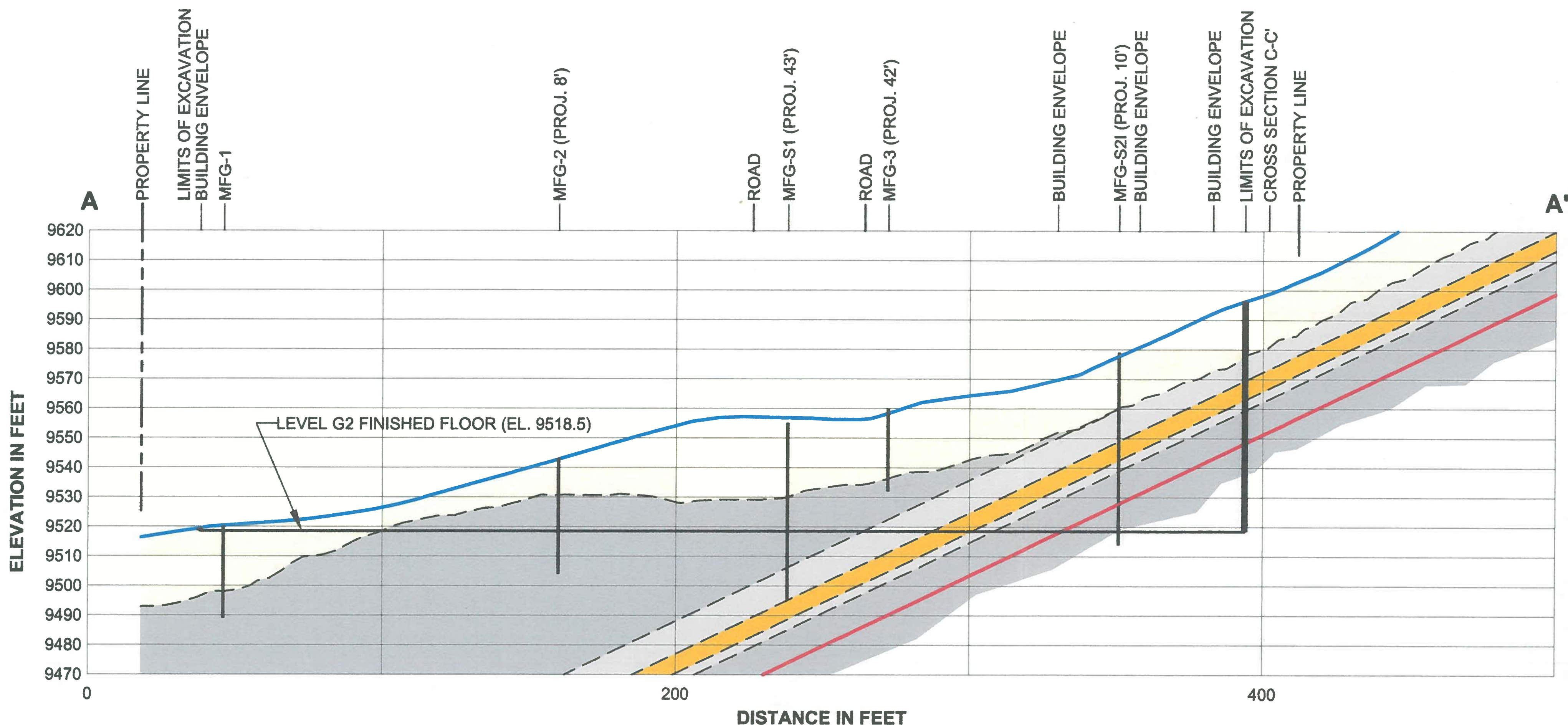

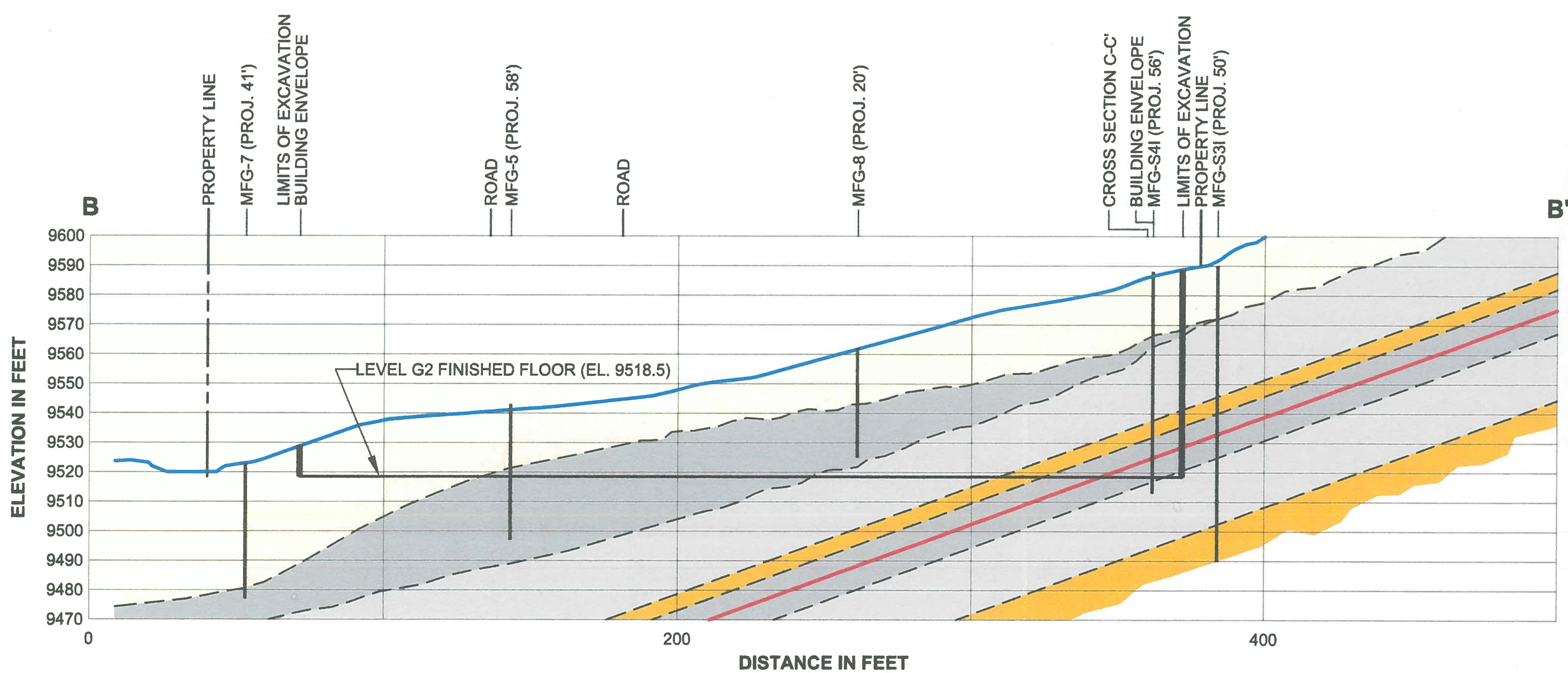

LEGEND:

- LANDSLIDE DEPOSIT
- SHALE
- INTERBEDDED SANDSTONE AND SHALE
- SANDSTONE
- COAL BED
- GEOLOGIC CONTACTS (DASHED WHERE APPROXIMATE)

NOTE:
THESE CROSS SECTIONS ARE BASED ON SUBJECTIVE INTERPRETATION OF BOREHOLE DATA AND GEOLOGIC MAPPING. THICKNESS AND ORIENTATION OF ROCK UNITS SHOULD BE CONSIDERED APPROXIMATE AND VARIABLE, PARTICULARLY WHERE PROJECTIONS ARE MADE BEYOND BOREHOLE LOCATIONS AND DEPTHS.

SCALE IN FEET
0 50'

LOT 161C–R
MOUNTAIN VILLAGE, CO
FIGURE 9–2

CROSS SECTIONS
A–A' AND B–B'

PROJECT: 181308	DATE: NOVEMBER 2006	
REV:	BY: TC	CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

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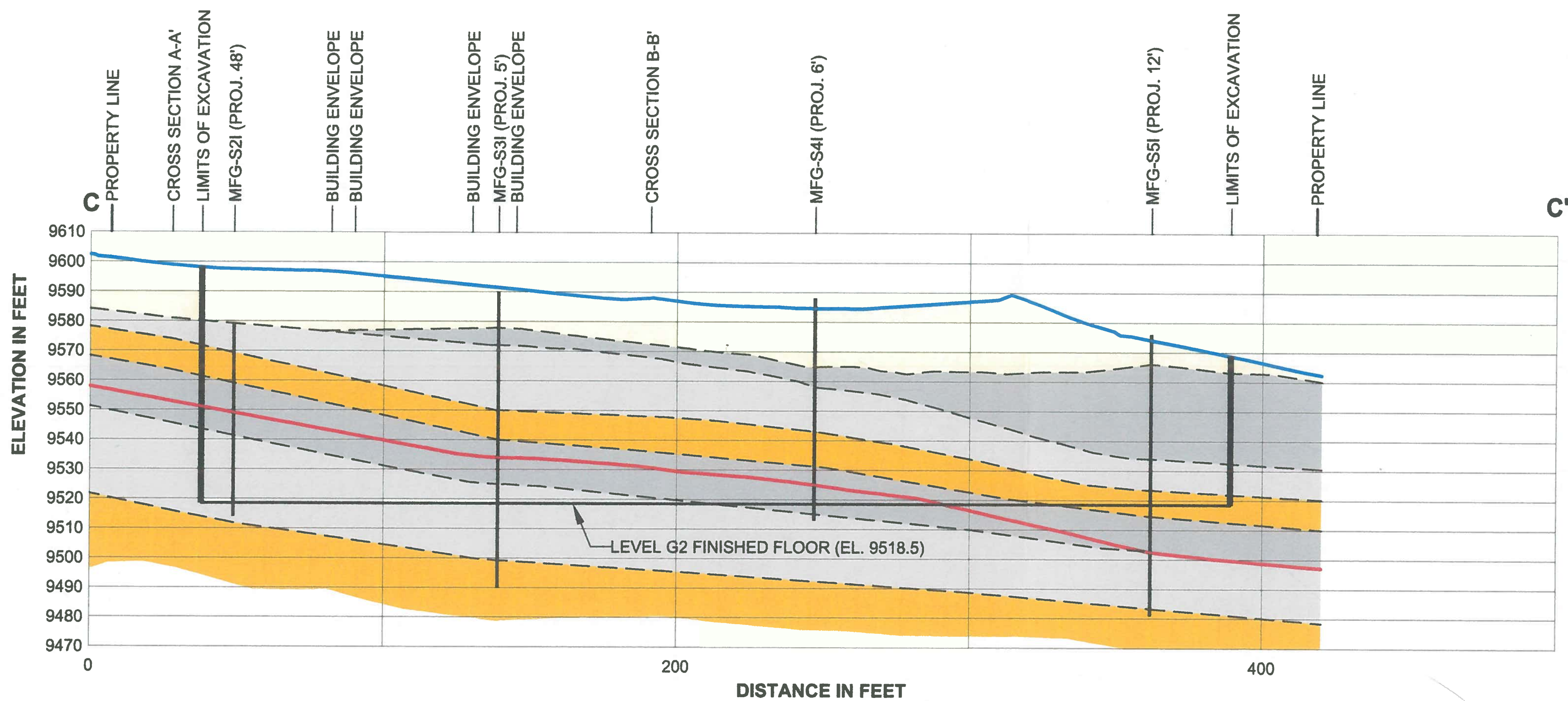


LEGEND:

- OVERBURDEN
- SHALE
- INTERBEDDED SANDSTONE AND SHALE
- SANDSTONE
- COAL BED

NOTE:
THESE CROSS SECTIONS ARE BASED ON SUBJECTIVE INTERPRETATION OF BOREHOLE DATA AND GEOLOGIC MAPPING. THICKNESS AND ORIENTATION OF ROCK UNITS SHOULD BE CONSIDERED APPROXIMATE AND VARIABLE, PARTICULARLY WHERE PROJECTIONS ARE MADE BEYOND BOREHOLE LOCATIONS AND DEPTHS.

SCALE IN FEET
0 50'

LOT 161C–R MOUNTAIN VILLAGE, CO		
FIGURE 9–3 CROSS SECTION C–C'		
PROJECT: 181308	DATE: NOVEMBER 2006	
REV:	BY: TC	CHECKED: MSA
MFG, Inc. *consulting scientists and engineers*		

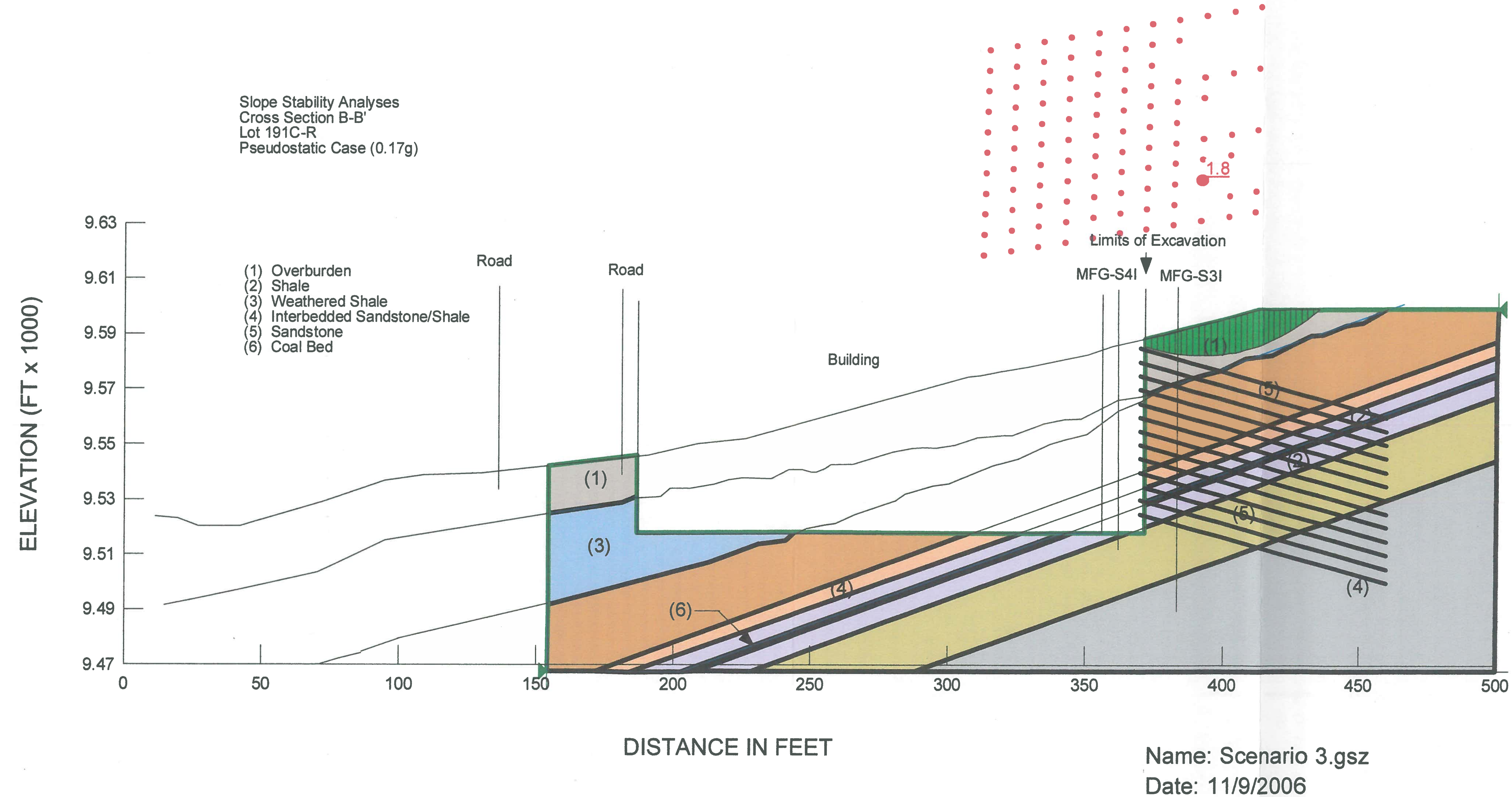
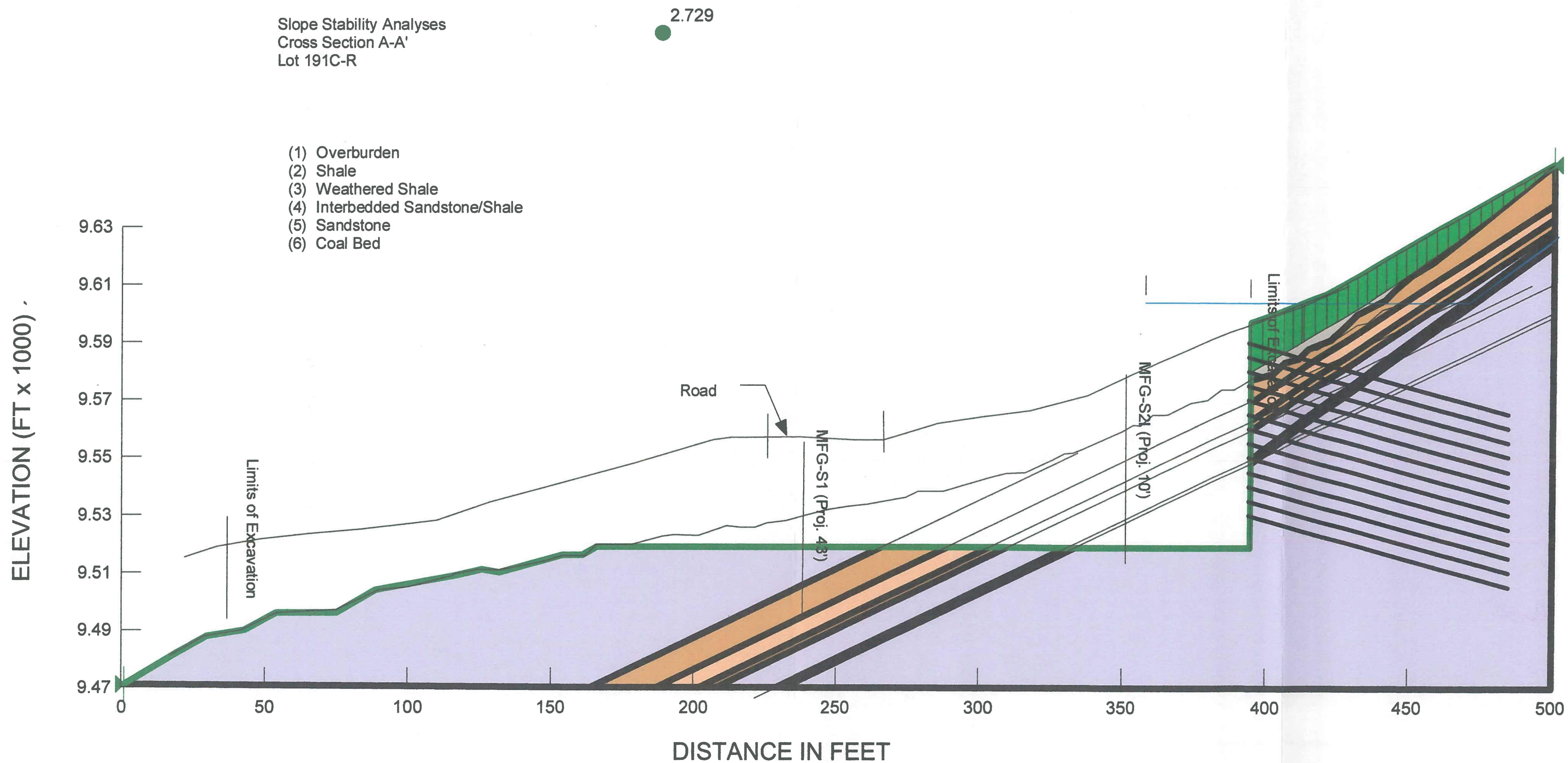


FIGURE 9-4
CROSS SECTION B-B'
SLOPE STABILITY SCENARIO 3

MFG, Inc.
consulting scientists and engineers

Date:	NOVEMBER 2006
Project:	181308
File:	SL-STA-3-7.dwg



Name: Scenario 7.gsz
Date: 11/9/2006

FIGURE 9-5
CROSS SECTION A-A'
SLOPE STABILITY SCENARIO 7

MFG, Inc.
consulting scientists and engineers

Date:	NOVEMBER 2006
Project:	181308
File:	SL-STA-3-7.dwg

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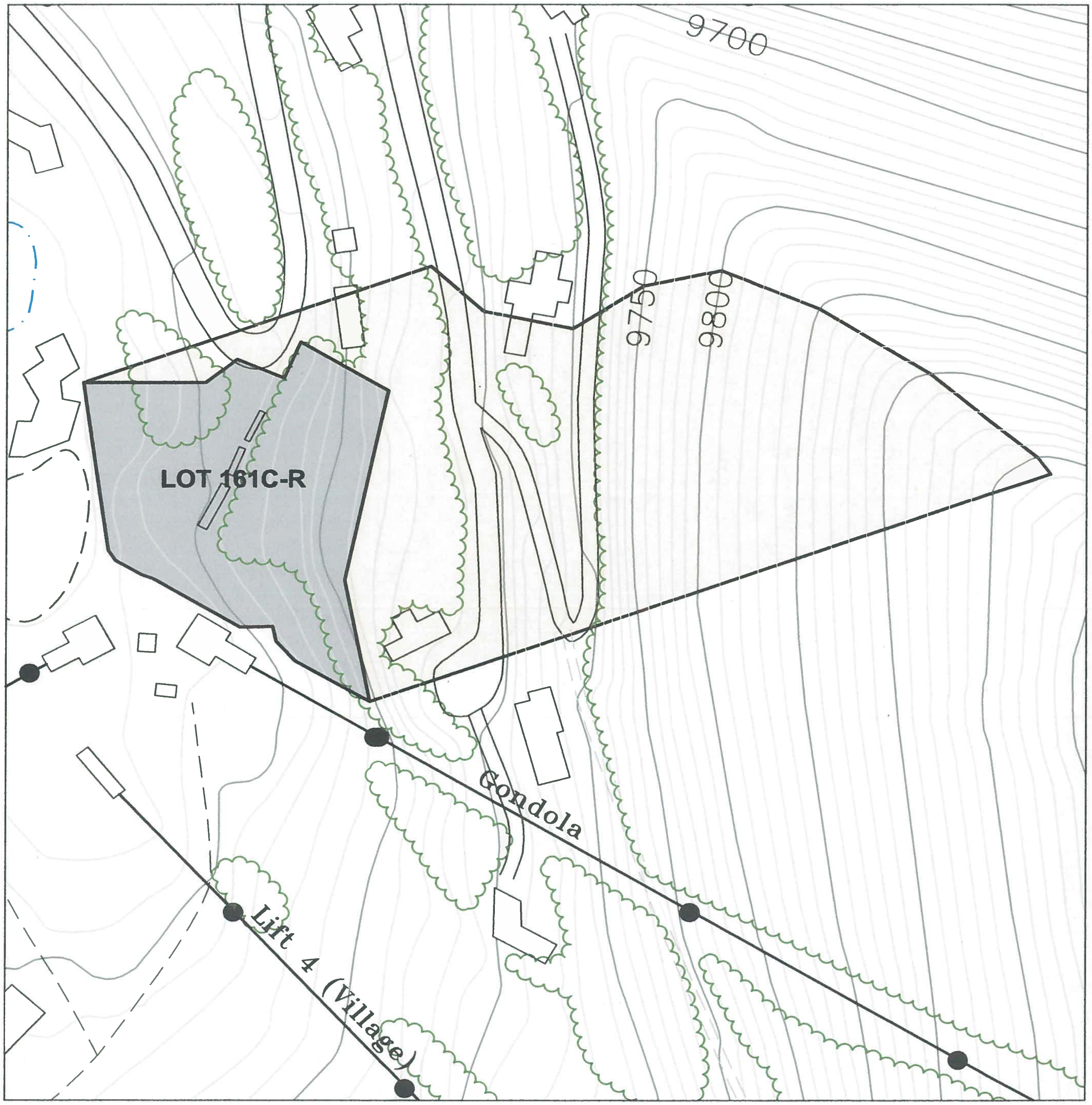

LEGEND

ESTIMATED UPGRADIENT EXTENT OF THE STORAGE BASIN (BASED ON A DIP ANGLE OF 26 DEGREES)

NOTE:
STORAGE BASIN AREA IS APPROXIMATELY 9.1 ACRES.

NORTH

SCALE IN FEET

0 200

LOT 161C–R MOUNTAIN VILLAGE, CO FIGURE 11–1	
ESTIMATED UPGRADIENT EXTENT OF CONFINED GROUNDWATER SYSTEM	
PROJECT: 181308	DATE: JANUARY 2007
REV:	BY: TGB CHECKED: MSA
MFG, Inc. *consulting scientists and engineers*	

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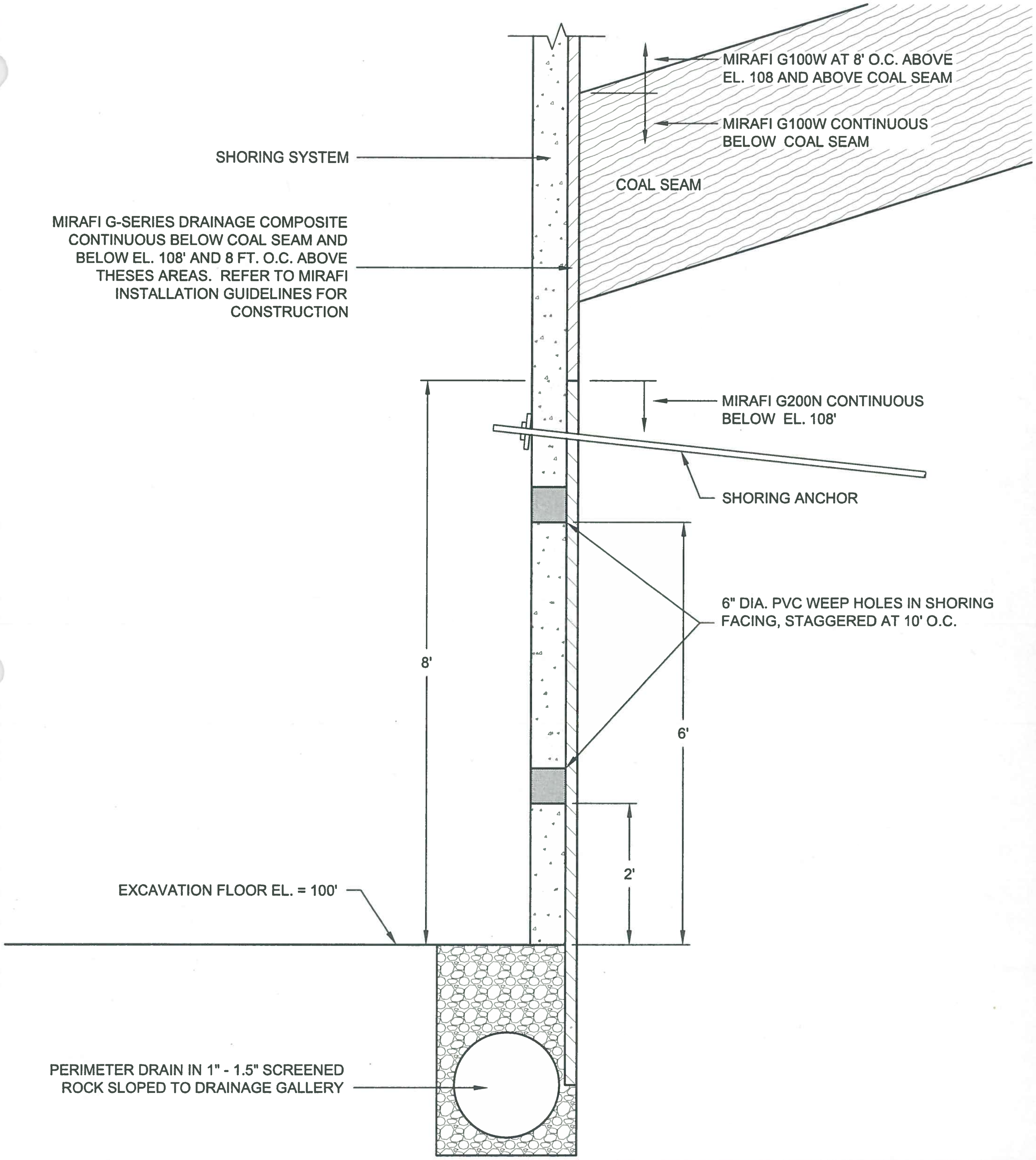

NOTE:

THE ACTUAL SHORING DRAINAGE DESIGN MUST ENSURE THAT EFFICIENT DRAINAGE BEHIND THE WALL IS MAINTAINED FOR THE SERVICE LIFE OF THE STRUCTURE. THE PERMANENT SHORING DRAINAGE SYSTEM SHOULD BE DESIGNED BY A PROFESSIONAL ENGINEER.

LOT 161C–R
MOUNTAIN VILLAGE, CO

FIGURE 11–2

CONCEPTUAL SHORING DRAINAGE DETAIL

PROJECT: 181308	DATE: NOVEMBER 2006	
REV:	BY: TGB	CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

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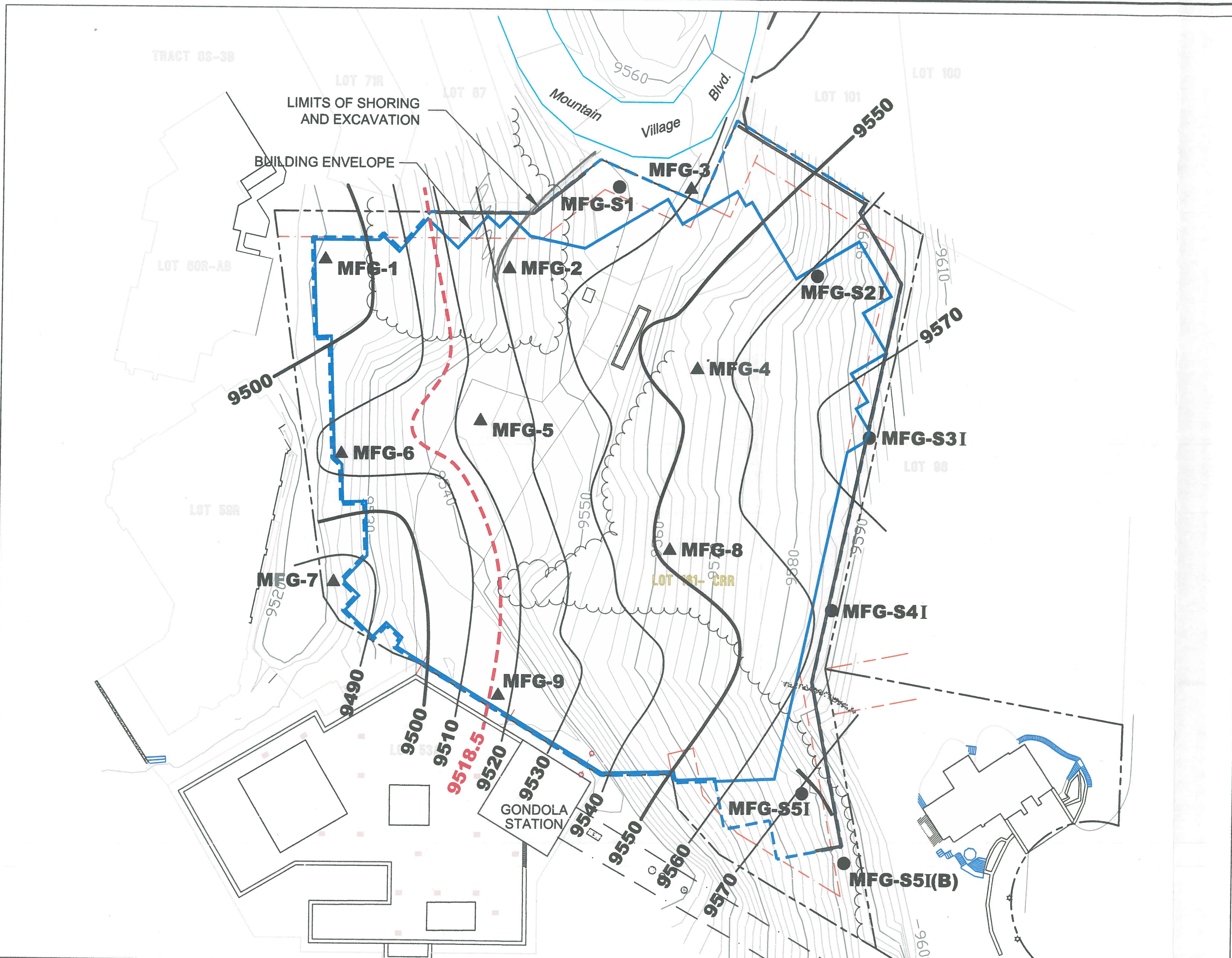

LEGEND

- ESTIMATED BEDROCK SURFACE ELEVATION CONTOURS
- APPROXIMATE LIMIT OF BEDROCK AT THE EXCAVATED FLOOR (ELEVATION 9518.5 FT.)
- SHORING INVESTIGATION BORING LOCATION
- FOUNDATION BORING LOCATION

NOTES:

1. BEDROCK ELEVATION CONTOURS WERE INTERPRETED FROM THE DEPTH TO BEDROCK IN OUR BORINGS AND TOPOGRAPHIC SURVEY PREPARED BY FOLEY ASSOCIATES, INC. (FINAL DRB EXISTING CONDITIONS, OCTOBER 3, 2006).

NORTH

SCALE IN FEET

0 60

LOT 161C–R
MOUNTAIN VILLAGE, CO

FIGURE 12–1

ESTIMATED BEDROCK SURFACE ELEVATION CONTOURS

PROJECT: 181308	DATE: JANUARY 2007	
REV:	BY: TGB	CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

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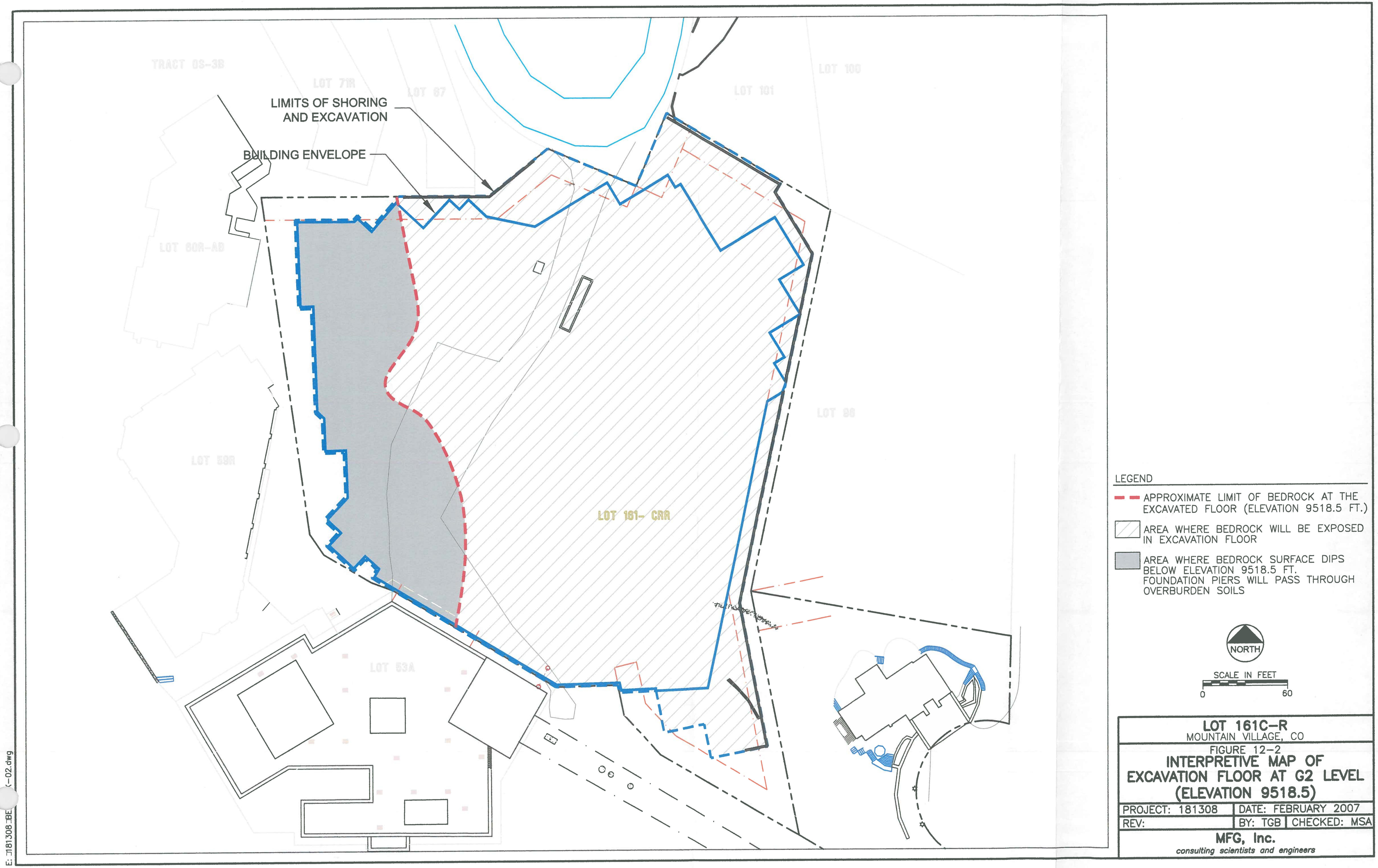
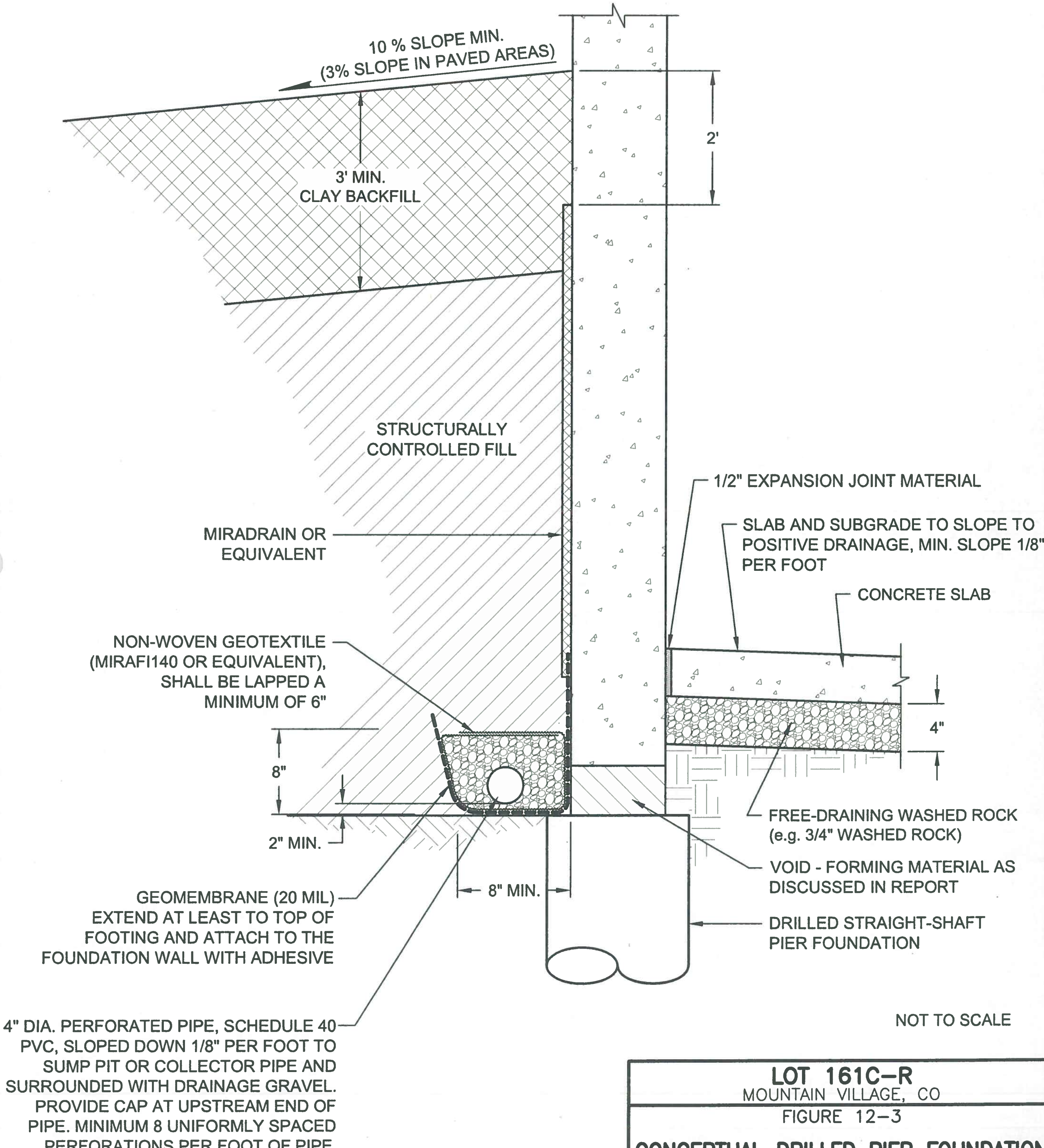


TRACT OS-3B
LOT 71R
LOT 67
LOT 101
LOT 100
LIMITS OF SHORING AND EXCAVATION
BUILDING ENVELOPE
LOT 60R-AB
LOT 59R
LOT 161- CRR
LOT 99
LOT 53A
LEGEND
APPROXIMATE LIMIT OF BEDROCK AT THE EXCAVATED FLOOR (ELEVATION 9518.5 FT.)
AREA WHERE BEDROCK WILL BE EXPOSED IN EXCAVATION FLOOR
AREA WHERE BEDROCK SURFACE DIPS BELOW ELEVATION 9518.5 FT. FOUNDATION PIERS WILL PASS THROUGH OVERBURDEN SOILS
NORTH
SCALE IN FEET
0
60
LOT 161C-R
MOUNTAIN VILLAGE, CO
FIGURE 12-2
INTERPRETIVE MAP OF EXCAVATION FLOOR AT G2 LEVEL (ELEVATION 9518.5)
PROJECT: 181308
DATE: FEBRUARY 2007
REV:
BY: TGB
CHECKED: MSA
MFG, Inc.
consulting scientists and engineers
E:\181308\BE...X-02.dwg



LOT 161C–R
MOUNTAIN VILLAGE, CO

FIGURE 12–3

CONCEPTUAL DRILLED PIER FOUNDATION EXTERIOR PERIMETER DRAIN DETAIL

PROJECT: 181308	DATE: FEBRUARY 2007	
REV:	BY: TGB	CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

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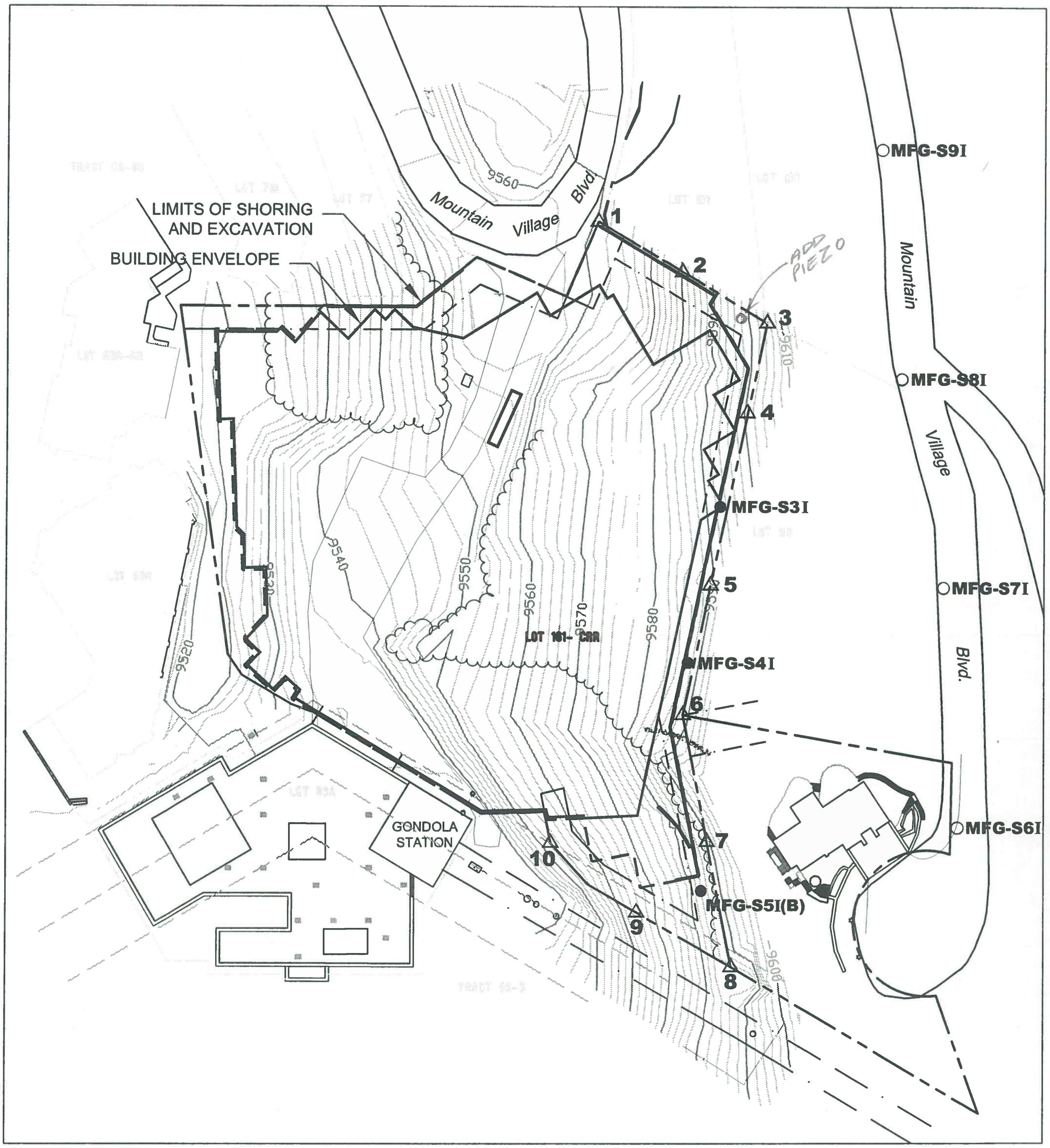

NORTH

SCALE IN FEET

0 100

LEGEND

- ● EXISTING INCLINOMETER LOCATION
- ○ PROPOSED INCLINOMETER
- △ PROPOSED SURVEY MONUMENT

SURVEY MONUMENTS

ID	EASTING	NORTHING
1	324671.5	471531.8
2	324729.3	471497.4
3	324787.2	471463.1
4	324773.9	471402.0
5	324749.2	471284.3
6	324730.4	471196.0
7	324747.1	471110.1
8	324763.8	471024.2
9	324699.0	471061.9
10	324639.9	471109.4

LOT 161C–R
MOUNTAIN VILLAGE, CO

FIGURE 14–1

SLOPE MONITORING PROGRAM SURVEY MONUMENT LOCATION

PROJECT: 181308	DATE: NOVEMBER 2006
REV:	BY: TGB / CHECKED: MSA

MFG, Inc.
consulting scientists and engineers

E: \181308\SURV—01.dwg