

SEISMIC EVALUATION OF SELECTED BUILDINGS AND STRUCTURES AT SCOTTY'S CASTLE

Death Valley National Park, Death Valley, California



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TABLE OF CONTENTS

EXECUTIVE SUMMARY..... 3

1.0 OBJECTIVE AND SCOPE..... 5

2.0 BUILDING DESCRIPTIONS 6

 2.1 DESCRIPTION OF EXISTING CONDITIONS 11

3.0 ANALYSIS AND EVALUATION OF EXISTING STRUCTURES 13

 3.1 ANALYSIS METHODOLOGY AND RESULTS 15

 3.2 SUMMARY OF ANALYSES 23

 3.3 PROPOSED RETROFIT STRATEGY..... 24

 3.4 REFERENCED DRAWINGS OF EXISTING CONDITIONS 27

4.0 REFERENCES..... 28

APPENDIX A – GUEST HOUSE EXISTING CONDITIONS..... 29

APPENDIX B – LONG SHED EXISTING CONDITIONS 31

APPENDIX C – CHIMES TOWER EXISTING CONDITIONS..... 33

Scotty's Castle Preliminary Seismic Evaluation –
 Guest House, Motel, & Chimes Tower
 Preliminary Seismic Evaluation

January 19, 2006
 Page 2 of 33

DESIMONE

EXECUTIVE SUMMARY

This report presents results of a preliminary seismic and structural evaluation of selected buildings and structures at the Scotty's Castle complex, located in Death Valley National Park in California. DeSimone Consulting Engineers, P.L.L.C., was retained by Carey & Company as part of a consultant team to prepare the Historic Structures Report for the National Park Services. This report describes our findings for the Guest House, the Long Shed and the Chimes Tower. The seismic evaluation of these buildings was performed in accordance with guidelines contained in FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings", published by the Federal Emergency Management Agency (FEMA), as well as in accordance with the 1997 Uniform Code for Building Conservation (UCBC), and the corresponding Secretary of the Interior's Standards.

Two levels of analyses were performed to evaluate the seismic performance of the structures, using: 1) FEMA 356 guidelines; and 2) UCBC 1997. Linear Static Procedures, as defined in FEMA 356, were used to perform the structural evaluation of the structures. The evaluation criteria were based upon the Basic Safety Earthquake 1 (BSE-1), which represents an earthquake having a 10% probability of exceedance in 50 years (475-year return period). The corresponding performance objective was selected to be "Collapse Prevention". A similar linear static procedure, as modified by the CBC, was used to evaluate the structures for conformance with the UCBC. The results of both analyses are presented side-by-side in this report.

The building structures were evaluated based upon the information obtained from existing drawings and information gathered during a field visit conducted by DeSimone personnel. No destructive or non-destructive testing was performed to establish properties of the existing building materials. In addition, no selective demolition or attempt to uncover hidden elements was made to establish configurations of the structural components.

Guest House

The Guest House is a one-story building with a basement. The structure is rectangular in plan, and measures roughly 86 ft x 30 ft. The gravity framing appears to be in good condition and adequate to support gravity loads. In order for the building to meet the stated seismic performance objective, we propose the following mitigation measures:

1. Increase the lateral load-resisting capacities of the existing stucco/ metal lath and plaster shear walls by either adding plywood shear panels, or replacing the hollow clay tiles by 4-in.-thick shotcrete at select locations.
2. Provide a positive roof diaphragm connection between the porch area and the east and west ends.
3. Provide positive anchorage of the existing walls to the wood roof diaphragm for resisting out-of-plane loads.
4. Repair hairline cracks in the basement walls to minimize moisture penetration into the structure.

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 3 of 33

DESIMONE

Long Shed

The Long Shed is a one-story wood building with no basement. The structure can be divided into two parts: the Garage portion (currently serves as the Gift Shop and Café) and the Motel portion (currently serving as office, storage and residential functions). Both parts are rectangular, and together they form an L-shape in plan. The Garage portion measures 97 ft x 36 ft, and the Motel portion measures approximately 15 ft x 200 ft. The gravity framing appears to be in good condition and adequate to support gravity loads. In order for the building to meet the stated seismic performance objective, we propose the following mitigation measures:

1. Increase the lateral load-resisting capacities of the existing metal lath and plaster shear walls by adding plywood shear panels at selected locations.
2. Introduce a concrete frame along grid line 1 in the Garage.
3. Provide a positive roof diaphragm connection between the porch and the main area of the Garage.
4. Provide positive anchorage of the existing walls to the wood roof diaphragm for resisting out-of-plane loads.
5. Repair holes and cracks in the exterior walls to minimize moisture penetration into the structure.

Chimes Tower

The Chimes Tower is a four-story structure of roughly 58 ft in height, with the main tower footprint measuring approximately 18 ft x 18 ft at its base. The first two stories are constructed of concrete, and the top half of the building is made of wood. We observed extensive cracking in the concrete columns at the base of the Chimes Tower, which were determined to be inadequate for supporting the gravity load. In order for the structure to meet the stated seismic performance objective, we propose the following mitigation measures:

1. Repair deteriorating concrete columns by epoxy injecting the cracks to create a sound concrete finish.
2. Strengthen walls supporting the Cupola level by adding ½-in. plywood sheathing on the inside and straps around openings on all four faces of the structure.
3. Repair loose tiles around the finial areas.
4. Repair cracks in the exterior walls to minimize moisture penetration into the structure.

1.0 OBJECTIVE AND SCOPE

The purpose of this report was to assess the seismic risks and provide preliminary recommendations for the retrofit of the Guest House, the Long Shed, and the Chimes Tower at the Scotty's Castle complex, located in Death Valley National Park in California. Observations regarding the general structural integrity are presented herein.

No destructive or non-destructive testing was performed to establish properties of the existing building materials. In addition, no selective demolition or attempt to uncover hidden elements was made to establish the configuration of structural components. A more detailed analysis than that performed herein would be required to develop a more precise characterization of the structures' expected seismic performance.

Our scope of services consisted of the following activities:

1. Review available architectural and structural drawings for the Guest House, the Long Shed, and the Chimes Tower to determine the nature of the designs and their primary structural characteristics.
2. Review available published information on the seismicity of the site.
3. Conduct a site visit to document existing conditions and prepare as-built drawings.
4. Prepare a preliminary seismic evaluation report for the structures with building descriptions, expected seismic performance under relevant standards, and proposed retrofit strategies.

The production of construction documents for strengthening recommendations is not included as a part of this scope of services, but can be provided by DeSimone as part of a supplemental contract, if desired.

2.0 BUILDING DESCRIPTIONS

Scotty's Castle is located within Death Valley National Park in California. This section describes the various structures at this complex that were evaluated for purposes of this report.

Guest House

The Guest House is a one-story wood structure with a concrete basement. The structure is rectangular in plan, with a main area of 86 ft x 30 ft and a kitchen area on the north face of 17 ft x 13 ft. The foundation system consists of a concrete slab on grade and spread footings. The foundation walls are of concrete. The ground floor is a concrete slab supported by the basement concrete walls and concrete columns with drop panels. According to the original drawings, the perimeter walls are made of wood studs with a layer of hollow clay tiles and insulex added for insulation. The interior of the walls is finished with metal lath and plaster, while the exterior is finished with stucco. The roof framing is comprised of wood rafters supported by wood trusses and straight tongue-and-groove sheathing. The roofing material is clay tiles. There are two living units on the first floor, with each unit having a loft area that is used as bedroom, that measures approximately 12 ft x 18 ft. The loft floor framing consists of wood rafters and wood sheathing with a finished wood floor. There is a porch in front of the living quarters at the south side of the building. The roof of the porch consists of wood rafters and tongue-and-groove sheathing. Wood posts support the south edge of the porch. Figure 2.1 shows the floor plan of the Guest House. Photo 2.1 shows the south elevation of this building.

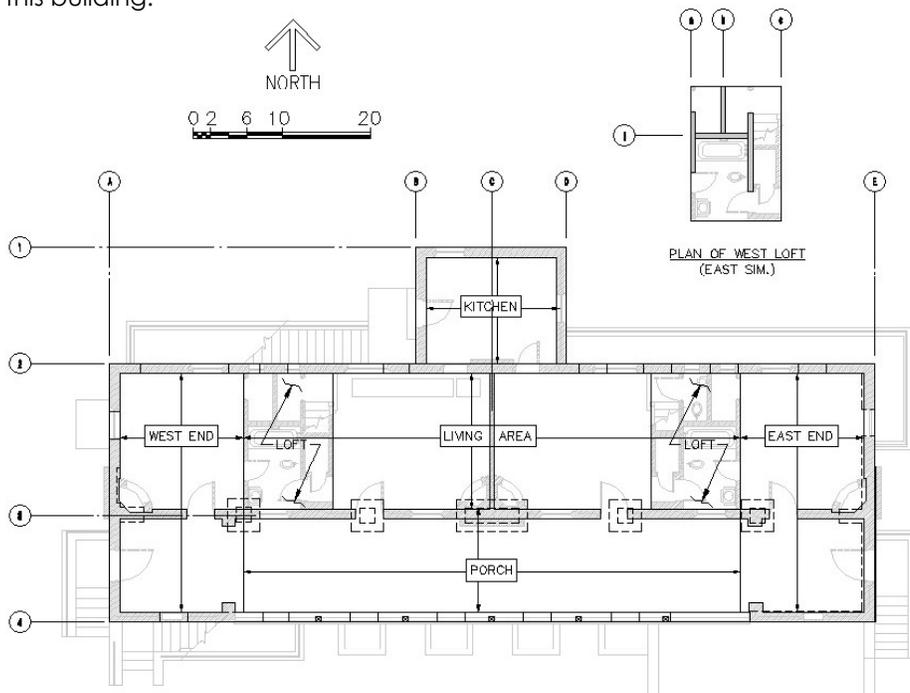


Figure 2.1: Guest House Layout Plan



Photo 2.1: South Elevation of Guest House

Long Shed

The Long Shed is a one-story wood structure with no basement. The structure can be divided into two parts: the Garage portion (currently used as a Gift Shop and Café) and the Motel portion (currently providing office, storage and residential spaces). Both portions are rectangular in plan. The Garage portion measures 97 ft x 36 ft, and there is a porch entrance area of 9 ft x 16 ft attached to the north end. The only drawing available for this structure was for the remodeling of part of the Garage (current Gift Shop). However, field observations provided different information than contained on the drawing. We believe that the foundation system is a continuous concrete footing. The perimeter and interior walls appear to be made of wood studs with plaster and metal lath on both sides. The roof framing consists of wood joists, rafters, and straight sheathing. The roofing material is clay tiles. The Motel portion measures roughly 15 ft x 200 ft. There is no existing drawing for this area. In addition, most of the structural elements are covered and their configuration could not, therefore, be established. However, by accessing the attic space above the suspended ceiling, it appeared that the framing in the office area is similar to that of the Garage. At the east end of the Motel portion, there appears to be an additional unit similar to the rest of the Motel portion. On the north face of the Motel portion, there is a canopy that is approximately 7 ft. wide. Clay tiles comprise the roofing material for the majority of the Motel portion except for the addition on the eastern end and the canopy on the north side. The roofing over these two areas is bituminous felt. Figure 2.2 shows the floor plan of the Long Shed. Photo 2.2 shows the exterior of the Garage portion (current Gift Shop) and the porch entrance area looking southeast. Photo 2.3 was taken in the courtyard area looking southwest where the Motel portion joins the Garage portion of the structure.

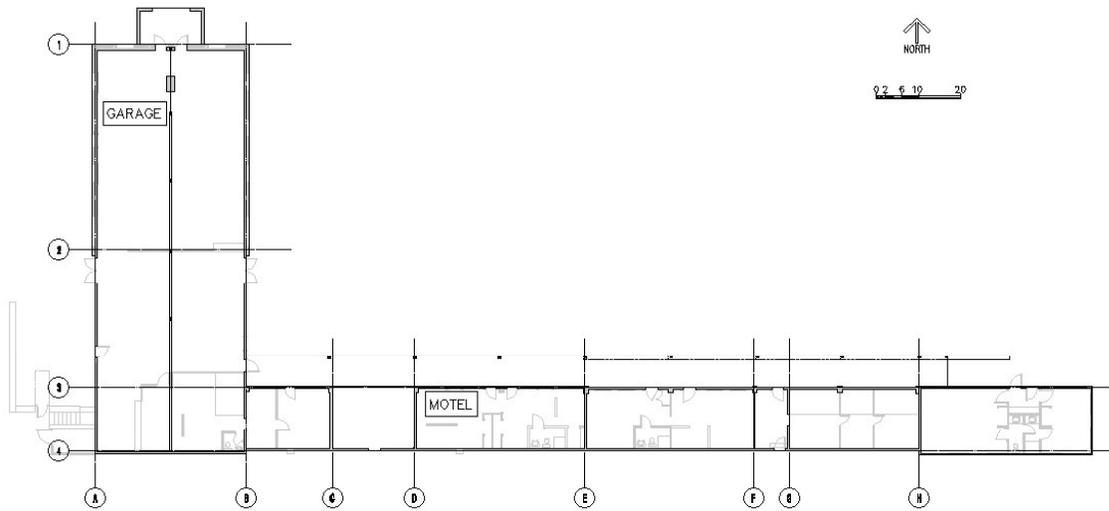


Figure 2.2: Long Shed Layout Plan



Photo 2.2: Garage and Entrance, Looking Southeast



Photo 2.3: Long Shed As Seen From Courtyard, Looking Southwest

Chimes Tower

The Chimes Tower is a four-story structure of roughly 58 ft in height, with the main tower footprint measuring approximately 18 ft x 18 ft at its base. The first two stories, namely the Mechanism floor and the Chimes Chamber floor, are constructed of concrete; the top half of the building, namely the Cupola and the roof, are made of wood. This building rests on a concrete foundation that comprises a slab-on-grade and spread footing. The concrete portion of the building has a concrete perimeter wall that tapers from a 15-inch thickness at the base to a 6-inch thickness at the top (the Chimes Chamber floor). The framing system for these two floors is comprised of a concrete slab with beams. There are two balconies and one stair landing, all of concrete construction, outside the footprint of the tower. Concrete walls extend from the base of the structure to support the north stair landing. The south balcony is supported by concrete columns and arches extending from the base of the structure. Concrete corbels extending from the tower support the west balcony. For the wood structure, the walls are built of wood studs, diagonal sheathing, and stucco/plaster on metal lath. The Cupola floor is a concrete slab supported on wood joists. Similar to the walls, the roof construction is comprised of wood rafters and diagonal lumber sheathing. The roofing material is clay tiles. Figure 2.3 shows the floor plans of the different levels of the Chimes Tower. Photo 2.4 shows the southeast elevation of the structure.

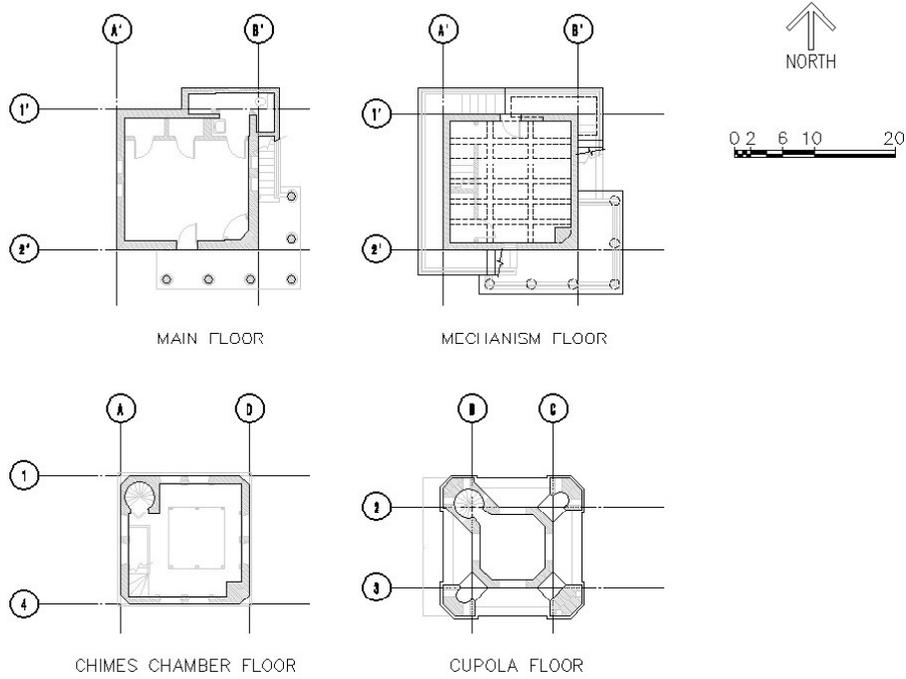


Figure 2.3: Chimes Tower Floor Layout Plan



Photo 2.4: Southeast Elevation of the Chimes Tower

2.1 Description of Existing Conditions

Based upon the visual survey conducted by DeSimone personnel, the condition of the existing structural elements is described as follows:

Guest House

1. The structure appears to be in good condition overall and very well maintained. The gravity framing of the structure, where exposed, was also found to be in good condition. Member sizes and connections of the roof trusses appear appropriate and adequate for their respective loadings.
2. The existing drawings provide adequate and accurate information for the as-built condition of the structure. The information provided in the existing drawings was confirmed during the site visit by visual inspection.
3. There was no sign of any major damage to the structure, and no sign of water seepage in the basement.
4. There are many hairline cracks in the concrete basement walls, which may suggest settlement of the structure.

Long Shed

1. The structure appears to be in good condition overall, and relatively well maintained. The gravity framing of the structure, where exposed, was found to be in good condition. Member sizes and connections appear appropriate and adequate for their respective loadings.
2. Existing drawings were available for the Garage portion of the building. However, our visual survey revealed that these drawings did not accurately reflect the as-built information. No drawings were available for the Motel portion of the building.
3. There was no sign of any major damage to the structure.
4. There are many holes and cracks in the exterior walls.

Chimes Tower

1. The concrete columns of the structure are deteriorating and have extensive cracks. The columns appeared to be not finished and the concrete was of poor quality.
2. There are many cracks on the exterior of the structure.
3. Tiles on the finial areas are loose and present a life-safety hazard to passers-by and visitors to this building.
4. Member sizes and connections appear appropriate and adequate for their respective loadings.

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

DESIMONE

January 19, 2006
Page 11 of 33

5. Existing drawings were available for most of the structure. The existing drawings provide adequate and accurate information for the as-built condition of the structure. The information provided in the existing drawings was confirmed during the site visit by visual inspection.
6. Aside from the specific deteriorations mentioned above, there was no sign of major damage of the structure.

Photographs documenting existing condition of the structures can be found in the corresponding appendices at the end of this report.

3.0 ANALYSIS AND EVALUATION OF EXISTING STRUCTURES

Linear static procedures were used to perform analyses and evaluations of the Guest House, the Long Shed and the Chimes Tower. The structures were modeled based upon the information obtained from the existing drawings and during the site visit by DeSimone personnel. No destructive or non-destructive testing was performed to establish properties of the existing building materials. No selective demolition or attempt to uncover hidden elements was made to establish configuration of structural components.

The analyses were performed based upon two independent criteria:

1. Federal Emergency Management Agency, Prestandard and Commentary for the Seismic Rehabilitation of Buildings, ASCE/FEMA 356, November 2000.
2. International Code of Building Officials, Uniform Code for Building Conservation (UCBC), 1997, and the Secretary of the Interior's Standards.

FEMA 356

The FEMA 356 guidelines are based upon "expected strength" level design associated with spectral-level seismic acceleration. The demand-capacity ratio of each structural component is computed and compared with an expected ductility factor m . The expected ductility factor m is defined in the FEMA 356 guidelines and is based upon both component testing as well as historical data. The response spectra that are generated are based upon national seismic hazard maps prepared by the USGS, using the specific location of the structure given by either zip code or longitude and latitude.

The selected FEMA 356 evaluation criteria was based upon the Basic Safety Earthquake 1 (BSE-1) that represents a 10% probability of exceedance in 50 years (475-year return period). The corresponding performance objective was selected to be "Collapse Prevention". This criterion is similar to the State Historic Building Code and is in accordance with the Secretary of Interior's Standards of Historic Preservation. Response spectra were computed based upon FEMA 356 guidelines. Two response spectra were obtained, one for the BSE-1 (a 10% probability of exceedance in 50 years event), also called the Design Basis Earthquake (DBE); and one for the BSE-2 (a 2% probability of exceedance in 50 years event), also called the Maximum Credible Earthquake (MCE). Figure 3.1 shows the generalized site response spectra for the BSE-1 and BSE-2 seismic events.

UCBC

The UCBC requirements, as modified by the CBC, are based upon "allowable strength design" and reduced response spectra. The response spectra are reduced by an overall reduction factor (R) to account for structure's inelastic behavior and expected global ductility.

The results of both analyses are presented side-by-side for comparison. The recommendations are based upon a combination of the results from both analyses, the Secretary of the Interior's Standards, our engineering judgment, and our knowledge as to the seismic performance of similar structures.

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

DESIMONE

January 19, 2006
Page 13 of 33

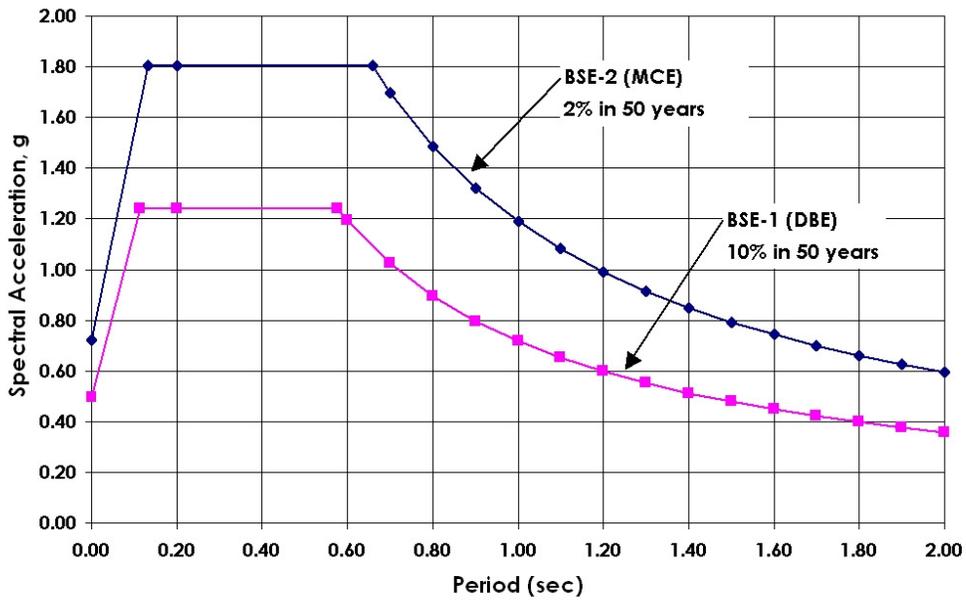


Figure 3.1: Generalized Site Response Spectra

3.1 Analysis Methodology and Results

Linear static procedures were used for the analyses. All interior and exterior building walls that qualify to act as shear walls (based upon their aspect ratios according to FEMA 356) were considered to resist the lateral earthquake forces.

The structural periods, total building weights, and base shears for each of the structures considered are presented in Table 3.1 below.

Table 3.1: Summary of Structures' Dynamic Characteristics

		Guest House	Long Shed	Chimes Tower	
				Wood	Concrete
Weight (lbs)		199,700	320,900	55,500	239,400
Period (sec)	FEMA 356	0.15	0.43	0.28	0.19
	UCBC	0.15	0.15	0.28	0.19
Base Shear	FEMA 356	1.80W	1.43W	1.62W	1.75W
	UCBC	0.28W	0.28W	0.28W	0.28W

The demand forces in the shear walls and roof/ floor diaphragms were computed and compared with corresponding allowable values according to the FEMA 356 and UCBC guidelines. The knowledge factor, κ , as defined in FEMA 356, was taken to be 1.0 based upon the quality of the information available for the analyses, coupled with the application of engineering judgment from the extensive experience DeSimone has as to the seismic performance of similar types of construction.

The evaluation is based upon the resultant Demand-Capacity Ratio (DCR). For the FEMA 356 evaluation, the element capacity is multiplied by the corresponding m factor to obtain a DCR. A DCR of less than 1.0 indicates that the element capacity is larger than the demand, and therefore, the element strength is adequate. A DCR larger than 1.0 indicates that the element does not have adequate strength to resist the demand loads, and therefore requires strengthening.

Guest House

Figure 3.2 shows the Guest House floor plan with grid lines for easy reference. Note that there is no lateral force-resisting element for the porch roof diaphragm in the north-south direction. The force is thus considered to be resisted by the walls in the east and west ends of the building, which would require connection of the roofs over these areas. In keeping with the spirit of the Secretary of Interior's Standards and to keep the changes to the historic structure to a minimum, we are adopting the results of UCBC for our retrofit recommendations.

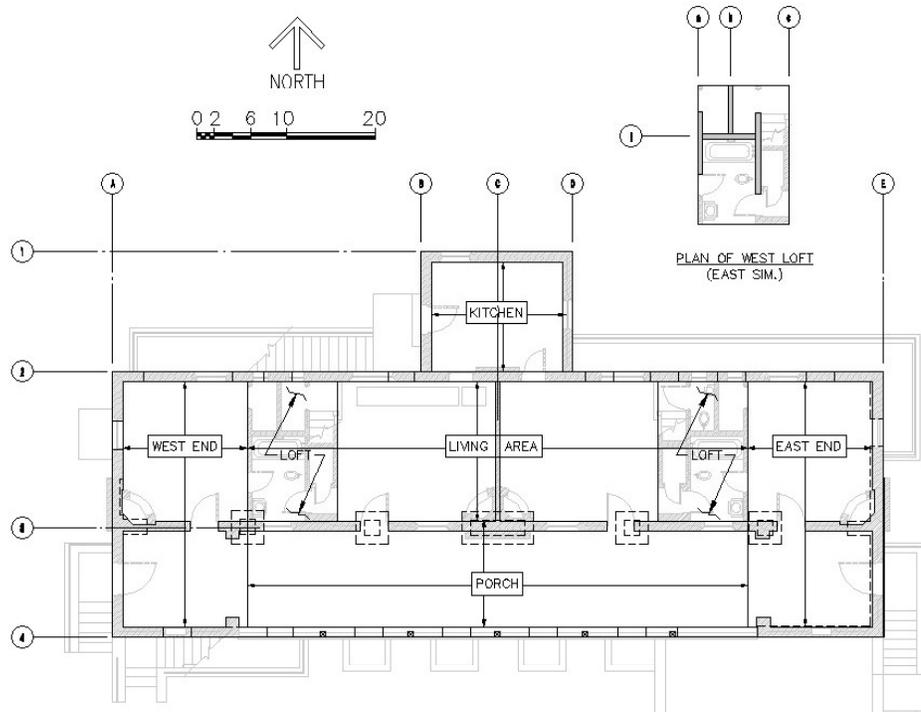


Figure 3.2: Guest House Floor Layout Plan

The seismic evaluation summary for the shear walls is presented in Table 3.2. A discussion of the results is as follows:

UCBC

The table shows that for the UCBC analysis, the shear walls along grid 3 (between the Living Area and the Porch), grid line A & E (along the west and east faces of the building), and grid C (in the Living Area) do not have adequate lateral load resisting capacity, with DCRs ranging from 1.44 to 2.07. All other walls are adequate for seismic loadings, with DCRs ranging from 0.24 to 0.79. The partition walls are adequate for supporting the loft areas with a low DCR of 0.24 in the east-west direction and 0.08 in the north-south direction.

FEMA 356

The evaluation criteria for FEMA 356 are more stringent and hence more walls are found to have inadequate lateral load-carrying capacity. All main walls, except those along grid 4 in the east and west ends of the building, have DCRs much greater than 1.0, with values ranging from 1.55 to 6.39, and hence require strengthening. However, as in the UCBC analysis, the partition walls were found to be adequate for supporting the loft areas, with a relatively low DCR of 0.67 in the east-west direction and 0.23 in the north-south direction.

Table 3.2: Guest House Shear Wall Seismic Evaluation Summary

Area	Grid Line	UCBC				FEMA 356, BSE-1, CP			
		Demand p/f	Capacity p/f	DCR	Comments	Demand p/f	Capacity p/f	DCR	Comments
East-West Direction									
West/East End	2	189	400	0.47	OK	1,735	1,120	1.55	NG
	3	315	400	0.79	OK	2,891	1,120	2.58	NG
	4	95	400	0.24	OK	867	875	0.99	OK
Kitchen	1	271	400	0.68	OK	2,489	1,225	2.03	NG
Living Area	2	252	400	0.63	OK	2,310	1,190	1.94	NG
Liv. A & Porch	3 long	609	400	1.52	NG	5,589	1,085	5.15	NG
	3 short	-	-	-	-	5,589	875	6.39	NG
Loft		97	400	0.24	OK	889	1,320	0.67	OK
North-South Direction									
West/East End & Porch	A/E short	576	400	1.44	NG	5,286	875	6.04	NG
	A/E long	-	-	-	-	5,286	1,260	4.20	NG
Kitchen	B/D	249	400	0.62	OK	2,282	875	2.61	NG
Living Area	C	829	400	2.07	NG	7,609	1,320	5.76	NG
Loft	a	30	400	0.08	OK	278	1,320	0.21	OK
	b	33	400	0.08	OK	306	1,320	0.23	OK
	c	31	400	0.08	OK	288	1,320	0.22	OK

Table 3.3 presents a summary of the roof diaphragm seismic evaluation. A discussion of the results is as follows:

UCBC Based on the UCBC criteria, the diaphragm between grids 3 and 4 in the east and west ends of the building is the only roof diaphragm having an adequate lateral load carrying capacity, with a DCR of 0.81. All other roof diaphragms have DCRs greater than 1.0, with values ranging from 1.15 to 3.92. The east-west direction controls for the loft floor diaphragm and it is adequate with a low DCR of 0.35.

FEMA 356 Based on the more stringent FEMA 356 criteria, all roof diaphragms were found to be inadequate for this building, with DCRs ranging from 2.48 to as high as 11.98. The heavy weight of the hollow clay tiles, coupled with the perimeter walls having minimal shear resisting capacity, is a very important factor in contributing to the poor seismic performance of this building.

Table 3.3: Guest House Diaphragm Seismic Evaluation Summary

Area	Grid Line	UCBC				FEMA 356, BSE-1, CP			
		Demand plf	Capacity plf	DCR	Comments	Demand plf	Capacity plf	DCR	Comments
East-West Direction									
West/East End	2-3	122	100	1.22	NG	1,115	300	3.72	NG
	3-4	81	100	0.81	OK	743	300	2.48	NG
Kitchen	1-2	176	100	1.76	NG	1,611	300	5.37	NG
Living Area	2-3	122	100	1.22	NG	1,115	300	3.72	NG
Porch	3-4	162	100	1.62	NG	1,487	300	4.96	NG
Loff		35	100	0.35	OK	324	300	1.08	say OK
North-South Direction									
West/East End	A/E	346	100	3.46	NG	3,172	300	10.57	NG
Kitchen	B/D	115	100	1.15	NG	1,053	300	3.51	NG
Living Area	C	392	100	3.92	NG	3,593	300	11.98	NG
Porch		392	100	3.92	NG	3,593	300	11.98	NG

Long Shed

Figure 3.3 shows the Long Shed floor plan with grid lines for easy reference. The porch area is only framed by wood posts and has no lateral force-resisting capacity in either direction. Along grid line 1, the wall has many large openings and thus cannot be considered effective to resist seismic forces. For similar reasons, the wall on the east face of the Motel is also considered ineffective. Along grid line 2, as can be seen in photo B.8 in Appendix B, one side of the plaster of the partition wall is discontinuous above the ceiling, and hence for analysis purposes only one side is considered effective. Given the simplicity of this building, it is our recommendation to adopt the UCBC criteria for this building over the more stringent requirements of FEMA 356. The results of the FEMA 356 analysis are provided here for comparison purposes only.

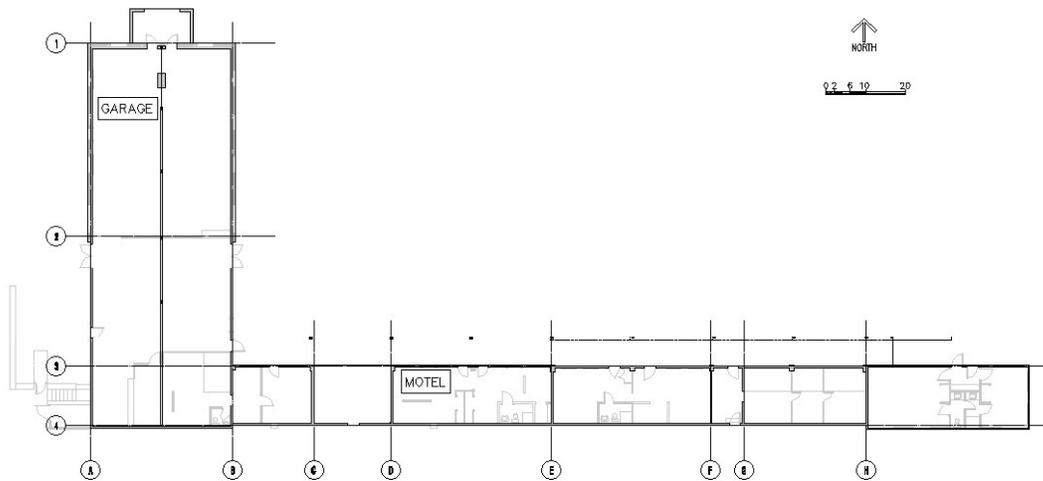


Figure 3.3: Long Shed Floor Layout Plan

The seismic evaluation summary for the Long Shed shear walls is presented in Table 3.4. A discussion of the results is as follows:

UCBC The table shows that for the UCBC analysis, the shear walls along lines E and H for the Long Shed in the north-south direction, and along lines 2 and 4 for the Garage in the east-west direction, do not have adequate lateral-load resisting capacity, with DCRs ranging from 1.08 to 5.56. All other walls are adequate for seismic loadings, with DCRs ranging from 0.20 to 0.95.

FEMA 356 As with the analysis for the Guest House, under the FEMA 356 criteria several walls were found to have inadequate lateral load carrying capacity. All walls in the north-south direction, as well as walls along grid lines 2 and 4 for the Garage, and along grid line 3 for the Long Shed in the east-west direction, have DCRs well above 1.0, with typical values ranging from 1.14 to 2.90. The highest DCR value was found to be 12.30 for the wall along grid line 2 for the Garage. Under FEMA 356 guidelines, all of these walls will require strengthening.

Table 3.4: Long Shed Shear Wall Seismic Evaluation Summary

Area	Grid Line	UCBC				FEMA 356, BSE-1, CP			
		Demand plf	Capacity plf	DCR	Comments	Demand plf	Capacity plf	DCR	Comments
North-South Direction									
Garage	A	298	400	0.74	OK	2,172	1,320	1.65	NG
	B	334	400	0.83	OK	2,435	1,320	1.84	NG
Motel	C	252	400	0.63	OK	1,840	1,320	1.39	NG
	D	378	400	0.95	OK	2,760	1,320	2.09	NG
	E	504	400	1.26	NG	3,680	1,320	2.79	NG
	F	303	400	0.76	OK	2,208	1,320	1.67	NG
	G	287	400	0.72	OK	2,093	1,320	1.59	NG
Motel End	H	525	400	1.31	NG	3,828	1,320	2.90	NG
East-West Direction									
Garage	2	1,112	200	5.56	NG	8,115	660	12.30	NG
	4	433	400	1.08	NG	3,162	1,320	2.40	NG
Motel	3	206	400	0.51	OK	1,500	1,320	1.14	NG
	4	82	400	0.20	OK	597	1,320	0.45	OK
Motel End	3	126	400	0.32	OK	920	1,320	0.70	OK
	4	114	400	0.29	OK	835	1,320	0.63	OK

Table 3.5 presents a summary of the roof diaphragm evaluation. A discussion of the results is as follows:

UCBC Based on the UCBC criteria, the diaphragms between grid lines 1-2, 2-4, and A-B for the Garage, D-E, E-F, G-H, and east of grid line H for the Long Shed, do not have adequate load carrying capacity. Typical overstress DCR values range from 1.24 to 2.72, with the highest value being 4.24 for the Garage diaphragm between grid lines 1-2 in the east-west direction.

FEMA 356 Based on FEMA 356 criteria, however, all diaphragms, with a minor exception of an 8 ft. x 15 ft. area between grid lines F-G, are inadequate. However, inadequacy of the diaphragm strength at such DCR values corresponds to an exceedance of only the serviceability criteria. Yielding of diaphragms in light frame wall buildings does not present a collapse hazard, as inherently, such buildings are able to deform substantially and therefore absorb energy.

Table 3.5: Long Shed Diaphragm Evaluation Summary

Area	Grid Line	UCBC				FEMA 356, BSE-1, CP			
		Demand plf	Capacity plf	DCR	Comments	Demand plf	Capacity plf	DCR	Comments
North-South Direction									
Garage	A-B	150	100	1.50	NG	1,096	300	3.65	NG
Motel	B-C	80	100	0.80	OK	585	300	1.95	NG
	C-D	80	100	0.80	OK	585	300	1.95	NG
	D-E	160	100	1.60	NG	1,170	300	3.90	NG
	E-F	160	100	1.60	NG	1,170	300	3.90	NG
	F-G	32	100	0.32	OK	234	300	0.78	OK
	G-H	124	100	1.24	NG	907	300	3.02	NG
	H-H'	245	100	2.45	NG	1,793	300	5.98	NG
Motel End	H'	272	100	2.72	NG	1,989	300	6.63	NG
East-West Direction									
Porch	1	72	100	0.72	OK	527	300	1.76	NG
Garage	1-2	424	100	4.24	NG	3,101	300	10.34	NG
	2-4	192	100	1.92	NG	1,404	300	4.68	NG
Motel	3	56	100	0.56	OK	410	300	1.37	NG
	3-4	60	100	0.60	OK	439	300	1.46	NG
Motel End	3	56	100	0.56	OK	410	300	1.37	NG
	3-4	64	100	0.64	OK	468	300	1.56	NG

Chimes Tower

Figure 3.4 shows the different level floor plans for the Chimes Tower for easy reference. The Chimes Tower has three stories and is comprised of a mix of construction materials. The Main Floor and the Mechanism Floor are constructed of concrete, whereas the levels above, namely the Chimes Chamber Floor and the Cupola Floor, are constructed of wood.

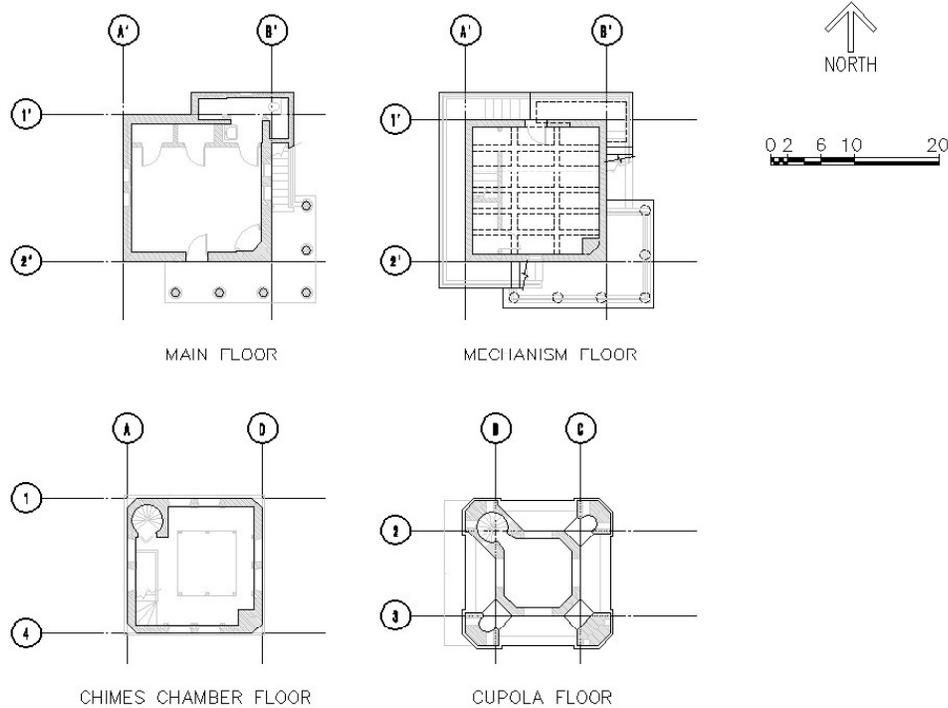


Figure 3.4: Chimes Tower Floor Layout Plan

The seismic evaluation summary for the Chimes Tower shear walls is presented in Table 3.6. The walls supporting the Cupola Floor have big openings on all four sides of the structure, and are considered not effective in resisting lateral forces due to their high aspect ratios.

For this particular structure, guidelines from UCBC and FEMA 356 give comparable results; hence discussion is included herein based on numeric values from the FEMA 356 analysis. For the walls that qualify as shear walls, all but two walls have capacities greater than the shear demand, with DCRs ranging from 0.47 to 0.83 per FEMA 356 criteria. The walls supporting the roof along grid lines 2 and B are the only walls that have inadequate capacities, with a DCR of 1.42 per FEMA 356 criteria. However, inadequacy of strength in walls comprised of wood studs and diagonal lumber sheathing at such DCR levels corresponds only to an exceedance of the serviceability criteria. Excessive deformation is expected to occur, but it would not induce collapse in the structure. It is our conclusion that these walls are adequate for the Collapse Prevention performance level, as discussed earlier in this section.

Table 3.6: Chimes Tower Shear Wall Seismic Evaluation Summary

Area	Grid Line	UCBC				FEMA 356, BSE-1, CP			
		Demand Capacity		DCR	Comments	Demand Capacity		DCR	Comments
		plf	plf			plf	plf		
Roof	2, B	373	250	1.49	NG	3,083	2,170	1.42	NG
	3, C	187	250	0.75	OK	1,541	2,170	0.71	OK
Chimes Chambe	1', 2'	1,611	6,831	0.24	OK	10,992	20,493	0.54	OK
	A', B'	1,410	6,831	0.21	OK	9,618	20,493	0.47	OK
Mechanism Floor	1'	4,262	11,384	0.37	OK	28,270	34,152	0.83	OK
	2'	2,740	11,384	0.24	OK	18,174	34,152	0.53	OK
	A', B'	3,196	11,384	0.28	OK	21,203	34,152	0.62	OK

The stair landing and balconies extending from the main Chimes Tower footprint were evaluated separately for their lateral load-carrying capacities. The north stair landing area, supported by concrete walls, was found to be adequate, with a DCR of 0.09 in the east-west direction and 0.07 in the north-south direction per FEMA 356. The south balcony is supported by columns along the south and east faces. The shear capacities of the columns, after repairing, were found to be adequate for the stated level of performance, with a DCR of 0.46 in the east-west direction and 0.61 in the north-south direction per FEMA 356.

Table 3.7 presents a summary of the roof/ floor diaphragm evaluation. The roof diaphragm comprises diagonal lumber sheathing and the floor diaphragms comprise 3-in.-thick concrete slabs. All diaphragms were found to be adequate based upon both FEMA 356 and UCBC criteria, and hence no strengthening is required.

Table 3.7: Chimes Tower Diaphragm Seismic Evaluation Summary

Area	UCBC				FEMA 356, BSE-1, CP			
	Demand Capacity		DCR	Comments	Demand Capacity		DCR	Comments
	plf	plf			plf	plf		
Roof	145	250	0.58	OK	1,201	1,500	0.80	OK
Cupola	674	3,415	0.20	OK	5,578	10,245	0.54	OK
Chimes Chambe	1,422	3,415	0.42	OK	9,421	10,245	0.92	OK
Mechanism Floor	1,491	3,415	0.44	OK	9,896	10,245	0.97	OK

3.2 Summary of Analyses

Based upon the analysis results presented in this section, certain elements in the Guest House, the Long Shed and the Chimes Tower do not have sufficient strength to conform to the stated performance objective. The performance objective was discussed earlier in this section. The following is a summary of the findings.

Guest House & Long Shed

1. Existing wood stud and stucco/ plaster on metal lath walls do not have adequate shear capacity to resist lateral loads.
2. Existing wood roof diaphragms are found to have inadequate shear capacity to resist the lateral loads.
3. The wall-to-roof diaphragm anchorage for in-plane forces does not appear to be adequate.

Chimes Tower

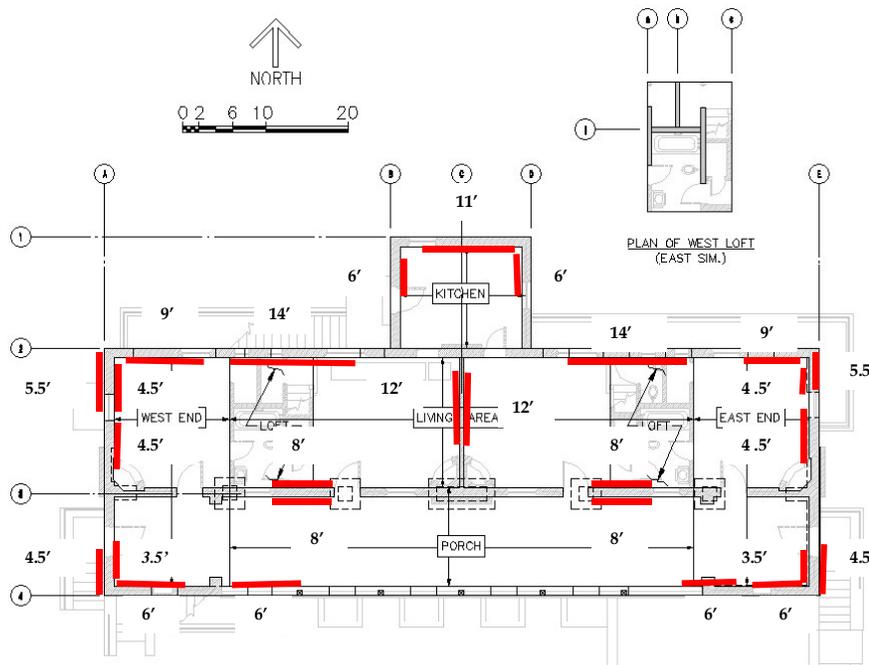
1. All walls that qualify as shear walls have adequate lateral load carrying capacities. However, the walls supporting the Cupola level do not qualify as shear walls and require strengthening.
2. All diaphragms are found to be adequate.

3.3 Proposed Retrofit Strategy

To mitigate issues stated above and to improve the building performance in order to meet the stated performance objective, the following strengthening measures are proposed. Figure 3.4 through 3.7 present the conceptual retrofit strategy.

Guest House

1. Increase the lateral load-resisting capacities of the existing stucco/ metal lath and plaster shear walls by either adding plywood shear panels (Figure 3.4), or replacing the hollow clay tile walls by 4-in.-thick shotcrete at select locations (Figure 3.5).
2. Provide roof diaphragm connection between the porch area and the east and west ends.
3. Strengthen existing wood roof diaphragms by adding plywood sheathing to the underside of the roof.
4. Provide positive anchorage of existing walls to the wood roof diaphragm for out-of-plane loads.
5. Repair hairline cracks in the basement walls to minimize moisture penetration into the structure.



**Figure 3.4: Guest House – Conceptual Seismic Rehabilitation Recommendation
Addition of Plywood Sheathing**

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

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January 19, 2006
Page 24 of 33

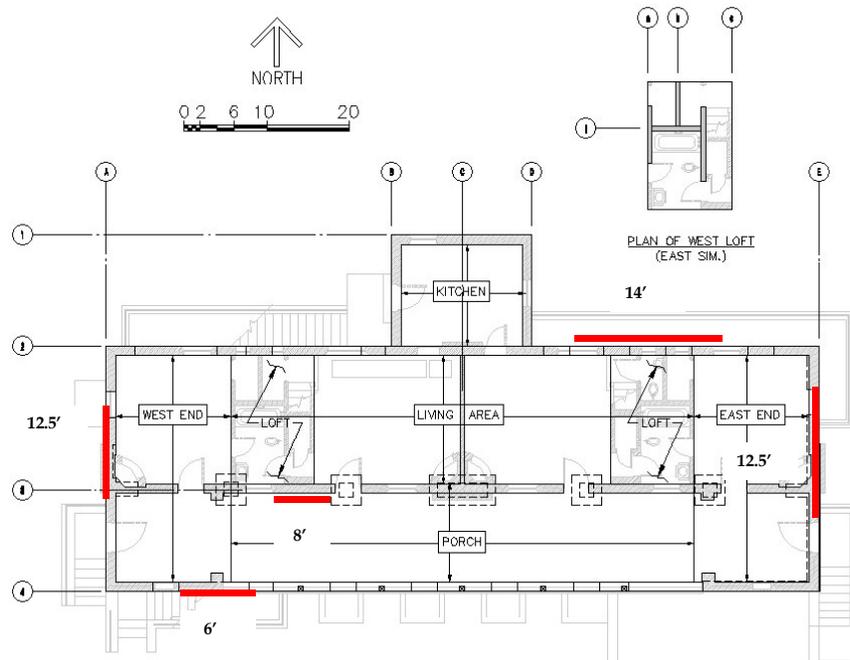


Figure 3.5: Guest House – Conceptual Seismic Rehabilitation Recommendation
Replacement of Hollow Clay Tile Walls with Shotcrete Walls

Long Shed

1. Increase the lateral load-resisting capacities of the existing metal lath and plaster shear walls by adding plywood shear panels at select locations (Figure 3.6).
2. Introduce a concrete frame along grid line 1 in the Garage.
3. Provide a positive roof diaphragm connection between the porch and the main area of the Garage.
4. Provide positive anchorage of existing walls to the wood roof diaphragm for out-of-plane loads.
5. Repair holes and cracks in the plaster walls to minimize moisture penetration into the structure.

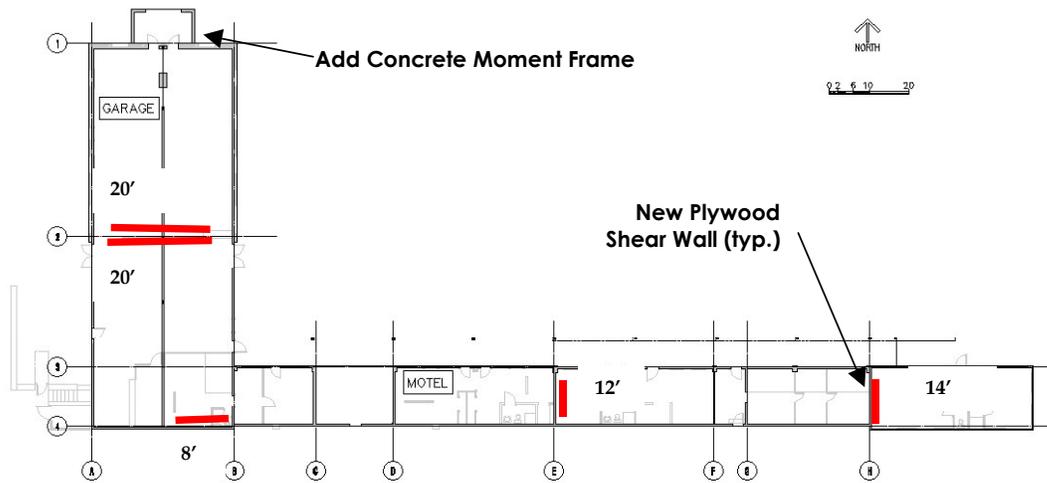


Figure 3.6: Long Shed – Conceptual Seismic Rehabilitation Recommendation for Shear Walls

Chimes Tower

1. Repair deteriorating columns by epoxy injecting the cracks to create a sound concrete finish.
2. Strengthen walls supporting the Cupola level by adding ½-in. plywood sheathing on the inside face of the walls and adding straps around openings to stiffen the wall elements.
3. Repair loose tiles on the finial areas.
4. Repair cracks on the exterior walls to minimize moisture penetration into the structure.

3.4 Referenced Drawings of Existing Conditions

The following drawings are referenced as a part of this report and were used to document the existing condition of the structures.

Guest House

- S7.1: Basement/ Foundation Plan of Existing Condition
- S7.2: First Floor Plan of Existing Condition
- S7.3: Roof Framing Plan of Existing Condition
- S7.4: Elevations and Details of Existing Condition

Long Shed

- S9.1: Roof Framing Plan of Existing Condition
- S9.2: Roof Framing Plan of Existing Condition
- S9.3: Roof Framing Plan of Existing Condition
- S9.4: Elevations and Details of Existing Condition

Chimes Tower

- S4.1: First and Second Floor Plan of Existing Condition
- S4.2: Third and Fourth Floor Plan of Existing Condition
- S4.3: Elevations and Details of Existing Condition

4.0 REFERENCES

1. Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, ASCE/FEMA 356, November 2000.
2. International Code of Building Officials, *Uniform Code for Building Conservation (UCBC)*, 1997.
3. International Code of Building Officials, *California Building Code (CBC)*, 2001.
4. International Code of Building Officials, *Uniform Building Code (UBC)*, 1997.
5. Original building drawings for the Guest House, dated October 25, 1927.
6. Original building drawings for the Garage remodel, not dated.
7. Original building drawings for the Chimes Tower, dated October 30, 1928.
8. Carey & Co., Measured Drawings.

APPENDIX A – GUEST HOUSE EXISTING CONDITIONS



Photo A.1: West Elevation of Guest House



Photo A.2: Kitchen Area of Guest House



Photo A.3: Porch Area of Guest House



Photo A.4: Post Detail at Porch, Guest House

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 29 of 33

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APPENDIX A – GUEST HOUSE EXISTING CONDITIONS (CONT'D.)



Photo A.5: Loft Area of Guest House



Photo A.6: Roof Truss Connection, Guest House



Photo A.7: Drop Panel, Guest House



Photo A.8: Cracks in Basement Wall, Guest House

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 30 of 33

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APPENDIX B – LONG SHED EXISTING CONDITIONS



Photo B.1: East Elevation of Motel



Photo B.2: South Face of Motel



Photo B.3: North Face of Motel (1 of 2)



Photo B.4: North Face of Motel (2 of 2)

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 31 of 33

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APPENDIX B – LONG SHED EXISTING CONDITIONS (CONT'D.)



Photo B.5: Interior of Chicken Coop, Motel



Photo B.6: Corridor at Motel



Photo B.7: Roof Framing, Garage



Photo B.8: Partition Wall, Garage

Scotty's Castle Preliminary Seismic Evaluation –
Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 32 of 33

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APPENDIX C – CHIMES TOWER EXISTING CONDITIONS



Photo C.1: Second Floor Framing, Chimes Tower



Photo C.2: Loose Tile at Finial, Chimes Tower



Photo C.3: Cracks in Columns at Chimes Tower



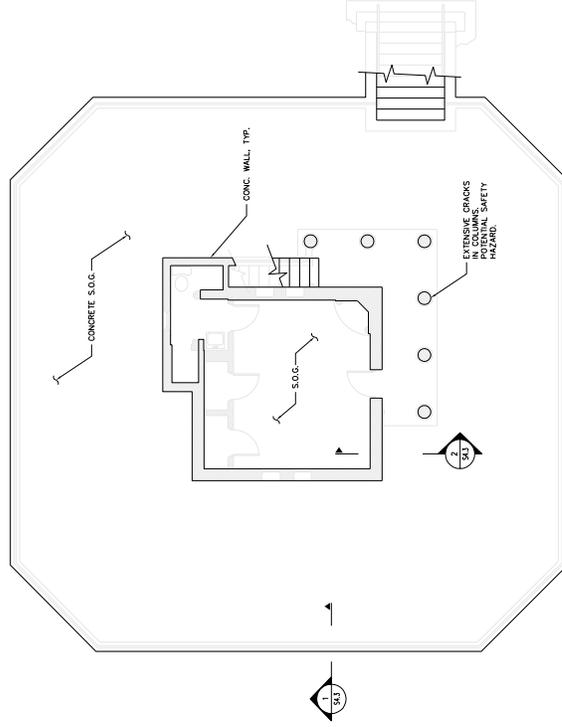
Photo C.4: Cracks in Column at Chimes Tower

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Guest House, Motel, & Chimes Tower
Preliminary Seismic Evaluation

January 19, 2006
Page 33 of 33

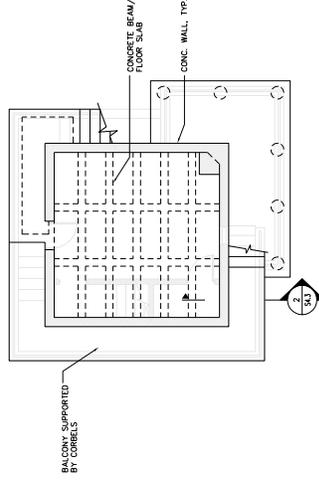
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143-41031 SHEET 128 OF 159	10/20/1928
143-41033A SHEET 3 OF 6	06/15/1931



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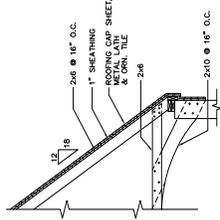
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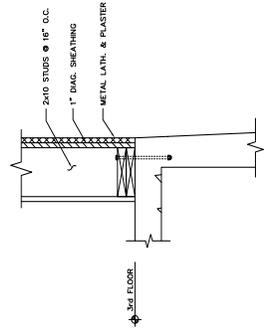


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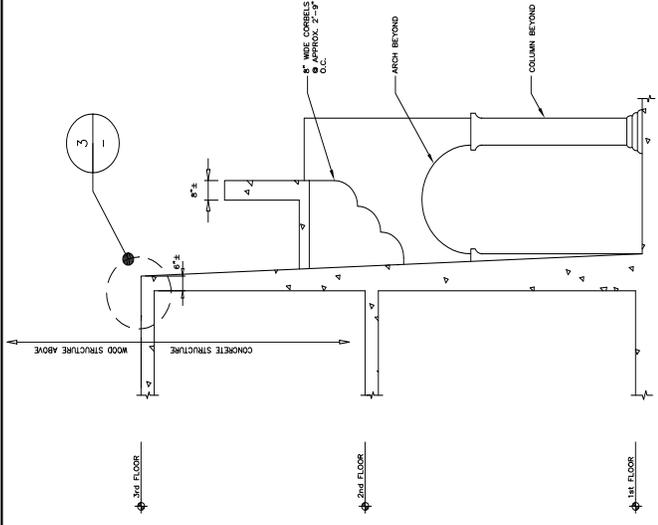
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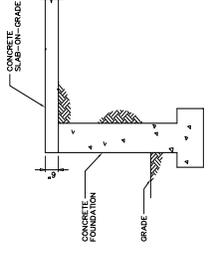
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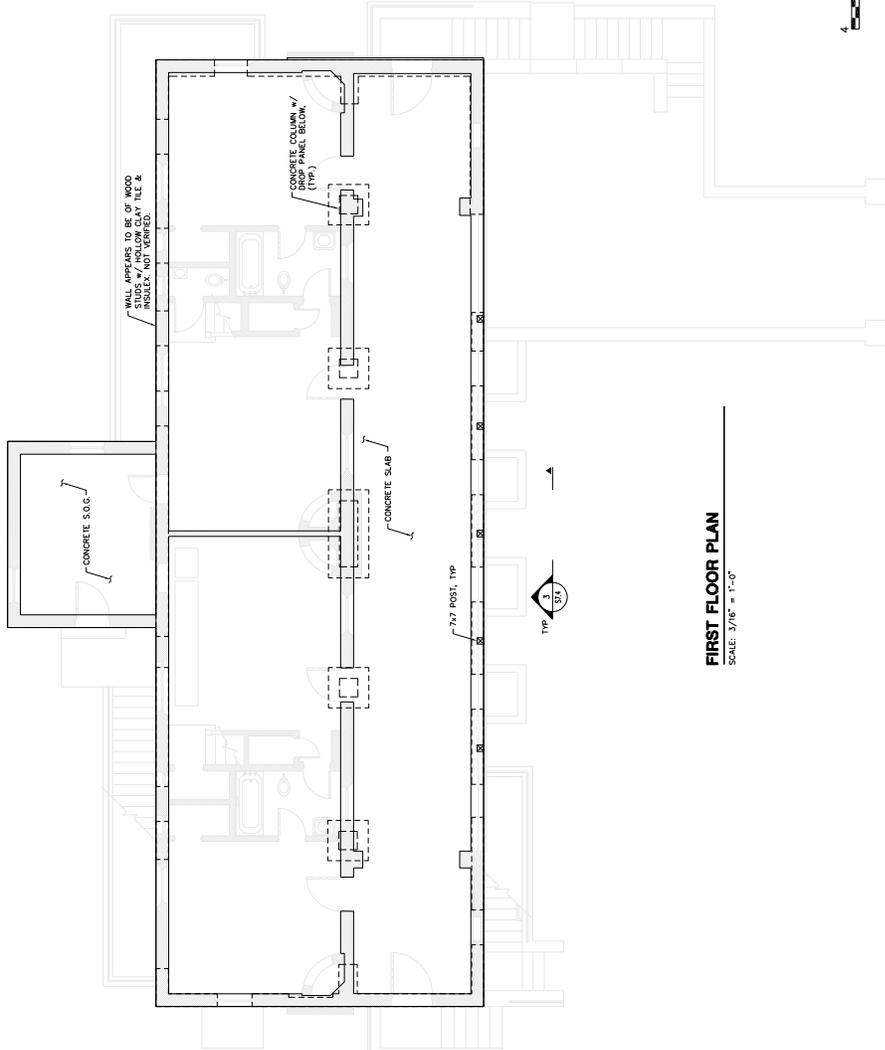
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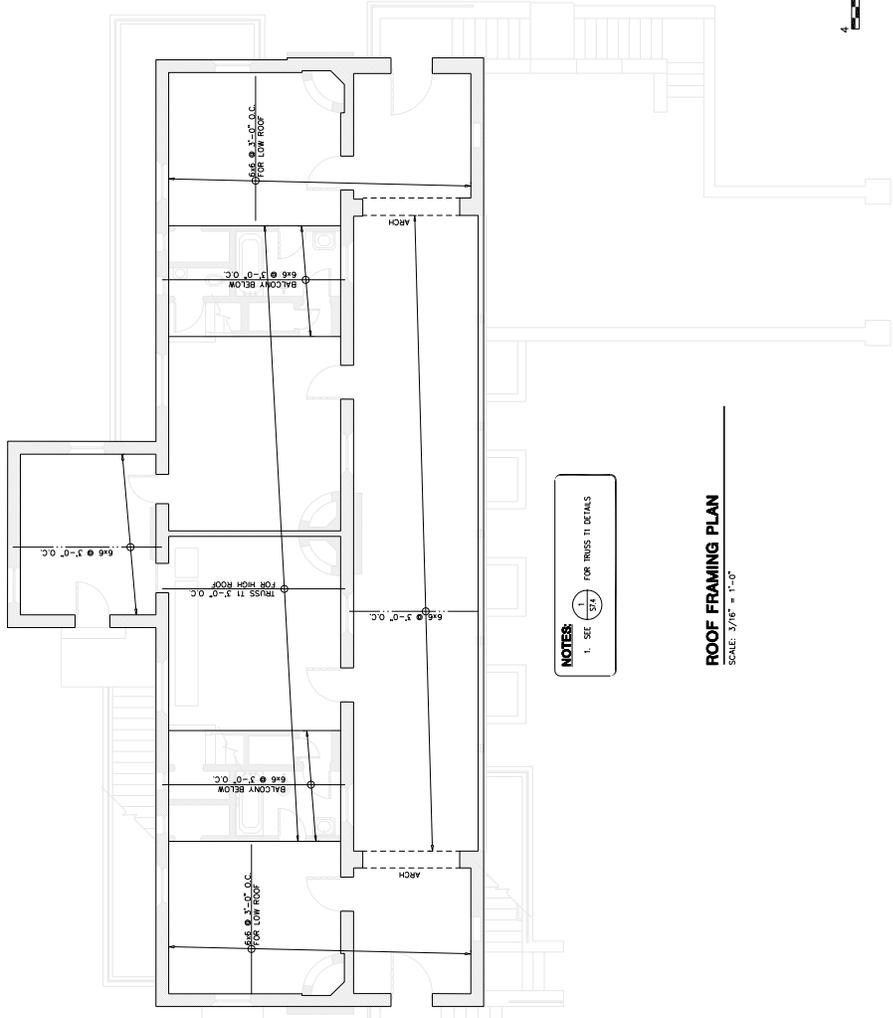


FIRST FLOOR PLAN
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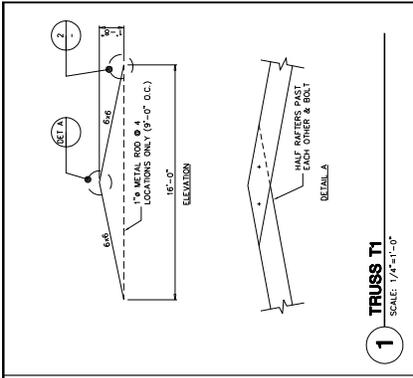
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ROOF FRAMING PLAN
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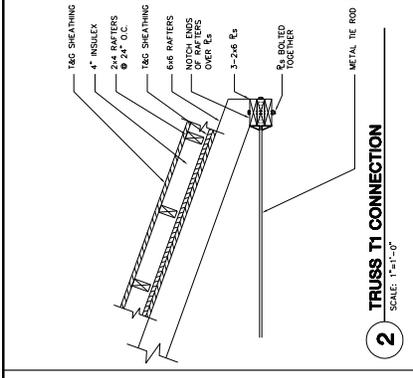


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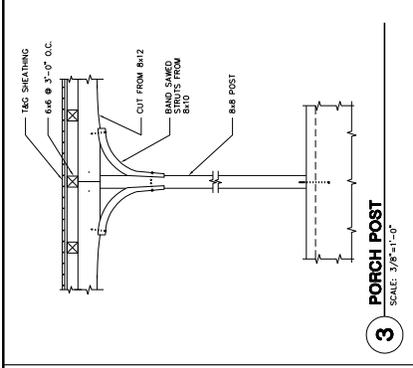
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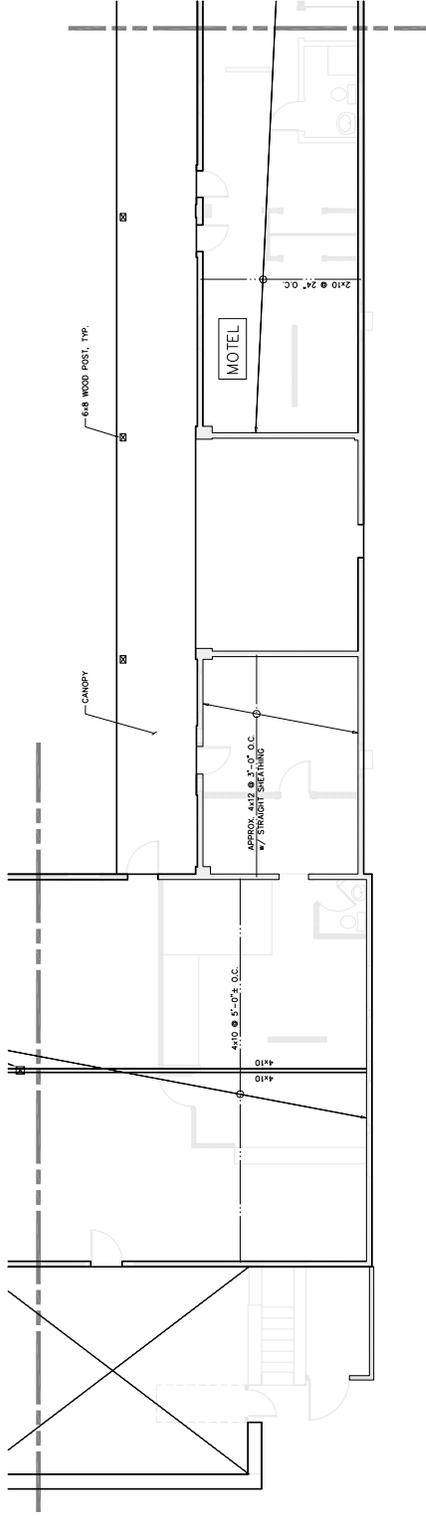
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3 PORCH POST
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GUEST HOUSE DETAILS		GUEST HOUSE DETAILS	
EXISTING CONDITION		EXISTING CONDITION	
DEATH VALLEY		DEATH VALLEY	
NATIONAL PARK		NATIONAL PARK	



ROOF FRAMING PLAN

SCALE: 3/16" = 1'-0"



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DRAWING NO. 332

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 NATIONAL PARK

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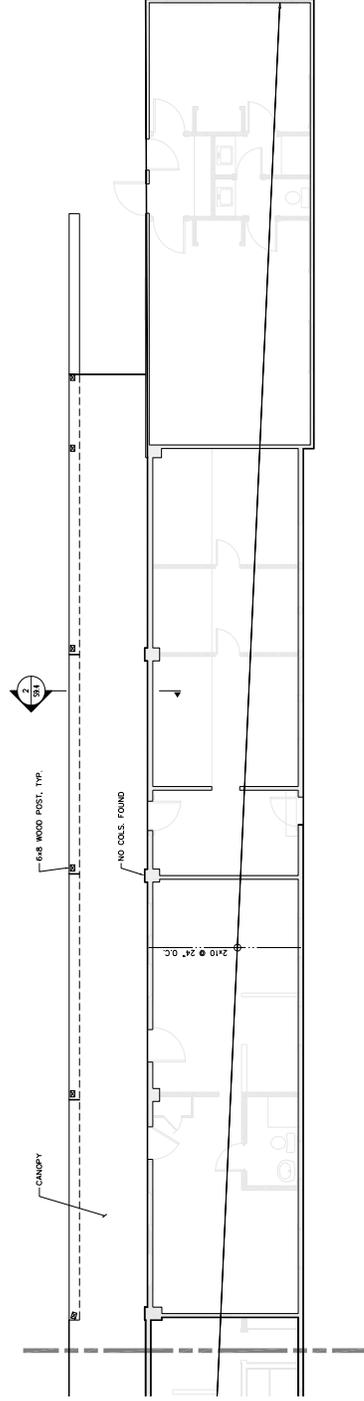
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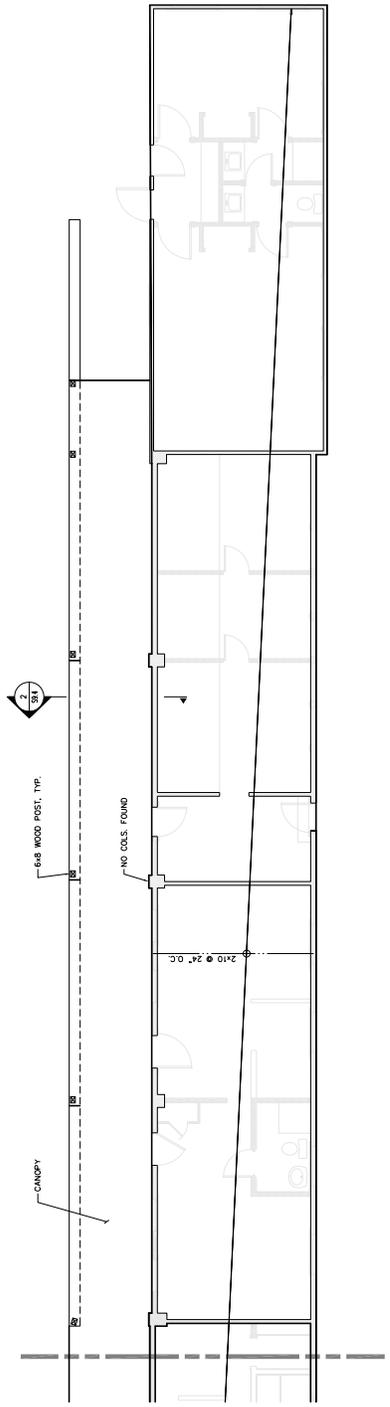


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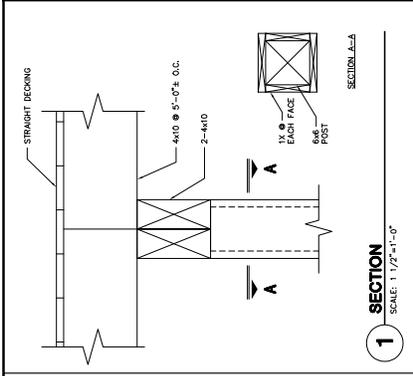
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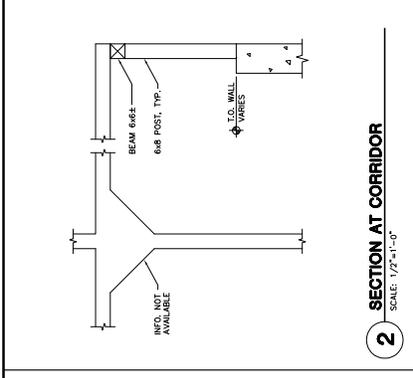


ROOF FRAMING PLAN
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2 SECTION AT CORRIDOR
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DEATH VALLEY	
NATIONAL PARK	

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1.0 Executive Summary

This report presents the results of the preliminary seismic and structural evaluation of the Stable building at the Scotty's Castle located at the Death Valley National Park in California. DeSimone Consulting Engineers, P.L.L.C., was retained by Carey & Company as part of a consultant team to prepare the Historic Structures Report for the National Park Services. DeSimone's scope of work comprises evaluation of the Stables building, the Guest House, the Long Shed/ Motel and the Chimes Tower. This report describes our findings for the Stables building only. A separate report will be issued for the Guest House, the Long Shed/ Motel and the Chimes Tower. The seismic evaluation was performed in accordance with the guidelines contained in FEMA 356, "Prestandard and Commentary for the Seismic Rehabilitation of Buildings" published by the Federal Emergency Management Agency (FEMA), as well as in accordance with the Uniform Code for Building Conservation (UCBC). The results of both analyses are presented in this report.

The Stable building consists of three attached structures: a stable, a shed, and a covered driveway that connects the two structures. The original stable was built in 1920 as a stand-alone structure and was extensively remodeled in 1928. In addition, in 1928 the other two structures were added, the shed and the covered driveway. All three structures are one story tall. The perimeter and interior walls are made of wood studs, metal lath and plaster except the shed building has concrete perimeter walls. The roof framing comprises wood trusses with tile roofs. The foundation system comprises strip and spread footings.

Two levels of analyses were performed to evaluate the structures: 1) FEMA 356 guidelines; and 2) UCBC 1997. Linear Static Procedures as defined in FEMA 356 were used to perform the structural evaluation of the structures. The evaluation criteria were based upon the Basic Safety Earthquake 1 (BSE-1), which represents an earthquake having a 10% probability of exceedance in 50 years (475 years return period). The corresponding performance objective was selected to be "Collapse Prevention". A similar linear static procedure, as modified by the CBC, was used to evaluate the structures for conformance with the UCBC. The UCBC analyses performed are for comparison purposes only and the final strengthening recommendations are based upon FEMA 356.

The building structures were evaluated based upon the information obtained from the existing drawings and information gathered during a field visit conducted by DeSimone personnel. No destructive or non-destructive testing was performed to establish properties of the existing building materials. No selective demolition or attempt to uncover hidden elements was made to establish configuration of structural components.

Based upon the observation and analyses, the gravity framing in the structures appear to be in good conditions and adequate to support gravity loads. However, the building structures do not have adequate lateral load resisting capacity. To mitigate these issues, and to improve the building's performance to meet the stated performance objective, following mitigation measures are proposed.

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 2 of 16

1. To increase the lateral load resisting capacity of the structure, strengthen existing metal lath and plaster shear walls by adding plywood shear panel at selected locations.
2. Strengthen existing wood roof diaphragm for the shed and the covered driveway structures by adding plywood sheathing at the underside of the truss.
3. Provide positive anchorage of concrete walls to the wood roof diaphragm for the shed building for out-of-plane loads.
4. Repair hairline plaster cracks in the perimeter walls to minimize moisture penetration into the structure.

2.0 Building Description

Scotty's Castle is located at the Death Valley National Park, California. The Stable building consists of three attached structures: a stable, a shed, and a covered driveway that connects the two structures. The original stable was built in 1920 as a stand-alone structure. In 1928, extensive remodeling of the stable took place and new perimeter walls as well as new roof structures were constructed. In addition, two new buildings were added namely, the shed and the covered driveway. All three structures are one story tall. Figure 2.1 shows the floor plan of the building.

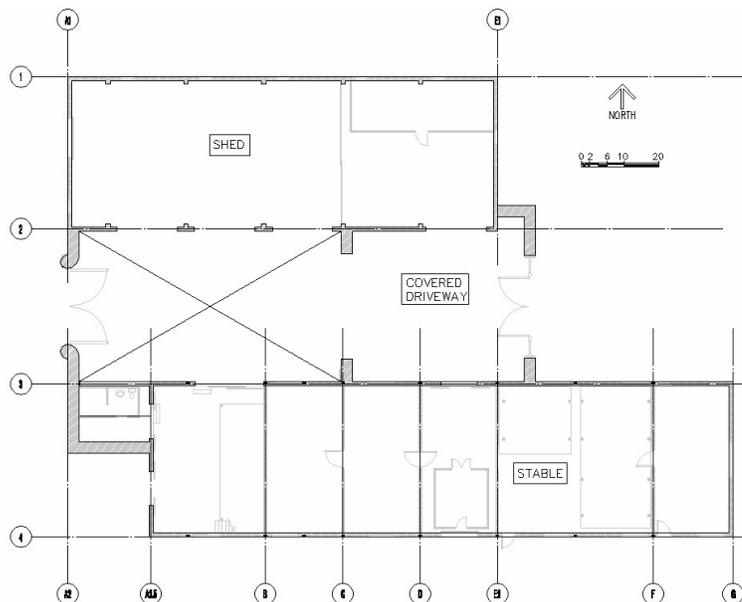


Figure 2.1: Floor Layout Plan

The stable is a one-story wood structure that is rectangular in plan, measures 135 feet x 36 feet. The foundation system consists of concrete continuous and spread footings. The perimeter walls are made of wood studs with metal lath and plaster. The roof framing comprises wood trusses with metal tension rods, rafters, and diagonal sheathing. Interior partitions appear to be of the same construction as the perimeter walls. The bathroom and the storage area to the west of the stable appear to be an addition after the major remodeling of the stable, and were not documented on the drawings from 1928. This area measures 17 feet x 19 feet, and appears to have wood stud walls similar to the rest of the stable. The roof framing comprises wood rafters and straight sheathing.

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 4 of 16

The shed, similar to the stable, is also a one-story structure rectangular in plan, and it measures 100 feet x 36 feet. The perimeter walls are concrete, with large arches on the south wall that opens to the courtyard. The roof framing is also similar to that of the stable. Unlike the stable, there are no interior partitions in the shed.

The covered driveway connects the stable and the shed. The arches, found on the east and west faces, are framed from studs with metal lath and plaster. The roof framing comprises of very simple trusses with wood and metal rod, ornamental diagonal blocking, and straight sheathing.

Photo 2.1 shows the west elevation of the stable building, and photo 2.2 was taken in the open courtyard showing relative locations of the structures.



Photo 2.1: West Elevation of Stable Building

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 5 of 16

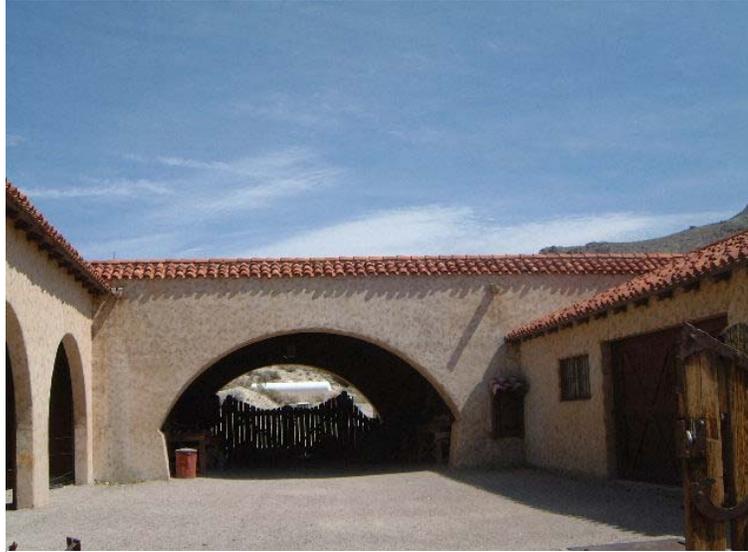


Photo 2.2: Left to right: shed, driveway, and stable. Photo taken in open courtyard looking east.

2.1 Description of Existing Conditions

Based upon the visual survey conducted by DeSimone personnel, the condition of the existing structural elements is described as follows:

1. The structures appear to be in good condition overall. The gravity framing of the structures, where exposed, is found to be in good condition. Member sizes and connections of roof trusses appear appropriate and adequate for their respective loading.
2. There was no sign of major damage of the structures. There are, however, some cracks on the exterior plaster of the structures, mostly around windows and under the tile roof.

3. Analysis and Evaluation of the Existing Structures

Linear static procedures were used to perform analyses and evaluations of the existing structures. The structures were modeled based upon the information obtained from the existing drawings and during the site visit by DeSimone personnel. No destructive or non-destructive testing was performed to establish properties of the existing building materials. No selective demolition or attempt to uncover hidden elements was made to establish configuration of structural components.

The analyses were performed based upon two independent criteria:

1. Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, ASCE/FEMA 356, November 2000.
2. International Code of Building Officials, *Uniform Code for Building Conservation (UCBC)*, 1997.

The FEMA 356 guidelines are based upon "expected strength" level design associated with spectral level seismic acceleration. The demand-capacity ratio of each structural component is computed and compared with an expected ductility factor m . The expected ductility factor m is provided in form of tables in the FEMA 356 guidelines and is based upon component testing as well as historical data. The response spectra are generated based upon national seismic hazard maps prepared by USGS by the specific location of the structure given by zip code or longitude and latitude.

The selected FEMA 356 evaluation criteria was based upon the Basic Safety Earthquake 1 (BSE-1) that represents 10% probability of exceedance in 50 years (475 years return period). The corresponding performance objective was selected to be "Collapse Prevention". This criterion is similar to the State Historic Building Code and in accordance with the Secretary of Interior Standards of Historic Preservation. Response spectra were computed based upon FEMA 356 guidelines. Two response spectra were obtained, one for the BSE-1 (10% in 50 years event, also called the Design Basis Earthquake, or DBE) and one for the BSE-2 (2% in 50 years event, also called the Maximum Credible Earthquake, or MCE) for comparison only. Figure 3.1 shows the generalized site response spectra for the BSE-1 and BSE-2 seismic events.

Scotty's Castle
 Preliminary Seismic Evaluation
 Stable Building
 Page 7 of 16

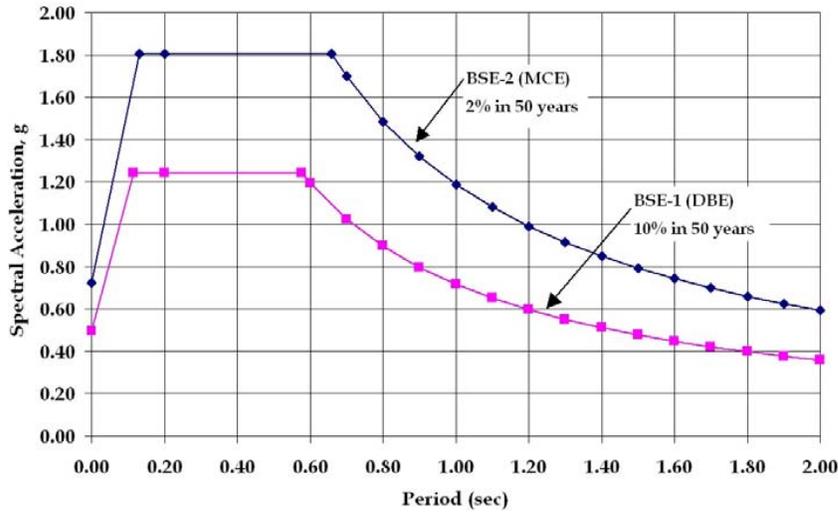


Figure 3.1 Generalized Site Response Spectra

The UCBC requirements, as modified by the CBC, are based upon “allowable strength design” and reduced response spectra. The response spectra are reduced by an overall reduction factor (R) to account for structure’s inelastic behavior and expected global ductility. The UCBC level of analysis is provided in this report for comparison purposes only. The structural evaluation and strengthening recommendations are based upon FEMA 356 criteria.

3.1 Analysis Methodology and Results

Linear Static Procedures were used for the analyses. All interior and exterior building walls that qualify to act as shear walls (based upon their aspect ratio according to FEMA 356) were considered to resist the earthquake forces.

The structural periods, total building weights, and base shears of each of the structures are presented in Table 3.1 below.

Table 3.1. Summary of Structures' Dynamic Characteristics

		Stable	Covered Driveway	Shed
Weight (lbs)		234,000	53,000	342,000
Period (sec)	FEMA 356	0.50	0.50	0.17
	UCBC	0.17	0.17	0.17
Base Shear	FEMA 356	1.34W	1.34W	1.77W
	UCBC	0.19W	0.19W	0.28W

The demand forces in the shear walls and roof diaphragms were computed and compared with corresponding allowable values according to the FEMA 356 and UCBC guidelines. The knowledge factor, ϕ , as defined in FEMA 356, is taken to be 1.0 based upon the information available for the analyses and an engineering judgment from our extensive experience on similar constructions.

The evaluation is based upon the demand-capacity ratios (DCR). For FEMA 356 evaluation, the element capacity is multiplied by the corresponding m factor to obtain a corresponding DCR. DCR of less than 1.0 indicates that the element capacity is larger than the demand and therefore, the element strength is adequate. DCR larger than 1.0 indicates that the element does not have adequate strength to resist the demand loads, and therefore requires strengthening.

Figure 3.2 show the building floor plan with grid line for easy reference.

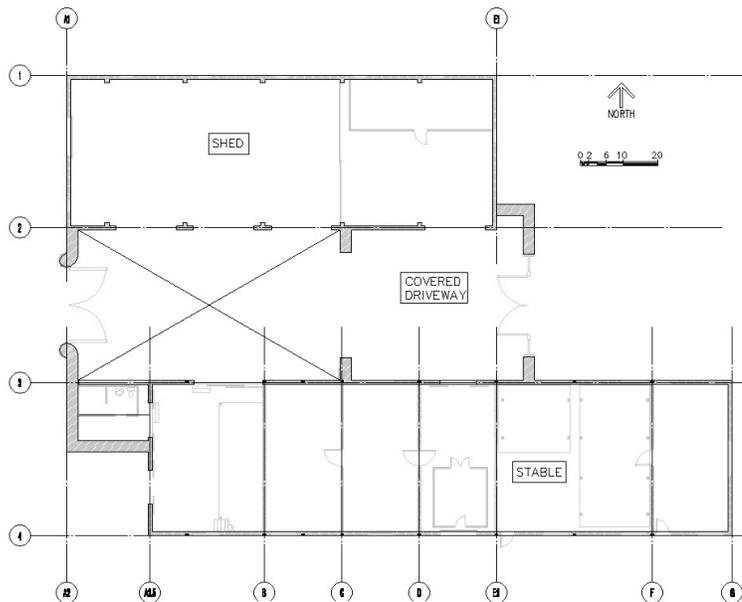


Figure 3.2: Floor Plan

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 9 of 16

The evaluation summary of shear walls is presented in Table 3.2. The Table shows that for the UCBC analysis, all walls in east-west and north-south directions have adequate lateral load resisting capacity. The DCR ranges from 0.03 to 0.75.

Table 3.2: Shear Wall Evaluation Summary

Grid Line	FEMA 356, BSE-1, CP				UCBC			
	Demand plf	Capacity plf	DCR	Comments	Demand plf	Capacity plf	DCR	Comments
East-West Direction								
1	3,056	13,661	0.22	OK	338	11,612	0.03	OK
2	10,554	13,661	0.77	OK	1,159	11,612	0.10	OK
3	2,159	1,320	1.64	NG	219	400	0.55	OK
4	1,171	1,320	0.89	OK	119	400	0.30	OK
North-South Direction								
A1	11,635	13,661	0.85	OK	1,288	11,612	0.11	OK
E1	13,152	13,661	0.96	OK	1,456	11,612	0.13	OK
A2	1,620	1,320	1.23	NG	164	400	0.41	OK
B	1,308	1,320	0.99	OK	133	400	0.33	OK
C	2,376	1,320	1.80	NG	241	400	0.60	OK
D	1,275	1,320	0.97	OK	129	400	0.32	OK
E2	2,965	1,320	2.25	NG	301	400	0.75	OK
F	1,864	1,320	1.41	NG	189	400	0.47	OK
G	523	1,320	0.40	OK	53	400	0.13	OK

For FEMA 356 analysis, in the east-west direction, all shear walls except the wall on grid line 3 have adequate strength to resist the applied loads. The shear wall on grid line 3 has a DCR of 1.64 and therefore, has inadequate strength to resist the lateral loads. In the north-south direction the walls on grid lines A2, C, E2 and F do not have adequate strength to resist lateral loads. The DCR for these walls ranges from 1.23 to 2.25. All other walls have DCR ratio less than 1.0 and therefore, have adequate lateral load resisting capacity.

Table 3.3 presents a summary of the roof diaphragm evaluation. The roof diaphragm over the shed building is found to be inadequate based upon both the UCBC and the FEMA criteria. The diaphragm for the covered driveway is also found to be inadequate to transfer earthquake forces in the east-west direction. The diaphragm for the stable is found to be satisfactory due to their relatively short spans.

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 10 of 16

Table 3.3: Diaphragm Evaluation Summary

Grid Line	FEMA 356, BSE-1, CP				UCBC			
	Demand plf	Capacity plf	DCR	Comments	Demand plf	Capacity plf	DCR	Comments
East-West Direction								
1 & 2	3,025	630	4.80	NG	335	250	1.34	NG
2 & 3	805	180	4.47	NG	81	100	0.81	OK
3	1,040	1,125	0.92	OK	106	250	0.42	OK
4	1,114	1,125	0.99	OK	113	250	0.45	OK
North-South Direction								
A1 & E1	8,405	630	13.34	NG	929	250	3.72	NG
A2 & A2.5	456	1,125	0.41	OK	44	250	0.18	OK
A2 & A2.5	698	1,125	0.62	OK	71	250	0.28	OK
A2.5 & B	400	1,125	0.36	OK	41	250	0.16	OK
A2.5 & B	387	1,125	0.34	OK	39	250	0.16	OK
B & C	460	1,125	0.41	OK	47	250	0.19	OK
B & C	631	1,125	0.56	OK	65	250	0.26	OK
C & D	486	1,125	0.43	OK	50	250	0.20	OK
C & D	462	1,125	0.41	OK	47	250	0.19	OK
D & E2	372	1,125	0.33	OK	38	250	0.15	OK
D & E2	577	1,125	0.51	OK	60	250	0.24	OK
E2 & F	551	1,125	0.49	OK	56	250	0.22	OK
E2 & F	1,081	1,125	0.96	OK	109	250	0.44	OK
F & G	847	1,125	0.75	OK	86	250	0.34	OK
F & G	244	1,125	0.22	OK	26	250	0.10	OK

3.3 Summary of Findings and Proposed Retrofit Strategy

Based upon the analysis results presented in this section, the shear walls and the diaphragms in the stable building do not have sufficient strength to conform to the stated performance objective. The performance objective was defined earlier in this section, and comprises a BSE-1 earthquake (10% in 50 years event) and Collapse Prevention performance. The following is a summary of the findings.

1. Existing wood stud and lath and plaster shear walls at the stable and the covered driveway do not have adequate shear capacity to resist lateral loads.
2. The shed and the covered driveway roof diaphragms are found to have inadequate shear capacity to resist the lateral loads.
3. The wall to roof diaphragm anchorage for in-plane forces does not appear to be adequate in all of the three structures. In addition, out of plane wall to diaphragm anchorage for the shed building is not adequate to resist the wall out-of-plane loads.

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Scotty's Castle
 Preliminary Seismic Evaluation
 Stable Building
 Page 11 of 16

To mitigate these issues and to improve the building performance in order to meet the stated performance objective, following measures are proposed. Figure 3.3 presents the conceptual retrofit strategy.

1. To strengthen shear walls, remove plaster on lath on selected walls and add ½” plywood sheathing to increase the shear capacity of the walls. Once the addition is made plaster can be applied to the plywood to match existing architecture.
2. To increase diaphragm capacity, add ½” plywood at the bottom of roof trusses. This method eliminates the need of removing the tiles on the roof and thus retains the original architecture.
3. Provide positive in-plane and out-of-plane wall to roof diaphragm anchorage.

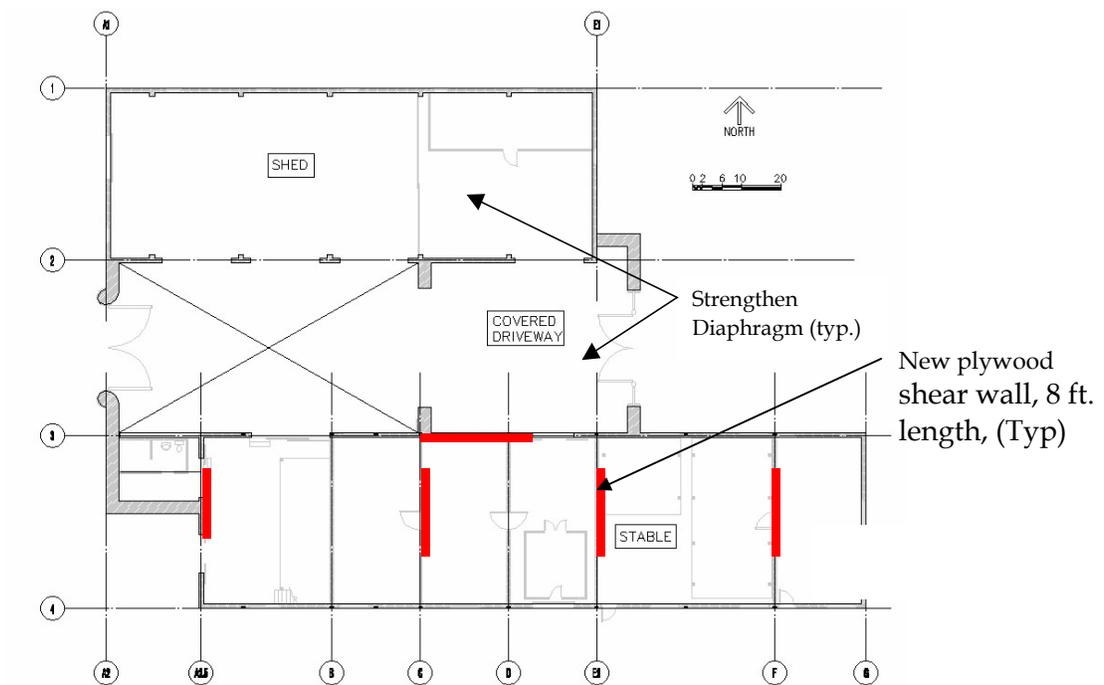


Figure 3.3: Conceptual Seismic Rehabilitation Recommendation

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 12 of 16

The following drawings are part of this report to document the existing condition of the structures.

- S8.1: Roof Framing Plan of Existing Condition
- S8.2: Roof Framing Plan of Existing Condition
- S8.3: Elevations and Details of Existing Condition

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 13 of 16

4. References

1. Federal Emergency Management Agency, *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*, ASCE/FEMA 356, November 2000.
2. International Code of Building Officials, *Uniform Code for Building Conservation (UCBC)*, 1997.
3. International Code of Building Officials, *California Building Code (CBC)*, 2001.
4. International Code of Building Officials, *Uniform Building Code (UBC)*, 1997.
5. Original building drawings dated February 18, 1928.
6. Carey & Co., Measured Drawings.

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 14 of 16

5. Photographs of Existing Conditions



Photo-1: North Face of Stable



Photo-2: South Face of Stable



Photo-3: Crack around Window, Stable



Photo-4: Roof Framing in Stable

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 15 of 16



Photo 5: Roof Truss End Configuration, Stable



Photo 6: Roof Truss Connection, Stable



Photo-7: Roof Framing, Covered Driveway



Photo-8: South Wall of Shed

Scotty's Castle
Preliminary Seismic Evaluation
Stable Building
Page 16 of 16



Photo-9: North Wall of Shed



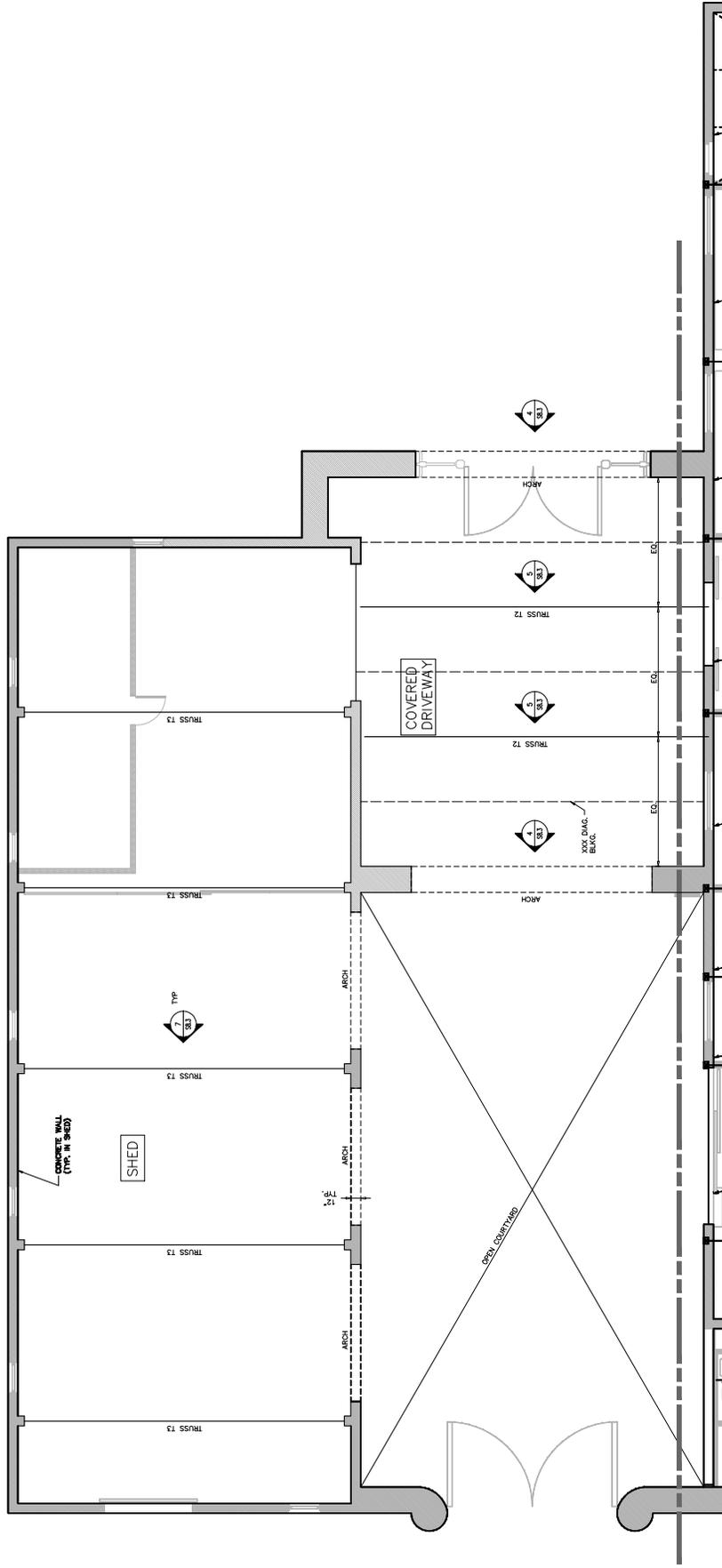
Photo-10: Crack under Roof, Shed



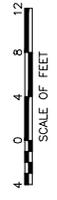
Photo-11: Damaged Tiles, Shed



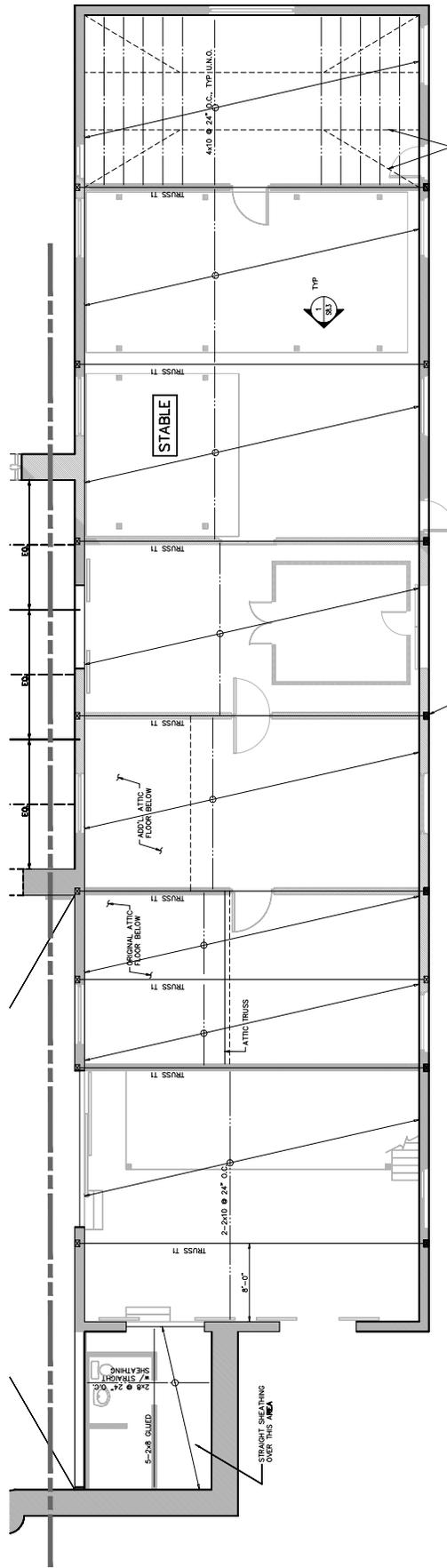
Photo-12: Roof Trusses and Framing, Shed



ROOF FRAMING PLAN
SCALE: 3/16" = 1'-0"



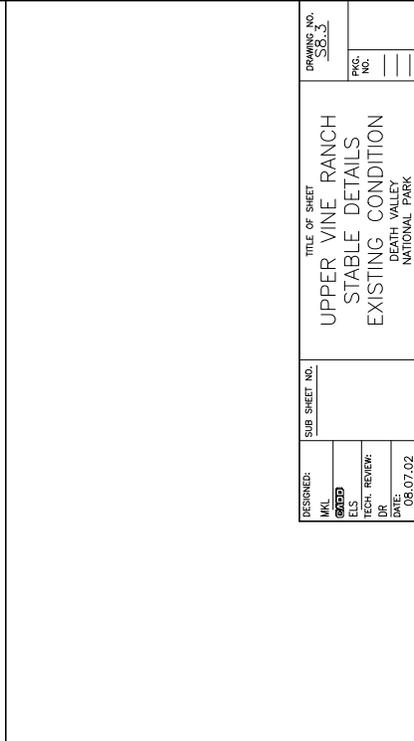
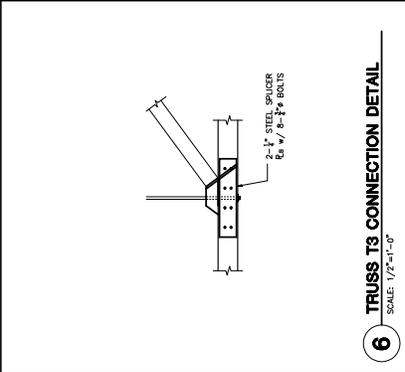
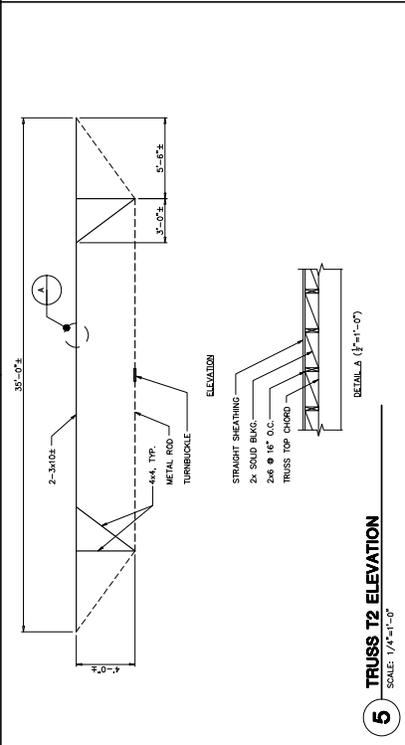
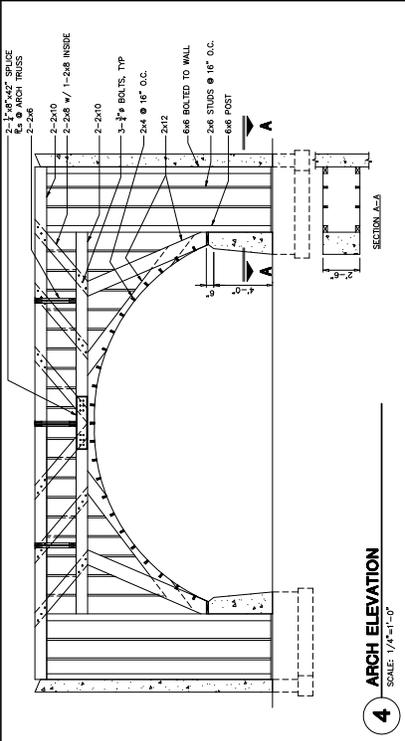
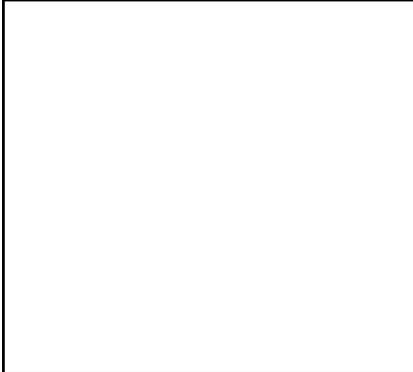
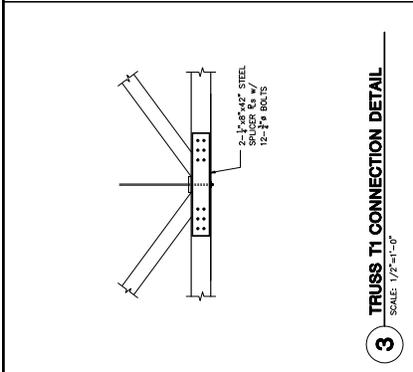
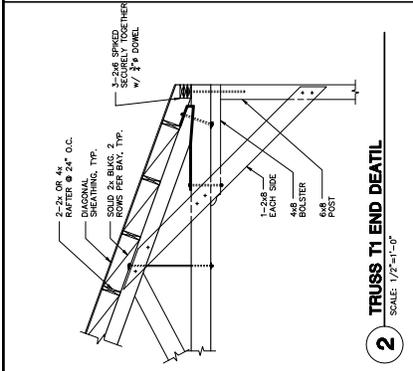
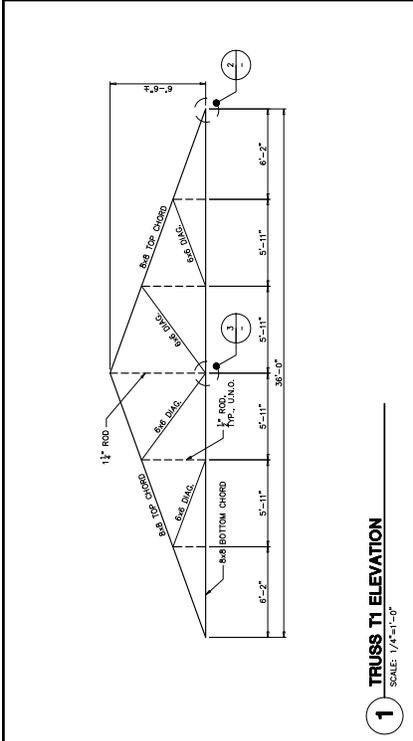
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TECH. REVIEWER: DR			PKG. NO.:
DATE: 06.07.02			



ROOF FRAMING PLAN
SCALE: 3/16" = 1'-0"



DESIGNED BY: MML	DATE: 08/08/07	DR:	DATE: 08/07/02
CHECKED BY: JL	DATE: 08/08/07	DR:	
TECH. REVIEWER: DR			
SUB SHEET NO.:		DRAWING NO. SB.Z	
TITLE OF SHEET: UPPER VINE RANCH STABLE EXISTING CONDITION DEATH VALLEY NATIONAL PARK			
PKG. NO.:		SHEET NO.:	



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DR:			EXISTING CONDITION	
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