# CITY OF WHEELER, OREGON

775 Nehalem Blvd, P.O. Box 177, Wheeler, OR 97147
Telephone: (503) 368-5767 / Fax: (503) 368-4273
Website: www.ci.wheeler.or.us / Email: cityofwheeler@nehalemtel.net

## LAND USE APPLICATION

Land Use Application # ~~2019.10 DR~~ 2019.10 DR 2019.10 CU

Property Owner: BOTT'S MARSH LLC Phone: 503.738.7282
Mailing Address: P.O. BOX 1161, SEASIDE. OREGON 97138
Applicant: KEN ULBRICHT Phone: 503.738.7282
Mailing Address: P.O. BOX 1161. SEASIDE. OREGON 97138
Email Contact: KENUL @ PACIFIER

Application Type(s): Check all that apply: * requires additional * information identified below

per email request 8.28.19

X Design Review (See: Section 11.050)
X Variance (See: Article 14)
X Conditional Use (See: Article 15)
___ Zone Boundary Change from ____ to: _____
___ Text Amendment (see attached verbiage)
___ Miscellaneous Review: ______
___ Lot Line Adjustment *
___ Partition *
___ Subdivision *
___ Planned Development *
___ Cluster Development *
X Consolidated Review Requested
___ Appeal of Decision #______
___ Floodplain Development Permit*

PROPERTY DESCRIPTION:
Site Address: NONE Present Use: VACANT LAND
Requested Use: HOTEL & COMMERCIAL BUILDING
Land Use Zone: WRC
Property Size: 4 AC. ☐ square feet / ☐ acres
Access: HIGHWAY 101
☐ Wetlands; ☐ Flood Zone** ______ BFE: ______; ☐ Waterway: ______
Tax Map: Township: ______ Range ______ Section ______ Tax Lot(s) ______
Survey: ☒ Yes; Recorded: ☐ Yes #(s): ______
Legal Description: SEE ATTACHED LEGAL DESCRIPTION
*(Subdivision Name, Block, Lot Number(s) / Partition and Lot Number(s) / Other Description)*

Provide three copies of the following information as necessary to depict the proposed use:
Draw plans to scale and include a north arrow. Information not listed may be required. In a Special Flood Hazard Area, a Floodplain Development Permit is required.

✓ Site Plan
___ Grading Plan
___ Utilities (Water/Sewer/Access)
✓ Property Survey
___ Lot Corner Elevations
✓ Geologic Site Investigation
___ Erosion Control Plan
___ Revegetation Plan
___ Civil Engineering
___ Agency Approvals*
✓ Title Report *
___ Phasing *
___ Storm Water Drainage Plan
✓ Construction Elevations
___ Tentative Plat *
___ Final Plat *
___ Open Space *
✓ Flood Elevation Certificate*

Property Owner(s) Signature: [signature] Date: 8.28.2019

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## FLOODPLAIN DEVELOPMENT PERMIT

OFFICE USE (Year, #): # W-_______-_______

A Floodplain Development Permit is required for all development in the Special Flood Hazard Area (SFHA) Zone A, AE, A1-A30, AH, or AO as identified on the FEMA Flood Insurance Rate Map. As property owner you are making application for a permit in a designated floodplain area and by signing this application you agree that all work shall be done in accordance with the requirements of the Wheeler Zoning Ordinance Article 9 Flood Hazard Overlay Zone and consistent with all other applicable City, State and Federal regulations. The work to be performed shall be described below and in appropriate attachments. This application does not create liability on the part of the City of Wheeler or any officer, or employee thereof for any flood damage that results from reliance on this application or any administrative decision made lawfully thereunder. These documents will be permanently retained by the City.

### PROPERTY OWNER(S)

Legally Recorder Property Owner(s) BETT'S MARSH LLC Phone 503-738-7282
Mailing Address P.O. BOX 1161 City SEASIDE State OR ZIP 97138
Property Owner Signature: ______________________ Date: ______________

### CONTRACTOR / INSTALLER

Building Contractor HAZEN INC. CCB # 210020 Phone 503-717-3811
Manuf. Home Installer NONE MDI # —— Phone ——
☒ Mail permit to: P.O. BOX 1161. SEASIDE, OR 97138

### LOCATION INFORMATION

Physical Address NONE ☐ Recorded Survey #______________
Zone(s) WRC; Lot Size: Dimensions, Area: 4.85 AC. square feet / acres
Legal Description SEE ATTACHED LEGAL DESCRIPTION
Township 2N Range 10W Section 2 Tax lot(s) 4600, 4700, 4200-S1, 4800

### A. DESCRIPTION OF WORK (Complete for all work)

1. Describe the Proposed Development:
   ☒ New Building ☐ Manufactured Home ☐ Improvement to Existing Building ☐ Filling ☐ Other
2. Size and Location of Proposed Development: Attach a Site Plan Drawn to Measurable Scale
3. In what Special Flood Hazard Area Zone(s) is the proposed development located? NONE
4. Identify the FEMA Flood Insurance Rate Map Panel number and revision date.
   Panel # 41057C0209F Date SEPTEMBER 28, 2018
5. Will other local, State or Federal permits be obtained? ☐ Yes ☒ No
   Type ______________________________
6. Is the proposed development in an identified floodway? ☐ Yes ☒ No
7. If yes to #6, is a "No Rise Certification" with supporting data attached? ☐ Yes ☐ No

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| B. | Complete for New Structures and Building Site |
|---|---|

1. Base Flood Elevation at the Site: 10 feet ☐ NGVD 29 ☒ NAVD 88
2. Required lowest flood elevation (including basement): ______ feet ☐ NGVD 29 ☐ NAVD 88
3. If the cost of the proposed construction equals or exceeds 50 percent of the market value of the structure, then the substantial improvement provisions shall apply.
4. Number of flood openings (vents) ______ and enclosed area ______ sq. feet below BFE.

| C. | Complete for Alterations, Additions, or Improvements to Existing Structures |
|---|---|

1. What is the estimated market value of the existing structure? $ ______
2. What is the cost of the proposed construction? $ ______
3. If the cost of the proposed construction equals or exceeds 50 percent of the market value of the structure, then the substantial improvement provisions shall apply.

| D. | Complete for Non-Residential Floodproofed Construction: |
|---|---|

1. Type of floodproofing method: N/A
2. The required floodproofing elevation is: ______ feet ☐ NGVD 29 ☐ NAVD 88
3. Floodproofing certification by a registered engineer is attached: ☐ Yes ☐ No

| E. | Complete for Subdivisions or Planned Unit Development: |
|---|---|

1. Will the subdivision or other development contain 50 lots or 5 acres? ☐ Yes ☐ No
2. If yes, does the plat or proposal clearly identify base flood elevations? ☐ Yes ☐ No
3. Are the 100 Year Floodplain and Floodway delineated on the site plan? ☐ Yes ☐ No

| THIS SECTION FOR ADMINISTRATION USE ONLY |
|---|

1. ☐ Permit Approved date ☐ Permit denied (Findings of Fact attached)
2. Elevation Certificate attached ☐ Yes ☐ No
3. As-Built lowest floor elevation: ______ feet ☐ NGVD 29 ☐ NAVD 88
4. Work Inspected by: ______________________ Date: ______
5. Local Administrator: ______________________ Date: ______

CONDITIONS OF APPROVAL ☐ See attached report with conditions dated ______

1. When construction is complete, prior to occupancy, submit an as-built elevation certificate.

# EXHIBIT A
# LEGAL DESCRIPTION

Parcel No. 1:
Starting at the quarter Section corner between Section 35, Township 3 North, Range 10 West of the Willamette Meridian and Section 2, Township 2 North, Range 10 West of the Willamette Meridian, in Tillamook County, Oregon; thence South 55° 35′ West 1791.2 feet to the Northwest corner of Rowe's Addition to Wheeler; thence South 83° 33′ West 344 feet to the place of beginning; thence North 88° 48′ West 100 feet; thence South 5° 54′ West 300.9 feet; thence South 88° 48′ East 228.4 feet; thence North 21° 37′ East 320 feet; thence North 88° 48′ West 215.4 feet to the place of beginning.

Parcel No. 2:
Beginning at a point North 13° 53′ West 223.8 feet from the initial point of Rowe's Addition to Wheeler, in Section 2, Township 2 North, Range 10 West of the Willamette Meridian, in Tillamook County, Oregon; and running thence South 21° 37′ West 216.9 feet to the Southerly line of the tract deeded to the Port of Nehalem by Deed recorded in Book 44, Pages 449-450 of Records of Deeds of Tillamook County, for the initial point of the tract hereby conveyed; thence North 88° 48′ West to a point 60 feet North of the Northwest corner of a tract of land conveyed to the Grantee herein by Deed recorded at Page 431, Book 44, of the Records of Deeds of said Tillamook County; thence South 60 feet to said Northwest corner of said tract; thence South 88° 48′ East 315.4 feet, more of less, to the West line of right of way of the Southern Pacific Company; thence North 21° 37′ East on the line of said right of way to a point South 88° 48′ East from the initial point of the tract hereby conveyed; thence North 88° 48′ West 30 feet to the said initial point.

Parcel No. 3:
Beginning at a point on the Westerly boundary of the Southern Pacific Railway, said point being 307.92 feet South, 228.72 feet West of the Initial Point of Rowe's Addition to the Town of Wheeler, Section 2, Township 2 North, Range 10 West of the Willamette Meridian, in Tillamook County, Oregon; thence South 21° 37′ West 75.00 feet; thence North 80° 27′ West to the low water line of the Nehalem Bay; thence following the low water line upstream to a point which is North 80° 27′ West from the place of beginning; thence South 80° 27′ East to the place of beginning.
EXCEPTING THEREFROM that portion of said premises lying in Tract 1 above.

Parcel No. 4:
Commencing at a point which is South 21° 37′ West 100 feet from a point which is North 13° 53′ West 223.8 feet from the initial point of Rowe's Addition to Wheeler; and thence North 88° 48′ West 150 feet; thence North 0° 02′ West 162.3 feet; thence North 68° 32′ West 109.8 feet; thence West to the line of low water in Nehalem Bay; thence Southerly along the line of low water to a point which is 60 feet North of a point which is North 88° 48′ West 100 feet from a point which is South 83° 33′ West 344 feet from the Initial Point of Rowe's Addition to Wheeler; thence South 88° 48′ East to a point 30 feet West of the West line of the right of way of the Southern Pacific Company; thence North 21° 37′ East on a line parallel to and 30 feet from said West line of said right of way to the point of beginning.

Parcel No. 5:
Beginning at a point North 3° 55′ East 427.4 feet from the Initial Point of Rowe's Addition To Wheeler; thence South 68° 23′ East 30 feet to a point on the Westerly right of way line of P.R. & N. Company, now Southern Pacific Company, and the true point of beginning of the tract to be described; thence North 21° 37′ East along said Westerly right of way line 395 feet to a point 10 feet Southerly from the foot of the dike as it existed on September 8, 1945; thence parallel to said dike and 10 feet distant from the foot thereof North 45° 09′ West 318 feet; thence North 88° 16′ West 319 feet to a mean high water line of the Nehalem River; thence continuing North 88° 16′ West to mean low water of said river; thence Southerly along said mean low water line 215 feet to a point North 68° 23′ West from the Northerly

corner of the tract conveyed to Wheeler Manufacturing Company by deed recorded in Book 43, page 234, Deed Records; thence South 68° 33' East 88 feet to the Northerly corner of said tract; thence South 68° 23' East along the North line of said tract 450.1 feet to the Northeasterly corner of said tract; thence South 21° 37' West 200 feet to the Southeasterly corner of said tract; thence South 68° 23' East 30 feet to the true point of beginning.

Parcel No. 6:
Beginning at a point North 3° 55' East 427.4 feet from the Initial Point of Rowe's Addition To The Town Of Wheeler, as platted and filed; and thence North 21° 37' East a distance of 200 feet; thence North 68° 23' West a distance of 450.1 feet, more or less, to the low waterline of the Nehalem River; thence along said low water line South 2° 24' West a distance of 218.9 feet; thence South 68° 23' East a distance of 361 feet to the place of beginning, all situated in Tillamook County, Oregon.
EXCEPTING from said lands a strip of land 30 feet in width immediately adjoining on the Westerly side of and parallel with the right of way of the Pacific Railway and Navigation Company across Lot 3, Section 2, Township 2 North, Range 10 West of the Willamette Meridian.

Parcel No. 7:
Beginning at a point North 13° 53' West 223.8 feet from the Initial Point of Rowe's Addition To The Town Of Wheeler; and thence North 21° 37' East 225 feet, more or less, to the Southerly line of the property described in deed recorded in Book 31, Page 571, of the Records of Deeds of Tillamook County, Oregon; thence South 68° 23' East 30 feet, more or less, to the right of way of the Pacific Railway and Navigation Company for its railroad, now owned by the Southern Pacific Company; thence Southerly along the line of said right of way a distance of 325 feet; thence North 68°23' West 30 feet to a point 100 feet distant and South 21°37' West from the beginning point herein first described; thence North 21°37' East 100 feet to the place of beginning.

Parcel No. 8:
Beginning at a point North 13°53' West 223.8 feet from the Initial Point of Rowe's Addition to the Town of Wheeler; thence North 21°37' East a distance of 225 feet; thence North 68°23' West a distance of 361 feet, more or less to the low water line of the Nehalem River; thence Southerly along the said low water line to a point North 68°23' West of the point of beginning; thence South 68°23' East a distance of 310.5 feet to the point of beginning.
ALSO: a tract of land beginning at a point North 13°53' West 223.8 feet from the Initial Point of said Rowe's Addition to the Town of Wheeler; and thence South 21°37' West 100 feet; thence North 88°48' West 150 feet; thence North 0°02' West 162.3 feet; thence South 68°28' East approximately 200 feet to the point of beginning, all in Section 2, Township 2 North, Range 10 West of the Willamette Meridian, Tillamook County, Oregon.

Parcel No. 9:
All that real property North of and adjacent to the following described new property line: South line of Smith property and North line of Werschkull property, located in Section 2, Township 2 North, Range 10 West of the Willamette Meridian, and lying Westward of the Southern Pacific Railroad right of way, will be a line which begins at a point marked by a 3/4" iron lag bold and located South 383.46 feet and West 289.93 feet from the Initial Point of Rowe's Addition to Wheeler and running from said point South 80°27' East 30 feet, more or less to the Westerly right of way line of Southern Pacific Railroad Co. and running from the same point North 80°27' West to Nehalem Bay.

Parcel No. 10:
A tract of land situated in Section 2, Township 2 North, Range 10 West, Willamette Meridian, Tillamook County, Oregon, as more particularly described as follows: Commencing at the Initial Point of Rowe's Addition to Wheeler; thence North 13°53' West 223.8 feet; thence South 21°37' West 100 feet to the point of beginning of the tract herein described; thence South 21°37' West 116.9 feet; thence South 88°48' East 32 feet to the West line of the Southern Pacific Railroad Company right of way; thence along said Southern Pacific Co. right of way line North 21°37' East 105.7 feet; thence North 68°23' West 30 feet to the point of beginning of the tract herein described.

Parcel No. 11:
The Southeast quarter of the Southwest quarter of Section 35, Township 3 North, Range 10 West of the Willamette Meridian, Tillamook County, Oregon, lying West of the center of U.S. Highway 101 and EXCEPTING that portion within the right of way of the Southern Pacific Railroad ALSO EXCEPTING Lot 5 in Section 35, Township 3 North, Range 10 West of the Willamette Meridian, and the tidelands fronting and abutting thereon.

Parcel No. 12:
Beginning at a point on the North line of Section 2, Township 2 North, Range 10 West of the Willamette Meridian, where the said line intersects the Westerly right of way line of the Southern Pacific Railroad; thence South 21°37′ West along the said railway line of said railroad to the most Easterly corner of that tract of land conveyed to 3. A. Lewis Shingle Co., a corporation by deed recorded September 6, 1955 in Book 149, at Page 223, Deed Records; thence North 45°09′ West 318 feet along the Northerly line of said Shingle Company Tract; thence continuing North 88°16′ West 319 feet along the said Northerly line of said Shingle Company Tract to the Westerly line of that tract of land conveyed to H.T. Botts by deed recorded August 6, 1927 in Book 58 at Page 10, Deed Records; thence North 7°30′ West along the said Westerly line of said Botts Tract to the North line of said section; thence East along the said section line to the point of beginning.

# BOTT'S MARSH LLC – CONDITIONAL USE APPLICATION

## Executive Summary

Bott's Marsh LLC is proposing to build a commercial development consisting of two separate buildings;

BUILDING I - Retail/wholesale fish and shellfish sales, with second floor residential apartments, and

BUILDING II - 45 room Hotel.

The applicant is requesting two conditional uses and one variance.

**Conditional Use No.1**

Residential use in conjunction with a permitted or conditional use where the street level shall be maintained as a commercial unit, in an ARTICLE 2, WRC ZONE – WATER RELATED-COMMERCIAL.

For Build No. 1- Retail/wholesale fish and shellfish sales, with second floor residential apartments, the retail/wholesale fish and shellfish sales are an out-right zone use (Section 2.010, Item #7). The second-floor residential apartments are a conditional use (Section 2.030, Item #12).

The residential use is proposed to be four rental apartments used for employee housing.

The first floor will be maintained as a commercial unit, which would qualify the second floor to be residential use as a conditional use.

**Conditional Use No.2.**

Hotel and Motel allowed as a conditional use in a WRC ZONE – WATER RELATED-COMMERCIAL.

Applicant believes that the hotel's design and site plan encompass the City of Wheeler's Priorities and Recommendations for Action outlined in their City of Wheeler Vision Report.

Applicant believes that the hotel's design and site plan encompass the City of Wheeler's Waterfront Development Plan and Transportation System Plan.

Applicant believes most of the out-right zones uses allowed in the WRC Zone are either outdated or obsolete. Commercial fishing on the Nehalem river is no longer allowed and commercial marine activities on the river are minimal at best. Dredging of the Nehalem River would be a challenge and would most likely fail any cost vs benefit rational.

Allowing a hotel on the property is consistent with the Vision Plan and provides an additional room tax revenue to the city by increased room tax dollars.

**SITE ANALYSIS DATA**

SQUARE FOOTAGE INFORMATION

| | Total Sqaure Footage | Building No. I Retail/Wholesale | Building No. II Hotel | Parking | Roads | Landscaping & Others |
|---|---|---|---|---|---|---|
| Sq. Footage | 111,078 | 6,907 | 15,604 | 17,424 | 10,260 | 60,883 |
| Percenatge | 100.00% | 6.22% | 14.05% | 15.69% | 9.24% | 54.81% |

PARKING REQUIREMENTS

| BUILDING No. I | Sq. ft. / eemployee | Setion 11.090 Requirements | | Number of Parking Sp. |
|---|---|---|---|---|
| Retail/Wholesale | | | | |
| Retail Area | 3000 | Sec 11.090-5(f) | 200 | 15 |
| # of employees | 8 employees | Sec 11.090-5(f) | 1 for 2 emp. | 4 |
| Storage/ Wholesale Area | 3907 | Sec 11.090-5(l) | 1000 | 4 |
| # of employees | 1 employee | Sec 11.090-5(l) | 1 for 1 emp. | 1 |
| | | | | |
| Second Floor-apartments | 4 units | Sec 11.090-5(f) | 1 for each unit | 4 |

| BUILDING No. II | | | | |
|---|---|---|---|---|
| Hotel | 44 Rooms | Sec 11.090-5(f) | 1 for each unit | 44 |
| # of employees | 5 employees | Sec 11.090-5(f) | 1 for 2 emp. | 3 |

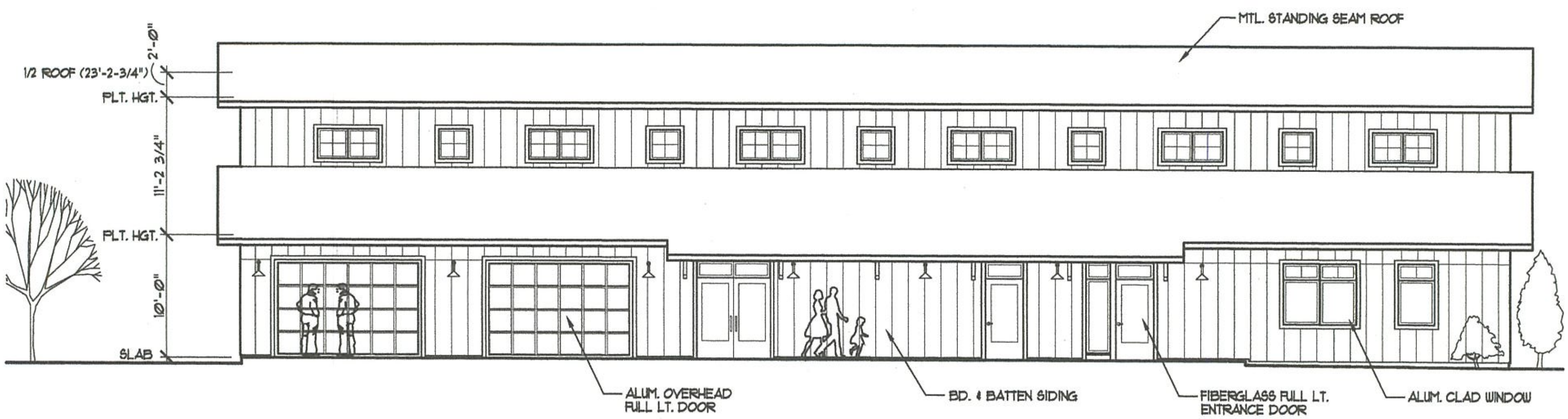
TOTAL SPACES NEEDED **75**

Available Spaces

| | |
|---|---|
| In front of Hotel | 39 |
| Located on Road | 28 |
| Available in service areas | 8 |
| TOTAL SPACES AVAILABLE | **75** |

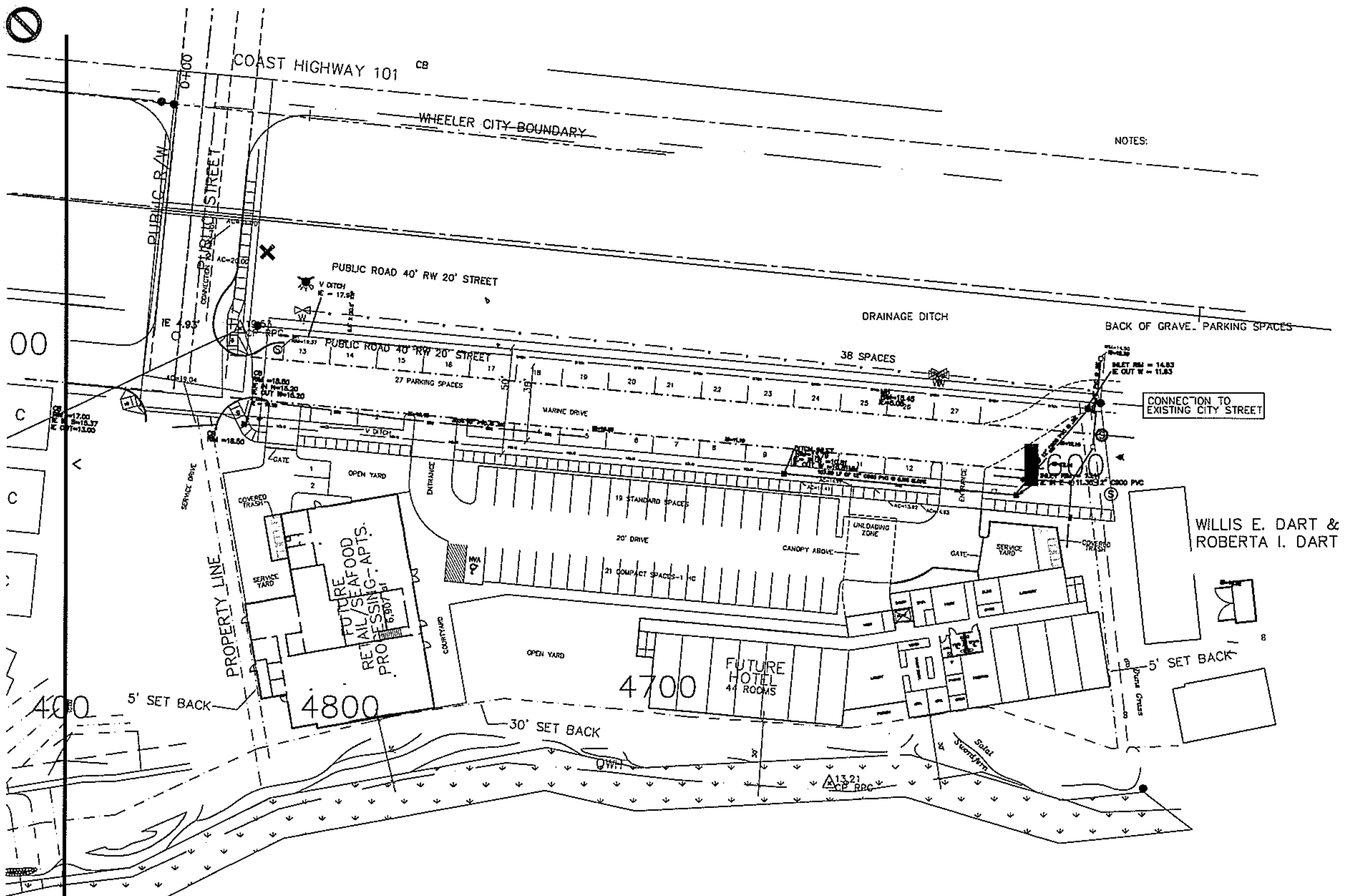

2 WEST ELEVATION

1/8"=1'-0"




3 NORTH ELEVATION

1/8"=1'-0"




1 SITE PLAN
1" = 20'-0"



THOMAS JOHNSON ARCHITECT
821 NW FLANDERS STREET, SUITE 210
PORTLAND, OREGON 97209

REGISTERED ARCHITECT
STATE OF OREGON

Owner:
NEHALEM BAY VILLAGE LLC
ATTN: KEN ULBRICHT
620 HOLLADAY STREET
SUITE #4
SEASIDE, OREGON 97138

Project:
BOTTS MARSH COMMUNITY BUILDING
WHEELER, OREGON

Sheet Title:

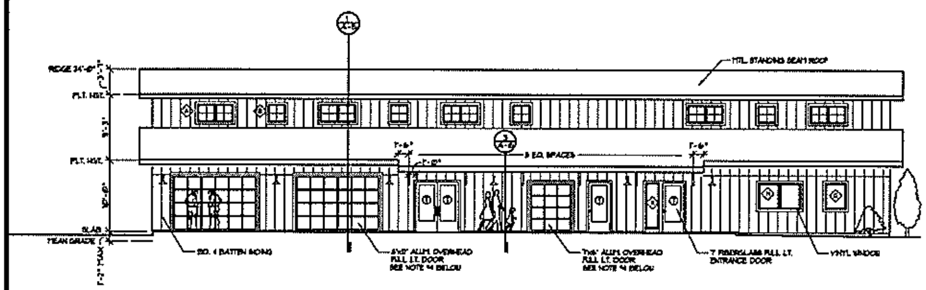
SHELL PERMIT SET

Date: AUGUST 22, 2019
Revisions:

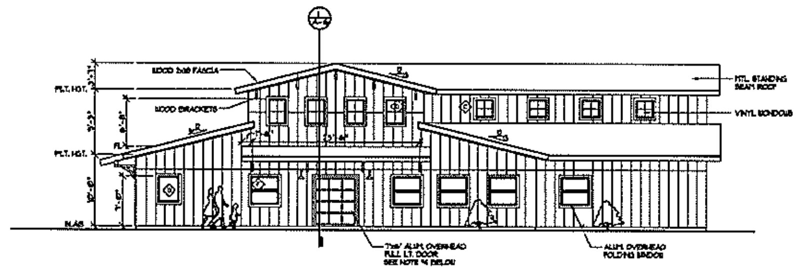
Drawn By: TJ
Checked By: TJ

Project Number: 2016-05
Sheet Number:

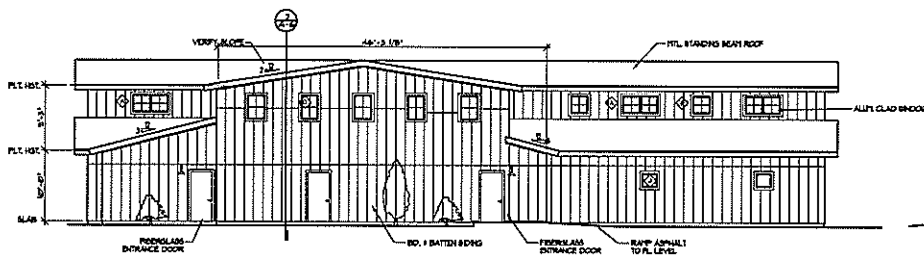
A-1 of 6



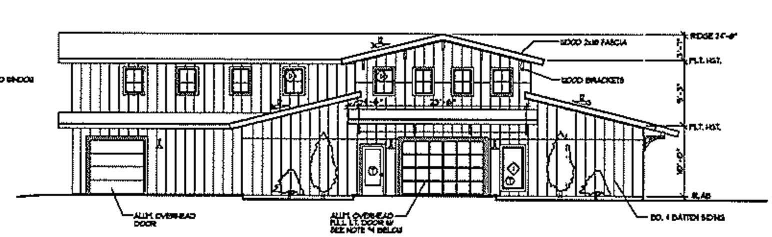

① SOUTH ELEVATION
1/8"=1'-0"




② WEST ELEVATION
1/8"=1'-0"




③ NORTH ELEVATION
1/8"=1'-0"




④ EAST ELEVATION
1/8"=1'-0"

**WINDOW SCHEDULE** (WIDTH x HGT.)

- (A) (TRU 30" x 30" VINYL FIXED
- (B) 30" x 30" VINYL FIXED
- (C) 30" x 36" VINYL CASEMENT
- (D) 30" x 42" VINYL CASEMENT
- (E) NOT USED
- (F) 48" x 48" ALUM. UPWARD FOLDING
- (G) 36" x 48" VINYL FIXED
- (H) (TRU 36" x 48" VINYL FIXED
- (I) 36" x "Q" VINYL FIXED
- (J) 24" x 24" VINYL AWNING
- (K) DOOR SIDELITE-TEMPERED

**WINDOW NOTES:**

1. VERIFY ALL WINDOW SIZES BEFORE ORDERING WINDOWS
2. PROVIDE SAFETY GLAZING PER CURRENT OREGON 2014 STRUCTURAL SPECIALITY CODE
3. (T) INDICATES TEMPERED GLAZING
4. PROVIDE SAFETY GLAZING PER CURRENT OREGON 2014 STRUCTURAL SPECIALITY CODE @ OVERHEAD DOORS



THOMAS JOHNSON ARCHITECT
421 NW FLANDERS STREET, SUITE 210
PORTLAND, OREGON 97209
503/679-3047
tjarchitect@questoffice.net

REGISTERED ARCHITECT STATE OF OREGON

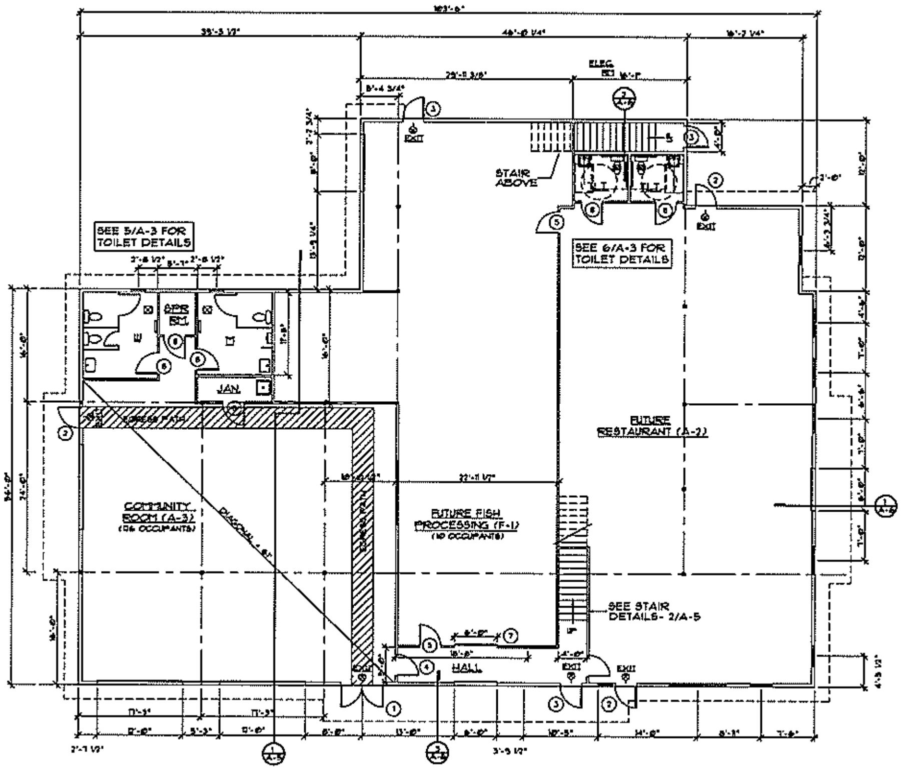
Owner:
NEHALEM BAY VILLAGE LLC
ATTN: KEN ULBRICHT
620 HOLLADAY STREET
SUITE #4
SEASIDE, OREGON 97138
503/738-7283

Project:
BOTTS MARSH COMMUNITY BUILDING
WHEELER, OREGON

Sheet Title:
ELEVATIONS

SHELL PERMIT SET

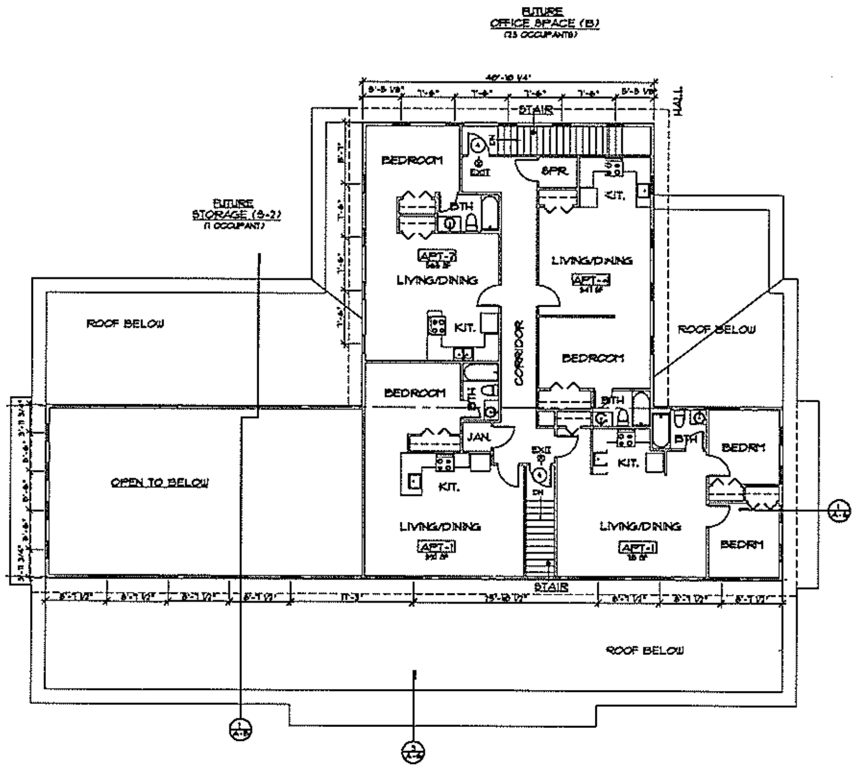
| | |
|---|---|
| Date: | AUGUST 22, 2019 |
| Revisions: | △ |
| Drawn By: | TJ |
| Checked By: | TJ |
| Project Number: | 2016-09 |
| Sheet Number: | A-4 of 6 |




## 1 FIRST FLOOR PLAN

1/8" = 1'-0"

TOTAL FIRST FLOOR




## 2 SECOND FLOOR PLAN

1/8" = 1'-0"

### DOOR SCHEDULE

| SYMBOL | SIZE | DESCRIPTION | FRAME | HARDWARE | DOOR FIN | REMARKS/NOTES |
|---|---|---|---|---|---|---|
| 1 | PR 3'-0" x 7'-0" | FULL LT ENTRY DOORS | WOOD | 1 | NAT FIR | 1, 3, 5, 6, 9 |
| 2 | 3'-0" x 7'-0" | FULL LT ENTRY DOOR | WOOD | 2 | NAT FIR | 1, 3, 5 |
| 3 | 3'-0" x 7'-0" | FULL LT ENTRY DOOR | WOOD | 3 | NAT FIR | 1, 3, 4, 5, 8 |
| 4 | 3'-0" x 7'-0" | WOOD FLUSH DOOR | WOOD | 4 | NAT FIR | 1, 3, 4, 5, 7 |
| 5 | 3'-0" x 7'-0" | WOOD FLUSH DOOR | WOOD | 5 | NAT FIR | 1, 3, 4, 5, 7 |
| 6 | 3'-0" x 7'-0" | WOOD FLUSH DOOR | WOOD | 2 | NAT FIR | 1, 3, 4, 5, 7 |
| 7 | 6'-0" x 7'-0" | ALUM. OVERHEAD DOOR | WOOD | | | |
| | | | | | | |

### HARDWARE GROUPS:

**GROUP 1**

- 6 PAIR BUTTS
- PANIC HARDWARE
- WEATHERSTRIPPING
- 2 CLOSERS
- EXTERIOR PULLS
- THRESHOLD

**GROUP 2**

- 1-1/2 PAIR BUTTS
- PANIC HARDWARE
- WEATHERSTRIPPING
- 1 CLOSER
- EXTERIOR PULL
- THRESHOLD

**GROUP 3**

- 1-1/2 PAIR BUTTS
- ENTRY HARDWARE
- WEATHERSTRIPPING
- 1 CLOSER
- THRESHOLD

**GROUP 4**

- 1-1/2 PAIR BUTTS
- LOCKSET SET

**GROUP 5**

- 1-1/2 PAIR BUTTS
- LOCKSET SET
- DEADBOLT

**GROUP 6**

- 1-1/2 PAIR BUTTS
- PULL-PUSH BARS
- 1 CLOSER

### DOOR NOTES:

1. EXTERIOR DOORS TO BE COMMERCIAL GRADE HEAVY DUTY FIBERGLASS DOORS.
2. INTERIOR DOORS TO BE STAINED WOOD (SIMPSON OE) SOLID CORE DOORS, RATED AS NOTED
3. ALL EXTERIOR DOORS TO HAVE SS. SILL PANS W/ END DAMS SET IN MASTIC
4. EXTERIOR DOORS W/ LITES TO BE TEMPERED GLAZING
5. ALL DOOR HARDWARE TO BE SCHLAGE, VON DUPRIN O.E.
6. PROVIDE MATCHING TEMPERED GLASS SIDELITES IF SHOWN
7. ALL DOORS AND HARDWARE TO BE ACCESSIBLE

### ROOM FINISH SCHEDULE

| ROOM | FLOOR | | | | BASE | | WALLS | | CLG | | REMARKS |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | CONCRETE | WOOD | WOOD | TILE | WOOD | TILE | PAINT | NONE | PAINT | NONE | |
| COMMUNITY ROOM | • | | | | • | | • | | | • | 1, 2, 5 |
| TOILETS | • | | | | | • | • | | • | | 1, 2, 3, 5 |
| HALL | • | | | | • | | • | | • | | 1, 2 |
| STAIR | | • | | | • | | • | | • | | 1, 4 |
| JANITOR RM | • | | | | • | | • | | • | | 1, 2 |
| ELECTRICAL RM | • | | | | | | | | | | 1, 2 |
| FUTURE FISH PROCESSING | | | | | | | | | | | UNFINISHED, 6 |
| FUTURE RESTAURANT | | | | | | | | | | | UNFINISHED, 6 |
| FUTURE STOR. & OFFICE | | | | | | | | | | | UNFINISHED, 6 |

### FINISH NOTES

1. ALL INTERIOR TRIM TO BE PAINTED WOOD
2. CEILING TO BE OPEN WHERE SHOWN, OTHERWISE GYP. BD.
3. WALL TILE TO MIN. 4'-0" AFF
4. FUTURE RUBBER TREADS & RISERS (W/ TI)
5. CONCRETE TO BE STAINED & SEALED
6. INSTALL WALL INSULATION ONLY

### GENERAL PLAN NOTES:

1. ALL DIMENSIONS ARE TO THE FACE OF STUD UNLESS NOTED OTHERWISE.
2. CONTRACTOR TO VERIFY ALL DIMENSIONS BEFORE BEGINNING WORK OR ORDERING MATERIALS.
3. CONTRACTOR TO COORDINATE ALL TRADES AND NOTIFY THE ARCHITECT OF ANY CONFLICTS THAT MIGHT ARISE.
4. ALL NEW INTERIOR WALLS TO BE 2x4 WOOD STUDS @ 16" OC W/ 5/8" GYP. BD.
5. CONTRACTOR TO VERIFY THE SIZE OF THE SPRINKLER AND ELECTRICAL ROOMS BEFORE FRAMING WALLS & POURING SLAB
6. ⊗- INSTALL EXIT SIGNS WHERE INDICATED
7. WALLS TO BE LEFT OPEN (NO GYP. BD.) IN SPACES NOTED AS FUTURE. INSTALL BATT INSULATION IN EXTERIOR WALLS.
8. CEILINGS TO BE LEFT OPEN (NO GYP. BD.) IN SPACES NOTED AS FUTURE.

### VENTILATION NOTES:

⊠ PROVIDE EXHAUST FANS IN ALL TOILETS
TOILETS W/ INTERMITTENT USE: MIN. 70 CFM
EXHAUST FANS TO BE CONTROLLED BY TIMING SWITCHES.

### EMERGENCY LIGHTING NOTES:

1. PROVIDE A MINIMUM OF 1 FOOT CANDLE ILLUMINATION FOR EMERGENCY LIGHTING (W/BATTERY BACK-UP) ALONG EGRESS PATH IN COMMUNITY ROOM, TOILET ROOMS, HALLS & STAIRWAY.
2. EMERGENCY LIGHTING FOR ALL OTHER AREAS TO BE INCLUDED W/ TI PERMITS WHEN PLANS HAVE BEEN ESTABLISHED.



THOMAS JOHNSON ARCHITECT
821 NW FLANDERS STREET, SUITE 210
PORTLAND, OREGON 97209
503/679-3047
tjarchitect@questoffice.net

REGISTERED ARCHITECT
STATE OF OREGON
PORTLAND, OREGON

Owner:
NEHALEM BAY VILLAGE LLC
ATTN: KEN DURRICHT
620 HOLLADAY STREET
SUITE #4
SEASIDE, OREGON 97138
503/738-7282

Project:
BOTTS MARSH COMMUNITY BUILDING
WHEELER, OREGON

Sheet Title:
FLOOR PLANS

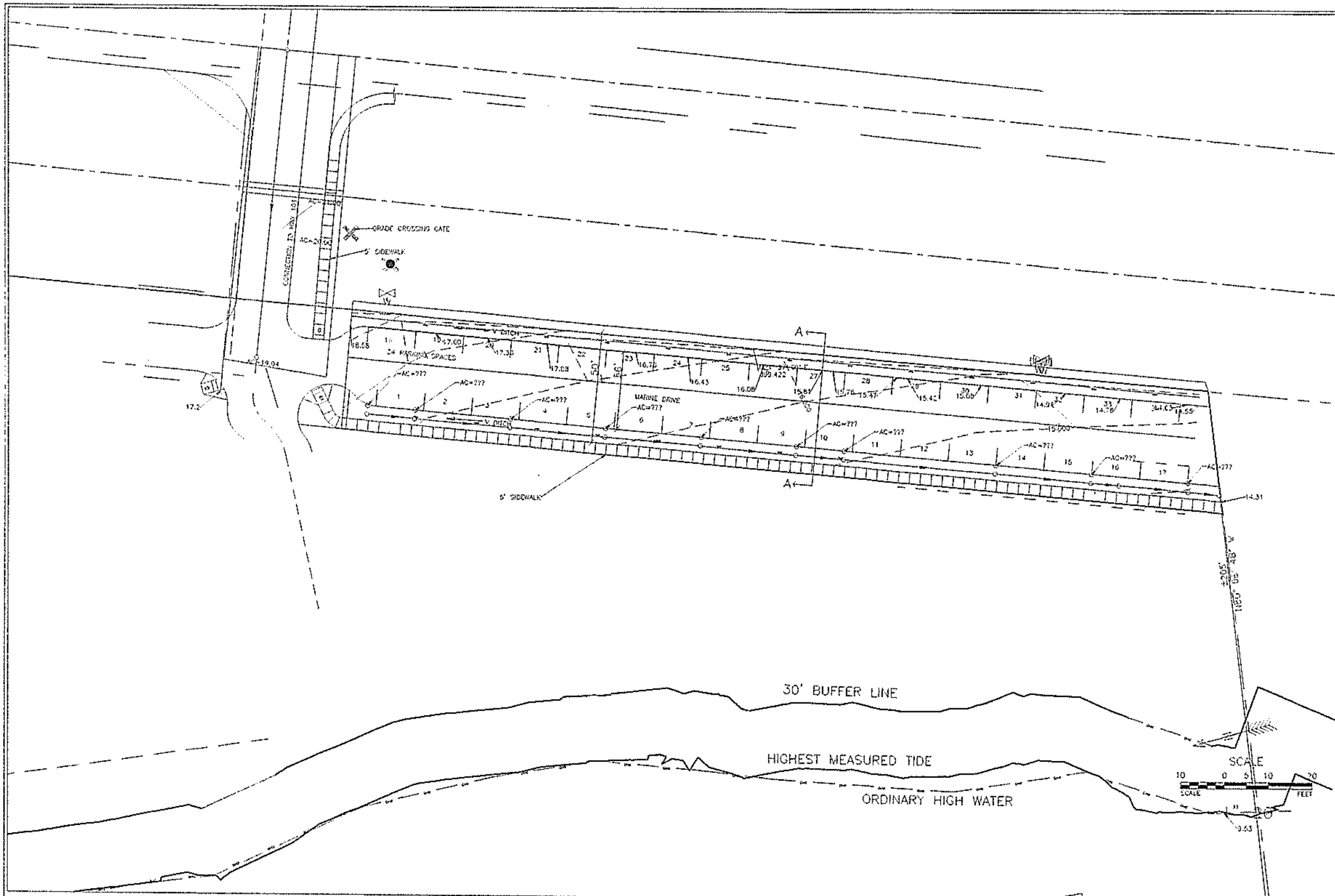
SHELL PERMIT SET

Date: AUGUST 22, 2019
Revisions:
△

Drawn By: TJ
Checked By: TJ

Project Number: 2016-09
Sheet Number:

A-2 of 6

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| GTK Drawn | 4/22/17 Date |
| GTK Checked | 4/22/17 Date |
| REVISIONS | |

REGISTERED PROFESSIONAL ENGINEER 10,053 OREGON FEB. 25, 1979 GREGORY KURAHASHI

EXPIRES 6/30/2019

GRADING PLAN

VILLAGE AT NEHALEM BAY

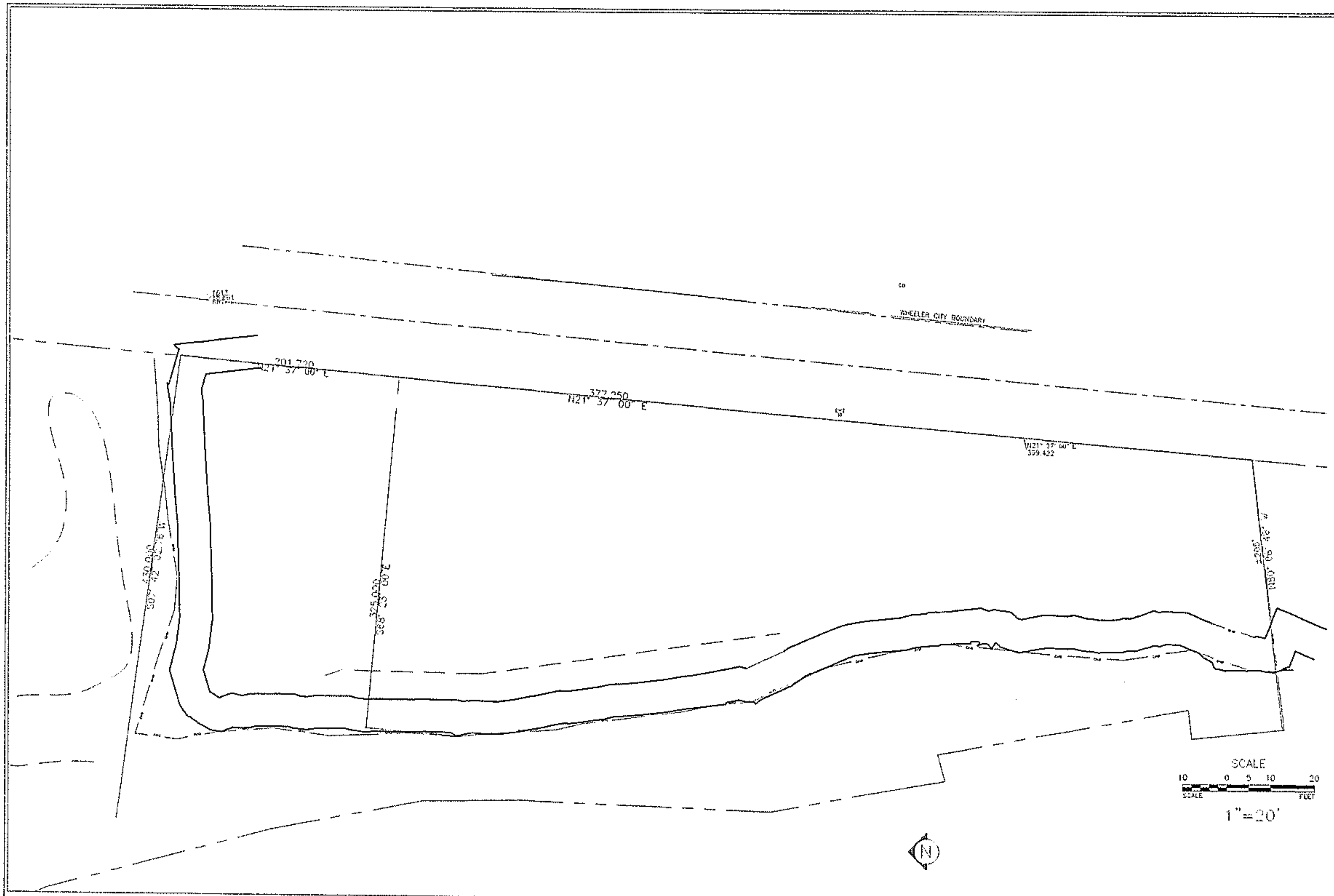
KEN ULBRICHT

620 HOLLADAY STREET SUITE #4
SEASIDE, OREGON 97138

KURAHASHI AND ASSOCIATES COMPANY
4470 SW HALL BLVD STE C
BEAVERTON, OR 97005

C 4.0
Sheet No.

Project No. 2752

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| Checked | Date |

REVISIONS

REGISTERED PROFESSIONAL ENGINEER 10,053 OREGON FEB 22, 1973 GREGORY KURAHASHI

EXPIRES 6/30/2019

SITE PLAN

VILLAGE AT NEHALEM BAY

KEN ULBRICHT

620 HOLLADAY STREET SUITE #4
SEASIDE, OREGON 97138

KURAHASHI AND ASSOCIATES COMPANY
4470 SW HALL BLVD STE C
BEAVERTON, OR 97005

C 2.0
Sheet No.

Project No. 2752

Chinook GeoServices Inc.

September 18, 2006

M.A.N. Developments
6107 Southwest Murray Boulevard, Suite 424
Beaverton, Oregon 97008
Attention: Mr. Mike Nelson

**Subject:** **Geotechnical Engineering Evaluation Report**
**Proposed 92-Unit Townhome Development**
**Marine Drive, Wheeler, Tillamook County, Oregon**
**CGI Report No. 06-035-1**

Dear Mr. Nelson:

Chinook GeoServices, Inc. (CGI) is pleased to submit our Geotechnical Engineering Evaluation Report for the proposed townhome development located in Wheeler, Oregon. This report includes the results of our literature research, field reconnaissance and subsurface exploration, laboratory testing, and geotechnical engineering conclusions and recommendations for the proposed construction, as well as recommendations for general site development.

We appreciate the opportunity to perform this study and look forward to continued participation during the design and construction phases of this project. Please contact Marcy Boyer or Warren Krager in our office at 360-695-8500 if you have any questions or if we may be of further service.

Respectfully submitted,

**CHINOOK GEOSERVICES, INC.**

Marcella M. Boyer, P.E., G.E.
Principal Geotechnical Engineer

R. Warren Krager, R.G., C.E.G.
Principal Engineering Geologist

Distribution: Addressee (1 electronic copy, 4 hard copies)

**GEOTECHNICAL ENGINEERING EVALUATION REPORT**

For the

**Proposed 92-Unit Townhome Development**
**Marine Drive, Wheeler, Tillamook County, Oregon**

Prepared for

**M.A.N. Developments**
**6107 Southwest Murray Boulevard, Suite 424**
**Beaverton, Oregon 97008**

Prepared by


**CHINOOK GEOSERVICES, INC.**
**1508 Broadway Street**
**Vancouver, Washington 98663**
**Telephone (360) 695-8500**
**Fax (360) 695-8510**


**CGI Report No. 06-035-1**

**September 18, 2006**

**Chinook GeoServices, Inc.**

CERTIFIED
OREGON
R. WARREN KRAGER
E957
ENGINEERING GEOLOGIST

Exp 12-31-06


**R. Warren Krager, R.G., C.E.G.**
**Principal Engineering Geologist**


REGISTERED PROFESSIONAL
ENGINEER
16,590
OREGON
JULY 20, 19[illegible]
MARCELLA M. BOYER

9/18/06

EXPIRES 12/31/07


**Marcella M. Boyer, P.E., G.E.**
**Principal Geotechnical Engineer**

## TABLE OF CONTENTS

## 1.0 EXECUTIVE SUMMARY

A subsurface exploration program and a geotechnical engineering evaluation of seismic liquefaction potential and subsurface conditions has been completed for the proposed 92-unit townhome development to be constructed at the north terminus of Marine Drive in Wheeler, Oregon. Nine test pits were excavated across the site to investigate the condition and thickness of the upper fill. To investigate the deeper native soils, two mud rotary borings were drilled—one each at the north and south ends of the site. Selected soil samples from both the borings and test pits were tested in the laboratory to determine index, compressibility, and strength properties of the soils encountered. In general, the subsurface conditions consisted of a thin layer of topsoil (no more than a foot in thickness), overlying 8½ to 17 feet of variable density, organic fill. The fill was underlain by soft/loose, compressible peat, sand, silt and clay to a depth of at least 101 ½ feet in boring B-1 and 33½ feet in boring B-2. We recommend groundwater be considered to be as shallow as 6 to 10 below existing grade, depending upon weather and tidal conditions.

The site has several geotechnical engineering concerns including:

- Soft, compressible fill 8½ to 17 feet in thickness that contains wood, logs and stumps and could settle several inches
- Soft, native peat and organic soils that are extremely compressible and could settle an additional 4 to 8 inches
- Soils that could liquefy and settle up to about 10 to 15 inches and slide laterally up to about 5 to 16 feet if the site is shaken by a major design earthquake
- The site is within the tsunami inundation zone and will be flooded if a major tsunami strikes the coastline

The proposed buildings cannot be supported on conventional shallow foundation systems because they cannot tolerate the excessive settlements caused by the geotechnical engineering concerns outlined above. We understand that the project team has already determined that pile foundation support is not economically viable. Based on these constraints, we recommend the use of either shallow, rigid, reinforced concrete raft or mat foundations supported on lightweight geofoam blocks. The raft or mat foundations will spread the building loads out uniformly across the building pads. The geofoam will replace existing site soils so that the weight of the new buildings will not be "felt" by the underlying soils. Ideally, this would mean that there would not be excessive settlement of the foundations. However, because of the complexity of the subsurface soil conditions, we recommend some excessive settlement (i.e. several inches) be planned for. As such, we recommend the use of flexible utility connections and we recommend the building entrances be constructed at least 1 to 2 feet high to allow for some building settlement. Site preparation of the building pads prior to geofoam block placement will require the removal of organics (wood debris) where more concentrated.

Details related to foundation design, and site preparation and construction considerations are included in subsequent sections of this report. The owner and/or designer should not rely solely on this Executive Summary and must read and evaluate the entire contents of this report prior to using our engineering recommendations to prepare the design/construction documents.

## 2.0 PROJECT INFORMATION

### 2.1 Project Authorization

Chinook GeoServices, Inc. (CGI) has completed a geotechnical engineering evaluation for the proposed townhome development to be constructed at the northern terminus of Marine Drive in Wheeler, Tillamook County, Oregon. Our services were authorized by Mr. Mike Nelson, owner of M.A.N. Developments on July 19, 2006 by signing CGI Proposal No. 06-P045 dated July 7, 2006. Our services were completed in general accordance with our proposal.

### 2.2 Project Description

Our current understanding of the project is based on limited information provided to us by Mr. Nelson including an undated, untitled site development plan. The 5-acre waterfront site will be developed with 17, three-story townhome buildings with a total of 92 living units. Four of the townhome buildings will also have some commercial space. There will also be 3 commercial buildings. The commercial space all appears to be located in the middle of the project. The building footprints will range in size from about 3,500 to 7,000 square feet. A pool—presumably below grade—is shown on the north-central portion of the site development plan. A promenade will be constructed at the west side of the middle of the site.

Below ground construction is not planned to our knowledge with the exception of utilities, and possibly the pool. According to Mr. Nelson, the site will not require any new fill to raise the grade elevation except at the promenade, where up to about 6 inches of fill will be needed. Finish floor of the proposed buildings roughly matches existing grade so no fill will be required under the buildings.

Building loads have not been provided to us. For the purposes of our engineering analyses, we have assumed the buildings will have maximum wall, column and floor loads of 4 kips per linear foot, 75 kips, and 0.150 kips per square foot, respectively. Due to the complexity of this project, it will be imperative that actual building loads be provided to us for review at some point during the design process.

### 2.3 Purpose and Scope of Services

The purpose of this study was to evaluate seismic liquefaction potential and geotechnical engineering subsurface conditions at the site to enable an evaluation of acceptable foundation recommendations for the proposed townhomes. We understand that you have already determined that supporting the proposed buildings on piles is not economically feasible and we have not included this alternative in our scope of services. Our scope of services included advancing 2 borings using mud rotary drilling techniques to investigate the native soil conditions, 9 test pits to investigate the overlying fill conditions, select laboratory testing, engineering analysis and preparation of this report.

## 3.0 SITE AND SUBSURFACE CONDITIONS

### 3.1 Site Description

A street address for the property was not available to us. The site is roughly located at the north terminus of Marine Drive and the west terminus of Hemlock Street. It is bordered by Port of Tillamook Bay Railroad tracks and Highway 101 to the east, Nehalem Bay to the west and north, and a residence and the Wheeler Marina (278 Marine Drive) to the south. The site is relatively level. Several feet of fill has been placed across the site in the past. Apparently, one of the former uses at the site was to transport and store logs. A review of an August 22, 2000 aerial map on the www.terraserver-usa.com website shows that logs were not being stored at that time. The site is also clear of any structures at that time. A July 1, 1985 topographic map from the same website indicates that several structures were located on the property. During our site investigation, we observed buried concrete foundations and concrete slab remnants at the surface of the site.

### 3.2 Geologic Background

According to the 1994 "Geologic Map of the Tillamook Highlands, Northwest Oregon Coast Range," (Open File Report 94-21) prepared by the US Department of the Interior, US Geologic Survey (USGS), the site geology consists of fluvial and estuarine Holocene deposits (deposits within the last 11,000 years). These deposits typically consist of unconsolidated clay, silt and gravel alluvium deposited along rivers and streams and stabilized tidal flat mud, sand and peat in Nehalem and Tillamook Bays. Discounting the upper fill, the site geology encountered in our borings and test pits appears to be consistent with the geologic conditions described above.

### 3.3 Geotechnical Subsurface Investigation

CGI completed 2 mud rotary borings and 9 backhoe test pits to evaluate the subsurface conditions (reference Appendix C for a description of the field investigation procedures). Based on the borings and test pits, the site subsurface consisted of the following soil units:

**TOPSOIL** – Brown, slightly moist silt topsoil was encountered in all exploration locations except boring B-2, which was located in a gravel parking area. The topsoil unit was generally firm and ranged in thickness from about 2 to 12 inches.

**FILL** – Variable fill was encountered immediately below the topsoil layer in all borings and test pits. The fill consisted of varying amounts of clay, silt, sand, gravel and cobbles. Varying degrees of construction debris, organics and wood pieces were also encountered in all test pits (see Photos 4 and 5 in the Photo Log attached for examples of wood pieces encountered). The fill thickness ranged from 8½ to 17 feet. It appears the fill was imported from a number of different sites based on the high variability of the material. N-values and pocket penetrometer readings indicate the fill was not fully compacted, although the general trend was that the upper portion of fill was much firmer and the bottom of the fill was noticeably softer/looser. The fill ranged from very soft to very dense. The moisture condition of the fill ranged from slightly moist near the ground surface to wet near the bottom of the fill. Note that TP-6 did not extend through this fill stratum into the native soil due to practical digging refusal on very dense gravel fill. In TP-8 it appears that we encountered the remnants of a buried building foundation at a depth of about 6 feet (see Photos 7 and 8 attached).

**PEAT** – Test pits TP-4 and TP-5 each encountered a 2-foot thick layer of fibrous peat at a depth of 12 and 11 feet, respectively. The peat was free of soil and appeared to be buried marsh grass, similar to what is observed west and north of the site. Peat was also observed in the native silt and clay soils immediately beneath the fill placed across the site, however, the peat content appeared to be small—10.6 and 17.7 percent in the samples tested (see Photo 6 attached).

**SILT AND CLAY WITH ORGANICS** – All borings and test pits (excluding TP-6 because it did not penetrate through the fill) encountered a soil unit that was predominantly silt and clay immediately beneath the peat layer in TP-4 and TP-5 and beneath the fill stratum of all other test pits and borings. This soil unit consisted of both low and high plasticity clay and silt based on Atterberg limits testing. The top couple of feet of this stratum generally included some wood pieces and moderate peat content. The organic content decreased with depth. It is likely the wood pieces were pushed into the native stratum by the weight of the fill above. In boring B-2, this silt/clay stratum included gravel and trace sand below a depth of 25 feet. Based on Standard Penetration Tests and pocket penetrometer readings, the strength of this unit was generally very soft to medium stiff, except in boring B-2 at the south end of the site where it was very stiff to hard starting at a depth of 25 feet. Moisture content of the samples tested ranged from 28 percent to 79 percent.

**SAND WITH SILT AND TRACE ORGANICS** – Boring B-1 was extended significantly deeper than the other boring and test pits. From a depth of 45 feet to 75 feet below grade, sand with silt and trace organics (shells and wood) was encountered. This stratum was generally wet and very loose to loose.

**SILT WITH TRACE ORGANICS** – From 75 feet to the termination depth of 101½ feet below grade, silt with trace organics was encountered in boring B-1. The stratum was generally very soft to medium stiff in consistency with moisture contents ranging from 47 percent to 57 percent.

The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The boring and test pit logs included in the appendix should be reviewed for specific information at individual exploration locations. These records include soil descriptions, stratifications, penetration resistances (automatic hammer equivalent), locations of the samples, and laboratory test data. The stratifications shown on the exploration logs represent the conditions only at the actual boring and test pit locations. Variations may occur and should be expected between exploration locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. The samples that were not altered by laboratory testing will be retained for 60 days from the date of this report and then will be discarded.

### 3.4 Groundwater Information

Groundwater levels are extremely difficult to accurately measure in mud rotary borings. We were, however, able to observe groundwater levels in the test pits at the time of excavation. Seepage was generally observed in the test pit sidewalls at a depth of 6 feet to 11 feet below existing grade. Standing groundwater was generally observed in the test pits at depths of 8 feet to 16 feet. Because the site soils within the seepage zone were generally cohesive with low permeability, it is probable that if the test pits were allowed to stand open long enough, the standing water would have risen close to the seepage depth. For planning purposes, we recommend the groundwater depth be anticipated to be about 6 to 10 feet below existing grade. Some fluctuations of groundwater levels should be anticipated with changing climatic conditions, tides, and/or changes in surface topography, construction activities and site use.

## 4.0 GEOTECHNICAL EARTHQUAKE ENGINEERING EVALUATION

### 4.1 Seismicity

There are three major types of earthquakes that can occur in Oregon: deep interplate or subduction zone earthquakes, moderately deep intraplate earthquakes and shallow crustal earthquakes. A review of the 1994 USGS map for this area (referenced earlier in this report) indicates the presence of several faults in the area. However, current understanding of the activity of these faults and the potential for an earthquake to occur within the life of the proposed structure is not very well understood. Additionally, there may be yet undiscovered faults in the area capable of generating significant ground motion at the subject site.

The Cascadia Subduction Zone, located approximately 100 kilometers (km), or roughly 60 miles, off the Oregon and Washington coasts, is an immense thrust fault and a potential source of earthquakes large enough to cause significant ground shaking at the subject site and potentially throughout western Oregon and Washington. Research over the last several years has shown that this offshore thrust fault zone has repeatedly produced large earthquakes every 300 to 700 years. Geologic research of ancient Japanese tsunami records along with dendrochronology (tree ring dating techniques) has established that the last large Cascadia Subduction Zone earthquake occurred in January of 1700 AD. Given the date of the last earthquake in 1700 and the average return interval, it is generally considered that Oregon could be impacted by a very large subduction zone earthquake at any time in the next 400 years or so. Although researchers do not agree on the likely magnitude of the next Cascadia Subduction Zone thrust fault earthquake, it is widely believed that an earthquake of moment magnitude (Mw) 8.5 to 9.5 is possible. The duration of strong ground shaking is estimated to be up to about two or three minutes, with minor shaking lasting several minutes. Aftershock earthquakes could continue for weeks or months after the main thrust fault rupture. It is also possible that large areas of the coastline will experience several feet of rapid, permanent subsidence as a result of a large Cascadia Subduction Zone earthquake. Remnants of salt water flooded and killed forests and rooted stumps in the surf are still present after the last Cascadia Subduction Zone earthquake. These geologic features provide evidence that up to 6 or more feet of coseismic subsidence may be possible. This amount of subsidence coupled with sea level rise from global warming will render many currently low lying coastal areas uninhabitable without substantial engineering solutions.

Additional earthquake sources in this region include fault ruptures within the subducting oceanic plates (interplate earthquake) and within the overlying North American continental crustal plate (crustal earthquake). Intraplate earthquakes originate at depths on the order of about 50 km (30 miles) within the remains of the Juan de Fuca Plate. These large earthquakes have occurred with historical frequency in western Washington and to a lesser extent in western Oregon. These earthquakes range up moment magnitude (Mw) 7.5 and have caused widespread damage in the southern Puget Sound and northwest Oregon region in 1949, 1965, 1971, and 2001.

Crustal earthquakes are relatively shallow, occurring within 10 to 20 km (6 to 12 miles) of the ground surface within the North American Plate. Oregon has experienced at least two significant crustal earthquakes in the recent past—the Scotts Mills (Mt. Angel) earthquake (Mw 5.6) on March 25, 1993 and the Klamath Falls earthquake (Mw 5.9) on September 20, 1993.

## 4.2 Seismic Hazards

While we have not performed a detailed site-specific seismic evaluation, we have done a preliminary analysis of seismic hazards which could potentially impact the site. These seismic hazards include:

- Severe ground shaking
- Ground surface subsidence
- Fault rupture
- Liquefaction, dynamic settlement and lateral spread
- Seiche
- Tsunami
- Earthquake-induced landslides

Based on the historical seismic activity in Oregon, it is possible that the proposed development will be impacted by *severe ground shaking* within its useful life. The severity of ground shaking at the site will be determined in part by the source of the earthquake (i.e. Cascadia Subduction Zone versus local shallow crustal). Structural design parameters required to address ground shaking are provided in Section 4.4 below.

The risk of *ground surface subsidence*, as previously discussed in Section 4.1, is possible due to either a Cascadia Subduction Zone earthquake, which causes the entire coastline to drop several feet, or liquefaction.

Because of the lack of historical data on fault activity in the area, *fault rupture* is difficult to predict. There was no evidence of past fault rupture at this site during our subsurface investigation, although the conditions at the site would make it difficult to observe fault rupture.

Based on our subsurface investigation and laboratory testing, *liquefaction* is possible at this site. Our liquefaction analysis will be discussed in detail in the next section.

A *seiche* is a standing wave in an enclosed or partially enclosed body of water. Ground shaking during an earthquake can cause the body of water to slosh back and forth, repeatedly flooding the margins of the body of water. Seiches can occur on lakes, reservoirs, bays and seas. Considering the close proximity of this site to Nehalem Bay, this site is at risk of being flooded by a seiche.

A *tsunami*, or seismic sea wave, is produced when a fault under the ocean floor shifts vertically, displacing the seawater above it. The Tsunami Inundation Map of the Nehalem Quadrangle, (DOGAMI Open File Report O-95-17), included as Figure 4, indicates that the site is within the tsunami inundation zone. Wave run-up heights are anticipated to range from 15 feet to 31 feet above sea level (Priest, 1995). This model is based on an estimated subduction zone earthquake of Mw=8.8 to 8.9 and does not include storm wave influence. **This site should be considered at significant risk to tsunami inundation.**

## 4.3 Liquefaction, Dynamic Settlement and Lateral Spread Evaluation

Liquefaction occurs when saturated deposits of loose to medium dense, cohesionless, fine-grained soils—generally sands and low plasticity silts and clays—are subjected to strong earthquake shaking. If these deposits are saturated and cannot drain rapidly, there will be an increase in pore water pressure. With increasing oscillation, the pore water pressure can increase to the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. Therefore, when the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil reduces to zero, and the soil deposit liquefies (i.e. acts like a fluid).

We performed our liquefaction analysis using the largely empirical *Simplified Procedure* originally developed by Seed and Idriss (1982), and since updated several times, including major revisions in 2004 (Cetin et al.). Our soil strength parameters in our analyses were based on the SPT and pocket penetrometer data collected in our borings.

For our liquefaction analysis, we assumed a groundwater depth of 7.5 feet below the existing ground surface. Based on our soil explorations, laboratory testing, and analysis, liquefiable soil was present in our borings below a depth of about 15 feet. We limited the depth of liquefiable soils to 50 feet below the ground surface, which is generally the standard of practice. Therefore, our liquefiable layer was from 15 feet to 50 feet.

Our evaluation was modeled after a large Cascadia Subduction earthquake with a moment magnitude of 8.5. While a greater magnitude earthquake could occur, the *Simplified Procedure* limits the magnitude to 8.5. Based on our engineering analyses and data collected in boring B-1 at the north end of the site, we estimate dynamic settlement resulting from liquefaction could be as much as 10 inches to 15 inches.

Boring B-2 at the south end of the site indicated the presence of high plasticity clay soils below a depth of about 15 feet. High plasticity clay soils are not considered liquefiable. A 7 foot to 8 foot thick, loose layer of course sand above the clay layer is considered liquefiable. Our analysis indicates up to about 3 inches or 4 inches of dynamic settlement where conditions are similar to B-2. Differential dynamic settlement is generally estimated at one-half to two-thirds of total dynamic settlement.

We performed lateral spread analyses in accordance with the method by Youd et al. (1999). Based on those results, we estimate as much as 5 feet of lateral spread could occur where subsurface soil conditions are similar to boring B-2 at the south end of the site and up to 16 feet of lateral spread could occur where subsurface conditions are similar to boring B-1 at the north end of the site. Our estimations are based on a very large ($M_w$ 9) Cascadia Subduction Zone earthquake. The amount of lateral spread occurring at the site would be different if the earthquake magnitude or source are different.

4.4 2004 SOSSC Seismic Design Parameters

In accordance with Table 1615.1.1 of the 2004 State of Oregon Structural Specialty Code (SOSSC), which is an amendment to the 2003 International Building Code (IBC), we recommend a **Site Class F** for this site when considering the average of the upper 100 feet. We recommend Site Class F because the site soils are potentially liquefiable. According to the 2002 USGS Earthquake Hazards website http://eqint.cr.usgs.gov/eq-men/html/lookup-2002-interp-06.html, the Peak Ground Acceleration (PGA) is 0.56, and the maximum considered earthquake (MCE) ground motions for the site (45.69374 degrees latitude and -123.88106 degrees longitude) are **$S_S$=1.324g** and **$S_1$=0.675g** (for Site Class B and 5 percent critical damping). The USGS website values are a more accurate interpolation of the values presented in Figure 1615(1) of the IBC.

In accordance with Note b in Tables 1615.1.2(1) and 1615.1.2.(2), Site Coefficients $F_a$ and $F_v$ for liquefiable soils may be obtained from the 2 referenced tables based on the Site Class determined without regard to liquefaction provided the building period is not greater than 0.5 seconds. We anticipate the three-story buildings being considered will have a building period not greater than 0.5 seconds. The building period can be estimated by multiplying the number of building stories times 0.1 seconds, or about 0.3 second period for the proposed three-story buildings. Should the project structural engineer determine that any of the building periods are greater than 0.5 seconds, then a site-specific geotechnical investigation and dynamic site response analysis would be required by the code.

For this site, ignoring liquefaction as justified above, we can use the Site Class E coefficients. Therefore, Site Coefficients are **$F_a$=0.9** and **$F_v$=2.40**. The adjusted MCE ground motions are **$S_{MS}$=1.192g** and **$S_{M1}$=1.620g** (for Site Class F and without regard to liquefaction). The return interval for these ground motions is 2 percent probability of exceedance in 50 years.

A site-specific seismic hazard study was beyond the present scope of services for this project.

## 5.0 GEOTECHNICAL ENGINEERING EVALUATION

### 5.1 Geotechnical Engineering Discussion

Based on our evaluation, the geotechnical engineering soil conditions are poor for this site, however the proposed development is suitable, provided the recommendations in this report are incorporated into the design and construction of the project and the risks associated with site development are understood. We understand from Mr. Nelson that supporting the proposed buildings on piles is not an economical option and that alternative foundation methods are necessary. The primary geotechnical factors influencing the proposed construction include:

1. **Variable Density Fill with Organics and Debris.** The native ground at this site likely use to be at the elevation of the swamplands immediately north of the site. At some point, it appears the site was filled to its current elevation, possibly in stages based on the different fill materials used. The fill was generally about 8½ feet to 17 feet in our borings and test pits and, for the most part, does not appear to have been properly compacted. Much of the organic content in the fill was trace to minor and is not anticipated to negatively impact the proposed construction. However, in some test pits, a portion of the fill consisted entirely of organics (wood pieces). It appears these might be buried wood piles. This occurred at TP-1 (4 to 7 feet below grade), TP-2 (3 ½ to about 8 feet below grade), TP-6 (5 to 8 feet below grade), and TP-7 (7 to 10 feet below grade) and could be present elsewhere throughout the site.

   The presence of a relatively thick covering of poorly compacted fill and organics will cause the proposed buildings to settle excessively when considering both total and differential settlement. Settlement due to organics could continue slowly for years as the material decomposes. The amount of total and differential static settlement is difficult to predict because the fill is so variable, but total static settlement could be several inches, with maximum differential static settlement as much as 90 percent to 100 percent of total settlement.

2. **Soft, Compressible Peat and Organic Clay.** All of the borings and test pits encountered organic soil. The upper fill primarily contained wood pieces, logs and stumps, while the underlying native soil primarily contained peat and rootlets. The organics in the upper fill will be prone to static settlement from building loads as well as long-term static settlement due to decay. The peat will not likely decay because it appears to be below the water table, however it is extremely compressible. Analyses of one-dimensional consolidation lab tests indicate the potential for up to about 4 to 8 inches of primary total static settlement that could take several years to occur.

3. **Potentially Liquefiable Soil.** In Section 4.3 above, we estimated that potentially liquefiable soil was present in boring B-1 at the north end of the site from about 15 feet to 50 feet. The liquefiable soil layer in B-2 was considered from 7½ to 15 feet. The test pits did not extend deep enough to evaluate liquefaction. Based on our evaluation of boring B-1, we estimated

up to 10 inches to 15 inches of potential total dynamic settlement. We estimated up to 3 inches to 4 inches of dynamic settlement in boring B-2. Differential dynamic settlement is generally estimated at ½ to 2/3 of total dynamic settlement. These settlement magnitudes would be extremely destructive to buildings supported on conventional shallow foundations.

4. **Lateral Spread.** Based on our analyses, we estimated up to 5 to 16 feet of lateral spread should a major earthquake impact the site.

5. **Seiche or Tsunami Event.** Damage to the property from a seiche or tsunami could be extensive.

6. **Potential Methane Gas Generation from Buried Organic Decomposition.** While methane gas risk and mitigation is not part of our expertise as geotechnical engineers, we should caution that buried organics like those found on this site are known to generate harmful methane gas, which then works its way to the surface and can collect in crawl spaces and basements. Since we do not anticipate that the proposed construction will include crawl spaces or basements, methane gas hazard does not appear to be a problem. But we do recommend you consult someone with expertise in mitigating the risk of methane gas as a precaution.

Based on the above geotechnical factors, and the understanding that a pile foundation system is not an acceptable alternative, we recommend the proposed buildings be supported on either shallow raft or mat foundations. A conventional shallow foundation system is not appropriate for this site.

A raft foundation normally consists of a reinforced concrete slab which extends over the entire loaded area. It is then stiffened with a matrix of criss-crossing reinforced concrete ribs or beams. Where the raft may be variable thickness, a mat foundation is generally uniform thickness throughout with 2 layers of reinforcing steel. The raft foundation system typically uses less concrete than a mat and may be more economical. However, excavation for the raft system can be more complicated. Ultimately, the foundation system selected needs be rigid enough to compensate for several inches of differential static and dynamic settlement.

With a raft or mat foundation, the buildings will still settle several inches over time. This means that eventually exterior flatwork (i.e. sidewalks, patios, driveways) may not match up well with building entrances. We recommend that exterior flatwork not be structurally connected to the buildings. We also recommend consideration be given to raising the buildings 1 to 2 feet above exterior grade and entering the buildings with steps. Then when the buildings settle over their useful life, the steps into the buildings can easily be modified as necessary. We also recommend that the buildings be constructed as early as possible in the construction schedule and then wait as late as possible in the construction schedule to construct exterior flatwork and attach underground utilities at the perimeter of the buildings. Specially designed, flexible utility connections should also be used to compensate for future building settlement.

We have considered methods for reducing the amount of potential static settlement. These methods include surcharging the site prior to construction and load compensation. The surcharge program would consist of temporarily placing fill on each building pad that is greater than the weight of the proposed building. We recommend a surcharge of at least twice the weight of the building. Based on our consolidation lab tests, it could take a couple of years for a surcharge program to be completed. While the surcharge will mitigate short-term settlement of the soft, compressible soil, it will not mitigate static settlement that will occur over several years due to decomposition of the upper organic soils and the slow settling peat and organic soil. So surcharging will significantly reduce the total amount of static settlement but it will not eliminate it completely. More discussion on the surcharge alternative is presented in Section 5.5. However, given the fact that it could take a couple of years for most of the static settlement to occur, we anticipate the surcharge program may not meet the project schedule needs.

The load compensation program would consist of "unloading" each building pad by at least the total weight of the building. This would normally be done by lowering the building pad grades, but we do not anticipate that this is feasible at this site. Therefore, we recommend consideration be given to replacing site soil beneath the building pads with lightweight rigid cellular polystyrene geofoam (geofoam). This material comes in block shapes and typically has a density of 1 to 2 pounds per cubic foot (pcf), compared to the site soils to be replaced, which have a density on the order of 110 to 120 pcf. Depending on the weight of the proposed buildings, the site may need to be excavated several feet and replaced with geofoam blocks. Ideally, this would eliminate all settlement concerns, except the potential settlement due to liquefaction. However, due to the complexity of the subsurface conditions, we recommend the design team assume that the buildings will still experience total static settlement of several inches. As such, we recommend the use of flexible utility connections and consideration be given to constructing the building entrances 1 to 2 feet higher than the adjacent grade so there is room to settle without interrupting the entrances. More detailed design discussion on this alternative is presented in Section 5.6.

In summary, we anticipate that the buildings will be supported on rigid, reinforced concrete raft or mat foundations bearing on geofoam blocks after the major concentrations of organics (wood, logs, stumps) have been removed from the upper fill. Ideally, this system will mitigate excessive settlement except dynamic settlement caused by an earthquake.

## 5.2 Site Preparation

We recommend that topsoil, vegetation, roots, wood, stumps and old buried foundations and utilities in the construction areas be stripped from the site. A representative of the Geotechnical Engineer should evaluate the near surface soils and determine of additional stripping or removal of previous improvements is needed at the time of construction. Utilities should be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations should be backfilled with properly compacted structural fill which is constructed as outlined in Section 5.3 of this report.

We recommend woody debris material be removed where the wood content is significant (i.e. TP-1, TP-2, TP-6 and TP-7). However, because the buildings will be supported on a rigid foundation that will mitigate settlement due to decomposition of the wood over time, it is not required that all organics be removed from the existing fill. Organic removal should be addressed by a representative of the Geotechnical Engineer during earthwork construction.

After stripping and excavating to the proposed subgrade level, building areas to support the building or pavement should be proofrolled with a heavily loaded tandem axle dump truck or similar rubber-tired vehicle. Soils that are observed to rut or deflect excessively under the moving load, or are otherwise judged to be unsuitable should be excavated and replaced with properly compacted fill. The proofrolling activities should be witnessed by a representative of the Geotechnical Engineer.

5.3 Fill Requirements

After subgrade preparation and observation have been completed, fill placement may begin if needed. The first layer of fill material should be placed in a relatively uniform horizontal lift on the prepared subgrade. Fill materials should be free of organic or other deleterious materials, have a maximum particle size less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. The on-site, non-organic soils are suitable for use as structural fill. Structural fill should be compacted to at least 95 percent of modified Proctor maximum dry density as determined by ASTM D1557.

Fill should be placed in maximum lifts of 8 inches of loose material and should be compacted within the range of 3 percentage points below to 2 percentage points above the optimum moisture content value. If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts. The fill should extend horizontally outward beyond the exterior perimeter of the building and footings a distance equal to the height of the fill or 5 feet; which ever is greater, prior to sloping. Also, fill should extend horizontally outward from the exterior perimeter of the pavement a distance equal to the height of the fill or 3 feet; whichever is greater, prior to sloping.

5.4 Foundation Recommendations – Raft or Mat

As discussed in Section 5.1 above, we recommend the buildings be supported on rigid, shallow foundations that can tolerate several inches of differential settlement. This could consist of shallow, rigid raft or mat foundations. A conventional shallow foundation system should not be used for this site.

Structural design of reinforced concrete raft or mat foundations may include an elastic analysis of the foundation and bearing soil. This analysis uses a coefficient of vertical subgrade reaction ($K_{v1}$) to represent the elastic properties of the bearing soil. The coefficient of subgrade reaction is a function of soil type, compressibility and foundation size. $K_{v1}$ represents the coefficient of subgrade reaction for a vertical load on a 1-square foot test plate. For the generally soft, compressible soils at this site, we recommend a soil coefficient of subgrade reaction ($K_{v1}$) of 43 kips per cubic foot (kcf)—roughly 25 pci—for the anticipated foundation bearing soils. The footings should be designed using a coefficient of subgrade reaction ($K_v$), which is determined for foundations supported on clayey soils, by dividing the subgrade modulus $K_{v1}$ by the footing width (B)—see equation below. For a raft or mat foundation, we recommend setting B equal to the lesser of the smallest column spacing or foundation width.

$$K_v = K_{v1} / B$$

where: $K_v$ = coefficient of subgrade reaction to be used by structural engineer, kcf
$K_{v1}$ = coefficient of subgrade reaction for 1-foot wide footing, kcf
B = footing width (lesser of the smallest column spacing or foundation width), feet

For example, if $K_{v1}$ is 43 pci as we recommended for this site, and the least column spacing is 20 feet, then the foundation coefficient of subgrade reaction to use in the structural design ($K_v$) is 2.15 kcf. $K_v$ may be increased by a factor of 1.5 for transient seismic loading conditions.

Where the structural engineer designs rigid foundations using spring constants, each spring constant should be determined by calculating $K_v$ as shown above and then multiplying $K_v$ by the tributary area supported by the spring (spring constant units: pounds/foot).

Shallow foundations may be designed to resist lateral loads using a passive earth pressure based on an equivalent fluid density of 350 pounds per cubic foot on footings poured "neat" against firm in-situ soils or properly backfilled structural fill. The recommended passive earth pressure is an ultimate value and does not include a factor of safety. The structural engineer should apply a factor of safety of 1.5 to obtain the "allowable" value. The upper 1 foot of soil should be neglected when determining passive earth pressure resistance acting against the vertical face of footings, unless the adjacent ground is paved.

We do not recommend using frictional resistance between the base of footings and subgrade to resist lateral loads on foundations because we anticipate that over time, portions of the foundations will not be in contact with the subgrade as the site settles. Because foundation sliding resistance cannot be used, it may be necessary to construct keyways in the foundations to increase the passive resistance.

Foundations exposed to weather should be embedded at least 12 inches below the adjacent exterior grade to prevent frost heave. Provided the bottom of the foundation conservatively terminates no deeper than 7 ½ feet below existing grade, we do not anticipate that the foundation will need to be designed to resist buoyant forces due to groundwater.

5.5 Surcharge

The surcharge program, if utilized, would consist of temporarily placing fill on each building pad that is at least twice the weight of the proposed building. To speed up the process, additional fill height can be added. Once total building loads are provided by the project Structural Engineer, we can provide final recommendations for the surcharge volume required. For preliminary planning purposes, assuming surcharge fill material with an in-place unit weight of about 100 pcf and three-story buildings with a total load of about 100 psf per floor, the minimum surcharge height would be about 6 feet.

The following are general specifications for surcharge construction:

- Topsoil beneath the surcharge should be removed and wasted or stockpiled for use in landscape areas on-site.
- The site should be filled to or above the desired site grade with structural fill as described in the Site Preparation section of this report.
- A 2-foot thick layer of clean, free-draining material such as sand should be placed over the surcharged pads to facilitate drainage. The surcharge process will likely generate a lot of water at the base of the surcharge fill as the underlying soft/loose soils are compressed.
- Settlement plates should be placed across the site area to be preloaded, as indicated by the geotechnical engineer, to allow monitoring of the consolidation.
- The material used for the surcharge should generally be granular (i.e. silty sand, sand or gravel) so that it is easier to place.
- The surcharge fill should be moderately compacted with a large vibrating smooth drum roller but it does not need to be compacted to meet a structural fill requirement (i.e. at least 95 percent) since it will removed.
- The surcharge fill should be placed and the surface sloped to provide runoff of precipitation. Side slopes should be no steeper than 1 horizontal to 1 vertical (1H:1V).
- The surcharge should extend out laterally at least 5 feet beyond the building footprints.
- Monitoring of the surcharge with survey readings should take place on a regular basis. This data should be forwarded to the Geotechnical Engineer for evaluation so that it can be determined when the surcharge program is complete (i.e. the surcharge load has essentially stopped settling). Based on our consolidation lab tests, it could take a couple of years for a surcharge program to be completed.

### 5.6 Geofoam

The geofoam material should comply with ASTM D6817-02, "Standard Specifications for Rigid Cellular Polystyrene Geofoam." Further information is needed about the actual total building weights and footprint before we can provide our final geofoam material and thickness recommendations, but we anticipate that either an EPS15, EPS22, or EPS29 material would be appropriate. Each of these products has a different unit weight and compression resistance (reference Table 1 in ASTM D6817 for minimum unit weight and compressibility characteristics for different contact pressures). Selection of the final product should be based on optimizing the geofoam unit weight and compressibility. Provided the EPS15 material meets the settlement criteria for the given contact pressure, it would be the most cost effective. EPS15 has a unit weight of about 1 pound per cubic foot, and will compress no more than 1 percent when loaded up to about 500 psf.

Geofoam is adversely affected by exposure to ultra-violet light. As such, it should be embedded in the ground and not exposed to sunlight. Prior to installation, we recommend the geofoam supplier perform compression testing on their product to confirm its compressibility. Previous experience indicates that the local geofoam suppliers use different quality polystyrene resin beads. The bead quality affects the geofoam compression resistance. The final recommendations for which geofoam product to use and how thick it should be should be based on the suppliers' compression testing.

For preliminary budgeting purposes, the EPS15 material is currently available from local suppliers for about $40 per cubic yard. For a building pad of about 7,000 square feet and a uniform foundation pressure of 300 pounds per square foot, roughly 720 cubic yards of geofoam would be required, at a cost of about $29,000.

Provided the bottom of the geofoam terminates no deeper than 7½ feet below existing grade, we do not anticipate that the foundation system will need to be designed to resist buoyant forces due to groundwater. We recommend a properly compacted granular leveling pad of at least 4 inches be placed between the native soil and geofoam.

## 6.0 CONSTRUCTION CONSIDERATIONS

CGI should be retained to provide observation and testing of construction activities involved in the foundation and earthwork related activities of this project. GCI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundation if not engaged to also provide construction observation and testing for this project.

### 6.1 Drainage and Groundwater Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for structures during construction. Positive site drainage should be maintained throughout construction activities. Excavated areas should be sloped toward one corner to allow water removal.

The site grading plan should be developed to provide rapid drainage of surface water away from the building and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slabs. Careful consideration should be given to the potential impact of landscaped areas and/or sprinkler systems on adjacent foundations and floor slabs. Roof run off should be conveyed at least 10 feet away from the building prior to discharge upon unpaved surfaces.

### 6.2 Excavations

Temporary earth slopes and trenches should be cut in accordance with Department of Labor Occupational Safety and Health Administration (OSHA) guidelines. Job site safety is the responsibility of the project contractor.

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our

October 9, 2018

Mr. Ken Ulbricht
Bott's Marsh LLC
P.O. Box 1161
Seaside, OR 97138

RE: Wheeler Development Site Traffic Analysis "Note to File"
(CTS Engineers Project No. OR07.031.T01)

Dear Ken:

In 2007 CTS Engineers conducted a traffic study (*"the 2007 traffic study"*) for The Point at Wheeler Landing (*"the 2007 development proposal"*), a proposed mixed use development located along the west side of Hwy 101 at Hemlock Street in Wheeler, Oregon.

The 2007 development proposal included 44 townhomes, 14 three-story live-work townhomes with first-floor retail space, and another 19,077 gross square feet of retail space on three pads. It faced opposition from residents concerned about the proposed land use density and traffic impacts. A key traffic concern was that sole access would be provided by adding a west leg to the existing Hwy 101/Hemlock Street intersection to connect the highway and Marine Drive, two parallel roadway facilities separated by a 100-foot corridor owned by the Port of Tillamook Bay and the former location of the Port of Tillamook Bay Railroad (POTB), which was an active shortline connecting Tillamook Bay to the Willamette Valley when the 2007 traffic study was prepared. The 2007 development proposal did not advance through the land use approval process. (In December 2007 the POTB corridor was severely damaged by a major storm. Some sections have been permanently abandoned. Other segments have been targeted for conversion from rail to trail. I am uncertain of the status of this specific segment of POTB).

Per your request I contacted City of Wheeler land use planner Sabrina Pearson to find out what the City needs in an update of the 2007 traffic study for a new land use application for the site, with less intense development (*"the 2018 development concept"*).

Ms. Pearson said the 2018 development concept would NOT require an update of the 2007 traffic study for the following reasons:

- The 40+/- residential units in the 2018 development concept would add fewer new vehicle trips on local roads than the development proposal analyzed in the 2007 traffic study.
- There have been no roadway changes or annexations since the 2007 traffic study was prepared, and only half a dozen or so new homes built in the City.
- Therefore existing traffic conditions in 2018 are likely similar to the baseline conditions analyzed in the 2007 traffic study.

- ODOT should be contacted to confirm applicable access permit requirements for connection of Hemlock Street to Hwy 101 from Marine Drive ("the Hemlock Street extension"). The Hemlock Street extension would pass through a 100-foot corridor separating Marine Drive and Hwy 101 that is the site of the former active shortline serving the Port of Tillamook Bay (the POTB). Current status of this segment of POTB would need to be confirmed with the Port and ODOT's Rail Division. The City Fire Department also needs to be consulted about the Hemlock Street extension. Shoulder widening may be required to meet minimum residential standards for fire access.

She said higher density development of the site could be possible if Marine Drive were improved to applicable standards to provide a second public street access. The City likely would require improvement of Marine Drive 1) north of the main Wheeler Marina entrance to the Dart property, 2) along the Dart property frontage between adjacent to the marina, and within the site boundary. (The paved width of Marine Drive appears to narrow north of the main Wheeler Marina entrance.)

The Bellevue office of CTS Engineers (425-455-7622) can provide civil engineering and design services you may need for access improvements required by the City and/or ODOT.

Sincerely,

Howard Roll, P.E.
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949-379-9921

40 Santa Comba
Irvine, CA 92606

c: Barry Knight, CTS Engineers

August 29, 2007

**Project No. OR07.031.T01**
***The Point At Wheeler Landing***

CTS Engineers

Doug W. Hooper
City Manager
PO Box 177
Wheeler OR 97147

**SUBJECT:** ***Traffic Analysis for the proposed The Point At Wheeler Landing along Hemlock Street and west side of Hwy 101 in Wheeler, OR.***

Dear Mr. Hooper:

As requested, a traffic impact analysis has been prepared for the buildout of the proposed The Point at Wheeler Landing residential and retail development on the west side of Hwy 101 /Hemlock Street on a site of approximately 8.9 acres zoned water related commercial / general commercial. This development will consist of 44 two-story townhomes, 14 three-story live-work townhomes which will have approximately 570 GSF of retail space and a garage on the first floor and the upper two levels as a residence and 3 two-story buildings that will have a total of approximately 19,077 GSF retail space. ***Figure 1*** contains a vicinity map of the proposed site and surrounding roadway system. Access to the site will be provided via a proposed west leg to the existing intersection of Highway 101 /Hemlock Street (see attached Figures). ***Figure 1A*** presents a site plan of the proposed development.

This traffic analysis includes a detailed assessment of the traffic impacts of the proposed The Point at Wheeler Landing and the growth in background traffic due to other sources. Based on the results of this analysis, it is concluded that the proposed development can be constructed without adversely affecting the traffic operational or safety characteristics of the adjacent roadway system. Specific findings of this study are as follows:

- When the entire site is developed, it is estimated that The Point At Wheeler Landing will generate approximately 652 ***net new*** vehicle trips during a typical weekday, including 38 vehicle trips during the AM peak hour and 62 vehicle trips during the PM peak hour.
- Analysis of future 2010 background traffic volumes that will exist regardless of buildout for the proposed development found that the study area intersection will operate at acceptable Levels of Service (LOS) C or better during the peak hour /30$^{th}$ HV with V/C ratio of 0.01 or better.
- Analysis of future 2010 traffic conditions with the buildout of The Point at Wheeler Landing found that the traffic generated by this development will not adversely impact future Levels of Service. These results indicate that the study area intersection will continue to operate at LOS C or better during the peak hour /30$^{th}$ HV with V/C ratio of 0.01 or better.

The following sections document the study's methodology, results, and major findings.

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**PROPOSED DEVELOPMENT**
The proposed The Point at Wheeler Landing residential and retail development on the west side of Hwy 101 /Hemlock Street on a site of approximately 8.9 acres zoned water related commercial /general commercial. This development will consist of 44 two-story townhomes, 14 three-story live-work townhomes which will have approximately 570 GSF of retail space and a garage on the first floor and the upper two levels as a residence and 3 two-story buildings that will have a total of approximately 19,077 GSF retail space. ***Figure 1*** contains a vicinity map of the proposed site and surrounding roadway system. Access to the site will be provided via a proposed west leg to the existing intersection of Highway 101 /Hemlock Street (see attached Figures). ***Figure 1A*** presents a site plan of the proposed development.

**STUDY AREA**
Based on preliminary discussion with ODOT staff and previous traffic impact analyses conducted by CTS Engineers in the City, one intersection was selected for the analysis during AM Peak, PM peak hour and $30^{th}$ DHV (design hour volume) conditions at the minor street stop-controlled intersection of Hwy 101 /Hemlock Street.

**EXISTING CONDITIONS**

**Transportation Facilities**
***Figure 2*** shows the approximate location of The Point at Wheeler Landing development and the surrounding roadway network including the lane configurations in the study area.

*Area Roadway System*
The main roadways in the study area include Hwy 101 and Hemlock Street. ***Table 1*** presents the characteristics of these roadways. Area roadway and sight distance photos are in the Appendix to the report. Both existing and future traffic analyses in this study were conducted assuming existing roadway conditions.

***Table 1: Summary of Study Area Roadway Characteristics***

| Street Name | Road Class | Width (Feet) | Speed Limit | Sidewalks | Bike Lane | On-Street Parking |
|---|---|---|---|---|---|---|
| **Hwy 101** (At Hemlock Street) | State Route | 28 | North-45 South-25 | East-Yes West-No | No | No |
| **Hemlock Street** | Local | 18 | - | No | No | No |

**Land Uses**
Land use in the immediate vicinity to the south along Hwy 101 is mostly retail and a public boat launch and dock along Marine Drive. Land to the east of the site across Hwy 101 consists of single family homes.

**Pedestrian and Bicycling Considerations**
Sidewalks currently exist along the east side of Hwy 101. To the north of the Hemlock Street, neither sidewalks nor bike lanes are present along Hwy 101. However, Hwy 101 has shoulder bikeways. The applicant will provide sidewalks/walking paths, curbs and gutters throughout the internal roadway system.

**Transit Considerations**
"The Wave" operates bus routes to/from Portland's Union Station. The Wave traverses Tillamook County, providing service to Manzanita, Nehalem, Wheeler, Rockaway Beach, Garibaldi, Bay City, Tillamook, Hebo and Cloverdale and from these locations to Portland. The nearest stop to the site is about a quarter mile from the site's southern edge.

**Existing Traffic Volumes**

A reconnaissance of the site and its vicinity was conducted. To assess the impact of buildout of the proposed The Point at Wheeler Landing, traffic operations were analyzed during both AM and PM weekday peak hours because these periods represent reasonable "worst case" for traffic scenarios in the study area. Furthermore, traffic operations were also analyzed during 30$^{th}$ DH.

*Traffic Volumes*

***Figure 3*** shows recent weekday AM and PM peak hour traffic volumes obtained at the key intersection in the vicinity of the site. Traffic volumes during PM peak hour within the study area were obtained from actual weekday peak hour manual traffic counts conducted during August of 2007 and the AM counts were obtained from tube counts conducted during the same time in August of 2007. These data revealed that the weekday PM peak hour occurs between 3:00 - 4:00 PM, and the typical AM peak hour between 7:00 – 9:00 AM is significantly lower. This inconsistency is typical of the nature of the proposed site location. Traffic volumes greater than 25 were rounded upward to the nearest five vehicles. For the purpose of this traffic study, only PM peak hour between 3:00 – 4:00 PM will be analyzed.

To evaluate intersections for existing and future operational deficiencies, ODOT requires analysis of 30$^{th}$ highest design hour volumes (30$^{th}$ DHV), which is the hourly volume of traffic that is exceeded only 29 hours over the entire year. To estimate 30$^{th}$ DHV, typical PM peak hour volumes are adjusted using a seasonal factor. The ODOT methodology contained in the TPAU Manual – *Developing Design Hour Volumes* calls for averaging the most recent five years of seasonal factors after first tossing out the highest and lowest factors for each month. The 30$^{th}$ DHV is determined by adjusting typical PM peak hour volumes with a seasonal factor determined using data from an appropriate Automatic Traffic Recorders (ATR) or, if there is no ATR nearby, from the most current seasonal trend table. Approximately 8 miles south of Hemlock Street along Hwy 101 /Washington Street intersection in Rockaway Beach, ODOT maintains an Automatic Traffic Recorder (ATR) (29-001). Based on the data from this ATR it is revealed that the seasonal adjustment for the month of July (ATR's peak month) and August (traffic count month) is approximately the same (i.e. July/August = 143% / 143% = ***1.00***). Therefore, the counts obtained in August ***will not require adjustment***. Again, as mentioned earlier, for the purpose of this traffic study only PM peak hour will be analyzed.

***Table 3*: Seasonal Adjustment Factors For OR 18-B (Based on ATR #29-001 on OR 9)**

| | **2006** | **2005** | **2004** | **2003** | **2002** | **Average** |
|---|---|---|---|---|---|---|
| **July** ***(Peak Month of the year)*** | 143% | 128% | 143% | 148% | 144% | **143%** |
| **August** ***(Traffic Count Month)*** | 142% | 124% | 141% | 148% | 146% | **143%** |

*Peak Hour Traffic Operations*

Traffic conditions at the key intersection in the study area were analyzed only during PM peak hour because the 30$^{th}$ DH along Hwy 101 is approximately the same as PM peak hour. Intersection operational analyses were conducted using the procedures in the ***2000 Highway Capacity Manual (HCM)*** for evaluating signalized and unsignalized intersections, which describe the traffic operations of an intersection in terms of its Volume to Capacity Ratio (V/C), Delay, Queue Length, and Level of Service (LOS). For unsignalized intersections, the intersection's LOS is stated relative to the most critical intersection approach or maneuver, typically the left turns from the minor street approach. For signalized intersections, the LOS is a function of the average vehicle delays that drivers on all approaches experience. For the section of Highway 101 in the vicinity of the site, ODOT standards require that all intersections operate at a V/C ratio of 0.75 or better (Aug 2005 Amendment to Table 6 in Policy 1F – Mobility Standards, 1999 Oregon Highway Plan). The V/C ratio is the ratio of hourly traffic volume to the theoretical maximum hourly volume of vehicles that a roadway section or approach can accommodate. The LOS worksheets for the results presented in this study are attached in the appendix to this report.

***Table 2*** presents the calculated results (V/C ratios) for our existing conditions analyses at all study area intersections based on the peak hour traffic volumes shown in ***Figure 3***. These results indicate that the study area intersection will operate at LOS C or better during weekday peak hour /30th HV with V/C of 0.01 or better during. These findings were confirmed during our general observations of traffic operations. Furthermore, queuing along Hemlock Street is minimal (1 vehicle during the peak hour) which was confirmed during our intersection volume counts.

**Table 2: Existing 2007 Weekday PM Peak Hour /30th DH Levels of Service**

| Intersection | AM Peak Hour | | | PM Peak Hour /30th HV | | |
|---|---|---|---|---|---|---|
| | Minor Street Stop Control | | | | | |
| | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS |
| **Hwy 101 /Hemlock Street** (Critical Approach: WB) | - | - | - | EB-16.0<br>WB-13.1 | 0.01<br>0.01 | C<br>B |

**Traffic Safety**
Collision records requested. Data pending.

*Intersection Sight Distance*
A general assessment of intersection sight distance was performed along the study area intersection. Photos in the Appendix illustrate sight distance at the study area intersections. Hwy 101 in the vicinity of Hemlock Street (site's proposed access) is relatively straight and flat. To the east, Hemlock Street terminates as a "dead end". ODOT standards require that intersection sight distances conform to **AASHTO – *A Policy on Geometric Design of Highways and Streets 2001,*** which requires that measurements be based on an eye height of three and one-half (3.5) feet above the controlled road at least fifteen (15) feet from the edge of the vehicle travel lane of the uncontrolled public road to an object height of three and one-half (3.5) feet on the uncontrolled public road. For Hwy 101, a state highway with a posted speed limit of 45 mph to the north of Hemlock Street and 25 mph to the south, AASHTO requires 500 feet of available clear sight distance for 45 mph and 280 feet for 25 mph. Our measurements from Hemlock Street found that sight distance exceeds 550 feet to the north, but is obstructed to the south due to vegetation. After the removal/trimming of vegetation, the sight distance to the south will be at least 400 feet, which exceeds the minimum criteria of 280 feet.

Based on the above and the field observations, it does not appear that the applicant needs to address any sight distance traffic safety problems in the immediate vicinity of the site.

## TRAFFIC IMPACT ANALYSIS

The impact of traffic generated by the full buildout of The Point at Wheeler Landing on the surrounding street system during the critical weekday peak hours was analyzed as follows:

- A three-year buildout was assumed, to the year 2010. Therefore, the existing traffic volumes were adjusted to estimate future 2010 background traffic conditions including other nearby developments expected to be completed before 2010.
- Total AM and PM peak hour trips both into and out of The Point at Wheeler Landing site were estimated for complete buildout conditions.
- Existing traffic volumes on the roadways surrounding the site and the site's proximity to major roadways were evaluated to estimate the trip distribution patterns in the study area for vehicle trips generated by the site.
- Estimated site-generated traffic volumes for the AM and PM peak hours were assigned to the roadway network and added to the estimated 2010 background traffic volumes to represent future traffic conditions with full buildout of the site.
- Future LOS and volume-to-capacity ratios (v/c ratios) at key intersections in the study area were examined under both background and full buildout traffic conditions.

### Future 2008 Background Traffic Volumes

The future year analysis, as required by ODOT (OAR 734-051-0180) for any single phase development with an anticipated ADT between 0 and 999, is the year of its opening. Full buildout of the proposed The Point at Wheeler Landing residential development is expected to occur by the end of 2010. To assess the likely future traffic conditions regardless of the proposed development, increases in traffic due to general growth as well as other proposed developments in the vicinity of the site were estimated. Discussion was held with City /ODOT staff to review the area. There are no other approved developments in the immediate area.

To assess the likely future traffic conditions regardless of the proposed development, increases in traffic due to general growth as well as other proposed developments in the vicinity of the site were estimated. Discussions/meetings were held with ODOT planning staff to review traffic growth trends along Hwy 101. This research found there were no other major developments in the near future that have been proposed or approved in the vicinity. To determine an appropriate background growth factor for developing design hour volumes (30$^{th}$ DHV) for this project, ODOT's TPAU (Transportation Planning and Analysis Unit) Future Volume Tables (FVTs) were used. These tables are based on historical volume trends from past years to project future volumes. Notably, data from mileposts north (MP 47.08) and south (MP 47.38) of the site were used to interpolate a more accurate result. As shown in ***Table 3*** and from the FVT at MP 47.08 along Highway 9 (US 101), the 2003 volume is shown as 5,400, the 2025 traffic volume is 7,100, and the r-squared value is 0.765. At MP 47.38 the 2003 volume was found to be 5,400, the 2025 traffic volume ,6800, and the r-squared value 0.93. Both of these r-squared values are acceptable and indicate a strong relationship between historical data points. Using this data, we first computed the 22 year growth factor for each milepost. At milepost 47.08 the 22 year growth factor is 1.31 (7,100/5,400), and at milepost 47.38 the 22 year growth rate is 1.25. Next, assuming a linear relationship, the average of these two 22 year growth factors was used to compute the annual growth factor: (((1.31+1.25)/2)-1)/22 = 0.0127, or 1.3% straight-line growth per year. Thus, to calculate the 2010 future background PM or 30$^{th}$ DHV's that correspond with this annual growth rate and full buildout year, existing peak hour volumes (30$^{th}$ HV) in ***Figure 3*** were increased by 3.9 percent (3 years x 1.3% per year = 3.9%) to account for other increases in traffic due to sources outside the study area during the next three years to 2010. The resulting weekday future 2010 background PM peak hour /30$^{th}$ DHVs are shown in ***Figure 5*** and the intersection capacity analysis results are shown in ***Table 4.***

**Table 3: Development of Future Growth Factor**

| Location | From ODOT/TPAU Future Volume Tables | | | 22 Year Growth Factor |
|---|---|---|---|---|
| | 2003 Volume | 2024 Volume | $R^2$ Value | |
| US 101, Milepost 47.08 | 5,400 | 7,100 | 0.765 | 1.31 |
| US 101, Milepost 47.38 | 5,400 | 6,800 | 0.939 | 1.25 |

These results indicate that under future background 2010 traffic conditions, traffic operations at study area intersections are expected to degrade only slightly when compared to the existing conditions analysis results. Intersection Levels of Service are similar to existing conditions. The critical mobility measure of V/C ratio is estimated to remain within acceptable criteria as results indicate in ***Table 4***.

**Table 4: Future Background 2010 Weekday PM Peak Hour /30th DH Levels of Service**

| Intersection | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|
| | Minor Street Stop Control | | | | | |
| | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS |
| **Hwy 101 /Hemlock Street** (Critical Approach: WB) | - | - | - | EB-16.5<br>WB-13.4 | 0.01<br>0.01 | C<br>B |

**Site-Generated Traffic Volumes**

***Figure 1A*** shows the proposed site plan for The Point at Wheeler Landing. The applicant is proposing to construct about 44 two-story townhomes, 14 three-story live-work townhomes which will have approximately 570 GSF of retail space and a garage on the first floor and the upper two levels as a residence. This development also has three 2-story buildings that will have a total of approximately 19,077 GSF retail space. The site is located along the west side of Hwy 101 /Hemlock Street intersection on a site of approximately 8.9 acres zoned water related commercial /general commercial. Access to the site will be provided via a new public street connection along the west side of Hwy 101 and aligned with existing Hemlock Street. The number of vehicle trips into and out of The Point at Wheeler Landing were estimated using standard trip generation rates for Townhomes (Land Use Code 230) and Shopping Center (Land Use Code 820) as presented in the ITE Trip Generation Report (7th Edition). Also, for the work portions of the live/work units, the trip rates from ITE Land Use Code 710 General Office are applied to the work portion (GSF) of these dwellings. This is a very conservative approach because the trip generation for the residential use includes people leaving the home to go to work, and the office rates include people driving to work. The primary concept of these live/work dwellings is that people do not have to leave their home to travel to a traditional office/work place. ***Thus, to remove double counts, trips generated by live portion from live/work units will not be included.*** Trip rates in this ITE publication are based on empirical observations performed at many similar sized developments located throughout the United States.

Due to the nature of retail land uses in the proposed site, a full understanding of the trip types that will be traveling to/from the site is necessary. In evaluating the traffic impact of retail uses, it is important to realize that the majority of vehicle trips to/from a retail use, such as the proposed The Point at Wheeler Landing will result from vehicles already on the road making trips for other purposes, such as getting to/from work or shopping at adjacent uses. This is particularly true for retail uses along major commuter routes. The first trip type, *pass-by trips*, comes from drivers who are already traveling along an adjacent street. As they pass by the site as part of their regular travel route, they turn into the site to make a purchase and then continue on their original route. The second trip type is *diverted trips* from other drivers already on the road, but who divert their route a few streets to enter the site. After they make their purchase they then return to their original route. The third trip type is totally new trips on the roadway system. These include nearby residents who leave their home or office and drive to make a purchase and then return home without making any other stops. To be conservative, we considered diverted as a new trip, so, for this study diverted trips are included in new trips.

Furthermore, it should also be noted that the proposed site includes both residential and retial. Based on ***ITE Trip Generation Report (7th Edition)***, a mixed-use development such as the proposed will likely have trips that remain internal to the site (for example, trips from an on-site residential to an on-site retail destination). Analysis of potential internal travel demand was performed by using the *Multi-Use Development Trip Generation and Internal Capture Worksheet* from the ***ITE Trip Generation Handbook.*** This worksheet is set up to estimate the amount of internal travel based on the size/type of land uses for a typical mixed-use development. For instance, a retail and residential use in the same mixed-use development would be expected to produce internal trips between the two. The generated trips from each of these land uses were put into the internal capture spreadsheet (shown in the *Appendix).* It was found that approximately 17 percent of the traffic was internal trips. These internal trips were removed from the total site generated trips before applying the pass-by reduction. Based on these trip rates, and as shown below in ***Table 5***, we estimate that The Point at Wheeler Landing will potentially generate approximately 652 new vehicle trips throughout a typical weekday, including 38 vehicle trips during the AM peak hour and 62 vehicle trips during the PM peak hour.

**Table 5: Trip Generation Estimate for Buildout of The Point at Wheeler Landing**

| Land Use/ Site Location | | Daily Trips | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|---|---|
| | | | Total | In | Out | Total | In | Out |
| Attached Townhomes (44 Units) (ITE Code 230) | | 258 | 19 | 3 | 16 | 23 | 15 | 8 |
| *Live Work Townhomes (14 Units) (ITE Code 230)* | | *82* | *6* | *1* | *5* | *7* | *5* | *2* |
| Gen. Office in Live/Work Townhomes (14 @ 572 GSF ea. = 8,008) (ITE Code 710) | | 88 | 12 | 11 | 1 | 12 | 2 | 10 |
| Shopping Center (19,077 GSF) - ITE Code 820A | | 819 | 20 | 12 | 8 | 72 | 35 | 37 |
| Internal Trips | 17% | 139 | 3 | 2 | 1 | 12 | 6 | 6 |
| Total External Trips | | 680 | 17 | 10 | 7 | 60 | 29 | 31 |
| Pass-by Trips | 55% | 374 | 9 | 5 | 4 | 33 | 16 | 17 |
| Total New /Diverted Trips | 45% | 306 | 7 | 4 | 3 | 27 | 13 | 14 |
| **Total Site Generated Trips** | | **1,165** | **51** | **26** | **25** | **107** | **52** | **55** |
| Total Internal Site Generated Trips | | 139 | 3 | 2 | 1 | 12 | 6 | 6 |
| Total External Site Generated Trips | | 1,026 | 48 | 24 | 24 | 95 | 46 | 49 |
| Total Pass-by Trips | | 374 | 9 | 5 | 4 | 33 | 16 | 17 |
| **Net New /Diverted Site Generated Trips** | | **652** | **38** | **18** | **20** | **62** | **30** | **32** |

**Distribution and Assignment of Site Generated Traffic**

Traffic generated by the proposed The Point at Wheeler Landing residential and retail development was assigned to the roadway network by considering existing travel patterns obtained from AM and PM peak hour counts at the intersections of Hwy 101 /Hemlock Street. ***Figure 6*** displays the trip distribution that was assumed from these vehicle trips generated by this development. It was found that approximately 60 percent of the traffic travels south along Hwy 101 and 40 percent travels north. ***Figure 7*** shows these trip distributions and assignments of traffic associated with the proposed The Point at Wheeler Landing during the peak hours /30$^{th}$ HV.

**Total Future 2008 Traffic Volumes and Levels of Service**

Total future 2010 peak hour /30th HV traffic volumes at the study area intersections were estimated by adding the background future traffic volumes displayed in ***Figure 5***, to the volumes that would be generated by buildout of The Point at Wheeler Landing shown in ***Figure 6***. Total future 2010 peak hour /30th HV traffic volumes with buildout of The Point at Wheeler Landing are shown in ***Figure 7***. The results of the intersection LOS analyses for total future 2010 traffic volumes are shown in ***Table 6***. These results indicate that with The Point at Wheeler Landing fully built out, delays will degrade slightly from future background conditions. However, the study area intersections will continue to operate at LOS C with V/C ratio of 0.01 or better during weekday peak hour /30th HV peak hours, which meets ODOT standards for intersection performance.

**Table 4: Future Background 2010 Weekday PM Peak Hour /30th DH Levels of Service**

| Intersection | AM Peak Hour | | | PM Peak Hour | | |
|---|---|---|---|---|---|---|
| | Minor Street Stop Control | | | | | |
| | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS | Avg Vehicle Delay (Sec/Veh) | V/C Ratio | LOS |
| **Hwy 101 /Hemlock Street** (Critical Approach: WB) | - | - | - | EB-15.1<br>WB-14.5 | 0.01<br>0.01 | C<br>B |

**SITE ACCESS AND CIRCULATION PLAN**

***Figure 1*** and ***Figure 1A*** show the vicinity and proposed site plan. The applicant is proposing to construct 44 two-story townhomes, 14 three-story live-work townhomes which will have approximately 570 GSF of retail space and a garage on the first floor and the upper two levels as a residence and 3 two-story buildings that will have a total of approximately 19,077 GSF retail space. Access to the site will be provided via a proposed west leg to the existing intersection of Highway 101 /Hemlock Street (see attached Figures).

**CONCLUSIONS**

Based on the results of the analysis described in this report, it is concluded that the proposed The Point at Wheeler Landing development can be constructed without adversely affecting traffic operations or safety in the vicinity of the site. Furthermore, key intersection and roadways in the study area can operate at acceptable Levels of Service when this development is built out. No specific off-site roadway improvements are recommended to accommodate this development or mitigate its impact.

If you have any questions relating to the data or analyses discussed in this report, please contact me directly.

Sincerely,

Rob Morast

Robert Morast, P.E.
Transportation Engineer

Attachments

REGISTERED PROFESSIONAL ENGINEER
16,451
OREGON
MAR. 18, 19[illegible]
ROBERT J. MORAST
8/29/07

EXPIRES 12-31-08

CTS ENGINEERS

April 1, 2008

**Project No. OR07.031.T01**
**The Point At Wheeler Landing**

Doug W. Hooper
City Manager
PO Box 177
Wheeler OR 97147

**RE: Addendum to the Traffic Impact Study Dated August 29, 2007 for The Point At Wheeler Landing on the west side of Hwy 101 /SW Hemlock Street Intersection in Wheeler, OR.**

Dear Mr. Hooper,

This letter is an addendum to the original traffic study dated August 29th 2007. It includes analysis of recent five years of crash history, and right and left turn lane criteria for the proposed The Point at Wheeler Landing development. The development is located on the west side of Hwy 101 /Hemlock Street on a site of approximately 8.9 acres zoned water related commercial / general commercial. As mentioned in the original report, this development will consist of 44 two-story townhomes, 14 three-story live-work townhomes which will have approximately 570 GSF of retail space and a garage on the first floor and the upper two levels as a residence and 3 two-story buildings that will have a total of approximately 19,077 GSF retail space. ***Figure 1*** contains a vicinity map of the proposed site and surrounding roadway system. Access to the site will be provided via a proposed west leg to the existing intersection of Highway 101 /Hemlock Street (see attached Figures). ***Figure 1A*** presents a site plan of the proposed development.

**Traffic Safety**
Collision records for the most recent five years of available data (Jan. 2002 to Dec. 2006) were obtained from Oregon Department of Transportation (ODOT) for the intersection of Highway 101 /Hemlock Street in the vicinity of the project site. This was analyzed to determine if traffic safety problems exist at the study area intersection in the vicinity of the site. ***Figure 4*** shows the location and type of reported incidents. A total of 4 crashes were reported, which equates to an average annual crash rate of 0.39 crashes per million entering vehicles. This number and rate of reported collisions are typical of the crashes experienced on similar roadways throughout the State.

**Left Turn Lane Warrants**
An analysis was conducted to determine if northbound vehicles turning left into the proposed site from Hwy 101 would meet warrants for requiring separate left turn lanes under total future 2010 traffic conditions. These warrants are based on the number of vehicles turning left, the posted speed limit or design speed, advancing volumes, and the opposing conflicting volumes during the critical PM peak hour. As shown in ***Table 1*** and based on Criterion 1 (vehicular volume) in the ***ODOT Left Turn Lane Warrant Criteria***, these projected future 2010 traffic volumes at the proposed site access intersections (Hwy 101 /Hemlock Street) ***meets*** warrant criterion for requiring a separate northbound left turn lane under both 45mph and 25mph zoned zone.


[illegible]
HILLSBORO OR [illegible]

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HILLSBORO, OR
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**Table 1: Results of Left Turn Warrant Analyses under Total Future 2010 Volume Conditions (Buildout of the The Point at Wheeler Landing)**

| Intersection | Total Future 2010 PM Peak Hour | | | ODOT Criteria | |
|---|---|---|---|---|---|
| | Design Speed (mph) | Left Turns (vph) | Opposing Plus Advancing Volume (vph/Lane) | Minimum Criteria (Lt Turns-vph) | Warrant Met? |
| Northbound Hwy 101 at Hemlock Street (*Site Access*) | 45 | 26 | 728 | 10 | Yes |
| Northbound Hwy 101 at Hemlock Street (*Site Access*) | 25 | 26 | 728 | 12 | Yes |

**In-bound Vehicle Queue Length Analysis**
Queuing analysis was conducted at the site's access intersections of Hwy 101 /Hemlock Street. As mentioned earlier, the proposed development meets warrants for a northbound left turn lane. Intersection operations reported in the original traffic study were evaluated using the methodology outlined in the 2000 Highway Capacity Manual (HCM). However, for the purpose of queue length, AASHTO 2-minute Rule or SimTraffic analysis software cannot be used at this intersection due to the railroad crossing running parallel to the Hwy 101. In discussions with the Port of Tillamook Bay staff, trains passing through this area are approximately 1,000 to 1,500 feet (20-30 cargo cars) in length and travel at about 10 mph within the City limit. It was also found that a train with 30 cargo cars in length crosses the railroad crossing in about 2-3 minutes. Approximately 30 seconds before arriving at the crossing, the system which closes the gates at a crossing works by sensing the presence of a train on the tracks at a fixed distance from the crossing. That distance is set such that a train moving at the maximum permissible speed (10 mph) for that section of track will not reach the crossing until enough time has passed to allow the gates to fully close (approximately 500 feet at Hemlock Street). After several seconds of flashing lights and ringing bells, the crossing gates begin to lower, which usually takes about 5-10 seconds. This phenomenon can also be verified as follows:

$$C_t = \left[\frac{L_T + D_L + W_I}{S_T}\right]$$

Where:
$C_t$ = Clearance time in seconds
$L_T$ = Length of the Train in feet (1,500 feet)
$D_L$ = Distance of the detection loop from the road crossing in feet (about 500 feet)
$W_I$ = Width of an intersection (approximately 50 feet)
$S_T$ = Speed of the Train is 10mph. (10 mph x 5280) /3600 = 14.67 ft/sec

$$C_t = \left[\frac{1{,}500 + 500 + 50}{14.67}\right]$$

$$C_t = 140\ sec \cong 2.33\ min \leq \mathbf{3\ minutes}$$

**(4 minutes under "worst case" scenario)**

As shown in ***Figure 7***, total future 2010 traffic volume with full buildout of The Point at Wheeler Landing will generate 26 northbound left turns entering into the site (Hemlock Street) from Hwy 101 during the PM peak hour. This equates to approximately 1 vehicle for every 2 minutes. However, due to railroad crossing at the site access a 4-minute "worst case" scenario is assumed which equates to a queue of 2 vehicles (50 feet). Furthermore, assuming there is a passenger car and a delivery truck in the queue, a storage length of 50-75 feet long should be sufficient.

**Right Turn Lane Warrants**
An additional analysis was also conducted to determine whether or not increased traffic along Hwy 101 at the proposed site access (Hwy 101 /Hemlock Street) intersections would meet warrants for requiring a separate right turn lane under total future 2010 traffic conditions with full buildout of the proposed development plan. As shown in ***Table 2***, and based on Criterion 1 (vehicular volume) in the ***ODOT Right Turn Lane Warrant Criteria***, the projected future 2010 southbound right turning vehicles in to the site from Hwy 101 ***do not meet*** the warrant criteria for requiring a separate right turn lane.

Similar to the analysis for left turn lane storage length, a right turning vehicle may also experience a waiting time of 3-4 minutes. Also, shown in ***Figure 7***, total future 2010 traffic volume with full buildout of The Point at Wheeler Landing will generate 23 northbound right turns entering into the site (Hemlock Street) from Hwy 101 during the PM peak hour. This equates to approximately 1 vehicle for every 2 minutes and 2 vehicles (50 feet) queue for the entire waiting period. Thus, it is desirable to provide a right-turn pocket to avoid blocking of thru-vehicles along Hwy 101 at Hemlock Street. Finally, it should be noted that the section of Hwy 101 just north of Hemlock Street is marked with a dashed yellow centerline for northbound vehicles only (indicating that passing is permitted. Furthermore, with the buildout of the proposed development, the segment of Hwy 101 will adopt a more urban character. Thus, it is recommended that ODOT should consider re-striping this section of Hwy 101 to a double solid yellow line and investigate moving the existing posted speed limit of 45 mph further to the north (approximately 500-1,000 feet).

**Table 2: Results of Right Turn Warrant Analyses under Total Future 2010 Volume Conditions (Buildout of the The Point at Wheeler Landing)**

| Intersection | Total Future 2010 PM Peak Hour | | | ODOT Criteria | |
|---|---|---|---|---|---|
| | Design Speed (mph) | Right Turns (vph) | Design Hour Volume (vph/Lane) | Minimum Criteria (Rt Turns-vph) | Warrant Met? |
| Southbound Hwy 101 at Hemlock Street (*Site Access*) | 45 | 23 | 393 | 29 | No |
| Southbound Hwy 101 at Hemlock Street (*Site Access*) | 25 | 23 | 393 | 61 | No |

**Outbound Vehicle Queuing Analysis**
Based on the request from ODOT staff, CTS Engineers conducted queuing analysis along the site's proposed access (Hemlock Street) location. A review of the site plan for the proposed development shows there is approximately 30 feet (≈ 1 Passenger car or small delivery truck) of storage between the highway intersection and the train dynamic envelope. As discussed in the original report, full buildout of the proposed The Point at Wheeler Landing generates about 48 external vehicle trips during the AM peak hour of which 24 are entering vehicles and 24 are exiting vehicles, and 95 vehicle trips during the PM peak hour of which 46 are entering vehicles and 49 are exiting vehicles as shown in the trip generation ***Table 5*** of the original report. Based on intersection capacity analysis (HCM 2000) results, queuing along the unsignalized intersections will be minimal (0-1 vehicles). Intersection capacity analysis worksheet is attached as an *Appendix* to this report which shows 95th percentile queues. In brief, the potential for queue spillback to the railroad track is limited.

However, it is recommended that the applicant install a warning sign (R8-8 in MUTCD) "DO NOT STOP ON TRACKS" and standard railroad (RR) pavement markings to keep cars from stacking on the tracks. It is also recommended that a storage space (W10-11 in MUTCD) sign supplemented by a word message storage distance (W10-11a) "30 FEET BETWEEN TRACKS & HIGHWAY" should be used. These signs should be mounted in advance of the railroad crossing to advice drivers of the space available for vehicle storage between the highway intersection and the highway-rail grade crossing. Furthermore, a storage space (W10-11b) "30 FEET BETWEEN HIGHWAY & TRACKS BEHIND YOU" may be mounted at the highway intersection under the STOP sign intersection to remind drivers of the storage space between the tracks and the highway intersection.

**CONCLUSIONS**

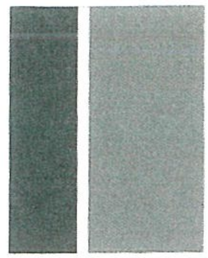
Based on the results of the analyses described in this memo, it is concluded that the proposed The Point at Wheeler Landing can be constructed without adversely affecting traffic operations or safety in the vicinity of the site. A possible mitigation to improve traffic operations at the intersection of Hemlock Street /Hwy 101 would be to install a separate northbound left turn lane and a southbound right turn lane. Furthermore, it is also recommended that ODOT should consider re-striping this section of Hwy 101 to a double solid yellow line and investigate moving the existing posted speed limit of 45 mph further to the north. With full buildout of the proposed development key intersections and roadways in the study area operate at acceptable LOS and capacity as reported in the original report. No other specific off-site roadway improvements are recommended to accommodate this development or mitigate its impact.

If there are any questions related to the data or analyses contained in this memo, please contact Arshad Syed.

Sincerely,

Robert J Morast, P.E., P.T.O.E.
Transportation Engineer

REGISTERED PROFESSIONAL ENGINEER
6451
OREGON
MAR. 16, 1992
ROBERT J. MORAST
Expires: 12/31/2008



HOTEL

Com. Building