Jensen – Van Lienden Associates, Inc.
GEOTECHNICAL ENGINEERING CONSULTANTS

Curtis N. Jensen
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December 17, 2008
Job No. U146AA

Navarro-by-the-Sea Center
PO Box 1710
Mendocino, California 95460

Attention: James Martin

Re: Geotechnical Engineering and Geologic Considerations
Navarro-by-the-Sea Center – Albion, CA

Gentlemen:

This report summarizes opinions concerning the geotechnical engineering and geologic conditions at the site of the Navarro-by-the-Sea Center at the mouth of the Navarro River near Albion, California. The center site is illustrated on Figure 1.

Opinions expressed in this report are based upon a collaborative evaluation of the site made by Jensen-Van Lienden Associates and Joyce Associates, geologic consultants. The scope of this evaluation included examination of and analyses from previously developed data regarding the site soil conditions, separate site examinations made by our personnel and those of Joyce Associates, interpretation of stereopair area photos and review of historic and published geologic information.

Joyce Associates prepared a separate discussion of the site geology and the issues presented by the site geology and topography for the project. This discussion is presented as a report in Appendix A.

SITE DESCRIPTION AND GEOTECHNICAL CONDITIONS

The Navarro-by-the-Sea Center historic buildings are located on a sand bar/stream terrace along the southeasterly side of the Navarro River, near the point where the river discharges into the Pacific Ocean. The terrace is relatively level beneath and in the beach area at the front of the buildings; the elevation change between the ground at the building front and the edge of the Navarro River was no more than a few feet on May 22, 2008, when we examined the site.

The rear of the buildings abuts the base of a hillside that rises moderately steeply to the top of bluffs adjacent to the river and the coastline. The hillside inclination varies somewhat, averaging about 1-1/2 horizontal to 1 vertical.

The stream terrace opposite the buildings is sparsely covered with grass and native, woody plants and trees. In contrast, the hillside is densely overgrown with vines, brush and trees.

Bauer Associates, Geotechnical Consultants, drilled four test borings close to the four corners of old motel building. Borings 3 and 4, near the front of the motel were drilled into the stream terrace and encountered a minor amount of fill over approximately 12 feet of loose silty sand. The silty sand overlies medium stiff-to-stiff sandy clay, which persisted to the maximum as-drilled depth (approximately 20 feet in test boring 4).

Borings 1 and 2 were drilled near the motel rear corners. These borings encountered a minor amount of silty sand and sandy clay soil resting on sandstone (sedimentary rock). Rock also outcrops in excavations behind both buildings as well as in the excavation above the beach park access road southwest of the site.

Groundwater migrated into all four of the Bauer Associates borings. They measured the level to which the groundwater accumulated; the measured level ranged from about ½ foot to 2 feet below the ground surface, i.e. the soils beneath the buildings are essentially fully saturated.

We understand that the project envisioned restoring the building(s) to their original, historic condition, and utilizing the restored building(s) as a visitor and research center. Thus, one or both buildings would be occupied during much of the workweek. We anticipate the restoration work will include construction of new foundations; recommendations for the foundations are given below.

GEOTECHNICAL CONCLUSIONS AND DISCUSSION

1. Liquefaction

As mentioned, the Bauer Associates' borings show the buildings are underlain by silty sand. This soil is probably similar to that exposed on the ground surface throughout much of the stream terrace, and along the beach adjacent to the river. The borings also show that the sand thickness beneath the motel increases with distance towards the river; the base of the upslope marks the edge of the stream terrace. It is reasonable to expect that the silty sand layer continues to increase in thickness towards the river, and it is conceivable that it is many tens of feet thick beneath the river itself.

The sparse Bauer Associates data suggest that the silty sand has a loose consistency (i.e. a low relative density) and they confirmed that it is saturated by groundwater. Loose, low density saturated sands and silty sands are susceptible to being liquefied if subjected to repeated shocks generated by a moderate earthquake with a source region near the site or a much larger earthquake with a more distant origin.

Liquefaction is the transformation of reasonably firm soil, having measureable strength and foundation bearing capacity, to a medium resembling a thick, viscous fluid possessing little or no shear strength and foundation bearing capacity. Liquefaction is a transient condition; drainage from the liquefied zone following cessation of the earthquake usually restores the sand to its original strength and supporting characteristics in a matter of minutes or hours. The mechanics of this transformation have been intensively researched and therefore are well known; they are not discussed in detail herein.

There is a reasonably high probability that the site will be subjected to an earthquake with a magnitude large enough to create liquefaction in the site silty sand. Therefore, in our opinion the associated liquefaction risk should be considered as part of the overall project planning.

We conclude that liquefaction could pose two potential hazards for the building(s) and the building occupants. These two hazards are discussed in following paragraphs.

As mentioned, liquefaction would substantially reduce the capacity of soil to support the building foundations. The soils directly beneath the foundations could then fail in bearing; bearing failures would cause the building(s) to experience large total and differential foundation settlements. While such settlements could damage the building(s), in our view, it is unlikely that they would cause the building to collapse and is therefore not a life safety issue. Furthermore, stiffening the foundation system, as discussed in more detail below, can, we believe, lessen damage from this hazard.

The other hazard is described by the term "lateral spreading". Because the liquefied soil has very low strength, there could be a tendency for the ground beneath the building(s), and the stream terrace as a whole, to move laterally towards the river channel after the silty sand liquefies, either before the earthquake ground motions stop or subsequently. This lateral movement should be minimized somewhat by the relatively flat inclination of the slope between the building fronts and the edge of the river, but the experience at other sites in other earthquakes indicates that even a liquefied area of modest slope can shift laterally, particularly if the ground motions are strong enough to accelerate the liquefied zone in the downslope direction (i.e. towards the river in this case). Lateral spreading is thus akin to a landslide that moves laterally on a nearly level failure surface. Judging from our site observations, and the Bauer Associates boring data, lateral spreading at the subject site is a real possibility.

Lateral spreading presents a life safety issue for the occupants of the building(s), if the lateral spreading movements were large enough to carry the building(s) some distance towards the river and if the foundation system is inadequately stiff and strong enough to prevent the building(s) from being structurally damaged by the movements. We believe this hazard can be mitigated sufficiently by constructing stiff, shallow foundation systems that would allow the building(s) to remain intact even if liquefaction and post-liquefaction lateral spreading and differential settlement were to occur. Such a system would enable the building(s) to remain relatively intact such that the occupants could safely exit the building following the earthquake.

Design criteria for a shallow and stiff foundation system are given in the Recommendation section.

Although the shallow stiff foundation system would, in itself, mitigate the life safety hazard, it is likely that the building(s) would not be salvageable if significant lateral spreading movements do occur. Unless the building(s) are restrained from moving if the lateral spreading occurs. Completely protecting the buildings from structural damage or even complete loss of use would require either treating the soils beneath in the vicinity of the building(s) to eliminate their liquefaction potential and/or to support the building(s) on deep foundations designed to accommodate forces associated with liquefaction lateral spreading. As an alternative to these expensive approaches, we suggest the building(s) be restrained by extending the rear foundation(s) into the site rock, effectively pinning the buildings to the rock at the base of the upslope. Specific criteria for this deepening are given below.

2. California Building Code Seismic Design Criteria

The site latitude is 39.19 degrees north and the site longitude is 123.76 degrees west.

We estimate that the site short period spectral acceleration is 1.581g, and that the site 1-second period spectral acceleration is 0.800g.

Given the proximity of rock beneath the site, particularly at the rear of the buildings, we estimate that the site class as defined in the California Building Code is C, or possibly D. If the renovation includes seismic structural upgrading, we suggest Class C for design, despite the liquefaction potential. Liquefaction of the underlying soils would theoretically damp the ground motions transmitted into the building(s) somewhat; because liquefaction cannot be predicted with 100% certainty, the building(s) should be designed for the scenario where liquefaction does not occur (i.e. with Class C).

RECOMMENDATIONS

1. Foundations

On the basis of the discussion given in the Conclusion and Discussion section of this report, we recommend that the renovated building(s) be re-supported upon spread footing foundation system(s). Footings should be made continuous around the perimeter of the structure(s), and be beneath interior bearing walls and new shear walls, if any.

In addition, continuous tie beams should be constructed to structurally connect the new footings. We recommend tie beams have spacings on the order of 15 feet. These tie beams should be constructed in both directions, i.e. parallel and perpendicular to the building long dimensions. These tie beams can be conventionally reinforced. Interior footings that are installed to support bearing or shear walls can be substituted for the tie beams. The intention of the tie beams is to connect the supporting footings with a grid system that provides rigidity and stiffness to the entire foundation system.

We recommend bearing footings be designed for an allowable bearing pressure of 1000 pounds per square foot for dead plus live loads. This value may be increased by the usual one-third factor to account for seismic loads. Regardless of the actual developed bearing pressures, we recommend that footings have a minimum width of 18 inches.

If lateral spreading beneath the building(s) is initiated, the laterally moving ground could transport the buildings northwesterly towards the river. The state of geotechnical engineering practice has not reached a point where accurate predictions of the total deformation the laterally spreading ground could undergo. However, order of magnitude estimates can be made; using the Bauer Associates data, we judge the movements could be large enough to shift the building(s) from their current (and future) sites several feet, possibly rendering them unusable for their intended purpose. Nonetheless, supported on foundations designed as recommended, we believe the building(s)would still be largely intact, at least to the extent that occupants could safely exit.

In our opinion, these potential building lateral movements can be prevented, or at least minimized to a large extent, by deepening the continuous footing supporting the building(s) rear walls into the rock that underlies the site. The rear wall footings would then pin the building(s) to the stable rock at the rear, and act to anchor the building(s), preventing them from shifting laterally as the underlying soil liquefies and laterally spreads. Drilled piers could also be used to pin the building to rock as well.

To accomplish the anchoring, we recommend that the rear wall continuous footing extend a minimum of 3 feet into the site rock, or 3 feet below the surface fill and silty sand, whichever is less. We anticipate that this would require relatively deep footing, cast into a trench excavated along the rear wall lines.

If drilled piers are designed to pin the building at the rear wall, the reaction against the piers should be modeled as a passive pressure of 300 psf per foot of depth, applied over two pier diameters, and beginning from finished grade. Piers should be no closer than two diameters, center to center.

If the building(s) are effectively pinned at their rear walls, the laterally spreading soil beneath the building(s) could exert drag forces on the remaining foundations. To minimize these forces, we recommend that the remaining new footings be constructed to bear at a depth no more than 8 inches below the lowest adjacent grade. Although this embedment depth is less than commonly required for spread footings (and perhaps less than required by the Building Code) it should be sufficient to develop the allowable bearing pressure recommended above.

The drag forces will create tensile stresses in the transverse footings and tie beams, which should be accounted for in the foundation structural design. We judge that the distribution and size of the drag forces will depend upon the foundation layout. Therefore, when the layout of the foundation system has been determined, we recommend it be transmitted to our firm for a review. The purpose of the review would be to develop specific criteria for the forces and stresses the structural engineer could use as input into the design. An initial estimate of the drag force can be made by assuming it is equal to the building dead load plus real live load exerted on the footings times a friction factor equal to 0.4.

As discussed, the building(s) could still settle differentially if the underlying soils do liquefy even if the building(s) are effectively pinned to the site rock. The differential settle should lead to cosmetic and architectural distress within the building(s).

However, we would expect this damage to be no greater than the damage associated with the ground shaking and inertial forces developed in the buildings themselves, and perhaps indistinguishable from the shaking damage. If necessary, the buildings could be releveled on the new foundations.

It would be acceptable to raise the grades beneath the building provided the bearing depth of the footings is no more than 8 inches as noted above. The footings could be formed and constructed first and the spaces within the grid of footings filled afterwards. The edges of these fills should be sloped down to prevent embedding the footings beyond the 8 inch depth.

2. Retaining Walls

New retaining walls may be required at the back of the building(s). Observations indicate that these walls would support either excavations into the site rock/stiff soils, or modest backfills placed between excavations and the walls themselves. Given this condition, we recommend the walls be designed utilizing pressures that would be developed by equivalent fluid weights with values given in Table I.

It would be preferable to backfill the walls, if necessary, with free draining granular materials, i.e. sand or gravel. This type of backfill would avoid having to compact a soil backfill to engineered fill standards.

Walls can be supported upon spread footings. Bearing pressure at the base of the footing will generate the existence to vertical loads and overturning moments; values for bearing pressure are given above.

The resistance to lateral loads will, we believe, be generated by both friction along the base of the footing and passive pressure against the leading edge and any footing shear key. We recommend a passive pressure of 300 psf per foot of depth. The friction resistance would be equal to a factor times the normal force on the footing. We estimate that the factor will be equal to 0.4.

Subsurface drains should be installed behind retaining walls to intercept seepage; the walls should also be thoroughly waterproofed. Details for our standard recommendation for subsurface drains are shown on Figure 2.

3. Construction Observation and Further Services

We recommend that our firm be retained to review building plans for the project in order to assess whether the design criteria presented in this report have been correctly incorporated into the project plans. In this regard, we suggest that the members of the project design team consult with us on an as-needed basis to answer any questions that may arise regarding the design criteria.

Our firm should be retained to be on site when excavations for foundations are being made. The purpose of our site observations would be to verify that the materials into which the excavations are being made are consistent with our expectations, and are the supporting materials as defined in the soil report. Being on site will also enable us to provide recommendations for changes to the project if conditions different from those anticipated are encountered during construction.

LIMITATIONS

The opinions expressed in this report are based on a visual examination of the property and on the scope of work described above. No test borings or any other type of subsurface investigation was undertaken. While we believe that our conclusions are well founded, it is possible that there may be undiscovered conditions that would cause us to revise our opinions and/or recommendations. The report, therefore, should not be construed to be any type of guarantee or insurance.

A more detailed study could be undertaken to confirm the accuracy of the opinions expressed in this report and to determine the specific soil and rock conditions existing on the lot. Such a study could include test borings, laboratory tests and/or other methods of investigation. We would be pleased to perform such a study if you desire.

We are pleased to have been of service to you on this project. Please do not hesitate to call us if you would like to discuss the contents of this report.

Very truly yours,

JENSEN-VAN LIENDEN ASSOCIATES, INC.

Curtis N. Jensen
G. E. # 438

REGISTERED PROFESSIONAL ENGINEER
CURTIS N. JENSEN
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TABLE I

ALLOWABLE EQUIVALENT FLUID WEIGHTS FOR DETERMINATION OF RETAINING WALL DESIGN BACKFILL PRESSURE

Slope Inclination Behind Wall Horizontal:Vertical	Equivalent Fluid Weight (pcf)
Level	35
4:1	40
3:1	45
2:1	55

Notes:

1. Slope inclination behind retaining walls should be no steeper than 2:1.

2. These equivalent fluid weights assume that the backfill will be drained (See Figure 2), and that the backfill will consist of free draining material (sand or gravel).

3. Linear interpolation can be used for backfill slopes with inclinations different from those shown above.

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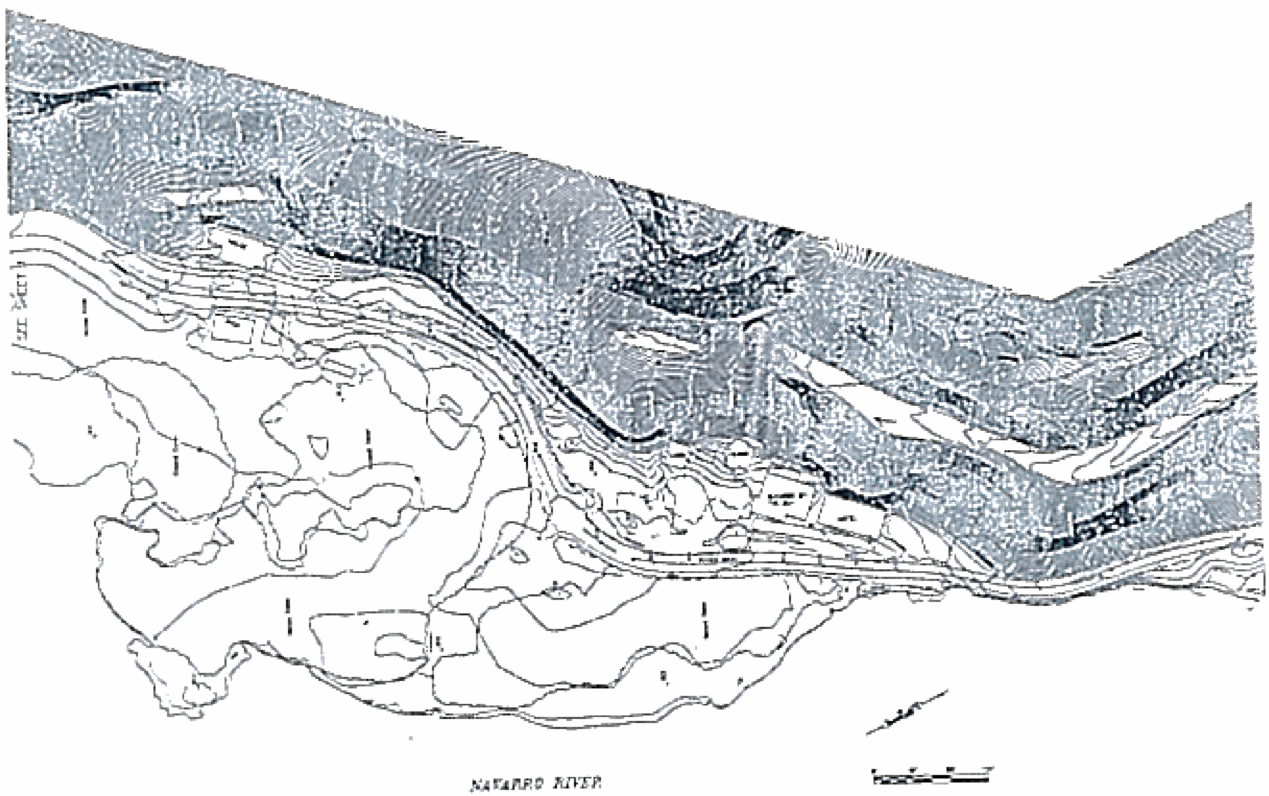

Jensen – Van Lienden
Associates, Inc.

SITE PLAN
NAVARRO BY THE SEA CENTER

Date	Figure	Job No.
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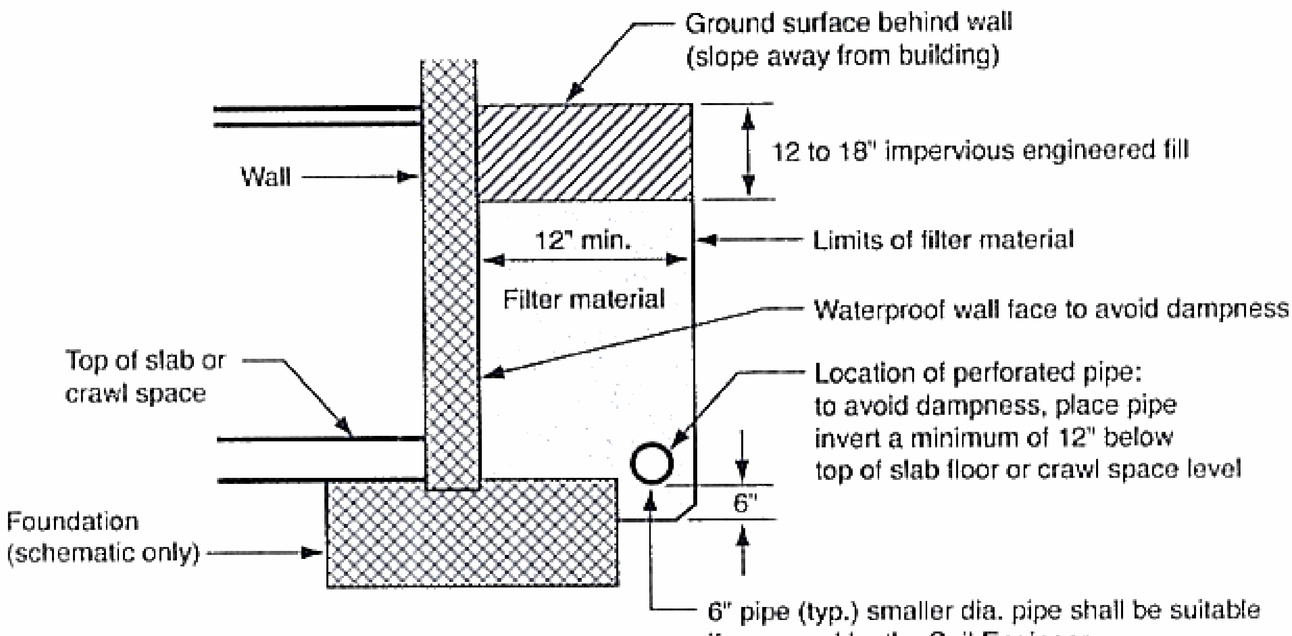
SUBDRAINAGE BEHIND RETAINING WALLS



TYPICAL SECTION
(Not to Scale)

Subdrain pipe shall be manufactured in accordance with the following requirements:

a. Acrylonitrile-butadiene-styrene (ABS) plastic pipe shall conform to the specifications for ABS plastic pipe given in ASTM Designation D2282 and ASTM Designation D2751. ABS pipe shall have a minimum pipe stiffness of 45 psi at 5% deflection when measured in accordance with ASTM Method D2412.

b. Polyvinyl chloride (PVC) pipe shall conform to AASHTO Designation M278. PVC pipe shall have a minimum pipe stiffness of 50 psi at 5% deflection when measured in accordance with ASTM Method D2412 except that pipe conforming to F758 shall be suitable. Schedule 40 PVC pipe shall be suitable.

Filter material for use in backfilling trenches around and over subdrain pipes shall consist of clean coarse sand and gravel or crushed stone conforming to the following requirements:

Sieve Size	% Passing Sieve
2"	100
3/4"	70 to 100
3/8"	40 to 100
#4	25 to 50
#8	15 to 45
#30	5 to 25
#50	0 to 20
#200	0 to 3

Class 2 "Permeable Material" conforming to the State of California Department of Transportation Standard Specifications, latest edition, Section 68-1.025 shall be suitable.

Clean, coarse gravel ("drain rock") shall be suitable, provided the subsurface drain is wrapped in an acceptable geotextile ("filter fabric").

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August 25, 2008

Job Number 276.01

Navarro–by–the-Sea Center
P.O. Box 1710
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Attn: Mr. James Martin

RE: Summary – Geologic Reconnaissance
Navarro–by–the-Sea Center
Mendocino County, California

Gentlemen:

INTRODUCTION

This report summarizes our geologic observations and data review for the geologic aspects of the proposed Navarro–by–the-Sea Center. The proposed project is to renovate an old hotel building to create a visitors center and office space. The subject building is located on the south side of the Navarro River, approximately one-tenth mile from the Pacific Ocean. The building is located adjacent to the lagoon at the mouth of the Navarro River on near-level ground at an elevation of a few feet above sea level. The rear of the building is immediately adjacent to a steep slope, which extends several hundred feet up slope to the south.

Previously, a geotechnical investigation of the site was performed by Bauer Associates (BA), although no report was apparently written. The logs of Bauer Associates borings were provided for our review.

During this project we collaborated with Jensen-Van Lienden Associates (JVLA) of Berkeley, California. JVLA provided geotechnical engineering consultation and evaluation for the project. Their recommendations are summarized in a letter dated July 18, 2008.

SCOPE OF WORK

The purpose of our geologic reconnaissance was to provide geologic information for use in project planning. Our scope of work included:

- performing a geologic site reconnaissance
- reviewing available published geologic maps
- reviewing one set of stereo-paired aerial photographs
- reviewing the Bauer Geotechnical boring logs
- providing geologic information to JVLA for use in their geotechnical evaluation of the site and reviewing their draft report
- preparation of the report

REVIEW OF PUBLISHED GEOLOGIC MAPS

Only broad-scale published geologic maps are available for the subject area. Jennings and Strand (1960) show that the hills in the site vicinity are underlain by sedimentary rocks of Cretaceous age, generally assigned to the Franciscan Assemblage. The low terrace that borders the river is mapped as alluvium of Quaternary age. Some marine terrace deposits are shown on the coastal terraces to the north and south of the site.

Jennings (1994) shows that the San Andreas Fault is approximately 2.5 miles west of the site. To the east, the closest known active fault is the Maacama fault, approximately 27 miles east of the site.

Youd and Hoose (1978) prepared a map of historical liquefaction features in Northern California. The map is rather generalized and based on historical reports from geologic reports, newspapers and other media. Youd and Hoose (1978) show no indications of liquefaction in the site vicinity. However, indications of lateral spreading, sand boils and other liquefaction-related phenomenon were reported in alluvial areas to the north and south, including Mendocino, Fort Bragg, Cleone, Inglenook, Westport, and Rockport. These areas are generally geologically similar to the subject site.

Lawson (1908) prepared an extensive report compiling observations made throughout the State of California of damage related to the great California Earthquake of 1906. Page 176 of that report describes the damage in Navarro as follows:

"Navarro, Mendocino County (F.E. Mathes) this town is an abandoned one, and the conspicuousness of its damage may perhaps in large measure be attributed to the neglected state of the buildings. Nearly every house, except for a few still occupied, suffered partial collapse of it's under pinning, so that from whatever point the town be viewed, it presents the same remarkable jumble of leaning, half ruin houses. Its location on the flat, alluvial bottom next to the river, probably contributed to the severity of the damage. In fact, the entire series of villages and towns visited on this section of the coast, this is the only one that stands on alluvial ground; all the others are built on firm rock terraces. The great wooden bridge at Navarro showed no damage whatever."

Lawson (2008) estimates the shaking intensity at Navarro at approximately IX on the Modified Mercalli Intensity Scale.

REVIEW OF AERIAL PHOTOGRAPHS

One set of black and white stereo-paired aerial photographs taken on July 14, 1954 were reviewed as a part of our reconnaissance (Photo ID AV133-03-2, 3) obtained form Pacific Aerial Surveys in Oakland, California. The photographs show that the subject building and the surrounding buildings are present at that time. Some of the buildings appear to be occupied. The old alignment of Highway 1 is still in use and the new alignment of Highway 1 has not been constructed. None of the houses south of the site, along the new alignment of Highway 1, were constructed by this time.

The old alignment of Highway 1, directly up slope of the site, is in use and appears to be in good condition. A moderately large fill, placed as a part of the construction of Highway 1, is present in the swale up slope of the buildings. We observed no indications of a landslide within the fill or on the slope above the subject site. The site development appears to have included excavation of a small cut slope at the rear of the buildings.

The slope immediately above the site is heavily vegetated. Above the new alignment of Highway 1, the terrain is less steep and includes near-level, grassy, marine terrace areas, intermixed with steeper slopes containing some brush and trees.

SITE RECONNAISSANCE

We performed a reconnaissance of the site and the surrounding area, including the portions of the current and former alignments of Highway 1 above the site. A cut slope of ranging up to approximately 10 feet in height is present at the rear of the hotel building. This cut was originally inclined at approximately 0.5:1 (h:v) but has been modified by sloughing and erosion. The remainder of the building site slopes gently to the north.

Both the current and former alignments of Highway 1 upslope of the site were constructed by cutting into the hill and placing fill on the downslope side. The fills appear stable on show no indications of landsliding. The cuts along the uphill side of the roads are typically inclined at about 1:1 and show some indications of localized sloughing and erosion. We found no evidence of landslides on the slope above the subject building.

The subject building is in relatively poor condition and appears to have experienced some settlement.

BAUER ASSOCIATES INVESTIGATION

Bauer Associates (2008) performed a geotechnical investigation of the motel building, which is immediately west of the hotel building. Four borings were drilled roughly at the corners of the building. Borings 1 and 2 at the rear of the building encountered a thin layer of silty sand and sandy clay ranging from six feet in thickness in Boring 1 to three feet in thickness in Boring 2. These soils were underlain by sandstone in both borings to the full depth explored, which was 7.5 and 4 feet, respectively. Borings 3 and 4, were located near the front of the motel building. Boring 3 encountered approximately one foot of clayey gravel fill underlain by a thin layer of sandy clay extending to a depth of about three feet. From three feet to 12 feet the boring encountered loose, saturated, silty sand. From 12 feet to the bottom of the boring at 19 feet, medium stiff to stiff sandy clay was encountered. Boring 4 encountered fill consisting of silty sandy gravel to a depth of approximately three feet. Below that, the boring encountered loose, saturated silty sand to a depth of 12 feet. Below that depth, the boring encountered medium stiff, saturated sandy clay to the full depth explored, which was approximately 20 feet. All four borings reported water levels varying from 1/2 to 2 feet after completion of the boring, indicating that the ground is saturated to near the ground surface.

The Bauer Associates borings show that the rear of the motel building is underlain by bedrock at a depth of a few feet. Conversely, bedrock was not encountered in either of the borings at the front of the building despite extending to depths as much as 20 feet.

Because of the close proximity of the subject building to the motel building, and the similarity of the topography, it is likely that the geologic conditions beneath the subject building are very similar to those encountered beneath the motel building.

CONCLUSIONS

The presence of loose, saturated, silty sand in the borings at the front of the motel building indicates that there is a significant risk of liquefaction at the site in the event of a moderate or large earthquake in the area. Blow counts within these materials were on the order of a few blows per foot, which indicates a loose and liquefiable condition. Therefore, we conclude that there is a significant risk of liquefaction at the site.

Presently, the risk of a large earthquake in the area is not well known, however it should be assumed that there is a significant risk of a large earthquake within the region during the next few decades. The most likely source of a large earthquake is the San Andreas Fault, located offshore approximately 2.5 miles west of the site. It is also possible that a large earthquake occurring on the Maacama fault 27 miles to the east of the site could cause liquefaction at the site.

In the event that a large earthquake causes liquefaction at the site, a number of affects could occur, including settlement of the on-site soils, sand boils, and lateral spreading (a lateral movement of the soil toward the river). It is beyond the scope of this report to predict the magnitude of settlement or lateral spreading that could occur, however it is likely that the settlement would be at least several inches. At present, it is difficult to predict the amount of lateral spreading that could occur, however the overall amount of lateral spreading may exceed the amount of settlement, and it could be up to several feet.

It is worth noting the description given by Lawson (2008), which was quoted earlier in this report. The description of the damage is suggestive that liquefaction and perhaps some lateral spreading occurred within the site sufficient to cause a loss of foundation report and significant distress to the existing buildings. The type of damage described at the site as a result of the 1906 earthquake is probably generally similar, but could be less severe than the damage resulting from a future large earthquake located closer to the site.

No active faults have been mapped within or projecting toward the subject site. We therefore conclude that the risk of fault rupture beneath the subject building is very low. We found no evidence that the slope above the site is unstable, therefore the risk of landsliding is judged to be low. The cut slope behind the building has experienced some sloughing and erosion. This problem can be mitigated by constructing a properly designed retaining wall.

LIMITATIONS

The conclusions and opinions expressed in this report are based on the study methods described in this report. This report was prepared to provide geologic opinions only. This report was prepared in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions. No other warranty expressed or implied is made.

CLOSURE

We trust this provides the information required at this time. Please call if you have any questions.

Yours very truly,

Joyce Associates

PROFESSIONAL GEOLOGIST
JAMES M. JOYCE
No. 1517
CERTIFIED
ENGINEERING
GEOLOGIST
STATE OF CALIFORNIA

James M. Joyce

James M. Joyce
Principal Engineering Geologist

REFERENCES

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Jennings, Charles W., and Strand, R, G. 1960, Geologic Map of California, Ukiah Sheet, California Division of Mines and Geology.

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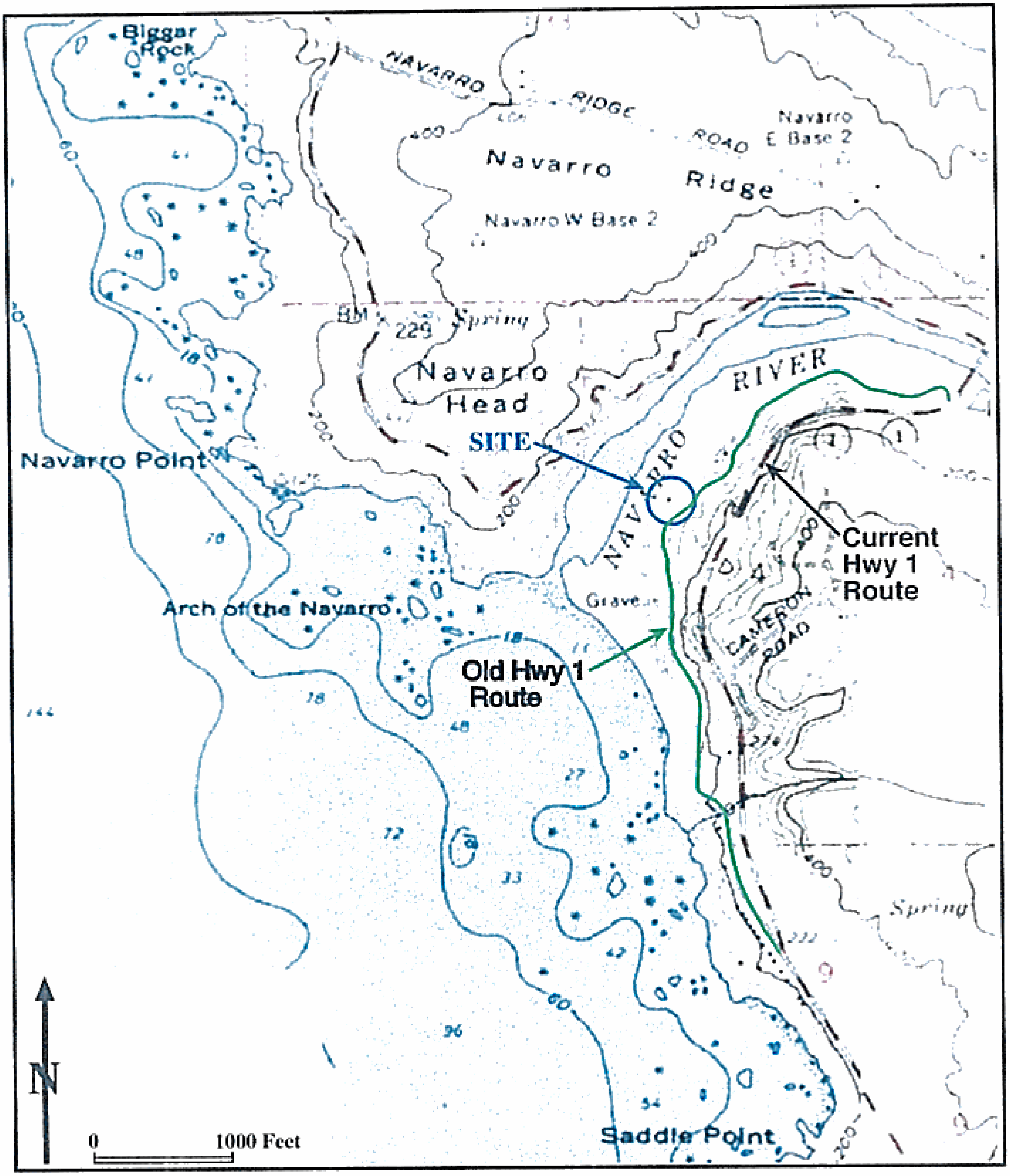

JOYCE ASSOCIATES

USGS Topographic Map
Navarro-by-the-Sea Center
Mendocino County, California

PLATE
1