Executive Summary

The following report is an analysis and confirmation of the lateral force resisting system in the Brunswick School Athletic Building. This building has a definitive long direction, referred to as X, nearly 450 feet spanning NW-SE and a short direction, called Y, 105 feet spanning NE-SW. Location of the directions, X and Y labeling can be seen on the enclosed first floor plan. Roof trusses span the 105 foot direction, and are spaced 35 feet on center. Load combinations are discussed; seismic loads controlled all of the analysis but roof uplift. This building consists of timber trusses on masonry walls; therefore the analysis includes a complete shear and torsion distribution, a check of roof uplift, and the effects of the lateral forces on the foundation. Shear and torsion distribution was performed over the 29 various shear walls, 12 in the short direction and 17 in the long direction. The included first floor plan shows the numbering these walls. Distribution over these walls was determined using an Excel spreadsheet, and results of wind, seismic, and a combination of wind and seismic loads is included. Shear loadings per each wall were calculated, and compared to the maximum shear resistance in 12”, 10”, and 8” CMU walls. All shear loads were well resisted by the shear reinforcement of 2-#9 bars at 16”. Roof uplift loads are distributed to each truss and compared to the dead load of the roof; uplift was found to be minimal compared to dead load. Overturning moment due to the lateral loads in the X and Y directions were calculated and forces resisting them were compared to the dead load of the foundation. The foundation is more than adequate to resist uplift. Overall, every member checked is sufficient to resist the calculated lateral loads.
**Lateral System and Load Path Summary**

The Brunswick School Athletic Building consists largely of masonry walls and a timber truss roof. This building consists of three sections: a hockey rink, a core or “brain” where locker rooms and weight rooms are located, and a basketball court. The following page shows the location of these three sections.

The masonry walls transfer all lateral loads to the foundation. Twenty-nine shear walls are also marked on the following page. Load on the roof is transferred into the walls, and this load, as well as that directly on the exterior walls is distributed to the either the 12 walls resisting load in the long direction, or the 17 walls resisting load in the short direction.
Lateral Load Combinations

Wind load on the building is not divided over the sections, but rather over the faces of the building, but seismic is very different. Because the core section has three floors, it has three separate seismic forces. The rink and court are only one story, and therefore only one seismic force each. The following sketches show wind and seismic forces over the building:

Wind loads in pounds per square foot.

Summation of the wind forces yielded a force in the long direction $P_Y=127.6$ kips, and in the short direction $P_X=666.6$ kips.

Seismic loads by location.

Seismic forces not labeled are $F_h=696.9$ kips and $F_b=439.2$ kips.

Total seismic forces $P_s=1804.5$ kips in either direction.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>X</th>
<th>Y</th>
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<tr>
<td>Unfactored Wind</td>
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<td>126.7 k</td>
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<td>1.3 W</td>
<td>866.6 k</td>
<td>164.7 k</td>
</tr>
<tr>
<td>1.0 E</td>
<td>1804.5 k</td>
<td>1804.5 k</td>
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<tr>
<td>1.3 W + 1.0 E</td>
<td>2671.1 k</td>
<td>1969.2 k</td>
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This chart shows the combinations of these loads.
The critical combination in each direction is $1.3 \text{ W} + 1.0 \text{ E}$.
Shear & Torsion Distribution

Shear and torsion reactions were distributed over the twenty-nine shear walls using an Excel spreadsheet. The spreadsheet calculated the relative rigidity of each wall and compared it to the total rigidity in each direction. The following chart shows the results in each direction:

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<tr>
<th></th>
<th>X</th>
<th>Y</th>
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<tbody>
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<td>Total Relative Rigidity</td>
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<td>Center of Area</td>
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<td>Center of Rigidity</td>
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<td>Eccentricity</td>
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</table>

Centers of Area and Rigidity are marked on the floor plan.

These factors were used to analyze each load case. The loads shown on the previous page were each entered into the spreadsheet, and direct shear, torsional shear, and the combination of direct and torsional shear were calculated for each wall. The wall with the most shear is Wall 1 (located farthest NE), with V=18.2 kips per foot of wall.

Drift due to these forces is also calculated in the spreadsheets. The largest case, combined wind and seismic in the Y direction, shows Δ= 0.00363 inches. Compared to the allowable drift for masonry walls, per IBC 2000, Δ= h/240 = 2.95”, all drift conditions are very small.

The spreadsheets for each loading condition are contained on the next three pages.
### Seismic Shear

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<tr>
<th>Wall #</th>
<th>size (&quot;CMU)</th>
<th>thickness (in)</th>
<th>depth (ft)</th>
<th>height (ft)</th>
<th>x (ft)</th>
<th>h/d</th>
<th>flexibility</th>
<th>Rigidity</th>
<th>R<em>x &amp; R</em>y</th>
<th>r</th>
<th>r^2</th>
<th>Direct V (kips)</th>
<th>Torsion V (kips)</th>
<th>Total V (kips)</th>
<th>Shear/length (kft/ft of wall)</th>
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\[ V_y = 1804.5 \text{ kips} \]
\[ C_{Mx} = 449.75/2 = 224.875 \text{ ft} \]
\[ C_{Rx} = 243.989 \]
\[ C_{My} = 144.67/2 = 72.335 \text{ ft} \]
\[ C_{Ry} = 69.293 \]

\[ \Sigma R = 331.265 \]
\[ \Sigma R_x = 80825.258 \]
\[ \Sigma R_y = 28334.174 \]
\[ \Sigma V = 1804.5 \]
\[ \Sigma V_x = 3491.875 \text{ ft-kips} \]

\[ M = \frac{3491.875 \text{ ft-kips}}{0.161 \text{ kips/ft}^2} \]

\[ e = 3.042 \text{ ft} \]
\[ M = 5489.663 \text{ ft-kips} \]

\[ M/J = 0.026 \]

\[ \Delta x = 3.03E-03 \text{ in} \]

\[ J = \sum r^2 = 214191.819 \text{ ft}^2 \]
\[ E = 1800 \text{ ksi} \]
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\[ V_y = 164.7 \text{kips} \]
\[ \sum V = 331.265 \text{kips} \]
\[ CR_x = \frac{243.998}{M} = 0.997 \text{ft} \]
\[ CR_y = \frac{3148.136}{M} = 1.643 \text{ft} \]

\[ J = \sum r^2 = 1.21 \text{ft}^2 \]
\[ E = 1800 \text{ksi} \]
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<th>depth (ft)</th>
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<td>-11.256</td>
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</table>

\[
V_y = 1969.2 \text{ kips}
\]

\[
CM_x = 449.75/2 = 224.875 \text{ ft}
\]

\[
CR_x = \sum R_x = 243.989
\]

\[
e = 19.114 \text{ ft}
\]

\[
M = 37640.012 \text{ ft-kips}
\]

\[
\bar{M}/J = 0.038 \text{ kips/ft}^2
\]

\[
\Delta x = 3.30E-03 \text{ in}
\]

\[
V_x = 2671.1 \text{ kips}
\]

\[
CM_y = 144.67/2 = 72.335 \text{ ft}
\]

\[
CR_y = \sum R_y = 69.293
\]

\[
e = 3.042 \text{ ft}
\]

\[
M = 8126.040 \text{ ft-kips}
\]

\[
\bar{M}/J = 0.038 \text{ kips/ft}^2
\]

\[
\Delta y = 3.63E-03 \text{ in}
\]

\[
J = \sum r^2 = 214191.819 \text{ ft}^2
\]

\[
E = 1800 \text{ ksi}
\]
Member Loading Checks

The following cases were checked: shear on masonry walls, roof uplift, and load on the foundation. Calculations can be reviewed in the included calculation section of this report.

Shear forces from the Excel spreadsheet in the previous section were checked. It was determined that each wall, reinforced with 2-#9 bars @ 16” could resist 40 kips per foot of wall. The worst case, in Wall 1, was 18.2 kips per foot, well below the allowable limit. All shear walls are sufficient to carry the shear load to the foundation.

The roof uplift was determined due to upward forces and overturning moment in the roof. The maximum uplift force is 2.5 kips per truss, spaced 35 feet on center. The dead load per 35 foot span of roof is 278.4 kips, much higher than the uplift load.

Two cases of overturning moment for the building were analyzed, one in each direction. The moment due to forces in the X direction were 107000 foot-kips, and reactions resisting this moment were 2.26 kips per foot of wall. The dead load per foot in this direction is 329.2 kips, much greater than the uplift on the foundation. Loads in the Y direction induce 93000 foot-kips of moment, or 2.0 kips per foot of wall. Dead load in this direction is 150.6 kips, again much greater than the uplift.
Summary

After analysis, it was determined that all of the shear walls in the Brunswick School Athletic Building are sufficient to carry the direct and torsional shear forces imposed by either wind or seismic loads, as well as a combination of these forces. The significant weight of this building caused seismic loads to control loadings. All roof and foundation uplift forces are more than balanced by dead load forces. Wall deflection values are well within the IBC limits.

The lateral system for this building is quite sufficient for the loads it will encounter.
\[ R = \frac{1}{\Delta} = \frac{3hP}{E_m t d} + \frac{Ph^3}{E_m t d^3} \]

\[ DF = \frac{R}{ER} \quad \text{ASSUME} \ P \ \& \ E_m \ \text{UNIFORM} \]

\[ R = \frac{P}{E_m} \cdot \frac{1}{t} \left[ \left( \frac{h}{d} \right)^3 + 3 \left( \frac{h}{d} \right) \right] \]

\[ \text{RELATIVE} \quad R = \frac{1}{t} \left[ \left( \frac{h}{d} \right)^3 + 3 \left( \frac{h}{d} \right) \right] \]

29 WALLS HAVE CMU & FOOTING

12 IN \ Y
17 IN \ X

WIND STORY FORCE

\[ \theta = 29^\circ \]

\[ P_x = \left( 10.7 + 11.1 \text{ psf} \right) \left( 15' \times 4'49\frac{1}{2}" \right) + \left( 18.24 + 11.4 \text{ psf} \right) \left( 14' \times 4'49\frac{1}{2}" \right) + 7.2 \cos 29^\circ \left( 29' \times 4'49\frac{1}{2}" \right) + 2.1 \cos 29^\circ \left( 29' \times 4'49\frac{1}{2}" \right) \]

\[ P_x = 1515 \text{k} + 186.0 \text{k} + 821 \text{k} + 246.4 \text{k} \]

\[ P_x = 666.6 \text{k} \]
\[ R_y = (19.2 + 12.1 \text{ psi}) (29' \times 105' 2'') + (11.4 + 18.2 \text{ psi})(14' \times 105' 2'') + (10.7' + 17.1 \text{ psi})(15' \times 105' 2'') \]
\[ = 47.7 \text{ k} + 53.6 \text{ k} + 35.4 \text{ k} \]
\[ = 136.7 \text{ k} \]

**Wall Notes:**

If wall is partially under ground:
- Disregard, use full size

1 → 12 → t → Actual Value
12" CMU = 11-5/8"
10" CMU = 9-7/8"

Excel sheet continues this work...

**Allowable Drift**

\[ \Delta = \frac{f}{1240} \]

\[ = \frac{5.9' \times 12''}{240} = 2.95'' \]

\[ f' = 2000 \text{ psi} \]

\[ E = 900 f' = 1800 \text{ psi} \]
FROM SI:

\[ F_h : \text{LOAD ON HOCKEY RINK} \]
\[ F_b : \text{LOAD ON BASKETBALL COURT} \]

**WEIGHT OF RINK:**

**Roof:**
\[ 378.4 \, \text{k} \, \text{per} \, 35' \times 6 \, \text{spans} = 1670.4 \, \text{k} \]

**Walls:**
\[ (29 + 29/2)(105.2) + 2(29)(247') = 18902.2 \, \text{sf} \times 12 \times \frac{1}{8} \, \text{in}^2 = 2343.9 \, \text{k} \]

\[ C_s = 0.1736 \]

\[ V = C_sW = 4014.3 \, \text{k} \times 0.1736 = 696.9 \, \text{k} \]

\[ F_h = V \, \text{BECAUSE ONLY 1 FLOOR} \]

\[ F_h = 690.9 \, \text{k} \]
WEIGHT OF B-BALL COURT

ROOF: \[ 278.4 \text{k per 35' x 4 spans} = 1113.6 \text{k} \]

WALLS:\[ \begin{align*}
(29 + 29/2) (105.2') + 2(29)(118') &= 11420.2 \text{sf} \\
&\times 12.4 \text{#/1k} \\
&= 1416.1 \text{k} \\
\end{align*} \]

\[ V = C_{SW} W = 2529.7 \times 0.1736 = 439.2 \text{k} \]

\[ F_H = V = 439.2 \text{k} \]

\[ P_E = 439.2 + 696.9 + 199.9 + 446.5 + 52 = 1804.5 \text{k} \]

COMBINED WIND & SEISMIC

<table>
<thead>
<tr>
<th>( x )</th>
<th>( 0.8W )</th>
<th>( 1.3W )</th>
<th>( 1.0E )</th>
<th>( 1.3W + 1.0E )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \frac{X}{101.4 \text{k}} )</td>
<td>533.3 k</td>
<td>866.6 k</td>
<td>1804.5 k</td>
<td>2671.1 k</td>
</tr>
</tbody>
</table>

CRITICAL: \( 1.3W + 1.0E \)

\[ P_x = 2671.1 \text{k} \]

\[ P_y = 1969.2 \text{k} \]
\( M = 866.6 \times (19.17') + 1804.5 \times (50.1') \)
\( = \frac{106,959.2^k}{100.959.2^k} \)

(See spreadsheet for \( V \))

\[ \bar{y}_n = \frac{\sum F_y}{\sum F} = \frac{664.6^k}{(10.7' + 11.1' + 18.2' + 11.4' + (7.2' \times 21.6') \times 0.92)} = 449.9' \]

\[ \bar{y}_E = \frac{\sum F_E}{\sum F} = \frac{(696.9' + 439.2' + 199.9')(58') + 52'(15') + 416.5'(29')}{(696.9' + 439.2' + 199.9' + 52' + 416.5')} \]

\[ = 50.1' \]

Resistance to \( M \)

\[ F_y = \frac{106,959.2^k}{105.2'} = 1016.7^k \]

1016.7^k distributed over 449.9' of wall

2259.9# uplift per foot

\[ = 2.26^k \ll 329.2^k \text{ - dead load per foot of foundation (21')} \]

\[ \therefore \text{ Foundation uplift OK} \]

\[ \frac{2259.9^l}{17' \times 12''} = 26.9 \text{ psi transferred to soil} \]

\( \ll 4000 \text{ psi soil} \)
\[ \Delta W = \frac{106.7^2}{(19.2 + 18.1)(105.2/2)} + \frac{11.4 + 18.244}{10.72 + 17.11} \times 105.2 \]

\[ = 17.8' \]

\[ M = 164.7^2(17.8') + 180.75^2(50.1') \]

\[ = 93333.3^{1/2} \]

**Resistance to M**

\[ F_y = 93333.3^{1/2}/449.9' = 207.5^{1/2} \]

\[ 207.5^{1/2} \text{ OVER } 105.2' \]

197.2# UPLIFT PER FOOT

*Less than that for Rx; OK*
SHEAR

WALL #1 (FAR NW) → WORST SHEAR

\[ V = 18.2 \text{ klfy} \quad \Rightarrow \quad 18.200 \text{ klfy} \]

\[ f_v = \frac{V}{A_e} = \frac{18.200 \text{ klfy}}{13.95 \text{ in} \cdot \text{lfy}} = 130.4 \text{ psi} \]

\[ F_s = 0.5 f_y = 0.5 (60,000) = 30,000 \]

\[ F_{min} = 141.7 \text{ psi} < f_v \]

\[ \text{MEAS: ACI 530.99 T-1992} \]

NEED REINF.

\[ F_v / F_{min} = 1.5 \sqrt{f_{min}} = 67.1 \text{ psi} < f_v \]

2 #9 BARS \( A_v = 2.0 \text{ in}^2 \)

\[ S = \frac{A_v f_s}{b j' f_v} = \frac{2.0 \cdot (24,000)}{(11.625) (0.9) (130.4)} = 35.2'' > 16'' \quad \text{OK} \]

2 #9 @ 16'' \( A_v = 2.0 \text{ in}^2 / \text{lfy} \)

\[ f_v = \frac{A_v f_s}{b j' s} = \frac{2.0 \cdot (24,000)}{(11.625) (0.9) (16 \text{ in})} \]

\[ = 286.7 \text{ psi} \]

\[ V = f_v A_e = 286.7 \text{ psi} \cdot 13.95 \text{ in}^2 \]

\[ = 40.5 \text{ klfy} \]

ALL 12'' WALLS OK \( V < 40\text{ klfy} \)

\[ f_v (10'') = 346.3 \text{ psi} \quad f_v (18'') = 437.2 \text{ psi} \]

\[ V = (346.3)(18.5) = 40.5 \text{ klfy} \quad V = 437.2(9.5) = 40.5 \text{ klfy} \quad \text{OK} \]
\[ \Sigma M_2 = 0 = 105.2 R_1 + (105.2 - 17.5) 7.3 k \]
\[ - (13.2 k + 39.7 k) (9.7') \]
\[ = 22.0 k (17.5') \]
\[ \therefore R_1 = 2.45 k \text{ PER 35' SPAN} \]

\[ \Sigma M_1 = 0 = 105.2 R_2 + 17.5' (7.3 k) \]
\[ - (13.2 k + 39.7 k) 9.7' - 22.0 (105.2 - 17.5) \]
\[ \therefore R_2 = 22.0 k \text{ PER 35' SPAN} \]

Roof Dead Load: 278.4 k PER 35' SPAN

\[ \therefore \text{UPLIFT OK} \]