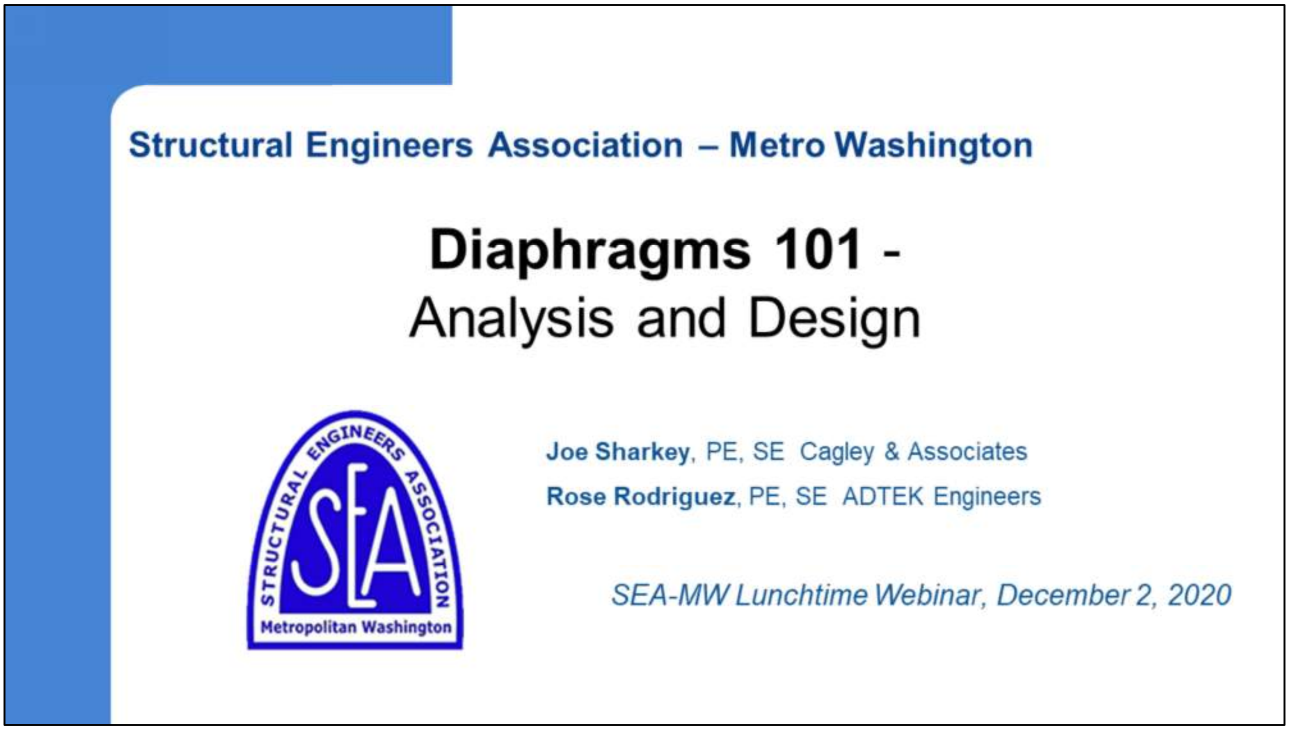


JOE

Good morning and welcome to Diaphragms 101. Today we will be discussing building diaphragms, an often overlooked piece of a buildings lateral system. Being a low seismic area, diaphragm loads for a typical building are usually not large enough to warrant an in depth review. That being said, diaphragm loads are not always generated by seismic loads and later we will discuss some common examples of other types of forces that can cause some pretty sizable loads which should be reviewed.

We will take questions at the end of the presentation… Please write your questions in the chat board.

Rose Rodriguez, Structural Team Leader at ADTEK Engineers, SEAMW Vice-Chair, 25 years of designing in low, moderate, and high seismic regions, and high wind regions.

Joe Sharkey, Project Manager at Cagley & Associates, SEAMW Treasurer, 12 years of design experience. Designs include wood, steel, concrete, and masonry.

Diaphragms 101 - Overview

1. Basics:
    a. Definition
    b. Types of Diaphragms: Materials
    c. Components of a Diaphragm
2. Design requirements
3. Historical analysis techniques
4. Modern analysis techniques
5. Examples: Steel, Wood, Concrete, Seismic
6. Additional Considerations

JOE

To get started today we will:

- Give some definitions
- Describe different types of diaphragms and the components of diaphragms
- Provide design requirements
- Give a brief history of analysis techniques
- Discuss modern analysis techniques
- And then try to spend a majority of our time working through some examples and additional considerations

1. Diaphragm Basics

Define, Review types, Identify components of a diaphragm

Rose
Define what is a diaphragm, types (materials) and components of a diaphragm
... and why do I have to check it.

1A. Diaphragm Basics - Definition

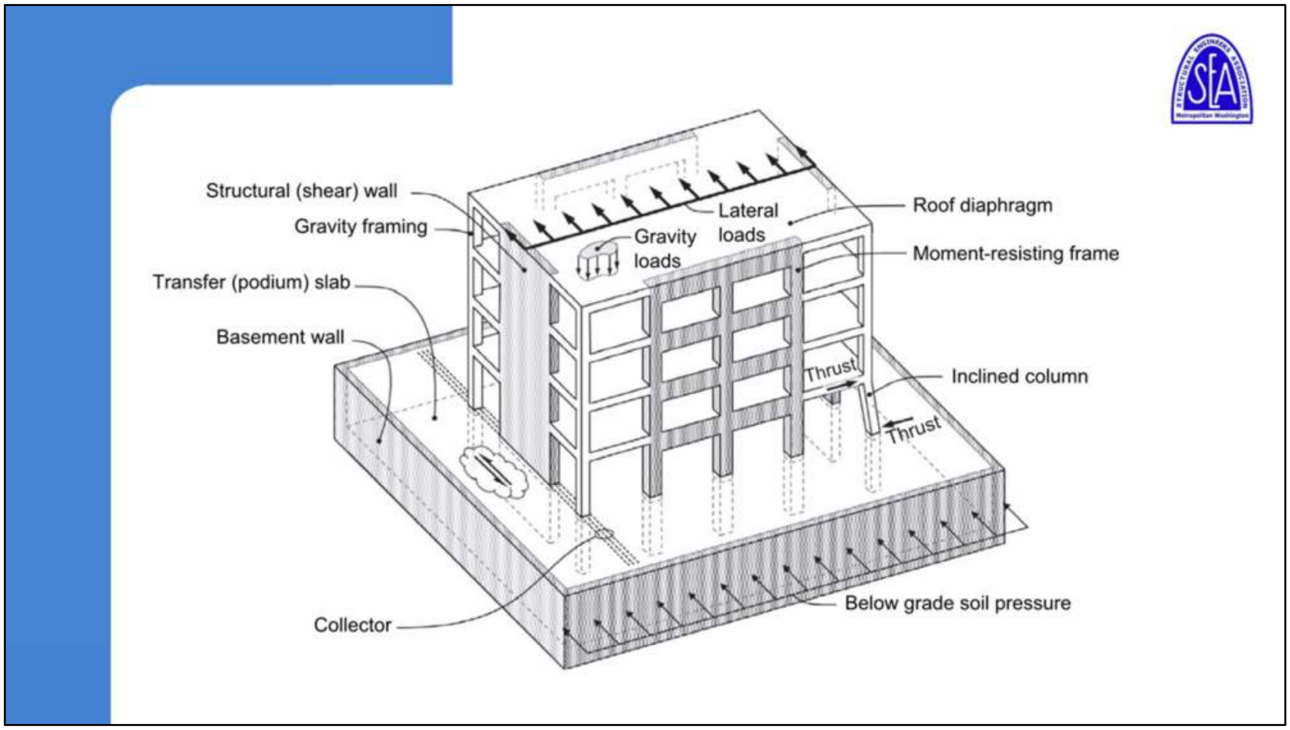
- Horizontal elements - the floors and roofs; they transmit lateral forces from the floor system to the vertical elements of the structural system.
- Diaphragms also tie vertical elements together thus stabilizing these elements.
- Role of the diaphragm - Maintain a continuous LOAD PATH
  - Resist wind, seismic, earth pressure, fluid loads
  - Resist Gravity loads

Rose

Generally, diaphragms are, horizontal elements - solid, planar elements.

THey are floors and roofs (but they can also consist of horizontal trusses)

Role of the diaphragm - Resist wind, seismic, earth pressure, fluid loads



ROSE
Diaphragms resist out of plane forces, mainly gravity loads of self weight and live loads, and wind uplift.

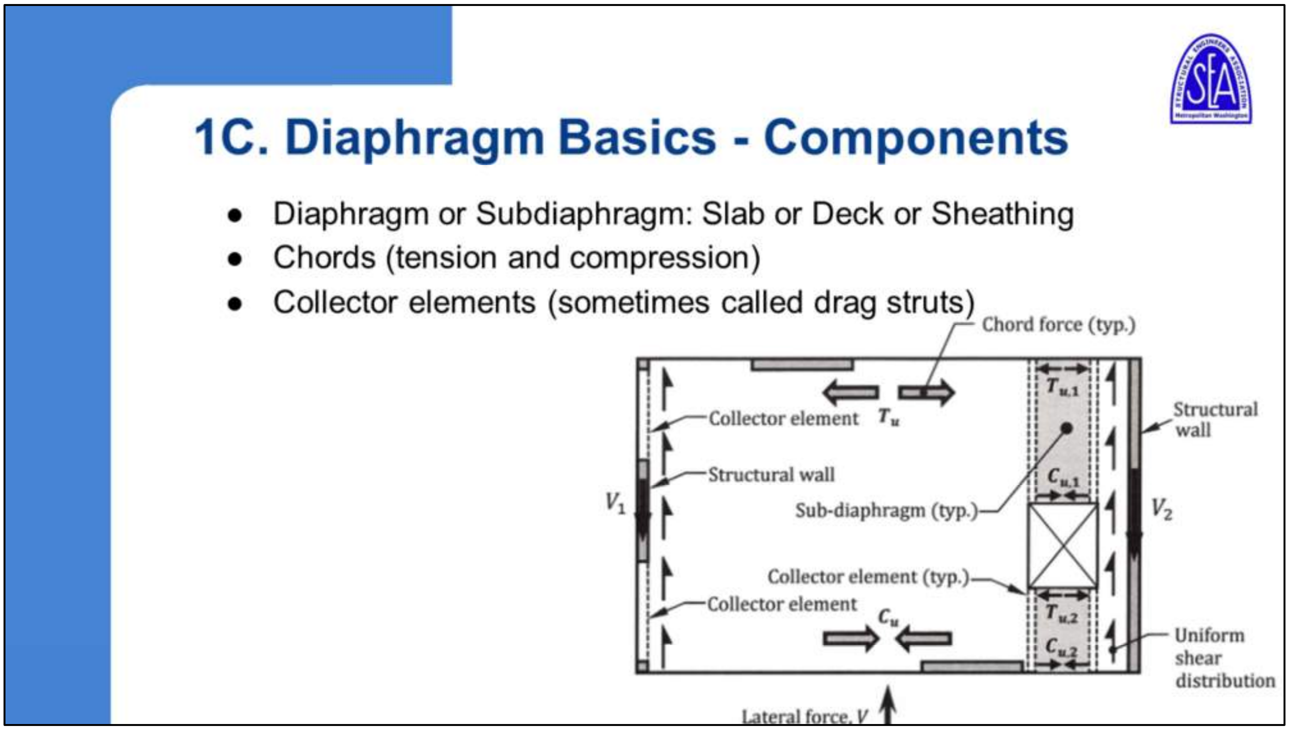
1. WIND: Wind loads are transferred from exterior cladding to the diaphragm which then takes the load to the vertical elements which resist lateral load.
2. SEISMIC: For earthquake loading, inertial forces start in the diaphragm and the tributary elements like exterior cladding; once the inertial force is generated by the ground motion, the load then is transferred by the diaphragm to the vertical elements.
3. SOIL: For subterranean building levels, soil pressure loads are transferred from the basement walls to the diaphragms.
   - If the soil loads are balanced, the load remains a compression load in the slab.
   - If the soil loads are unbalanced in the case of a partial basement, the loads are transferred from the diaphragm to the vertical elements. This will be one of the examples Joe will present later in the presentation.
1. For buildings where the footprint changes from smaller to bigger, or where vertical elements change stiffness, we get transfer forces. A simple example of this is a concrete podium slab - Once you hit the basement or podium level, the load will transfer out of the moment frames or flexible shearwalls and travel through the diaphragm into the much stiffer vertical element - the basement walls.
2. COLUMNS: Diaphragms brace columns providing lateral support to resist buckling; Also, if inclined columns have horizontal forces called thrusts that must be transferred by the diaphragm to a resisting element (like a shearwall).

1B. Diaphragm Basics - Types

- Concrete - Cast-in-place concrete, either conventionally framed slabs or post-tensioned slabs, with or without beams
- Concrete - prestressed, precast concrete elements fastened together with embedded steel plates or by a topping slab
- Composite concrete slab on steel deck
- Steel roof deck
- Wood - wood sheathing and CLT
- Gypcrete topping on board

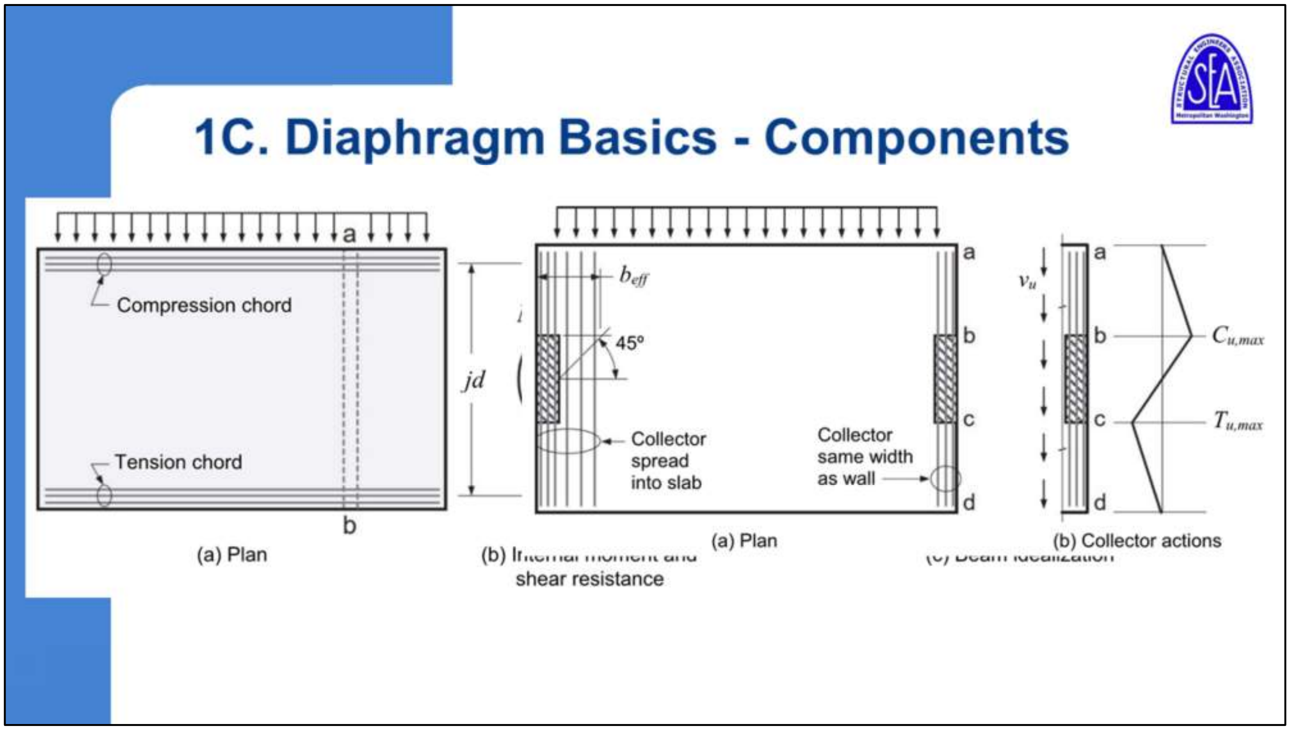
- CLASSIFICATIONS: Rigid, semi-rigid, flexible

ROSE
Role of the diaphragm - Resist wind, seismic, earth pressure, fluid loads



ROSE
Role of the diaphragm - Resist wind, seismic, earth pressure, fluid loads
Resisting these loads generates in the diaphragm several forces: in-plane shears, axial loads, bending forces.

- Diaphragm: main roof or floor planar element
- Chords: resist compression in tension perpendicular to the main lateral force
- Collector elements: pull force into the vertical elements

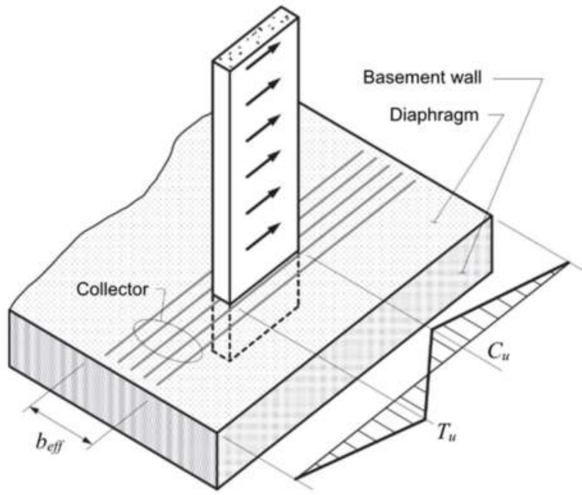


ROSE
Starting with the most simplified analysis technique for a diaphragm, we idealize this rectangular floor plate as a deep beam.
The moment and shear diagrams are shown on the right.
If we take a cross-section thru the deep beam, we see there are

Large moments occur at midspan which generates large T/C forces in the chords at the edges of the floor, and relatively small forces near the vertical elements.

Conversely, Large shears occur near the supports, creating large diaphragm shears near the vertical elements.

1C. Diaphragm Basics - Components



ROSE
Diaphragms and Collectors can also take the force from one vertical element, to the diaphragm and back to a stiffer vertical element
Podium slab is taking not just large vertical loads, but also large horizontal loads.
These transfer forces can be the largest forces in the diaphragm and must not be overlooked by the designer.

2. Diaphragm Design Requirements

Failure mechanisms and Codes

ROSE -> JOE
Now something that usually catches everyone's attention is what happens if we do not review our diaphragms?

2A. Diaphragm Design Requirements- Failure

- If the role of the diaphragm is to resist wind, seismic, earth pressure, fluid loads, and eccentric gravity loads… What happens when diaphragms are inadequate?
  - Load Path is incomplete.
  - Diaphragms and connections can fail - as shown in the following examples.
  - Diaphragm detaches from roof/floor and leads to wall collapse, then roof collapse.
  - Diaphragm detachment leads to increased unbraced height - column or wall is then unable to sustain gravity load.

JOE

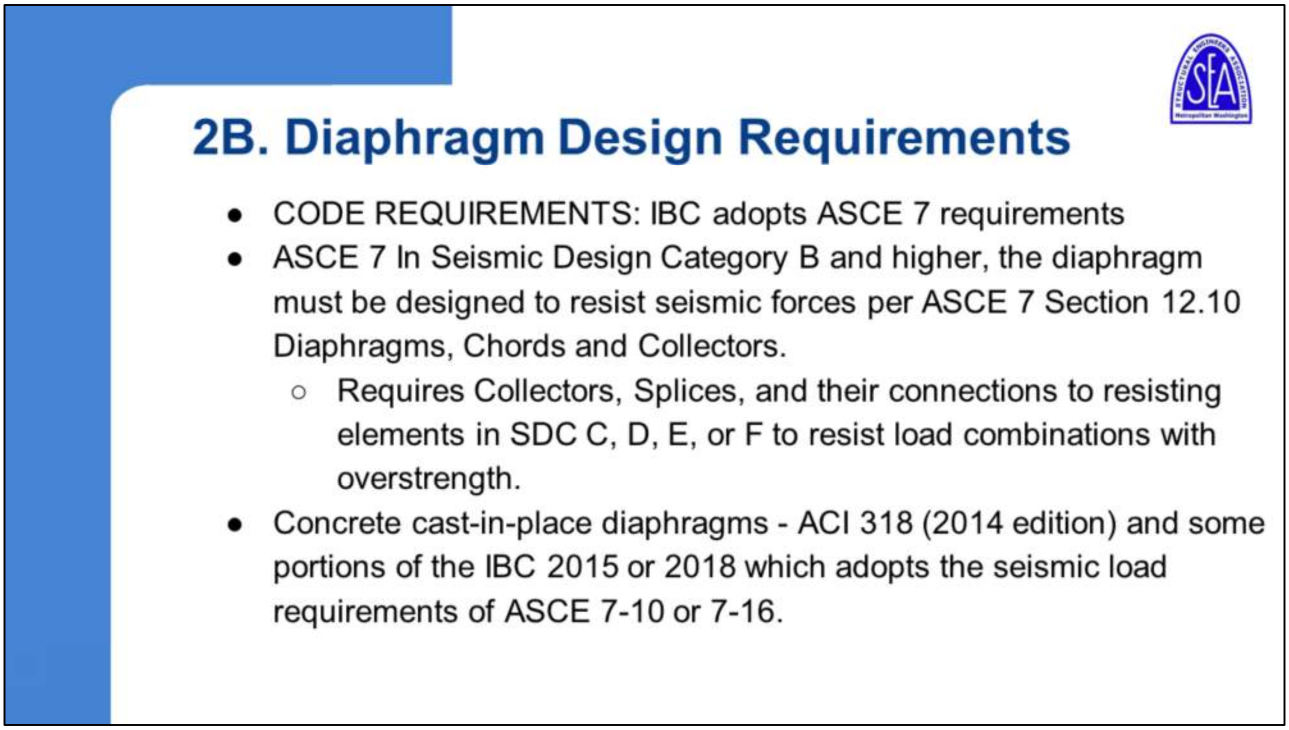
If we do not review our diaphragms or at a minimum provide an adequate and rational load path, diaphragms can fail and in some extreme cases can cause some serious issues, like collapse. Some examples of diaphragm failures would be:

1. If the floor or roof connections to bearing walls are insufficient, the out -of plane wall can detach from the roof or floor and collapse due to insufficient out-of-plane bracing; Now that your wall or vertical support is gone, collapse of the roof isn't far behind.
2. Diaphragm detachment also leads to increased unbraced heights of columns which can lead to further collapse.

JOE

First photo is a building that was damaged in the 1992 Landers, California 7.3 magnitude Earthquake - Inadequate anchorage and collector design lead to the roof pulling away from it’s support and ultimately collapsing.

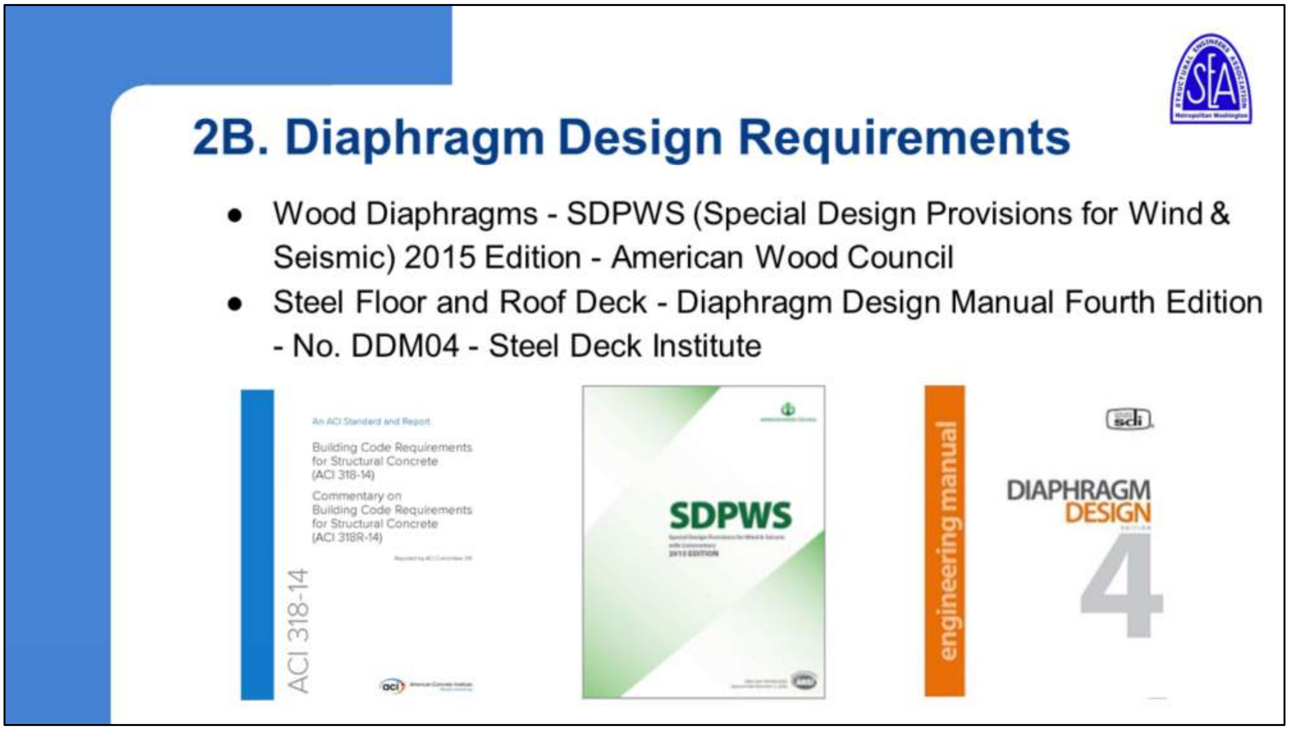
Second photo is a building that was damaged in the 2015 Gorkha, Nepal 7.8 magnitude earthquake - A partial out-of-plane wall failure occured due to roof diaphragm flexibility. Essentially the diaphragm deflected until the wall collapsed from the eccentricity and the roof went along for the ride.



JOE

- Unfortunately, the code is open to interpretation so you will need to use your engineering judgement and accepted design approaches.
- Several compilations of accepted design approaches are listed at the end of this presentation.

- IBC Adopts ASCE 7
- ASCE 7 does not have too much guidance for wind, but it does have a lot of seismic requirements
    a. defines load combinations with overstrength factor Ω_0. *Equations take both vertical and horizontal seismic load effects into account.*
- ACI 318 contains design, detailing and inspection requirements for concrete Diaphragms



JOE

1. Wood Diaphragms - SDPWS (Special Design Provisions for Wind and Seismic) which provides methods for calculating diaphragm deflections, sets limits on diaphragm aspect ratios, and provides shear capacities for diaphragms withstanding wind and seismic loads.

1. Steel Floor and Roof Deck - Diaphragm Design Manual which includes design guidelines for diaphragm strength and stiffness, fasteners and connections, and warping and stiffness properties.

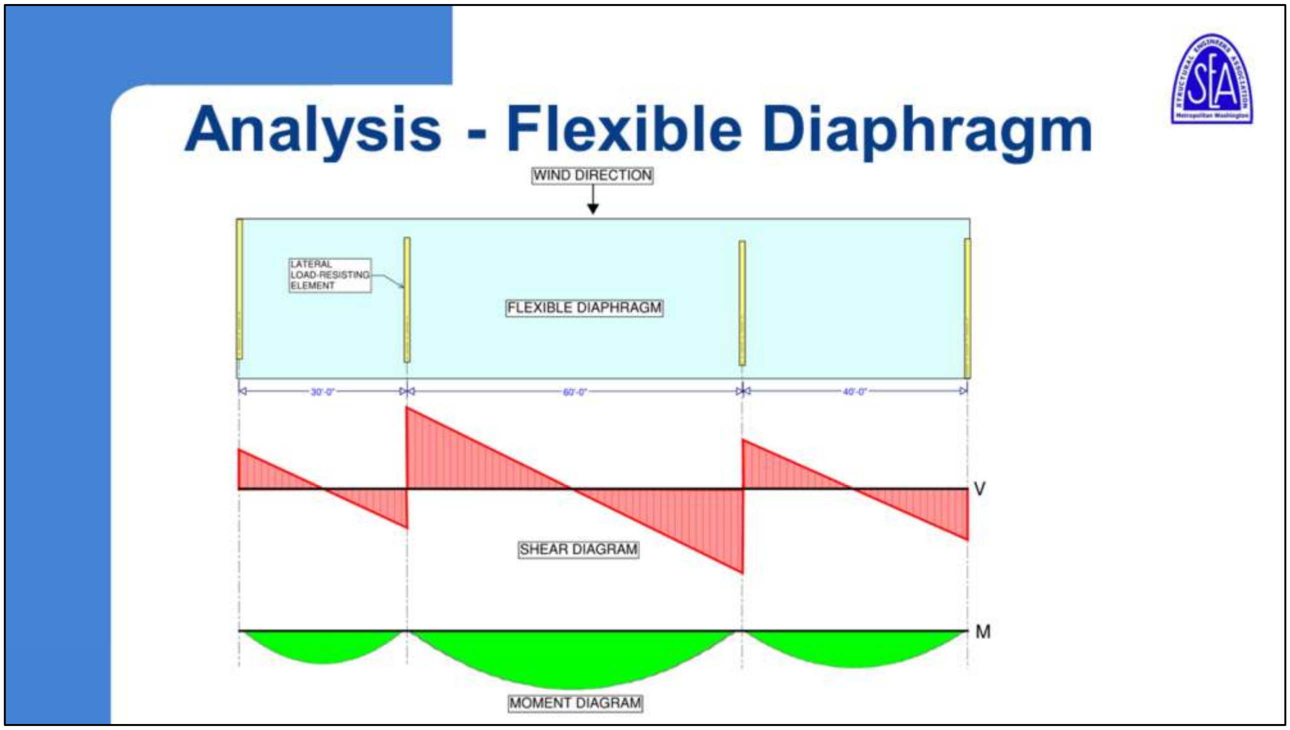
3. Diaphragm - Historical Analysis Techniques

Flexible, Rigid, or somewhere inbetween

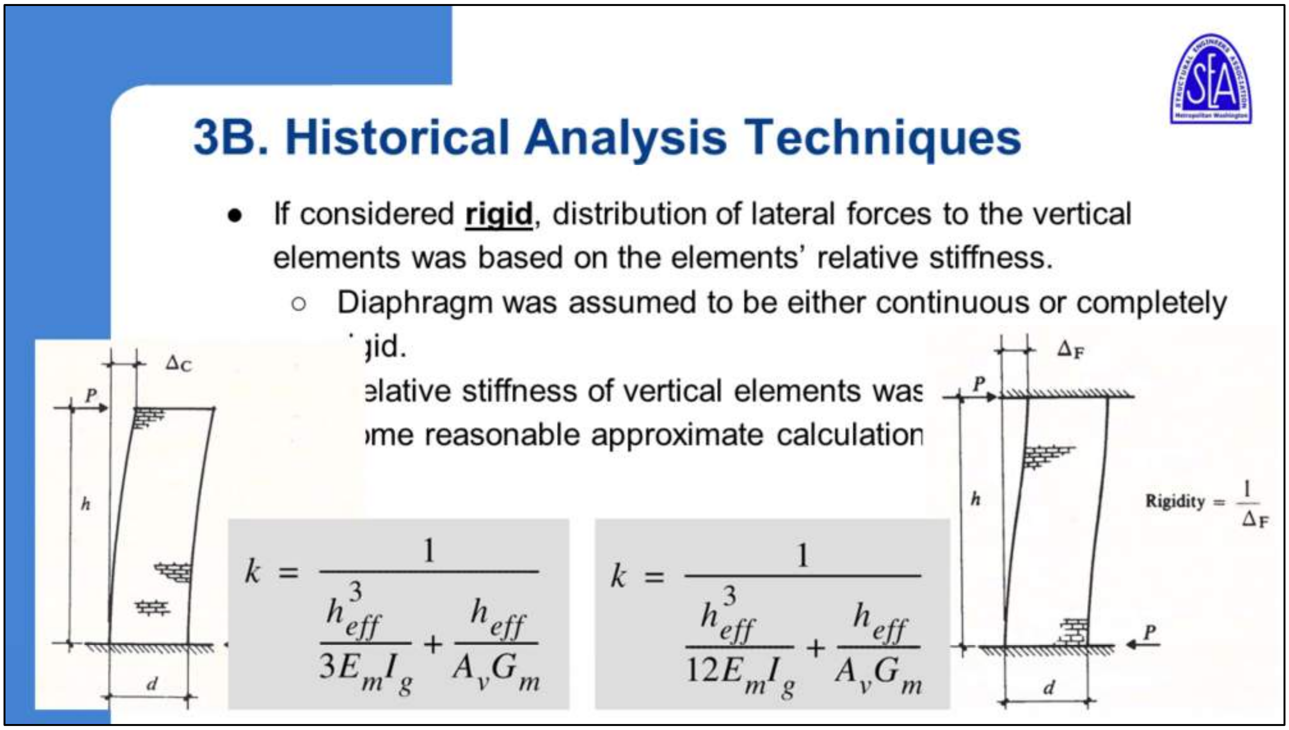
ROSE
Historically, diaphragms were either analyzed as either completely flexible or infinitely rigid.

3A. Historical Analysis Techniques

- If considered **flexible**, the diaphragm was assumed to simply span between adjacent vertical elements that resisted load.
  - The floor or roof system was assumed to be discontinuous.
  - Forces "tributary" to the vertical elements were calculated as the sum of the simple span reactions to those elements.

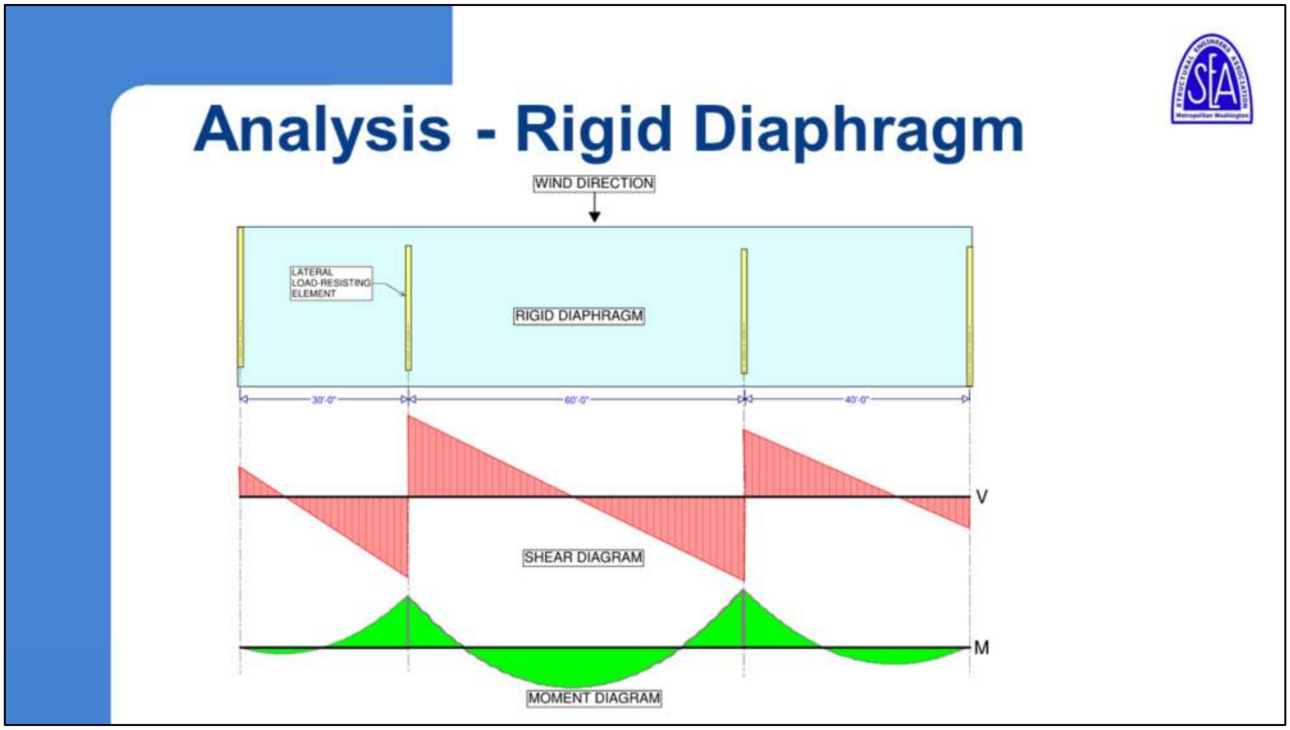


Diaphragm is discontinuous-
Tributary area method of load distribution.



Need more discussion points here…

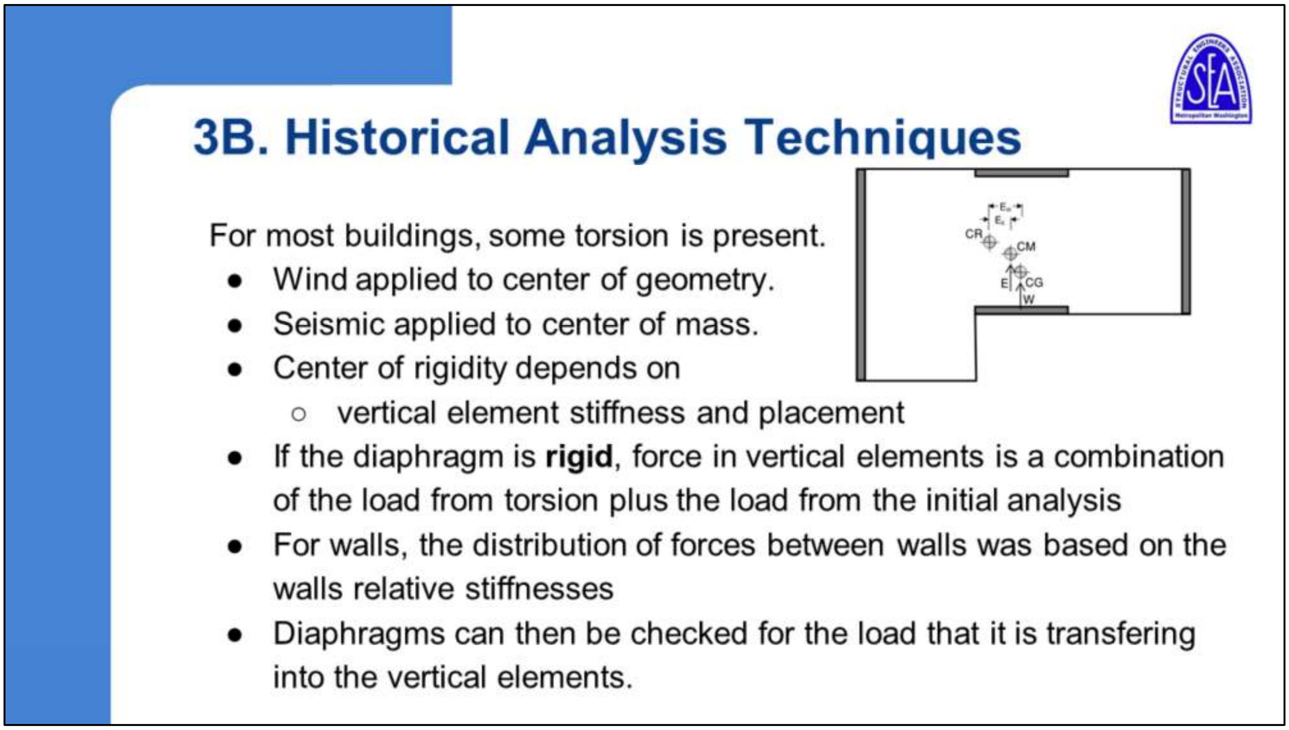
1. Equations for calculating the relative stiffness of certain types of lateral load-resisting elements have been around for quite a while.
    a. For masonry shearwall, load distribution between certain members could be estimated by calculating their relative stiffness
    b. The two equations shown are for cantilevered shear walls and fixed shearwalls.
    c. The denominator is the deflection caused by moment and shear.
    d. Stiffness “k” is the inverse of deflection



This is a continuous diaphragm; there are various techniques to determine the beams moment and shear diagrams :

Methods of calculating these diagrams are

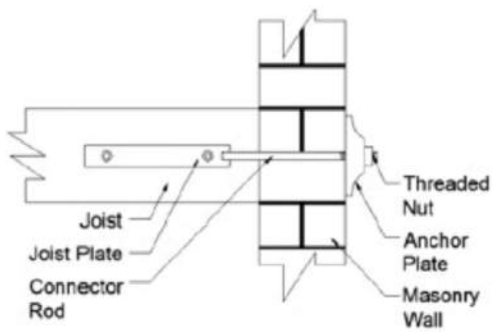
1. The moment distribution method developed in 1930
2. Beam diagrams in the Beam section of the Steel Construction Manual by AISC



1. To throw a wrinkle into this analysis, if the lateral load does not line up with the centroid of resistance, torsion is created.
    a. Wind load applied to center of wind area, usually the center of geometry
    b. Seismic loads applied at the center of mass
    c. Center of rigidity is resisting these loads.
2. Torsional effects are distributed by a rigid diaphragm; this torsion is caused by the eccentricity between applied load and resistance.
    a. The torsion is a moment applied to the center of rigidity.
    b. This increases load within vertical lateral load resisting elements thus increasing load within diaphragm and it’s connections to these elements which must be accounted for.
3. Once the load resisted by a vertical elements has been calculated, the diaphragm is checked the diaphragm strength and attachments.

3C. Historical Analysis Techniques

- If it was unclear if a diaphragm was flexible or rigid, the analysis was “enveloped” and the designer considered the answer was somewhere between the flexible and rigid analysis results.



1. Enveloping a solution is reasonable for most materials, certainly for concrete which can crack to redistribute the load.
2. And if all else failed, the designer could just throw in some diaphragm connections between the floor framing and the exterior wall. These diaphragm connection plates are very popular as retrofits. These are also called earthquake rosettes in high seismic zones.

4. Diaphragm - Modern Analysis Techniques

Additional analysis options are now available and widely used

Joe

Now that we've discussed historical design techniques, we can take a look at more modern techniques that are available to us today.

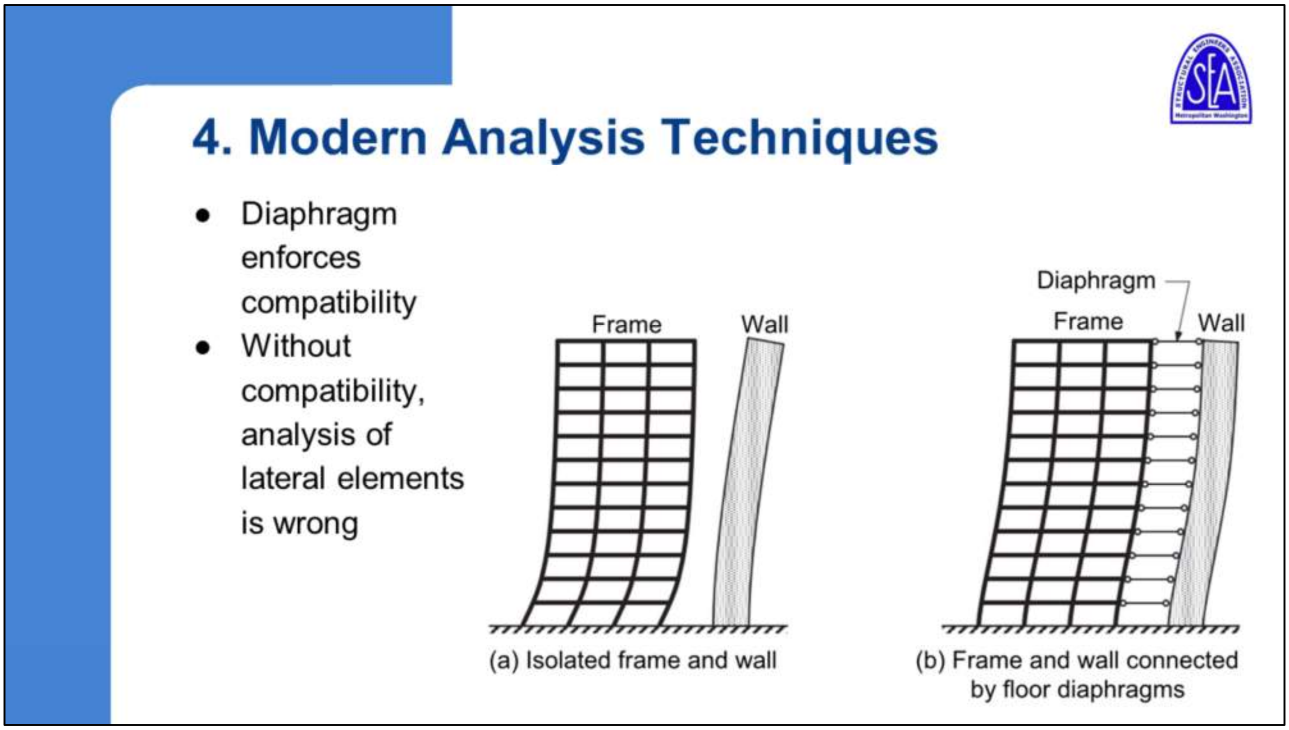
4. Modern Analysis Techniques

Simple 2-D idealizations → complex computer finite element analysis.

- Simple Models: 2-D idealizations are appropriate for rectangular floor plate with vertical elements evenly distributed with similar stiffnesses
  - Use 2-D beam modeling with pinned supports or spring supports
  - Use elastic or inelastic modeling of diaphragm “beam” element
  - Simple Strut and Tie analysis
- More Complex models: 3-D modeling appropriate for
  - Irregularly shaped floor plates
  - Vertical elements with dissimilar stiffnesses and drift profiles
  - Vertical element discontinuities
  - a moderate to high seismic load

1. Today we have everything from simple 2-d idealizations all the way to super complex computer finite element analysis options.
2. Even the 2-D modeling of today is much more complex than it used to be.
   a. 2-D models are appropriate for rectangular floor plates with vertical elements evenly distributed with similar stiffnesses.
   b. The diaphragm can be idealized as a beam with the lateral supports being modeled as pinned, or if you wanted to be add a bit more complexity, as springs that mimic the relative stiffness of the lateral supports. We will discuss spring modeling a little later in the steel roof deck design example.
   c. This beam idealization can even be taken a step further and modeled inelastically to determine what happens if the concrete diaphragm cracks, when your stresses are high enough.
   d. Another method that can be implemented is Strut and Tie
3. 3-D modeling, while more complex, actually adds relatively little additional effort if you are already utilizing ETABS or Ram Concept for your structural analysis modeling.
   a. This level of sophistication is appropriate for irregularly shaped floors, vertical elements with dissimilar stiffnesses, drift profiles, vertical element discontinuities, or for a moderate to high seismic load.

1. How you decide what method to use will vary by how complex your structure is. The analysis needs to be sufficiently complex to represent how lateral forces are flowing through the building.



JOE

1. Modern computer analysis programs make it easier for us to identify and quantify the transfer forces that occur in most structures.
2. These transfer forces can be much larger than a wind load at a particular floor, or a seismic inertial force.

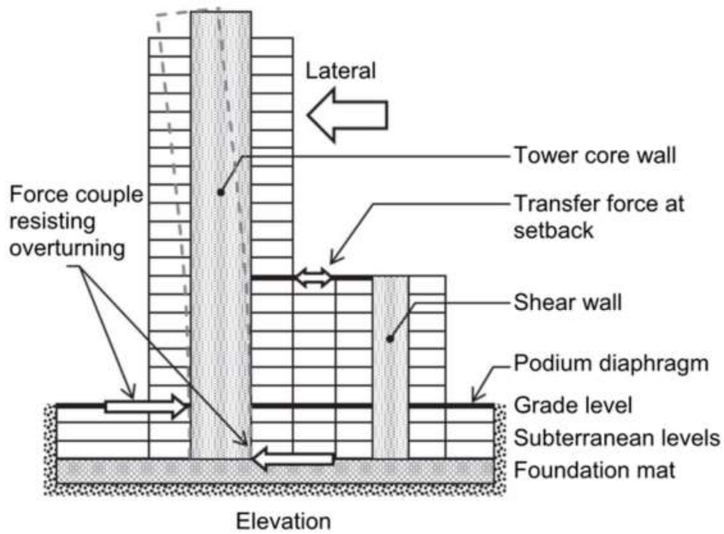
One type of transfer force is shown here.

- The moment frame and the shearwall are in line but they have different displacement shapes and different stiffnesses when they are isolated.
- The image on the right looks more like a shape we would want to happen, ie the building is working as a single unit. To achieve this the diaphragm needs to enforce compatibility between these elements. Forcing this type of compatibility can result in a significant amount of force being transferred through the diaphragm.
- An incorrectly designed and detailed diaphragm will crack and fail, causing the building to act more similar to the isolated condition on the left, making your rigid diaphragm analysis in Ram Structural System completely wrong.

Ultimately we want to maintain relatively stiff and damage-free diaphragms.

4. Modern Analysis Techniques

- Transfer forces at vertical offsets
- Vertical discontinuities



JOE
Most transfer forces occur at vertical discontinuities or offsets.
The example shown here displays both issues.
As you work your way down the building, more walls are introduced. These walls are shorter and thus stiffer than the taller one which will result in the taller wall shedding load at the floor level, through the diaphragm, to these stiffer walls.

This is a very common occurrence in buildings with basement walls. At a basement level, you typically have a significant amount of wall length. Again, this causes the taller more flexible walls to shed load. But how does that load transfer to these basement walls? The diaphragm. This results in a force significantly higher that the typical inertial or wind loads that are being placed on an individual level. Even a wind load can create a high enough force to cause concern. The force at this level can actually be so high that you can create a shear reversal at the base of the taller wall. If these forces are not accounted for and are large enough to fail the diaphragm, the forces will find a new load path and redistribute, and possibly make the forces you have designed your lateral resisting elements for incorrect.

4. Modern Analysis Techniques - Flexible

- Flexible Diaphragm
  - ASCE 7-16 Section 12.3.1.1 - Diaphragms constructed of untopped steel decking or wood structural panels are permitted to be idealized as flexible.
  - ASCE 7-16 Section 12.3.1.3 - Diaphragms are permitted to be idealized as flexible where the max in-plane diaphragm deflection is more than two times the average story drift.
  - Loads applied to lateral resisting elements are based on tributary width.

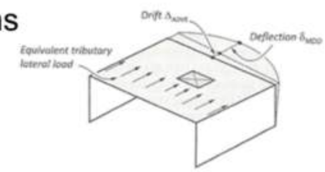


FIGURE 12.3-1 Flexible Diaphragm

Joe

- Diaphragms can be categorized as either flexible, rigid, or semi-rigid
- ASCE 7 defines diaphragm types within the seismic chapter. **These definitions are acceptable to apply to wind or other load types.**
- Concrete is **never** a flexible diaphragm

4. Modern Analysis Techniques - Flexible

- Flexible Diaphragm
  - Pros
    - Simple load path
    - Behavior is easy to understand
  - Cons
    - Can be overly simplistic and possibly overly conservative depending on building layout.
    - Can be tedious to apply nodal loads.

Joe

4. Modern Analysis Techniques - Rigid

- Rigid Diaphragm
  - ASCE 7-16 Section 12.2.1.2 - Diaphragms of concrete slabs or concrete-filled metal deck with span-to-depth ratios of 3 or less in structures that have no horizontal irregularities are permitted to be idealized as rigid.
  - IBC 2018 Section 1604.4 - Diaphragm is rigid when the lateral deformation of the diaphragm is less than or equal to two times the average story drift.
  - Loads applied to lateral resisting elements are based on stiffness of the lateral resisting elements.

Joe

- Note that this IBC provision is similar but opposite(???) to ASCE 7's definition of flexible.
- More complex than a simple diaphragm but still simple enough that a lateral computer model will run relatively quickly.

4. Modern Analysis Techniques - Rigid

- Rigid Diaphragm
  - Pros
    - Computer models run very quickly.
    - Easy to understand results.
  - Cons
    - Diaphragm forces are unavailable.
    - Axial forces are hidden within the diaphragm in computer models and are not applied to frame beams. This can be unconservative when designing braced frames where axial loads within the beams can be significant.
    - Large openings are not considered.

Joe

4. Modern Analysis Techniques - Semi-Rigid

- Semi-rigid Diaphragm
  - ASCE 7 - Unless a diaphragm can be idealized as either flexible or rigid, the structural analysis shall explicitly include consideration of the stiffness of the diaphragm (i.e., semi-rigid)
  - ASCE 7-16 Section 12.3.1.2 does not allow diaphragms with horizontal irregularities to be analyzed as Rigid.
  - Buildings with irregular shapes or large openings.

Joe

4. Modern Analysis Techniques - Semi-Rigid

- Semi-rigid Diaphragm
  - Pros
    - Most accurate; Considered the most rational in determining actual distribution of forces.
    - Diaphragm forces and deflections can be reported.
  - Cons
    - Very significant calculation effort.
    - Need to input diaphragm properties.
    - Meshing of diaphragms within models dramatically increases the time it takes a model to run.
- Simplified semi-rigid analysis is acceptable alternate.
- Enveloping rigid and flexible is acceptable alternate.

JOE

- Dramatically increases the time it takes to run a model. For diaphragms that are very stiff, solid concrete slabs for example, will give similar results as a rigid diaphragm.
- Can define a single floor as semi-rigid, in a building with a large floor opening on one level for example, and the remaining floors as rigid to decrease model run time.

5. Design Examples

5A. Wind on Steel Roof Deck -
Single and Multi-span (rigid and semi-rigid)

5B. Wind on Wood Roof -
Flexible Roof with component design

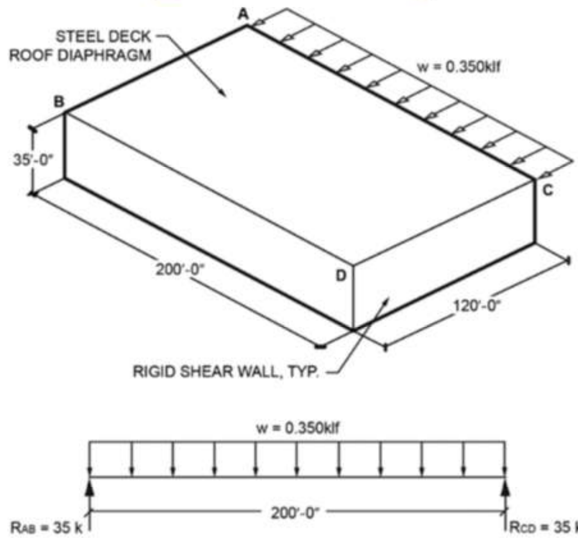
5C. Soil Pressure on Concrete Deck -
Finite Element Analysis

5D. Seismic Analysis

Rose
Define what is a diaphragm, types (materials) and components of a diaphragm ... and why do I have to check it.

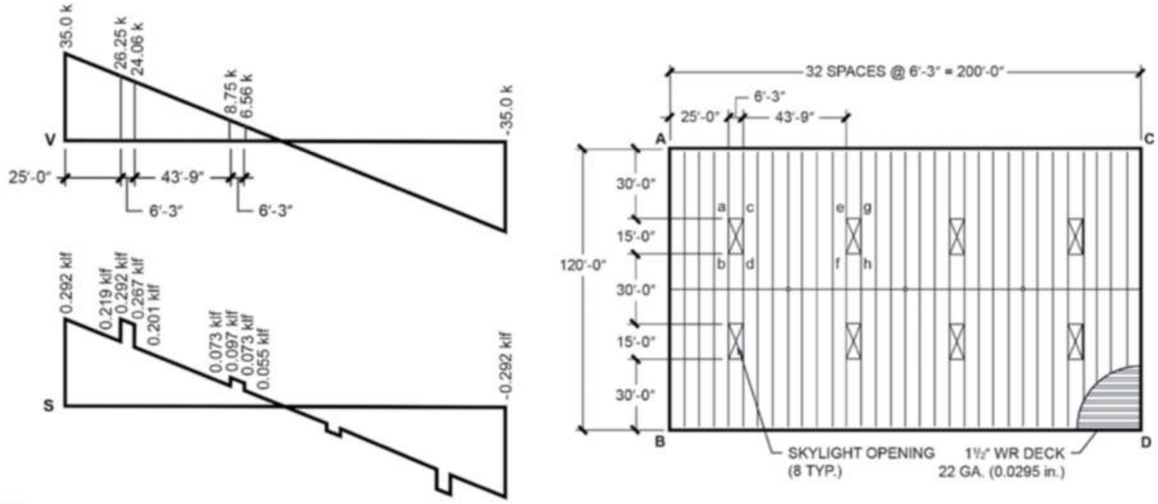
5A. Diaphragm Design Example - Wind on Steel

- Shear walls on either side of a large box store
- Uniform roof and parapet height = constant load
- Rigid Diaphragm
- Single span



ROSE
Service level wind pressure of 20 psf = windward + leeward pressures

5A. Example-Wind on Steel Roof Deck



n Steel Roof Deck

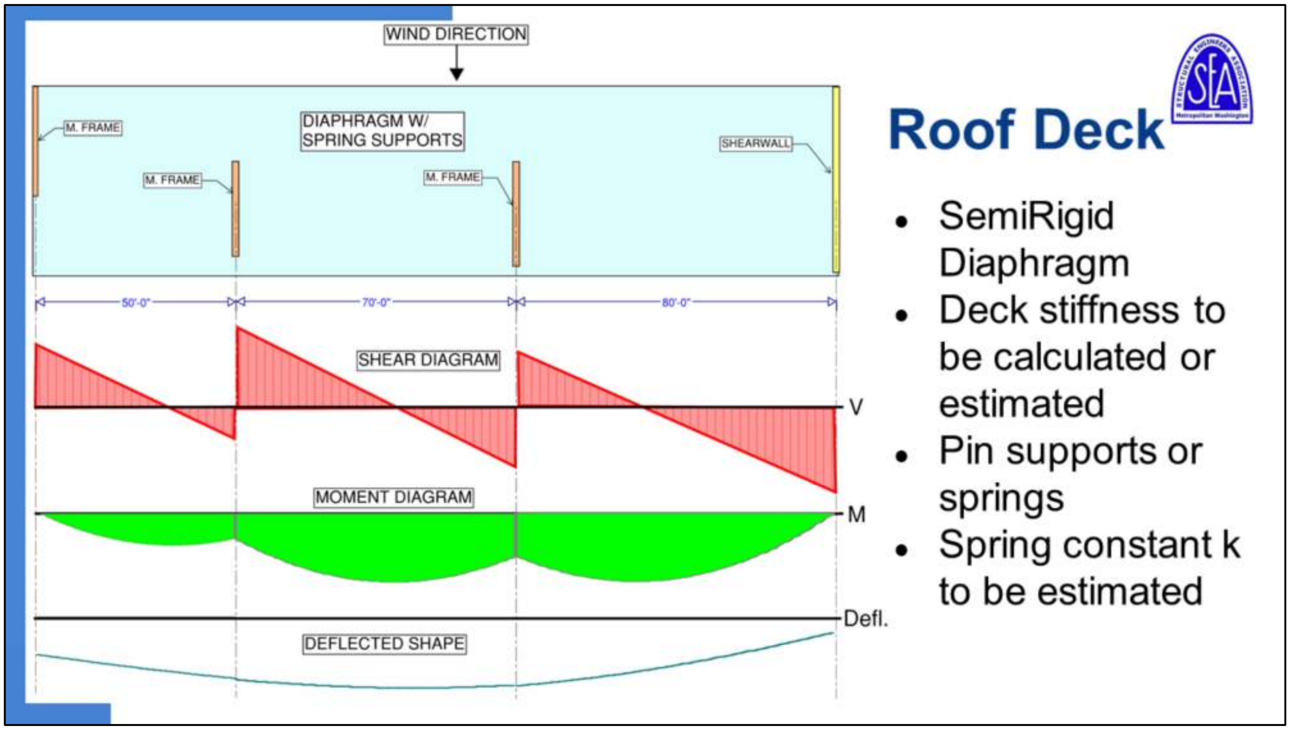
TABLE NO. 21A - B 20 GA. DIAPHRAGM DESIGN

Design Thickness = 0.0358 in.
Support Fasteners: 5/8" Puddle Welds
Side Lap Fasteners: # 10 Screws

Φ (EQ): 0.55 Ω (EQ): 3.00
Φ (Wind): 0.70 Ω (Wind): 2.35
Φ (Other): 0.60 Ω (Other): 2.65

Fill Type	Support Fastener Pattern	Side Lap Conn. per Span	Nominal Diaphragm Shear Strength (plf) – Center to Center Span (ft. - in.)														K_1 (ft.$^{-1}$)
			4 - 0	4 - 6	5 - 0	5 - 6	5 - 9	6 - 0	6 - 3	6 - 6	6 - 9	7 - 0	7 - 6	8 - 0	8 - 6	9 - 1	
No Fill (Bare Deck)	36/9 (B) D_n = 68	0	1671	1493	1334	1205	1149	1097	[illegible]	1006	966	928	860	801	749	695	0.357
		1	1827	1649	1488	1345	1283	1225	-	-	-	-	-	-	-	-	0.299
		2	1977	1789	1631	1485	1416	1354	1296	1243	1193	1148	1065	993	929	-	0.258
		3	2121	1924	1757	1616	1550	1482	1419	1361	1307	1257	1168	1089	1020	949	0.226
		4	2258	2054	1880	1732	1665	1604	1542	1479	1421	1367	1270	1185	1110	1033	0.202
		5	2388	2178	1999	1844	1775	1710	1650	1593	1535	1477	1373	1281	1201	1118*	0.182
		6	2512	2298	2113	1953	1881	1814	1751	1692	1636	1584	1475	1377*	1291*	1202*	0.166
		7	2630	2413	2224	2059	1985	1915	1850	1788	1731	1676	1577*	1473*	1382*	1287*	0.152
		8	2741	2522	2330	2161	2085	2013	1946	1882	1822	1766	1662*	1569*	1472*	1372*	0.141
		9	2847	2627	2432	2260	2182	2109	2039	1974	1912*	1854*	1747*	1650*	1562*	1456*	0.131
		10	2947	2726	2530	2356	2276	2201	2130	2063*	2000*	1940*	1829*	1730*	1640*	1541*	0.122
	36/7 (B) D_n = 68	0	1038	918	822	743	709	678	[illegible]	622	597	574	533	497	465	433	0.535
		1	1219	1089	976	883	843	806	-	-	-	-	-	-	-	-	0.415
		2	1384	1245	1129	1023	976	934	895	859	825	794	738	689	646	-	0.340
		3	1542	1391	1265	1159	1110	1062	1018	977	939	904	841	785	737	686	0.287
		4	1693	1531	1396	1282	1231	1184	1140	1095	1053	1014	943	881	827	771	0.249
		5	1836	1666	1522	1400	1346	1295	1248	1204	1163	1123	1046	977	917	856	0.219
		6	1971	1795	1644	1515	1458	1404	1354	1307	1263	1222	1147	1074	1008	940	0.196
		7	2099	1917	1761	1626	1566	1509	1456	1407	1360	1317	1237	1166	1098	1025	0.178
		8	2219	2034	1873	1734	1671	1611	1556	1504	1455	1409	1325	1250	1183	1109*	0.162
		9	2333	2145	1981	1837	1772	1710	1653	1599	1548	1500	1412	1333	1262*	1187*	0.149
		10	2439	2250	2083	1936	1869	1806	1747	1691	1638	1588	1496	1414*	1339*	1261*	0.138
	36/5 (B) D_n = 428	0	946	849	760	687	655	626	[illegible]	574	552	530	492	458	429	398	0.642
		1	1100	994	906	827	789	754	-	-	-	-	-	-	-	-	0.477
		2	1243	1129	1033	950	913	879	845	811	779	750	697	651	610	-	0.380
		3	1375	1255	1152	1063	1023	986	951	919	888	860	799	747	700	652	0.315
		4	1496	1372	1264	1171	1128	1089	1051	1016	984	953	896	843	791	737	0.270
		5	1606	1480	1370	1272	1228	1186	1147	1110	1075	1042	981	927	878	822	0.236
		6	1706	1580	1467	1367	1322	1278	1237	1199	1162	1128	1064	1006	954	900	0.209
		7	1796	1671	1558	1457	1410	1366	1324	1284	1246	1210	1143	1083	1028	970	0.188
		8	1878	1755	1643	1540	1493	1448	1405	1364	1325	1288	1219	1157	1099	1039	0.171
		9	1951	1831	1720	1618	1571	1525	1482	1440	1401	1363	1292	1227	1168	1105*	0.156
		10	2017	1901	1792	1691	1644	1598	1554	1512	1472	1434	1362	1295	1235*	1170*	0.144
	36/4 (B) D_n = 608	0	725	643	574	518	494	471	[illegible]	432	414	398	368	342	320	296	0.802
		1	876	794	724	658	627	599	-	-	-	-	-	-	-	-	0.561
		2	1013	923	847	781	751	724	[illegible]	668	642	617	573	535	501	-	0.431
		3	1134	1040	959	888	856	826	798	772	747	724	676	631	591	550	0.350
		4	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	892	864	837	812	765	723	681	635	0.294
		5	1334	1240	1156	1080	1045	1011	980	950	922	895	846	801	760	717	0.254
		6	1416	1324	1240	1163	1128	1094	1062	1031	1002	974	922	875	832	786	0.224
		7	1487	1398	1316	1240	1204	1170	1137	1106	1076	1047	994	945	900	852	0.200
		8	1549	1464	1384	1309	1274	1240	1207	1175	1145	1116	1061	1011	964	915	0.180
		9	1603	1523	1445	1372	1337	1304	1271	1240	1209	1180	1125	1073	1026	975	0.164
		10	1651	1574	1500	1429	1395	1362	1330	1299	1269	1239	1184	1132	1084	1032	0.151

- ICC-ES Evaluation Report for New Millenium
- 1.5 B Roof Deck - 20 gage
- 6'-3" c/c support spacing
- 36/4 - ⅝" dia. weld to support
- (4) side lap fasteners per span (#10 screws @ 18")
- 892 plf nominal V strength
- Divide by 2.35
- Allowable shear = **380 plf**



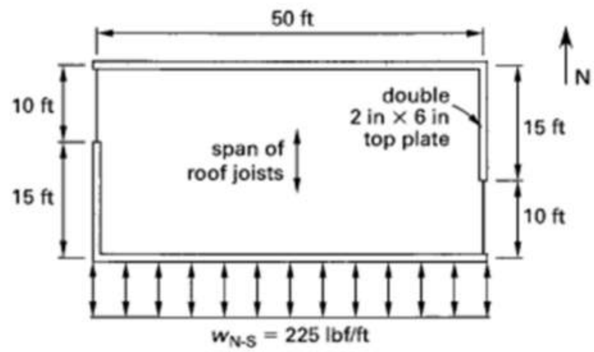
Maybe this works but it's the forces are difficult to transfer.
PRELIMINARY analysis - Does this layout makes sense, is the deck strong enough?
Try another layout
Let's try the same 200' x 120' long building with a different lateral system to create a multi-span diaphragm instead of a single span diaphragm.

1. Here we have 3 moment frames and 1 shearwall.
2. Test this out with a 2-D beam analysis with pinned and roller supports
3. Or we can use a 2-D beam analysis with spring supports.
    a. The spring constant k can be iterated such that the spring deflection matches your RamSteel output, or can be estimate using a simplified formula.
    b. Alternately, the spring constant k can be guesstimated based on the relative stiffness of the lateral elements.
4. The deck in-plane stiffness also needs to be estimated.
    a. Based on pure shear deformation, warping deformation, connection slip
    b. AISI D310-17: AISI Design Guide - Design Examples for the Design of Profiled Steel Diaphragm Panels Based on AISI S310-16, 2017 Edition
5. The moment has dropped significantly, which lowers the chord forces.
    a. The single span moment was 1750 k-ft creating chord forces of
    b. The new multi-span diaphragm has
        i. A max. moment of 560 k-ft which is 32% of the original moment.

        ii. A max. Shear of 22 k which is 63% of the original shear.

5B. Diaphragm Design Example Wood - Flexible

- Wind Load = 225 lbf/ft
- 50' x 25'
- LRFD
- No. 2 SPF



JOE

Next is a flexible wood diaphragm.

Diaphragm Design Example
Wood - Flexible

- L/W = 50/25 = 2:1

4.2.4 Diaphragm Aspect Ratios

Size and shape of diaphragms shall be limited to the aspect ratios in Table 4.2.4.

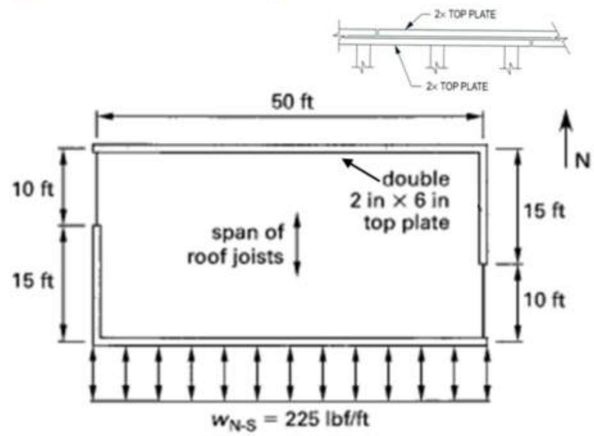
Table 4.2.4 **Maximum Diaphragm Aspect Ratios** (Horizontal or Sloped Diaphragms)

Diaphragm Sheathing Type	Maximum L/W Ratio
Wood structural panel, unblocked	3:1
Wood structural panel, blocked	4:1
Single-layer straight lumber sheathing	2:1
Single-layer diagonal lumber sheathing	3:1
Double-layer diagonal lumber sheathing	4:1

- First and foremost is to check your aspect ratio.
- In our case we have a 2:1 which meets the maximum aspect ratio of 4:1 for a blocked diaphragm.
- The closer you get to these aspect ratio, the more concerned you should get about diaphragm deflection, which we will calculate in a later slide.

Diaphragm Design Example
Wood
Chord Forces

- $M = W_{N\text{-}S}L^2/8$
  $= 225 \text{ lbf/ft} \times (50\text{ft})^2/8$
  $= 70{,}313 \text{ ft-lbf}$
- $T = M/B = 70{,}313 \text{ ft-lbf}/50 \text{ ft}$
  $= 1406 \text{ lbf}$



We start by simplifying the diaphragm as a beam simply supported by the end walls. Use WL2/8 to get our moment.
Divide our moment by our diaphragm depth and we find our chord force of 1406 lbs.
Our chord is the made up of the 2x6 top plate that runs continuously along the top of the wall.

Diaphragm Design Example
Wood
Chord Forces

- F_t = 450psi
- C_D = Not used for LRFD
- C_F = 1.3
- C_m = C_t = Ci = 1.0
- K_F = 2.7
- Φ = 0.80
- *λ = 1.0 (Wind Combo)*
- *F'_T = 1264 psi*
- *A (2x6) = 8.25in2*
- *f_t = 1406lbf/8.25in2 = 171psi*
- *171psi < 1264psi*

Table 4.3.1 Applicability of Adjustment Factors for Sawn Lumber

		ASD only	ASD and LRFD										LRFD only		
		Load Duration Factor	Wet Service Factor	Temperature Factor	Beam Stability Factor	Size Factor	Flat Use Factor	Incising Factor	Repetitive Member Factor	Column Stability Factor	Buckling Stiffness Factor	Bearing Area Factor	Format Conversion Factor K_F	Resistance Factor ϕ	Time Effect Factor
$F_b' = F_b$	x	C_D	C_M	C_t	C_L	C_F	C_{fu}	C_i	C_r	-	-	-	2.54	0.85	λ
$F_t' = F_t$	x	C_D	C_M	C_t	-	C_F	-	C_i	-	-	-	-	2.70	0.80	λ

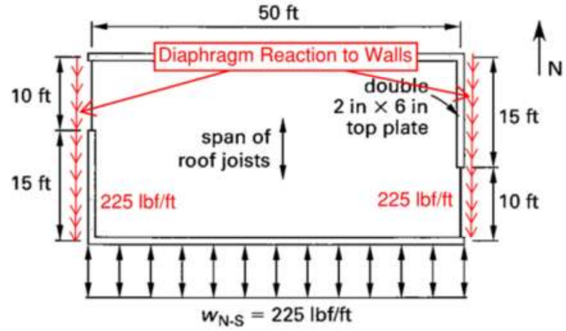
USE WITH TABLE 4A ADJUSTMENT FACTORS

Species and commercial grade	Size classification	Design values in pounds per square inch (psi): Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	E_{min}	Specific Gravity[4] G	Grading Rules Agency
SPRUCE-PINE-FIR										
Select Structural	2" & wider	1,250	700	135	425	1,400	1,500,000	550,000	0.42	NLGA
No. 1/ No. 2		875	450	135	425	1,150	1,400,000	510,000		
No. 3		500	250	135	425	650	1,200,000	440,000		
Stud	2" & wider	675	350	135	425	725	1,200,000	440,000		
Construction	2" - 4" wide	1,000	500	135	425	1,400	1,300,000	470,000		
Standard		550	275	135	425	1,150	1,200,000	440,000		
Utility		275	125	135	425	750	1,100,000	400,000		

Next we flip open our NDS and find our tensile strength and applicable factors from table.

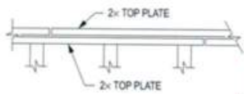
Diagram Design Example Wood Collector

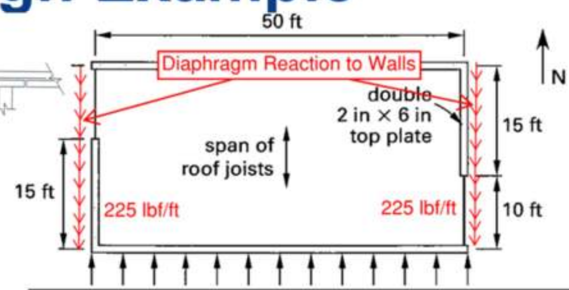
- R = $W_{N\text{-}S}L/2$ = 225lbf/ft x 50ft/2
  = 5625 lbf
- V = R/L = 5625lbf / 25ft = 225 lbf/ft
- T = 225 lbf/ft x 10ft = 2250 lbf
- From previous Calc $F'_T = 1264\ psi$
- $f_t = 2250\ lbf/8.25in2 = 275\ psi$
- *275 psi < 1264 psi*



Next we find our collector loads. The collectors in our case are the 10' openings on either side of the structure.
We again idealize our diaphragm as a simply supported beam and find our shear using WL/2.
Divide this shear by our diaphragm depth to find a force/ft.
Then multiply this force by the length of our 10' collector and end up with a tension/compression chord of 2250 lbs.
This force must be drug back into the wall using the double top plate.
Divide this force by a single 2x6 and we find our tensile force to be 275 psi which is less than our allowable so we are good.

Diaphragm Design Example
Wood
Collector





- Double 2x6 top plate splice
- Controlling T = 2250 lbf
- Tensile force will load nails in shear.
- Z = 100 lb for 10d common nail.
- C_D = Not used for LRFD
- C_{di} = 1.1 (Diaphragm Factor)
- K_F = 3.32
- Φ = 0.65
- *λ = 1.0 (Wind Combo)*
- Z' = 100 x 1.1 x 3.32 x 0.65 x 1.0
- Z' = 237 lbs/nail, 2250/237 = 10 Nails each side of splice

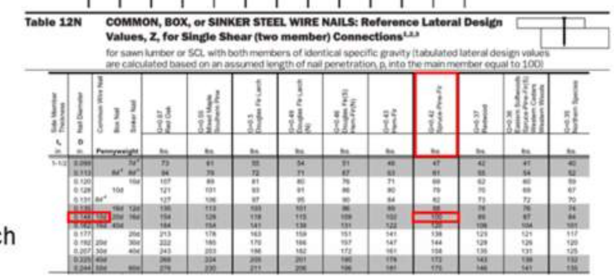


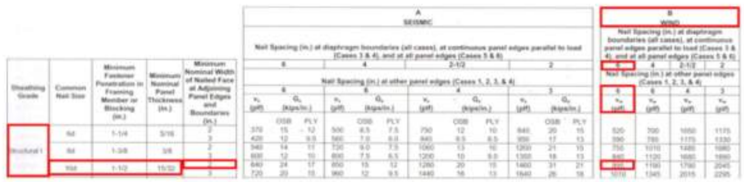
Table 12N **COMMON, BOX, or SINKER STEEL WIRE NAILS: Reference Lateral Design Values, Z, for Single Shear (two member) Connections**

for sawn lumber or SCL with both members of identical specific gravity (tabulated lateral design values are calculated based on an assumed length of nail penetration, p, into the main member equal to 10D)

Next we determine how many nails that are required in our splice.
Our collector tensions was the controlling load.
This load must be transferred between the boards through nail shear.
Again we open our NDS and find our Z for a 10d nail, multiply it my the appropriate factors and find we have a capacity of 237 lbs/nail and we end up needing 10 nails each side of the splice, ie 20 nails total at any given splice.

Diaphragm Design Example Wood Sheathing

- 15/32" Structural I Sheathing
- 10d Nails
- V = 225 lbf/ft
- Φ_D = 0.80
- Specific Gravity Factor = [1-(0.5-G)] = [1-(0.5-0.42)] = 0.92
- Boundary Nail Spacing = 6"
- Panel Edge Nail Spacing = 6"
- v_w = 895 lbf/ft
- v'_w = 895 x 0.80 x 0.92 = 658 lbf/ft > 225 lbf/ft



Then we can check to make sure our sheathing type, thickness, and nail pattern is adequate.
Using SDPWS we can find the tabulated shear capacity, apply our factors for LRFD and wood type and find that our capacity of 658 plf is greater than the demand we had previously found of 225 plf.

5B. Diaphragm Design Example Wood - Flexible

- ASD Load
  225 lbf/ft x 0.6 = 135 lbf/ft
- Deflection
  γ = 180000 $D^{1.5}$ = 180000 x $0.148^{1.5}$
  = 10248 lb/in/nail
  Δc = 2(T or C)/(γn) = 2(2250x.6)/(10248 x 6)
  = 0.044"

Chord Deformation + Shear/Panel Shear/Nail Slip + Chord Splice Slip

$$\frac{5(135/12)(50x12)3}{8(1400000)(8.25)(25x12)} + \frac{0.25(135/12)(50x12)}{1000(17)} + \frac{7\ \text{splices}(25x12)(0.044)}{2(50x12)} = 0.176"$$

(50x12)/0.176" = L/3409

4.2.2 Deflection

$$\delta_{dia} = \frac{5vL^3}{8EAW} + \frac{0.25vL}{1000G_a} + \frac{\sum x\Delta_c}{2W} \qquad (4.2\text{-}1)$$

where:

E = modulus of elasticity of diaphragm chords, psi

A = area of chord cross-section, in.^{2}

G_a = apparent diaphragm shear stiffness from nail slip and panel shear deformation, kips/in. (from Column A, Tables 4.2A, 4.2B, 4.2C, or 4.2D)

L = diaphragm length, ft

v = induced unit shear in diaphragm, lbs/ft

W = diaphragm width, ft

x = distance from chord splice to nearest support, ft

Δ_c = diaphragm chord splice slip, in., at the induced unit shear in diaphragm

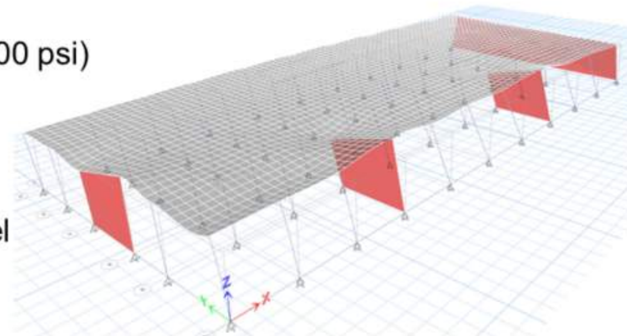
δ_{dia} = maximum mid-span diaphragm deflection determined by elastic analysis, in.

- Finally we check our deflection
  - Remember, we were not close to the maximum aspect ratio of 4:1 so we wouldn’t expect the diaphragm to deflect very far.

- Deflection is a combination of 3 components. Chord deformation, shear/panel shear/nail slip, and chord splice slip. Chord deformation in most cases is negligible. Nail slip and panel shear is where a majority of the deflection occurs.
- We find our deflection to be 0.176” or L/3409. So very stiff for this load.

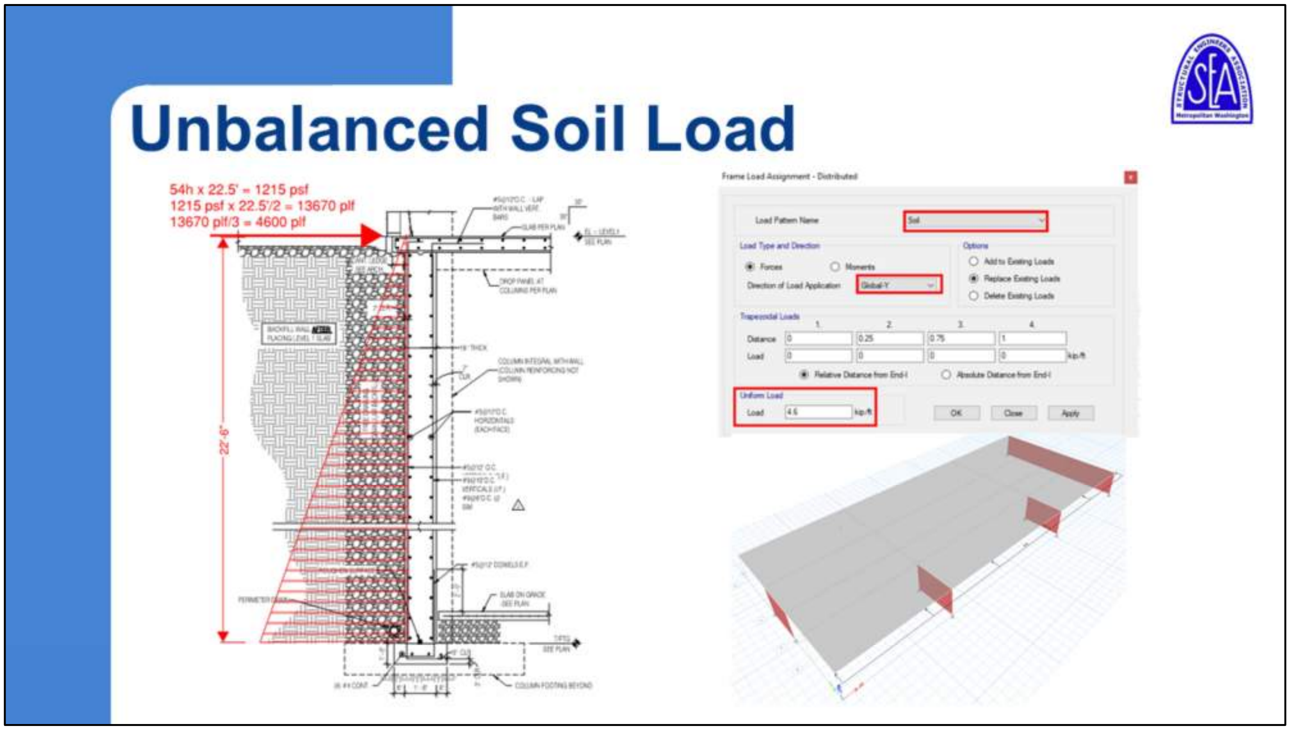
5C. Diaphragm Design Example Concrete - Semi Rigid

- 9” 2-Way Concrete Slab (5000 psi)
- 122’ x 284’
- 22’-6” Unbalanced Soil Load
- Walkout Basement
- ETABS Finite Element Model



JOE

Walk-out basement with unbalanced soil load which is from a project we had recently designed.
9” concrete slab with 22’-6” of unbalance soil load.
We will use etabs to review this as a semi-rigid diaphragm.

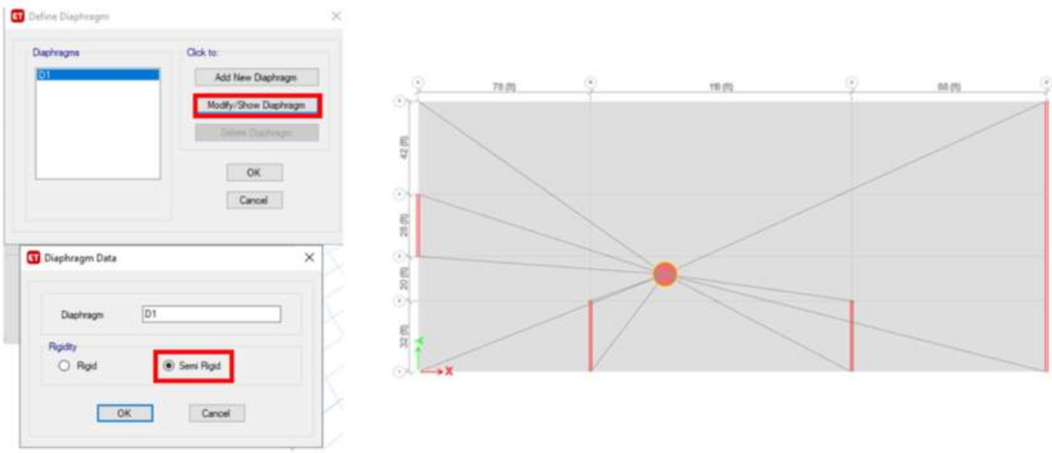


Using our geotechnical report you can find a soil pressure and can see that this is a very sizable load of 4.6 k/ft.

There are multiple ways to lessen this load but this must be accounted for in some manner. This could have been designed as a retaining wall or the base of the wall could have also been designed to be fixed to drag some load away from the diaphragm.

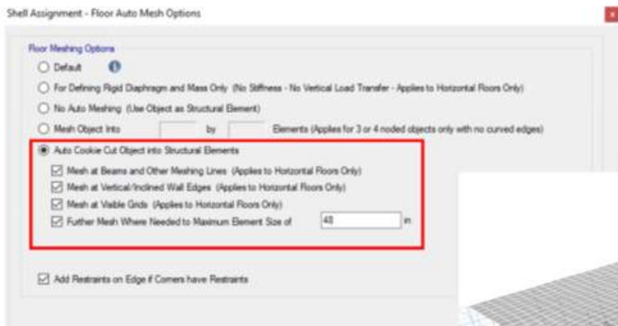
In our case we braced this load with the diaphragm and added additional intermediate shear walls for just the below grade level to cut down on the diaphragm span. As you'll see in the later slides the issue isn't necessarily the diaphragm strength but it's connection to the shear walls,

Define Diaphragm as Semi Rigid

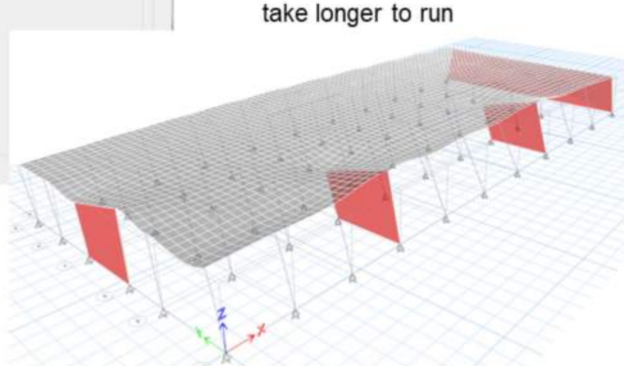


Define our diaphragm as Semi-rigid

Define Mesh Options



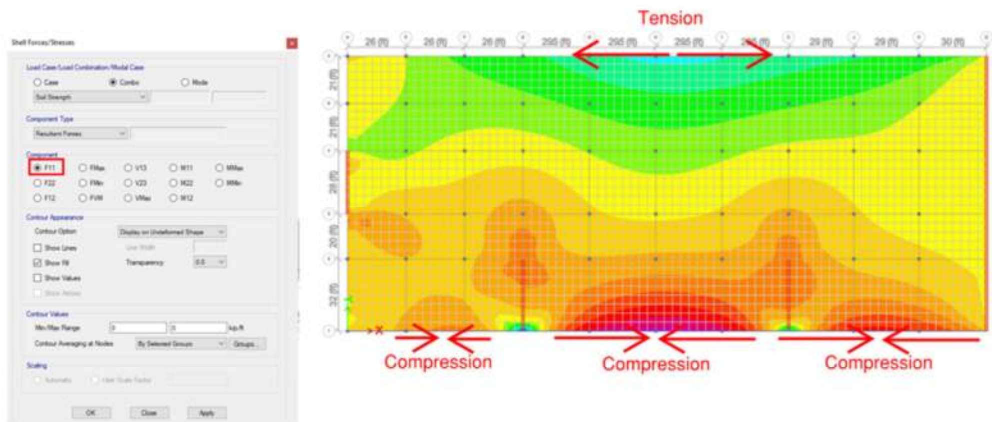
- Mesh to be 1/5 to 1/3 of the bay length or wall length.
- 4' x 4' is common preliminary mesh size
- Can mesh smaller but model will take longer to run



Define our finite element mesh.
The recommended mesh sizes range from 1/5th to 1/3rd of the bay or wall length.
It really depends on how precise you want or need to be. The penalty for a smaller mesh is a model that takes significantly longer to run.
A common mesh size to start with is 4' x 4' or smaller

Display Shell Forces/Stresses

- Find Ultimate Loads for Design of Concrete
- 1.2D + 1.6(L + H)



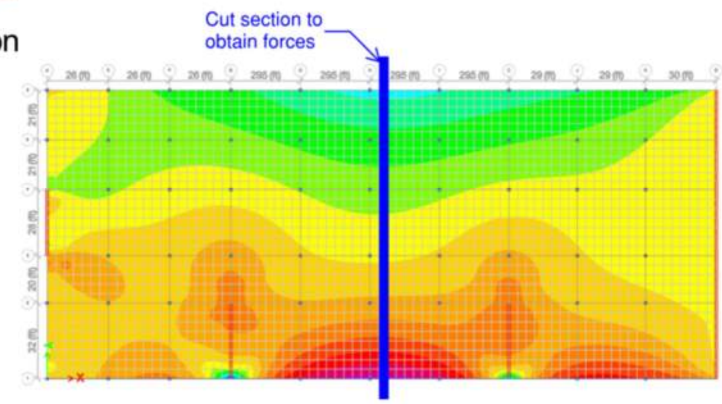
After running the model, we start our checks.

If the stresses exceed the concrete cracking stress, inelastic effects could to be taken into account if you are worried about load distribution
lateral deflections should be okay in a slab this substantial.

Similar to our simple beam analogies, it is easy to understand that the bottom part of this diaphragm is in compression and the top is in tension.

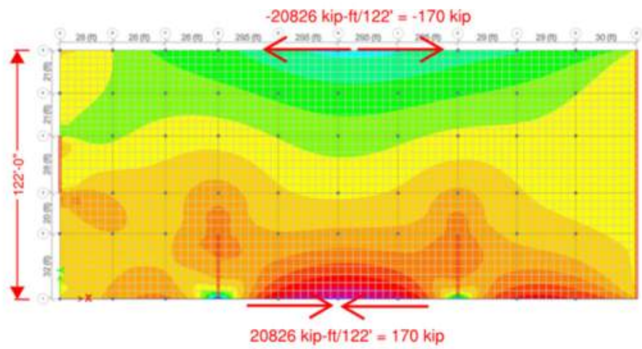
Cut Section to Display Diaphragm Forces Chord Design

Display forces at location of maximum bending



First we want to find our chord steel at our point of maximum moment so we cut a section in order to obtain the forces.

Chord Forces

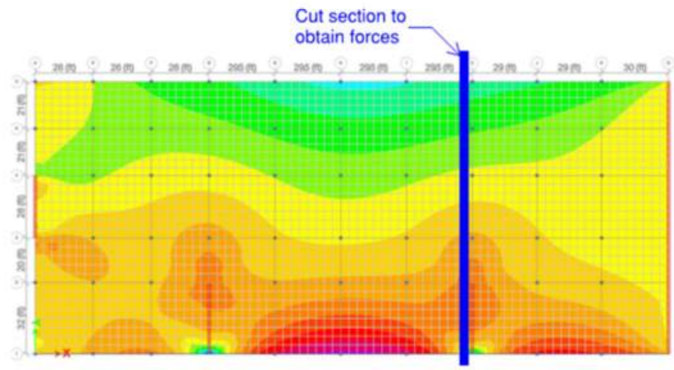


- As = Tu/Φfy
- As = 170k / (0.9*60 ksi) = 3.15 in^2
- 10 #5 within h/4 of tension end (ACI 318 12.5.2.3) = 122'/4 = 30'-6"
- Compression zone usually does not require direct consideration in a concrete diaphragm (except where a strut is close to an opening)

Divide this moment out by our diaphragm depth to find our required tension capacity. Divide this by 0.9 As Fy to find our required area of steel.
And we find that we are good with 10#5 which ACI recommends be placed within h/4 of the tension end.
The compression zone does not usually require direct consideration unless you have an odd case such as a opening near a slab edge. You would then want to check this small sliver of concrete for compression.

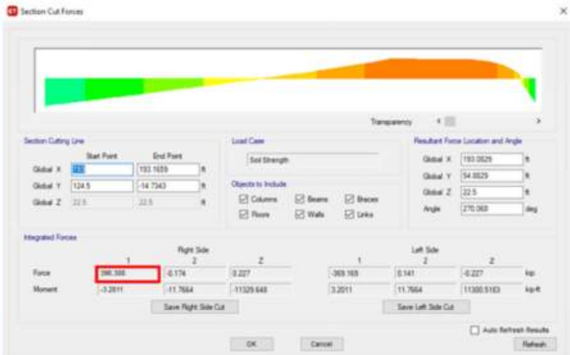
Cut Section to Display Diaphragm Forces Connection to Shear Wall

Display forces adjacent to shear wall



Now we want to check our connection to the shear wall so we cut a section just next to the wall in question.

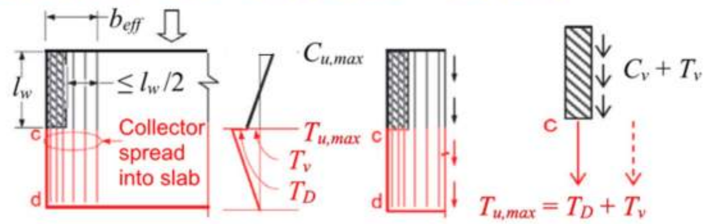
Diaphragm Shear Force



- Every section of diaphragm to be designed/checked.
- $\Phi V_n = \Phi A_{cv}\,(2\lambda\sqrt{f'_c} + \rho_t f_y)$
- $\Phi V_n = \dfrac{0.75 \times 9" \times (122' \times 12) \times 2 \times 1 \times \sqrt{5000}}{1000}$
- $\Phi V_n = 1398k > 397k$

Remember you would want to check every section of a diaphragm but in our case it is obvious the areas that control. Special consideration should be taken around openings or other discontinuities.
Using phi x 2sqrt f'c we can find our allowable shear strength and see that it is greater than our demand.

Connection to Shear Wall

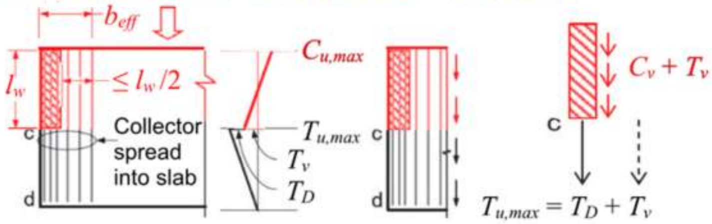


(a) Partial plan (b) Collector actions (c) Force transfer to wall

- Force transfer to is a combination of collector compression, tension, and shear friction.
- Calc has been done for ½ of collector. Opposite side of wall would need to be reviewed for additional collector forces
- b_{eff} = 1'/2 + 32'/2 = 16.5'
- v_{ucd} = 397 k / 122' = 3.25 k/ft
- $T_{u,max} = v_u l_{cd}$ = 3.25 k/ft x 90' = 293k
- $A_{slcd} = T_{u,max}/\Phi f_y$ = 293k / (0.9*60 ksi) = 5.43 in^2
- 18 # 5 in chord additional to slab steel since this is a permanent load.

Now for where our forces become significant, at the connections.
Force transfer to the shear wall will be through a combination of load drug in from the collector and direct shear transfer from the slab.
This calc has only been done for one side of the wall. The same calc would need to be done by cutting a section on the other side.
We first find our collector width which can be approximated as the wall width + ½ the wall length.
We take our shear found earlier and divide this by the diaphragm depth.
We then multiply this by the collector length, similar to the wood example, and find a tension of 293k.
Sizing our tensile reinforcement we then find that we need 5.43 in2 in the collector or 18 #5.
This is clearly too many bars to be developed directly into the shear wall so a majority of this force will need to be drug into the side of the wall.
For this, we are using shear friction.

Connection to Shear Wall

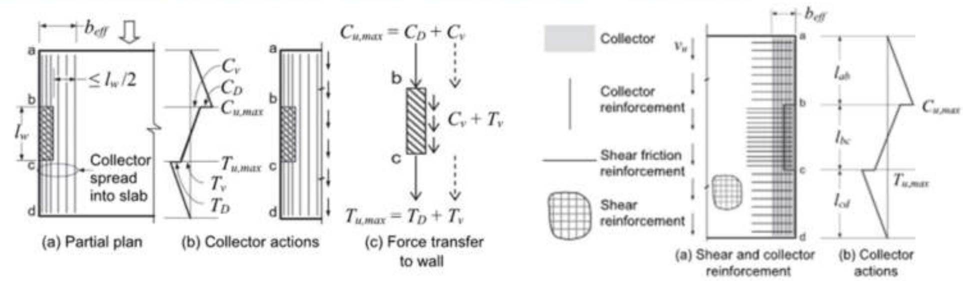


(a) Partial plan (b) Collector actions (c) Force transfer to wall

- Can practically only extend 1#5 collector bar into shear wall.
  - Remainder of load required to be transferred through shear friction.
- $T_{u,max} = T_D + T_v = 293k$ (Drag strut + Shear friction)
- $T_D = 16k$ (1 #5 Drag strut)
- $T_v = 293k - 16k = 277k$ (needs to be transferred thru Shear friction)
- At the wall, the shear friction dowels must transfer 3.25 k/ft + drag strut force
  $V_{ubc} = v_u l_{bc} + T_v = C_v + T_v = 3.25\text{ k/ft} \times 32' + 277k = 381k$

The amount of shear friction we need to drag into the wall is the remaining force not yet developed from the collector + the direct shear along the wall length itself.
If we only develop 1 #5 into the wall we can back calculate that we are left with 277 k from the collector.
Adding this to the direct shear in the wall from the slab we now have to transmit 381k through shear friction.

Connection to Shear Wall



- Size shear friction reinforcement
- A_{vf} = 381k/(0.75 x 60ksi x 1.4 (cast monolithically))
- A_{vf} = *6.1 in^2*
- *6.1 in^2/32' = 0.19in^2/ft = #4 @ 12" o.c.*

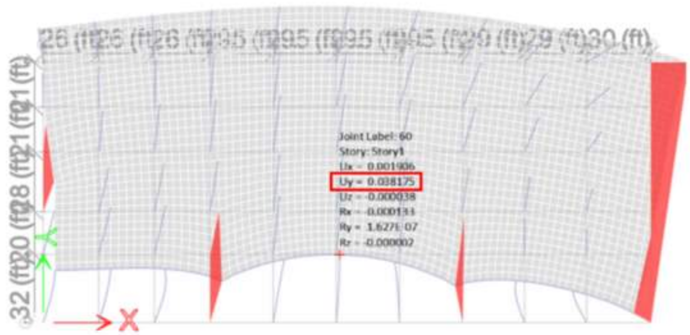
<u>Note</u>: If these were seismic forces, the overstrength factor, Ωo, would be applied to the collector design.

You can then use the shear friction equations from ACI to determine how much reinforcement needs to cross this joint.
In our case we can use #4 @12" o.c.
This scenario works well with concrete but would have been very difficult or impossible with a composite slab. For comparison a composite metal deck can only practically transfer approximately 2klf but our example is closer 12 k/ft at the face of wall. There's no way this would have worked without additional lateral elements to reduce these connection forces.

a. If you are thinking of using wood diaphragms to resist unbalanced earth pressure, be careful and space your vertical elements very closely.

Check Diaphragm Deflection

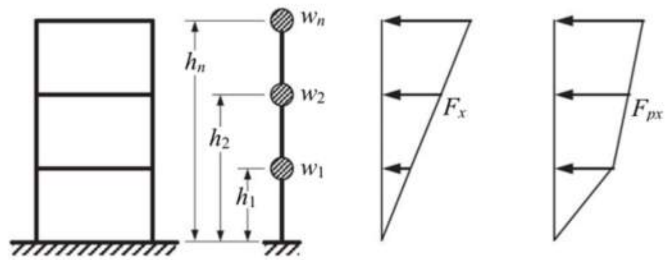
- Find Deflection Using Allowable Combinations D + H (Soil) + L
- Max Deflection = 0.038”
- (118’x12 in/ft)/0.038”
  = L/37263



To finish this example up i thought it would be interesting just to show you how stiff a concrete diaphragm really is. Even with this lateral earth pressure the deflections are practically negligible. The issues really do become the connections and also how this is resisted at the foundation level. At a minimum, if you have some large unbalanced soil pressure, or level that is shedding load to other lateral elements, you should give this some thought at the beginning of a project with some quick hand calcs to determine the magnitude of the forces because it may steer you towards a certain material, spacing of lateral elements, or overall thickness of your diaphragm.

5D. Diaphragm Design - Seismic Calcs

- ASCE 7-16 requires design of diaphragms in Seismic Design Category B thru F



(a) Structure (b) Model (c) Forces for vertical element design (d) Forces for diaphragm design

ROSE

The seismic diaphragm forces are large and often control over wind. Why are they so large?

1. Multistory buildings have numerous vibration modes under seismic excitation.
2. Total acceleration response is a combination of the responses of each individual vibration mode.
3. It has been agreed upon that it is overly-conservative to design the main lateral force resisting system of the building for the maximum of each mode occurring at the same time on each floor, and the code recognizes this and allows the entire system to be designed for a lower force.
4. On the other hand, each floor diaphragm must be strong enough to transfer its own inertial forces to the vertical resisting elements. These inertial forces are the tributary mass x acceleration.
5. As we said earlier, we want to maintain relatively stiff and damage-free diaphragms. That is why diaphragms are designed for essentially linear behaviour under earthquakes.

5D. Diaphragm Design - Seismic Calcs

- ASCE 7-16 Chapter 12 SEISMIC DESIGN REQUIREMENTS, Section 12.10 DIAPHRAGMS, CHORDS, AND COLLECTORS
- Diaphragm force is the larger of :
  - Fx at a floor level from main analysis or
  - the diaphragm force Fpx

The diaphragm design force F_{px}, where

$$F_{px} = \frac{\sum_{i=x}^{n} F_i}{\sum_{i=x}^{n} w_i} w_{px} \qquad \text{(ASCE 7 Eq. 12.10-1)}$$

but not less than

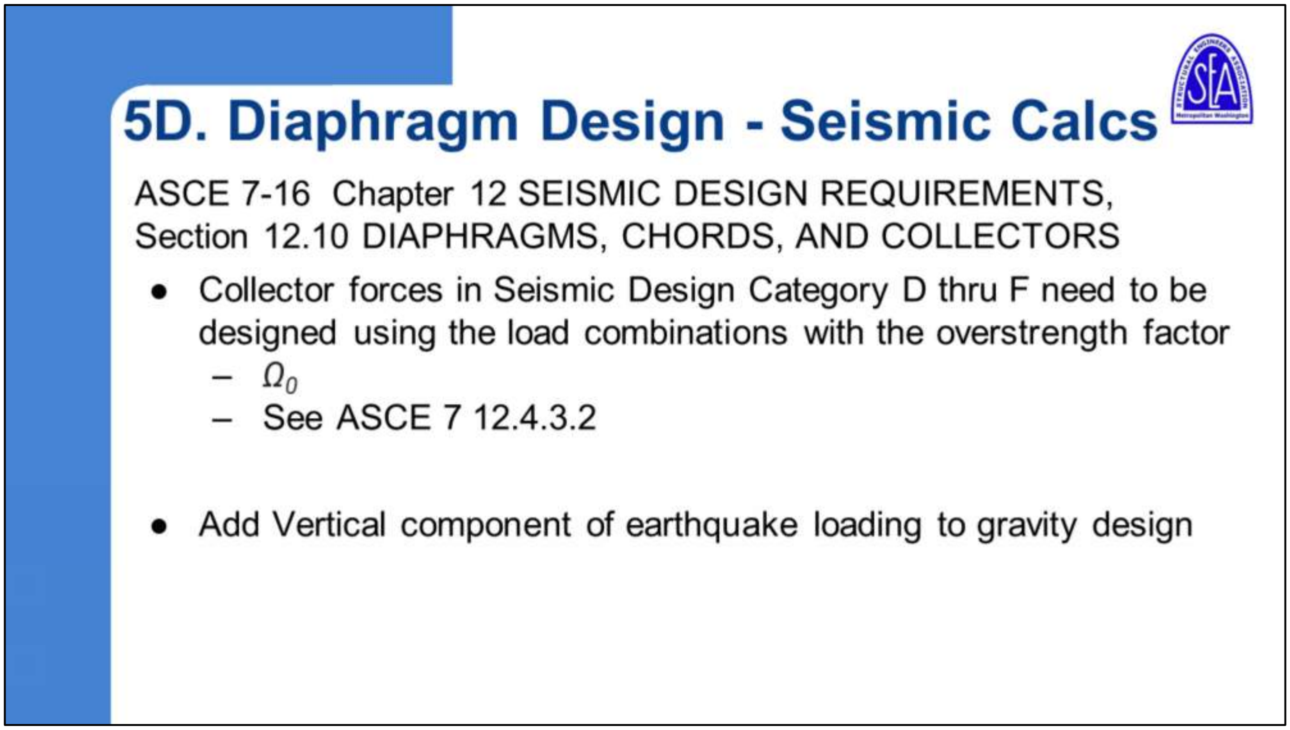
$$F_{px,\min} = 0.2 S_{DS} I_e w_{px}$$

and need not exceed

$$F_{px,\max} = 0.4 S_{DS} I_e w_{px}$$

ROSE
Inertial forces are the larger of the force Fx from the ELFP at that level, or Fpx. Fpx is usually larger, so let's examine this more carefully.

1. F*i* is often based on the Equivalent Lateral Force Procedure
   but the diaphragm inertial force can also be the force at level *i* from the Modal Response Spectrum Analysis.
1. Once this inertial force is determined, it is then added to any transfer forces due to vertical offsets, etc. as previously discussed.



ROSE

1. Chord forces in Seismic Design category C, thru F need to be designed for the overstrength Omega

2. Finally, *floors and roofs also resist vertical component of earthquake loading.*

6. Additional Considerations

Diaphragm discontinuities, vertical offsets, other geometric issues

Expansion Joints and locating vertical load-resisting element

Rose

6A. Additional Considerations

Diaphragm discontinuities, vertical offsets, other geometric issues

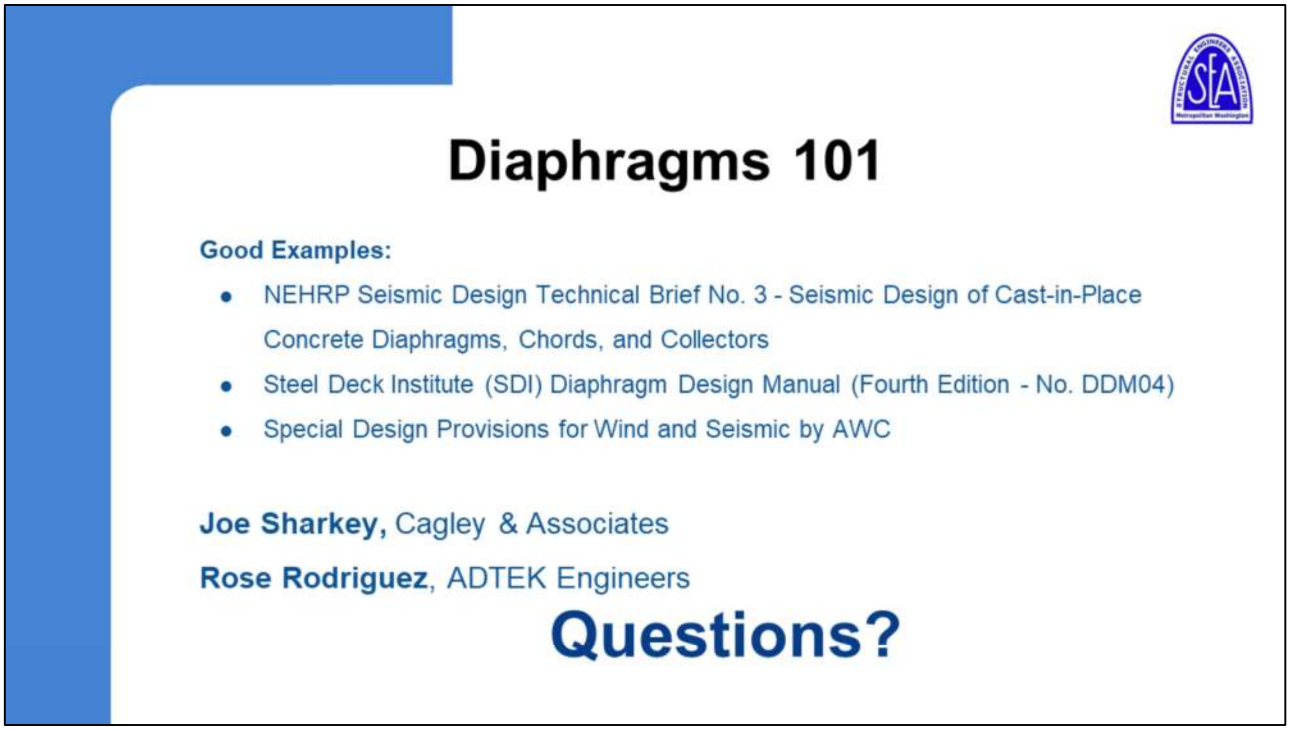
- Add in-plane bracing to stiffen/strengthen an inadequate diaphragm
    - Exposed horizontal X-bracing,
    - Horizontal trusses under the steel deck,
    - Reinforce a composite slab with rebar, etc.
- Vertical Floor Diaphragm Offset - Create continuity with misc members
    - deep beams, diagonal kickers, analysis columns in bending
- Ramps create all sorts of issues:
    - Create short columns created near ramps can generate high shears
    - Ramp parallel to force, it is a strut
    - if forces are perpendicular to a ramp, the ramp acts as an inclined shear wall.

6B. Additional Considerations

What to consider when locating Expansion Joints

- Expansion joints are important in that they limit the length of a diaphragm, preventing issues such as excessive shrinkage cracking in the slab, or shrinkage causing induced lateral loads in vertical elements.
    - 5-story moment frame in Utah that was over 200 ft long; the weld shrinkage was so significant that it pulled the columns out-of-plumb
    - 7-story hospital in Mississippi with bow-tie shape footprint - slab pulled away from the shearwalls because the diaphragm was 450' long on the diagonal.
- Note how far the closest vertical element is from the joint and verify the diaphragm can cantilever from the last vertical element

Rose
clearly, diaphragms stop at expansion joints.



- I hope we have sufficiently scared you into checking your diaphragms.
- Diaphragms are an integral part of the load path and failure of the diaphragm means an incomplete load path.
- To see some good step by step diaphragm design, see the following documents.