

## COLUMN DESIGN FOR FLEXURAL (NON DUCTILE UN-BRACED)

### DESIGN CODE ACI 318-2019

#### General Data

<i>ColumnNo</i> :	C9	
<i>Level</i> :	0 m to 6.2m	
<i>FrameType</i> := 1	-----	1- for Non Ductile, 2 for Special
<i>B</i> := 24 <i>in</i>	-----	Width of the Column
<i>D</i> := 36 <i>in</i>	-----	Depth of the Column
<i>f'c</i> := 3.5 <i>ksi</i>	-----	Grade of Concrete (Cylindrical Strength)
<i>fy</i> := 60 <i>ksi</i>	-----	Grade of Main Reinforcement
<i>fyt</i> := 60 <i>ksi</i>	-----	Grade of Secondary Reinforcement
<i>Cc</i> := 2 <i>in</i>	-----	Nominal Cover to Beam Tension Reinforcement
<i>Es</i> := 20009 <i>ksi</i>	-----	Modulus of elasticity of reinforcement
$\phi_{BM}$ := 0.65	-----	Capacity reduction factor for Bending Moment
$\phi_{shear}$ := 0.75	-----	Capacity reduction factor for Shear and torsion
<i>ptmax</i> := 4	-----	Maximum % reinforcement
<i>ptmin</i> := 1	-----	Minimum % reinforcement
$\epsilon_{cu}$ := 0.003	-----	Permissible Strain in Concrete
<i>lux</i> := 212.6 <i>in</i>	-----	Clear Floor Height along Major Direction@lux
<i>luy</i> := 212.6 <i>in</i>	-----	Clear Floor Height along Minor Direction@luy
<i>NoOfFloors</i> := 1	-----	Number of Floors considered after merged of levels
<i>NoOfColumnsInGroup</i> := 1	-----	Number of column in Group
<i>ColumnType</i> := 1	-----	1- for Braced, 2 for Un-Braced
<i>MinimumEccentricityCheck</i>	-----	Simultaneously (Both Axis)
<i>CodeDefinedDBRatio</i> := 4		

$$DBRatio := \begin{cases} \text{if } \frac{D}{B} < 4 \\ \quad \text{"DesignAsColumn"} \\ \text{else} \\ \quad \text{"DesignAsWall"} \end{cases} = \text{"DesignAsColumn"}$$

#### Reinforcement Provided

$$\begin{aligned} \phi 1 &:= 1 \text{ in} \\ N1 &:= 4 \\ \phi 2 &:= 0.75 \text{ in} \\ N2 &:= 18 \end{aligned}$$

$$A_{st} := \frac{\pi \cdot \phi 1^2}{4} \cdot N1 + \frac{\pi \cdot \phi 2^2}{4} \cdot N2 = 11.094 \text{ in}^2 \quad \text{----- Area of Reinforcement Provided}$$

#### Flexural Design (Analysis Forces)

<i>AnalysisReferenceNo</i>	4040
<i>LoadCombination</i>	[8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) - (LOAD 3: LOAD CASE 3 EQ-X)
<i>CriticalLoacation</i>	Top Joint
<i>Put</i> := 618.49 <i>kip</i>	----- Ultimate moment from analysis at Top Node

$M_{uxt} := 80.24 \text{ kip} \cdot \text{ft}$	-----	Ultimate moment from analysis along D at Top Node
$M_{uyt} := -718.01 \text{ kip} \cdot \text{ft}$	-----	Ultimate moment from analysis along B at Top Node
$V_{uxt} := -51.04 \text{ kip}$	-----	Ultimate Shear from analysis along D at Top Node
$V_{uyt} := -11.68 \text{ kip}$	-----	Ultimate Shear from analysis along B at Top Node
$P_{ub} := 618.49 \text{ kip}$	-----	Ultimate moment from analysis at Bottom Node
$M_{uxb} := -157.3 \text{ kip} \cdot \text{ft}$	-----	Ultimate moment from analysis along D at Bottom Node
$M_{uyb} := 319.91 \text{ kip} \cdot \text{ft}$	-----	Ultimate moment from analysis along B at Bottom Node
$V_{uxb} := -51.04 \text{ kip}$	-----	Ultimate Shear from analysis along D at Bottom Node
$V_{uyb} := -11.68 \text{ kip}$	-----	Ultimate Shear from analysis along B at Bottom Node

### Sway Calculation (Stability Index)

*For\_Global\_X\_Direction*

*LoadCombination* [13] : 1.2(LOAD 1: LOAD CASE 1) +1.2 (LOAD 2: LOAD CASE 2)-1.2 (LOAD 3: LOAD CASE 3 EQ-X)

$StoreyHeight(A) := 20.34 \text{ ft}$	-----	Height of Story Considered for design
$GravityLoad(B) := 13775.05 \text{ kip}$	-----	Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 0.126 \text{ in}$	-----	Relative Displacement for Story selected
$StoreyShear(D) := 89.96 \text{ kip}$	-----	Story Shear for Story selected

$$StabilityIndex_{QD} := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.079$$

$Check := \begin{cases} \text{if } StabilityIndex_{QD} \leq 0.05 & \text{=} \text{"Sway Frame(UnBraced)"} \\ \text{"Non Sway Frame(Braced)"} & \text{----- Clause 6.6.4.4.1} \\ \text{else} & \text{----- Clause 6.6.4.3} \\ \text{"Sway Frame(UnBraced)"} & \end{cases}$

*For\_Global\_Y\_Direction*

*LoadCombination* [13] : 1.2(LOAD 1: LOAD CASE 1) +1.2 (LOAD 2: LOAD CASE 2)+1.2 (LOAD 3: LOAD CASE 3 EQ-Y)

$StoreyHeight(A) := 20.34 \text{ ft}$	-----	Height of Story Considered for design
$GravityLoad(B) := 13775.05 \text{ kip}$	-----	Summation of Axial Load of Story selected
$RelativeDisplacement(C) := 0.09 \text{ in}$	-----	Relative Displacement for Story selected
$StoreyShear(D) := 67.47 \text{ kip}$	-----	Story Shear for Story selected

$$StabilityIndex_{QB} := \frac{GravityLoad(B)}{StoreyHeight(A)} \cdot \frac{RelativeDisplacement(C)}{StoreyShear(D)} = 0.075$$

$Check := \begin{cases} \text{if } StabilityIndex_{QB} \leq 0.05 & \text{=} \text{"Sway Frame(UnBraced)"} \\ \text{"Non Sway Frame(Braced)"} & \text{----- Clause 6.6.4.3} \\ \text{else} & \\ \text{"Sway Frame(UnBraced)"} & \end{cases}$

### Effective Length Factor

$EffectiveLengthFactorAlongD := 1.66$

$EffectiveLengthFactorAlongB := 1.32$

### Calculation of Slenderness Check

Along D:

$$EffectiveLengthD := EffectiveLengthFactorAlongD \cdot l_{uy} = 352.916 \text{ in}$$

$$A := D \cdot B = 864 \text{ in}^2 \quad \text{-----} \quad \text{Area of Cross Section}$$

$$I_{xxD} := B \cdot \frac{D^3}{12} = (9.331 \cdot 10^4) \text{ in}^4 \quad \text{-----} \quad \text{Moment of Inertia}$$

$$rD := \sqrt{\left(\frac{I_{xxD}}{A}\right)} = 10.392 \text{ in} \quad \text{-----} \quad \text{Radius of Gyration}$$

$$k_D := EffectiveLengthFactorAlongD = 1.66$$

$$Actual\_Slender\_D := \frac{EffectiveLengthD}{rD} = 33.959$$

$$LerPerD := 22 \quad \text{-----} \quad \text{Clause 6.2.5.1(a)}$$

$$Check1 := \begin{cases} \text{if } LerPerD > Actual\_Slender\_D \\ \quad \text{“Column Not Slender Along B”} \\ \text{else if } Actual\_Slender\_D < 100 \\ \quad \text{“Slender\_Use Approximate method”} \\ \text{else} \\ \quad \text{“Slender\_P–delta effect (revise)”} \end{cases} = \text{“Slender\_Use Approximate method”}$$

Along B:

$$EffectiveLengthB := EffectiveLengthFactorAlongB \cdot l_{ux} = 280.632 \text{ in}$$

$$A := D \cdot B = 864 \text{ in}^2 \quad \text{-----} \quad \text{Area of Cross Section}$$

$$I_{xxB} := D \cdot \frac{B^3}{12} = (4.147 \cdot 10^4) \text{ in}^4 \quad \text{-----} \quad \text{Moment of Inertia}$$

$$rB := \sqrt{\left(\frac{I_{xxB}}{A}\right)} = 6.928 \text{ in} \quad \text{-----} \quad \text{Radius of Gyration}$$

$$Actual\_Slender\_B := \frac{EffectiveLengthB}{rB} = 40.506$$

$$LerPerB := 22 \quad \text{-----} \quad \text{Clause 6.2.5.1(a)}$$

$$Check1 := \begin{cases} \text{if } LerPerB > Actual\_Slender\_B \\ \quad \text{“Column Not Slender Along B”} \\ \text{else if } Actual\_Slender\_B < 100 \\ \quad \text{“Slender\_Use Approximate method”} \\ \text{else} \\ \quad \text{“Slender\_P–delta effect (revise)”} \end{cases} = \text{“Slender\_Use Approximate method”}$$

### Calculation of Slenderness Moment - Un-Braced (Sway) Frame

$$\beta_{dns} := 0.6 \quad \text{-----} \quad \text{Clause 6.6.4.4.4}$$

$$E_c := 57000 \cdot \sqrt{\left(\frac{f'_c}{\text{psi}}\right)} = 3372165.476 \quad \text{-----} \quad \text{Clause 19.2.2.1}$$

$$I_g := D \cdot \frac{B^3}{12} = (4.147 \cdot 10^4) \text{ in}^4 \quad \text{-----} \quad \text{Moment of Inertia of Section}$$

$$EI := 0.288675 \cdot E_c \cdot I_g = (4.037 \cdot 10^{10}) \text{ in}^4$$

$$\delta_{ns} := \left\| \begin{array}{l} \text{if } StabilityIndex\_QD > 1 \\ \quad \left\| \begin{array}{l} 1.5 \\ \text{else} \\ \frac{1}{(1 - StabilityIndex\_QD)} \end{array} \right\| \\ \end{array} \right\| = 1.086 \quad \text{-----} \quad \text{Clause 6.6.4.5.3}$$

Along D:

$$M1s := 15.06 \text{ kip} \cdot \text{ft} \quad \text{-----} \quad \text{factored end moment on compression member at the end at which M1 acts, due to loads that cause appreciable side-sway. Clause 6.6.4.6}$$

$$M2s := -155.91 \text{ kip} \cdot \text{ft}$$

Columns	Level	Analysis No	Load Case	Location (ft)	P (kip)	Mx (kip-ft)	My (kip-ft)	ShearX (kip)	ShearY (kip)	Torsion (kip-ft)
C9	20.34117 ft	4040	LOAD 1: LOAD CASE 1	0	-2.59	0.16	-0.07	0.57	-0.25	0
			LOAD 1: LOAD CASE 1	20.34	-2.59	5.19	11.49	0.57	-0.25	0
			LOAD 2: LOAD CASE 2	0	126.3	-0.44	64.43	-10.81	-0.41	0
			LOAD 2: LOAD CASE 2	20.34	126.3	7.85	-155.3	-10.81	-0.41	0
			LOAD 3: LOAD CASE 3 EQ-X	0	0.06	155.91	1.91	-0.15	8.41	0
			LOAD 3: LOAD CASE 3 EQ-X	20.34	0.06	-15.06	-1.05	-0.15	8.41	0
			LOAD 4: LOAD CASE 4 EQ-Y	0	2.65	-0.08	67.12	-5.35	0	0
			LOAD 4: LOAD CASE 4 EQ-Y	20.34	2.65	0.01	-41.7	-5.35	0	0

The above value is from the Basic load case

$$\delta s M1s := \delta_{ns} \cdot M1s = 16.353 \text{ kip} \cdot \text{ft}$$

$$\delta s M2s := \delta_{ns} \cdot M2s = -169.292 \text{ kip} \cdot \text{ft}$$

$$M1 := 80.24 \text{ kip} \cdot \text{ft}$$

$$M2 := -157.3 \text{ kip} \cdot \text{ft}$$

$$M1ns := (M1 - M1s) = 65.18 \text{ kip} \cdot \text{ft}$$

$$M2ns := (M2 - M2s) = -1.39 \text{ kip} \cdot \text{ft}$$

factored end moment on compression member at the end at which M1 acts, due to loads that cause no appreciable sides-way. Clause 6.6.4.6

Columns	Level	Analysis No	Load Comb Ref No	Load Comb Analysis No	Load Comb Description	Location (ft)	P (kip)	Mx (kip-ft)	My (kip-ft)	ShearX (kip)	ShearY (kip)	Torsion (kip-ft)
C9	20.34117 ft	4040	1	100	1.4 (LOAD 1: LOAD CASE 1)	0	-3.63	0.22	-0.09	0.8	-0.35	0
			1	100	1.4 (LOAD 1: LOAD CASE 1)	20.34	-3.63	7.27	16.08	0.8	-0.35	0
			2	101	1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: LOAD CASE 2)	0	198.97	-0.51	103.01	-16.61	-0.95	0
			2	101	1.2 (LOAD 1: LOAD CASE 1) +1.6 (LOAD 2: LOAD CASE 2)	20.34	198.97	18.78	-234.69	-16.61	-0.95	0
			3	102	1.2 (LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: LOAD CASE 2)	0	60.04	-0.03	32.14	-4.72	-0.5	0
			3	102	1.2 (LOAD 1: LOAD CASE 1) +0.5 (LOAD 2: LOAD CASE 2)	20.34	60.04	10.15	-63.87	-4.72	-0.5	0
			4	103	1.2 (LOAD 1: LOAD CASE 1)	0	-3.11	0.19	-0.08	0.68	-0.3	0
			4	103	1.2 (LOAD 1: LOAD CASE 1)	20.34	-3.11	6.23	13.78	0.68	-0.3	0
			6	105	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD CASE 2)	0	494.9	154.79	259.37	-41.09	5.79	0
			6	105	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD CASE 2)	20.34	494.9	37.08	-576.29	-41.09	5.79	0
			7	106	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD CASE 2)	0	497.48	-1.19	324.58	-46.3	-2.62	0
			7	106	4 (LOAD 1: LOAD CASE 1) +4 (LOAD 2: LOAD CASE 2)	20.34	497.48	52.15	-616.94	-46.3	-2.62	0
			8	107	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2)	0	618.49	-157.3	319.91	-51.04	-11.68	0
			8	107	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2)	20.34	618.49	80.24	-718.01	-51.04	-11.68	0
			9	108	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2)	0	615.9	-1.31	254.7	-45.83	-3.27	0
			9	108	5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2)	20.34	615.9	65.18	-677.36	-45.83	-3.27	0



*LoadCombination*

[8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) -  
(LOAD 3: LOAD CASE 3 EQ-X)

*CriticalLocation*

Bottom Joint

$$P_u := 618.49 \text{ kip}$$

----- Factored Axial force

$$M_{ux1} := -157.3 \text{ kip} \cdot \text{ft}$$

----- Factored Bending Moment along Major Direction

$$M_{uy1} := 319.91 \text{ kip} \cdot \text{ft}$$

----- Factored Bending Moment along Minor Direction

$$A_g := B \cdot D = 864 \text{ in}^2$$

----- Cross Section Area

$$A_{st} = 11.094 \text{ in}^2$$

----- Area of Reinforcement Provided

$$p_t := \frac{A_{st} \cdot 100}{A_g} = 1.284$$

----- % Reinforcement Provided

$$\phi P_n \max := 0.8 \cdot \phi B M \cdot (0.85 \cdot f'_c \cdot (A_g - A_{st}) + (A_{st} \cdot f_{y\_1})) = 1665.571 \text{ kip}$$

----- Clause 22.4.2.2

$$\text{Check} := \begin{cases} \text{if } P_u \leq \phi P_n \max & = \text{"Ok"} \\ \text{else} & \\ \text{"Revise"} & \end{cases}$$

### Minimum Ast Calculation

$$f_{y\_2} := \begin{cases} \text{if } f_y > 80 \text{ ksi} & = 60 \text{ ksi} \\ 80 \text{ ksi} & \\ \text{else} & \\ f_y & \end{cases}$$

----- Clause 22.4.2.1

*LoadCombination*

[8] : 5 (LOAD 1: LOAD CASE 1) +5 (LOAD 2: LOAD CASE 2) -  
(LOAD 3: LOAD CASE 3 EQ-X)

$$P_{u\_max} := 681.465 \text{ kip}$$

$$A_{stReq} := \frac{0.442 \cdot f'_c \cdot A_g - P_{u\_max}}{(0.442 \cdot f'_c) - (0.52 \cdot f_{y\_2})} = -22.094 \text{ in}^2$$

----- Clause 22.4.2.2

$$p_t := \begin{cases} \text{if } A_{stReq} \leq 0 & = 0 \\ 0 & \\ \text{else} & \\ \frac{A_{stReq}}{A_g} & \end{cases}$$

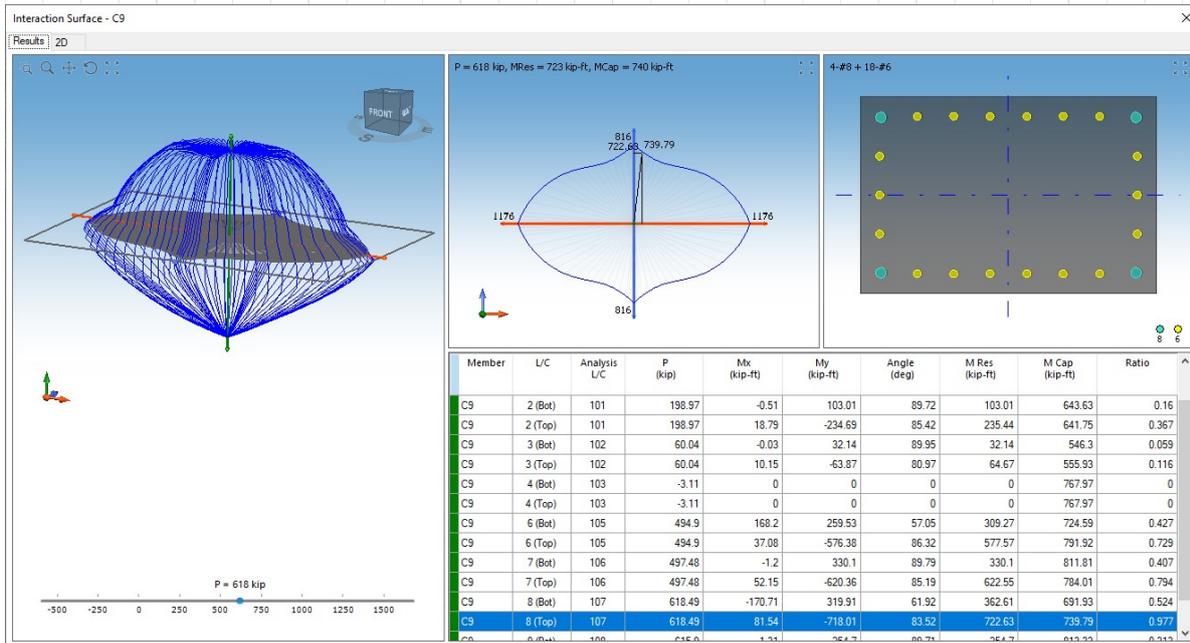
$$p_{tmin} := \max(0.5, p_t) = 0.5$$

----- Clause 10.3.1.2

$$UserDefinedp_t := 1$$

$$Finalp_t := \max(p_{tmin}, UserDefinedp_t) = 1$$

# PM Curve



## Resultant Moment (Combined Action)

### Moment Capacity Check

$$P_{\text{calculated}} := \frac{A_{st} \cdot 100}{A_g} = 1.28 \quad \text{-----} \quad \% \text{ Reinforcement Provided}$$

$$\text{LoadAngle} := 83.52 \quad \text{-----} \quad \tan^{-1} (M_{uy\_Final} / M_{ux\_Final})$$

$$M_{res} := \sqrt{M_{ux\_Final}^2 + M_{uy\_Final}^2} = 722.535 \text{ kip} \cdot \text{ft} \quad \text{-----} \quad \text{Resultant Bending Moment}$$

$$\phi M_{cap} := 739.79 \text{ kip} \cdot \text{ft} \quad \text{-----} \quad \text{Moment Capacity from PM Curve}$$

$$\text{CapacityRatio} := \frac{M_{res}}{\phi M_{cap}} = 0.977 \quad \text{-----} \quad \text{Clause 10.4.2}$$

$$\text{Check} := \left\{ \begin{array}{l} \text{if CapacityRatio} < 1 \\ \quad \text{“Ok”} \\ \text{else} \\ \quad \text{“Revise”} \end{array} \right\} = \text{“Ok”} \quad \text{-----} \quad \text{Clause 10.4.2}$$

## Design Data for All Load Combination

Column/Wall : C9  
 Level : 0 ft To 20.34117 ft  
 Frame Type = Non-Ductile  
 Design Code = ACI 318 - 19  
 Grade Of Concrete (f'c) = 3.5 ksi  
 Grade Of Steel (Main) = 60 ksi  
 Grade Of Steel (Shear) = 60 ksi  
 Column B = 24 in  
 Column D = 36 in  
 Clear Cover = 2 in  
 Pt = 1.29 %

### Design Table :

Member	LOC	L/C	Analysis L/C No	Pu (kip)	Analysis		Mslr or MC		Design			Mcap (kip-ft)	Capacity Ratio
					Mx (kip-ft)	My (kip-ft)	Mx (kip-ft)	My (kip-ft)	Mux (kip-ft)	Muy (kip-ft)	MuRes (kip-ft)		
4040	BOT	1	100	-3.63	0.22	-0.09	-	-	0.00	0.00	0.00	767.43	0
4040	TOP	1	100	-3.63	7.27	16.08	-	-	0.00	0.00	0.00	767.43	0
4040	BOT	2	101	198.97	-0.51	103.01	-0.51	103.01	-0.51	103.01	103.01	643.64	0.16
4040	TOP	2	101	198.97	18.78	-234.69	18.78	-234.69	18.78	-234.69	235.44	641.75	0.367
4040	BOT	3	102	60.04	-0.03	32.14	-0.03	32.14	-0.03	32.14	32.14	546.31	0.059
4040	TOP	3	102	60.04	10.15	-63.87	10.15	-63.87	10.15	-63.87	64.67	555.93	0.116
4040	BOT	4	103	-3.11	0.19	-0.08	-	-	0.00	0.00	0.00	767.97	0
4040	TOP	4	103	-3.11	6.23	13.78	-	-	0.00	0.00	0.00	767.97	0
4040	BOT	6	105	494.90	154.79	259.37	168.20	259.53	168.20	259.53	309.27	718.65	0.43
4040	TOP	6	105	494.90	37.08	-576.29	37.08	-576.38	37.08	-576.38	577.57	788.52	0.732
4040	BOT	7	106	497.48	-1.19	324.58	-1.20	330.09	-1.20	330.09	330.10	810.69	0.407
4040	TOP	7	106	497.48	52.15	-616.94	52.15	-620.37	52.15	-620.37	622.55	779.51	0.799
4040	BOT	8	107	618.49	-157.30	319.91	-170.71	319.91	-170.71	319.91	362.60	685.32	0.529
4040	TOP	8	107	618.49	80.24	-718.01	81.54	-718.01	81.54	-718.01	722.62	732.50	0.987
4040	BOT	9	108	615.90	-1.31	254.70	-1.31	254.70	-1.31	254.70	254.70	802.85	0.317
4040	TOP	9	108	615.90	65.18	-677.36	65.18	-677.36	65.18	-677.36	680.49	741.03	0.918