POLITECNICO DI MILANO
POLO REGIONALE DI LECCO
Faculty of Engineering
Master of Science in Civil Engineering
Academic Year 2009-2010

Finite Element Analysis of Non-structural Wall Panels under Seismic Action

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Acknowledgement

I would like to acknowledge my tutor Prof. Ing. Marco di Prisco for his advices and instructions for this thesis.

Also, I would like to thank all my classmates for their help and support.

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I. Introduction

The purpose of this thesis is to analyze the behavior of external wall panel (non-structural element) of industrial building under seismic action. The building is single storey precast reinforced concrete frame for manufacturing uses. The main interesting aspect lies in analyzing the reaction forces at the panel nodes which transmit shear forces from the columns to the panel and represent the seismic force acting on non-structural panel.

This report compare results obtained by 4 different methods listed below. Effect of different connection details of panel joints and different modeling approaches will also be discussed here.

1. Linear Static Method (Lateral Force methods specified by EN8), using simplified formula as below:

Base shear force: $F_b = S_d\left(T_1\right)\cdot m\cdot\lambda$

Natural period: $T_1 = C_t \cdot H^{3/4}$ or (better) $T = 2\cdot\pi\cdot\sqrt{\frac{m}{k}}$

Distribution of horizontal seismic force: $F_i = F_b \cdot \frac{z_i \cdot m_i}{\sum z_j \cdot m_j}$

2. Empirical Formula for Non-structural Panel (EN8)

Seismic action on panel: $F_a = \frac{S_a \cdot W_a}{q_a}$

Where $S_a = \alpha\cdot S\cdot\left[\frac{3\cdot\left(1+Z/H\right)}{1+\left(1-T_a/T_1\right)^2}-0.5\right]$

3. Spectral Analysis by Finite Element Software Diana (Linear Dynamic)
4. Non-linear Dynamic Analysis by Diana

In this report, we compare the solutions obtained by different calculation methods. Analysis in the following sections will show that: the results obtained by the first two methods are significantly different. As a result, a numerical analysis by finite element program Diana is necessary to determine which method is better representing the reality. The finite element analysis is capable of simulating more accurate boundary conditions and connections as well as the actual mass distribution. It is generally believed that the result obtained by finite element method (FEM) is reliable and more accurate (this is also due to some experimental data we already have in hands). So, we may use the result obtained by Diana to check the validity of the previous two methods.

The analysis in this project includes:
- Linear Static Analysis by Formula of Lateral Force Method
- EN8 empirical formula for Non-structural element
- different boundary conditions for FEM (fixed, pin, roller)
- Linear Dynamic Analysis by Diana
- Non-linear Dynamic Analysis by Diana

The calculation is simulated under different circumstances, that is for different seismic zones in Italy and for different soil type. The seismic action is characterized by Type I design response spectrum. All the design response spectra in use is according to EN1998-1.

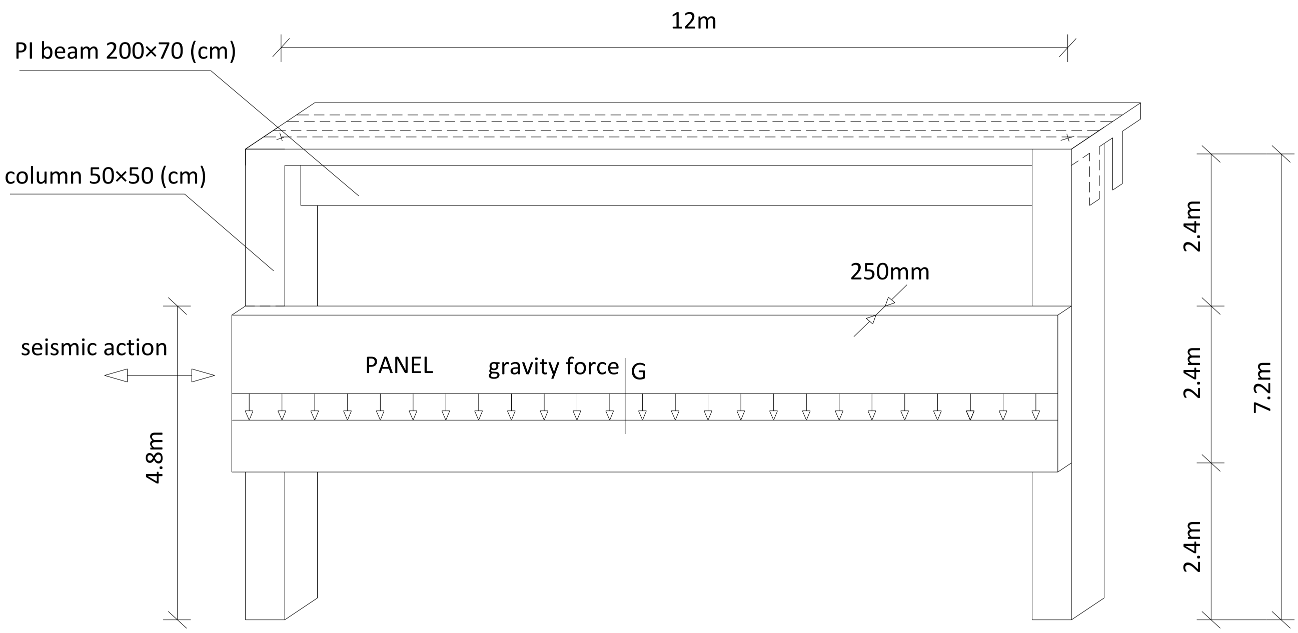
The structure locates in seismic zone Messina. It’s a frame structure with roof tiles and external panels. According to the category of importance classes of buildings, it is considered type II building: single floor or double floors with horizontal panels. Thus the importance factor is 1.0 in consideration.

II. Linear Static Analysis

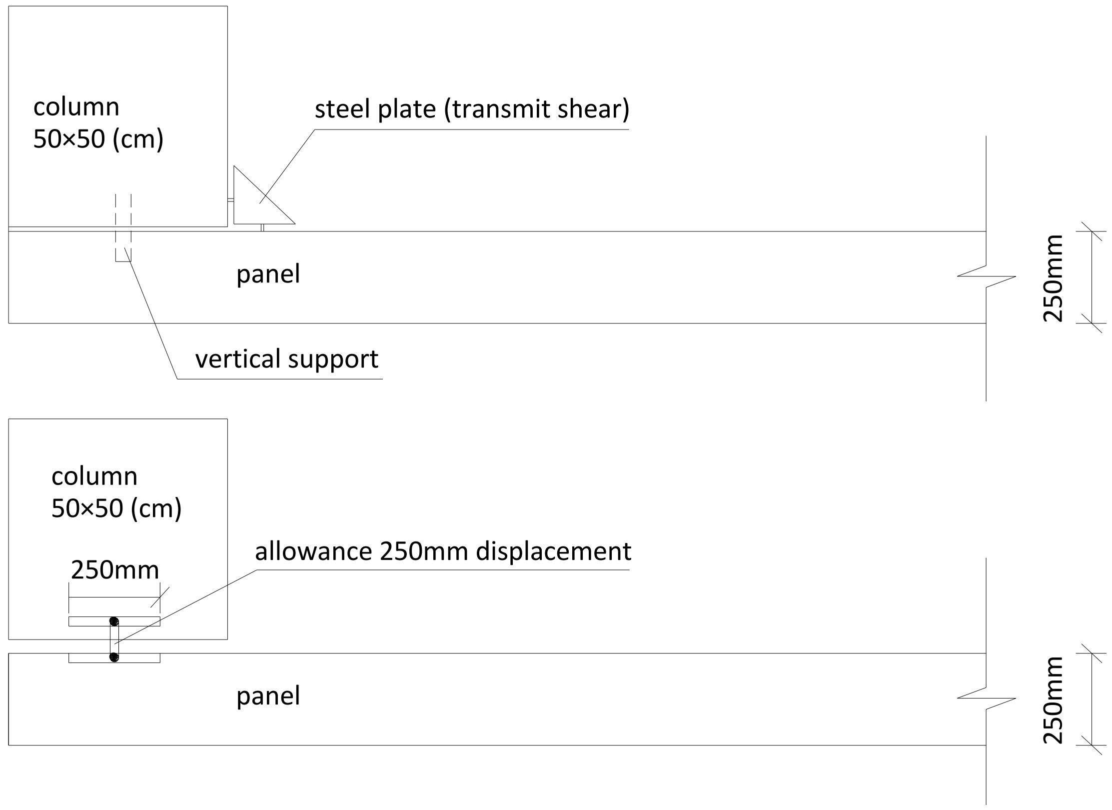
1. Model of Analysis (one floor & single span)

--Geometry of the Model--

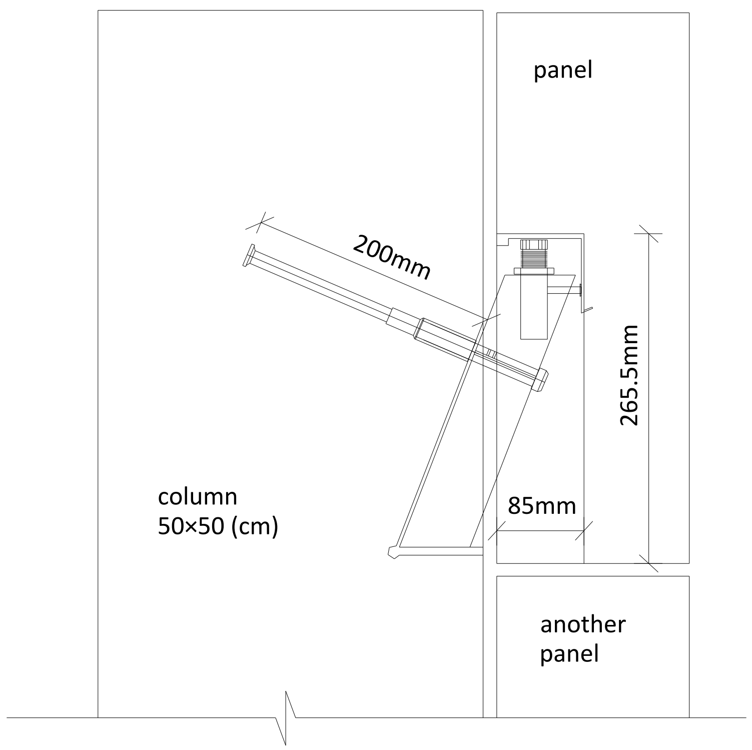
To check the validity of the formula suggested by Eurocode, I build a model with only one panel in the middle of a single span and one storey frame which has a total height of 7.2m. The geometry and details of the connections is plot below. The panel is in the middle of the height. While the panel can also cover the top 2.4m or at bottom, analysis approach for panels locate in different places will be similar to the one locates in the middle.



The panel in the middle of the column is 12m long, 2.4m wide and 250mm thick. The panel is attached to the outer surface of the external columns. For panel, only consider its mass and gravity force as distributed load while its contribution to the lateral stiffness of the structure will not be taken into account at first. Vertical supports are at the two bottom joints of the panel. There is a steel plate which can transmit lateral shear to the column from panel (see detail connection plot), it locates in one bottom joint, assume left here. There are certain allowance for displacement at top joints of about 250mm whose detailing is also plot below.

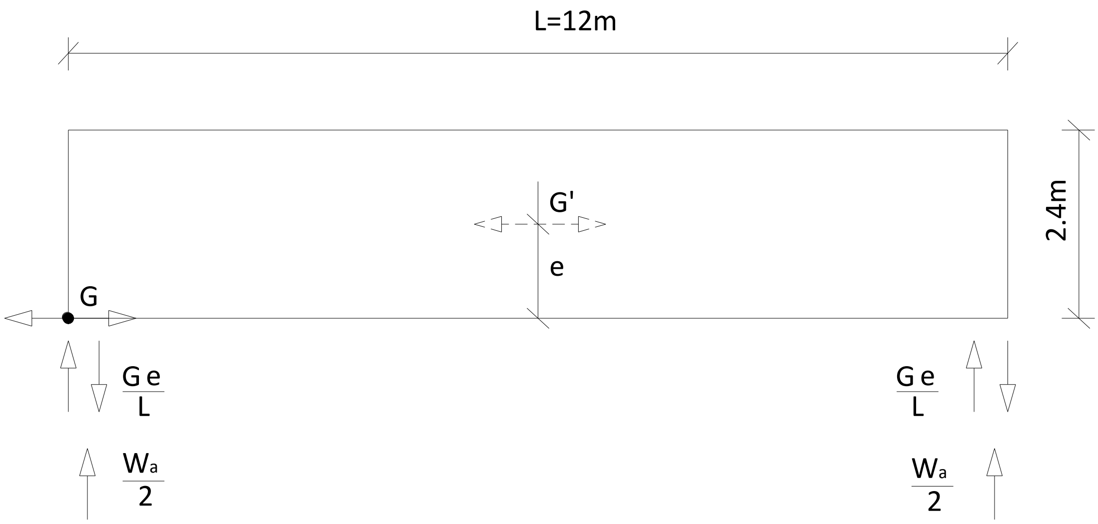


See detail connection below, we can assume a well attached connection in orthogonal to panel direction. Detailed panel column connection according to the CAD drawing HALFEN-DEHA "sezione e pianta-pannello aperto":



-Consideration of Eccentricity-

The centre of mass of the panel lies in the middle point, however the support is on the lower boundary. As a result, there is an eccentricity “e”. Consider the total mass concentrating at the lower left nodal of the panel while the “e” results in two vertical axial forces in both columns, one upwards and one downwards. So, in this case the height of the panel measured from ground is 2.4m.



G’ is the horizontal inertial force, which is replaced by G=G’ acting on the lower left node and one couple forces $\frac{G\,e}{L}$ pointing at opposite directions. The couple forces contribute to vertical axial forces and transmit directly to the base, therefore no effect on lateral forces.

-B.C- (boundary condition)

Boundary condition at two upper nodes C and D are rollers, but with allowance of movement within 250mm. There is a triangular steel plate at node A, so it’s pin connected. In real case, friction at vertical support is not taken into account because in construction application, workers simply put down the panel, there is no enough work space to guarantee a fixed connection at the joint A. However the steel plate can transmit lateral forces in shear direction.

The node B on the bottom right is a special joint connection. In reality, because of friction, when the ratio between shear force and normal force is small, the joint is not able to slide horizontally, it is pin connected to column. However, when the ratio is large, the joint exceeds its friction limit and begin to slide like a roller with a constant frictional force μF. It can hardly be distinguished in linear static analysis since only lump model and simple boundary condition are considered. However, in the finite element numerical model we will simulate the upper and lower bound condition of the real case.

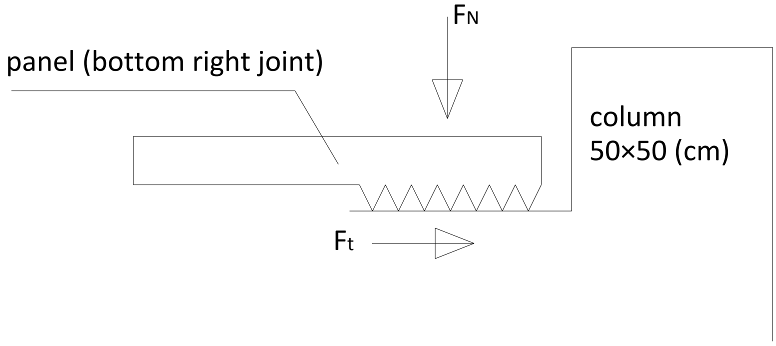
Connection Details:

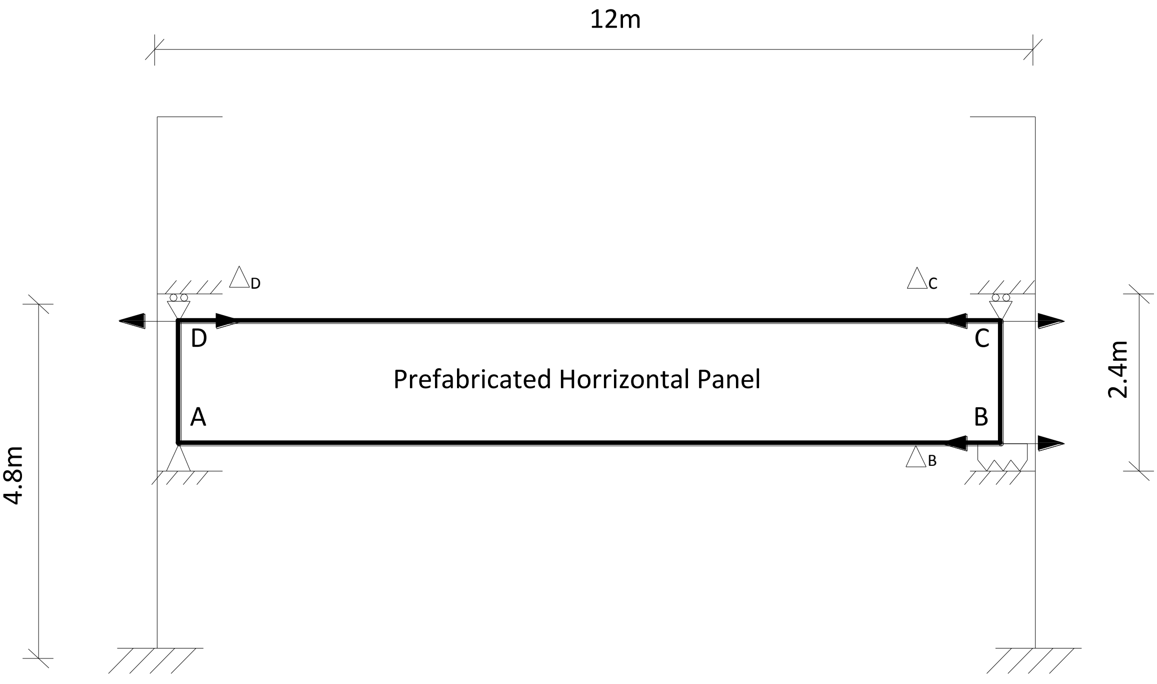
- Before initiation of sliding, the joint connection is **PIN CONNECTED**
- Threshold value for beginning of sliding is

$$F_t = \mu F_N \qquad \mu: 0.1 \sim$$

- After sliding, the joint become a **ROLLER WITH FRICTION FORCE**
  The shear force transmitted from panel joint to column is equal to the friction force which is also proportional to the normal force

$$F_a = F_t = \mu \ F$$

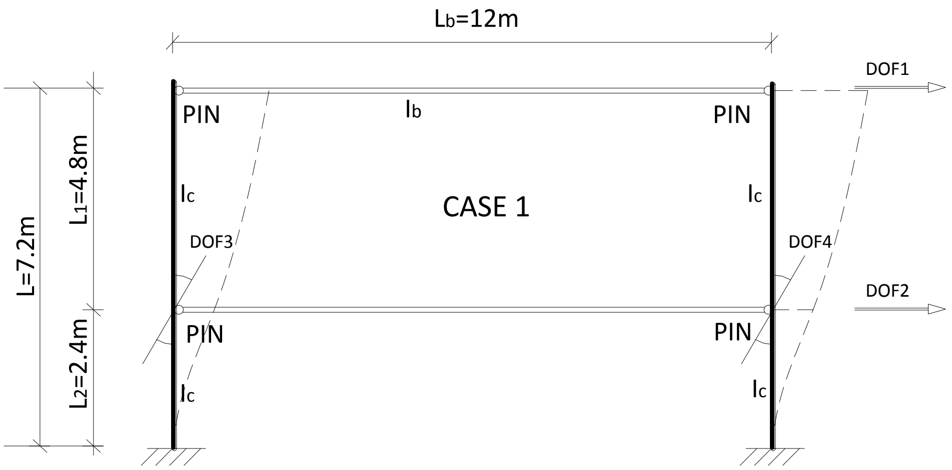




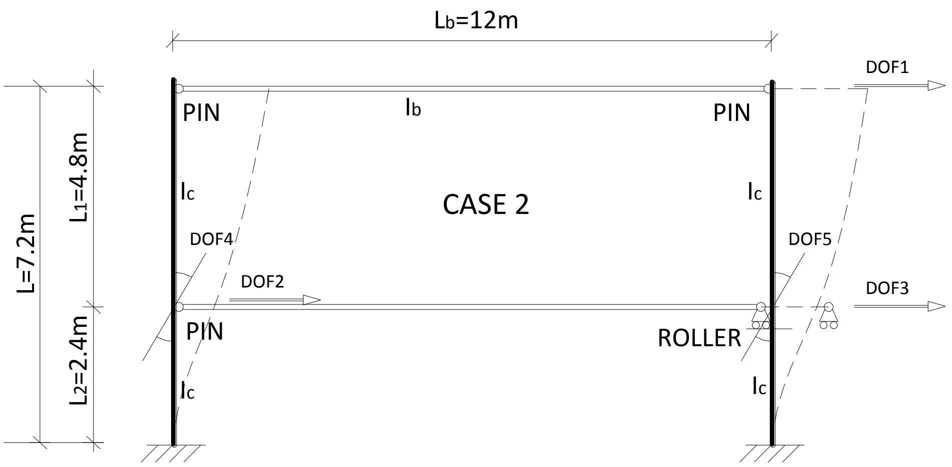
-The Mass Lumps Model-

Due to the connection of beam-columns and panel node B to column. There are four cases to be analyzed. The joint connection of node B-column is not exactly pin or roller, actually, it should be able to slide with a constant friction value μF when exceeding the friction limit for shear force. However I consider two extreme situations here, that is the upper bound (roller) and lower bound (pin). As is plot in the figure below, several mass lumps are considered, with both translational DOFs and rotational DOFs.

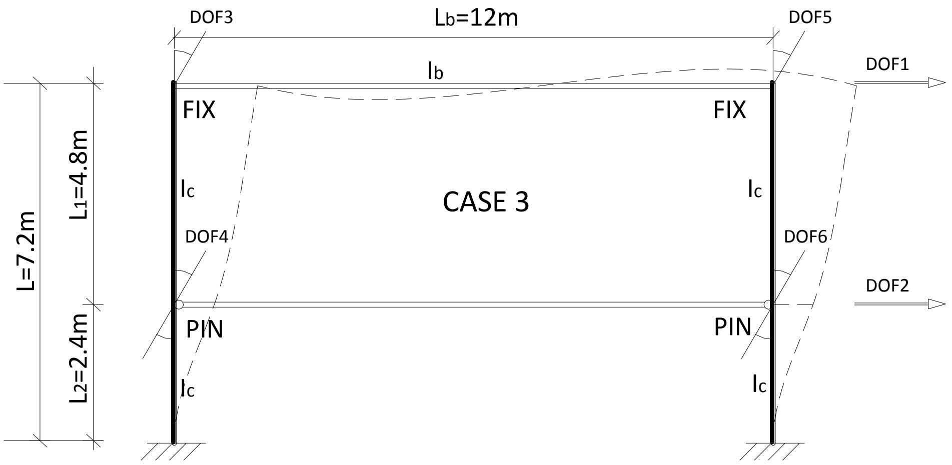
The mass of columns are considered to be concentrated on level 2.4m and 7.2m taken into account effective zone: half length of adjacent spans. The load of roof tiles is taken into account as weight (mass) acting on beam level. In Case 2 and Case 4, the weight of panel is concentrated on left joint of the column. In the cases with a roller, there are 2 translational DOFs at panel level.



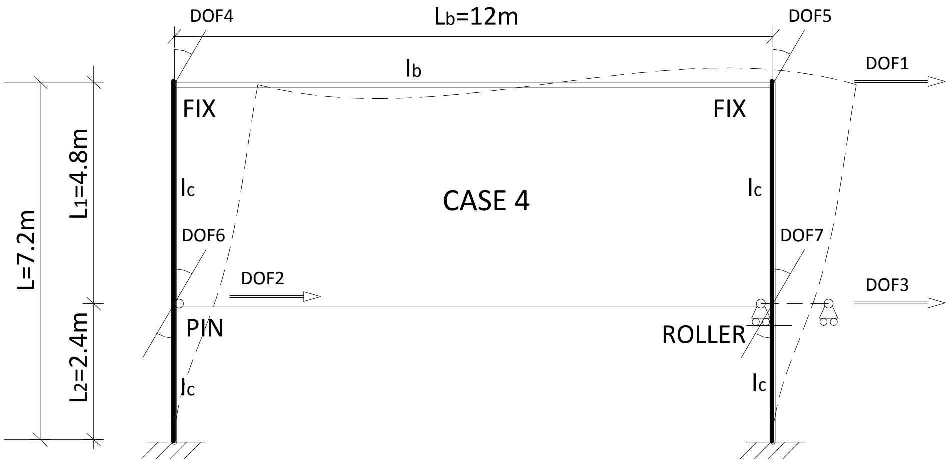
Case1: 4 pin connections; 2 translational DOFs, 2 rotational DOFs, axis symmetric situation



Case2: 3pins and 1 roller, 3 translational DOFs, 2 rotational DOFs



Case3: 2pins with fixed beam, 2 translational DOFs, 4 rotational DOFs



Case4: 1 pin +1 roller + 2 fix, 3 translational DOFs, 4 rotational DOFs

-Materials-

The column is precast reinforced concrete:

Material for C45/55, is $\gamma_c = 25.5\,\mathrm{kN/m^3}$, E=38200 $\mathrm{N/mm^2}$, ν=0.15.

The panel is mainly made of polymer content----a composition of concrete and some light material and is prefabricated, for polystyrene as suggested by EN1991-1: the density is 0.3 $\mathrm{kN/m^3}$.However the density here according to some document is given in 2-D form in $\mathrm{m^2}$: that is $4\,\mathrm{kN/m^2}$.

For roof tiles, $0.4\,\mathrm{kN/m^2}$.

-Element Type and Cross-sections-

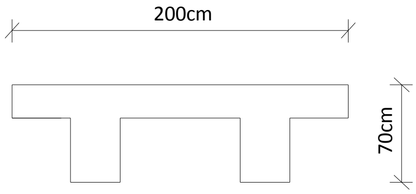
Columns:

-Dimension: 50cm×50cm

-Area: 2500$\mathrm{cm^2}$

-Moment of inertial of principle axis (I3): 520833$\mathrm{cm^4}$

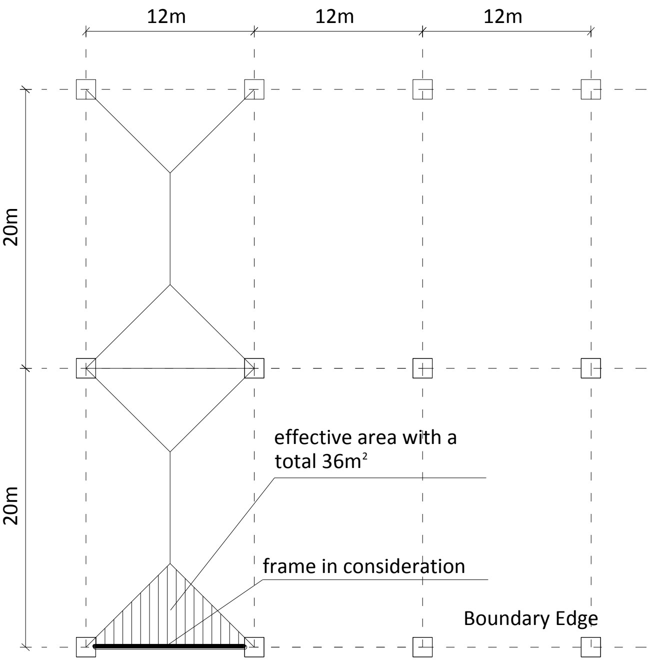
Beam:



-Dimension: π-beam 200cm×70cm
-Area: 3340cm^2
-Moment of inertial: 1684192cm^4
Panel:
-Dimension: 12m×2.4m×0.25m
-Surface Area: 28.8m^2
Roofing System:
The frame in consider is a part of a complete industrial building plot as below, assuming the span in our case locates at the left corner.
For 2-way slab with 4 fixed end lines, the tributary area for load transmission is plot as below. This is also according to the plane geometry of the whole structure. We could derive the dead weight load acting on this single frame on the top beam.

The dead weight could be simplified as uniformly distributed along the beam for convenience:

$$g_k = \frac{0.4 \times 36}{12} = 1.2\, kN/m$$



- Load Combination-

Load Combination under seismic action and ground acceleration according to EN1990, there are two cases to be consider.

For EQU: $Com1 = \gamma_G G_k + \gamma_{Q,1} Q_k + \gamma_{Q,2} \psi_{0.2} S$

For SEISMIC: $Com2 = G_k + \gamma_1 A_{Ek} + \psi_{2.1} Q_k + \psi_{2.2} S$

Take SEISMIC combination in the analysis since it's more critical. As defined in EN1991-1 (6.3.4), for roof category H "Roofs not accessible except for normal maintenance and repair", the live load per square meter is $0.4\,\mathrm{kN/m^2}$, as a result $q_k = 1.2\,\mathrm{kN/m}$.

For snow $s = \mu_i C_e C_t S_k = 0.8 \times 1.6 = 1.28\,kN/m^2$, $s = 3.84\,\mathrm{kN/m}$

Other coefficient $\gamma_1 = 1.0$; $\psi_{2,1} = \psi_{2,2} = 0.3$; A_{Ek} is seismic action

As a result, I can calculate the total load acting on the top beam from the roof system:

$$W_{roof} = g_k L_b + \psi_{2.1} q_k L_b + \psi_{2.2} s L_b = 1.2 \times 12 + 0.3 \times 12 \times (1.2 + 3.84) = 32.544kN$$

-Mass and Stiffness-

First calculate the weight of the horizontal prefabricated panel.

The weight of panel:

$$W_a = 4 \times A_a = 4 \times 28.8 = 115.2kN$$

The distributed weight of columns (weight per meter length):

$$w_c = 2 \cdot b_c \cdot h_c \cdot \gamma_c = 2 \times 0.5 \times 0.5 \times 25 = 12.5\,kN/m$$

The weight of beam:

$$W_b = A_b \cdot L_b \cdot \gamma_c = 3340 \times 10^{-4} \times 12 \times 25 = 100.2kN$$

$$W_{roof} = 32.544kN$$

The total weight of the frame can be calculated;

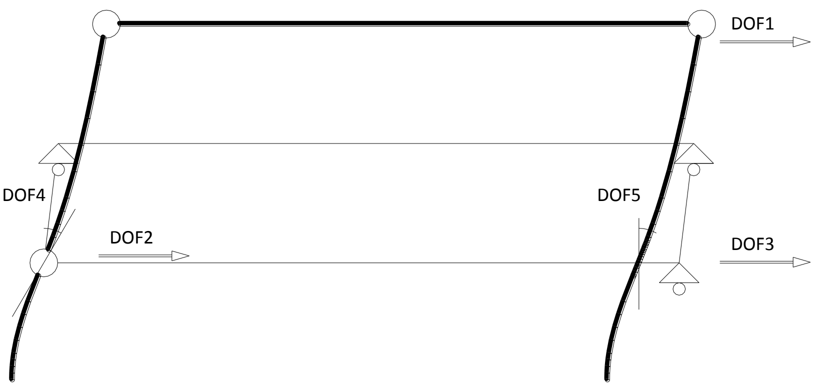
$$m_{total} = W_b + W_r + W_a + W_c = 100.2 + 32.544 + 115.2 + 12.5 \times 7.2 = 337.944kN$$

Mass & Stiffness Matrix Case 1)

(N,m)						
No. of column	E	I	L1	L2	LB	IB
2	3.82E+10	5.21E-03	4.8	2.4	12	1.68E-02
DOF						
4						

	1	2	3	4	
M1	162.744	0	0	0	(kN)
2	0	160.2	0	0	
3	0	0	0	0	
4	0	0	0	0	
K1	$2 \times \frac{3E_cI_c}{L_1^3}$	$-2 \times \frac{3E_cI_c}{L_1^3}$	$-\frac{3E_cI_c}{L_1^2}$	$-\frac{3E_cI_c}{L_1^2}$	(-)
2	$-2 \times \frac{3E_cI_c}{L_1^3}$	$2 \times \left(\frac{3E_cI_c}{L_1^3} + \frac{12E_cI_c}{L_2^3}\right)$	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	
3	$-\frac{3E_cI_c}{L_1^2}$	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	$\frac{3E_cI_c}{L_1} + \frac{4E_cI_c}{L_2}$	0	
4	$-\frac{3E_cI_c}{L_1^2}$	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	0	$\frac{3E_cI_c}{L_1} + \frac{4E_cI_c}{L_2}$	
K1	1.0794E+04	-1.0794E+04	-2.5906E+04	-2.5906E+04	(kN,m)
2	-1.0794E+04	3.5621E+05	-1.8134E+05	-1.8134E+05	
3	-2.5906E+04	-1.8134E+05	4.5595E+05	0.0000E+00	
4	-2.5906E+04	-1.8134E+05	0.0000E+00	4.5595E+05	

Mass & Stiffness Matrix Case 2)



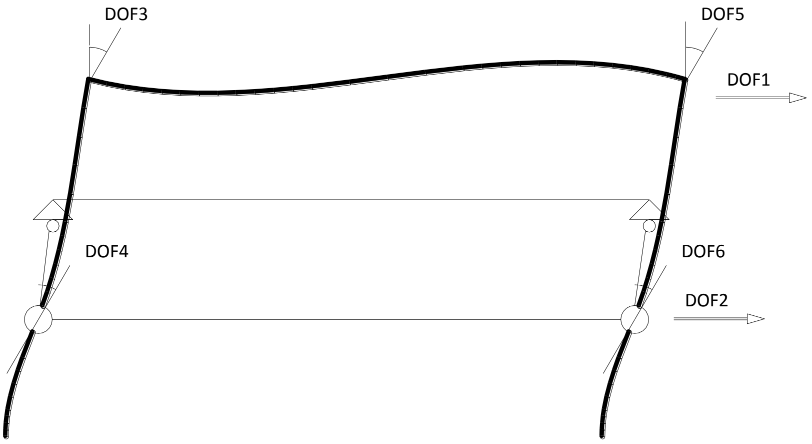
(N,m)

No. of column	E	I	L1	L2	LB	IB
2	3.82E+10	5.21E-03	4.8	2.4	12	1.68E-02

DOF
5

	1	2	3	4	5	
M1	162.744	0	0	0	0	(kN)
2	0	137.7	0	0	0	
3	0	0	22.5	0	0	
4	0	0	0	0	0	
5	0	0	0	0	0	
K1	$2 \times \frac{3E_cI_c}{L_1^3}$	$-\frac{3E_cI_c}{L_1^3}$	$-\frac{3E_cI_c}{L_1^3}$	$-\frac{3E_cI_c}{L_1^2}$	$-\frac{3E_cI_c}{L_1^2}$	(-)
2	$-\frac{3E_cI_c}{L_1^3}$	$\frac{3E_cI_c}{L_1^3} + \frac{12E_cI_c}{L_2^3}$	0	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	0	
3	$-\frac{3E_cI_c}{L_1^3}$	0	$\frac{3E_cI_c}{L_1^3} + \frac{12E_cI_c}{L_2^3}$	0	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	
4	$-\frac{3E_cI_c}{L_1^2}$	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	0	$\frac{3E_cI_c}{L_1} + \frac{4E_cI_c}{L_2}$	0	
5	$-\frac{3E_cI_c}{L_1^2}$	0	$\frac{3E_cI_c}{L_1^2} - \frac{6E_cI_c}{L_2^2}$	0	$\frac{3E_cI_c}{L_1} + \frac{4E_cI_c}{L_2}$	
K1	1.0794E+04	-5.3971E+03	-5.3971E+03	-2.5906E+04	-2.5906E+04	(kN m)
2	-5.3971E+03	1.7810E+05	0.0000E+00	-1.8134E+05	0.0000E+00	
3	-5.3971E+03	0.0000E+00	1.7810E+05	0.0000E+00	-1.8134E+05	
4	-2.5906E+04	-1.8134E+05	0.0000E+00	4.5595E+05	0.0000E+00	
5	-2.5906E+04	0.0000E+00	-1.8134E+05	0.0000E+00	4.5595E+05	

Mass & Stiffness Matrix Case 3)



(N,m)

No. of column	E	I	L1	L2	LB	IB
2	3.82E+10	5.21E-03	4.8	2.4	12	1.68E-02

DOF

6

	1	2	3	4	5	6	
M1	162.744	0	0	0	0	0	(kN)
2	0	160.2	0	0	0	0	
3	0	0	0	0	0	0	
4	0	0	0	0	0	0	
5	0	0	0	0	0	0	
6	0	0	0	0	0	0	
K1	$2\times\frac{12E_cI_c}{L_1^3}$	$-2\times\frac{12E_cI_c}{L_1^3}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	(-)
2	$-2\times\frac{12E_cI_c}{L_1^3}$	$\frac{12E_cI_c}{L_1^3}+\frac{12E_cI_c}{L_2^3}$	$\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	$\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	
3	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}$	$\frac{4E_cI_c}{L_1}+\frac{4E_bI_b}{L_b}$	$\frac{2E_cI_c}{L_1}$	$\frac{2E_bI_b}{L_b}$	0	
4	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	$\frac{2E_cI_c}{L_1}$	$\frac{4E_cI_c}{L_1}+\frac{4E_cI_c}{L_2}$	0	0	
5	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^3}$	$\frac{2E_bI_b}{L_b}$	0	$\frac{4E_cI_c}{L_1}+\frac{4E_bI_b}{L_b}$	$\frac{2E_cI_c}{L_1}$	
6	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	0	0	$\frac{2E_cI_c}{L_1}$	$\frac{4E_cI_c}{L_1}+\frac{4E_cI_c}{L_2}$	
K1	4.3177E+04	-4.3177E+04	-5.1812E+04	-5.1812E+04	-5.1812E+04	-5.1812E+04	(kN,m)
2	-4.3177E+04	3.8859E+05	5.1812E+04	-1.5544E+05	5.1812E+04	-1.5544E+05	
3	-5.1812E+04	5.1812E+04	3.8025E+05	8.2899E+04	1.0723E+05	0.0000E+00	
4	-5.1812E+04	-1.5544E+05	8.2899E+04	4.9740E+05	0.0000E+00	0.0000E+00	
5	-5.1812E+04	5.1812E+04	1.0723E+05	0.0000E+00	3.8025E+05	8.2899E+04	
6	-5.1812E+04	-1.5544E+05	0.0000E+00	0.0000E+00	8.2899E+04	4.9740E+05	

Mass & Stiffness Matrix Case 4)

(N,m)

No. of column	E	I	L1	L2	LB	IB
2	3.82E+10	5.21E-03	4.8	2.4	12	1.68E-02

DOF
7

	1	2	3	4	5	6	7	
M1	162.744	0	0	0	0	0	0	(kN)
2	0	137.7	0	0	0	0	0	
3	0	0	22.5	0	0	0	0	
4	0	0	0	0	0	0	0	
5	0	0	0	0	0	0	0	
6	0	0	0	0	0	0	0	
7	0	0	0	0	0	0	0	
K1	$2\times\frac{12E_cI_c}{L_1^3}$	$-\frac{12E_cI_c}{L_1^3}$	$-\frac{12E_cI_c}{L_1^3}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	$-\frac{6E_cI_c}{L_1^2}$	(-)
2	$-\frac{12E_cI_c}{L_1^3}$	$\frac{12E_cI_c}{L_1^3}+\frac{12E_cI_c}{L_2^3}$	0	$\frac{6E_cI_c}{L_1^2}$	0	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	0	
3	$-\frac{6E_cI_c}{L_1^2}$	0	$\frac{12E_cI_c}{L_1^3}+\frac{12E_cI_c}{L_2^3}$	0	$\frac{6E_cI_c}{L_1^2}$	0	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	
4	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}$	0	$\frac{4E_cI_c}{L_1}+\frac{4E_bI_b}{L_b}$	$\frac{2E_bI_b}{L_b}$	$\frac{2E_cI_c}{L_1}$	0	
5	$-\frac{6E_cI_c}{L_1^2}$	0	$\frac{6E_cI_c}{L_1^2}$	$\frac{2E_bI_b}{L_b}$	$\frac{4E_cI_c}{L_1}+\frac{4E_bI_b}{L_b}$	0	$\frac{2E_cI_c}{L_1}$	
6	$-\frac{6E_cI_c}{L_1^2}$	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	0	$\frac{2E_cI_c}{L_1}$	0	$\frac{4E_cI_c}{L_1}+\frac{4E_cI_c}{L_2}$	0	
7	$-\frac{6E_cI_c}{L_1^2}$	0	$\frac{6E_cI_c}{L_1^2}-\frac{6E_cI_c}{L_2^2}$	0	$\frac{2E_cI_c}{L_1}$	0	$\frac{4E_cI_c}{L_1}+\frac{4E_cI_c}{L_2}$	
K1	4.3177E+04	-2.1588E+04	-2.1588E+04	-5.1812E+04	-5.1812E+04	-5.1812E+04	-5.1812E+04	(kN,m)
2	-2.1588E+04	1.9430E+05	0.0000E+00	5.1812E+04	0.0000E+00	-1.5544E+05	0.0000E+00	
3	-2.1588E+04	0.0000E+00	1.9430E+05	0.0000E+00	5.1812E+04	0.0000E+00	-1.5544E+05	
4	-5.1812E+04	5.1812E+04	0.0000E+00	3.8025E+05	1.0723E+05	8.2899E+04	0.0000E+00	
5	-5.1812E+04	0.0000E+00	5.1812E+04	1.0723E+05	3.8025E+05	0.0000E+00	8.2899E+04	
6	-5.1812E+04	-1.5544E+05	0.0000E+00	8.2899E+04	0.0000E+00	4.9740E+05	0.0000E+00	
7	-5.1812E+04	0.0000E+00	-1.5544E+05	0.0000E+00	8.2899E+04	0.0000E+00	4.9740E+05	

--Seismic Action--

Design horizontal response spectrum for SLC (Limit State of Collapse)

Formula:

According to EN1998-1, for the horizontal components of the seismic action the design spectrum, $S_d(T)$, shall be defined by the following expressions:

$$0 \le T \le T_B \qquad S_d(T) = a_g \cdot S \cdot \left[\frac{2}{3} + \frac{T}{T_B} \cdot \left(\frac{2.5}{q} - \frac{2}{3} \right) \right]$$

$$T_B \le T \le T_C \qquad S_d(T) = a_g \cdot S \cdot \frac{2.5}{q}$$

$$T_C \le T \le T_D \qquad S_d(T) = \begin{cases} = a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \left[\dfrac{T_C}{T} \right] \\ \ge \beta \cdot a_g \end{cases}$$

$$T_D \le T \qquad S_d(T) = \begin{cases} = a_g \cdot S \cdot \dfrac{2.5}{q} \cdot \left[\dfrac{T_C T_D}{T^2} \right] \\ \ge \beta \cdot a_g \end{cases}$$

Where:

$S_d(T)$ is design response spectrum, a function of T

T is the vibration period of linear single-degree-of-freedom system

a_g is the design ground acceleration on type A ground

$a_g = \gamma_1 a_{gR}$

γ_1 is topographical amplification effect on different important classes of structures, for type II normal, $\gamma_1 = 1.0$.

T_B is the lower limit of the period of the constant spectral acceleration branch

T_C is the upper limit of the period of the constant spectral acceleration branch

T_D is the value defining the beginning of the constant displacement response range of the spectrum

S soil factor according to ground type.

β is the lower bound factor for horizontal design spectrum β=0.2 as recommended

q behavior factor

For structure regular in elevation $q_0 = 3.0\,\alpha_u/\alpha_1$ for DCM.

For one-storey building $\alpha_u/\alpha_1 = 1.1$

$k_w = 1.0$ reflecting the previous failure mode

$q = q_0 k_w \ge 1.5$

For Ground Type A,B,C,D and for Type I response spectrum

Type A: S=1.0 $T_B = 0.15(s)$ $T_C = 0.4(s)$ $T_D = 2.0(s)$

Type B: S=1.2 $T_B = 0.15(s)$ $T_C = 0.5(s)$ $T_D = 2.0(s)$

Type C: S=1.15 $T_B = 0.2(s)$ $T_C = 0.6(s)$ $T_D = 2.0(s)$

Type D: S=1.35 $T_B = 0.2(s)$ $T_C = 0.8(s)$ $T_D = 2.0(s)$

2. Eigen Modes Analysis

Using Matlab to calculate Free Vibration Periods of the 4 models. Here I choose lateral force method for analysis(EN1998-1 4.3.3.2), according to sub-clause (1), "This type of analysis may be applied to buildings whose response is not significantly affected by contributions from modes of vibration higher than the fundamental mode in each principal direction." I take into account only the effect of first mode.

Case 1)
EIGEN VECTORS
-0.0000 0.0077 -0.0011 0.0000
-0.0000 0.0012 0.0077 -0.0000
7.0036 0.0009 0.0030 7.0036
7.0036 0.0009 0.0030 -7.0036
EIGEN VALUES
211491.617 13.735 115.171 211491.596
First Mode Period: 0.457 [s]
Second Mode Period: 0.0087 [s]
Case 2)
EIGEN VECTORS
-0.0000 -0.0000 -0.0004 0.0077 -0.0011
-0.0000 -0.0000 0.0000 0.0012 0.0084
-0.0001 0.0000 0.0208 0.0011 -0.0002
0.0106 3.1321 -0.0000 0.0009 0.0033
3.1321 -0.0106 0.0083 0.0009 -0.0001
EIGEN VALUES
66879.751 66879.554 215.284 13.734 87.712
First Mode Period: 0.458 [s]
Second Mode Period: 0.011 [s]
Case 3)
EIGEN VECTORS
-0.0000 -0.0000 0.0076 -0.0018 -0.0000 -0.0000
-0.0000 0.0000 0.0018 0.0076 -0.0000 -0.0000
15.1856 16.1213 0.0004 -0.0014 6.9286 21.0356
16.1213 -15.1856 0.0013 0.0024 21.0356 -6.9286
15.1856 16.1213 0.0004 -0.0014 -6.9286 -21.0356
16.1213 -15.1856 0.0013 0.0024 -21.0356 6.9286
EIGEN VALUES
751367.417 633730.458 24.312 131.039 717450.732 490964.833
First Mode Period: 0.258 [s]
Second Mode Period: 0.0076 [s]
Case 4)
EIGEN VECTORS
-0.0000 -0.0000 -0.0000 -0.0000 -0.0007 0.0075 -0.0017
-0.0000 0.0000 -0.0000 -0.0000 0.0002 0.0019 0.0082
-0.0000 0.0000 0.0000 0.0000 0.0208 0.0018 -0.0009
4.8021 5.0980 2.1910 -6.6520 0.0013 0.0004 -0.0021
4.8021 5.0980 -2.1910 6.6520 -0.0049 0.0004 0.0006
5.0980 -4.8021 6.6520 2.1910 -0.0002 0.0013 0.0027
5.0980 -4.8021 -6.6520 -2.1910 0.0072 0.0013 -0.0006
EIGEN VALUES
237603.247 200403.198 226877.861 155256.744 236.971 24.299 97.495
First Mode Period: 0.259 [s]
Second Mode Period: 0.01 [s]

3. Linear Static Method

see: EN1998-1 (4.3.3.2)

Base shear force: $F_b = S_d\left(T_1\right) \cdot m \cdot \lambda$

Forces acting on each lump: $F_i = F_b \cdot \dfrac{z_i \cdot m_i}{\sum z_j \cdot m_j}$

$S_d(T_1)$ is the ordinate of design spectrum
m is the total mass of the structure
λ is correction factor
For $T_1 < 2T_c$, storey less than two, λ=1.0
F_i is the horizontal force acting on storey i
z_i, z_j are the heights of the masses m_i, m_j above the level of application of the seismic action
m_i, m_j are mass lumps as calculated before

For different Seismic Zone(1~5) and different Ground Type(A~E), we have different value of $S_d(T_1)$ and different F_a values. From Zone1A to Zone5E, there are 25 cases in total.

4. EN8 Formula for Non-structural Panels

Formula for lateral force acting on non-structural panel specified by EN1998-1 (4.3.5.2)

$$F_a = \frac{S_a \cdot W_a}{q_a}$$

$$S_a = \alpha \cdot S \cdot \left[\frac{3 \cdot \left(1 + Z/H\right)}{1 + \left(1 - T_a/T_1\right)^2} - 0.5 \right]$$

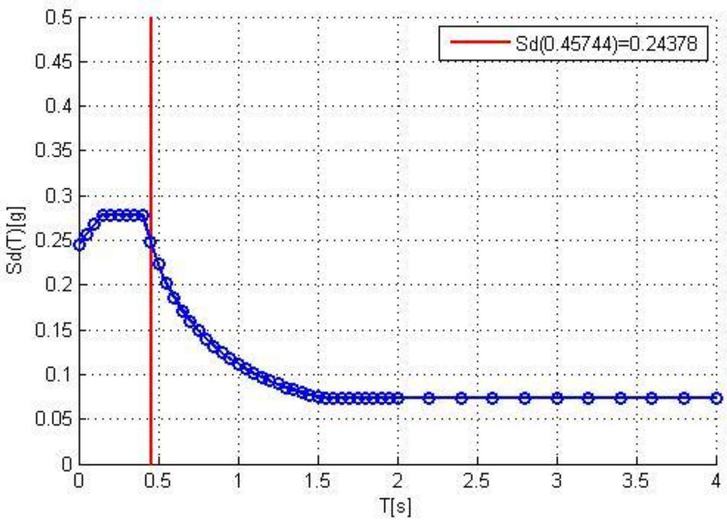
F_a is the horizontal seismic force, acting at the centre of mass of the non-structural element in the
most unfavorable direction
T_a the fundamental vibration period of non-structural element
T_1 the fundamental vibration period of the whole structure
W_a self-weight of the panel
S_a seismic coefficient apply on non-structural element
S Coefficient of soil type
q_a behavior factor of non-structural element taken into account the non-linear ductile behavior
α ratio of design ground acceleration and acceleration of gravity =Ag
Z distance from center of the panel mass to the ground

5. Results of LS Analysis

F_a and F_a/W_a are functions of (T_a/T_1) and Soil Type S. Notice that zone1~5 will affect the coefficient α, which coincides with the value a_g. Later we will compare the results obtained by the 2 methods in the charts. Take Zone1A For example:

Case 1)

$T_1 = 0.457(s)$, $a_g = 0.368(g)$, Ground Type A, Response spectrum Type I $S_d = 0.244(g)$.

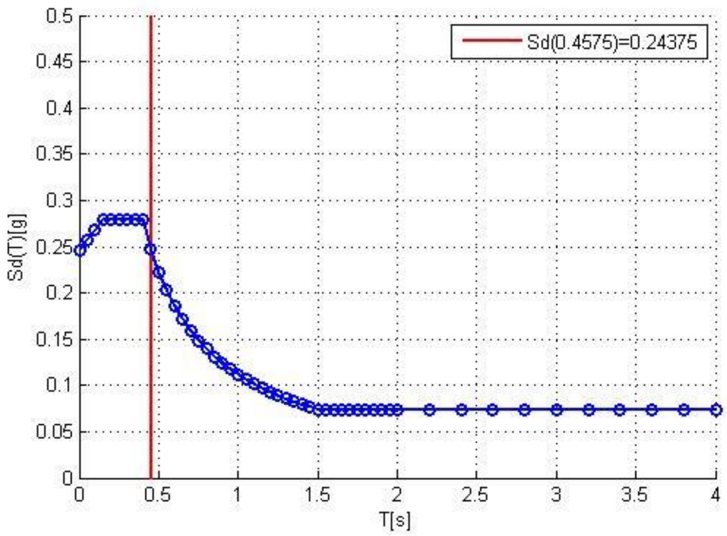


Linear Static Method: Fa=13.987 [kN]; Fa/Wa=0.121

EN8 Non-structural: Fa=74.189 [kN]; Fa/Wa=0.644

Case 2)

$T_1 = 0.458(s)$, $a_g = 0.368(g)$, Ground Type A, Response spectrum Type I $S_d = 0.244(g)$.

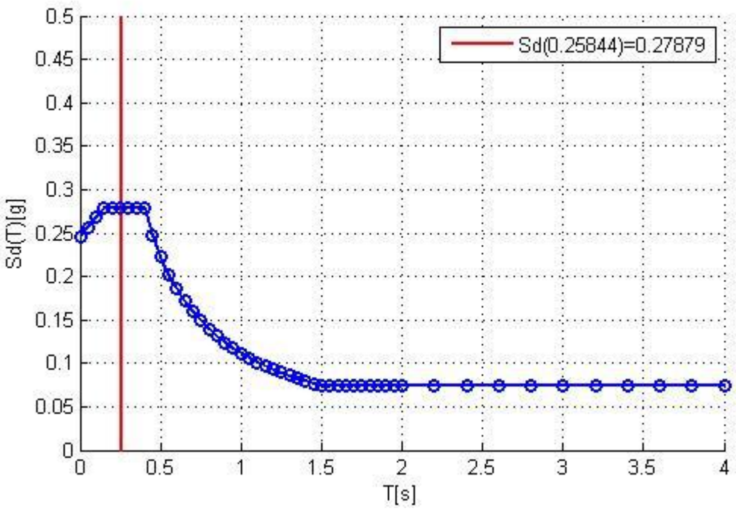


Linear Static Method: Fa=13.985 [kN]; Fa/Wa=0.121

EN8 Non-structural: Fa=74.189 [kN]; Fa/Wa=0.644

Case 3)

$T_1 = 0.258(s)$, $a_g = 0.368(g)$, Ground Type A, Response spectrum Type I $S_d = 0.279(g)$.

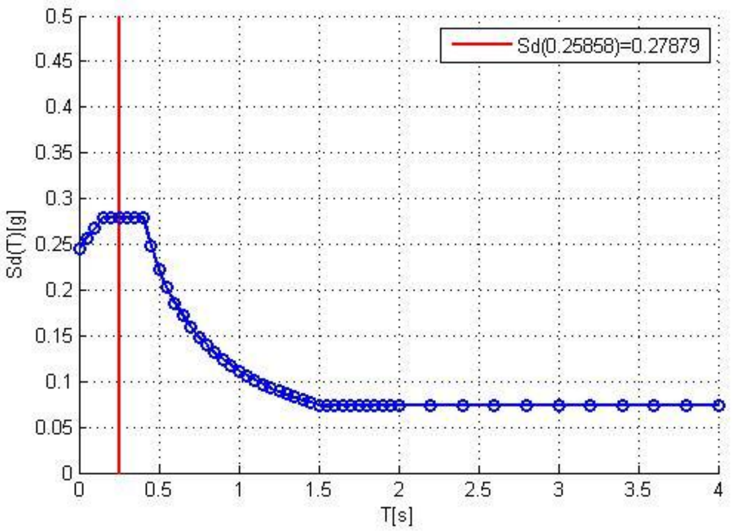


Linear Static Method:
Fa=15.995 [kN]; Fa/Wa=0.139

EN8 Non-structural:
Fa=74.189 [kN]; Fa/Wa=0.644

Case 4)

$T_1 = 0.259(s)$, $a_g = 0.368(g)$, Ground Type A, Response spectrum Type I $S_d = 0.279(g)$.



Linear Static Method:
Fa=15.995 [kN]; Fa/Wa=0.139

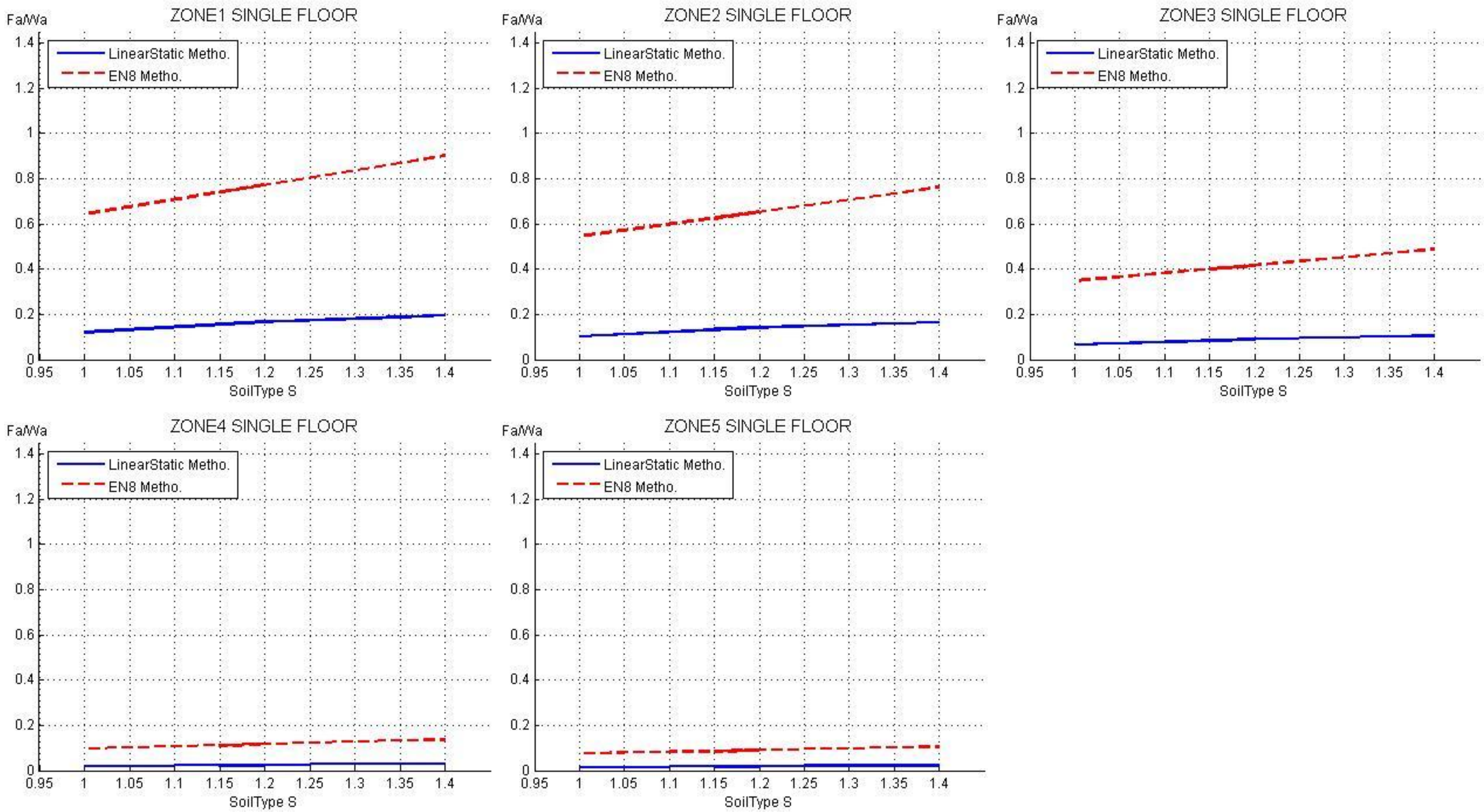
EN8 Non-structural:
Fa=74.189 [kN]; Fa/Wa=0.644

Note that Fa is the seismic force acting on non-structural element. In the analysis we obtain the force F2 acting on the panel height level which value is corresponding to the mass M2.
By proportion, we get:

$$F_a = F_2 \times \frac{M_a}{M_2} = F_2 \times \frac{W_a}{W_2}$$

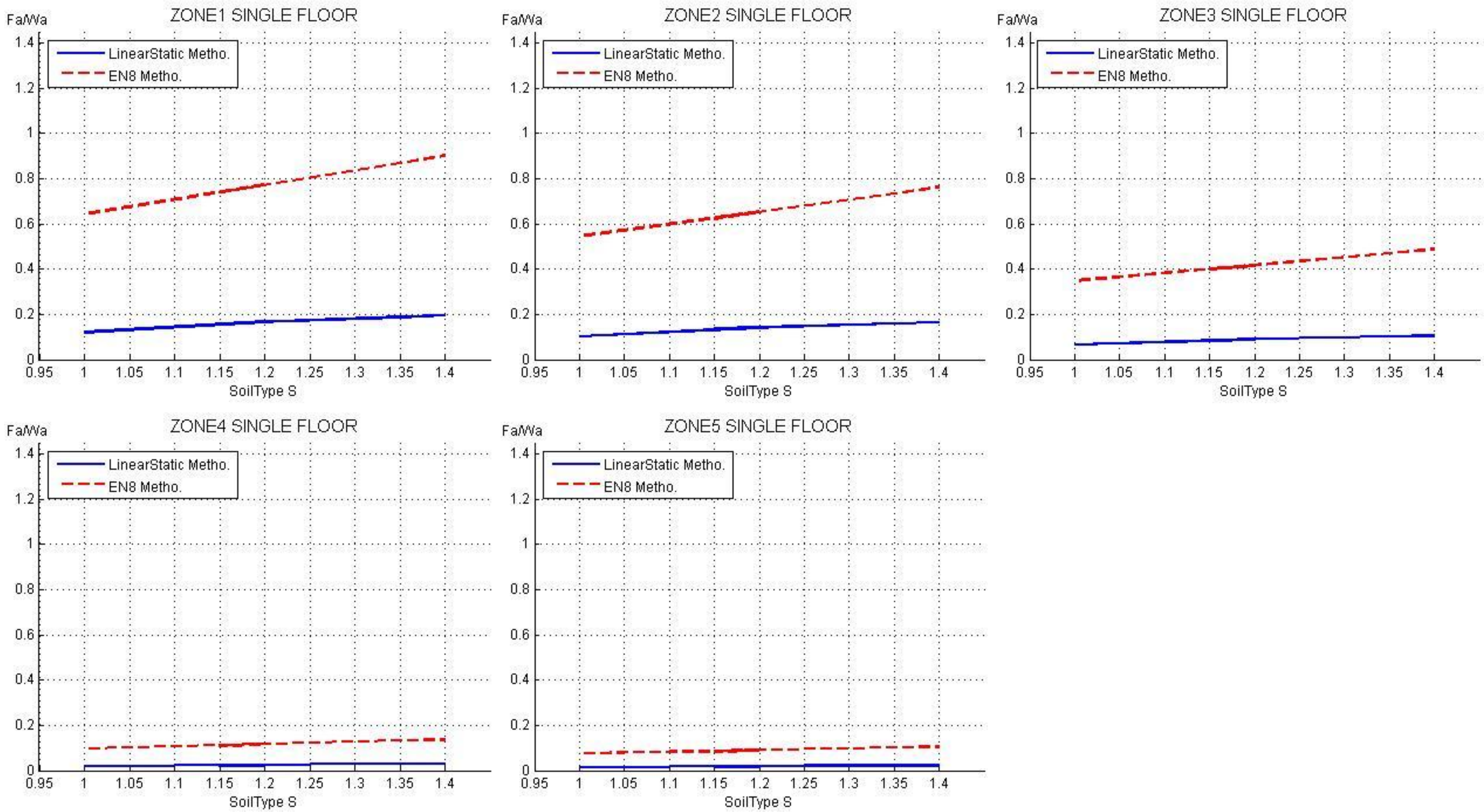
Comparison of Linear Static Method and EN8 empirical formula----Case 1)

F_a/W_a varied with SoilType S for 5 seismic zones (notes: plots drawn by Matlab)



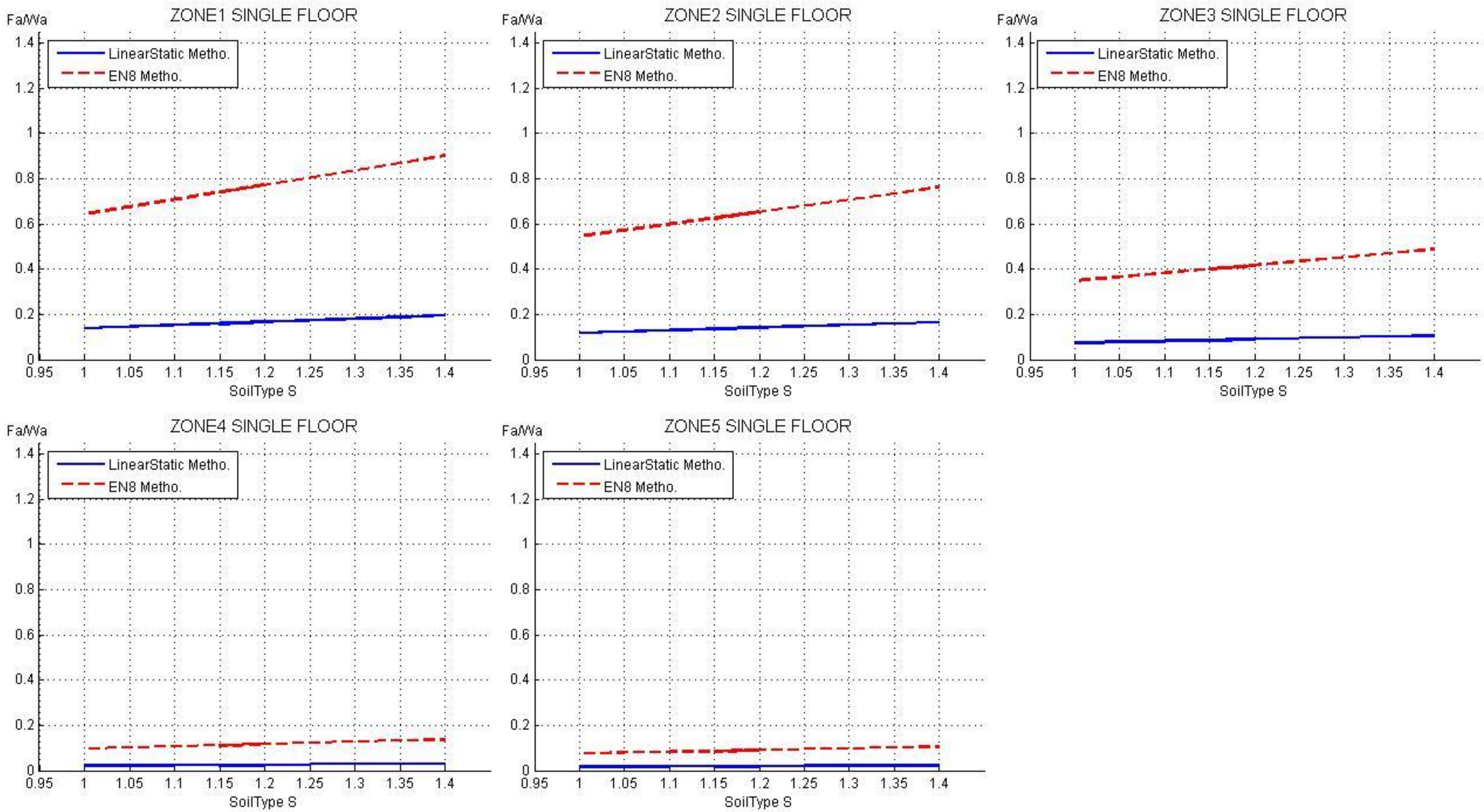
Comparison of Linear Static Method and EN8 empirical formula----Case 2)

F_a/W_a varied with SoilType S for 5 seismic zones (notes: plots drawn by Matlab)



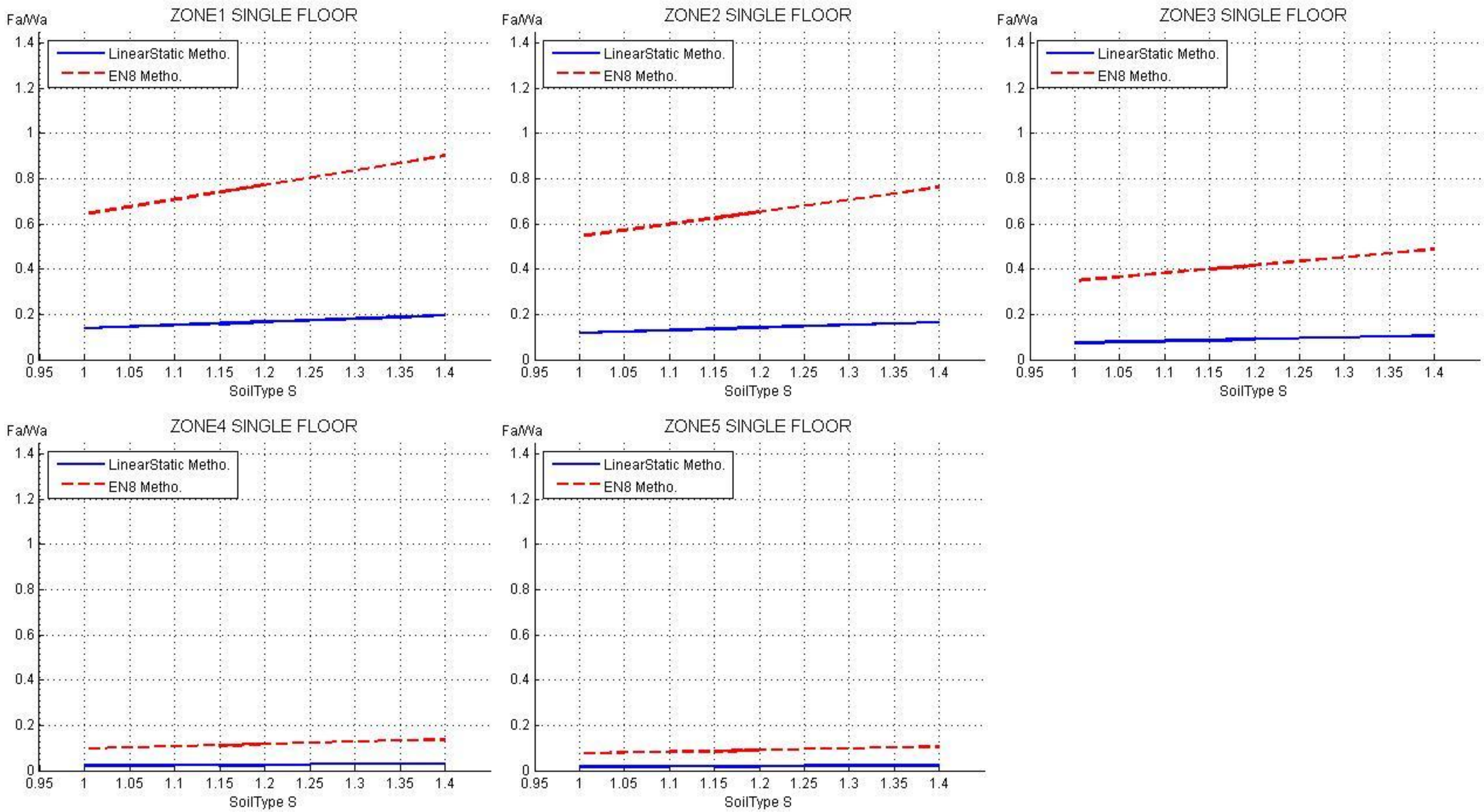
Comparison of Linear Static Method and EN8 empirical formula----Case 3)

F_a/W_a varied with SoilType S for 5 seismic zones (notes: plots drawn by Matlab)



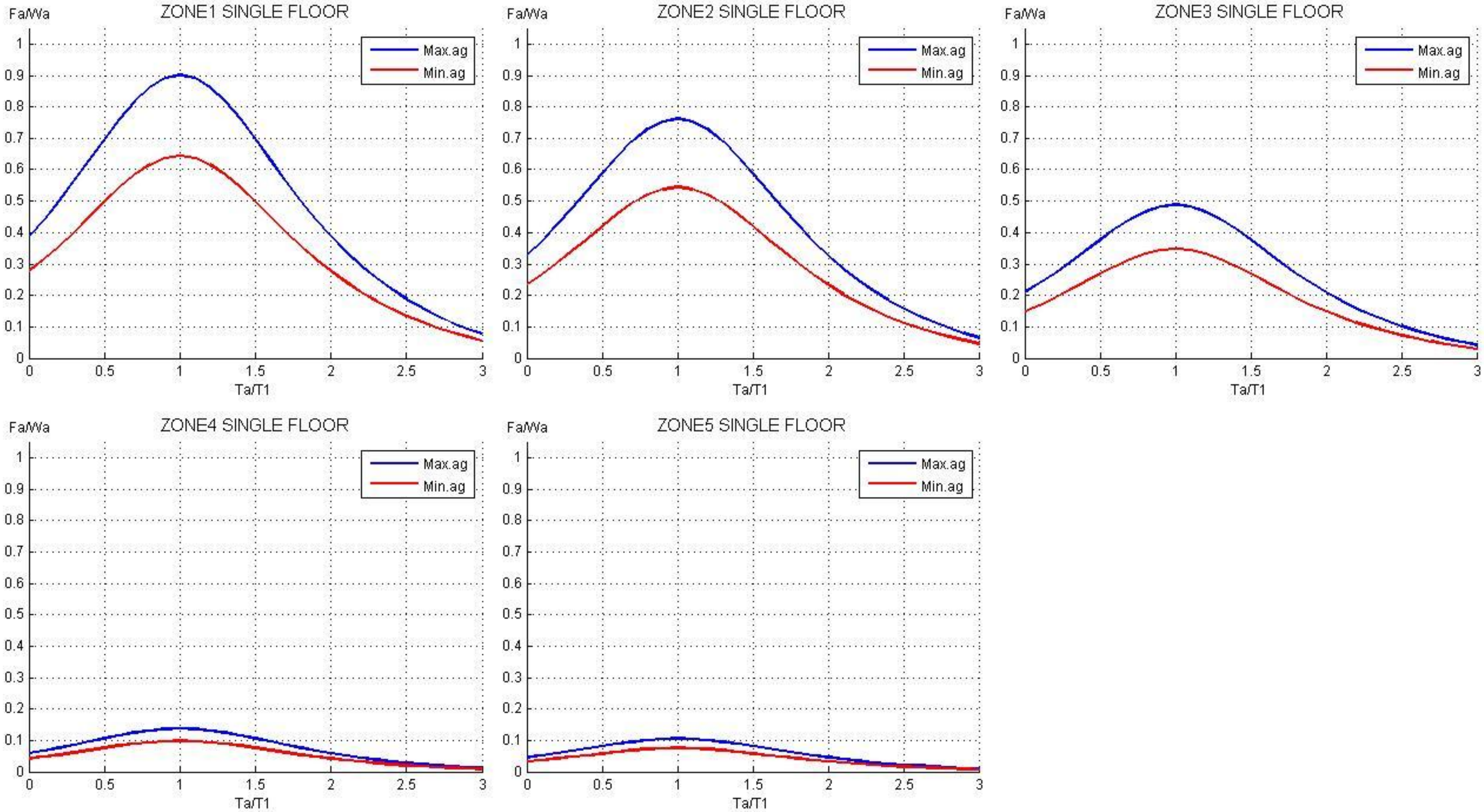
Comparison of Linear Static Method and EN8 empirical formula----Case 4)

F_a/W_a varied with SoilType S for 5 seismic zones (notes: plots drawn by Matlab)



Value of F_a/W_a varied with Vibration Period of Non-structural Panel

F_a/W_a for Max.ag and Min.ag according to different soil types S and for 5 seismic zones (notes: all possible values lie between two curves)



--Conclusions--

- the relative lateral seismic force F_a/W_a is approximately in linear relationship with Soil coefficient S. The larger the value S the larger the force (in ratio form).

- Increasing the index of zones from zone1 to zone5, the value declines as well as the slope of the interpolating lines.

- The value obtained by EN8 formula for non-structural part is much larger than LS Method in zones with the largest ground acceleration a_g, while the difference is smaller in small ground acceleration case.

- The result obtained by (F_a/W_a)~S plot may indicate an over-estimation of seismic force with EN8 formula for non-structural element. It will be checked with FEA later.

- From the second series of comparison graphs, we can see that assuming $T_a/T_1 = 1.0$ is on the safe side, since the seismic force is in parabolic relations with T_a/T_1, and the peak value corresponding to $T_a/T_1 = 1.0$ (that means, from physic point of view, the free vibration of panel is at the same pace with vibration of the overall structure).

- Since the real relation between T_a and T_1 is difficult to be accurately predicted, we should, as solving for other general engineering problems, assuming a critical value of $T_a/T_1 = 1.0$.

III. Linear Dynamic Analysis by Diana

Use Dynamic analysis---earthquake spectral analysis with the given response spectrum.
Now list below some important steps, take Case1 for example (beam-columns pin connected, panel-columns also pin connected.)

1. Finite Element Modal

--Geometry--
Create the finite element model in the Design main window of iDiana (the pre-processor). Start the interactive session end enter a name for the new model which is `NONSTRUCTURAL-PANEL'.

In the "Analysis and Units" dialog box, indicate that this is a model for structural analysis in a 2D situation. Also accept the default units [m, kg, s, N, celsiu]. It can also use "structural plain strain". However, since we have to define details element section like "PI beam" and this option only available when choosing "Structural 2D", thus a 2D model will be defined. In this case the model is 2D structural plain stress.

```
FEMGEN NONSTRUCTURAL-PANEL
Analysis and unit: (M, kg, s, N, C)
Analysis Selection
Model Type: !Structural 2D
```

To define the geometry of the model, create points at the supported feet of the column and at the column-beam, column-panel junctions. The meshing in next step will sub-divide the element into much smaller segments and will create more points consequently. We may define only some major and critical points here to sketch the profile. Then connect points with lines.

```
GEOMETRY POINT 0.0 0.0
GEOMETRY POINT 0.0 2.4
……
GEOMETRY LINE P1 P2
GEOMETRY LINE P2 P3
……
```

Create a surface to represent the panel by defining 4 corner points (0,2.4,0), (0,4.8,0), (12,4.8,0), (12,2.4,0), which assigned to P11, P12, P13, P14 respectively. The name of the surface is "PANEL".

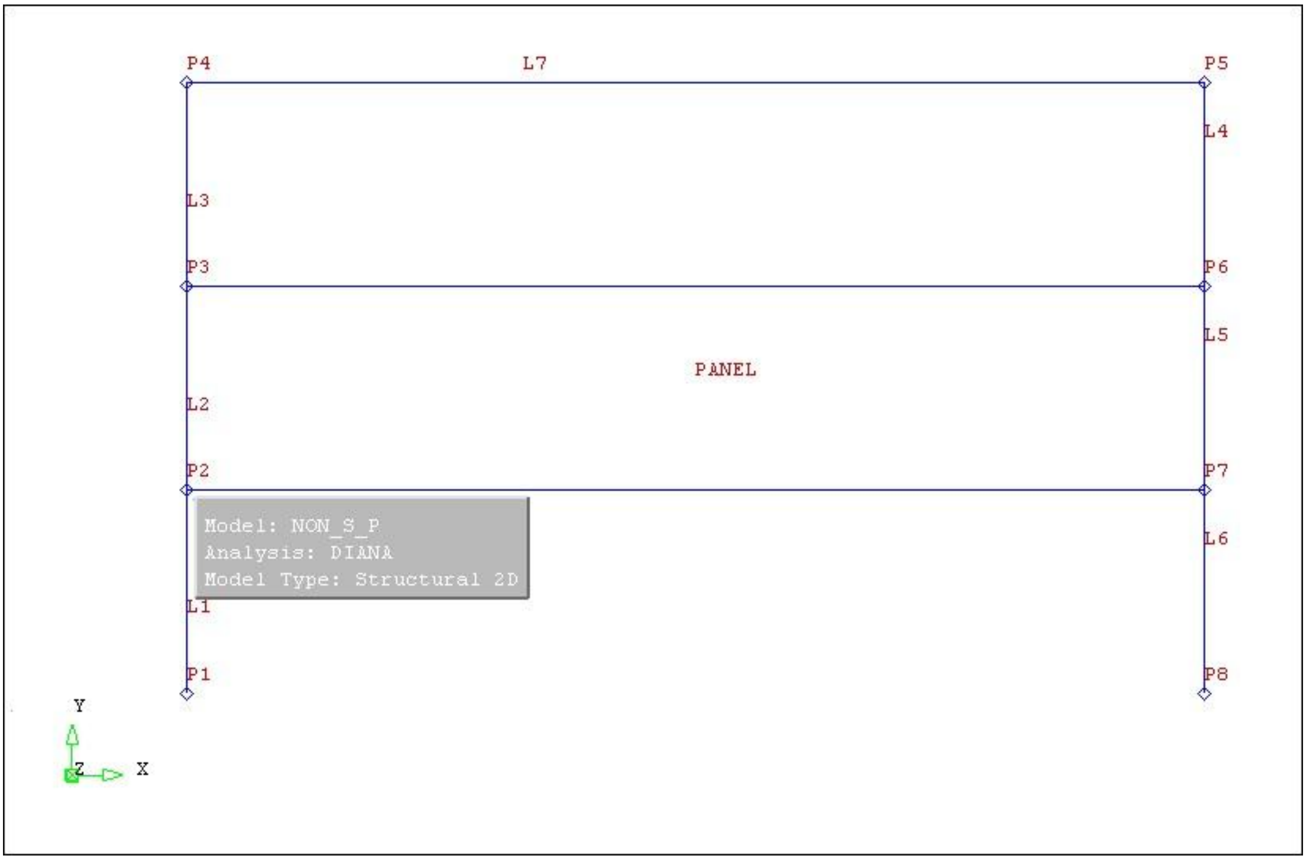
```
GEOMETRY SURFACE 4POINTS PANEL P11 P12 P13 P14
```

Construct sets: Define 3 sets here (SET1~SET3) each represents the panel, the column and the beam. Each set includes series of points, lines, and surface. Notice that the coordinate of (P2 and P11), (P3 and P12), (P6 and P13), (P7 and P14) are the same, but P2~P7 belongs to column set(SET2) while P9~P12 belongs to panel(SET1). Similarly the coordinate of (P4 and P9), (P5 and P10) are the same, but P4,P5 belongs to column while P9, P10 are points of beam. Interaction between them will be defined latter in "TYING" module.

```
CONSTRUCT SET OPEN SET1
CONSTRUCT SET APPEND SURFACE PANEL
CONSTRUCT SET CLOSE
CONSTRUCT SET OPEN SET2
CONSTRUCT SET APPEND L1 L2 L3 L4 L5 L6
CONSTRUCT SET CLOSE
CONSTRUCT SET OPEN SET3
CONSTRUCT SET APPEND L7
CONSTRUCT SET CLOSE
```

To check the assignment of element types, loading, boundary constraints, material and physical properties we may use the following LABEL and VIEW commands.

```
VIEW GEOMETRY SET1 VIOLET
LABEL GEOMETRY LINES SET1 RED
LABEL GEOMETRY POINTS
EYE FRAME
```



--Mesh--

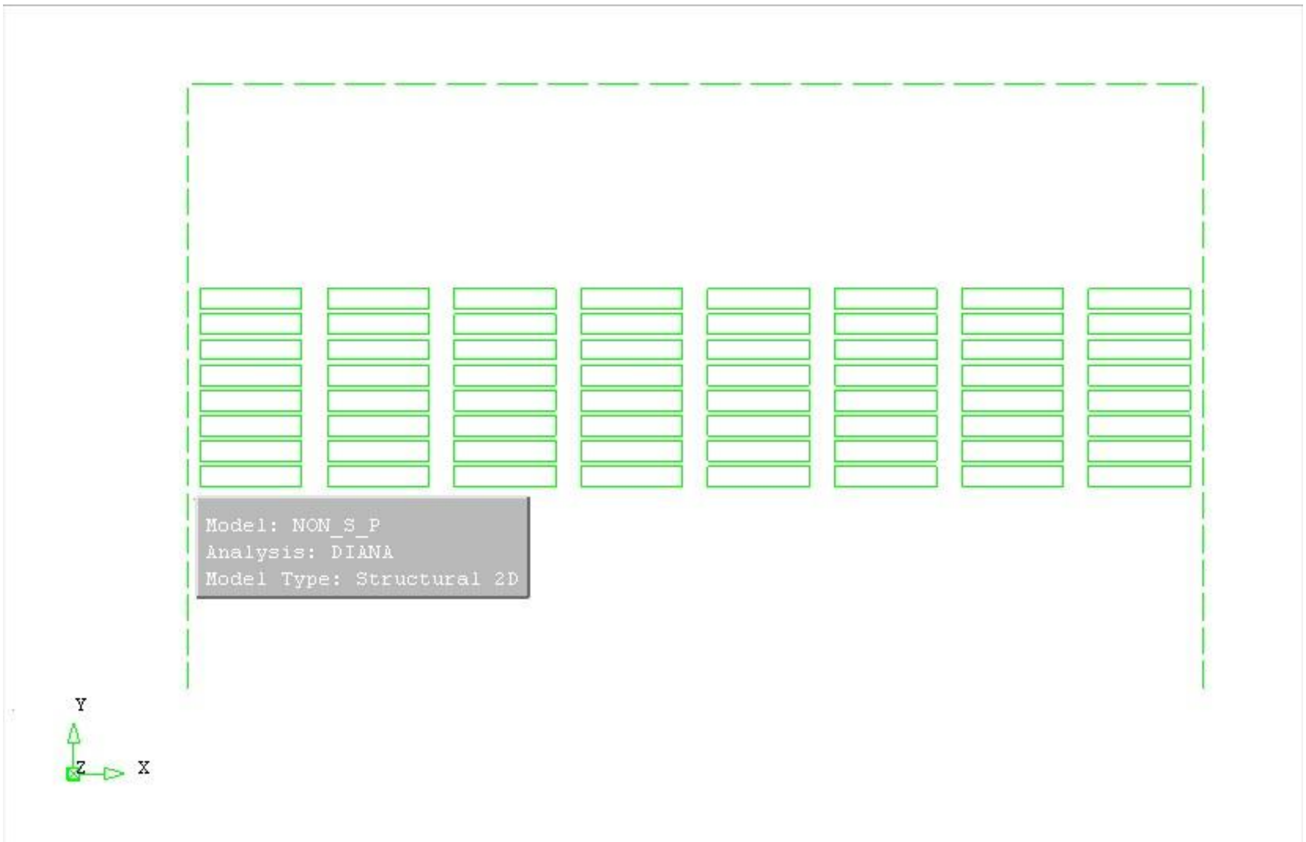
Use MESHING commands to create the finite element mesh. First assign the L6BEN element type to all elements in beam and columns then mesh via the TYPES option. It's the typical beam element type. Then specify the approximate size of the elements via the DIVISION ELSIZE option. Finally the mesh is generated via the GENERATE option. The VIEW and LABEL commands cause the mesh to be plotted including labels for the node and element numbers. Note that we apply the SHRINK option to get a "shrunken elements" view, otherwise it would be impossible to distinguish the individual line elements.

Here I divide each line of column set into 6 and top beam into 20 segments, while divide panel into 8×8 pieces.

"L6BEN" is Bending beam, 2 nodes, 2-D, Timoshenko or Bernoulli.

"Q8MEM" is Quadrilateral plane stress, 4 nodes, linear.

```
MESHING TYPES SET2 BE2 L6BEN
MESHING TYPES SET3 BE2 L6BEN
MESHING TYPES SET1 Q8MEM
MESHING DIVISION LINE SET2 6
MESHING DIVISION LINE SET3 20
MESHING DIVISION SURFACE SET1 8
MESHING GENERATE
VIEW MESH
VIEW OPTIONS SHRINK
```



--Material and Physical Properties--

Define the material and physical properties in the Property Manager dialog box. The data is in accordance with previous chapter "Element Type and Cross-sections".

View →Property Manager...
→Property Manager
Materials Material Name: PLATE
→Linear Elasticity →Isotropic
YOUNG 3.820000E+10
POISON 2.000000E-01
Mass →Mass Density DENSIT 1.631000E+03
Physical Properties →Physical Property Name: THICK
Interface →Plane Stress→ Thickness THICK 2.500000E-01

Materials Material Name: MCOLUMN
→Linear Elasticity →Isotropic
YOUNG 3.820000E+10
POISON 2.000000E-01
Mass →Mass Density DENSIT 2.548400E+03
Physical Properties →Physical Property Name: REC
Geometry →Beam →Class-I →Predefined shapes
RECTAN (5.000000E-01 5.000000E-01)

Materials Material Name: MBEAM
→Linear Elasticity →Isotropic
YOUNG 3.820000E+10
POISON 2.000000E-01
Mass →Mass Density DENSIT 3.376122E+03
Physical Properties →Physical Property Name: PIBEAM
Geometry →Beam →Class-I →Arbitrary shapes
CROSSE 3.340000E-01
INERTI 1.6840000E-02

Also take into account the roof load on beam by adding it to the density of the beam, as a result, DENSIT increase from original value 2.548400E+03 to 3.376122E+03. The "PI beam" is not predefined, but fortunately we can create it by setting Cross-section Area and Moment of Inertial about its principle axis. Now assign these material and physical properties to three sets.

PROPERTY ATTACH SET1 PLATE THICK
PROPERTY ATTACH SET2 MCOLUMN REC
PROPERTY ATTACH SET3 MBEAM PIBEAM

--Boundary Condition and Joints Connections--

Define connections at the base supports. Create a new set only consist of two base points P1 and P8, the name is "FEET". Then set it fixed to the ground. ENCASTRE command indicate a fixed B.C.

CONSTRUCT SET OPEN FEET
CONSTRUCT SET APPEND POINTS P1 P8

```
CONSTRUCT SET CLOSE
PROPERTY BOUNDARY CONSTRAINT FEET ENCASTRE
```

The difficulty lies in define 'TYING' for beam-column joints & panel-column joints. Since there are multiple(two) points coincide at the same coordinate, define a multi-points constrains for the 2 joints of beam-column and 4 corner joints of the panel plate.

For the beam-column joints, they are pin connected in Case 1), so the multi-points constrains is in translational DOF in X,Y while rotational DOF is set free. Similarly, the two lower joints of the panel are fixed in translation in X,Y direction and free in rotational DOF. The top two joints only fixed in Y direction, means free to move horizontally, however displacement should be check for the allowance 25mm. The command "Rigid Connection" may be used here.

```
PROPERTY BOUNDARY MPC RCONNECT P4 P9 X Y
PROPERTY BOUNDARY MPC RCONNECT P5 P10 X Y
PROPERTY BOUNDARY MPC RCONNECT P2 P11 X Y
PROPERTY BOUNDARY MPC RCONNECT P3 P12 Y
PROPERTY BOUNDARY MPC RCONNECT P6 P13 Y
PROPERTY BOUNDARY MPC RCONNECT P7 P14 X Y
```

--Loads--

Only ground excitation and gravity load in consider, the magnitude is 3.61008 (0.368×g) for Zone1 excitation. Notice that loading for different cases can be set latter in Spectra Definition. We may set g=9.81 here.

```
PROPERTY LOADS BASE FEET 9.81 X
PROPERTY LOADS GRAVITY ALL -9.81 Y
```

Now finish building the model, create input file for Diana.

```
UTILITY WRITE DIANA
yes
FILE CLOSE
yes
```

The principle directions of the model defined by the following matrix:

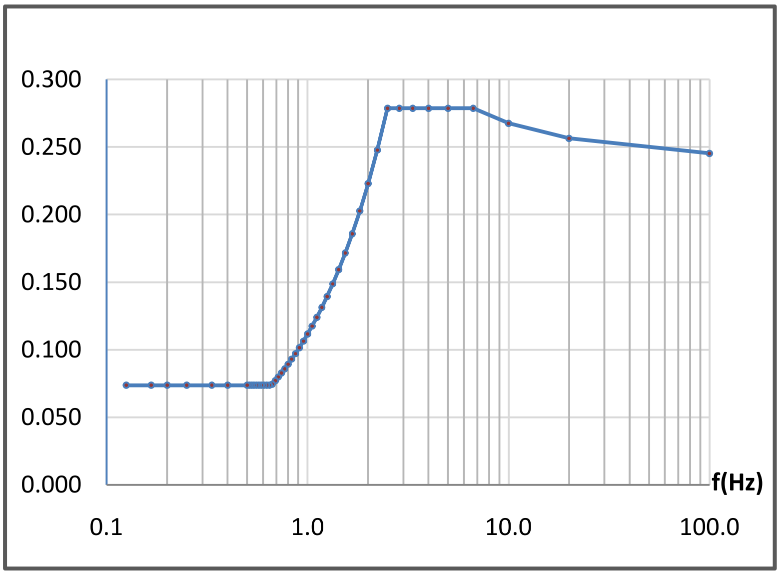
```
'DIRECTIONS'
     1    1.000000E+00    0.000000E+00    0.000000E+00
     2    0.000000E+00    1.000000E+00    0.000000E+00
     3    0.000000E+00    0.000000E+00    1.000000E+00
```

-See complete input file (*.dat) in Appendix B

2. Earthquake Spectral Analysis

--Frequency-based Response Spectrum--

The Design response spectrum in EuroCode 8 is given in amplification factor S_d versus period T [s], now here the spectrum is defined in S_d versus frequency f [Hz].
For example (Max Zone1 GroundType A):



Create input file (*.dat)
The format is as follows:

```
'FREQLO'
LOAD 1
FREQUE 0.100 0.125 0.167 0.200… \
FACTOR 0.009 0.009 0.009 0.009… \
'END'
```

The input acceleration is the base excitation 9.81 X which defined in “load 1 × factors”. Consider as many as possible nodes as in EuroCode Seismic Response spectrum Curve(See Appendix A). This input file will later be used in spectral analysis command file.

--Writing the Command Files--

The command file (*.com) consists of two part: one for modal(eigen) analysis via “*EIGEN” modulus, and one for spectral analysis via “*SPECTR” modulus.

Firstly, there are several modes in consider. Higher modes are not dominating the behaviour, however notice that mode 8, 10 have high mass participation in Y direction so they should also be included. Then read file "freqlo_zone1_A.dat" which we create before for spectrum input. Furthermore, ask for output of the maximum force amplitudes f_{max} for eigen modes 1~10 individually, and for the superposed maximum forces f_s according to the ABS and the SRSS rule. This leads to the following batch commands.

```
------------------------------------------------------------------------------------------------------------------------
*EIGEN
TYPE FREEVI MASS CONSIS
BEGIN EXECUT
  NMODES=10
END EXECUT
BEGIN OUTPUT TABULAR
  DISPLA
END OUTPUT
*END
------------------------------------------------------------------------------------------------------------------------
------------------------------------------------------------------------------------------------------------------------
*INPUT
READ FILE="freqlo_zone1_A.dat" TABLE FREQLO
*SPECTR
BEGIN MODEL
  EVALUA
  ASSEMB
END MODEL
EIGEN OFF
BEGIN RESPON
BEGIN OUTPUT FEMVIEW BINARY
SELECT MODES 1-10
DISPLA
FORCE RESIDU
END OUTPUT
BEGIN OUTPUT FEMVIEW BINARY
SELECT MODES 1-10 ABS SRSS
DISPLA
FORCE RESIDU
END OUTPUT
BEGIN OUTPUT    TABULAR
SELECT MODES 1-5 8 10 ABS SRSS
DISPLA
FORCE RESIDU
END OUTPUT
BEGIN OUTPUT    TABULAR
BEGIN SELECT
NODES 1-59
MODES 1-5 8 10 ABS SRSS
END SELECT
STRESS FORCE NODES
STRESS MOMENT NODES
END OUTPUT
BEGIN OUTPUT    FEMVIEW BINARY
BEGIN SELECT
NODES 1-59
MODES 1-10 ABS SRSS
END SELECT
STRESS FORCE NODES
STRESS MOMENT NODES
END OUTPUT
END RESPON
*END
------------------------------------------------------------------------------------------------------------------------
```

The linear static analysis is also run for gravity load so as to combine the result with seismic load case. The combination rule is due to seismic combination defined in EN1990. The two possible combinations are:
Left Seismic + Gravity
Right Seismic + Gravity

The command modulus for linear static analysis is as below:

```
--------------------------------------------------------------------------------
*INPUT
*LINSTA
BEGIN OUTPUT TABULA
  DISPLA
  STRESS FORCE
  STRESS MOMENT
END OUTPUT
*END
--------------------------------------------------------------------------------
```

--Reading the Output Files--
As specified in the above command file, the output files have two forms:
FEMVIEW BINARY: Graphical, interaction with iDiana
TABULAR: In the form of table which can be opened in “txt” format
Selected output are:
-Maximum Nodal displacement “DISPLA”
-Residual forces vector “FORCERESIDU”
-Nodal forces in local coordinate X & Y direction “STRESS FORCE NODES”
-Nodal Moment about principle axis “STRESS MOMENT NODES”
And there are two rules:
ABS:
The absolute maximum of the forces will be the sum of the maximum forces of all contributing modes separately. This absolute maximum is calculated by the ABS rule. In general it is not likely that all modes reach their maximum at the same moment.

$$E = \sum_{i=1\sim10} \max\{E_i\}$$

SRSS:
Rule which takes the Square Root of the Summed Squares, more realistic than ABS rule.

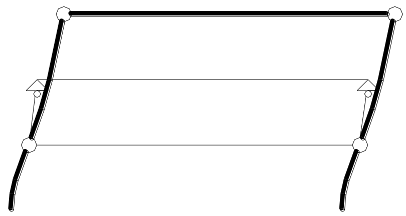
$$E = sqrt\left\{\sum_{i=1\sim10} E_i^2\right\}$$

Note that E represents the effect of earthquake response.
Results are saved in .tb & .out & .M70 files.

3. Results of Linear Dynamic Analysis

(Also compare the result obtain by previous non-numerical method)

Case 1)------ 4 DOF



--Eigen Analysis--

TOTAL MASS [kg]
0.344E+05
EIGEN-FREQUENCIES: (Hz)
0.22517D+01(1) 0.81845D+01(2) 0.17370D+02(3) 0.32067D+02(4)
0.32748D+02(5) 0.48872D+02(6) 0.55399D+02(7) 0.69194D+02(8)
0.94281D+02(9) 0.11879D+03(10)

MODE	FREQUENCY[Hz]	PERIOD[s]	MASS.P.X[%]	MASS.P.Y[%]
1	2.25E+00	0.4441	64.076	0.000
2	8.18E+00	0.1222	0.000	32.485
3	1.74E+01	0.0576	30.055	0.000
4	3.21E+01	0.0312	0.000	0.000
5	3.27E+01	0.0305	0.000	26.291
6	4.89E+01	0.0205	0.000	5.793
7	5.54E+01	0.0181	1.797	0.000
8	6.92E+01	0.0145	0.000	7.163
9	9.43E+01	0.0106	0.018	0.000
10	1.19E+02	0.0084	0.000	21.540

Comparison:

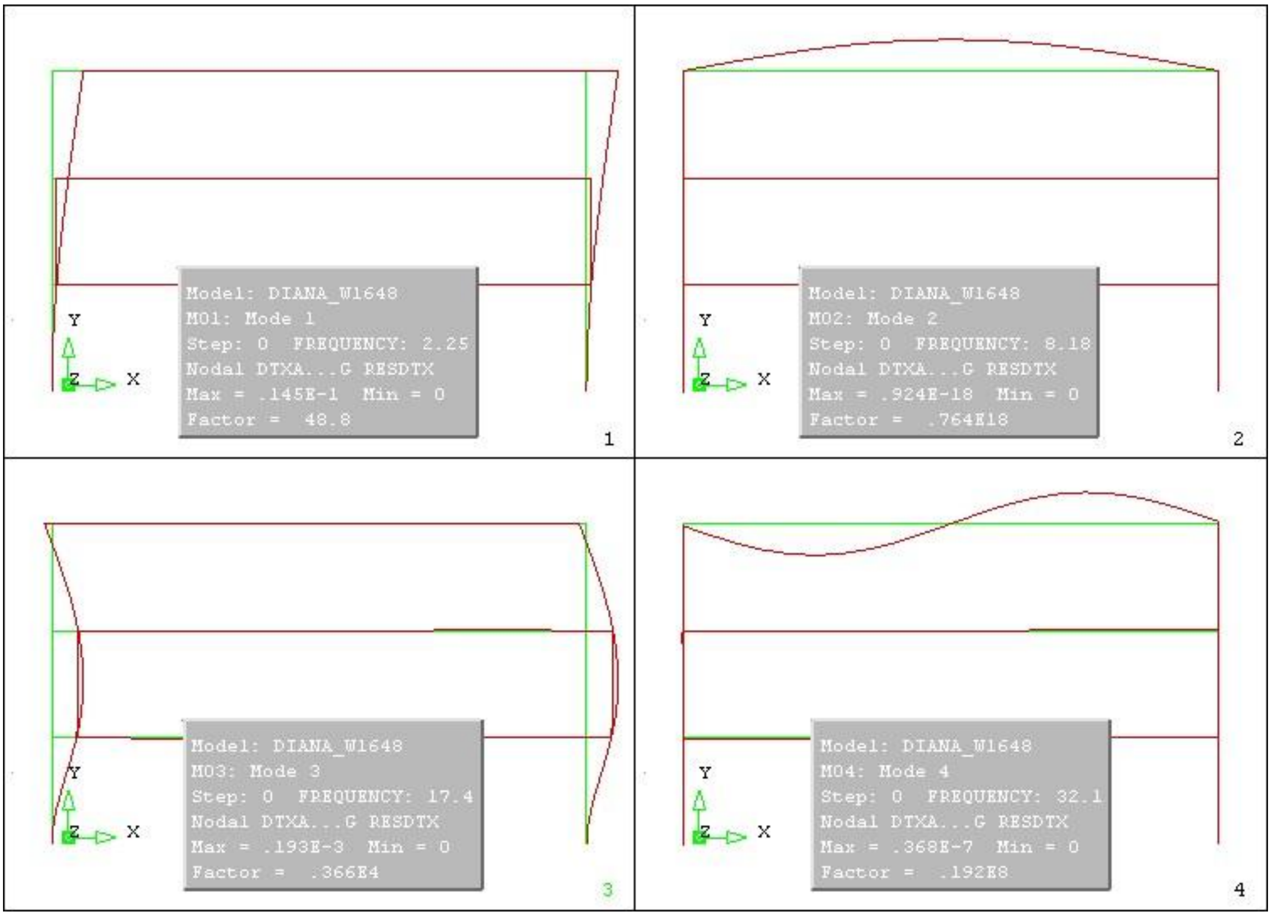
The total mass calculated by Diana is the same as calculated by LSM before, both equal to 0.344E+05 (kg) ,The first Mode Period computed by LSM in previous chapter is: 0.457s, which value is very close to the FEA result: 0.444s.

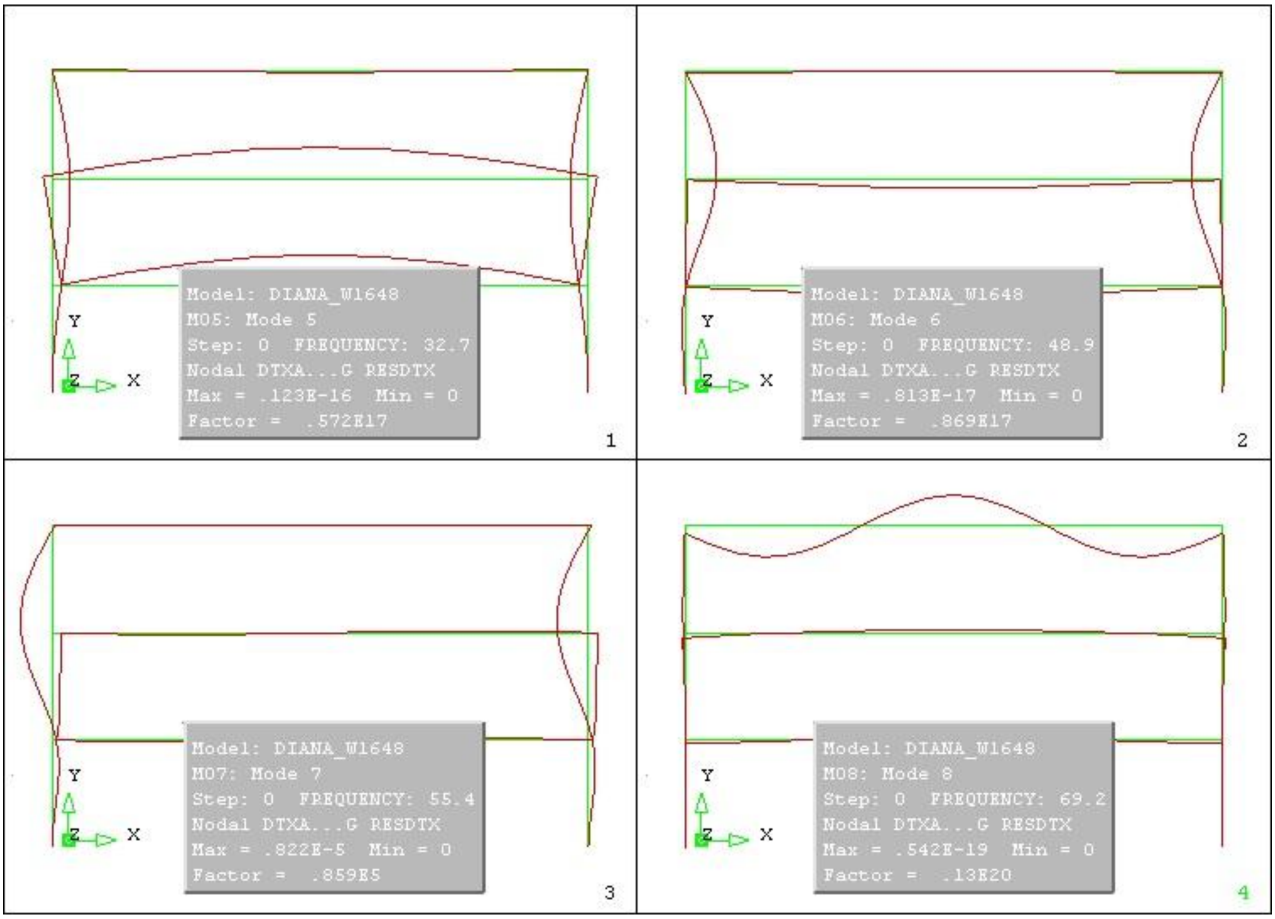
In linear static method, we consider only first mode, however in FEM, we have many mass lumps, and take the first 10th eigen modes in regards.

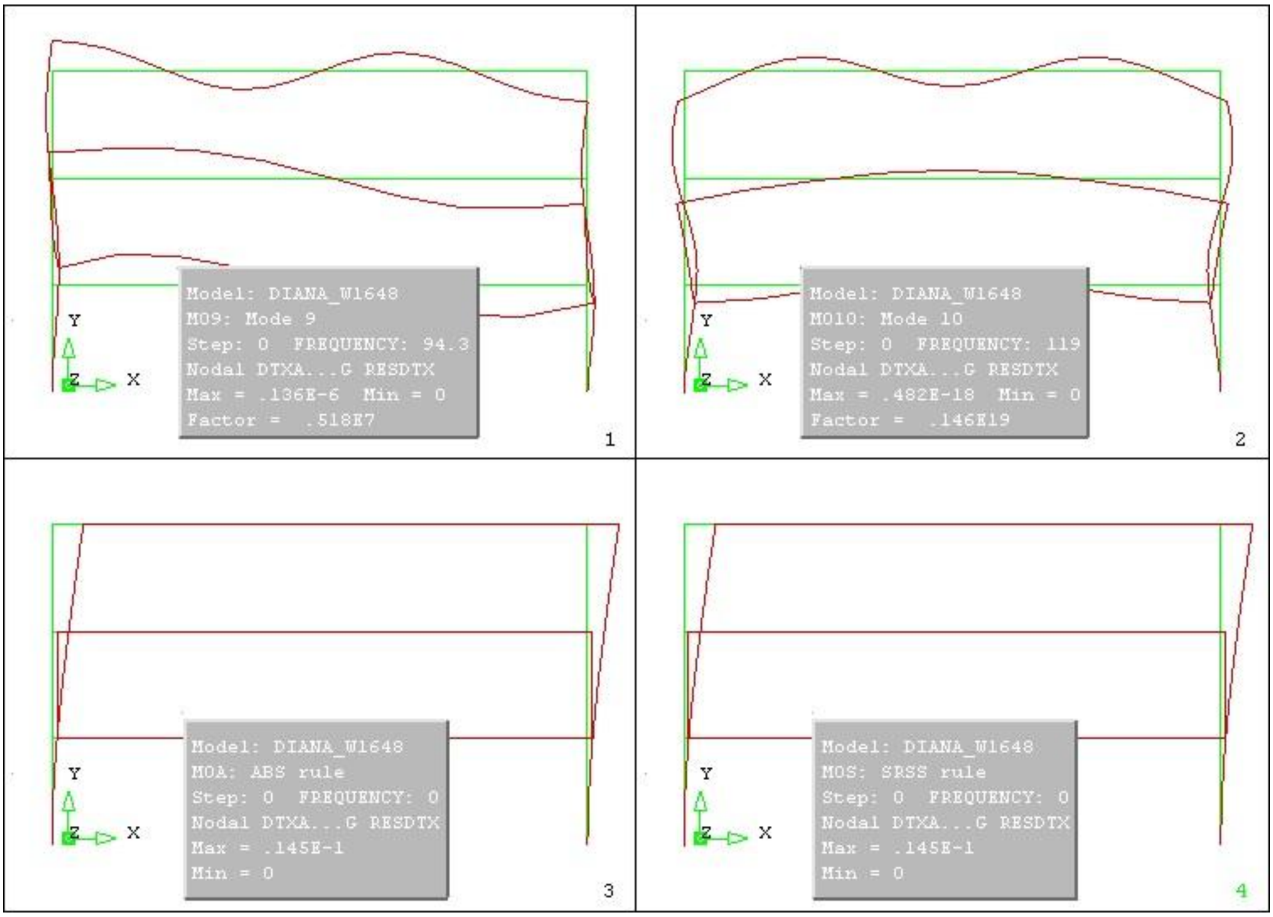
--Deform Shapes for Different Modes--

Open .M70 file in iDiana. (Interaction with iDiana)
Plot deformed shapes for each 10 modes and combination shape according to the two rules.

```
FEMVIEW SPECTR
RESULTS LOADCASE MO1
RESULTS NODAL DTXA...G RESDTX
PRESENT SHAPE
RESULTS LOADCASE MO2
PRESENT SHAPE
……
RESULTS LOADCASE MOA
PRESENT SHAPE
RESULTS LOADCASE MOS
PRESENT SHAPE
```

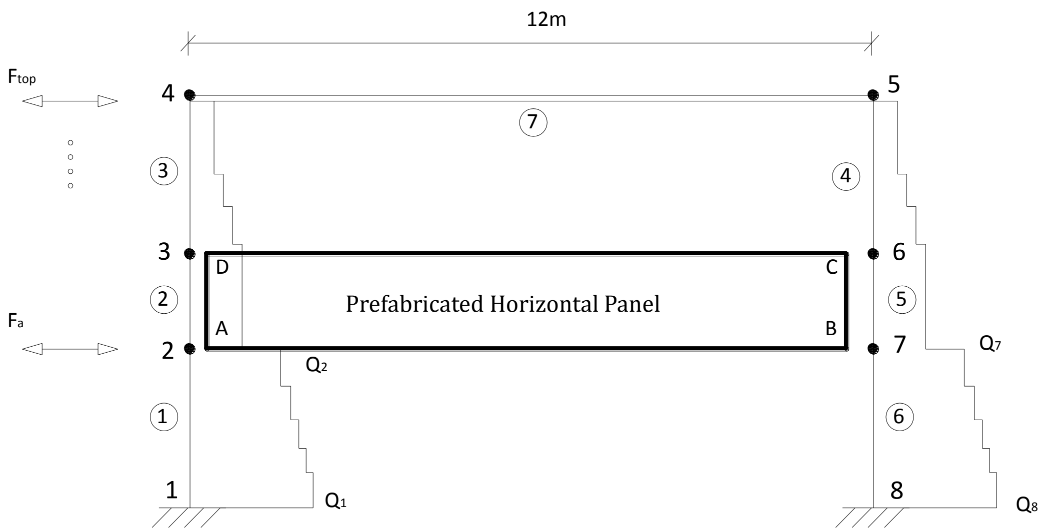

Model: DIANA_W1648
M01: Mode 1
Step: 0 FREQUENCY: 2.25
Nodal DTXA...G RESDTX
Max = .145E-1 Min = 0
Factor = 48.8
1
Model: DIANA_W1648
M02: Mode 2
Step: 0 FREQUENCY: 8.18
Nodal DTXA...G RESDTX
Max = .924E-18 Min = 0
Factor = .764E18
2
Model: DIANA_W1648
M03: Mode 3
Step: 0 FREQUENCY: 17.4
Nodal DTXA...G RESDTX
Max = .193E-3 Min = 0
Factor = .366E4
3
Model: DIANA_W1648
M04: Mode 4
Step: 0 FREQUENCY: 32.1
Nodal DTXA...G RESDTX
Max = .368E-7 Min = 0
Factor = .192E8
4


Model: DIANA_W1648
M05: Mode 5
Step: 0 FREQUENCY: 32.7
Nodal DTXA...G RESDTX
Max = .123E-16 Min = 0
Factor = .572E17
1
Model: DIANA_W1648
M06: Mode 6
Step: 0 FREQUENCY: 48.9
Nodal DTXA...G RESDTX
Max = .813E-17 Min = 0
Factor = .869E17
2
Model: DIANA_W1648
M07: Mode 7
Step: 0 FREQUENCY: 55.4
Nodal DTXA...G RESDTX
Max = .822E-5 Min = 0
Factor = .859E5
3
Model: DIANA_W1648
M08: Mode 8
Step: 0 FREQUENCY: 69.2
Nodal DTXA...G RESDTX
Max = .542E-19 Min = 0
Factor = .13E20
4


Model: DIANA_W1648
MO9: Mode 9
Step: 0 FREQUENCY: 94.3
Nodal DTXA...G RESDTX
Max = .136E-6 Min = 0
Factor = .518E7
1
Model: DIANA_W1648
MO10: Mode 10
Step: 0 FREQUENCY: 119
Nodal DTXA...G RESDTX
Max = .482E-18 Min = 0
Factor = .146E19
2
Model: DIANA_W1648
MOA: ABS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .145E-1
Min = 0
3
Model: DIANA_W1648
MOS: SRSS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .145E-1
Min = 0
4

--Internal Forces--

Our aim is to check the seismic force acting on non-structural member, so the most important task is to obtain the shear force at the level of panel-columns bottom joints, so that to compare the results F_a/W_a with previous two methods. Internal forces of some important nodes are listed below. The following results are for ZONE1 G.Type A, results for other cases are plot in the comparison curves.



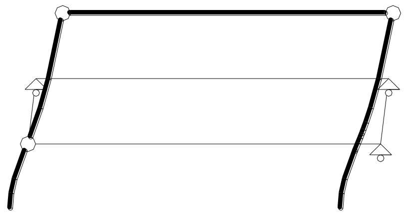
Only Seismic Load (Absolute_Max nodal internal forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.713E+03	4.037E+04	1.95E+05			
		2	2.708E+03	3.927E+04	1.19E+05			
	2	2	1.290E+03	2.499E+04	1.19E+05	1.418E+03	1.428E+04	1.007E+01
		3	1.296E+03	2.530E+04	5.96E+04			
	3	3	1.80E+01	2.54E+04	5.96E+04			
		4	1.35E+01	2.37E+04	2.66E+01			
	4	5	1.35E+01	2.37E+04	2.66E+01			
		6	1.80E+01	2.54E+04	5.96E+04			
	5	6	1.296E+03	2.530E+04	5.96E+04			
		7	1.290E+03	2.499E+04	1.19E+05			
	6	7	2.708E+03	3.927E+04	1.19E+05	1.418E+03	1.428E+04	1.007E+01
		8	2.713E+03	4.037E+04	1.95E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.252E+03	3.023E+04	1.75E+05			
		2	2.248E+03	2.967E+04	1.09E+05			
	2	2	1.073E+03	2.434E+04	1.09E+05	1.175E+03	5.330E+03	4.536E+00
		3	1.079E+03	2.324E+04	5.21E+04			
	3	3	1.49E+01	2.29E+04	5.21E+04			
		4	1.07E+01	2.03E+04	2.40E+01			
	4	5	1.07E+01	2.03E+04	2.40E+01			
		6	1.49E+01	2.29E+04	5.21E+04			
	5	6	1.079E+03	2.324E+04	5.21E+04			
		7	1.073E+03	2.434E+04	1.09E+05			
	6	7	2.248E+03	2.967E+04	1.09E+05	1.175E+03	5.330E+03	4.536E+00
		8	2.252E+03	3.023E+04	1.75E+05			

As a result: [N]

Base shear	Fa =ΣΔQ (Symmetric)
ABS: Qy(1)+Qy(8)= 8.074E+04	ABS: ΔQ(2)×2= 2.856E+04
SRSS: Qy(1)+Qy(8)= 6.046E+04	SRSS: ΔQ(2)×2= 1.066E+04

Case 2)------ 5 DOF



--Eigen Analysis--

TOTAL MASS [kg]
0.344E+05
EIGEN-FREQUENCIES: (Hz)
0.22514D+01(1) 0.81845D+01(2) 0.13179D+02(3) 0.26822D+02(4)
0.32067D+02(5) 0.33304D+02(6) 0.52139D+02(7) 0.69184D+02(8)
0.84586D+02(9) 0.94322D+02(10)

MODE	FREQUENCY[Hz]	PERIOD[s]	MASS.P.X[%]	MASS.P.Y[%]
1	2.25E+00	0.4442	64.207	0.000
2	8.18E+00	0.1222	0.000	32.493
3	1.32E+01	0.0759	26.234	0.112
4	2.68E+01	0.0373	3.704	0.000
5	3.21E+01	0.0312	0.000	0.000
6	3.33E+01	0.0300	0.040	29.741
7	5.21E+01	0.0192	0.714	2.272
8	6.92E+01	0.0145	0.000	7.250
9	8.46E+01	0.0118	1.064	0.000
10	9.43E+01	0.0106	0.000	0.000

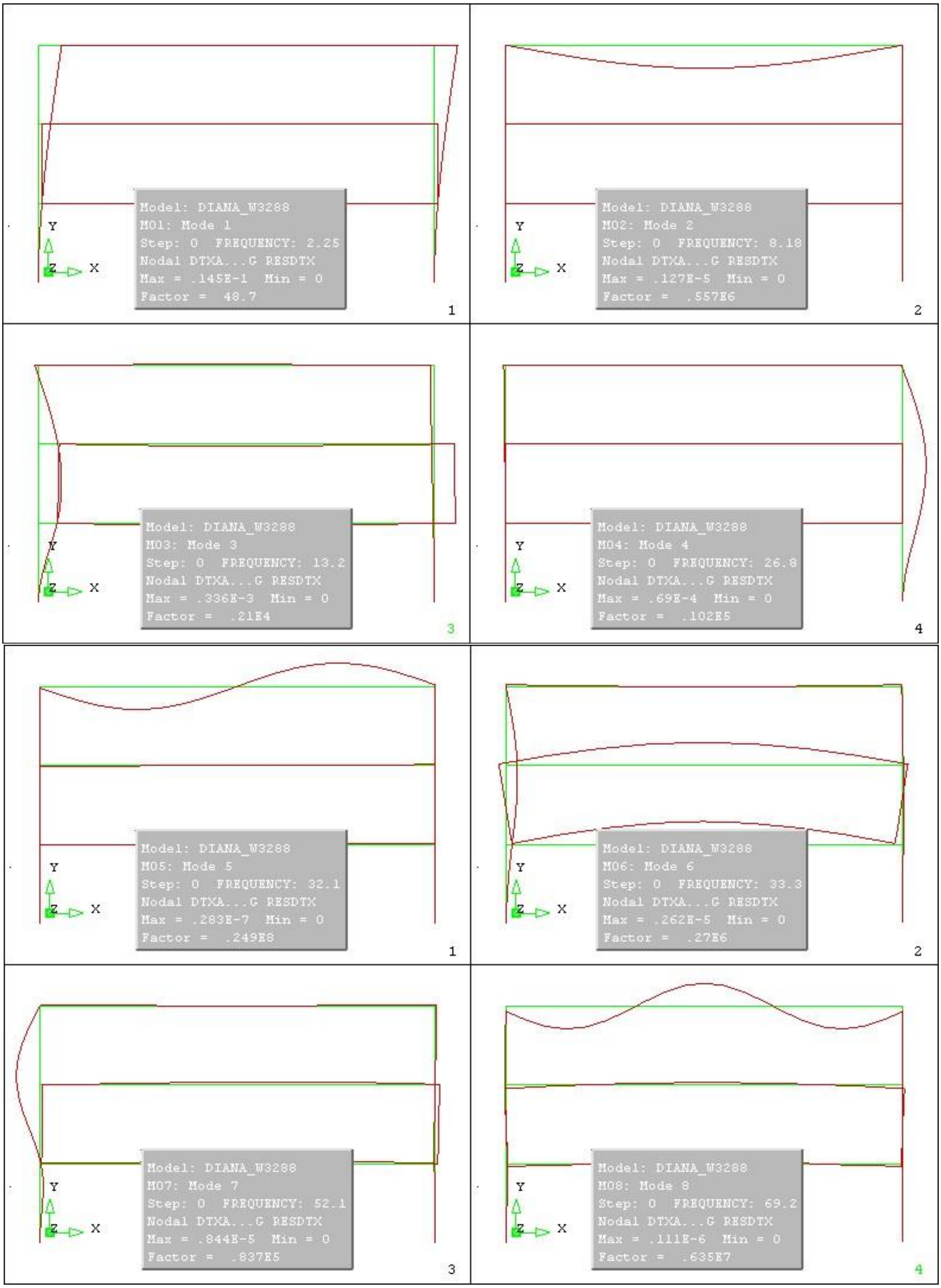
Comparison:

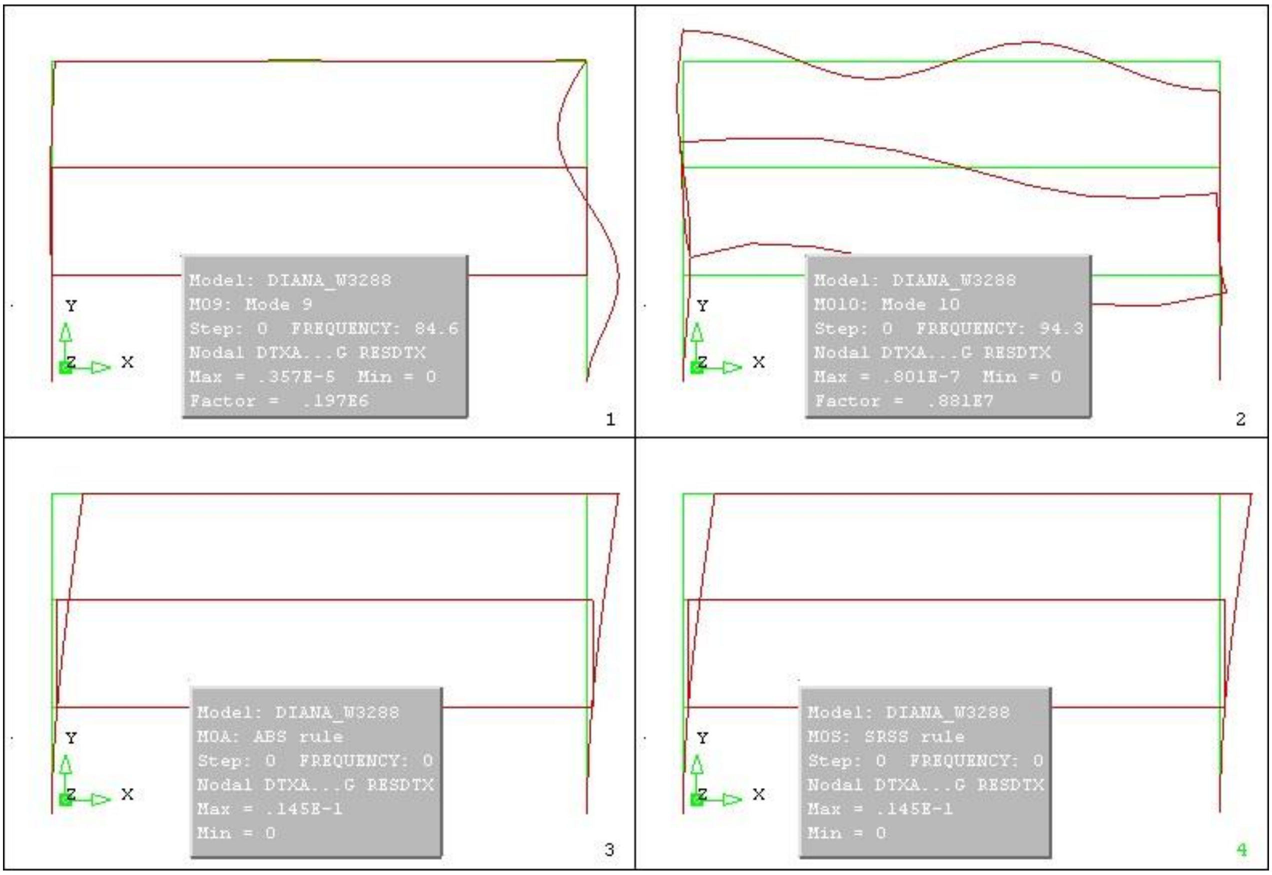
The total mass calculated by Diana is the same as calculated by LSM before, both equal to 0.344E+05 (kg) ,The first Mode Period computed by LSM in previous chapter is: 0.458s, which value is very close to the FEA result: 0.444s.

--Deform Shapes for Different Modes--

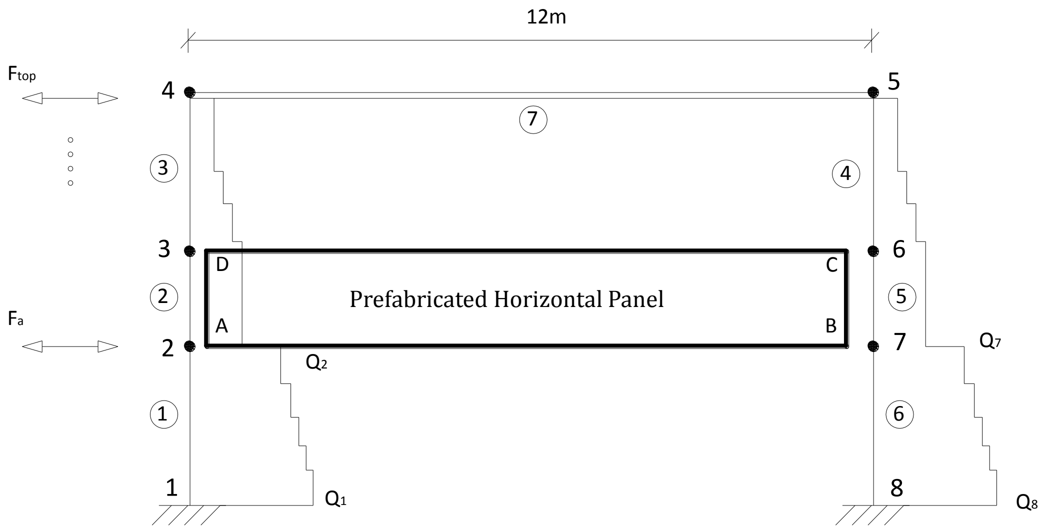
Open .M70 file in iDiana. (Interaction with iDiana)
Plot deformed shapes for each 10 modes and combination shape according to the two rules.

```
FEMVIEW SPECTR
RESULTS LOADCASE MO1
RESULTS NODAL DTXA...G RESDTX
PRESENT SHAPE
RESULTS LOADCASE MO2
PRESENT SHAPE
……
RESULTS LOADCASE MOA
PRESENT SHAPE
RESULTS LOADCASE MOS
PRESENT SHAPE
```


Model: DIANA_W3288
M01: Mode 1
Step: 0 FREQUENCY: 2.25
Nodal DTXA...G RESDTX
Max = .145E-1 Min = 0
Factor = 48.7
1
Model: DIANA_W3288
M02: Mode 2
Step: 0 FREQUENCY: 8.18
Nodal DTXA...G RESDTX
Max = .127E-5 Min = 0
Factor = .557E6
2
Model: DIANA_W3288
M03: Mode 3
Step: 0 FREQUENCY: 13.2
Nodal DTXA...G RESDTX
Max = .336E-3 Min = 0
Factor = .21E4
3
Model: DIANA_W3288
M04: Mode 4
Step: 0 FREQUENCY: 26.8
Nodal DTXA...G RESDTX
Max = .69E-4 Min = 0
Factor = .102E5
4
Model: DIANA_W3288
M05: Mode 5
Step: 0 FREQUENCY: 32.1
Nodal DTXA...G RESDTX
Max = .283E-7 Min = 0
Factor = .249E8
1
Model: DIANA_W3288
M06: Mode 6
Step: 0 FREQUENCY: 33.3
Nodal DTXA...G RESDTX
Max = .262E-5 Min = 0
Factor = .27E6
2
Model: DIANA_W3288
M07: Mode 7
Step: 0 FREQUENCY: 52.1
Nodal DTXA...G RESDTX
Max = .844E-5 Min = 0
Factor = .837E5
3
Model: DIANA_W3288
M08: Mode 8
Step: 0 FREQUENCY: 69.2
Nodal DTXA...G RESDTX
Max = .111E-6 Min = 0
Factor = .635E7
4


Model: DIANA_W3288
M09: Mode 9
Step: 0 FREQUENCY: 84.6
Nodal DTXA...G RESDTX
Max = .357E-5 Min = 0
Factor = .197E6
1
Model: DIANA_W3288
M010: Mode 10
Step: 0 FREQUENCY: 94.3
Nodal DTXA...G RESDTX
Max = .801E-7 Min = 0
Factor = .881E7
2
Model: DIANA_W3288
MOA: ABS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .145E-1
Min = 0
3
Model: DIANA_W3288
MOS: SRSS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .145E-1
Min = 0
4

--Internal Forces--



Only Seismic Load (Absolute_Max nodal internal forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.40E+03	5.35E+04	2.15E+05			
		2	2.39E+03	5.24E+04	1.26E+05			
	2	2	4.18E+03	2.53E+04	1.26E+05	1.80E+03	2.71E+04	1.51E+01
		3	4.15E+03	2.69E+04	6.32E+04			
	3	3	1.38E+02	2.70E+04	6.32E+04			
		4	1.03E+02	2.54E+04	2.67E+01			
	4	5	9.23E+01	2.28E+04	2.75E+01			
		6	1.32E+02	2.39E+04	5.64E+04			
	5	6	1.41E+03	2.39E+04	5.64E+04			
		7	1.45E+03	2.72E+04	1.12E+05			
	6	7	3.97E+03	2.77E+04	1.12E+05	2.52E+03	4.70E+02	1.86E-01
		8	3.99E+03	2.85E+04	1.77E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.68E+03	3.80E+04	1.82E+05			
		2	1.67E+03	3.72E+04	1.09E+05			
	2	2	3.38E+03	2.40E+04	1.09E+05	1.71E+03	1.32E+04	7.75E+00
		3	3.38E+03	2.31E+04	5.21E+04			
	3	3	8.04E+01	2.28E+04	5.21E+04			
		4	5.34E+01	2.04E+04	2.39E+01			
	4	5	5.12E+01	2.05E+04	2.40E+01			
		6	7.98E+01	2.32E+04	5.26E+04			
	5	6	1.10E+03	2.36E+04	5.26E+04			
		7	1.11E+03	2.48E+04	1.11E+05			
	6	7	3.16E+03	2.50E+04	1.11E+05	2.05E+03	1.40E+02	6.84E-02
		8	3.16E+03	2.52E+04	1.71E+05			

As a result: [N]

Base shear

ABS: Qy (1)+Qy (8)= 8.20E+04

SRSS: Qy (1)+Qy (8)= 6.32E+04

Fa =ΣΔQ

ABS: ΔQ(2)+ ΔQ(7)= 2.76E+04

SRSS: ΔQ(2)+ ΔQ(7)= 1.34E+04

Case 3)------ 6 DOF

--Eigen Analysis--

TOTAL MASS [kg]
0.344E+05
EIGEN-FREQUENCIES: (Hz)
0.38870D+01(1) 0.11058D+02(2) 0.20306D+02(3) 0.33942D+02(4)
0.35343D+02(5) 0.59237D+02(6) 0.71642D+02(7) 0.75126D+02(8)
0.94316D+02(9) 0.12217D+03(10)

MODE	FREQUENCY[Hz]	PERIOD[s]	MASS.P.X[%]	MASS.P.Y[%]
1	3.89E+00	0.2573	71.965	0.000
2	1.11E+01	0.0904	0.000	33.748
3	2.03E+01	0.0492	22.334	0.000
4	3.39E+01	0.0295	0.000	26.841
5	3.53E+01	0.0283	0.889	0.000
6	5.92E+01	0.0169	0.000	0.967
7	7.16E+01	0.0140	0.763	0.000
8	7.51E+01	0.0133	0.000	14.229
9	9.43E+01	0.0106	0.023	0.000
10	1.22E+02	0.0082	0.000	18.238

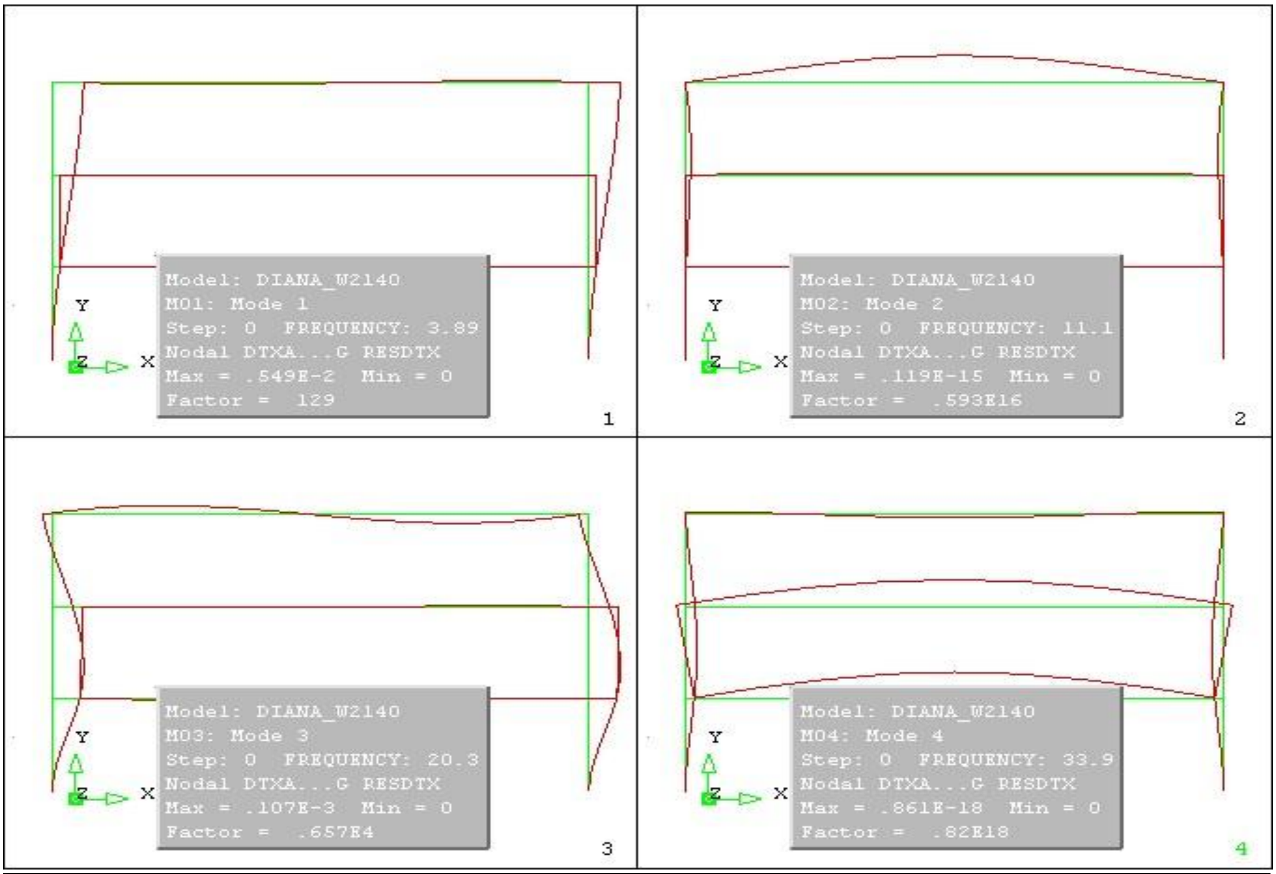
Comparison:

The total mass calculated by Diana is the same as calculated by LSM before, both equal to 0.344E+05 (kg) ,The first Mode Period computed by LSM in previous chapter is: 0.258s, which value is very close to the FEA result: 0.257s.

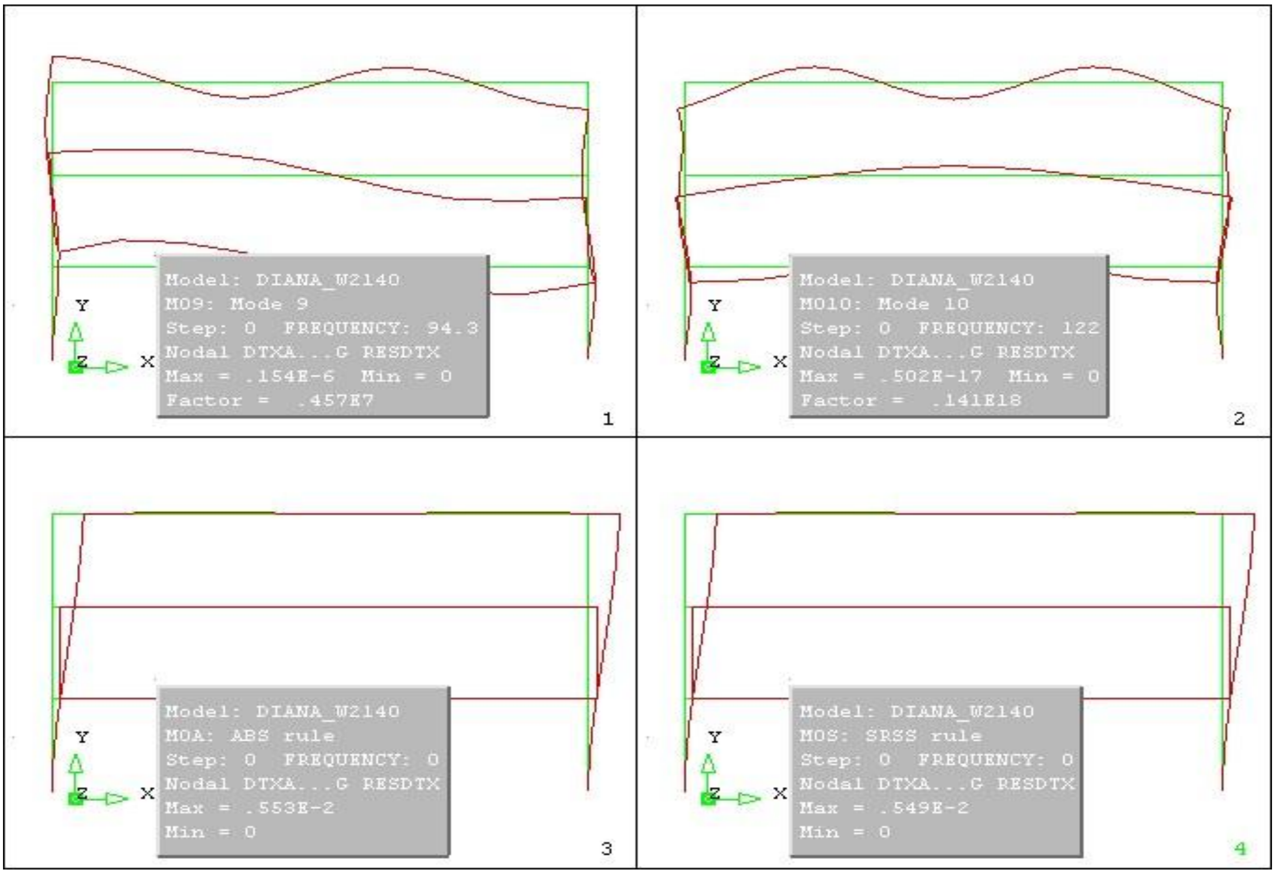
--Deform Shapes for Different Modes--

Open .M70 file in iDiana. (Interaction with iDiana)
Plot deformed shapes for each 10 modes and combination shape according to the two rules.

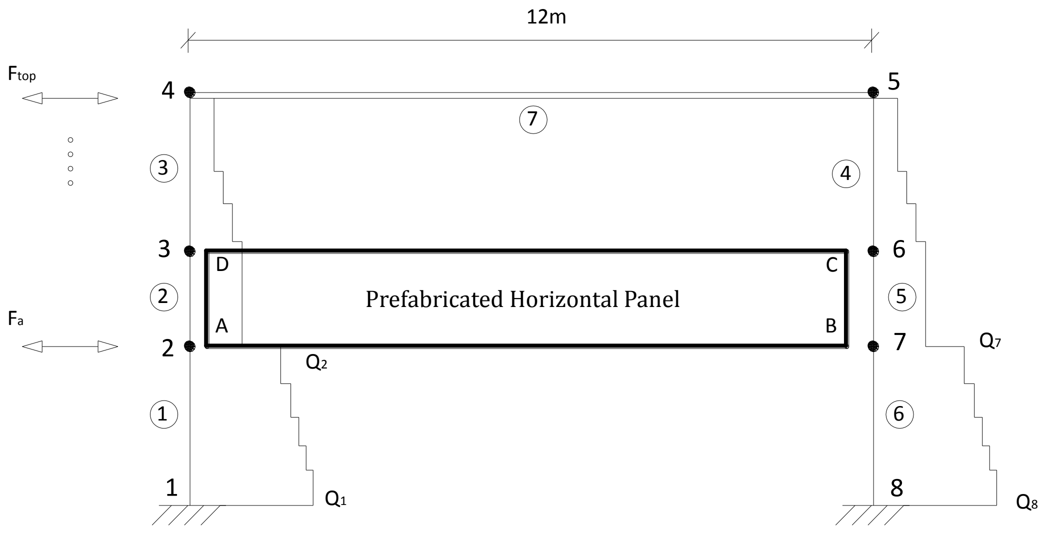
```
FEMVIEW SPECTR
RESULTS LOADCASE MO1
RESULTS NODAL DTXA...G RESDTX
PRESENT SHAPE
RESULTS LOADCASE MO2
PRESENT SHAPE
……
RESULTS LOADCASE MOA
PRESENT SHAPE
RESULTS LOADCASE MOS
PRESENT SHAPE
```


Model: DIANA_W2140
M01: Mode 1
Step: 0 FREQUENCY: 3.89
Nodal DTXA...G RESDTX
Max = .549E-2 Min = 0
Factor = 129
1
Model: DIANA_W2140
M02: Mode 2
Step: 0 FREQUENCY: 11.1
Nodal DTXA...G RESDTX
Max = .119E-15 Min = 0
Factor = .593E16
2
Model: DIANA_W2140
M03: Mode 3
Step: 0 FREQUENCY: 20.3
Nodal DTXA...G RESDTX
Max = .107E-3 Min = 0
Factor = .657E4
3
Model: DIANA_W2140
M04: Mode 4
Step: 0 FREQUENCY: 33.9
Nodal DTXA...G RESDTX
Max = .861E-18 Min = 0
Factor = .82E18
4
Y
Z
X


Model: DIANA_W2140
M05: Mode 5
Step: 0 FREQUENCY: 35.3
Nodal DTXA...G RESDTX
Max = .101E-4 Min = 0
Factor = .702E5
1
Model: DIANA_W2140
M06: Mode 6
Step: 0 FREQUENCY: 59.2
Nodal DTXA...G RESDTX
Max = .997E-18 Min = 0
Factor = .708E18
2
Model: DIANA_W2140
M07: Mode 7
Step: 0 FREQUENCY: 71.6
Nodal DTXA...G RESDTX
Max = .327E-5 Min = 0
Factor = .216E6
3
Model: DIANA_W2140
M08: Mode 8
Step: 0 FREQUENCY: 75.1
Nodal DTXA...G RESDTX
Max = .11E-18 Min = 0
Factor = .641E19
4
Y
Z
X


Model: DIANA_W2140
MO9: Mode 9
Step: 0 FREQUENCY: 94.3
Nodal DTXA...G RESDTX
Max = .154E-6 Min = 0
Factor = .457E7
1
Model: DIANA_W2140
MO10: Mode 10
Step: 0 FREQUENCY: 122
Nodal DTXA...G RESDTX
Max = .502E-17 Min = 0
Factor = .141E18
2
Model: DIANA_W2140
MOA: ABS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .553E-2
Min = 0
3
Model: DIANA_W2140
MOS: SRSS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .549E-2
Min = 0
4

--Internal Forces--



Only Seismic Load (Absolute_Max nodal internal forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.857E+04	4.397E+04	1.34E+05			
		2	1.855E+04	4.276E+04	4.66E+04			
	2	2	1.076E+04	3.012E+04	4.66E+04	7.790E+03	1.264E+04	1.623E+00
		3	1.074E+04	3.025E+04	3.24E+04			
	3	3	1.58E+04	2.99E+04	3.24E+04			
		4	1.57E+04	2.64E+04	9.43E+04			
	4	5	1.57E+04	2.64E+04	9.43E+04			
		6	1.58E+04	2.99E+04	3.24E+04			
	5	6	1.074E+04	3.025E+04	3.24E+04			
		7	1.076E+04	3.012E+04	4.66E+04			
	6	7	1.855E+04	4.276E+04	4.66E+04	7.790E+03	1.264E+04	1.623E+00
		8	1.857E+04	4.397E+04	1.34E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.534E+04	3.528E+04	1.20E+05			
		2	1.534E+04	3.474E+04	3.87E+04			
	2	2	9.210E+03	2.875E+04	3.87E+04	6.130E+03	5.990E+03	9.772E-01
		3	9.205E+03	2.697E+04	2.92E+04			
	3	3	1.43E+04	2.64E+04	2.92E+04			
		4	1.42E+04	2.29E+04	8.79E+04			
	4	5	1.42E+04	2.29E+04	8.79E+04			
		6	1.43E+04	2.64E+04	2.92E+04			
	5	6	9.205E+03	2.697E+04	2.92E+04			
		7	9.210E+03	2.875E+04	3.87E+04			
	6	7	1.534E+04	3.474E+04	3.87E+04	6.130E+03	5.990E+03	9.772E-01
		8	1.534E+04	3.528E+04	1.20E+05			

As a result: [N]

Base shear

ABS: Qy(1)+Qy(8)= 8.794E+04

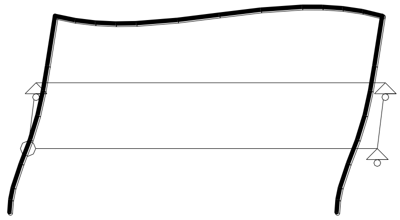
SRSS: Qy(1)+Qy(8)= 7.056E+04

Fa =ΣΔQ (Symmetric)

ABS: ΔQ(2)×2= 2.528E+04

SRSS: ΔQ(2)×2= 1.198E+04

Case 4)------ 7 DOF



--Eigen Analysis--

TOTAL MASS [kg]
0.344E+05
EIGEN-FREQUENCIES: (Hz)
0.38843D+01(1) 0.10231D+02(2) 0.15662D+02(3) 0.31460D+02(4)
0.33958D+02(5) 0.39698D+02(6) 0.64499D+02(7) 0.74532D+02(8)
0.94313D+02(9) 0.97333D+02(10)

MODE	FREQUENCY[Hz]	PERIOD[s]	MASS.P.X[%]	MASS.P.Y[%]
1	3.88E+00	0.2574	72.473	0.000
2	1.02E+01	0.0977	1.245	28.159
3	1.57E+01	0.0638	18.560	4.787
4	3.15E+01	0.0318	0.787	2.930
5	3.40E+01	0.0294	0.000	25.735
6	3.97E+01	0.0252	1.895	1.058
7	6.45E+01	0.0155	0.093	0.000
8	7.45E+01	0.0134	0.153	11.594
9	9.43E+01	0.0106	0.034	0.000
10	9.73E+01	0.0103	0.800	1.666

Comparison:

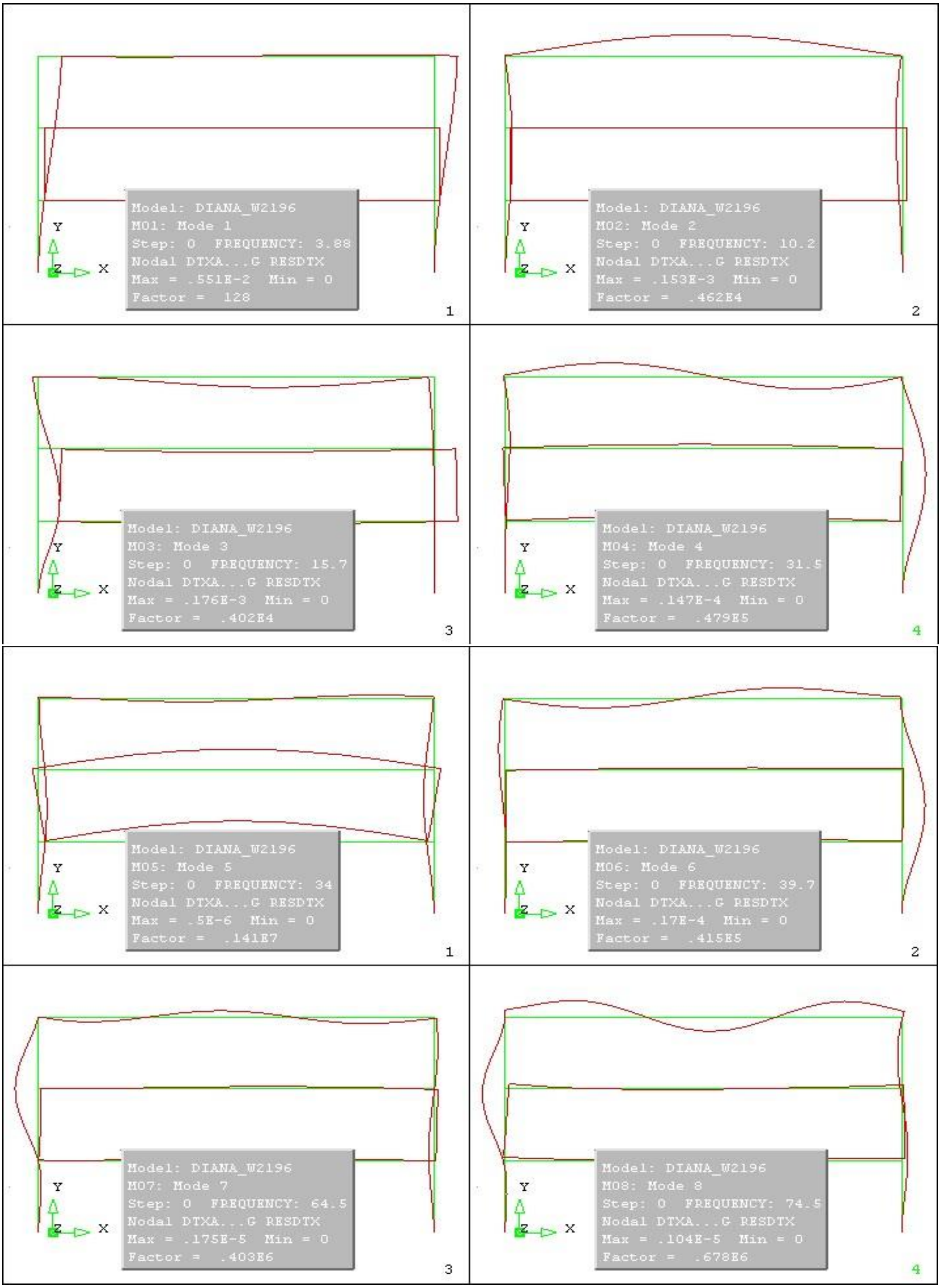
The total mass calculated by Diana is the same as calculated by LSM before, both equal to 0.344E+05 (kg) ,The first Mode Period computed by LSM in previous chapter is: 0.259s, which value is very close to the FEA result: 0.257s.

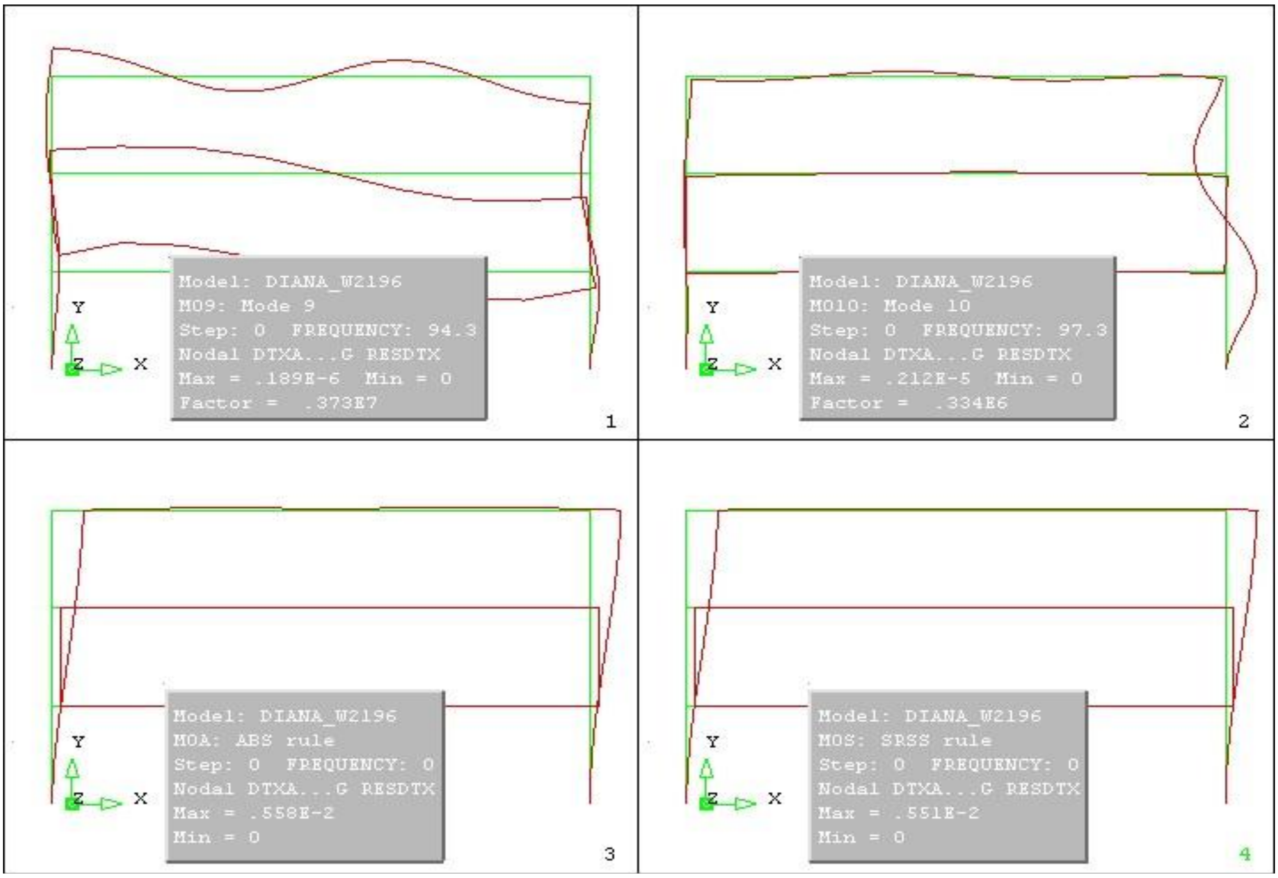
--Deform Shapes for Different Modes--

Open .M70 file in iDiana. (Interaction with iDiana)

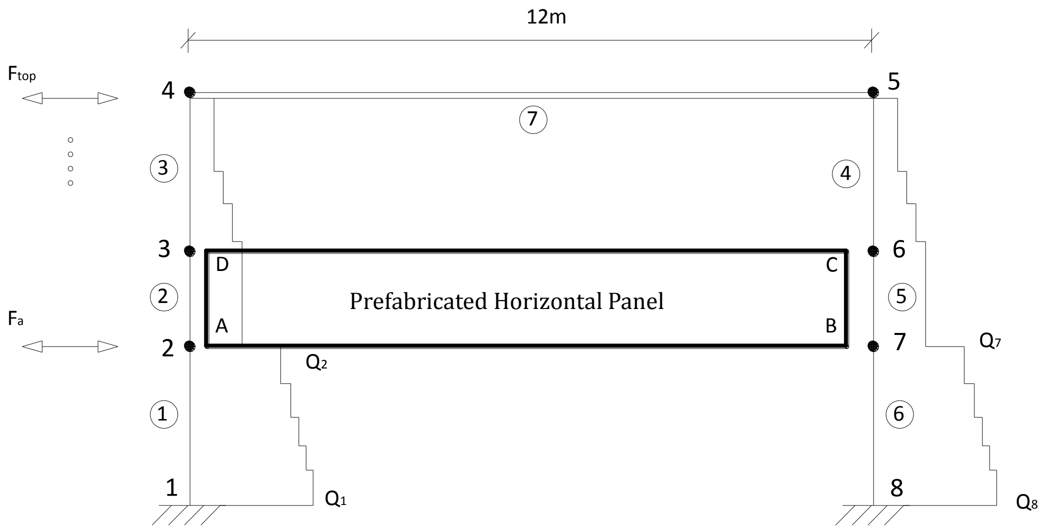
Plot deformed shapes for each 10 modes and combination shape according to the two rules.

```
FEMVIEW SPECTR
RESULTS LOADCASE MO1
RESULTS NODAL DTXA...G RESDTX
PRESENT SHAPE
RESULTS LOADCASE MO2
PRESENT SHAPE
……
RESULTS LOADCASE MOA
PRESENT SHAPE
RESULTS LOADCASE MOS
PRESENT SHAPE
```


Model: DIANA_W2196
M01: Mode 1
Step: 0 FREQUENCY: 3.88
Nodal DTXA...G RESDTX
Max = .551E-2 Min = 0
Factor = 128
1
Model: DIANA_W2196
M02: Mode 2
Step: 0 FREQUENCY: 10.2
Nodal DTXA...G RESDTX
Max = .153E-3 Min = 0
Factor = .462E4
2
Model: DIANA_W2196
M03: Mode 3
Step: 0 FREQUENCY: 15.7
Nodal DTXA...G RESDTX
Max = .176E-3 Min = 0
Factor = .402E4
3
Model: DIANA_W2196
M04: Mode 4
Step: 0 FREQUENCY: 31.5
Nodal DTXA...G RESDTX
Max = .147E-4 Min = 0
Factor = .479E5
4
Model: DIANA_W2196
M05: Mode 5
Step: 0 FREQUENCY: 34
Nodal DTXA...G RESDTX
Max = .5E-6 Min = 0
Factor = .141E7
1
Model: DIANA_W2196
M06: Mode 6
Step: 0 FREQUENCY: 39.7
Nodal DTXA...G RESDTX
Max = .17E-4 Min = 0
Factor = .415E5
2
Model: DIANA_W2196
M07: Mode 7
Step: 0 FREQUENCY: 64.5
Nodal DTXA...G RESDTX
Max = .175E-5 Min = 0
Factor = .403E6
3
Model: DIANA_W2196
M08: Mode 8
Step: 0 FREQUENCY: 74.5
Nodal DTXA...G RESDTX
Max = .104E-5 Min = 0
Factor = .678E6
4
Y
Z
X


Model: DIANA_W2196
MO9: Mode 9
Step: 0 FREQUENCY: 94.3
Nodal DTXA...G RESDTX
Max = .189E-6 Min = 0
Factor = .373E7
1
Model: DIANA_W2196
MO10: Mode 10
Step: 0 FREQUENCY: 97.3
Nodal DTXA...G RESDTX
Max = .212E-5 Min = 0
Factor = .334E6
2
Model: DIANA_W2196
MOA: ABS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .558E-2
Min = 0
3
Model: DIANA_W2196
MOS: SRSS rule
Step: 0 FREQUENCY: 0
Nodal DTXA...G RESDTX
Max = .551E-2
Min = 0
4
Y
Z
X

--Internal Forces--



Only Seismic Load (Absolute_Max nodal internal forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.337E+04	5.650E+04	1.52E+05			
		2	2.330E+04	5.529E+04	5.10E+04			
	2	2	1.671E+04	3.289E+04	5.10E+04	6.590E+03	2.240E+04	3.399E+00
		3	1.653E+04	3.220E+04	3.59E+04			
	3	3	2.22E+04	3.18E+04	3.59E+04			
		4	2.19E+04	2.82E+04	1.02E+05			
	4	5	2.08E+04	2.60E+04	9.55E+04			
		6	2.10E+04	2.87E+04	3.21E+04			
	5	6	1.533E+04	2.930E+04	3.21E+04			
		7	1.548E+04	3.151E+04	4.25E+04			
	6	7	2.457E+04	3.176E+04	4.25E+04	9.090E+03	2.500E+02	2.750E-02
		8	2.462E+04	3.296E+04	1.18E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.586E+04	4.142E+04	1.28E+05			
		2	1.585E+04	4.080E+04	3.75E+04			
	2	2	1.004E+04	2.815E+04	3.75E+04	5.810E+03	1.265E+04	2.177E+00
		3	1.002E+04	2.653E+04	3.06E+04			
	3	3	1.50E+04	2.60E+04	3.06E+04			
		4	1.50E+04	2.25E+04	8.77E+04			
	4	5	1.47E+04	2.36E+04	8.92E+04			
		6	1.47E+04	2.72E+04	2.81E+04			
	5	6	1.045E+04	2.781E+04	2.81E+04			
		7	1.046E+04	2.973E+04	4.13E+04			
	6	7	1.638E+04	2.993E+04	4.13E+04	5.920E+03	2.000E+02	3.378E-02
		8	1.639E+04	3.026E+04	1.14E+05			

As a result: [N]

Base shear

ABS: Qy(1)+Qy(8)= 8.946E+04

SRSS: Qy(1)+Qy(8)= 7.168E+04

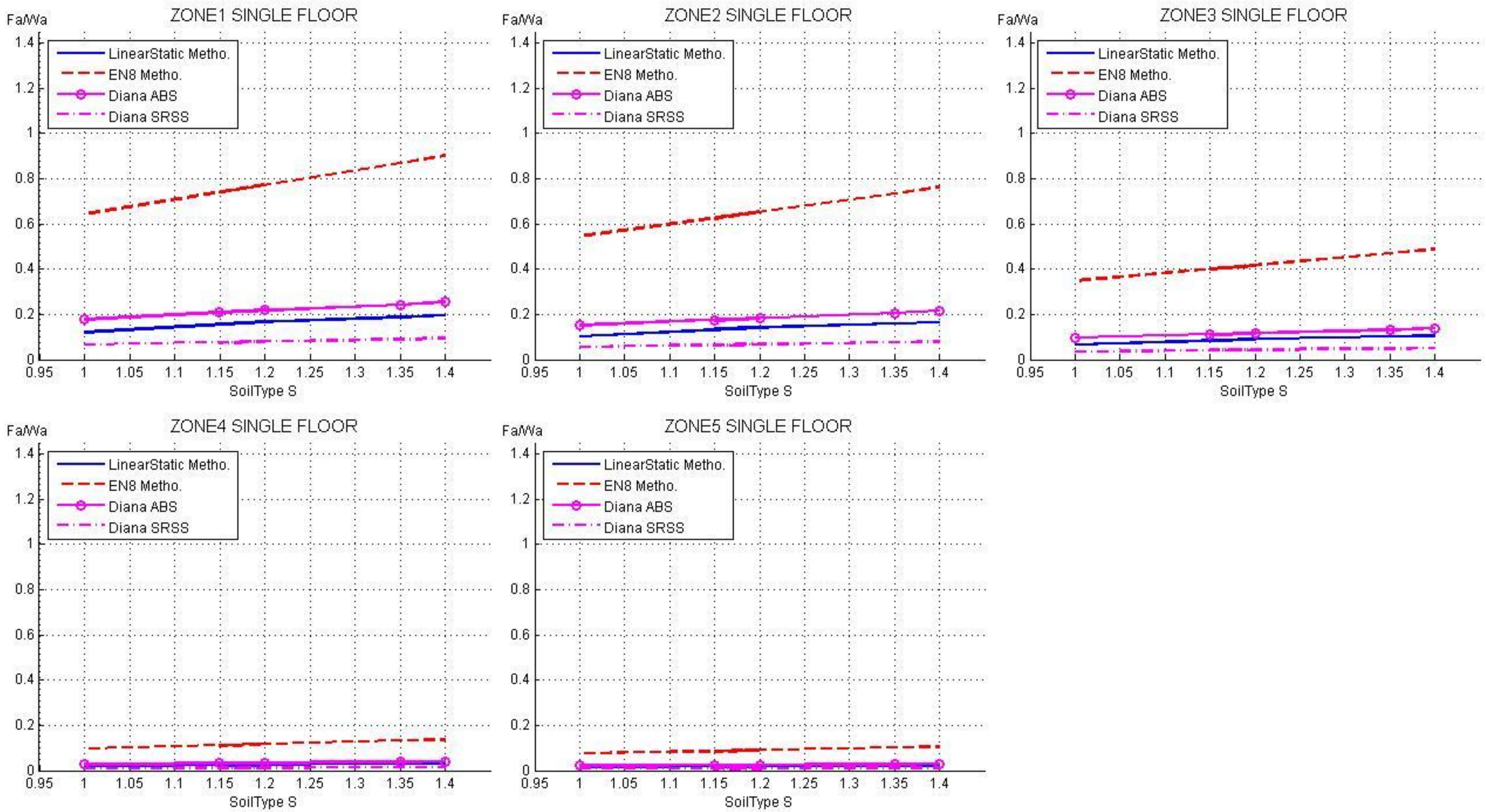
Fa =ΣΔQ

ABS: ΔQ(2)+ ΔQ(7)=2.265E+04

SRSS: ΔQ(2)+ ΔQ(7)=1.285E+04

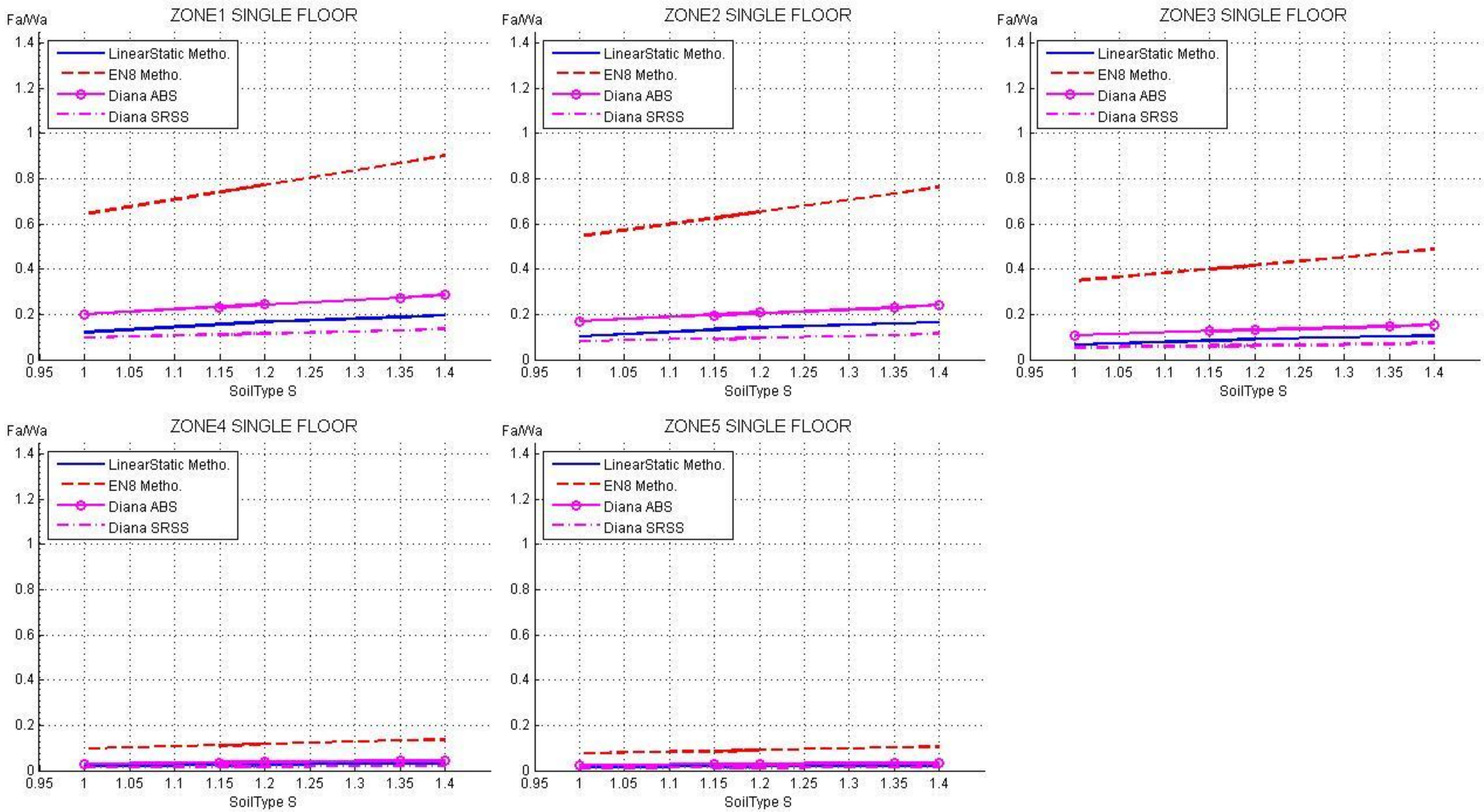
Comparison of Numerical Results and Results of LSM ----Case 1)

F_a/W_a varied with SoilType S for 5 seismic zones



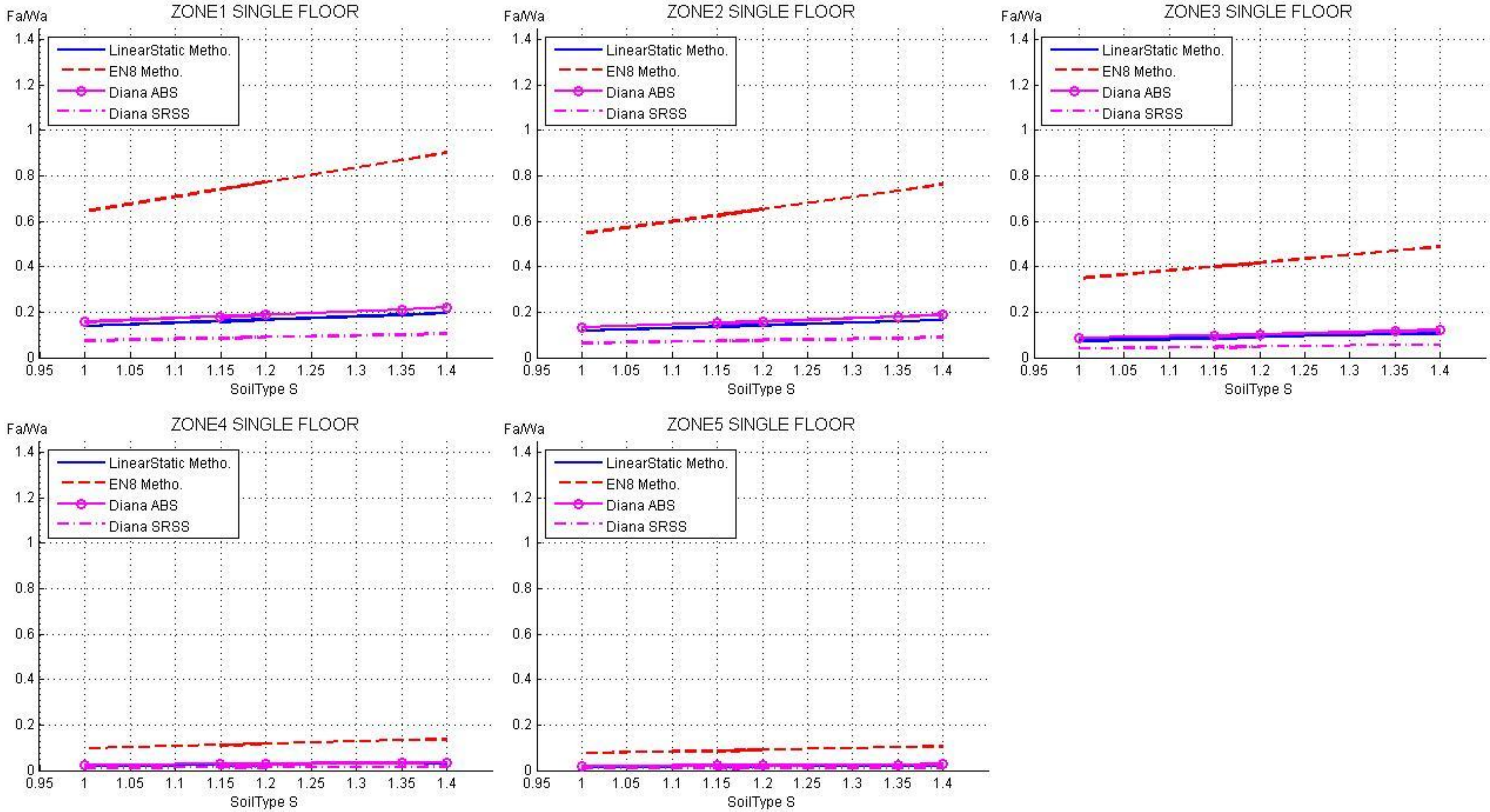
Comparison of Numerical Results and Results of LSM ----Case 2)

F_a/W_a varied with SoilType S for 5 seismic zones



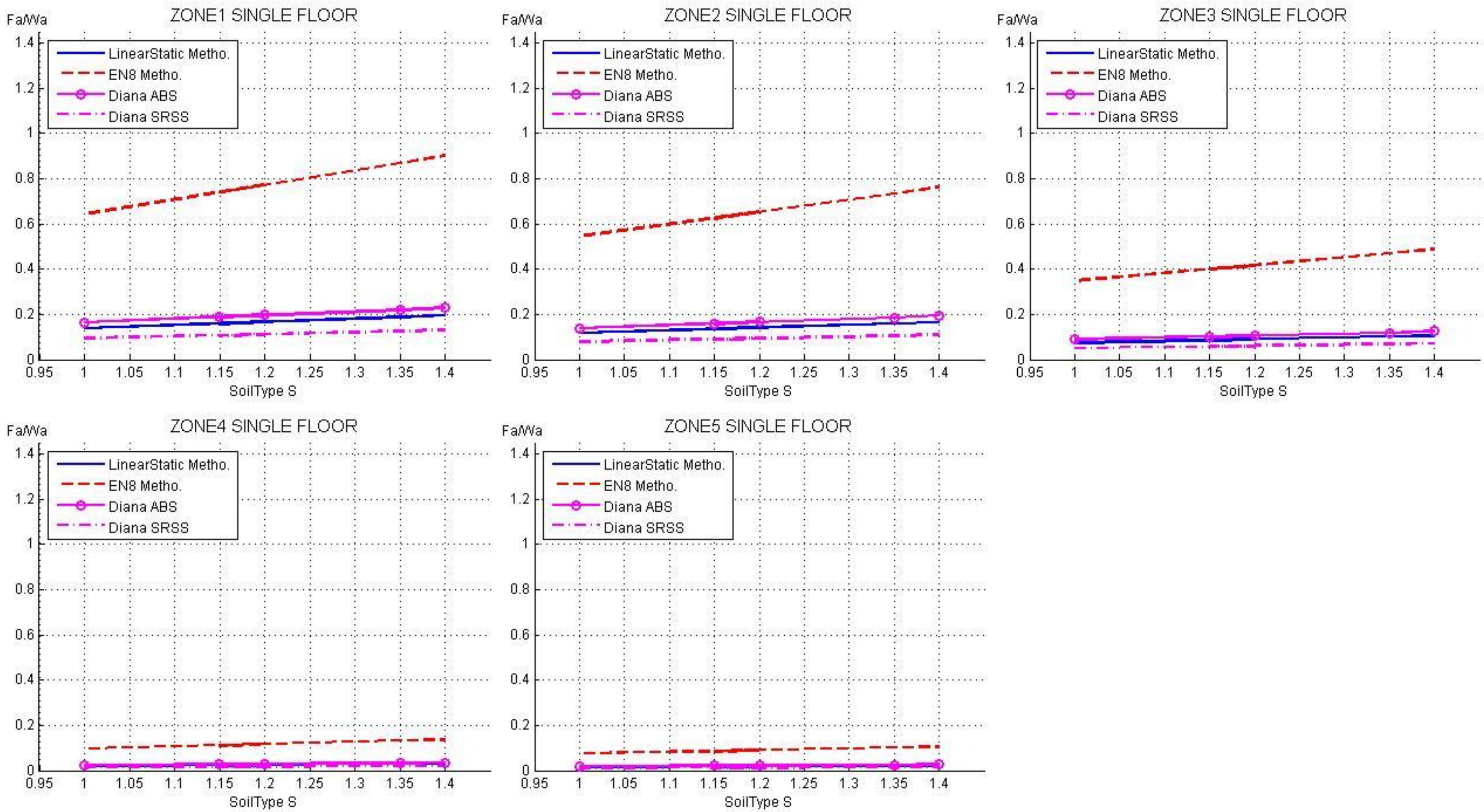
Comparison of Numerical Results and Results of LSM ----Case 3)

F_a/W_a varied with SoilType S for 5 seismic zones



Comparison of Numerical Results and Results of LSM ----Case 4)

F_a/W_a varied with SoilType S for 5 seismic zones



--Conclusions--

- The results obtained by EN8 empirical formula is much larger than the values calculated by other methods.

-The results calculated by LSM (blue lines) are in good accordance with numerical results as most of the data lie between the lines of two combination rules. In addition, the tendencies of increment with respect to soil coefficient are approximately the same for LSM and numerical method (same slope of lines).

-ABS combination rule could be more critical than SRSS combination rule in an earthquake spectral analysis

-Boundary Condition Case2 could be more critical, because the Fa/Wa values of Case2 is apparently larger than other cases. This is mainly due to 2 pinned & 1 roller boundaries of beam-column and panel-column connections.

-All values are small under low magnitude of seismic action, thus can hardly be distinguished for different analysis methods.

-Non-linear analysis should be performed to see if the result is more close to linear dynamic & Linear static results or EN8 empirical formula’s.

IV. Non-linear Dynamic Analysis by Diana

In non-linear seismic analysis of the 2-D frame, the concept of modal analysis and modes combination is no longer used, because these are generally used in linear analysis. Instead, we apply direct input of earthquake waveform which is Ag-t curve (the magnitude of seismic acceleration versus time). Both element types and material definition in Diana should be modified in accordance with nonlinear analysis.

In nonlinear batch commands(the executive file), we no longer use spectral analysis, whose modulus starts with key word: *SPECTR. Instead, *NONLIN modulus is employed in order to run a transient dynamic analysis with iteration steps. It is also not necessary to define a response spectrum curve(seismic response Sd versus frequency), instead, define a time-load relationship in 'TIMELO' sub-modulus for seismic action Ag.

The major difference with linear analysis, which is the outline for executive file(*.com):

```
------------------------------------------------------------------------------------------------------------
*NONLIN
BEGIN MODEL
….
END MODEL
BEGIN TYPE
….
END TYPE
BEGIN EXECUT
START
TIME
[ PHYSIC _ _ _ ]
[ ITERAT _ _ _ ]
[ SOLVE _ _ _ ]
[ OUTPUT _ _ _ ] …
END EXECUT
*END
------------------------------------------------------------------------------------------------------------
```

Create new constitutive laws for non-linear behaviour of the materials. Run linear spectral analysis first with the new model, then run non-linear analysis with the same model so as to compare the results. We are interested in the point that: in a non-linear analysis with a direct input of seismic wave, the force Fa acting on non-structural panel will be the same as in linear approach or more close to the value calculated with EN8 empirical formula.

Discussed below are for 4 types of connection conditions Case1~Case4 as before and for 4 different magnitudes of Ag taken from ITACA waveform curve. And also shows a comparison among Linear Static; EN8 empirical formula; Linear Dynamic (Diana); Non-linear (Diana) methods.

1. Seismic Waveform

In order to be more realistic and accurate, getting access to the seismic information data base in Italy: ITACA. And found a list of specific earthquake records during past several decades and situated in different earthquake zones in Italy. It is quite difficult to choose the exact records which are highly representative of 5 ZONES and for all 5 soil types, but we can choose from them 4 different levels of magnitudes for type B soil (because we have more records for this soil type than others).

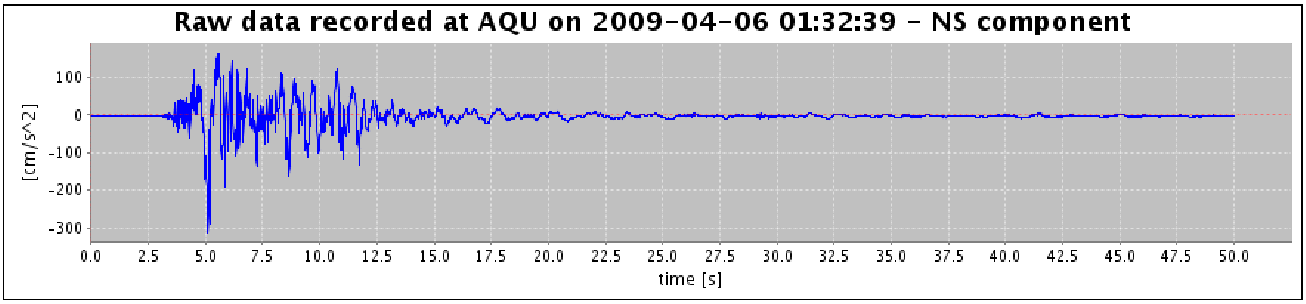
Peak Ground Acceleration Values for each of them are:
PGA=0.25g; 0.215g; 0.15g; 0.089g

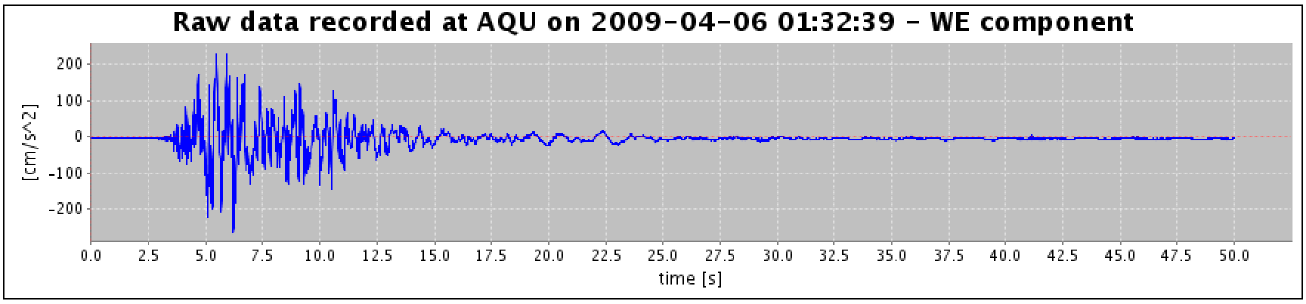
One example of selecting the time-load curve is explained below:
Search for [Magnitude (MW or ML): 6~6.5; Epicenter distance [Km]: <=30km; Corrected PGA [cm/s2]: 300~400].
3 records are found. A possible record to represent ZONE1~2-GTB is:

Date	MW	ML	Stat. Code	EC8	R epi. [km]	Corr. PGA [cm/s2]	PGV [cm/s]	PGD [cm]
2009-04-06 01:32:39	6.3	5.8	AQU	B*	6.018	301.827	29.503	8.9969

The event took place in L'Aquila, with an epicenter only 6.018km's deep from the ground surface. The * sign implies reliable data. "Corr" means corrected value which has eliminated the disturbances and noises of the wave.

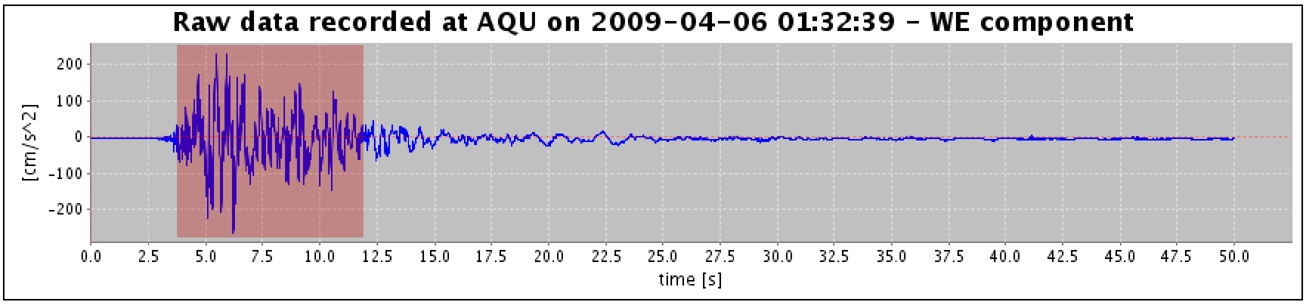
Click "Details" to expand this record:
We have 3 components of the waveform: NS; WE; UP. Choose a critical direction:
The peak acceleration lies in Waveform NS -301.8269 [cm/s2], however the average value is higher in WE direction. The wave is also more symmetrically distributed in "+" & "-" directions. Then choose Waveform WE as design input. We may also check for NS wave anyway.



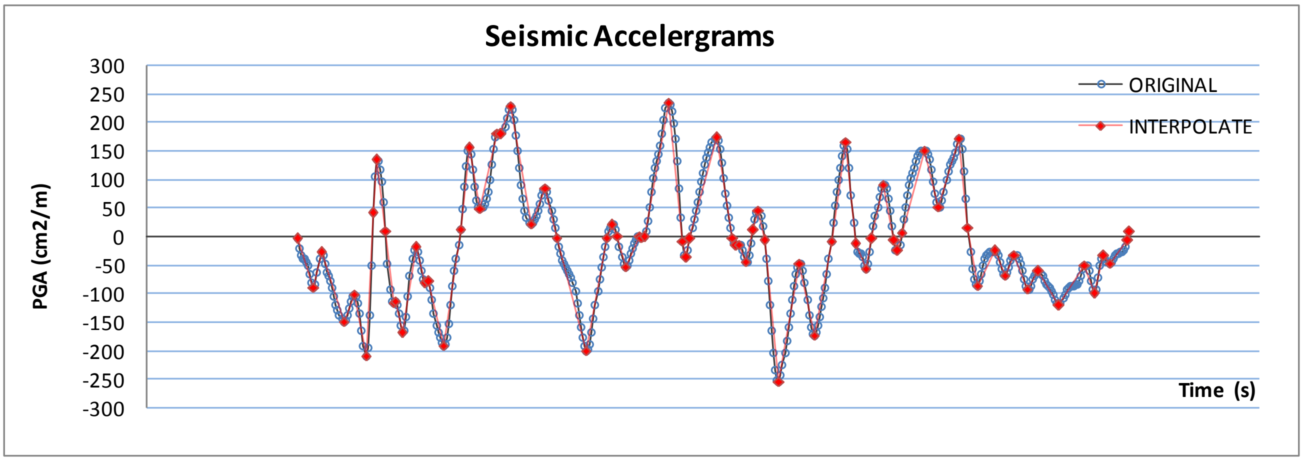


The peak value in this graph is -254.4940 [cm/s2], at 6.2s, and T90 Effective duration [s] is 7.69s, Approximately from 3.5s to 11s. We can also have a look at the velocity plot and displacement plot. But the input data only consist of acceleration. So, we don't need to use them.

Take the effective duration from the earthquake wave as input: 3.5s~11s (7.5s)



The waveform as seismic input may consist of a huge among of points with 0.005s interval, that will cause laborious computation efforts and consume a lot of time as well as create instability in iteration process. To simplify the wave, take away mid points between convexes and concaves using linear interpolation as an approximation. Here shows a small piece of waveforms:



Red points are selected ones. As a result, interpolation red lines seems to be a good approximation, and the nodal number is greatly reduced.

Write input file in Diana as: (with N*0.005s time interval)

```
'TIMELO'
LOAD 1
TIMES 0.000 0.005… 7.500 /
FACTOR 0.000 …0.000 /
```

Since the unit of acceleration for ITACA waveform is [cm/s2], we have input LOAD 1: Base Excitation value <u>0.01 in X direction</u>.
See Appendix C for waveforms of different magnitudes. (B soil)

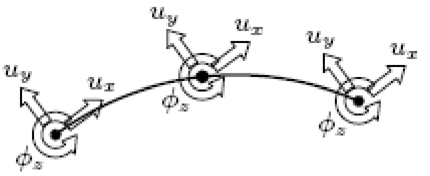
2. Process in Diana

Geometry of the structure, units and model type are the same as in linear case.
Analysis unit: (M, kg, s, N, C); Model Type: !Structural 2D
Define SET1: panel, SET2: columns, SET3: beam

--Choice of Element Type--
Class-III fully numerically integrated *Mindlin* beam elements. These elements may be used in linear and in geometric and physic nonlinear analysis.

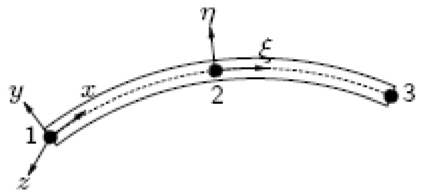
Unlike Classical Beam Elements which are based on Bernoulli theory, Class-III beam element is under the assumption that the cross-section of the beam remains plane but not necessarily perpendicular to the slope of the beam axis. The class-III beam elements are based on the so-called *Mindlin-Reissner* theory which <u>does</u> take shear deformation into account. Another advantage of the class-III beam elements compared to other classes, is that they may be curved due to the fact that they have more than two nodes.

Variables of class-III beam, in 2-D case, we have:



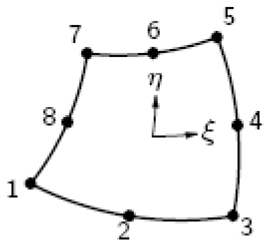
$$\mathbf{u}_e = \begin{Bmatrix} u_x \\ u_y \\ \phi_z \end{Bmatrix}$$

In this analysis case, choose element type <u>**CL9BE**</u> (curved, 3 nodes, 2-D) for 2 columns and beam. Which has 9DOF, and which type belongs to Class- III.



We may apply higher order elements for example CL12B, CL15B, however CL9BE will be sufficient in this case.

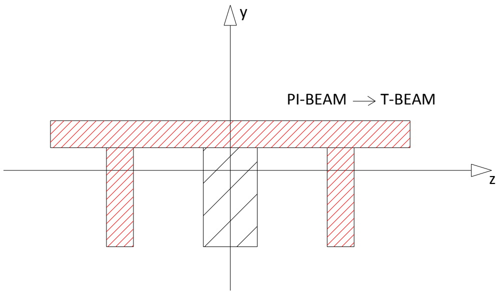
Choice of element type for panel
Non-linear analysis favors high order shell element. Here select <u>**CQ16M**</u> (quadrilateral, 8 nodes) which belongs to plane stress elements class. Out of plane moment/shear are not considered.



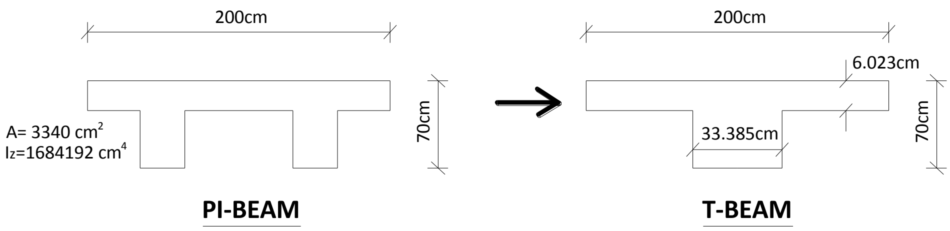
It is an isoparametric plane stress element and is based on quadratic interpolation and Gauss integration. The panel can be considered as “plane stress” because the plate is thin and is characterized by the fact that the stress components perpendicular to the surface are zero.

--Physical Properties--
We can not directly input cross-section area and moment of inertial in non-linear beam element Class-III, because the class-II and class-III beam elements are integrated in the area of cross-section, not only along the bar axis.

To model PI-beam, we have several approaches. For Class-III beam, specify arbitrary cross-sections with a number of quadrilateral zones. However in 2-D case, this methods could be complex. A simplified method is to use 2-D equivalent "T" beam, that is to transfer a "PI-beam" with cross-section properties: -Area: 3340cm^2 -Moment of inertial: 1684192cm^4 into a T-beam.



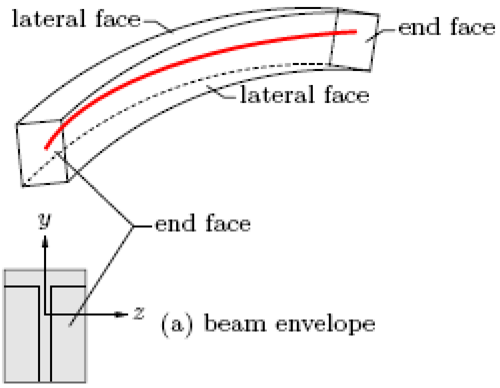
The left and right webs have merges in the middle which yields the same moment of inertial about local axis Z without violating the original section properties including the cross-sections area.

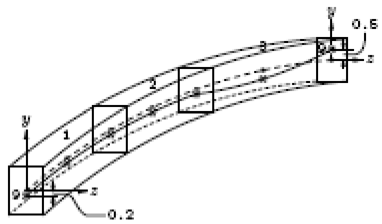


Column sections and thickness of the panel are the same as in linear case.

Take shear stress into account, assuming a constant shear stress along the cross-section. However, in real case, it is expected a non-homogeneous distribution of shear along the section depth. Thus for Beam "Class-III", we have shear stress correction factor (shape factor) Sr=1.2 by default.

--Embedded Reinforcement--
The program will automatically determine the envelope of the beam/column sections and check if the embedded reinforcement is within the envelope of the beam. That means i-Diana will check for compatibility of the rebars with the sections. There are several ways to define a rebar in 2-D case, by means of different patterns of lines: A straight line defined by two end points, A curved one determined by 3 nodes, a third order one defined by 4 points and so on. In our case, since the rebars are straight, we determine several straight lines connecting two end cross-sections of the beam.



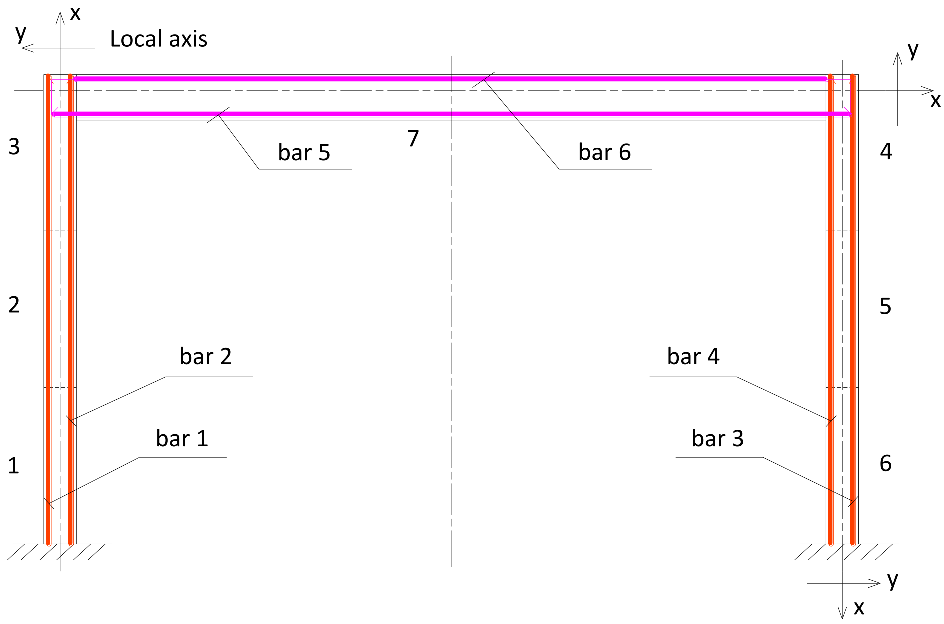


Applying “Element-to-Element” Method which explicitly specify location for the rebars. By defining eccentricity ‘EC’, the software automatically generate a line connecting starting point and end point passing through a series of elements. Inside each element, we have 3 location points which are specified by the program by default. The rebar segments within elements are called particles. Here define two parallel rebars inside the section with positive and negative eccentricities respectively. They are symmetric to the neutral axis x and defined in local y direction.

The input command for embedded rebars is:

```
'REINFO'
LOCATI
        No.       BAR
              LOCALY
               /1-3/    ±0.215
              /4-6/     ±0.215
               /7/         +0.217 -0.413
```

“LOCALY” indicates eccentricity in local y direction. For T section the neutral axis lies in 0.25166m measured from section top. Concrete cover is 35mm. “No.” could be i=1,2,3…6. See figure below: (1,2,…i are elements)

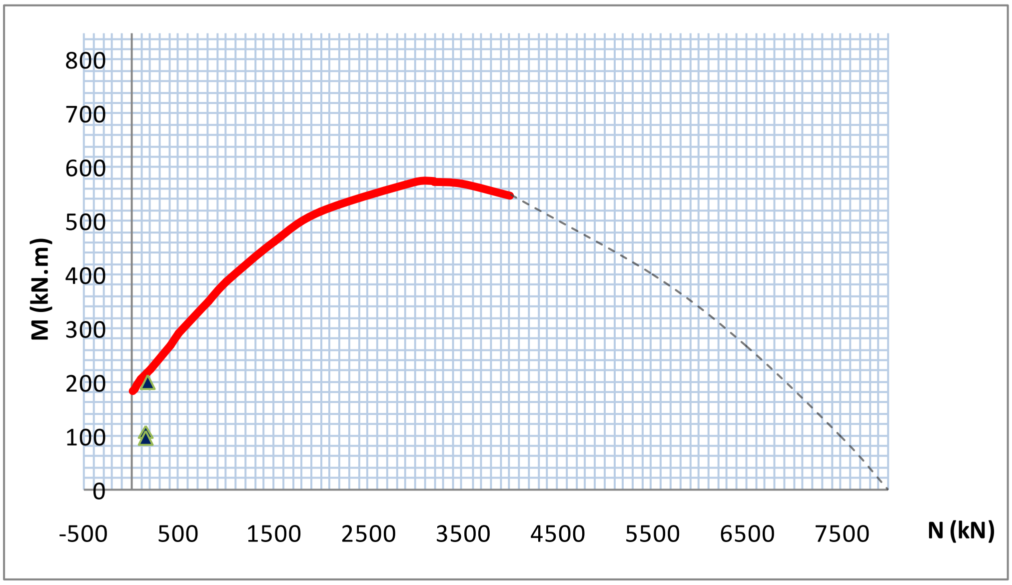


-Design of reinforcement-

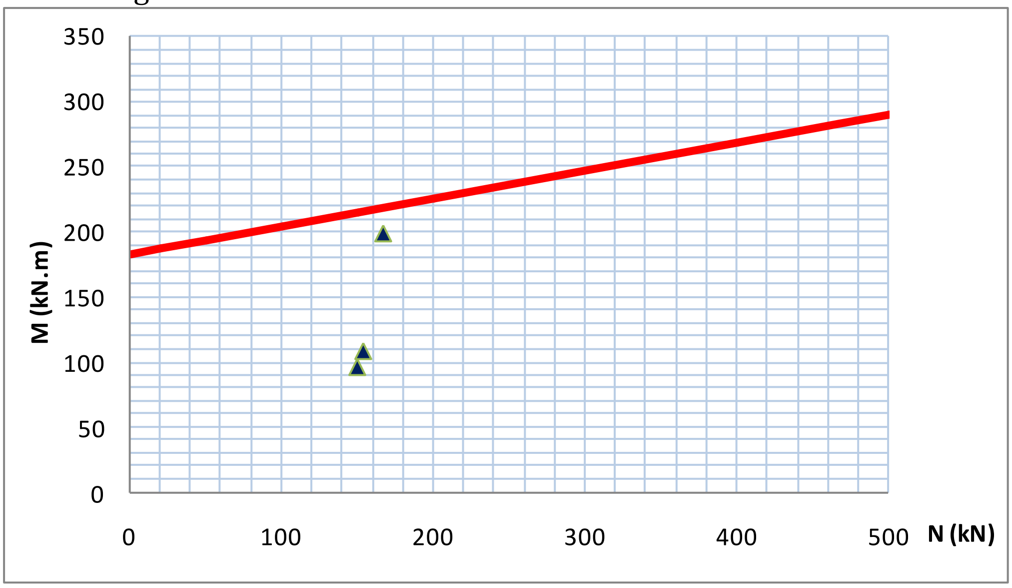
In left-seismic case we have critical sections at 1 & 8 for columns, end sections 4 & 5 for beam. The amount of reinforcement must be carefully chosen because this will be very sensitive to the non-linear analysis result. Here apply a uniform rebar assignment through the entire length of columns and beam.

Run the linear analysis first, find the critical value of bending moment + axial force combination, we obtain design value of internal forces. We get from linear analysis the following critical combination at node 1 or 8.

1. Nx=166.9kN; Mz=198.8kN.m (Case2-0.25g)
2. Nx=153.4kN; Mz=108.4kN.m (Case3-0.25g)
3. Nx=149.2kN; Mz=95.83kN.m (Case4-0.25g)



Zoomed figure



Applying a reinforcement of 4×Φ18 on each side, we get the above locus. Check for force state (M+Nc) for 3 critical combinations. They are all within the locus. Note that gravity force on columns should be always taken into account, because this will largely influence the bending resistance capacity of the sections. As a result:
Reinforcement for Columns: As=As'=1017.876 mm^2 ; Mrd=218.77kN.m>Med=198.8kN.m

Reinforcement of beam can be designed according to the minimum requirement, because beam section is strong and has a critical design moment of 153.2kN.m. Apply 6×Φ14 & 4×Φ14 in tensile zone and compression zone separately. As a result:
Reinforcement for beam: As'=615.752cm^2; As=923.628mm^2 ; Mrd=237.97kN.m>153.2kN.m

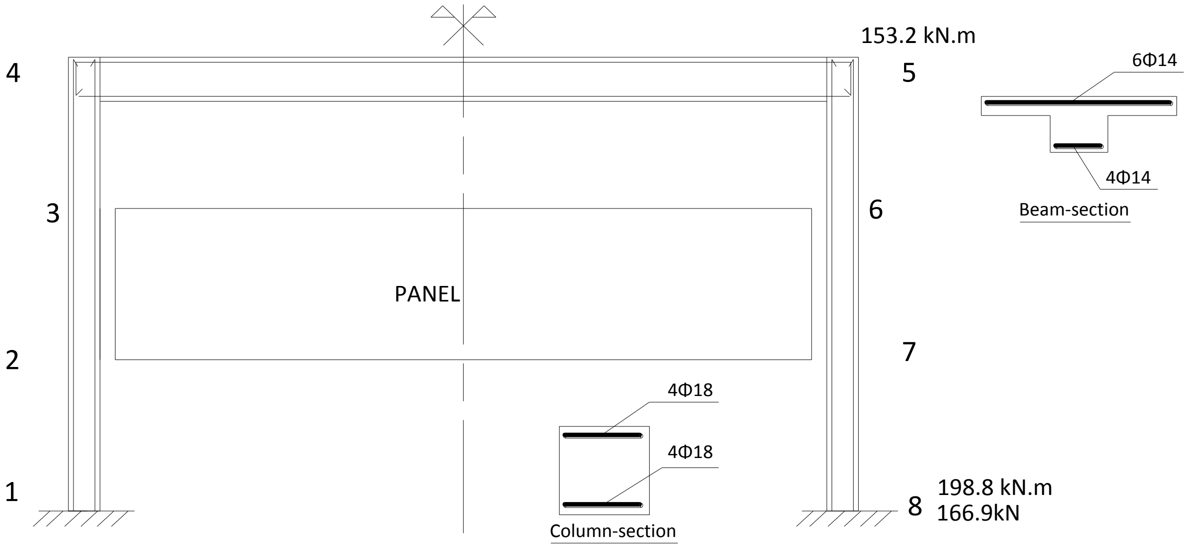
Apply same amount reinforcement along the entire beam/columns. Input rebar area As as physical properties in i-Diana.
For bar 1~4 As=1017.876 mm^2
For bar 5 As=615.752mm^2
For bar 6 As=923.628mm^2

In order to compare with linear case before, the reinforcement chosen above should also fulfill the requirement of DCM defined in EN1998-1-1, because we have been performing the analysis under the assumption of DCM. In linear dynamic method, use a behavior factor q=3.3 in order to take into account the energy dissipation of the structural members in post yielding. Here we explicitly design for reinforcement and directly setting non-linear properties for materials.

Some key requirement for DCM are listed below:
-Principle ones:
1. Concrete of a class lower than C16/20 shall not be used in primary seismic elements.
2. With a few exceptions, only ribbed bars shall be used as reinforcing steel in critical regions of primary seismic elements.
3. Reinforcing steel of Class B-C shall be used in critical regions of primary seismic elements.
-ULS verifications and detailing
1. For primary seismic beam, a region up to $l_{cr} = h_w$ (denotes the depth of the beam) from an end cross-section where the beam frames into a beam-column joint should be considered as being critical regions.
Here, $l_{cr} = 0.7(m)$
2. Ductility requirement for DCM in critical zone of beam is deemed to satisfied, if the following conditions are met:
(a) compression zone reinforcement is no less than half the amount of reinforcement provided in tensile zone
(b) maximum and minimum reinforcement ratio in tensile zone

$$\rho_{min} = 0.5\left(\frac{f_{ctm}}{f_{yk}}\right) < \rho < \rho_{max} = \rho' + \frac{0.0018}{\mu_{\emptyset}\varepsilon_{sy,d}} \cdot \frac{f_{cd}}{f_{yd}}$$

Where $\mu_{\emptyset} = 1 + 2(q_0 - 1)\,T_c/T_1 = 9.939$ (For $T_1 < T_c$)
$\rho_{min} = 4.13 \times 10^{-3} < \rho = 4.16 \times 10^{-3} < \rho_{max} = 8.813 \times 10^{-3}$
ρ and ρ' are reinforcement ratio in tension zone and compression zone
3. critical zone length of columns is defined by the formula below:
$l_{cr} = \max\{h_c;\ l_{cl}/6\,;0.45\}$=1.08(m)
4. For columns need to check for total longitudinal reinforcement ratio for
$0.01 \le \rho_L = 0.0175 \le 0.04$
All the above criteria are fulfilled, so the reinforcement we designed can guarantee a DCM case.


153.2 kN.m
4
5
6Φ14
4Φ14
Beam-section
3
6
PANEL
2
7
4Φ18
4Φ18
1
8
198.8 kN.m
166.9kN
Column-section

--Material Properties--

-Concrete-

Total Strain Crack Model

This constitutive rule include 4 parts: general input; tensile parameters; compression parameters; shear parameters.
Define Young's modulus, Density, Poison Ratio as before in linear case.
For crack type, select "Rotate Crack" , assuming the direction of cracks keep rotating with principle direction of the strain vector.

<u>Viscous Damping</u>

Rayleigh damping factor a & b (constant values)
ξ=0.05 for concrete.

$a = 2w_1 w_2 \beta; \quad b = 2\beta; \quad \alpha = \frac{w_1}{w_2} \quad \beta = \frac{(1-\alpha)\xi}{w_2 - \alpha w_1}$

w_1, w_2 from linear analysis

Case1) $f_1 = 3.89E+00[Hz]$; $f_2 = 1.11E+01[Hz]$ → a=1.81E+00; b=1.06E-03
Case2) $f_1 = 3.88E+00[Hz]$; $f_2 = 1.02E+01[Hz]$ → a=1.77E+00; b=1.13E-03
Case3) $f_1 = 2.25E+00[Hz]$; $f_2 = 8.18E+00[Hz]$ → a=1.11E+00; b=1.53E-03
Case4) $f_1 = 2.25E+00[Hz]$; $f_2 = 8.18E+00[Hz]$ → a=1.11E+00; b=1.53E-03

```
--------------------------------------------------------------------------------
'MATERIALS'
…
i (Material Number)
RAYLEI     1.110000E+00     1.530000E-03
…
--------------------------------------------------------------------------------
```

<u>Tensile</u>

To simplify, choose brittle failure criteria. Use characteristic value here, Diana will automatically consider a safety factor. Hence we have TEENSTR=2.66 Mpa

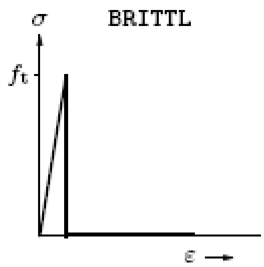
<u>Shear</u>

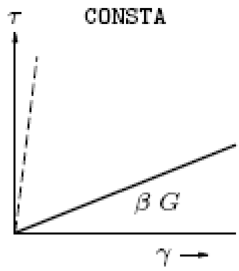
For constant shear retention, common choice of factor β=0.01

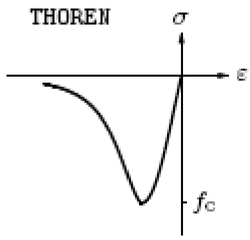
<u>Compression</u>

"Thoranfeldt" constitutive model fits well in our case, choose COMSTR=45 Mpa for C45/55.

As a result, we have defined all the strength and hardening behaviour for concrete, see plot below. Use characteristic value directly in non-linear analysis.







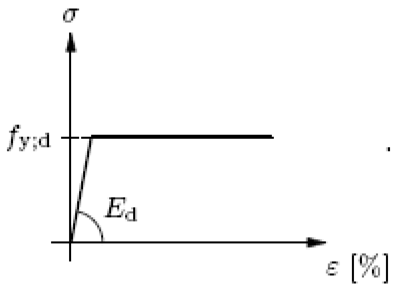
Consider the relative short duration of the earthquake waveform, creep/shrinkage and other long-term effect are not taken into account.

```
'MATERIALS'
YOUNG       3.820000E+10
  POISON    1.500000E-01
  DENSIT    2.548400E+03
  TOTCRK ROTATE
  TENCRV BRITTL
  TENSTR 2.66E+06
  SHRCRV CONSTA
  BETA 0.01
  COMCRV THOREN
  COMSTR 4.5E+07
```

-Reinforcing Steel-

For steel reinforcing bars, we may apply Von-Mises criteria. Characteristic value of yielding strength (both compression & tensile) f_{yk}=460 Mpa. Diana considers a reduction factor γ_s=1.15.

Assume a elastic-perfect plastic constitutive rule with elastic modulus E=200Gpa.



```
'MATERIALS'
YOUNG 2.000000E+11
YIELD VMISES
YLDVAL 4.60000E+08
HARNEN STRAIN
HARDIA 4.6E+08 0 4.6E+08 0.0015 4.6E+08 0.05
```

All steps in the pre-processor i-Diana has done, generate a input file, see appendix D.

--Writing Command File--

Start with common heading “NONLIN”, in “EXECUT” modulus, define automatic time increment having a range of (0.0001s~0.002s), and time steps number about a thousand so that to reach a time duration of 7.5s. According to some documentations, time increment Δt, and total time period t_{end} should satisfy the following:

$\Delta t \approx \frac{1}{20} T_i$; $t_{end} > 2T_i$

Theoretically, the time increment of each step should be approximately 1/20 of the free vibration period of the highest mode (consider 2nd mode, other modes are not dominating) in order to take into account the contribution of the high modes. The end time should be at least 2 times the free vibration period of the primary mode (1st mode) so that the period can be repeated at least twice and the result is considered reliable. In our case, choose time increment (0.0001s~0.002s) and effective duration 7.5s fits perfectly the above criteria.

Iteration rule: Newton-Regular

Check tolerance for Displacement and Force both: TOLCON=1.0E-02 relative to “norms”, and with a maximum iteration size 35.

Define in modulus “PHYSIC” rebar bonding condition as bond or not bonded. This is quite important part of the model analysis, bonding condition fits well when cracks are not formed. After crack formation, the plane section assumption can hardly hold true in numerical analysis, as a result, slip condition can be assumed. However, in Diana, under slip option, change of strain is uniformly applied to the whole reinforcement . Since there are friction and interlock between rebars and its mother element---concrete due to ribbed bars, a “BOND” condition should be applied.

Command file is as below:

```
*NONLIN
MODEL
BEGIN TYPE
TRANSI DYNAMI
END TYPE

BEGIN EXECUT
BEGIN PHYSIC
BOND REINFO    15 18 19 20 16 17
END PHYSIC
BEGIN TIME
BEGIN STEPS
  BEGIN AUTOMA
  SIZE=7.500
  MAXSIZ=0.002
  MINSIZ=0.0001
  END AUTOMA
END STEPS
END TIME
BEGIN ITERAT
MAXITE=35
METHOD NEWTON REGULA
FORCE CONTIN TOLCON=1.0E-02
DISPLA CONTIN TOLCON=1.0E-02
```

```
END ITERAT

BEGIN OUTPUT TABULAR
BEGIN SELECT
STEPS LAST MAX
NODES 13 25 37 38 50 62 1 74 116 188 196 124
END SELECT
DISPLA TOTAL TRANSL GLOBAL X Y
END OUTPUT
BEGIN OUTPUT TABULAR
BEGIN SELECT
STEPS LAST MIN
NODES 13 25 37 38 50 62 1 74 116 188 196 124
END SELECT
DISPLA TOTAL TRANSL GLOBAL X Y
END OUTPUT
BEGIN OUTPUT TABULAR
BEGIN SELECT
STEPS LAST MAX
ELEMEN 1 6 7 12 13 18 19 24 25 30 31 36
END SELECT
STRESS FORCE X Y NODES
STRESS MOMENT Z NODES
END OUTPUT
BEGIN OUTPUT TABULAR
BEGIN SELECT
STEPS LAST MIN
ELEMEN 1 6 7 12 13 18 19 24 25 30 31 36
END SELECT
STRESS FORCE X Y NODES
STRESS MOMENT Z NODES
END OUTPUT
END EXECUT
*END
```

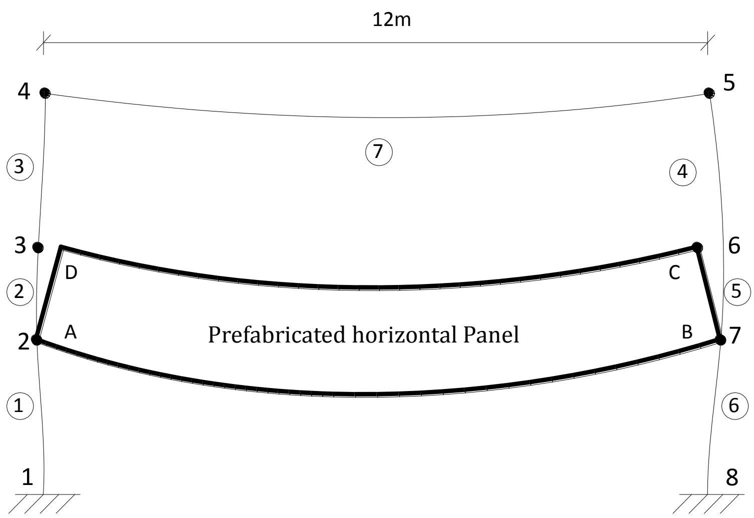
---

3. Consideration of Gravity Load

Linear static analysis has showed that: even under simply gravity load, the panel has certain magnitude of expansion. That means, in a two pined panel-column joints model (Case1), the expansion of bottom edge of the panel will push against the column and thus cause the shear force and bending moment in columns.

Since we are interested in the effect of seismic lateral force acting on panel, not the phenomena of panel deformation itself, besides, the shear forces caused by bottom edge elongation are symmetric which have no influence on seismic lateral force acting on non-structural panel. Certain approach should be taken to minimize the expansion effect. One of the most effective way is to set a constrain in the bottom edge of the panel, that is define a master node in the column joint which directly connected to the panel, then set a series of slave panel joints having the same displacement as master one in Global X direction. By adding this extra constrain. We are able to minimize the unfavorable effect to an acceptable level.

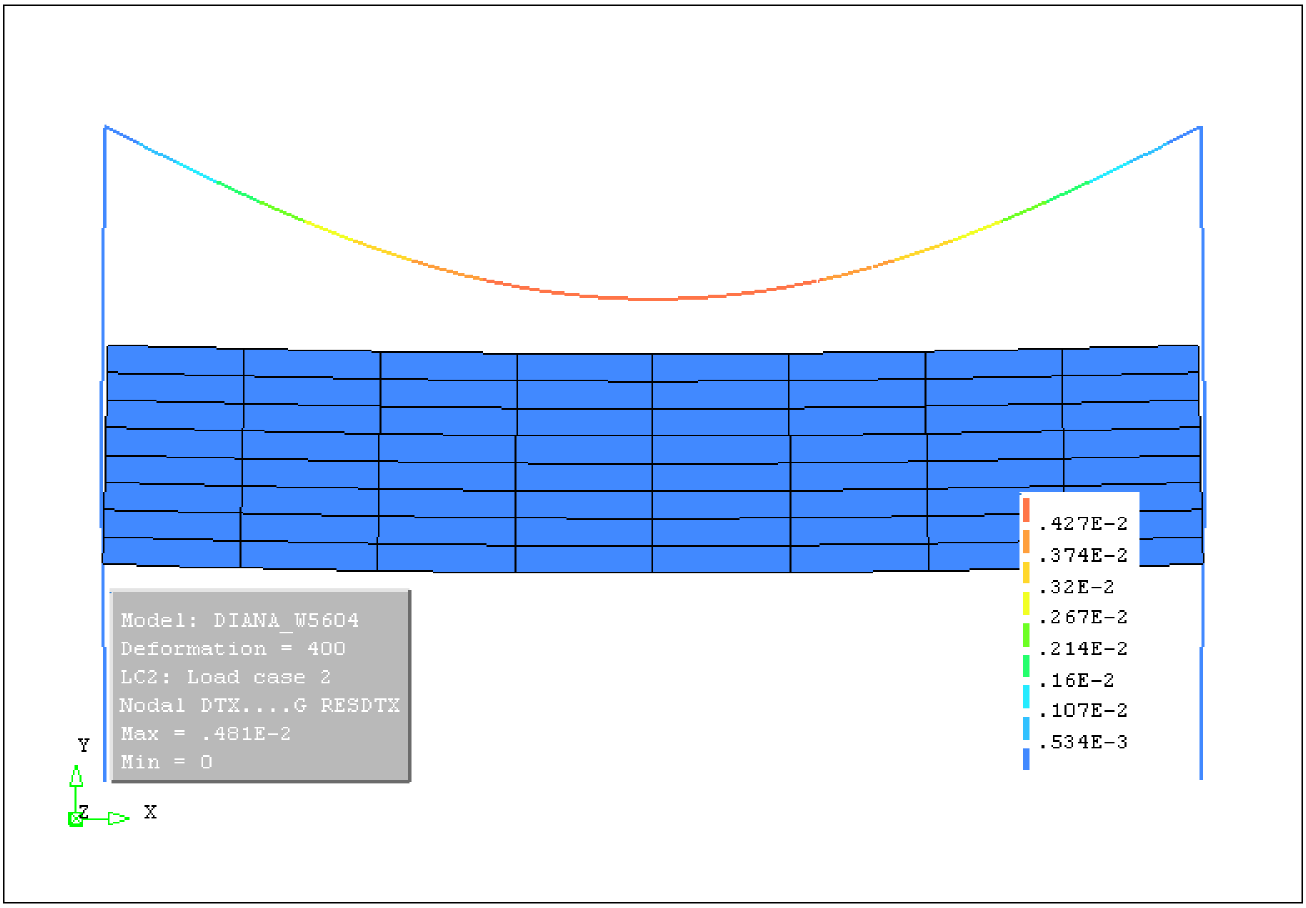
Comparison in following table shows the significant improvement after adding the constrains to the panel and also the top beam.

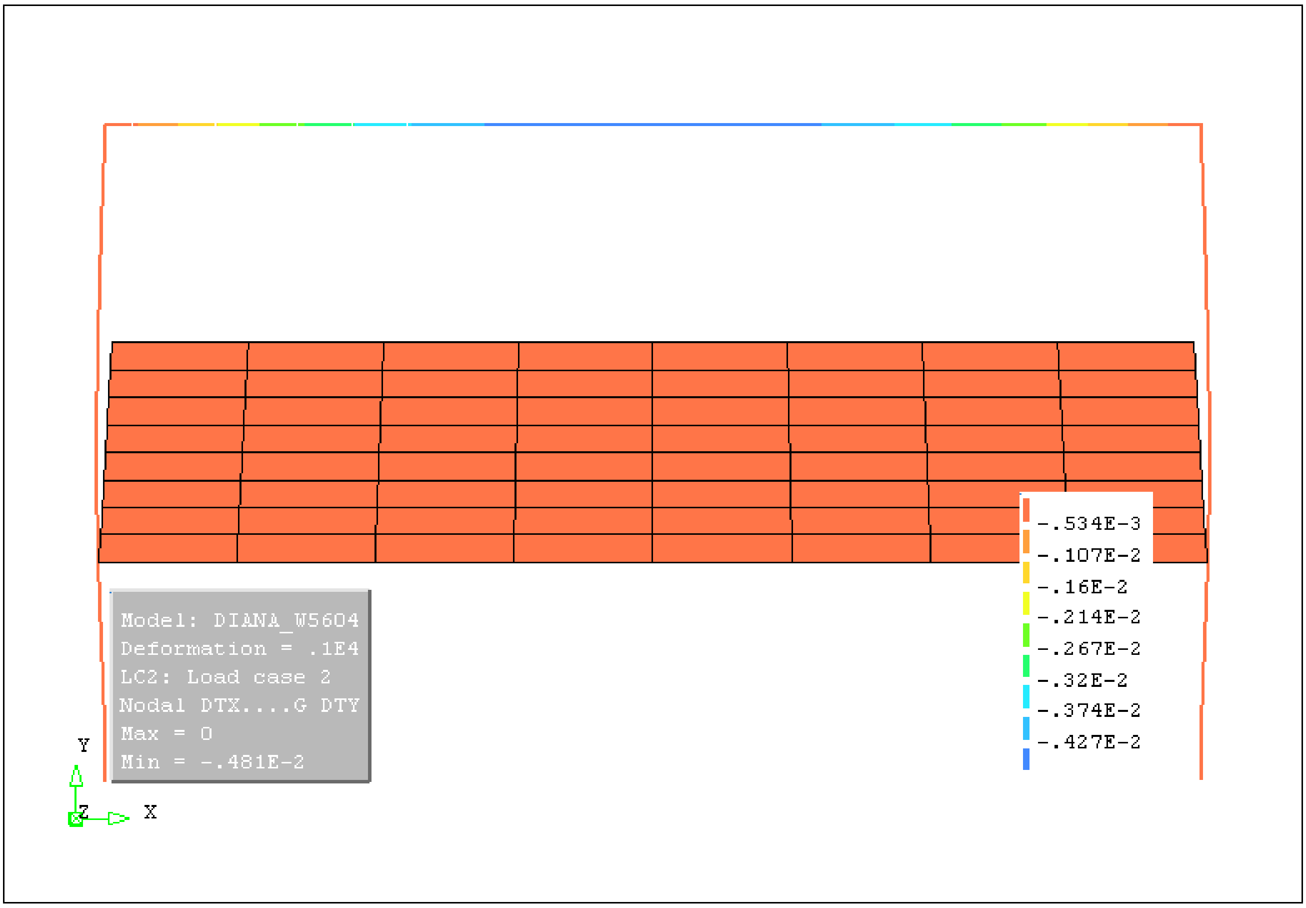


Nodal and Element Convention

B.C: Beam with 2 Pinned Ends under Gravity Force (Case1)

Without Constrain						
Elmnr	Nodnr	Nx	Qy	Mz	DX	DY
1	1	-1.690E+05	6.718E+03	-1.049E+04		
	2	-1.540E+05	6.718E+03	5.631E+03	-7.875E-05	-4.058E-05
2	2	-1.061E+05	-1.173E+03			
	3	-9.114E+04	-1.173E+03	2.815E+03	-8.040E-05	-6.141E-05
3	3	-8.138E+04	-1.173E+03			
	4	-6.638E+04	-1.173E+03	3.038E-10	-5.516E-07	-8.394E-05
With Constrain						
1	1	-1.690E+05	3.497E-10	1.630E-08		
	2	-1.540E+05	3.497E-10	1.714E-08	2.191E-16	-3.892E-05
2	2	-1.008E+05	-3.572E-09			
	3	-8.578E+04	-3.572E-09	8.568E-09	8.527E-16	-6.141E-05
3	3	-8.138E+04	-3.572E-09			
	4	-6.638E+04	-3.572E-09	1.937E-12	1.713E-15	-7.921E-05


Model: DIANA_W5604
Deformation = 400
LC2: Load case 2
Nodal DTX....G RESDTX
Max = .481E-2
Min = 0
.427E-2
.374E-2
.32E-2
.267E-2
.214E-2
.16E-2
.107E-2
.534E-3
Y
Z
X


Model: DIANA_W5604
Deformation = .1E4
LC2: Load case 2
Nodal DTX....G DTY
Max = 0
Min = -.481E-2
-.534E-3
-.107E-2
-.16E-2
-.214E-2
-.267E-2
-.32E-2
-.374E-2
-.427E-2
Y
Z
X

Plot from i-Diana post processor clearly shows a expanding behavior of the panel under gravity loading only, and that's probably because the structure has a beam and panel of 12m span.

Notice that the shear force caused by dilatants has a value around 7.0E+03 N for each column, but the lateral force acting on the panel "Fa" in a low PGA case is just a few kilo-Newtons. In such case, the behavior of the system under earthquake load is not dominated by the seismic action but by the gravity, and as a result, the maximum response will not achieved around peak ground acceleration point. This will bring about undesirable result and non-convergence in non-linear calculation. In case of fixed end of the beam, this phenomena is reasonably small, thus we can release the constrain anyway.

Applying "Tyings" to panel bottom line and to the beam

```
-----------------------------------------------------------------------------------------------------------------------------------
'TYINGS'
EQUAL TR 1
    / 75-115 38 / 37 (Left column joint to the beam)
EQUAL TR 1
    / 116 125 134 143 152 161 170 179 188 205 222 239 256 273 290 307 324 62 / 13 (Left column joint to the panel)
-----------------------------------------------------------------------------------------------------------------------------------
```

*ONLY for CASE1

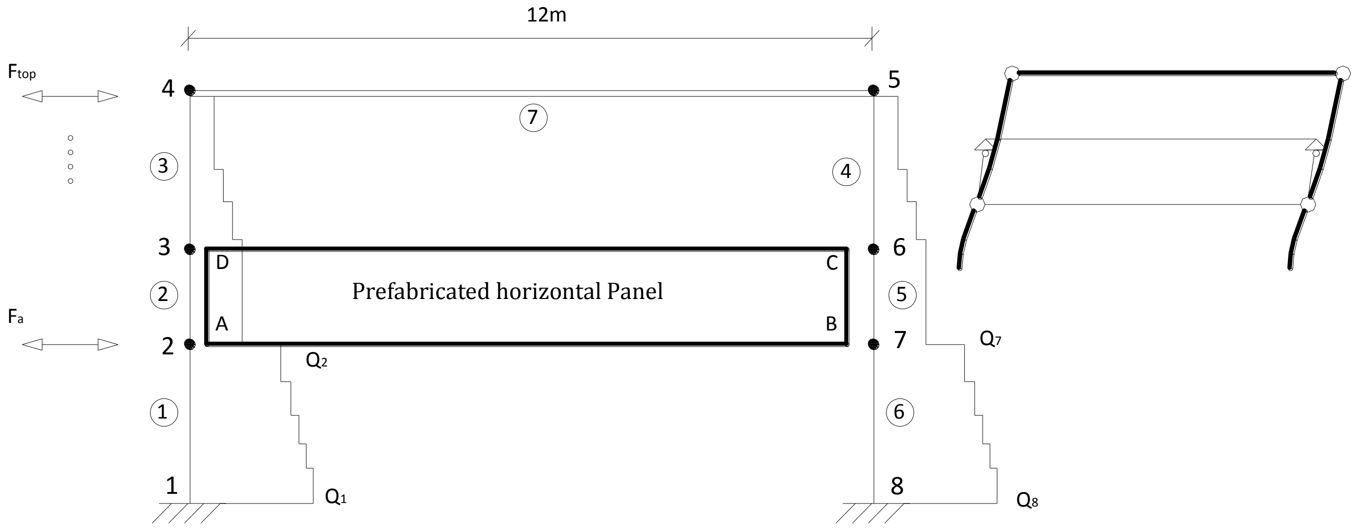
Combine Deadweight in Time-history Analysis:
In a non-linear analysis, principle of superposition is longer used. Combine gravity load and seismic waveform in time-history. Gravity force is 0 at initiation, keep increasing for a few time step to its full value 9.81kN/m2. This will allow Diana iteration process to predict a reasonable stiffness value k at initial time steps so that reduce the risk of iteration failure.

```
-----------------------------------------------------------------------------------------------------------------------------------
'LOADS'
CASE 2
WEIGHT
2       -9.81
LOAD 2
TIMES 0 0.15 7.5 /
FACTOR 0 1 1 /
-----------------------------------------------------------------------------------------------------------------------------------
```

The axial force in column will greatly influence the moment resistance at ULS as well as the yielding of the section. As a result, the deadweight in a non-linear analysis should always be present. In Case3 and Case4 (fixed beam ends), the deadweight will also cause moment and shear in the columns, so, certain calculation method should be used to separate the "shear" transmitted to panel by ground excitation from the one by gravity load.

4. Results of Linear Dynamic Analysis

For Case1, Ag=0.25g (For other magnitude of Ag, see the comparison curve)



--Internal Force--

Only Seismic Load (Abs_Max Nodal Internal Forces) [N,m]

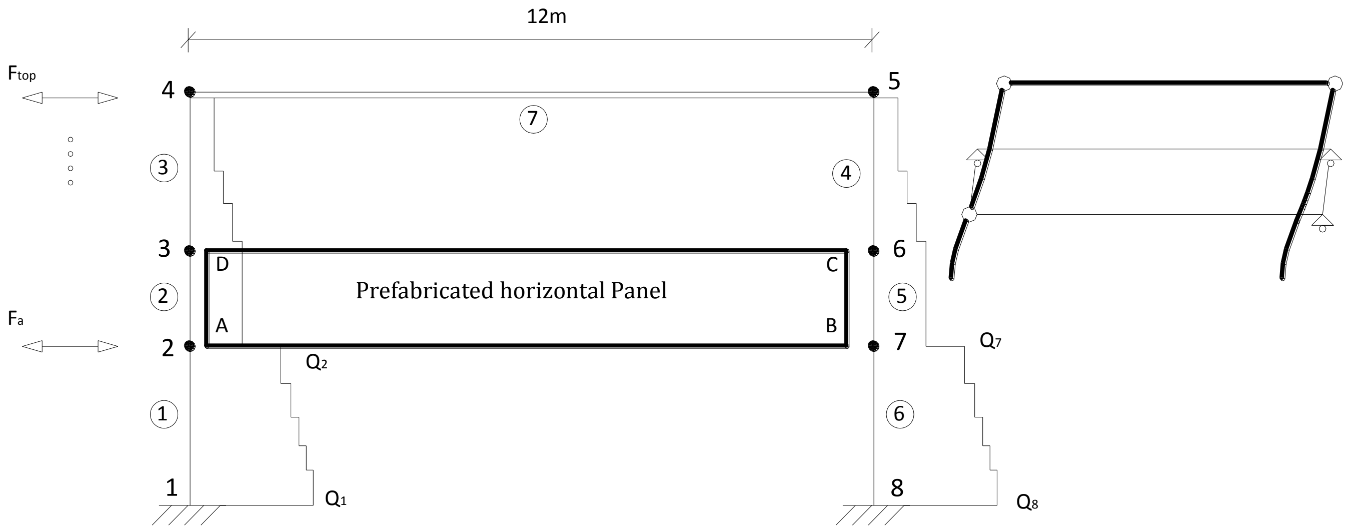
ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.373E+03	3.681E+04	1.81E+05			
		2	2.366E+03	3.559E+04	1.11E+05			
	2	2	1.614E+03	2.364E+04	1.11E+05	7.520E+02	1.195E+04	1.589E+01
		3	1.608E+03	2.364E+04	5.53E+04			
	3	3	1.24E+02	2.37E+04	5.53E+04			
		4	9.42E+01	2.15E+04	2.43E+01			
	4	5	9.42E+01	2.15E+04	2.43E+01			
		6	1.24E+02	2.37E+04	5.53E+04			
	5	6	1.608E+03	2.364E+04	5.53E+04			
		7	1.614E+03	2.364E+04	1.11E+05			
	6	7	2.366E+03	3.559E+04	1.11E+05	7.520E+02	1.195E+04	1.589E+01
		8	2.373E+03	3.681E+04	1.81E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.915E+03	2.793E+04	1.64E+05			
		2	1.911E+03	2.735E+04	1.02E+05			
	2	2	1.278E+03	2.293E+04	1.02E+05	6.330E+02	4.420E+03	6.983E+00
		3	1.283E+03	2.167E+04	4.89E+04			
	3	3	1.15E+02	2.17E+04	4.89E+04			
		4	8.81E+01	1.87E+04	2.23E+01			
	4	5	8.81E+01	1.87E+04	2.23E+01			
		6	1.15E+02	2.17E+04	4.89E+04			
	5	6	1.283E+03	2.167E+04	4.89E+04			
		7	1.278E+03	2.293E+04	1.02E+05			
	6	7	1.911E+03	2.735E+04	1.02E+05	6.330E+02	4.420E+03	6.983E+00
		8	1.915E+03	2.793E+04	1.64E+05			

As a result: [N]

Base shear
ABS: Qy(1)+Qy(8)= 7.362E+04
SRSS: Qy(1)+Qy(8)= 5.586E+04

Fa = ΣΔQ (Symmetric)
ABS: ΔQ(2)×2= 2.390E+04
SRSS: ΔQ(2)×2= 8.840E+03

For Case2, Ag=0.25g (For other magnitude of Ag, see the comparison curve)



--Internal Force--

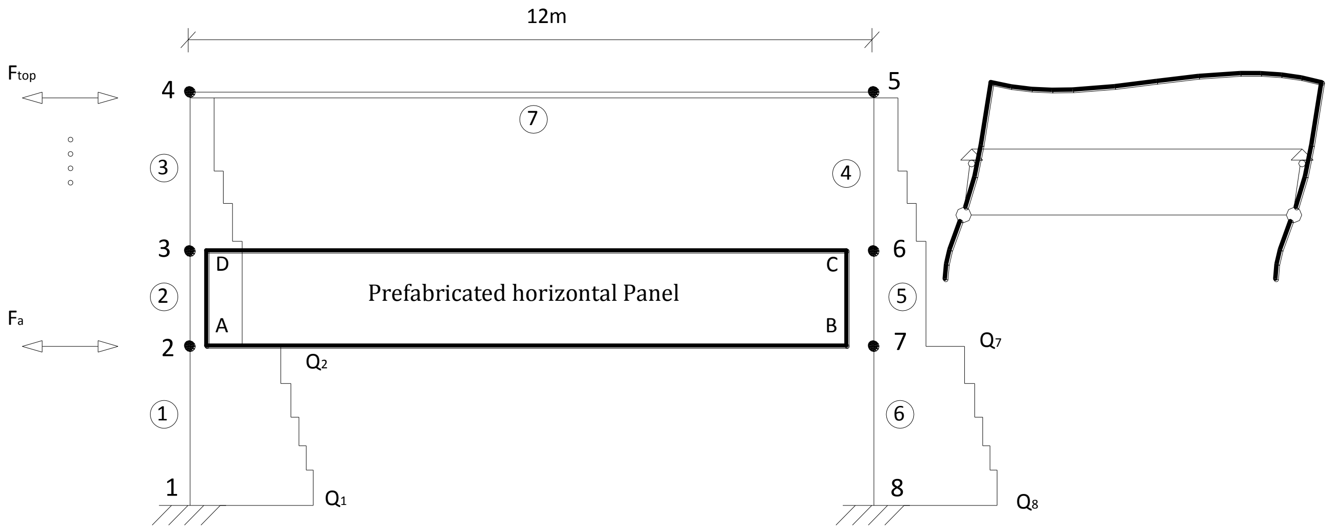
Only Seismic Load (Abs_Max Nodal Internal Forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	2.124E+03	4.811E+04	1.99E+05			
		2	2.109E+03	4.686E+04	1.17E+05			
	2	2	4.814E+03	2.355E+04	1.17E+05	2.705E+03	2.331E+04	8.617E+00
		3	4.775E+03	2.492E+04	5.82E+04			
	3	3	1.38E+02	2.49E+04	5.82E+04			
		4	9.35E+01	2.29E+04	2.43E+01			
	4	5	8.83E+01	2.09E+04	2.56E+01			
		6	1.30E+02	2.25E+04	5.27E+04			
	5	6	1.506E+03	2.245E+04	5.27E+04			
		7	1.551E+03	2.555E+04	1.05E+05			
	6	7	3.504E+03	2.555E+04	1.05E+05	1.953E+03	0.000E+00	0.000E+00
		8	3.522E+03	2.649E+04	1.66E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.412E+03	3.434E+04	1.70E+05			
		2	1.409E+03	3.357E+04	1.02E+05			
	2	2	3.813E+03	2.260E+04	1.02E+05	2.404E+03	1.097E+04	4.563E+00
		3	3.811E+03	2.153E+04	4.87E+04			
	3	3	8.67E+01	2.15E+04	4.87E+04			
		4	4.88E+01	1.87E+04	2.22E+01			
	4	5	4.74E+01	1.89E+04	2.24E+01			
		6	8.64E+01	2.20E+04	4.94E+04			
	5	6	1.135E+03	2.196E+04	4.94E+04			
		7	1.146E+03	2.338E+04	1.04E+05			
	6	7	2.705E+03	2.338E+04	1.04E+05	1.559E+03	0.000E+00	0.000E+00
		8	2.710E+03	2.366E+04	1.60E+05			

As a result: [N]

Base shear		Fa = ΣΔQ	
ABS:	Qy(1)+Qy(8)= 7.460E+04	ABS:	ΔQ(2)+ΔQ(7)= 2.331E+04
SRSS:	Qy(1)+Qy(8)= 5.800E+04	SRSS:	ΔQ(2)+ΔQ(7)= 1.097E+04

For Case3, Ag=0.25g (For other magnitude of Ag, see the comparison curve)



--Internal Force--

Only Seismic Load (Abs_Max Nodal Internal Forces) [N,m]

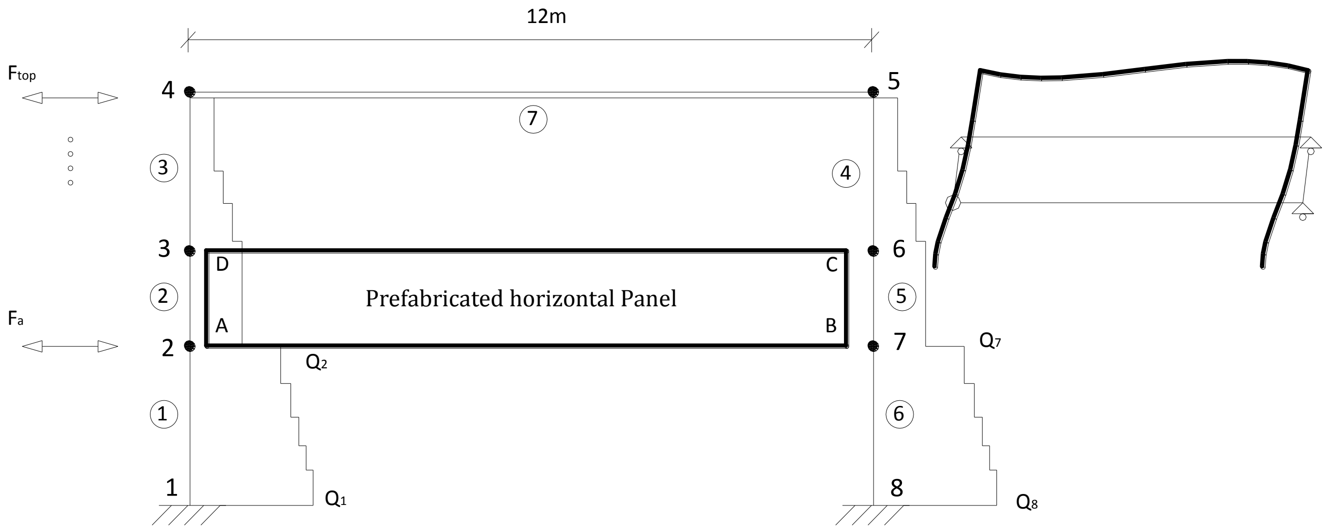
ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.563E+04	3.728E+04	1.13E+05			
		2	1.561E+04	3.598E+04	3.96E+04			
	2	2	1.161E+04	2.546E+04	3.96E+04	4.000E+03	1.052E+04	2.630E+00
		3	1.157E+04	2.550E+04	2.73E+04			
	3	3	1.33E+04	2.55E+04	2.73E+04			
		4	1.32E+04	2.20E+04	7.98E+04			
	4	5	1.32E+04	2.20E+04	7.98E+04			
		6	1.33E+04	2.55E+04	2.73E+04			
	5	6	1.157E+04	2.550E+04	2.73E+04			
		7	1.161E+04	2.546E+04	3.96E+04			
	6	7	1.561E+04	3.598E+04	3.96E+04	4.000E+03	1.052E+04	2.630E+00
		8	1.563E+04	3.728E+04	1.13E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.297E+04	2.990E+04	1.02E+05			
		2	1.296E+04	2.931E+04	3.29E+04			
	2	2	9.789E+03	2.442E+04	3.29E+04	3.171E+03	4.890E+03	1.542E+00
		3	9.783E+03	2.261E+04	2.46E+04			
	3	3	1.20E+04	2.26E+04	2.46E+04			
		4	1.20E+04	1.90E+04	7.43E+04			
	4	5	1.20E+04	1.90E+04	7.43E+04			
		6	1.20E+04	2.26E+04	2.46E+04			
	5	6	9.783E+03	2.261E+04	2.46E+04			
		7	9.789E+03	2.442E+04	3.29E+04			
	6	7	1.296E+04	2.931E+04	3.29E+04	3.171E+03	4.890E+03	1.542E+00
		8	1.297E+04	2.990E+04	1.02E+05			

As a result: [N]

Base shear
ABS: Qy(1)+Qy(8)= 7.456E+04
SRSS: Qy(1)+Qy(8)= 5.980E+04

Fa = ΣΔQ(Symmetric)
ABS: ΔQ(2)×2= 2.104E+04
SRSS: ΔQ(2)×2= 9.780E+03

For Case4, Ag=0.25g (For other magnitude of Ag, see the comparison curve)



--Internal Force--

Only Seismic Load (Abs_Max Nodal Internal Forces) [N,m]

ABS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.984E+04	4.782E+04	1.29E+05			
		2	1.978E+04	4.654E+04	4.32E+04			
	2	2	1.791E+04	2.782E+04	4.32E+04	1.870E+03	1.872E+04	1.001E+01
		3	1.775E+04	2.709E+04	3.03E+04			
	3	3	1.88E+04	2.71E+04	3.03E+04			
		4	1.86E+04	2.35E+04	8.64E+04			
	4	5	1.76E+04	2.17E+04	8.08E+04			
		6	1.78E+04	2.45E+04	2.71E+04			
	5	6	1.631E+04	2.454E+04	2.71E+04			
		7	1.646E+04	2.674E+04	3.61E+04			
	6	7	2.073E+04	2.674E+04	3.61E+04	4.270E+03	0.000E+00	0.000E+00
		8	2.079E+04	2.792E+04	1.00E+05			
SRSS	Elmnr	Nodnr	Nx	Qy	Mz	ΔN	ΔQ	ΔQ/ΔN
	1	1	1.344E+04	3.508E+04	1.08E+05			
		2	1.344E+04	3.442E+04	3.18E+04			
	2	2	1.076E+04	2.390E+04	3.18E+04	2.680E+03	1.052E+04	3.925E+00
		3	1.074E+04	2.223E+04	2.58E+04			
	3	3	1.27E+04	2.22E+04	2.58E+04			
		4	1.27E+04	1.87E+04	7.41E+04			
	4	5	1.24E+04	1.96E+04	7.54E+04			
		6	1.24E+04	2.33E+04	2.37E+04			
	5	6	1.112E+04	2.331E+04	2.37E+04			
		7	1.114E+04	2.526E+04	3.51E+04			
	6	7	1.387E+04	2.526E+04	3.51E+04	2.730E+03	0.000E+00	0.000E+00
		8	1.387E+04	2.563E+04	9.63E+04			

As a result: [N]

Base shear

ABS: Qy(1)+Qy(8)= 7.574E+04

SRSS: Qy(1)+Qy(8)= 6.071E+04

Fa = ΣΔQ

ABS: ΔQ(2)+ΔQ(7)= 1.872E+04

SRSS: ΔQ(2)+ΔQ(7)= 1.052E+04

5. Results of Non-Linear Dynamic Analysis

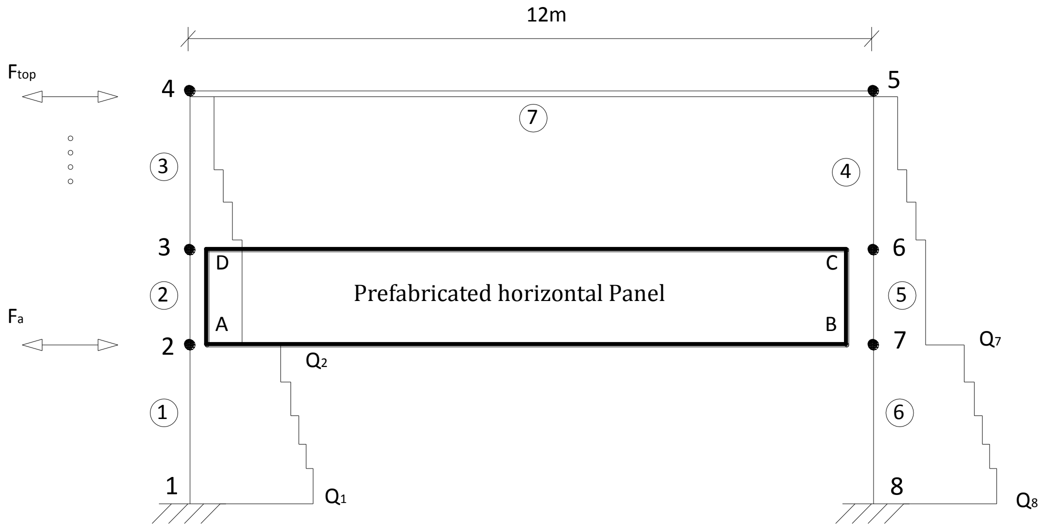
--Determining Critical Time Steps--

For different time-acceleration curves we need to find out the critical time step, that is the exact time point when the structure reaches its maximum response, or more specifically in our case: the Fa value acting on external wall panel. Request in Diana output file “STRESS FORCE X NODES” for important nodes (beam-column joints, column-panel joints), also the “STRESS TOTAL CAUCHY GLOBAL XX NODES” at panel nodes connected to columns.

Selection Criteria

Since hundreds of time steps involves in the analysis, it’s difficult to check one step by one step, we should alternatively choose several suspected ones. Select time steps according to the maximum (minimum) values of the above forces and stress, and we get a series of potential critical time steps.

Run the analysis again, and explicitly specify internal nodal force at the selected time steps, then compare ΔQ (at node 2 and 7 in the figure below) at different time step. Select the peak value of ΔQ, and this is the maximum Fa transmitted to panel during a real seismic event.

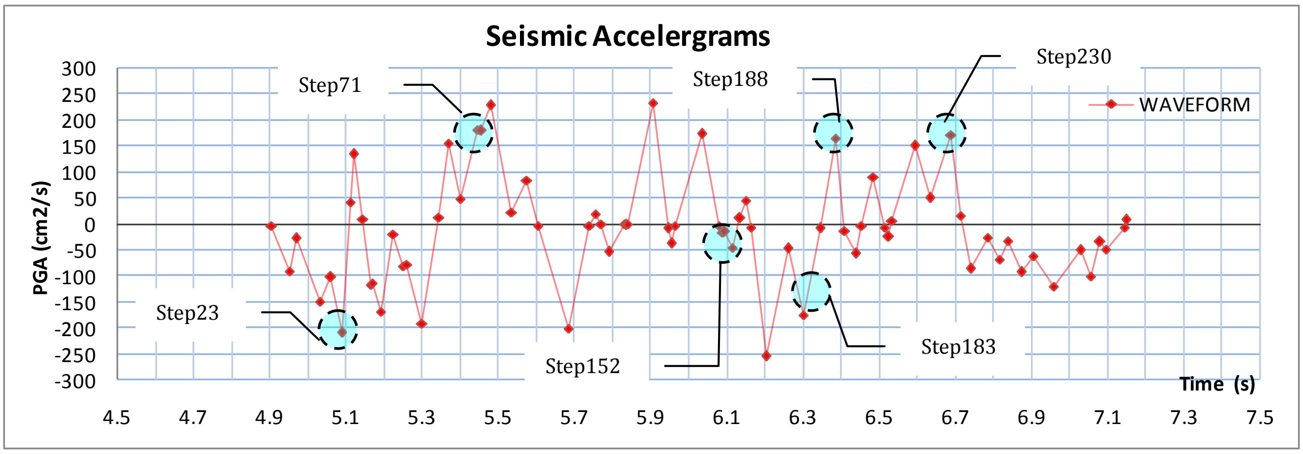


Here shows an example of calculation for Fa in Case1, Ag=0.25g waveform.

Select from output file the following critical time steps

Elmnr	Nodnr	Nx	Qy
(Extreme results maximum)			
1	2	-5.921E+03 (1)	4.990E+04 (230)
2	2	-3.516E+03 (1)	3.330E+04 (152)
(Extreme results minimum)			
1	2	-1.682E+05 (228)	-4.457E+04 (183)
2	2	-1.414E+05 (188)	-3.438E+04 (188)
Elmnr	**Nodnr**	**SXX**	**SYY**
Panel	A	-1.283E+05 (71)	-4.401E+05 (71)
Panel	B	-1.306E+05 (23)	-4.484E+05 (23)

Get time steps 230; 152; 183; 188; 71; 23 and compare” ΔQ” achieved at those times.



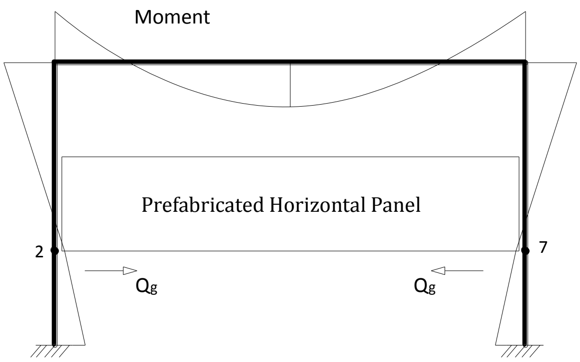
CASE1 0.25g	step 23	Qy	Real Time	step 152	Qy	Real Time
	2	-2.37E+03	5.09E+00	2	3.52E+04	6.09E+00
	2	-2.46E+03	**Fa=ΔQ**	2	3.33E+04	**Fa=ΔQ**
	7	-2.46E+03	**2.14E+04**	7	3.33E+04	**3.68E+03**
	7	-2.37E+04		7	3.51E+04	
	step 71	Qy	Real Time	step 183	Qy	Real Time
	2	2.01E+04	5.46E+00	2	-4.46E+04	6.33E+00
	2	1.08E+04	**Fa=ΔQ**	2	-3.17E+04	**Fa=ΔQ**
	7	1.08E+04	**1.86E+04**	7	-3.17E+04	**2.61E+04**
	7	2.00E+04		7	-4.50E+04	
	step 188	Qy	Real Time	step 230	Qy	Real Time
	2	-3.21E+04	6.37E+00	2	4.99E+04	6.69E+00
	2	-3.44E+04	**Fa=ΔQ**	2	2.60E+04	**Fa=ΔQ**
	7	-3.44E+04	**4.71E+03**	7	2.60E+04	**4.72E+04**
	7	-3.20E+04		7	4.93E+04	

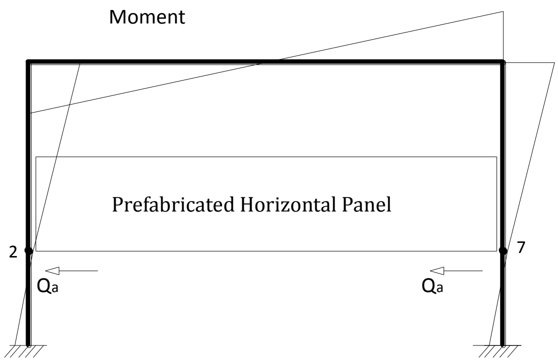
* ΣΔQ=|ΔQ(2)|+ |ΔQ(7)|= Fa

Using the formula above, we get step230 the critical one. We should choose the absolute maximum value of lateral seismic force through the whole time duration. In real case, under the specific seismic even, it's expected a peak value of Fa equal to 47.2kN in this case. In a symmetric condition, the lateral seismic force acting on each column node could be 23.6kN.

Here shows another example of calculation for Fa in Case3, Ag=0.25g waveform.

In Case3 and Case4, with a fixed beam-column connection. We have shear force also under gravity load, thus need to separate the shear force caused by seismic action from the shear force caused by gravity load.

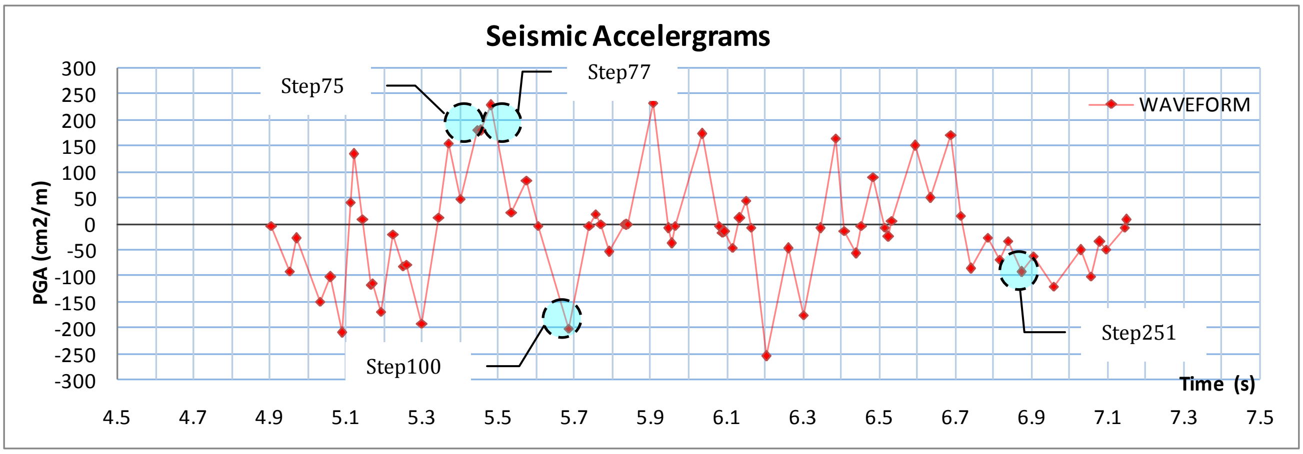




We can see that shear force exerted by external wall panel is symmetric under gravity load and skew-symmetric under seismic action. So, calculate Fa using the correlation formula below:

$\Delta Q_2 = Q_a - Q_g;\ \Delta Q_7 = Q_a + Q_g;\quad \Delta Q_2 + \Delta Q_7 = 2Q_a = F_a$

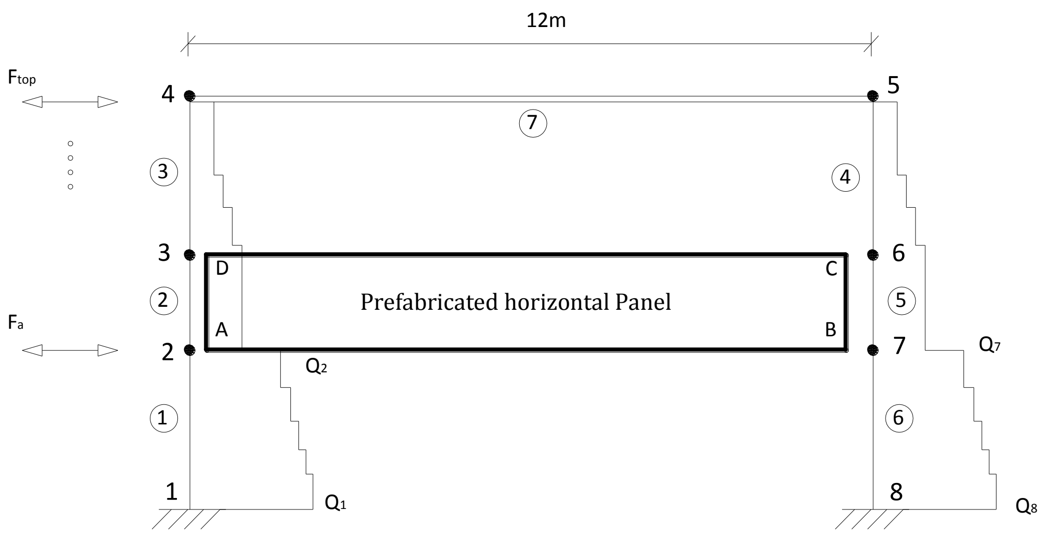
ΔQ_i are not using absolute values but the shear force in lower section minus upper section.



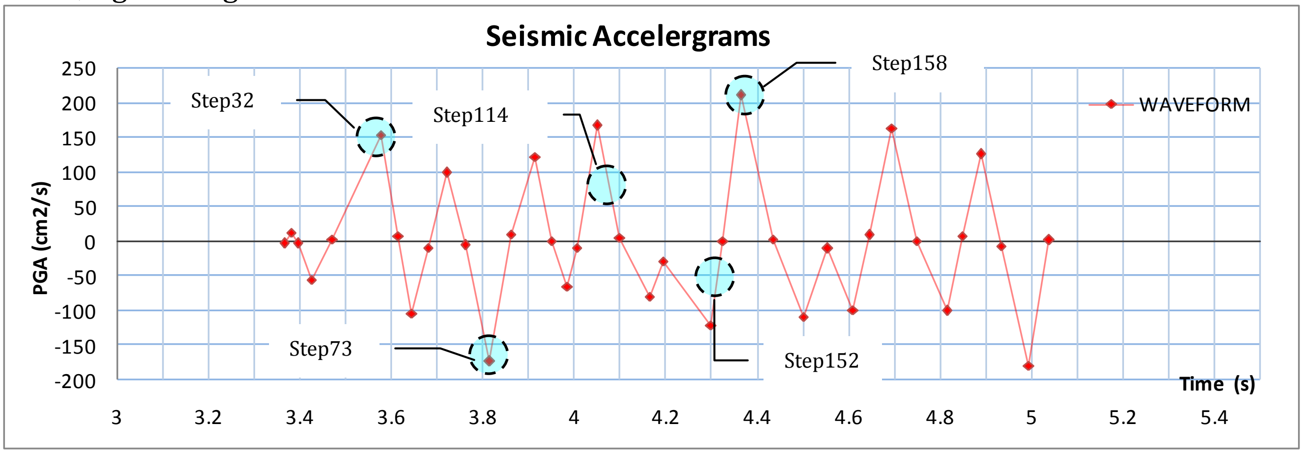
CASE3 0.25g	step 75	Qy	Real Time	step 77	Qy	Real Time
	2	4.21E+04	5.49E+00	2	4.01E+04	5.51E+00
	2	5.18E+04	**Fa=ΔQ**	2	5.30E+04	**Fa=ΔQ**
	7	1.70E+04	**3.07E+04**	7	1.83E+04	**2.42E+04**
	7	5.73E+04		7	5.54E+04	
	step 100	Qy	Real Time	step 251	Qy	Real Time
	2	-4.87E+04	5.69E+00	2	-4.63E+04	6.87E+00
	2	-1.02E+04	**Fa=ΔQ**	2	-1.34E+04	**Fa=ΔQ**
	7	-4.58E+04	**2.86E+04**	7	-4.81E+04	**1.68E+04**
	7	-3.42E+04		7	-3.20E+04	

* ΣΔQ=|ΔQ(2)+ ΔQ(7)|= Fa

Applying the same approach, obtain the critical lateral force Fa for different cases:

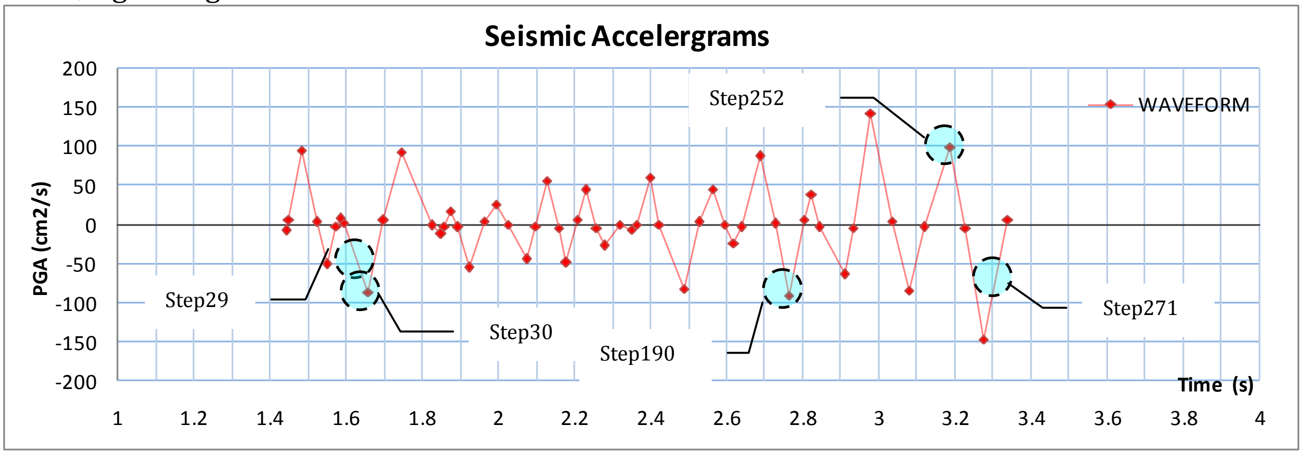


Case1, Ag=0.215g



CASE1 0.215g	step 32	Qy	Real Time	step 73	Qy	Real Time
	2	9.08E+03	3.56E+00	2	-1.81E+04	3.82E+00
	2	2.26E+03	**Fa=ΔQ**	2	-8.83E+03	**Fa=ΔQ**
	7	2.26E+03	**1.36E+04**	7	-8.83E+03	**1.86E+04**
	7	9.08E+03		7	-1.82E+04	
	step 114	Qy	Real Time	step 152	Qy	Real Time
	2	2.22E+04	4.08E+00	2	-2.82E+04	4.32E+00
	2	1.56E+04	**Fa=ΔQ**	2	-2.12E+04	**Fa=ΔQ**
	7	1.56E+04	**1.33E+04**	7	-2.12E+04	**1.39E+04**
	7	2.22E+04		7	-2.82E+04	
	step 158	Qy	Real Time			
	2	-1.68E+04	4.36E+00			
	2	-2.39E+04	**Fa=ΔQ**			
	7	-2.38E+04	**1.40E+04**			
	7	-1.68E+04				

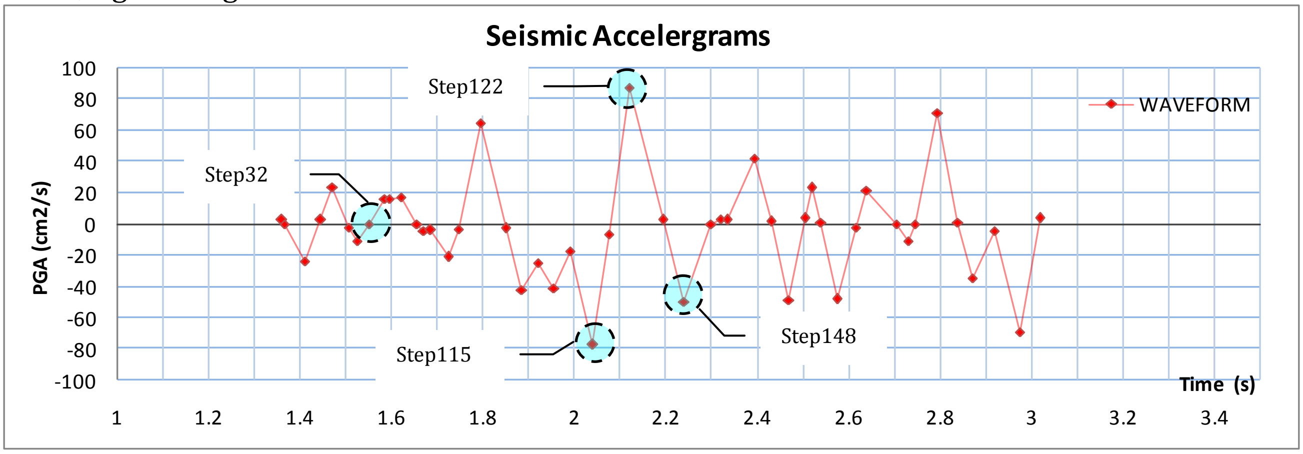
Case1, Ag=0.15g



CASE1 0.15g	step 29	Qy	Real Time	step 30	Qy	Real Time
	2	-8.99E+02	1.63E+00	2	-2.01E+03	1.64E+00
	2	1.64E+03	**Fa=ΔQ**	2	9.97E+02	**Fa=ΔQ**
	7	1.64E+03	**5.07E+03**	7	9.97E+02	**6.02E+03**
	7	-8.99E+02		7	-2.01E+03	
	step 190	Qy	Real Time	step 252	Qy	Real Time
	2	9.21E+03	2.76E+00	2	1.43E+04	3.19E+00
	2	1.18E+04	**Fa=ΔQ**	2	8.61E+03	**Fa=ΔQ**

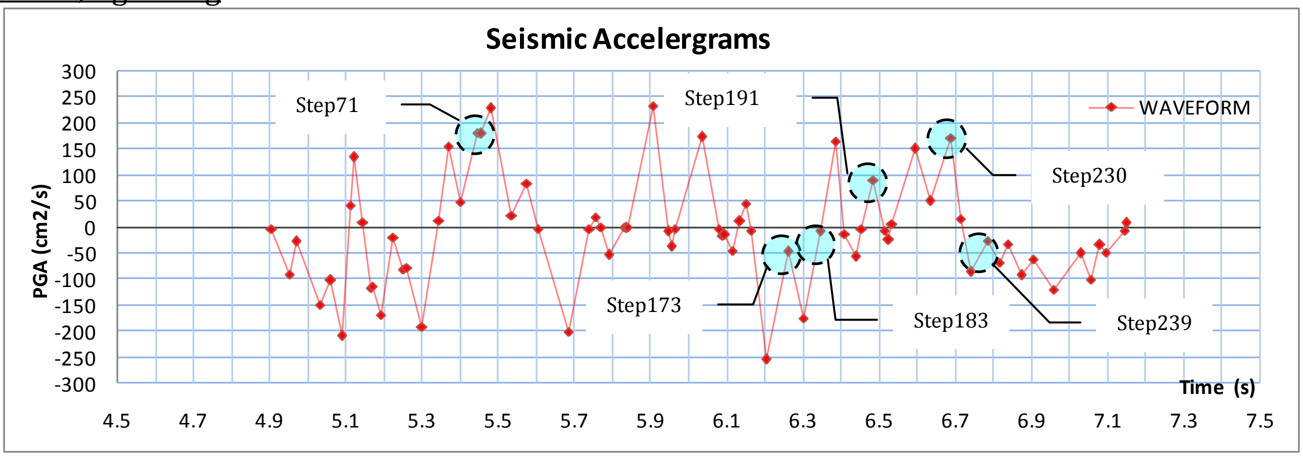
7	1.18E+04	**5.22E+03**	7	8.61E+03	**1.15E+04**
7	9.21E+03		7	1.44E+04	
step 271	Qy	Real Time			
2	-1.31E+04	3.33E+00			
2	-1.12E+04	**Fa=ΔQ**			
7	-1.12E+04	**3.80E+03**			
7	-1.31E+04				

Case1, Ag=0.089g



CASE1 0.089g	step 32	Qy	Real Time	step 115	Qy	Real Time
	2	-2.23E+02	1.55E+00	2	-1.13E+04	2.05E+00
	2	-1.95E+02	**Fa=ΔQ**	2	-6.83E+03	**Fa=ΔQ**
	7	-1.95E+02	**5.70E+01**	7	-6.83E+03	**8.98E+03**
	7	-2.23E+02		7	-1.13E+04	
	step 122	Qy	Real Time	step 148	Qy	Real Time
	2	-7.83E+03	2.09E+00	2	9.42E+03	2.25E+00
	2	-8.29E+03	**Fa=ΔQ**	2	1.06E+04	**Fa=ΔQ**
	7	-8.29E+03	**9.14E+02**	7	1.01E+04	**1.26E+03**
	7	-7.83E+03		7	1.02E+04	

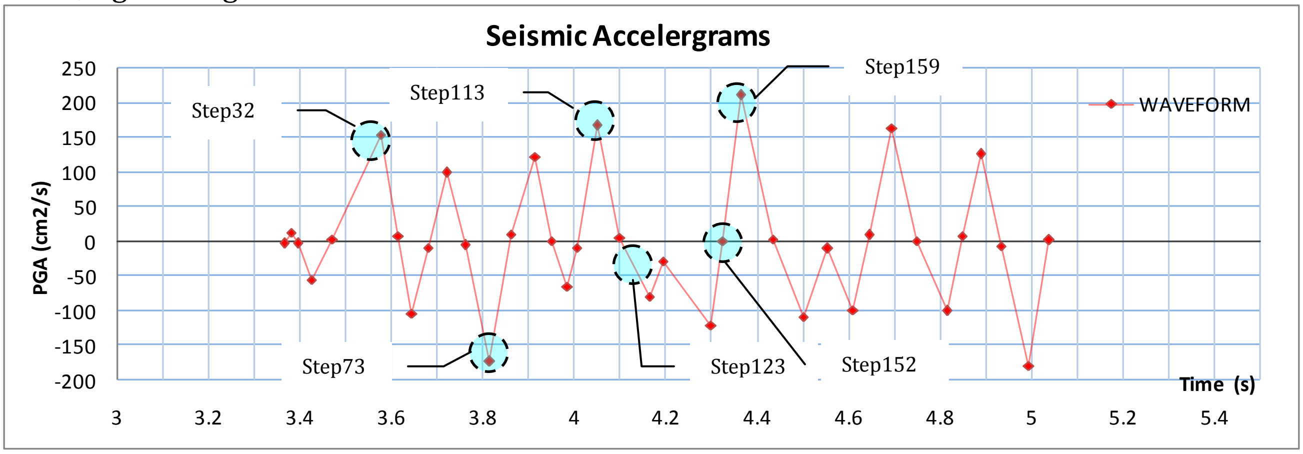
Case2, Ag=0.25g



CASE2 0.25g	step 71	Qy	Real Time	step 173	Qy	Real Time
	2	2.80E+04	5.46E+00	2	-2.28E+04	6.25E+00
	2	8.96E+03	**Fa=ΔQ**	2	7.28E+02	**Fa=ΔQ**
	7	0.00E+00	**1.90E+04**	7	0.00E+00	**2.36E+04**
	7	0.00E+00		7	0.00E+00	

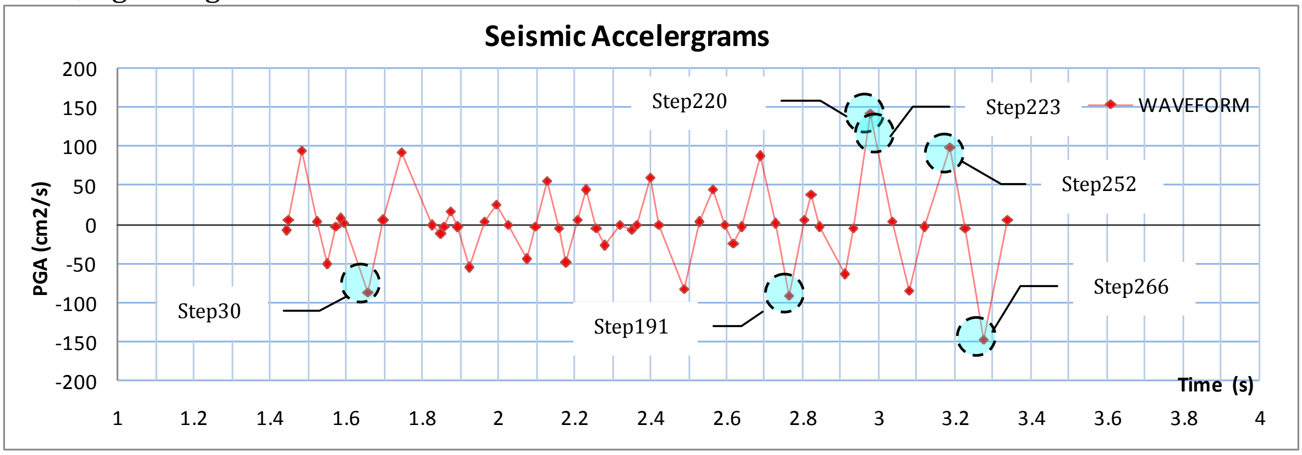
step 183	Qy	Real Time	step 191	Qy	Real Time
2	-5.34E+04	6.33E+00	2	-3.41E+04	6.46E+00
2	-2.89E+04	**Fa=ΔQ**	2	-3.38E+04	**Fa=ΔQ**
7	0.00E+00	**2.45E+04**	7	0.00E+00	**3.60E+02**
7	0.00E+00		7	0.00E+00	
step 230	Qy	Real Time	step 239	Qy	Real Time
2	5.53E+04	6.69E+00	2	3.06E+04	6.77E+00
2	2.34E+04	**Fa=ΔQ**	2	3.22E+04	**Fa=ΔQ**
7	0.00E+00	**3.18E+04**	7	0.00E+00	**1.53E+03**
7	0.00E+00		7	0.00E+00	

Case2, Ag=0.215g



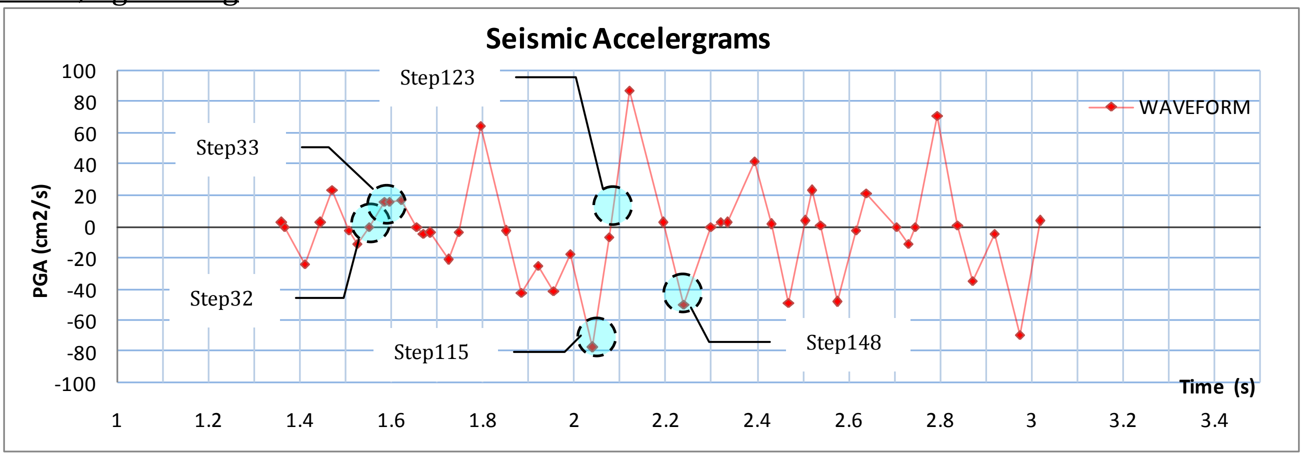
CASE2 0.215g	step 32	Qy	Real Time	step 73	Qy	Real Time
	2	1.51E+04	3.56E+00	2	-2.50E+04	3.82E+00
	2	1.23E+03	**Fa=ΔQ**	2	-7.39E+03	**Fa=ΔQ**
	7	0.00E+00	**1.38E+04**	7	0.00E+00	**1.76E+04**
	7	0.00E+00		7	0.00E+00	
	step 113	Qy	Real Time	step 123	Qy	Real Time
	2	2.98E+04	4.07E+00	2	1.71E+04	4.13E+00
	2	1.32E+04	**Fa=ΔQ**	2	1.89E+04	**Fa=ΔQ**
	7	0.00E+00	**1.67E+04**	7	0.00E+00	**1.80E+03**
	7	0.00E+00		7	0.00E+00	
	step 152	Qy	Real Time	step 159	Qy	Real Time
	2	-3.45E+04	4.32E+00	2	-8.47E+03	4.37E+00
	2	-2.00E+04	**Fa=ΔQ**	2	-2.49E+04	**Fa=ΔQ**
	7	0.00E+00	**1.45E+04**	7	0.00E+00	**1.64E+04**
	7	0.00E+00		7	0.00E+00	

Case2, Ag=0.15g



CASE2 0.15g	step 30	Qy	Real Time	step 191	Qy	Real Time
	2	-5.40E+03	1.64E+00	2	5.88E+03	2.77E+00
	2	1.48E+03	**Fa=ΔQ**	2	1.23E+04	**Fa=ΔQ**
	7	0.00E+00	**6.88E+03**	7	0.00E+00	**6.43E+03**
	7	0.00E+00		7	0.00E+00	
	step 220	Qy	Real Time	step 223	Qy	Real Time
	2	-6.34E+03	2.97E+00	2	1.05E+03	2.99E+00
	2	-1.30E+04	**Fa=ΔQ**	2	-1.17E+04	**Fa=ΔQ**
	7	0.00E+00	**6.67E+03**	7	0.00E+00	**1.27E+04**
	7	0.00E+00		7	0.00E+00	
	step 252	Qy	Real Time	step 266	Qy	Real Time
	2	1.89E+04	3.19E+00	2	-1.60E+04	3.29E+00
	2	7.76E+03	**Fa=ΔQ**	2	-1.69E+03	**Fa=ΔQ**
	7	0.00E+00	**1.11E+04**	7	0.00E+00	**1.44E+04**
	7	0.00E+00		7	0.00E+00	

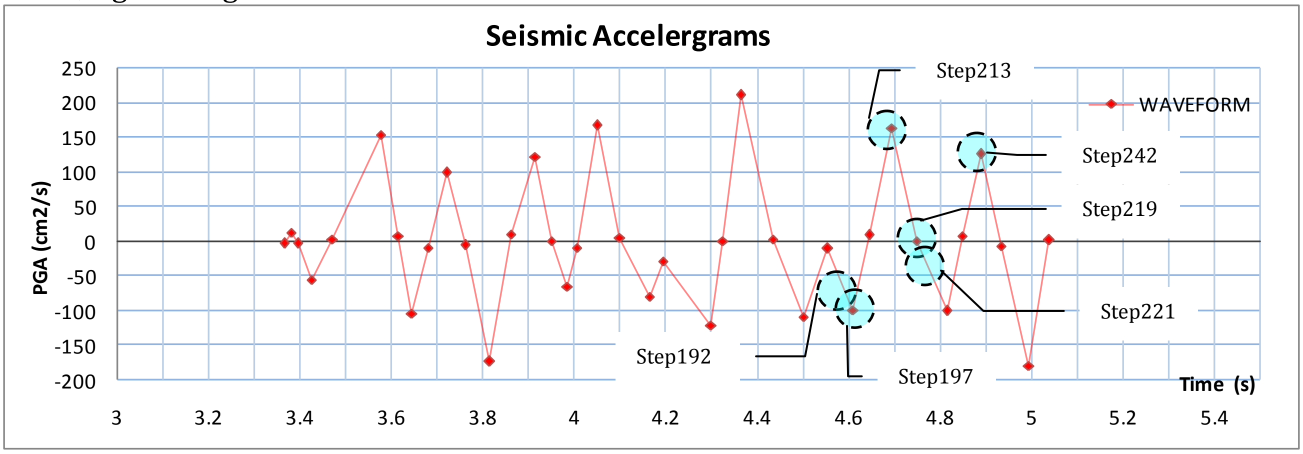
Case2, Ag=0.089g



CASE2 0.089g	step 32	Qy	Real Time	step 33	Qy	Real Time
	2	-4.81E+02	1.55E+00	2	2.14E+02	1.56E+00
	2	-1.83E+02	**Fa=ΔQ**	2	-1.46E+02	**Fa=ΔQ**
	7	0.00E+00	**2.97E+02**	7	0.00E+00	**3.60E+02**
	7	0.00E+00		7	0.00E+00	
	step 115	Qy	Real Time	step 123	Qy	Real Time
	2	-1.60E+04	2.05E+00	2	-5.39E+03	2.09E+00
	2	-6.13E+03	**Fa=ΔQ**	2	-8.41E+03	**Fa=ΔQ**

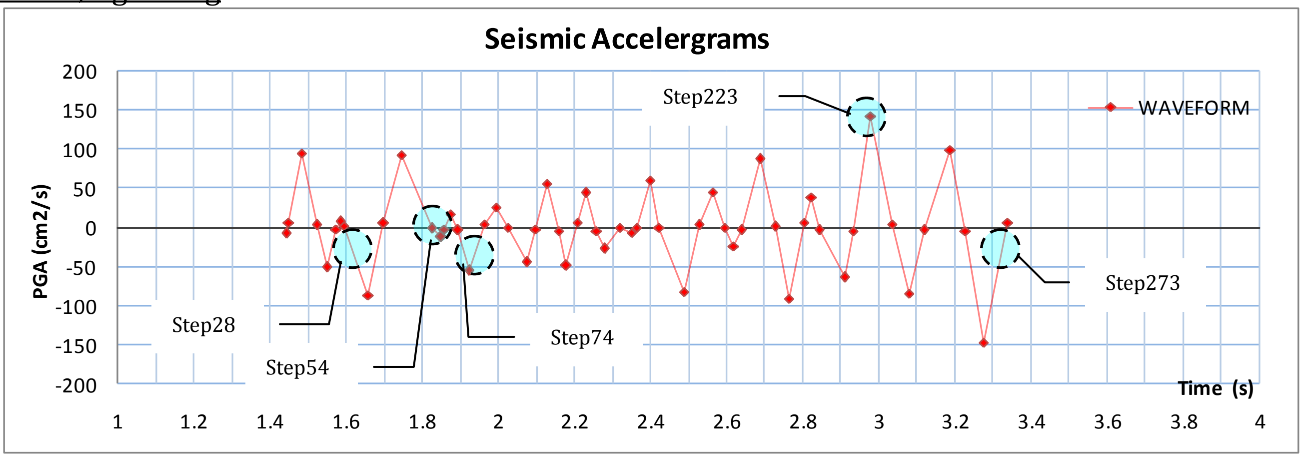
7	0.00E+00	**9.91E+03**	7	0.00E+00	**3.02E+03**
7	0.00E+00		7	0.00E+00	
step 148	Qy	Real Time			
2	7.90E+03	2.25E+00			
2	1.08E+04	**Fa=ΔQ**			
7	0.00E+00	**2.90E+03**			
7	0.00E+00				

Case3, Ag=0.215g



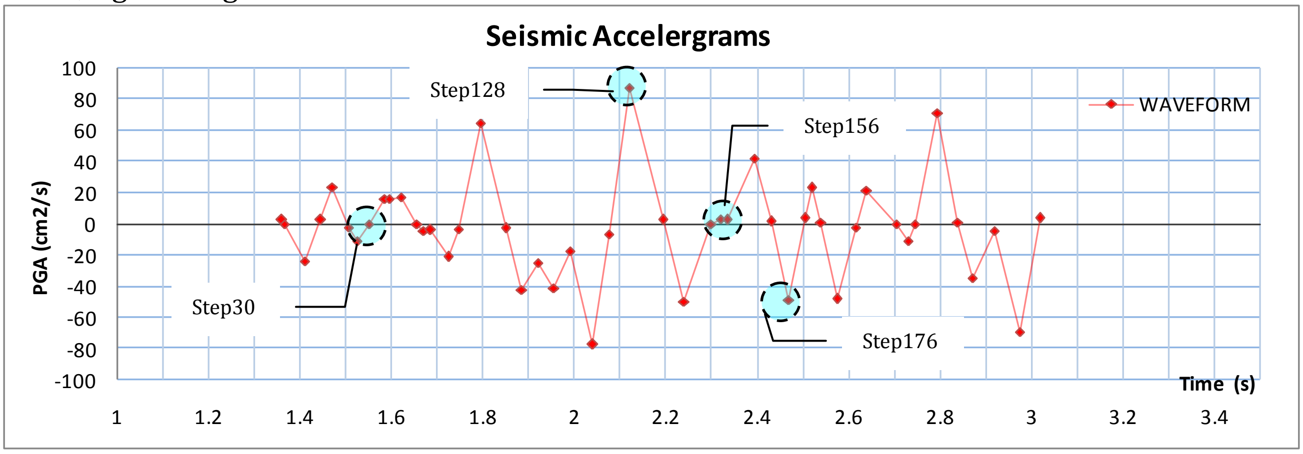
CASE3 0.215g	step 192	Qy	Real Time	step 197	Qy	Real Time
	2	-4.80E+04	4.58E+00	2	-4.14E+04	4.61E+00
	2	-1.44E+04	**Fa=ΔQ**	2	-7.30E+03	**Fa=ΔQ**
	7	-4.97E+04	**1.70E+04**	7	-4.40E+04	**1.66E+04**
	7	-3.31E+04		7	-2.64E+04	
	step 213	Qy	Real Time	step 219	Qy	Real Time
	2	2.62E+04	4.70E+00	2	3.68E+04	4.74E+00
	2	4.14E+04	**Fa=ΔQ**	2	5.27E+04	**Fa=ΔQ**
	7	4.56E+03	**2.10E+04**	7	1.96E+04	**1.57E+04**
	7	4.07E+04		7	5.11E+04	
	step 221	Qy	Real Time	step 242	Qy	Real Time
	2	3.58E+04	4.75E+00	2	-4.61E+04	4.88E+00
	2	5.31E+04	**Fa=ΔQ**	2	-1.89E+04	**Fa=ΔQ**
	7	1.93E+04	**1.40E+04**	7	-5.33E+04	**5.25E+03**
	7	5.05E+04		7	-3.13E+04	

Case3, Ag=0.15g



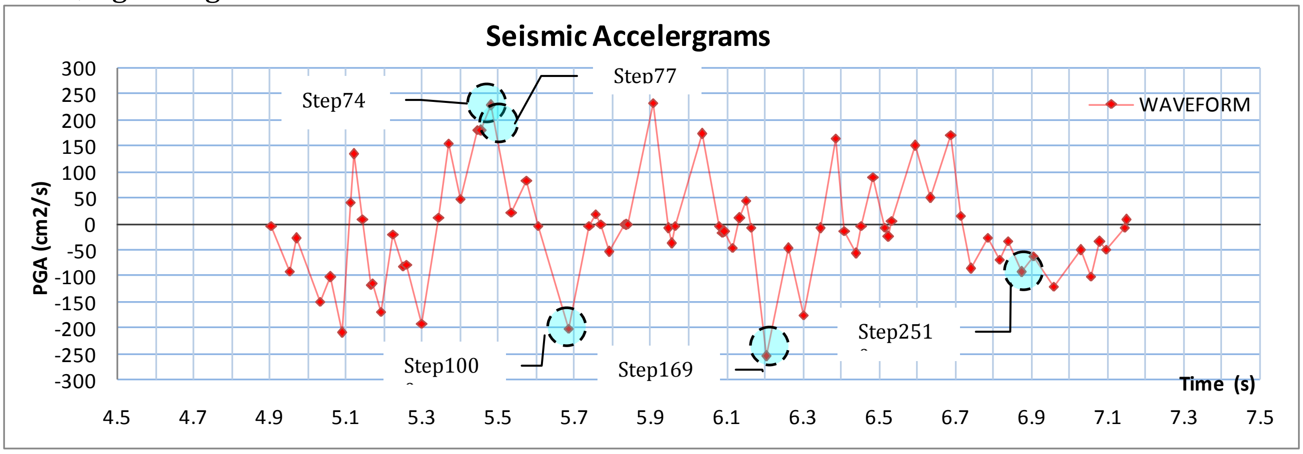
CASE3 0.15g	step 28	Qy	Real Time	step 54	Qy	Real Time
	2	-1.31E+04	1.63E+00	2	1.63E+04	1.81E+00
	2	1.52E+04	**Fa=ΔQ**	2	3.68E+04	**Fa=ΔQ**
	7	-2.56E+04	**5.24E+03**	7	3.67E+01	**6.64E+03**
	7	-2.58E+03		7	2.72E+04	
	step 74	Qy	Real Time	step 223	Qy	Real Time
	2	-2.43E+04	1.95E+00	2	9.12E+03	2.99E+00
	2	2.11E+03	**Fa=ΔQ**	2	2.50E+04	**Fa=ΔQ**
	7	-3.42E+04	**7.38E+03**	7	-1.15E+04	**1.47E+04**
	7	-1.52E+04		7	1.91E+04	
	step 273	Qy	Real Time			
	2	-2.42E+04	3.34E+00			
	2	1.09E+03	**Fa=ΔQ**			
	7	-3.53E+04	**4.89E+03**			
	7	-1.49E+04				

Case3, Ag=0.089g



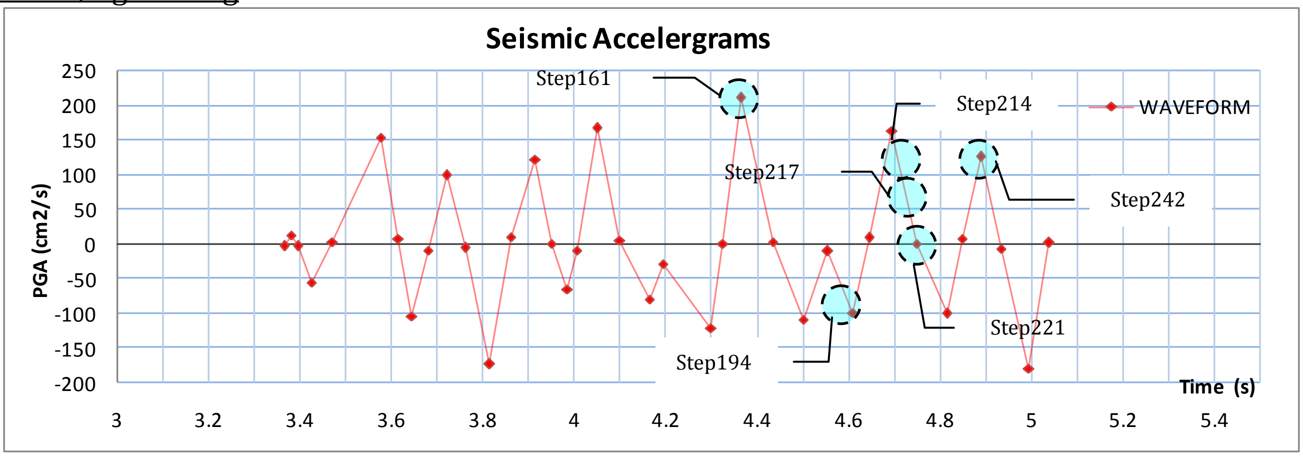
CASE3 0.089g	step 30	Qy	Real Time	step 128	Qy	Real Time
	2	-4.05E+03	1.54E+00	2	7.19E+03	2.13E+00
	2	2.26E+04	**Fa=ΔQ**	2	2.52E+04	**Fa=ΔQ**
	7	-1.94E+04	**3.08E+02**	7	-1.20E+04	**1.01E+04**
	7	6.97E+03		7	1.61E+04	
	step 156	Qy	Real Time	step 176	Qy	Real Time
	2	-2.23E+04	2.31E+00	2	1.39E+04	2.43E+00
	2	2.66E+03	**Fa=ΔQ**	2	3.40E+04	**Fa=ΔQ**
	7	-3.38E+04	**4.39E+03**	7	-2.32E+03	**5.04E+03**
	7	-1.33E+04		7	2.28E+04	

Case4, Ag=0.25g



CASE4 0.25g	step 74	Qy	Real Time	step 77	Qy	Real Time
	2	7.34E+04	5.48E+00	2	7.06E+04	5.51E+00
	2	4.05E+04	**Fa=ΔQ**	2	4.43E+04	**Fa=ΔQ**
	7	0.00E+00	**3.29E+04**	7	0.00E+00	**2.64E+04**
	7	0.00E+00		7	0.00E+00	
	step 100	Qy	Real Time	step 251	Qy	Real Time
	2	-4.15E+04	5.69E+00	2	-3.36E+04	6.87E+00
	2	-1.31E+04	**Fa=ΔQ**	2	-1.73E+04	**Fa=ΔQ**
	7	0.00E+00	**2.84E+04**	7	0.00E+00	**1.63E+04**
	7	0.00E+00		7	0.00E+00	
	step 169	Qy	Real Time			
	2	-1.81E+04	6.21E+00			
	2	9.27E+03	**Fa=ΔQ**			
	7	0.00E+00	**2.74E+04**			
	7	0.00E+00				

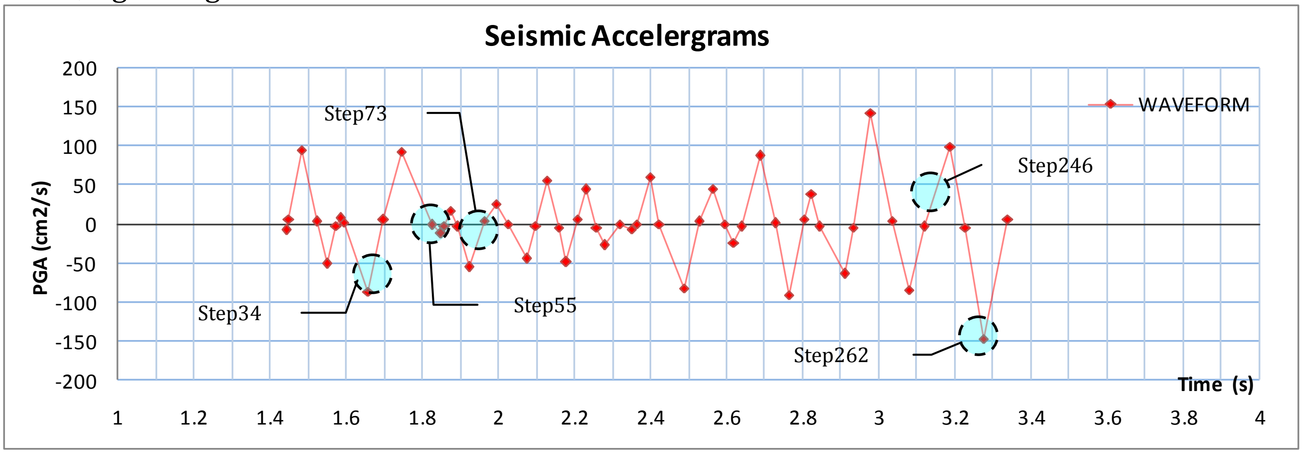
Case4, Ag=0.215g



CASE4 0.215g	step 194	Qy	Real Time	step 214	Qy	Real Time
	2	-3.62E+04	4.59E+00	2	5.94E+04	4.71E+00
	2	-1.70E+04	**Fa=ΔQ**	2	3.59E+04	**Fa=ΔQ**
	7	0.00E+00	**1.92E+04**	7	0.00E+00	**2.35E+04**
	7	0.00E+00		7	0.00E+00	
	step 217	Qy	Real Time	step 221	Qy	Real Time
	2	6.34E+04	4.73E+00	2	6.02E+04	4.75E+00
	2	4.35E+04	**Fa=ΔQ**	2	4.72E+04	**Fa=ΔQ**

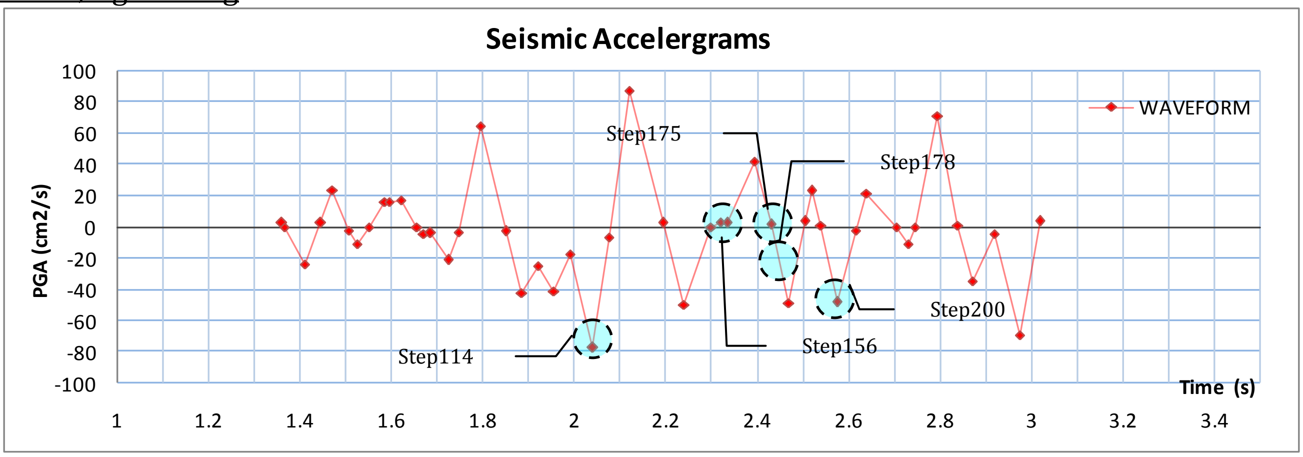
7	0.00E+00	**1.99E+04**	7	0.00E+00	**1.30E+04**
7	0.00E+00		7	0.00E+00	
step 242	Qy	Real Time	step 161	Qy	Real Time
2	-2.96E+04	4.88E+00	2	4.25E+04	4.38E+00
2	-2.50E+04	**Fa=ΔQ**	2	2.08E+04	**Fa=ΔQ**
7	0.00E+00	**4.61E+03**	7	0.00E+00	**2.17E+04**
7	0.00E+00		7	0.00E+00	

Case4, Ag=0.15g



CASE4 0.15g	step 34	Qy	Real Time	step 55	Qy	Real Time
	2	-6.56E+03	1.67E+00	2	3.74E+04	1.82E+00
	2	4.64E+03	**Fa=ΔQ**	2	3.02E+04	**Fa=ΔQ**
	7	0.00E+00	**1.12E+04**	7	0.00E+00	**7.21E+03**
	7	0.00E+00		7	0.00E+00	
	step 73	Qy	Real Time	step 246	Qy	Real Time
	2	-9.62E+03	1.94E+00	2	-4.46E+03	3.15E+00
	2	5.22E+01	**Fa=ΔQ**	2	-3.33E+03	**Fa=ΔQ**
	7	0.00E+00	**9.67E+03**	7	0.00E+00	**1.13E+03**
	7	0.00E+00		7	0.00E+00	
	step 262	Qy	Real Time			
	2	2.87E+04	3.26E+00			
	2	3.11E+04	**Fa=ΔQ**			
	7	0.00E+00	**2.42E+03**			
	7	0.00E+00				

Case4, Ag=0.089g

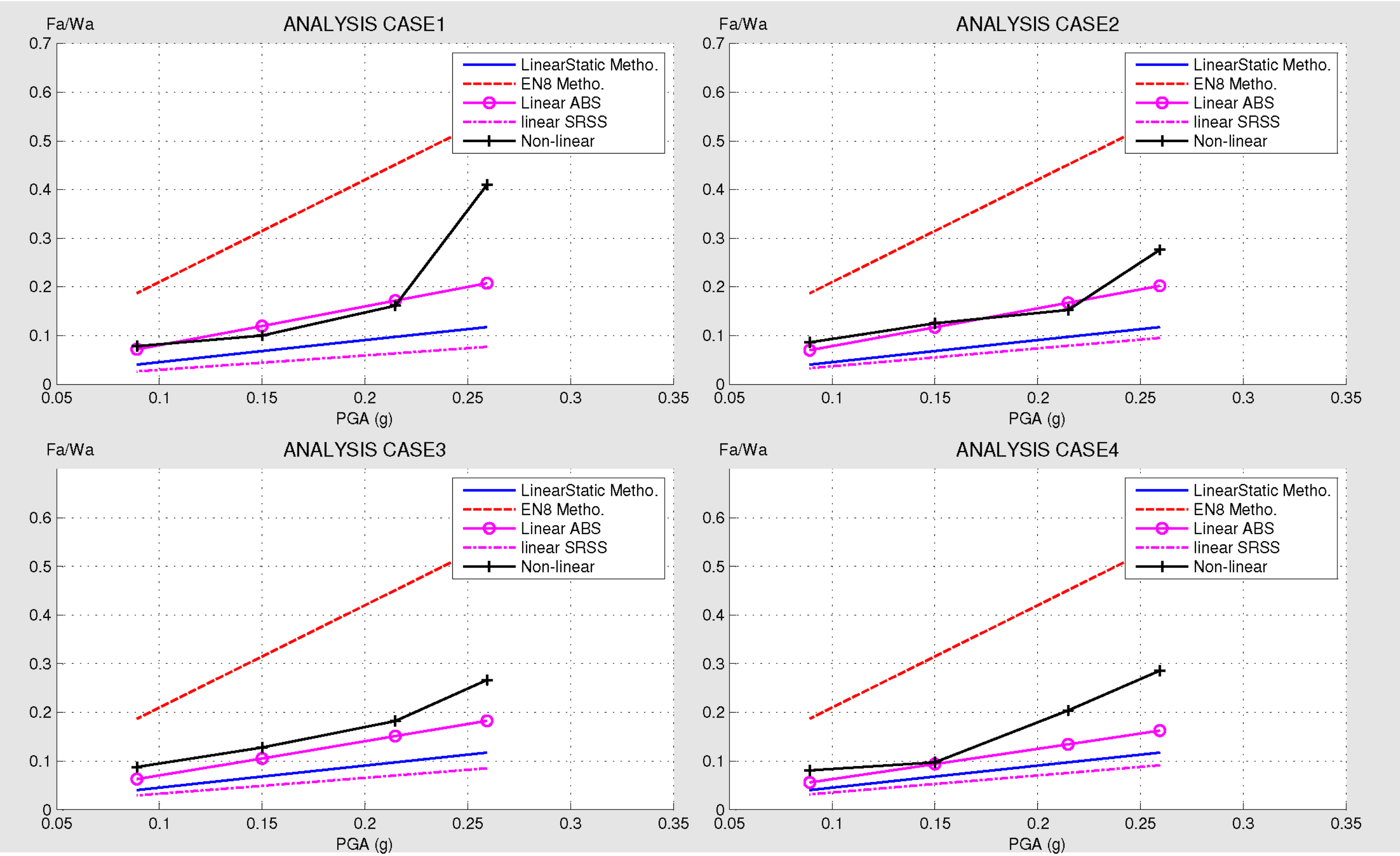


CASE4 0.089g	step 114	Qy	Real Time	step 156	Qy	Real Time
	2	-6.80E+02	2.04E+00	2	-5.48E+03	2.31E+00
	2	8.62E+03	**Fa=ΔQ**	2	-1.16E+03	**Fa=ΔQ**
	7	0.00E+00	**9.30E+03**	7	0.00E+00	**4.32E+03**
	7	0.00E+00		7	0.00E+00	
	step 175	Qy	Real Time	step 178	Qy	Real Time
	2	3.34E+04	2.43E+00	2	3.27E+04	2.44E+00
	2	2.80E+04	**Fa=ΔQ**	2	2.89E+04	**Fa=ΔQ**
	7	0.00E+00	**5.46E+03**	7	0.00E+00	**3.73E+03**
	7	0.00E+00		7	0.00E+00	
	step 200	Qy	Real Time			
	2	-7.66E+03	2.58E+00			
	2	1.14E+03	**Fa=ΔQ**			
	7	0.00E+00	**8.80E+03**			
	7	0.00E+00				

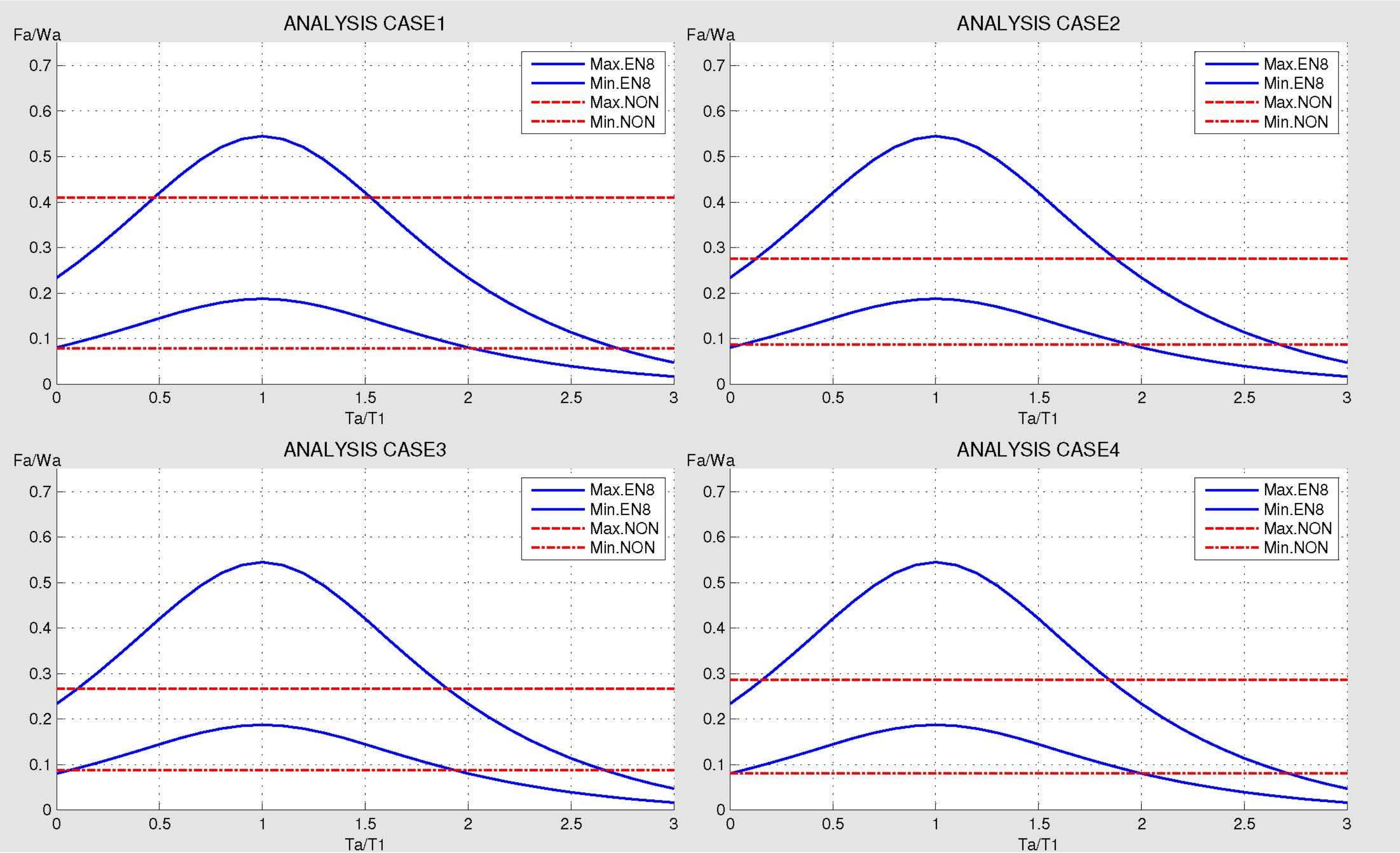
--Comparison of Results--
Values of Fa obtained by non-linear dynamic analysis:

	0.25g	0.125g	0.15g	0.089g
CASE1	Step183 6.33s Fa=4.72E+04	Step158 4.36s Fa=1.86E+04	Step252 3.19s Fa=1.15E+04	Step115 2.05s Fa=8.98E+03
CASE2	Step173 6.25s Fa=3.18E+04	Step159 4.37s Fa=1.76E+04	Step266 3.29s Fa=1.44E+04	Step115 2.05s Fa=9.91E+03
CASE3	Step100 5.69s Fa=3.07E+04	Step213 4.70s Fa=2.10E+04	Step223 2.99s Fa=1.47E+04	Step128 2.13s Fa=1.01E+04
CASE4	Step169 6.21s Fa=3.29E+04	Step161 4.38s Fa=2.35E+04	Step34 1.67s Fa=1.12E+04	Step114 2.04 Fa=9.30E+03

Comparison Curve of Fa/Wa obtained by Different Method:



Determining Ta/T1 with Non-linear Results:



--Conclusions--

-we can see from the above graph that non-linear results are in good accordance with linear dynamic and linear static analysis for the first 3 points. While the last point shows that Fa value calculated from the specific seismic waveform "0.25g" is quite big compared with linear results.

-EN8 empirical formula for non-structural element is still debatable because of the exaggerated results.

-In non-linear analysis, each data is related to a specific waveform, as a result, their relationship may not be linear with respect to peak ground acceleration.

-The Fa value corresponding to the first two PGA levels are approximately the same for all 4 Boundary conditions. While the third one is most critical in Case4 and the fourth one is most critical in Case1.

-Notice that the peak value of Fa is generally not achieved at the time when the ground acceleration also reach its peak value coincidently. We can predict that, change of wave patterns and durations may affect the above results.

-The very big value in Case1 Non-linear curve (Waveform 0.25g) may be due high magnitude of seismic wave in one single direction (+ or-) repeating again and again rapidly.

-Extract Ta/T1 ratio with non-linear results to understand which could be the real value to adopt in EN8 empirical formula. As we can see, according to non-linear analysis, the range of real value Ta/T1 is between the red lines (Max.Non & Min.Non) for the 4 boundary cases.

APPENDIX A

Max Zone1 $a_g = 0.368[g]$, Ground Type A, Response Spectrum Type I

Parameter Independent	
ag	0.368
S	1.000
TB	0.150
TC	0.400
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	1
Ground Type	A

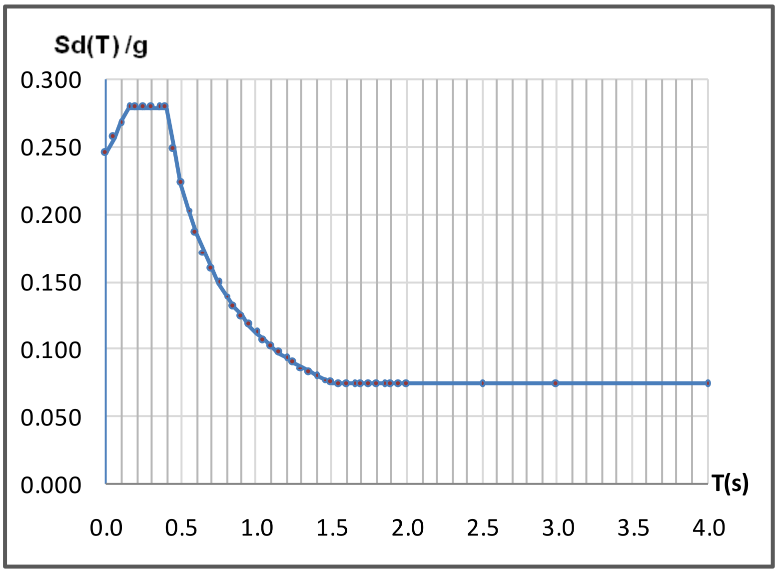
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.245
	0.05	0.256
	0.10	0.268
TB ←	0.15	0.279
	0.20	0.279
	0.25	0.279
	0.30	0.279
	0.35	0.279
TC ←	0.40	0.279
	0.45	0.248
	0.50	0.223
	0.55	0.203
	0.60	0.186
	0.65	0.172
	0.70	0.159
	0.75	0.149
	0.80	0.139
	0.85	0.131
	0.90	0.124
	0.95	0.117
	1.00	0.112
	1.05	0.106
	1.10	0.101
	1.15	0.097
	1.20	0.093
	1.25	0.089
	1.30	0.086
	1.35	0.083
	1.40	0.080
	1.45	0.077
	1.50	0.074
	1.55	0.074
	1.60	0.074
	1.65	0.074
	1.70	0.074
	1.75	0.074
	1.80	0.074
	1.85	0.074
	1.90	0.074
	1.95	0.074
TD ←	2.00	0.074
	2.50	0.074
	3.00	0.074
	4.00	0.074
	5.00	0.074
	6.00	0.074
	8.00	0.074
	10.00	0.074

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2]\ ;\ \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



Max Zone1 $a_g = 0.368[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.368
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q₀	3.300
Kw	1.000

Primary Parameter	
Zone	1
Ground Type	B

Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.294
	0.05	0.308
	0.10	0.321
TB ←	0.15	0.335
	0.20	0.335
	0.25	0.335
	0.30	0.335
	0.35	0.335
	0.40	0.335
	0.45	0.335
TC ←	0.50	0.335
	0.55	0.304
	0.60	0.279
	0.65	0.257
	0.70	0.239
	0.75	0.223
	0.80	0.209
	0.85	0.197
	0.90	0.186
	0.95	0.176
	1.00	0.167
	1.05	0.159
	1.10	0.152
	1.15	0.145
	1.20	0.139
	1.25	0.134
	1.30	0.129
	1.35	0.124
	1.40	0.119
	1.45	0.115
	1.50	0.112
	1.55	0.108
	1.60	0.105
	1.65	0.101
	1.70	0.098
	1.75	0.096
	1.80	0.093
	1.85	0.090
	1.90	0.088
	1.95	0.086
TD ←	2.00	0.084
	2.50	0.074
	3.00	0.074
	4.00	0.074
	5.00	0.074
	6.00	0.074
	8.00	0.074
	10.00	0.074

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

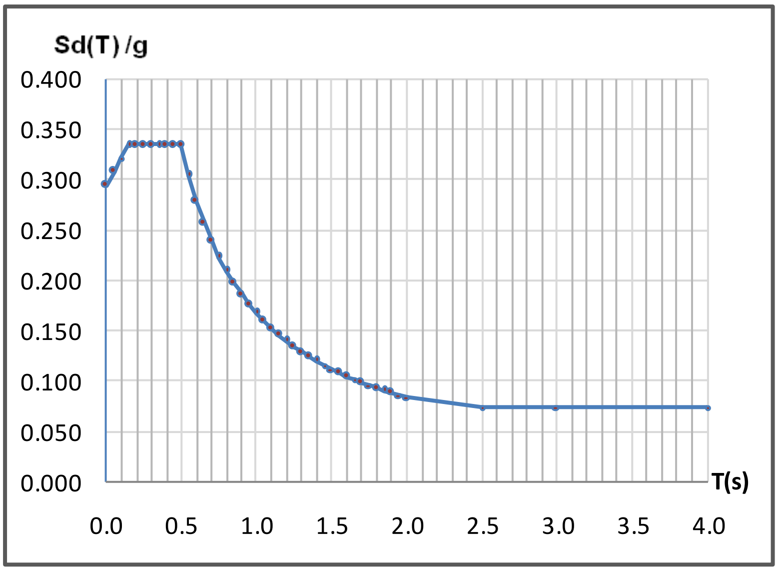
$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$

$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$

$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$

$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$

Notes: See EN1998-1(3.2.2)



Max Zone1 $a_g = 0.368[g]$, Ground Type C, Response Spectrum Type I

Parameter Independent	
ag	0.368
S	1.150
TB	0.200
TC	0.600
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	1
Ground Type	C

Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.282
	0.05	0.292
	0.10	0.301
	0.15	0.311
TB ←	0.20	0.321
	0.25	0.321
	0.30	0.321
	0.35	0.321
	0.40	0.321
	0.45	0.321
	0.50	0.321
	0.55	0.321
TC ←	0.60	0.321
	0.65	0.296
	0.70	0.275
	0.75	0.256
	0.80	0.240
	0.85	0.226
	0.90	0.214
	0.95	0.202
	1.00	0.192
	1.05	0.183
	1.10	0.175
	1.15	0.167
	1.20	0.160
	1.25	0.154
	1.30	0.148
	1.35	0.142
	1.40	0.137
	1.45	0.133
	1.50	0.128
	1.55	0.124
	1.60	0.120
	1.65	0.117
	1.70	0.113
	1.75	0.110
	1.80	0.107
	1.85	0.104
	1.90	0.101
	1.95	0.099
TD ←	2.00	0.096
	2.50	0.074
	3.00	0.074
	4.00	0.074
	5.00	0.074
	6.00	0.074
	8.00	0.074
	10.00	0.074

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

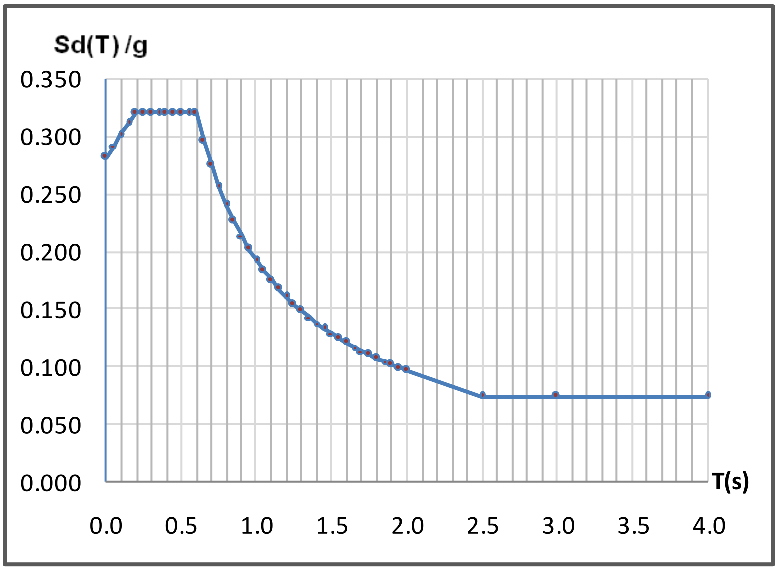
$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$

$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$

$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] \; ; \; \beta \times a_g\right\}$$

$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \; ; \; \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone1 $a_g = 0.368[g]$, Ground Type D, Response Spectrum Type I

Parameter Independent	
ag	0.368
S	1.350
TB	0.200
TC	0.800
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	1
Ground Type	D

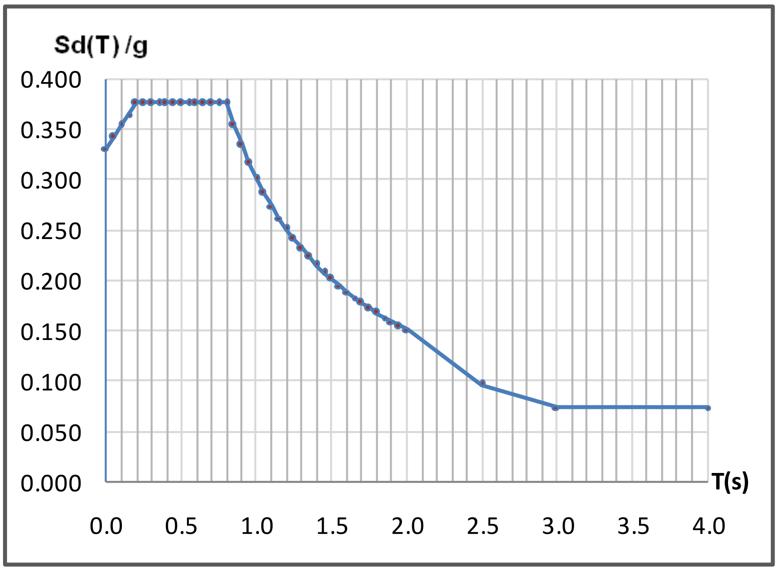
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.331
	0.05	0.342
	0.10	0.354
	0.15	0.365
TB ←	0.20	0.376
	0.25	0.376
	0.30	0.376
	0.35	0.376
	0.40	0.376
	0.45	0.376
	0.50	0.376
	0.55	0.376
	0.60	0.376
	0.65	0.376
	0.70	0.376
	0.75	0.376
TC ←	0.80	0.376
	0.85	0.354
	0.90	0.335
	0.95	0.317
	1.00	0.301
	1.05	0.287
	1.10	0.274
	1.15	0.262
	1.20	0.251
	1.25	0.241
	1.30	0.232
	1.35	0.223
	1.40	0.215
	1.45	0.208
	1.50	0.201
	1.55	0.194
	1.60	0.188
	1.65	0.182
	1.70	0.177
	1.75	0.172
	1.80	0.167
	1.85	0.163
	1.90	0.158
	1.95	0.154
TD ←	2.00	0.151
	2.50	0.096
	3.00	0.074
	4.00	0.074
	5.00	0.074
	6.00	0.074
	8.00	0.074
	10.00	0.074

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] \; ; \; \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \; ; \; \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone1 $a_g = 0.368[g]$, Ground Type E, Response Spectrum Type I

Parameter Independent	
ag	0.368
S	1.400
TB	0.150
TC	0.500
TD	2.000
β	0.200
q。	3.300
Kw	1.000

Primary Parameter	
Zone	1
Ground Type	E

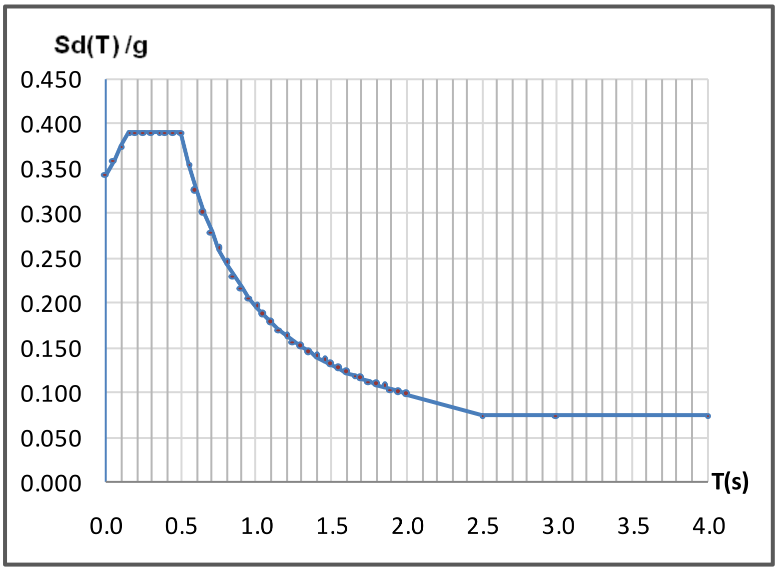
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.343
	0.05	0.359
	0.10	0.375
TB ←	0.15	0.390
	0.20	0.390
	0.25	0.390
	0.30	0.390
	0.35	0.390
	0.40	0.390
	0.45	0.390
TC ←	0.50	0.390
	0.55	0.355
	0.60	0.325
	0.65	0.300
	0.70	0.279
	0.75	0.260
	0.80	0.244
	0.85	0.230
	0.90	0.217
	0.95	0.205
	1.00	0.195
	1.05	0.186
	1.10	0.177
	1.15	0.170
	1.20	0.163
	1.25	0.156
	1.30	0.150
	1.35	0.145
	1.40	0.139
	1.45	0.135
	1.50	0.130
	1.55	0.126
	1.60	0.122
	1.65	0.118
	1.70	0.115
	1.75	0.112
	1.80	0.108
	1.85	0.105
	1.90	0.103
	1.95	0.100
TD ←	2.00	0.098
	2.50	0.074
	3.00	0.074
	4.00	0.074
	5.00	0.074
	6.00	0.074
	8.00	0.074
	10.00	0.074

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \ ; \ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \ ; \ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone2 $a_g = 0.311[g]$, Ground Type A, Response Spectrum Type I

Parameter Independent	
ag	0.311
S	1.000
TB	0.150
TC	0.400
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	2
Ground Type	A

Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.207
	0.05	0.217
	0.10	0.226
TB ←	0.15	0.236
	0.20	0.236
	0.25	0.236
	0.30	0.236
	0.35	0.236
TC ←	0.40	0.236
	0.45	0.209
	0.50	0.188
	0.55	0.171
	0.60	0.157
	0.65	0.145
	0.70	0.135
	0.75	0.126
	0.80	0.118
	0.85	0.111
	0.90	0.105
	0.95	0.099
	1.00	0.094
	1.05	0.090
	1.10	0.086
	1.15	0.082
	1.20	0.079
	1.25	0.075
	1.30	0.072
	1.35	0.070
	1.40	0.067
	1.45	0.065
	1.50	0.063
	1.55	0.062
	1.60	0.062
	1.65	0.062
	1.70	0.062
	1.75	0.062
	1.80	0.062
	1.85	0.062
	1.90	0.062
	1.95	0.062
TD ←	2.00	0.062
	2.50	0.062
	3.00	0.062
	4.00	0.062
	5.00	0.062
	6.00	0.062
	8.00	0.062
	10.00	0.062

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

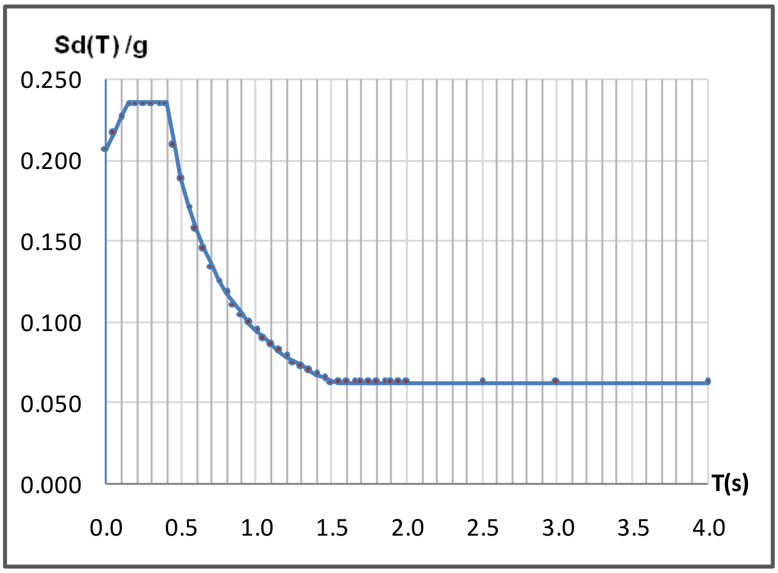
$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$

$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$

$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] \; ; \; \beta \times a_g\right\}$$

$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \; ; \; \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone2 $a_g = 0.311[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.311
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	2
Ground Type	B

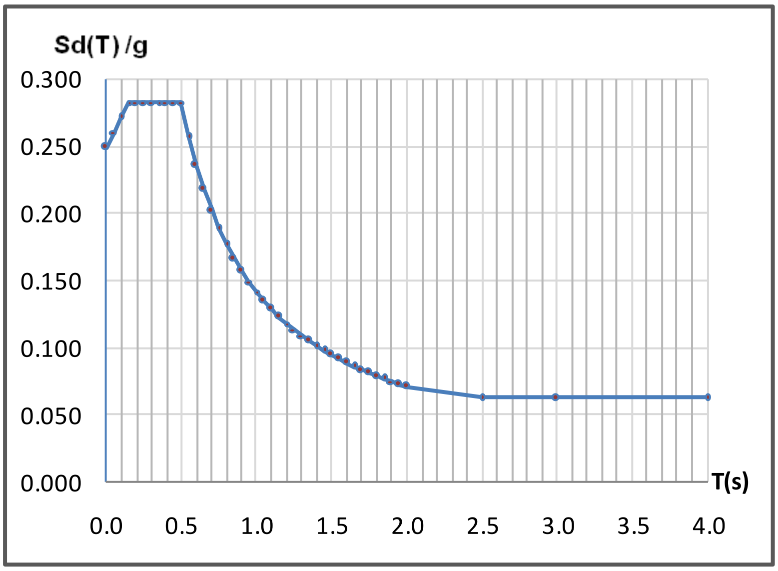
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.249
	0.05	0.260
	0.10	0.271
TB ←	0.15	0.283
	0.20	0.283
	0.25	0.283
	0.30	0.283
	0.35	0.283
	0.40	0.283
	0.45	0.283
TC ←	0.50	0.283
	0.55	0.257
	0.60	0.236
	0.65	0.217
	0.70	0.202
	0.75	0.188
	0.80	0.177
	0.85	0.166
	0.90	0.157
	0.95	0.149
	1.00	0.141
	1.05	0.135
	1.10	0.129
	1.15	0.123
	1.20	0.118
	1.25	0.113
	1.30	0.109
	1.35	0.105
	1.40	0.101
	1.45	0.097
	1.50	0.094
	1.55	0.091
	1.60	0.088
	1.65	0.086
	1.70	0.083
	1.75	0.081
	1.80	0.079
	1.85	0.076
	1.90	0.074
	1.95	0.072
TD ←	2.00	0.071
	2.50	0.062
	3.00	0.062
	4.00	0.062
	5.00	0.062
	6.00	0.062
	8.00	0.062
	10.00	0.062

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



Max Zone2 $a_g = 0.311[g]$, Ground Type C, Response Spectrum Type I

Parameter Independent	
ag	0.311
S	1.150
TB	0.200
TC	0.600
TD	2.000
β	0.200
q。	3.300
Kw	1.000

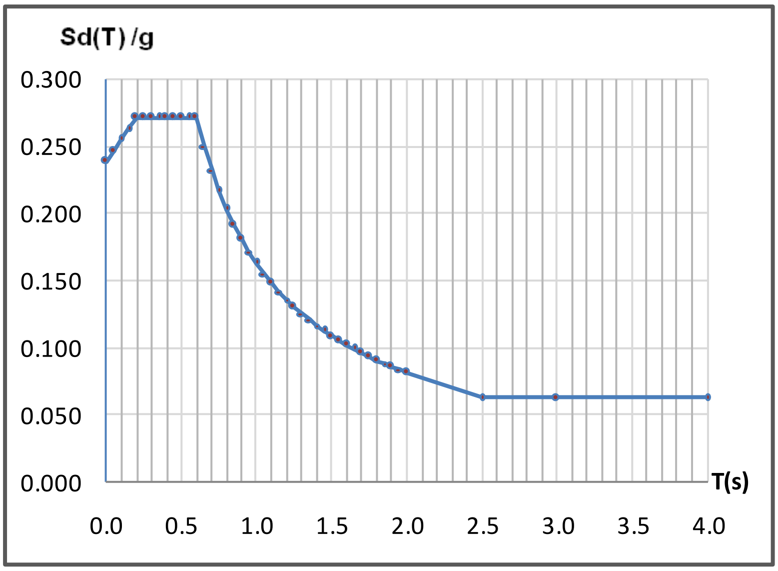
Primary Parameter	
Zone	2
Ground Type	C

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.238
	0.05	0.247
	0.10	0.255
	0.15	0.263
TB ←	0.20	0.271
	0.25	0.271
	0.30	0.271
	0.35	0.271
	0.40	0.271
	0.45	0.271
	0.50	0.271
	0.55	0.271
TC ←	0.60	0.271
	0.65	0.250
	0.70	0.232
	0.75	0.217
	0.80	0.203
	0.85	0.191
	0.90	0.181
	0.95	0.171
	1.00	0.163
	1.05	0.155
	1.10	0.148
	1.15	0.141
	1.20	0.135
	1.25	0.130
	1.30	0.125
	1.35	0.120
	1.40	0.116
	1.45	0.112
	1.50	0.108
	1.55	0.105
	1.60	0.102
	1.65	0.099
	1.70	0.096
	1.75	0.093
	1.80	0.090
	1.85	0.088
	1.90	0.086
	1.95	0.083
TD ←	2.00	0.081
	2.50	0.062
	3.00	0.062
	4.00	0.062
	5.00	0.062
	6.00	0.062
	8.00	0.062
	10.00	0.062

Max Zone2 $a_g = 0.311[g]$, Ground Type D, Response Spectrum Type I

Parameter Independent	
ag	0.311
S	1.350
TB	0.200
TC	0.800
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

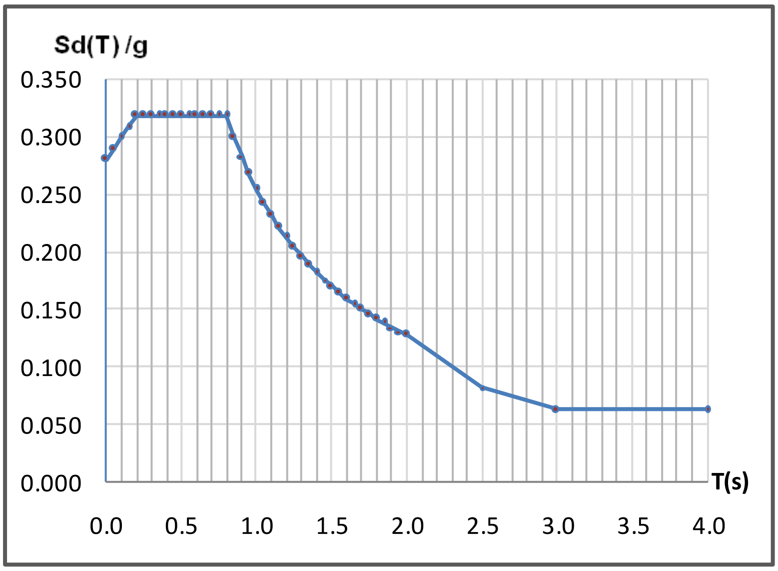
Primary Parameter	
Zone	2
Ground Type	D

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \; ; \; \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.280
	0.05	0.289
	0.10	0.299
	0.15	0.309
TB ←	0.20	0.318
	0.25	0.318
	0.30	0.318
	0.35	0.318
	0.40	0.318
	0.45	0.318
	0.50	0.318
	0.55	0.318
	0.60	0.318
	0.65	0.318
	0.70	0.318
	0.75	0.318
TC ←	0.80	0.318
	0.85	0.299
	0.90	0.283
	0.95	0.268
	1.00	0.254
	1.05	0.242
	1.10	0.231
	1.15	0.221
	1.20	0.212
	1.25	0.204
	1.30	0.196
	1.35	0.188
	1.40	0.182
	1.45	0.175
	1.50	0.170
	1.55	0.164
	1.60	0.159
	1.65	0.154
	1.70	0.150
	1.75	0.145
	1.80	0.141
	1.85	0.138
	1.90	0.134
	1.95	0.130
TD ←	2.00	0.127
	2.50	0.081
	3.00	0.062
	4.00	0.062
	5.00	0.062
	6.00	0.062
	8.00	0.062
	10.00	0.062

Max Zone2 $a_g = 0.311[g]$, Ground Type E, Response Spectrum Type I

Parameter Independent	
ag	0.311
S	1.400
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	2
Ground Type	E

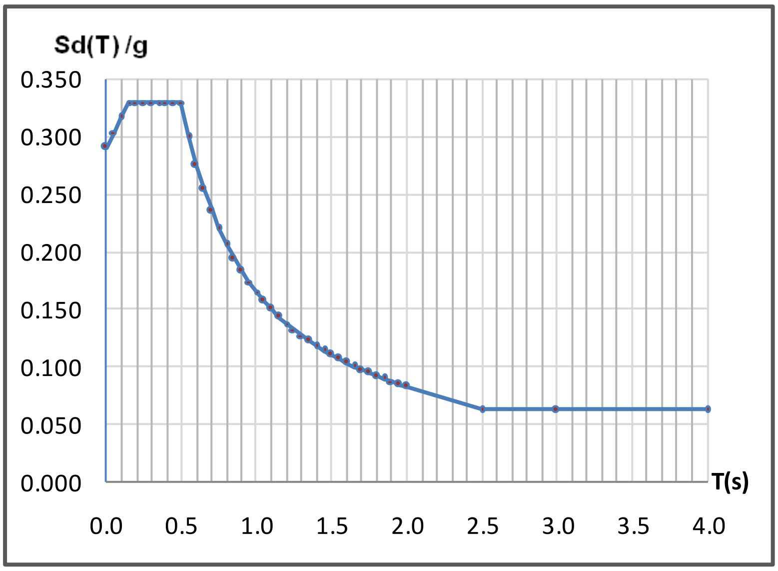
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.290
	0.05	0.303
	0.10	0.317
TB ←	0.15	0.330
	0.20	0.330
	0.25	0.330
	0.30	0.330
	0.35	0.330
	0.40	0.330
	0.45	0.330
TC ←	0.50	0.330
	0.55	0.300
	0.60	0.275
	0.65	0.254
	0.70	0.236
	0.75	0.220
	0.80	0.206
	0.85	0.194
	0.90	0.183
	0.95	0.174
	1.00	0.165
	1.05	0.157
	1.10	0.150
	1.15	0.143
	1.20	0.137
	1.25	0.132
	1.30	0.127
	1.35	0.122
	1.40	0.118
	1.45	0.114
	1.50	0.110
	1.55	0.106
	1.60	0.103
	1.65	0.100
	1.70	0.097
	1.75	0.094
	1.80	0.092
	1.85	0.089
	1.90	0.087
	1.95	0.085
TD ←	2.00	0.082
	2.50	0.062
	3.00	0.062
	4.00	0.062
	5.00	0.062
	6.00	0.062
	8.00	0.062
	10.00	0.062

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone3 $a_g = 0.199[g]$, Ground Type A, Response Spectrum Type I

Parameter Independent	
ag	0.199
S	1.000
TB	0.150
TC	0.400
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	3
Ground Type	A

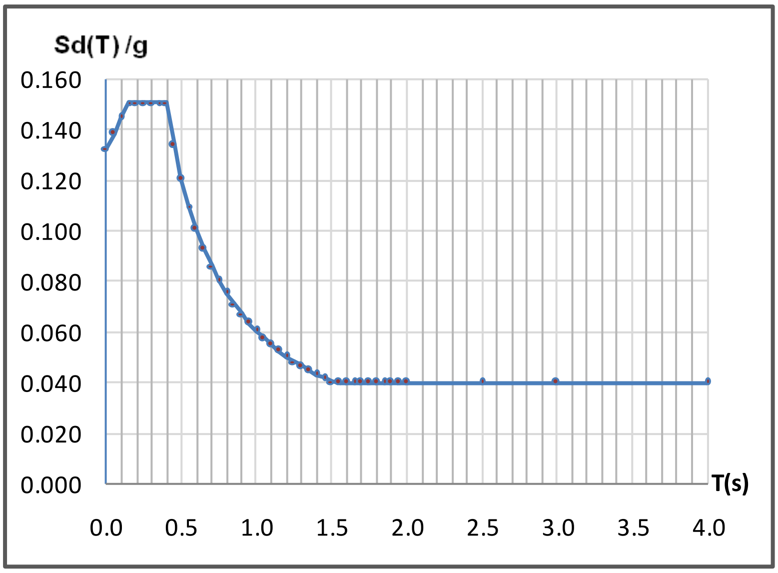
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.133
	0.05	0.139
	0.10	0.145
TB ←	0.15	0.151
	0.20	0.151
	0.25	0.151
	0.30	0.151
	0.35	0.151
TC ←	0.40	0.151
	0.45	0.134
	0.50	0.121
	0.55	0.110
	0.60	0.101
	0.65	0.093
	0.70	0.086
	0.75	0.080
	0.80	0.075
	0.85	0.071
	0.90	0.067
	0.95	0.063
	1.00	0.060
	1.05	0.057
	1.10	0.055
	1.15	0.052
	1.20	0.050
	1.25	0.048
	1.30	0.046
	1.35	0.045
	1.40	0.043
	1.45	0.042
	1.50	0.040
	1.55	0.040
	1.60	0.040
	1.65	0.040
	1.70	0.040
	1.75	0.040
	1.80	0.040
	1.85	0.040
	1.90	0.040
	1.95	0.040
TD ←	2.00	0.040
	2.50	0.040
	3.00	0.040
	4.00	0.040
	5.00	0.040
	6.00	0.040
	8.00	0.040
	10.00	0.040

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



Max Zone3 $a_g = 0.199[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.199
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q₀	3.300
Kw	1.000

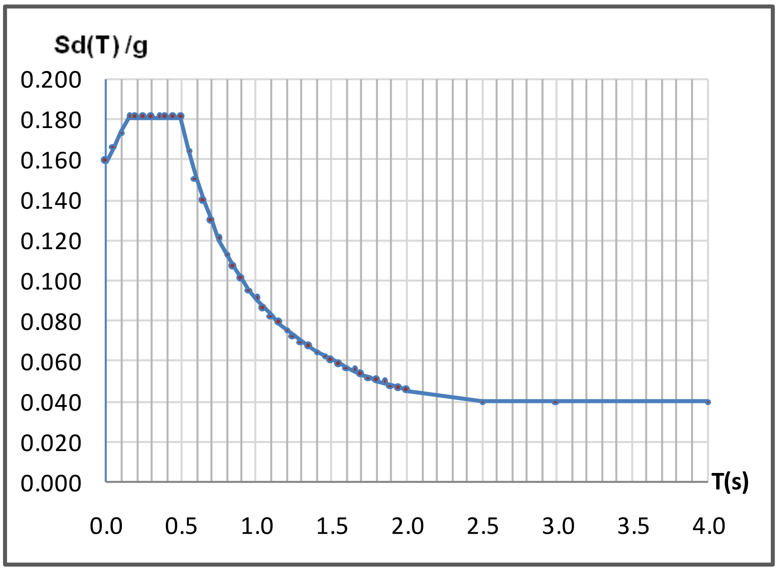
Primary Parameter	
Zone	3
Ground Type	B

Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.159
	0.05	0.166
	0.10	0.174
TB ←	0.15	0.181
	0.20	0.181
	0.25	0.181
	0.30	0.181
	0.35	0.181
	0.40	0.181
	0.45	0.181
TC ←	0.50	0.181
	0.55	0.164
	0.60	0.151
	0.65	0.139
	0.70	0.129
	0.75	0.121
	0.80	0.113
	0.85	0.106
	0.90	0.101
	0.95	0.095
	1.00	0.090
	1.05	0.086
	1.10	0.082
	1.15	0.079
	1.20	0.075
	1.25	0.072
	1.30	0.070
	1.35	0.067
	1.40	0.065
	1.45	0.062
	1.50	0.060
	1.55	0.058
	1.60	0.057
	1.65	0.055
	1.70	0.053
	1.75	0.052
	1.80	0.050
	1.85	0.049
	1.90	0.048
	1.95	0.046
TD ←	2.00	0.045
	2.50	0.040
	3.00	0.040
	4.00	0.040
	5.00	0.040
	6.00	0.040
	8.00	0.040
	10.00	0.040

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] \; ; \; \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \; ; \; \beta \times a_g\right\}$$
Notes: See EN1998-1(3.2.2)



Max Zone3 $a_g = 0.199[g]$, Ground Type C, Response Spectrum Type I

Parameter Independent	
ag	0.199
S	1.150
TB	0.200
TC	0.600
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

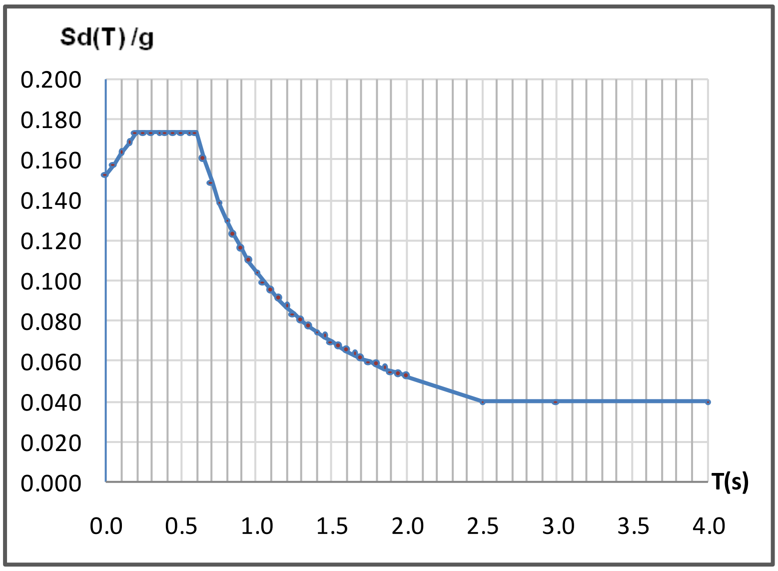
Primary Parameter	
Zone	3
Ground Type	C

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.153
	0.05	0.158
	0.10	0.163
	0.15	0.168
TB ←	0.20	0.173
	0.25	0.173
	0.30	0.173
	0.35	0.173
	0.40	0.173
	0.45	0.173
	0.50	0.173
	0.55	0.173
TC ←	0.60	0.173
	0.65	0.160
	0.70	0.149
	0.75	0.139
	0.80	0.130
	0.85	0.122
	0.90	0.116
	0.95	0.109
	1.00	0.104
	1.05	0.099
	1.10	0.095
	1.15	0.090
	1.20	0.087
	1.25	0.083
	1.30	0.080
	1.35	0.077
	1.40	0.074
	1.45	0.072
	1.50	0.069
	1.55	0.067
	1.60	0.065
	1.65	0.063
	1.70	0.061
	1.75	0.059
	1.80	0.058
	1.85	0.056
	1.90	0.055
	1.95	0.053
TD ←	2.00	0.052
	2.50	0.040
	3.00	0.040
	4.00	0.040
	5.00	0.040
	6.00	0.040
	8.00	0.040
	10.00	0.040

Max Zone3 $a_g = 0.199[g]$, Ground Type D, Response Spectrum Type I

Parameter Independent	
ag	0.199
S	1.350
TB	0.200
TC	0.800
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	3
Ground Type	D

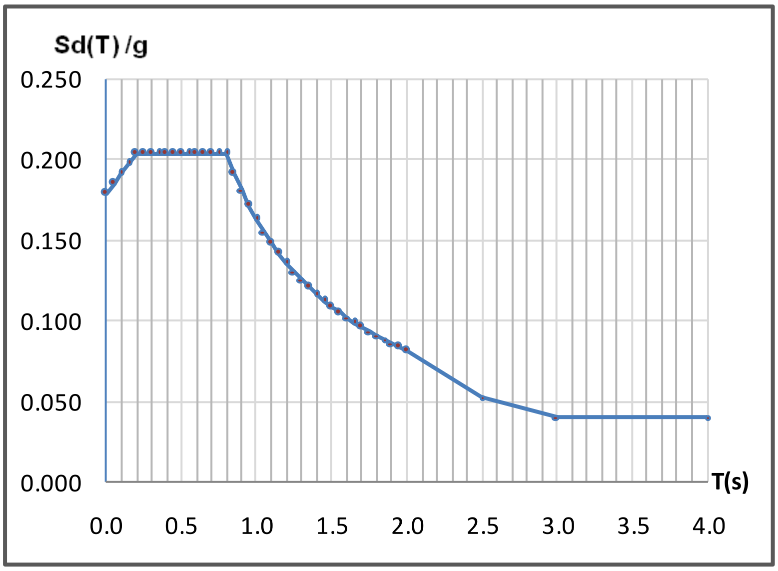
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.179
	0.05	0.185
	0.10	0.191
	0.15	0.197
TB ←	0.20	0.204
	0.25	0.204
	0.30	0.204
	0.35	0.204
	0.40	0.204
	0.45	0.204
	0.50	0.204
	0.55	0.204
	0.60	0.204
	0.65	0.204
	0.70	0.204
	0.75	0.204
TC ←	0.80	0.204
	0.85	0.192
	0.90	0.181
	0.95	0.171
	1.00	0.163
	1.05	0.155
	1.10	0.148
	1.15	0.142
	1.20	0.136
	1.25	0.130
	1.30	0.125
	1.35	0.121
	1.40	0.116
	1.45	0.112
	1.50	0.109
	1.55	0.105
	1.60	0.102
	1.65	0.099
	1.70	0.096
	1.75	0.093
	1.80	0.090
	1.85	0.088
	1.90	0.086
	1.95	0.083
TD ←	2.00	0.081
	2.50	0.052
	3.00	0.040
	4.00	0.040
	5.00	0.040
	6.00	0.040
	8.00	0.040
	10.00	0.040

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] \;;\; \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] \;;\; \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone3 $a_g = 0.199[g]$, Ground Type E, Response Spectrum Type I

Parameter Independent	
ag	0.199
S	1.400
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	3
Ground Type	E

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

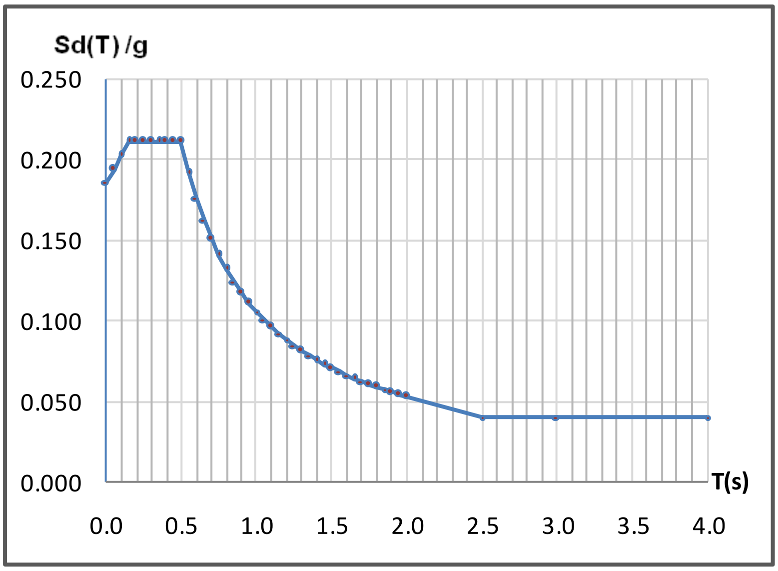
$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$

$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$

$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$

$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.186
	0.05	0.194
	0.10	0.203
TB ←	0.15	0.211
	0.20	0.211
	0.25	0.211
	0.30	0.211
	0.35	0.211
	0.40	0.211
	0.45	0.211
TC ←	0.50	0.211
	0.55	0.192
	0.60	0.176
	0.65	0.162
	0.70	0.151
	0.75	0.141
	0.80	0.132
	0.85	0.124
	0.90	0.117
	0.95	0.111
	1.00	0.106
	1.05	0.101
	1.10	0.096
	1.15	0.092
	1.20	0.088
	1.25	0.084
	1.30	0.081
	1.35	0.078
	1.40	0.075
	1.45	0.073
	1.50	0.070
	1.55	0.068
	1.60	0.066
	1.65	0.064
	1.70	0.062
	1.75	0.060
	1.80	0.059
	1.85	0.057
	1.90	0.056
	1.95	0.054
TD ←	2.00	0.053
	2.50	0.040
	3.00	0.040
	4.00	0.040
	5.00	0.040
	6.00	0.040
	8.00	0.040
	10.00	0.040

Max Zone4 $a_g = 0.056[g]$, Ground Type A, Response Spectrum Type I

Parameter Independent	
ag	0.056
S	1.000
TB	0.150
TC	0.400
TD	2.000
β	0.200
q◦	3.300
Kw	1.000

Primary Parameter	
Zone	4
Ground Type	A

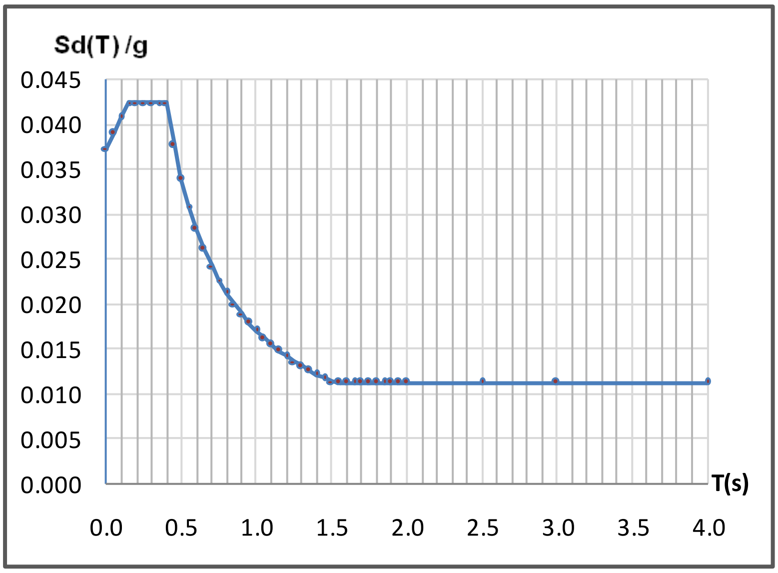
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.037
	0.05	0.039
	0.10	0.041
TB ←	0.15	0.042
	0.20	0.042
	0.25	0.042
	0.30	0.042
	0.35	0.042
TC ←	0.40	0.042
	0.45	0.038
	0.50	0.034
	0.55	0.031
	0.60	0.028
	0.65	0.026
	0.70	0.024
	0.75	0.023
	0.80	0.021
	0.85	0.020
	0.90	0.019
	0.95	0.018
	1.00	0.017
	1.05	0.016
	1.10	0.015
	1.15	0.015
	1.20	0.014
	1.25	0.014
	1.30	0.013
	1.35	0.013
	1.40	0.012
	1.45	0.012
	1.50	0.011
	1.55	0.011
	1.60	0.011
	1.65	0.011
	1.70	0.011
	1.75	0.011
	1.80	0.011
	1.85	0.011
	1.90	0.011
	1.95	0.011
TD ←	2.00	0.011
	2.50	0.011
	3.00	0.011
	4.00	0.011
	5.00	0.011
	6.00	0.011
	8.00	0.011
	10.00	0.011

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



Max Zone4 $a_g = 0.056[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.056
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

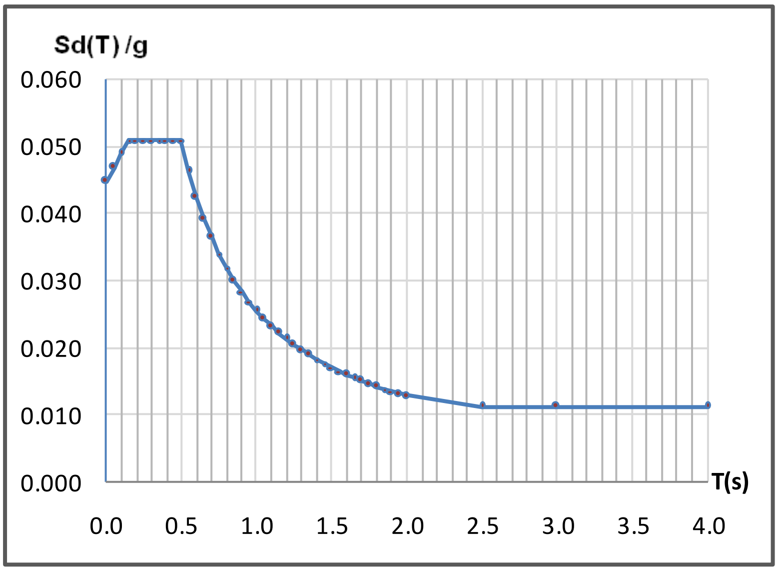
Primary Parameter	
Zone	4
Ground Type	B

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right] ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right] ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.045
	0.05	0.047
	0.10	0.049
TB ←	0.15	0.051
	0.20	0.051
	0.25	0.051
	0.30	0.051
	0.35	0.051
	0.40	0.051
	0.45	0.051
TC ←	0.50	0.051
	0.55	0.046
	0.60	0.042
	0.65	0.039
	0.70	0.036
	0.75	0.034
	0.80	0.032
	0.85	0.030
	0.90	0.028
	0.95	0.027
	1.00	0.025
	1.05	0.024
	1.10	0.023
	1.15	0.022
	1.20	0.021
	1.25	0.020
	1.30	0.020
	1.35	0.019
	1.40	0.018
	1.45	0.018
	1.50	0.017
	1.55	0.016
	1.60	0.016
	1.65	0.015
	1.70	0.015
	1.75	0.015
	1.80	0.014
	1.85	0.014
	1.90	0.013
	1.95	0.013
TD ←	2.00	0.013
	2.50	0.011
	3.00	0.011
	4.00	0.011
	5.00	0.011
	6.00	0.011
	8.00	0.011
	10.00	0.011

Max Zone4 $a_g = 0.056[g]$, Ground Type C, Response Spectrum Type I

Parameter Independent	
ag	0.056
S	1.150
TB	0.200
TC	0.600
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	4
Ground Type	C

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

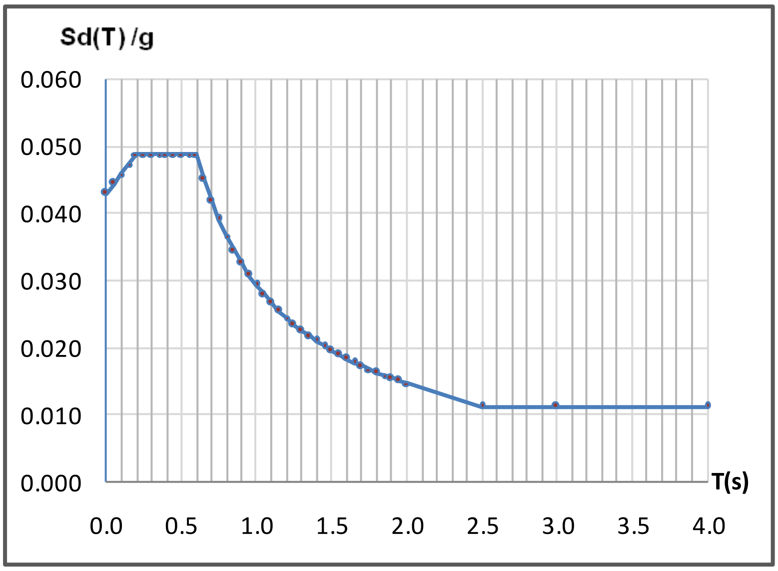
$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$

$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$

$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\}$$

$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2]\ ;\ \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.043
	0.05	0.044
	0.10	0.046
	0.15	0.047
TB ←	0.20	0.049
	0.25	0.049
	0.30	0.049
	0.35	0.049
	0.40	0.049
	0.45	0.049
	0.50	0.049
	0.55	0.049
TC ←	0.60	0.049
	0.65	0.045
	0.70	0.042
	0.75	0.039
	0.80	0.037
	0.85	0.034
	0.90	0.033
	0.95	0.031
	1.00	0.029
	1.05	0.028
	1.10	0.027
	1.15	0.025
	1.20	0.024
	1.25	0.023
	1.30	0.023
	1.35	0.022
	1.40	0.021
	1.45	0.020
	1.50	0.020
	1.55	0.019
	1.60	0.018
	1.65	0.018
	1.70	0.017
	1.75	0.017
	1.80	0.016
	1.85	0.016
	1.90	0.015
	1.95	0.015
TD ←	2.00	0.015
	2.50	0.011
	3.00	0.011
	4.00	0.011
	5.00	0.011
	6.00	0.011
	8.00	0.011
	10.00	0.011

Max Zone4 $a_g = 0.056[g]$, Ground Type D, Response Spectrum Type I

Parameter Independent	
ag	0.056
S	1.350
TB	0.200
TC	0.800
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

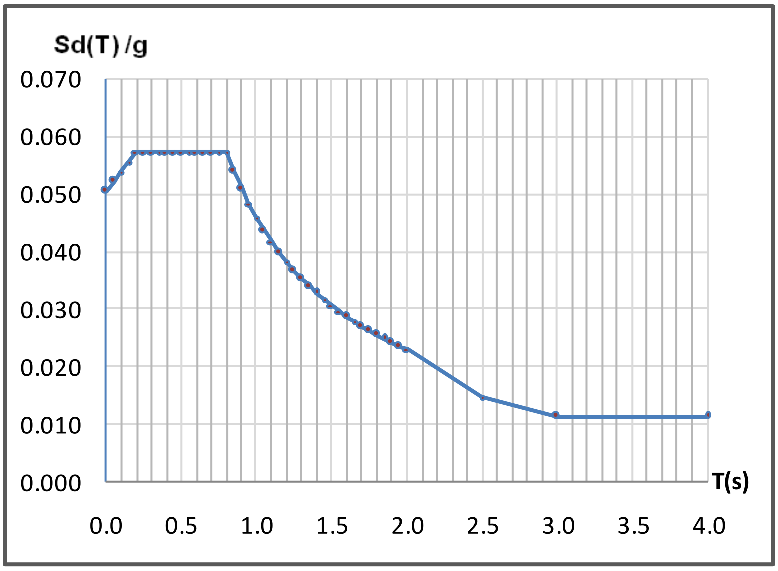
Primary Parameter	
Zone	4
Ground Type	D

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.050
	0.05	0.052
	0.10	0.054
	0.15	0.056
TB ←	0.20	0.057
	0.25	0.057
	0.30	0.057
	0.35	0.057
	0.40	0.057
	0.45	0.057
	0.50	0.057
	0.55	0.057
	0.60	0.057
	0.65	0.057
	0.70	0.057
	0.75	0.057
TC ←	0.80	0.057
	0.85	0.054
	0.90	0.051
	0.95	0.048
	1.00	0.046
	1.05	0.044
	1.10	0.042
	1.15	0.040
	1.20	0.038
	1.25	0.037
	1.30	0.035
	1.35	0.034
	1.40	0.033
	1.45	0.032
	1.50	0.031
	1.55	0.030
	1.60	0.029
	1.65	0.028
	1.70	0.027
	1.75	0.026
	1.80	0.025
	1.85	0.025
	1.90	0.024
	1.95	0.023
TD ←	2.00	0.023
	2.50	0.015
	3.00	0.011
	4.00	0.011
	5.00	0.011
	6.00	0.011
	8.00	0.011
	10.00	0.011

Max Zone4 $a_g = 0.056[g]$, Ground Type E, Response Spectrum Type I

Parameter Independent	
ag	0.056
S	1.400
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	4
Ground Type	E

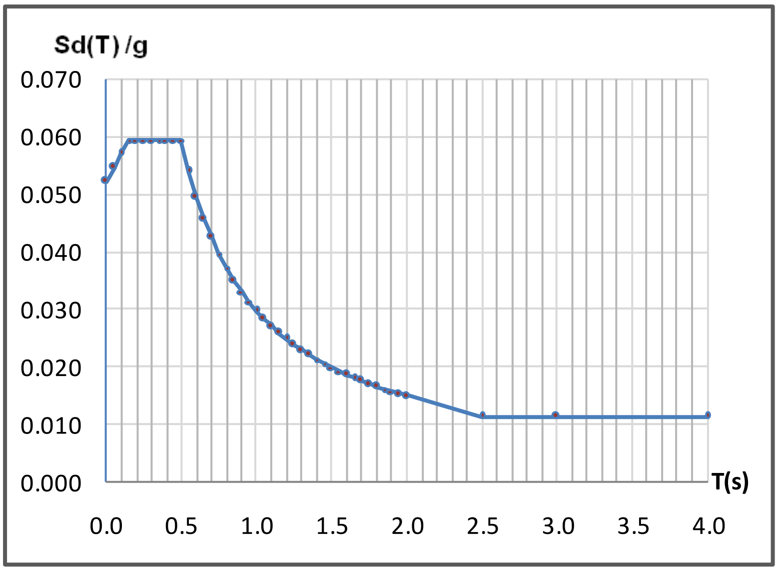
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.052
	0.05	0.055
	0.10	0.057
TB ←	0.15	0.059
	0.20	0.059
	0.25	0.059
	0.30	0.059
	0.35	0.059
	0.40	0.059
	0.45	0.059
TC ←	0.50	0.059
	0.55	0.054
	0.60	0.049
	0.65	0.046
	0.70	0.042
	0.75	0.040
	0.80	0.037
	0.85	0.035
	0.90	0.033
	0.95	0.031
	1.00	0.030
	1.05	0.028
	1.10	0.027
	1.15	0.026
	1.20	0.025
	1.25	0.024
	1.30	0.023
	1.35	0.022
	1.40	0.021
	1.45	0.020
	1.50	0.020
	1.55	0.019
	1.60	0.019
	1.65	0.018
	1.70	0.017
	1.75	0.017
	1.80	0.016
	1.85	0.016
	1.90	0.016
	1.95	0.015
TD ←	2.00	0.015
	2.50	0.011
	3.00	0.011
	4.00	0.011
	5.00	0.011
	6.00	0.011
	8.00	0.011
	10.00	0.011

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2]\ ;\ \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



Max Zone5 $a_g = 0.043[g]$, Ground Type A, Response Spectrum Type I

Parameter Independent	
ag	0.043
S	1.000
TB	0.150
TC	0.400
TD	2.000
β	0.200
q◦	3.300
Kw	1.000

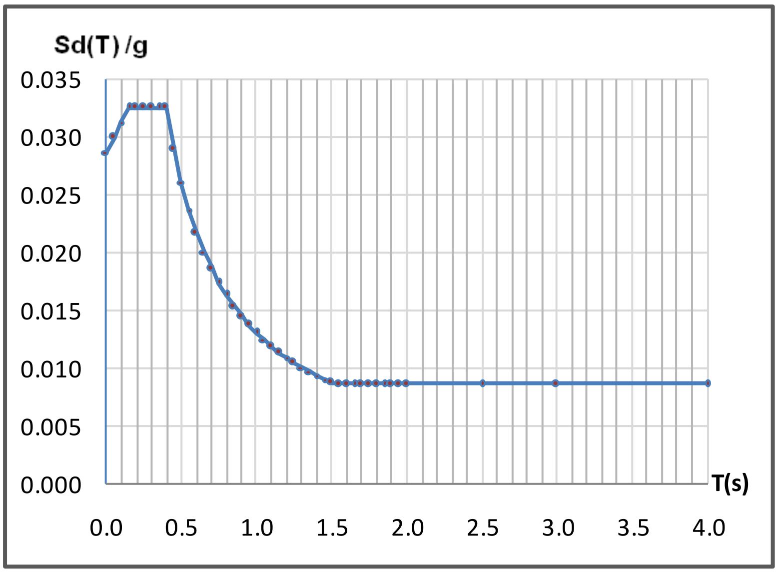
Primary Parameter	
Zone	5
Ground Type	A

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.029
	0.05	0.030
	0.10	0.031
TB ←	0.15	0.033
	0.20	0.033
	0.25	0.033
	0.30	0.033
	0.35	0.033
TC ←	0.40	0.033
	0.45	0.029
	0.50	0.026
	0.55	0.024
	0.60	0.022
	0.65	0.020
	0.70	0.019
	0.75	0.017
	0.80	0.016
	0.85	0.015
	0.90	0.014
	0.95	0.014
	1.00	0.013
	1.05	0.012
	1.10	0.012
	1.15	0.011
	1.20	0.011
	1.25	0.010
	1.30	0.010
	1.35	0.010
	1.40	0.009
	1.45	0.009
	1.50	0.009
	1.55	0.009
	1.60	0.009
	1.65	0.009
	1.70	0.009
	1.75	0.009
	1.80	0.009
	1.85	0.009
	1.90	0.009
	1.95	0.009
TD ←	2.00	0.009
	2.50	0.009
	3.00	0.009
	4.00	0.009
	5.00	0.009
	6.00	0.009
	8.00	0.009
	10.00	0.009

Max Zone5 $a_g = 0.043[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.043
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	5
Ground Type	B

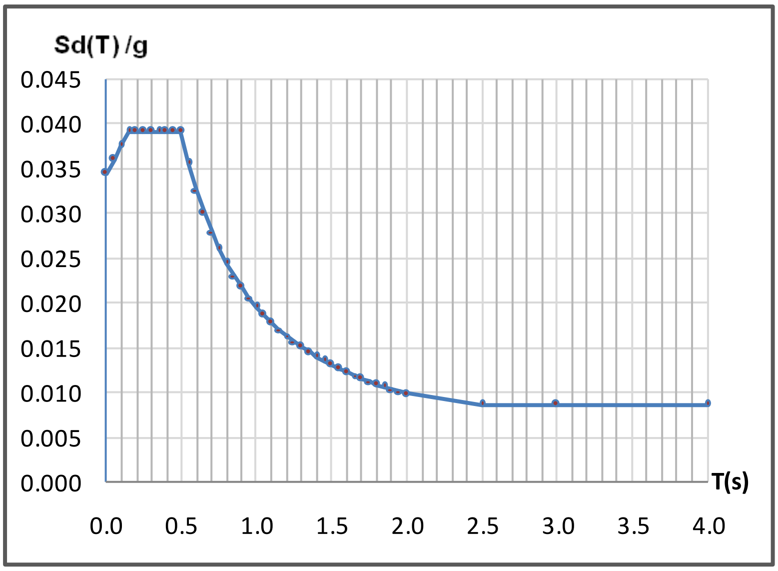
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.034
	0.05	0.036
	0.10	0.038
TB ←	0.15	0.039
	0.20	0.039
	0.25	0.039
	0.30	0.039
	0.35	0.039
	0.40	0.039
	0.45	0.039
TC ←	0.50	0.039
	0.55	0.036
	0.60	0.033
	0.65	0.030
	0.70	0.028
	0.75	0.026
	0.80	0.024
	0.85	0.023
	0.90	0.022
	0.95	0.021
	1.00	0.020
	1.05	0.019
	1.10	0.018
	1.15	0.017
	1.20	0.016
	1.25	0.016
	1.30	0.015
	1.35	0.014
	1.40	0.014
	1.45	0.013
	1.50	0.013
	1.55	0.013
	1.60	0.012
	1.65	0.012
	1.70	0.011
	1.75	0.011
	1.80	0.011
	1.85	0.011
	1.90	0.010
	1.95	0.010
TD ←	2.00	0.010
	2.50	0.009
	3.00	0.009
	4.00	0.009
	5.00	0.009
	6.00	0.009
	8.00	0.009
	10.00	0.009

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2]\ ;\ \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



Max Zone5 $a_g = 0.043[g]$, Ground Type C, Response Spectrum Type I

Parameter Independent	
ag	0.043
S	1.150
TB	0.200
TC	0.600
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

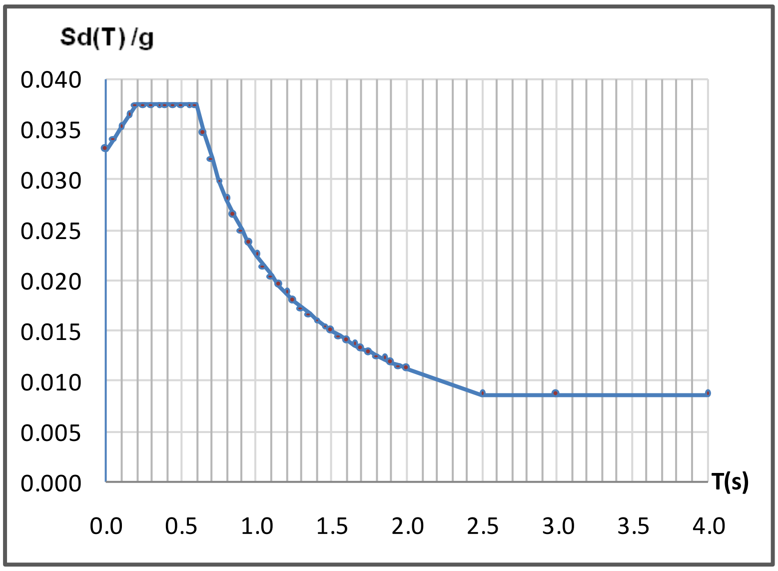
Primary Parameter	
Zone	5
Ground Type	C

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.033
	0.05	0.034
	0.10	0.035
	0.15	0.036
TB ←	0.20	0.037
	0.25	0.037
	0.30	0.037
	0.35	0.037
	0.40	0.037
	0.45	0.037
	0.50	0.037
	0.55	0.037
TC ←	0.60	0.037
	0.65	0.035
	0.70	0.032
	0.75	0.030
	0.80	0.028
	0.85	0.026
	0.90	0.025
	0.95	0.024
	1.00	0.022
	1.05	0.021
	1.10	0.020
	1.15	0.020
	1.20	0.019
	1.25	0.018
	1.30	0.017
	1.35	0.017
	1.40	0.016
	1.45	0.016
	1.50	0.015
	1.55	0.015
	1.60	0.014
	1.65	0.014
	1.70	0.013
	1.75	0.013
	1.80	0.012
	1.85	0.012
	1.90	0.012
	1.95	0.012
TD ←	2.00	0.011
	2.50	0.009
	3.00	0.009
	4.00	0.009
	5.00	0.009
	6.00	0.009
	8.00	0.009
	10.00	0.009

Max Zone5 $a_g = 0.043[g]$, Ground Type D, Response Spectrum Type I

Parameter Independent	
ag	0.043
S	1.350
TB	0.200
TC	0.800
TD	2.000
β	0.200
q。	3.300
Kw	1.000

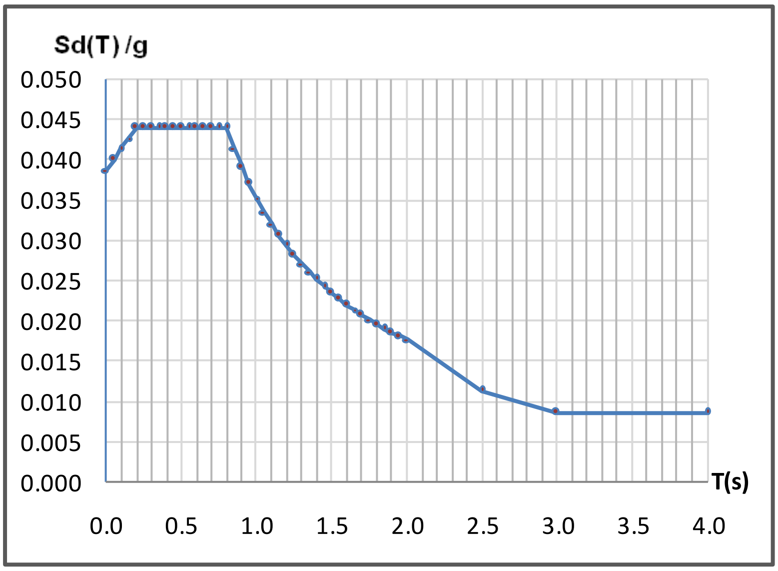
Primary Parameter	
Zone	5
Ground Type	D

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C/T\right]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.039
	0.05	0.040
	0.10	0.041
	0.15	0.043
TB ←	0.20	0.044
	0.25	0.044
	0.30	0.044
	0.35	0.044
	0.40	0.044
	0.45	0.044
	0.50	0.044
	0.55	0.044
	0.60	0.044
	0.65	0.044
	0.70	0.044
	0.75	0.044
TC ←	0.80	0.044
	0.85	0.041
	0.90	0.039
	0.95	0.037
	1.00	0.035
	1.05	0.034
	1.10	0.032
	1.15	0.031
	1.20	0.029
	1.25	0.028
	1.30	0.027
	1.35	0.026
	1.40	0.025
	1.45	0.024
	1.50	0.023
	1.55	0.023
	1.60	0.022
	1.65	0.021
	1.70	0.021
	1.75	0.020
	1.80	0.020
	1.85	0.019
	1.90	0.019
	1.95	0.018
TD ←	2.00	0.018
	2.50	0.011
	3.00	0.009
	4.00	0.009
	5.00	0.009
	6.00	0.009
	8.00	0.009
	10.00	0.009

Max Zone5 $a_g = 0.043[g]$, Ground Type E, Response Spectrum Type I

Parameter Independent	
ag	0.043
S	1.400
TB	0.150
TC	0.500
TD	2.000
β	0.200
q₀	3.300
Kw	1.000

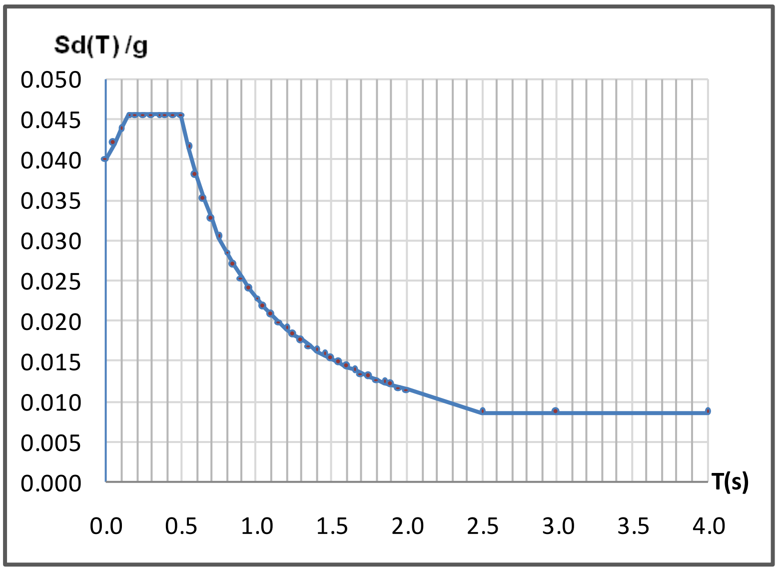
Primary Parameter	
Zone	5
Ground Type	E

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.040
	0.05	0.042
	0.10	0.044
TB ←	0.15	0.046
	0.20	0.046
	0.25	0.046
	0.30	0.046
	0.35	0.046
	0.40	0.046
	0.45	0.046
TC ←	0.50	0.046
	0.55	0.041
	0.60	0.038
	0.65	0.035
	0.70	0.033
	0.75	0.030
	0.80	0.029
	0.85	0.027
	0.90	0.025
	0.95	0.024
	1.00	0.023
	1.05	0.022
	1.10	0.021
	1.15	0.020
	1.20	0.019
	1.25	0.018
	1.30	0.018
	1.35	0.017
	1.40	0.016
	1.45	0.016
	1.50	0.015
	1.55	0.015
	1.60	0.014
	1.65	0.014
	1.70	0.013
	1.75	0.013
	1.80	0.013
	1.85	0.012
	1.90	0.012
	1.95	0.012
TD ←	2.00	0.011
	2.50	0.009
	3.00	0.009
	4.00	0.009
	5.00	0.009
	6.00	0.009
	8.00	0.009
	10.00	0.009

PGA of time-history curve 1: $a_g = 0.2594[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.260
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q。	3.300
Kw	1.000

Primary Parameter	
Zone	1~2
Ground Type	B

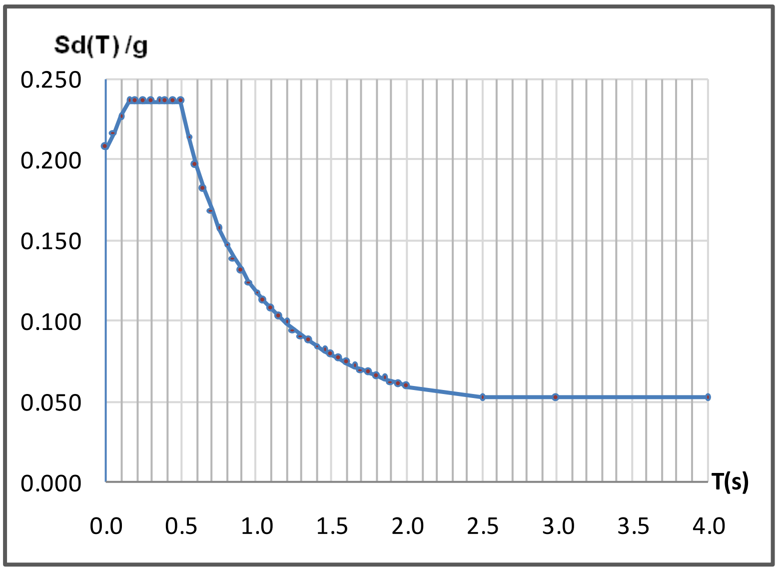
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.208
	0.05	0.217
	0.10	0.226
TB ←	0.15	0.236
	0.20	0.236
	0.25	0.236
	0.30	0.236
	0.35	0.236
	0.40	0.236
	0.45	0.236
TC ←	0.50	0.236
	0.55	0.214
	0.60	0.197
	0.65	0.181
	0.70	0.168
	0.75	0.157
	0.80	0.147
	0.85	0.139
	0.90	0.131
	0.95	0.124
	1.00	0.118
	1.05	0.112
	1.10	0.107
	1.15	0.103
	1.20	0.098
	1.25	0.094
	1.30	0.091
	1.35	0.087
	1.40	0.084
	1.45	0.081
	1.50	0.079
	1.55	0.076
	1.60	0.074
	1.65	0.071
	1.70	0.069
	1.75	0.067
	1.80	0.066
	1.85	0.064
	1.90	0.062
	1.95	0.060
TD ←	2.00	0.059
	2.50	0.052
	3.00	0.052
	4.00	0.052
	5.00	0.052
	6.00	0.052
	8.00	0.052
	10.00	0.052

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \; ; \; \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \; ; \; \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



PGA of time-history curve 2: $a_g = 0.214807[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.215
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	2~3
Ground Type	B

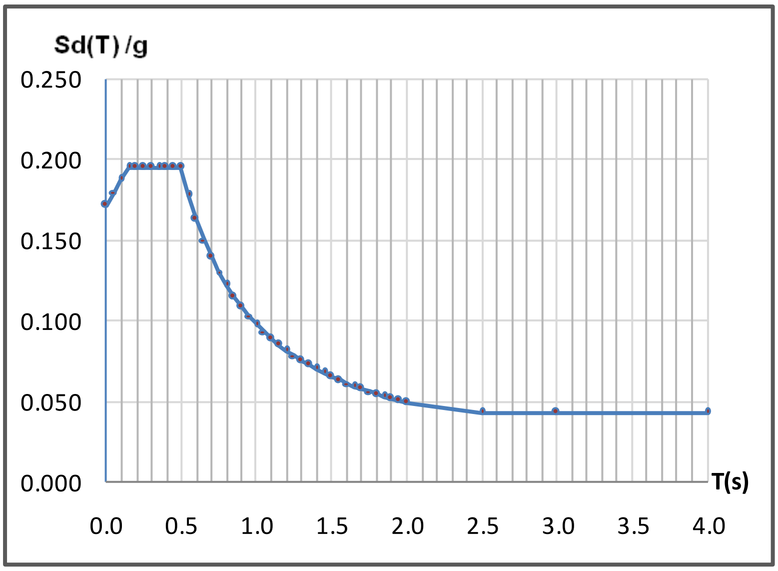
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.172
	0.05	0.180
	0.10	0.187
TB ←	0.15	0.195
	0.20	0.195
	0.25	0.195
	0.30	0.195
	0.35	0.195
	0.40	0.195
	0.45	0.195
TC ←	0.50	0.195
	0.55	0.178
	0.60	0.163
	0.65	0.150
	0.70	0.139
	0.75	0.130
	0.80	0.122
	0.85	0.115
	0.90	0.108
	0.95	0.103
	1.00	0.098
	1.05	0.093
	1.10	0.089
	1.15	0.085
	1.20	0.081
	1.25	0.078
	1.30	0.075
	1.35	0.072
	1.40	0.070
	1.45	0.067
	1.50	0.065
	1.55	0.063
	1.60	0.061
	1.65	0.059
	1.70	0.057
	1.75	0.056
	1.80	0.054
	1.85	0.053
	1.90	0.051
	1.95	0.050
TD ←	2.00	0.049
	2.50	0.043
	3.00	0.043
	4.00	0.043
	5.00	0.043
	6.00	0.043
	8.00	0.043
	10.00	0.043

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



PGA of time-history curve 3: $a_g = 0.150032[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.150
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

Primary Parameter	
Zone	3~4
Ground Type	B

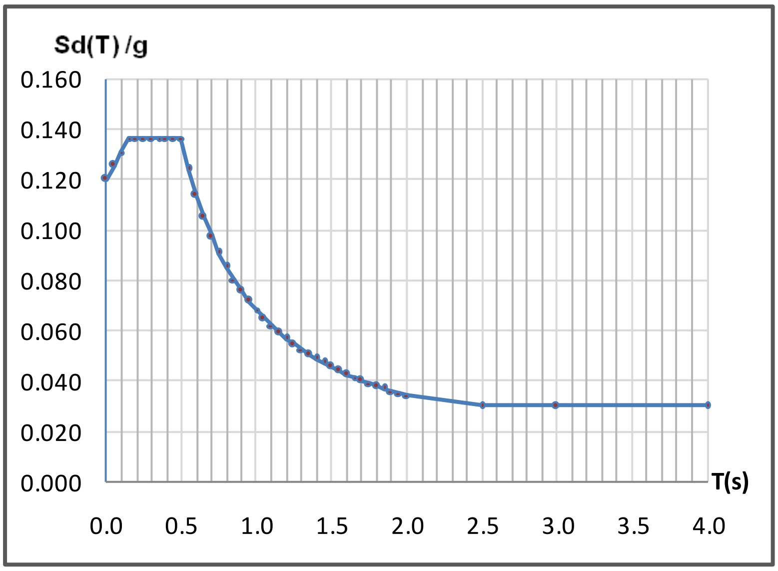
Computed Parameter	
q	3.300
Sd	see table

	T[s]	Se[g]
	0.00	0.120
	0.05	0.125
	0.10	0.131
TB ←	0.15	0.136
	0.20	0.136
	0.25	0.136
	0.30	0.136
	0.35	0.136
	0.40	0.136
	0.45	0.136
TC ←	0.50	0.136
	0.55	0.124
	0.60	0.114
	0.65	0.105
	0.70	0.097
	0.75	0.091
	0.80	0.085
	0.85	0.080
	0.90	0.076
	0.95	0.072
	1.00	0.068
	1.05	0.065
	1.10	0.062
	1.15	0.059
	1.20	0.057
	1.25	0.055
	1.30	0.052
	1.35	0.051
	1.40	0.049
	1.45	0.047
	1.50	0.045
	1.55	0.044
	1.60	0.043
	1.65	0.041
	1.70	0.040
	1.75	0.039
	1.80	0.038
	1.85	0.037
	1.90	0.036
	1.95	0.035
TD ←	2.00	0.034
	2.50	0.030
	3.00	0.030
	4.00	0.030
	5.00	0.030
	6.00	0.030
	8.00	0.030
	10.00	0.030

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot \left[2/3 + T/T_B \cdot (2.5/q - 2/3)\right]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T]\ ;\ \beta \times a_g\right\}$$
$$T_D \le T \quad S_d(T) = \max\left\{a_g \cdot S \cdot 2.5/q \cdot \left[T_C T_D/T^2\right]\ ;\ \beta \times a_g\right\}$$

Notes: See EN1998-1(3.2.2)



PGA of time-history curve 1: $a_g = 0.089134[g]$, Ground Type B, Response Spectrum Type I

Parameter Independent	
ag	0.089
S	1.200
TB	0.150
TC	0.500
TD	2.000
β	0.200
q∘	3.300
Kw	1.000

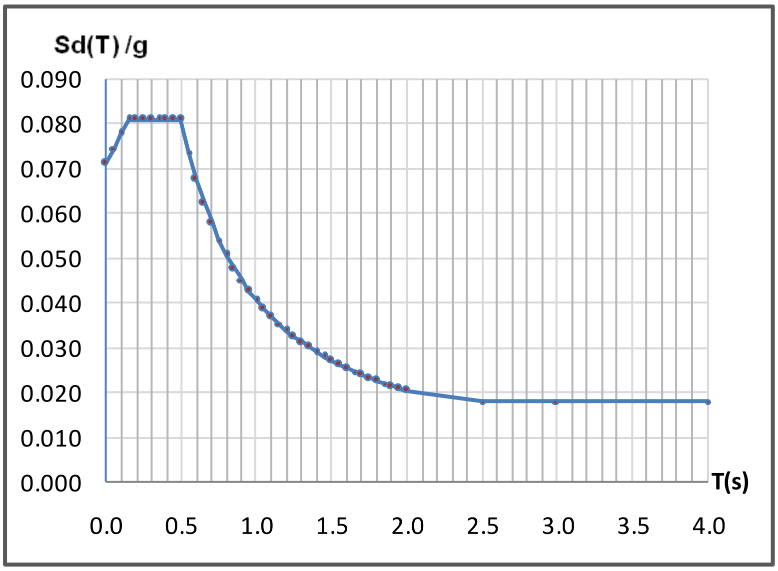
Primary Parameter	
Zone	4~5
Ground Type	B

Computed Parameter	
q	3.300
Sd	see table

$S_d(T)$ is design response spectrum, a function of T
T is the fundamental vibration period
a_g is the design ground acceleration
T_B is the lower limit of the period
T_C is the upper limit of the period
T_D beginning of the constant displacement response
S soil factor according to ground type.
β is the lower bound factor for horizontal spectrum
q behavior factor
k_w reflecting the previous failure mode
Notes: details of each parameter defines in EN1998-1

$$0 \le T \le T_B \quad S_d(T) = a_g \cdot S \cdot [2/3 + T/T_B \cdot (2.5/q - 2/3)]$$
$$T_B \le T \le T_C \quad S_d(T) = a_g \cdot S \cdot 2.5/q$$
$$T_C \le T \le T_D \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C/T] \ ; \ \beta \times a_g\}$$
$$T_D \le T \quad S_d(T) = \max\{a_g \cdot S \cdot 2.5/q \cdot [T_C T_D/T^2] \ ; \ \beta \times a_g\}$$

Notes: See EN1998-1(3.2.2)



	T[s]	Se[g]
	0.00	0.071
	0.05	0.075
	0.10	0.078
TB ←	0.15	0.081
	0.20	0.081
	0.25	0.081
	0.30	0.081
	0.35	0.081
	0.40	0.081
	0.45	0.081
TC ←	0.50	0.081
	0.55	0.074
	0.60	0.068
	0.65	0.062
	0.70	0.058
	0.75	0.054
	0.80	0.051
	0.85	0.048
	0.90	0.045
	0.95	0.043
	1.00	0.041
	1.05	0.039
	1.10	0.037
	1.15	0.035
	1.20	0.034
	1.25	0.032
	1.30	0.031
	1.35	0.030
	1.40	0.029
	1.45	0.028
	1.50	0.027
	1.55	0.026
	1.60	0.025
	1.65	0.025
	1.70	0.024
	1.75	0.023
	1.80	0.023
	1.85	0.022
	1.90	0.021
	1.95	0.021
TD ←	2.00	0.020
	2.50	0.018
	3.00	0.018
	4.00	0.018
	5.00	0.018
	6.00	0.018
	8.00	0.018
	10.00	0.018

APPENDIX B

INPUT FILE OF DIANA FINITE ELEMENT MODEL (LINEAR)

CASE 1)

```
FEMGEN MODEL      : NON_S_P
ANALYSIS TYPE     : Structural 2D
MODEL DESCRIPTION : 1F1B1Pfp
'UNITS'
LENGTH   M
TIME     SEC
TEMPER   CELSIU
FORCE    N
'COORDINATES' DI=2
    1    0.000000E+00    0.000000E+00
    2    0.000000E+00    4.000000E-01
    3    0.000000E+00    8.000001E-01
    4    0.000000E+00    1.200000E+00
    5    0.000000E+00    1.600000E+00
    6    0.000000E+00    2.000000E+00
    7    0.000000E+00    2.400000E+00
    8    0.000000E+00    2.800000E+00
    9    0.000000E+00    3.200000E+00
   10    0.000000E+00    3.600000E+00
   11    0.000000E+00    4.000000E+00
   12    0.000000E+00    4.400000E+00
   13    0.000000E+00    4.800000E+00
   14    0.000000E+00    5.200000E+00
   15    0.000000E+00    5.600000E+00
   16    0.000000E+00    6.000000E+00
   17    0.000000E+00    6.400000E+00
   18    0.000000E+00    6.800000E+00
   19    0.000000E+00    7.200000E+00
   20    1.200000E+01    7.200000E+00
   21    1.200000E+01    6.800000E+00
   22    1.200000E+01    6.400000E+00
   23    1.200000E+01    6.000000E+00
   24    1.200000E+01    5.600000E+00
   25    1.200000E+01    5.200000E+00
   26    1.200000E+01    4.800000E+00
   27    1.200000E+01    4.400000E+00
   28    1.200000E+01    4.000000E+00
   29    1.200000E+01    3.600000E+00
   30    1.200000E+01    3.200000E+00
   31    1.200000E+01    2.800000E+00
   32    1.200000E+01    2.400000E+00
   33    1.200000E+01    2.000000E+00
   34    1.200000E+01    1.600000E+00
   35    1.200000E+01    1.200000E+00
   36    1.200000E+01    8.000000E-01
   37    1.200000E+01    4.000001E-01
   38    1.200000E+01    0.000000E+00
   39    0.000000E+00    7.200000E+00
   40    6.000000E-01    7.200000E+00
   41    1.200000E+00    7.200000E+00
   42    1.800000E+00    7.200000E+00
   43    2.400000E+00    7.200000E+00
   44    3.000000E+00    7.200000E+00
```

45	3.600000E+00	7.200000E+00
46	4.200000E+00	7.200000E+00
47	4.800000E+00	7.200000E+00
48	5.400000E+00	7.200000E+00
49	6.000000E+00	7.200000E+00
50	6.600000E+00	7.200000E+00
51	7.200000E+00	7.200000E+00
52	7.800000E+00	7.200000E+00
53	8.400000E+00	7.200000E+00
54	9.000000E+00	7.200000E+00
55	9.600000E+00	7.200000E+00
56	1.020000E+01	7.200000E+00
57	1.080000E+01	7.200000E+00
58	1.140000E+01	7.200000E+00
59	1.200000E+01	7.200000E+00
60	0.000000E+00	2.400000E+00
61	0.000000E+00	2.700000E+00
62	0.000000E+00	3.000000E+00
63	0.000000E+00	3.300000E+00
64	0.000000E+00	3.600000E+00
65	0.000000E+00	3.900000E+00
66	0.000000E+00	4.200000E+00
67	0.000000E+00	4.500000E+00
68	0.000000E+00	4.800000E+00
69	1.500000E+00	2.400000E+00
70	1.500000E+00	2.700000E+00
71	1.500000E+00	3.000000E+00
72	1.500000E+00	3.300000E+00
73	1.500000E+00	3.600000E+00
74	1.500000E+00	3.900000E+00
75	1.500000E+00	4.200000E+00
76	1.500000E+00	4.500000E+00
77	1.500000E+00	4.800000E+00
78	3.000000E+00	2.400000E+00
79	3.000000E+00	2.700000E+00
80	3.000000E+00	3.000000E+00
81	3.000000E+00	3.300000E+00
82	3.000000E+00	3.600000E+00
83	3.000000E+00	3.900000E+00
84	3.000000E+00	4.200000E+00
85	3.000000E+00	4.500000E+00
86	3.000000E+00	4.800000E+00
87	4.500000E+00	2.400000E+00
88	4.500000E+00	2.700000E+00
89	4.500000E+00	3.000000E+00
90	4.500000E+00	3.300000E+00
91	4.500000E+00	3.600000E+00
92	4.500000E+00	3.900000E+00
93	4.500000E+00	4.200000E+00
94	4.500000E+00	4.500000E+00
95	4.500000E+00	4.800000E+00
96	6.000000E+00	2.400000E+00
97	6.000000E+00	2.700000E+00
98	6.000000E+00	3.000000E+00
99	6.000000E+00	3.300000E+00
100	6.000000E+00	3.600000E+00
101	6.000000E+00	3.900000E+00

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   103     6.000000E+00     4.500000E+00
   104     6.000000E+00     4.800000E+00
   105     7.500000E+00     2.400000E+00
   106     7.500000E+00     2.700000E+00
   107     7.500000E+00     3.000000E+00
   108     7.500000E+00     3.300000E+00
   109     7.500000E+00     3.600000E+00
   110     7.500000E+00     3.900000E+00
   111     7.500000E+00     4.200000E+00
   112     7.500000E+00     4.500000E+00
   113     7.500000E+00     4.800000E+00
   114     9.000000E+00     2.400000E+00
   115     9.000000E+00     2.700000E+00
   116     9.000000E+00     3.000000E+00
   117     9.000000E+00     3.300000E+00
   118     9.000000E+00     3.600000E+00
   119     9.000000E+00     3.900000E+00
   120     9.000000E+00     4.200000E+00
   121     9.000000E+00     4.500000E+00
   122     9.000000E+00     4.800000E+00
   123     1.050000E+01     2.400000E+00
   124     1.050000E+01     2.700000E+00
   125     1.050000E+01     3.000000E+00
   126     1.050000E+01     3.300000E+00
   127     1.050000E+01     3.600000E+00
   128     1.050000E+01     3.900000E+00
   129     1.050000E+01     4.200000E+00
   130     1.050000E+01     4.500000E+00
   131     1.050000E+01     4.800000E+00
   132     1.200000E+01     2.400000E+00
   133     1.200000E+01     2.700000E+00
   134     1.200000E+01     3.000000E+00
   135     1.200000E+01     3.300000E+00
   136     1.200000E+01     3.600000E+00
   137     1.200000E+01     3.900000E+00
   138     1.200000E+01     4.200000E+00
   139     1.200000E+01     4.500000E+00
   140     1.200000E+01     4.800000E+00
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    2 L6BEN  2 3
    3 L6BEN  3 4
    4 L6BEN  4 5
    5 L6BEN  5 6
    6 L6BEN  6 7
    7 L6BEN  7 8
    8 L6BEN  8 9
    9 L6BEN  9 10
   10 L6BEN  10 11
   11 L6BEN  11 12
   12 L6BEN  12 13
   13 L6BEN  13 14
   14 L6BEN  14 15
   15 L6BEN  15 16
   16 L6BEN  16 17
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66 Q8MEM 70 71 80 79
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69 Q8MEM 73 74 83 82
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72 Q8MEM 76 77 86 85
73 Q8MEM 78 79 88 87

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   75 Q8MEM  80 81 90 89
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   77 Q8MEM  82 83 92 91
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   79 Q8MEM  84 85 94 93
   80 Q8MEM  85 86 95 94
   81 Q8MEM  87 88 97 96
   82 Q8MEM  88 89 98 97
   83 Q8MEM  89 90 99 98
   84 Q8MEM  90 91 100 99
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   86 Q8MEM  92 93 102 101
   87 Q8MEM  93 94 103 102
   88 Q8MEM  94 95 104 103
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  119 Q8MEM  129 130 139 138
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/ 1-36 /  3
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/ 57-120 /  1
/ 37-56 /  2
/ 1-36 /  3
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    DENSIT    1.631000E+03
  2 YOUNG     3.820000E+10
    POISON    2.000000E-01
    DENSIT    3.376122E+03
  3 YOUNG     3.820000E+10
    POISON    2.000000E-01
    DENSIT    2.548400E+03
'GEOMETRY'
  1 THICK     2.500000E-01
  2 CROSSE    3.340000E-01
    INERTI    1.684000E-02
  3 RECTAN    5.000000E-01    5.000000E-01
'GROUPS'
ELEMEN
  1 SET1 / 57-120 /
NODES
  2 SET1_N / 60-140 /
ELEMEN
  3 SET2 / 1-36 /
NODES
  4 SET2_N / 1-38 /
ELEMEN
  5 SET3 / 37-56 /
NODES
  6 SET3_N / 39-59 /
  7 FEET / 1 38 /
'SUPPORTS'
 / 1 38 /   TR     1
 / 1 38 /   RO     1
 / 1 38 /   TR     2
 / 1 38 /   RO     2
 / 1 38 /   TR     3
 / 1 38 /   RO     3
'TYINGS'
FIX TR 1
  19    39 TR 1 1.0
FIX TR 2
  19    39 TR 2 1.0
FIX TR 1
  20    59 TR 1 1.0
FIX TR 2
  20    59 TR 2 1.0
FIX TR 1
   7    60 TR 1 1.0
FIX TR 2
   7    60 TR 2 1.0
FIX TR 2
  13    68 TR 2 1.0
FIX TR 2
  26   140 TR 2 1.0
FIX TR 1
  32   132 TR 1 1.0
FIX TR 2
  32   132 TR 2 1.0
'LOADS'
CASE 1
```

```
BASE
    1    9.81
CASE 2
WEIGHT
    2    -9.81000
'DIRECTIONS'
   1   1.000000E+00   0.000000E+00   0.000000E+00
   2   0.000000E+00   1.000000E+00   0.000000E+00
   3   0.000000E+00   0.000000E+00   1.000000E+00
   'END'
```

CASE 2)

The same as case 1) only differs with 'TYINGS' modulus.

```
….
'TYINGS'
FIX TR 1
  19    39 TR 1 1.0
FIX TR 2
  19    39 TR 2 1.0
FIX TR 1
  20    59 TR 1 1.0
FIX TR 2
  20    59 TR 2 1.0
FIX TR 1
   7    60 TR 1 1.0
FIX TR 2
   7    60 TR 2 1.0
FIX TR 2
  13    68 TR 2 1.0
FIX TR 2
  26   140 TR 2 1.0
FIX TR 2
  32   132 TR 2 1.0
….
```

CASE 3)

The same as case 1) only differs with 'TYINGS' modulus.

```
….
'TYINGS'
FIX TR 1
  19    39 TR 1 1.0
FIX TR 2
  19    39 TR 2 1.0
FIX RO 3
  19    39 RO 3 1.0
FIX TR 1
  20    59 TR 1 1.0
FIX TR 2
  20    59 TR 2 1.0
FIX RO 3
  20    59 RO 3 1.0
FIX TR 1
   7    60 TR 1 1.0
FIX TR 2
   7    60 TR 2 1.0
FIX TR 2
  13    68 TR 2 1.0
```

```
FIX TR 2
  26   140 TR 2 1.0
FIX TR 1
  32   132 TR 1 1.0
FIX TR 2
  32   132 TR 2 1.0
….
```

CASE 4)

The same as case 1) only differs with ‘TYINGS’ modulus.

```
….
'TYINGS'
FIX TR 1
  19    39 TR 1 1.0
FIX TR 2
  19    39 TR 2 1.0
FIX RO 3
  19    39 RO 3 1.0
FIX TR 1
  20    59 TR 1 1.0
FIX TR 2
  20    59 TR 2 1.0
FIX RO 3
  20    59 RO 3 1.0
FIX TR 1
   7    60 TR 1 1.0
FIX TR 2
   7    60 TR 2 1.0
FIX TR 2
  13    68 TR 2 1.0
FIX TR 2
  26   140 TR 2 1.0
FIX TR 2
  32   132 TR 2 1.0
….
```

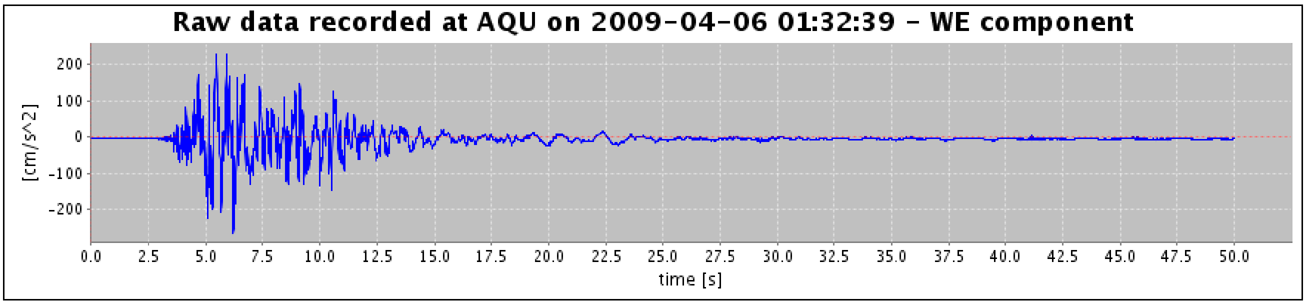
APPENDIX C

Seismic Acceleration Diagram

Date	MW	ML	Stat. Code	EC8	R epi. [km]	Corr. PGA [cm/s2]	PGV [cm/s]	PGD [cm]
2009-04-06 01:32:39	6.3	5.8	AQU	B*	6.018	301.827	29.503	8.9969
1976-09-15 03:15:18	5.9	6.1	FRC	B	17.294	258.438	9.787	1.9475
1981-01-16 00:37:45	5.2	4.6	CR3	B*	10.494	147.181	5.218	0.44277
1980-12-01 19:04:29		4.6	CR3	B*	2.213	87.440	4.363	0.51137

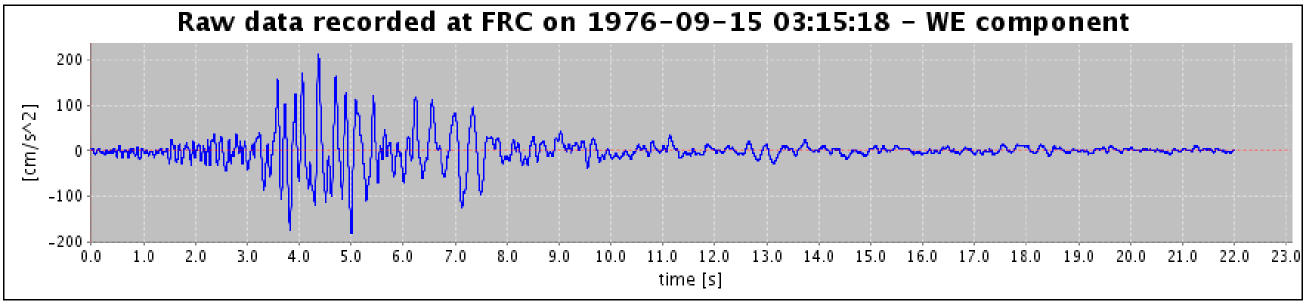
Wave Form

PGA 1 **0.25g**



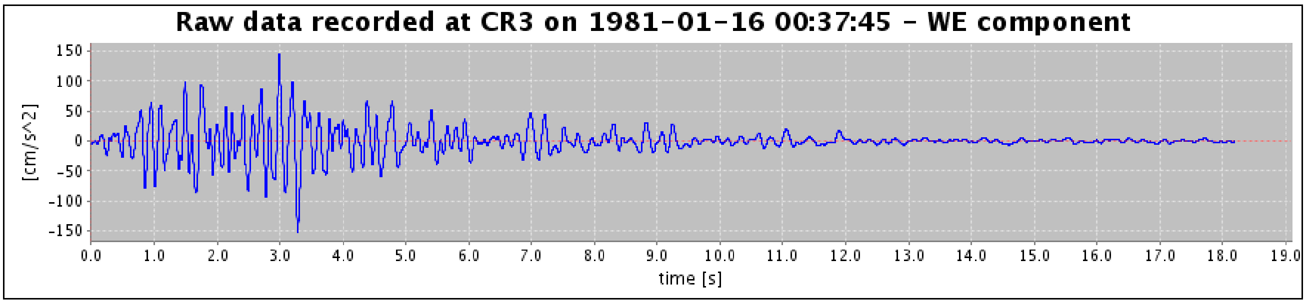
Peak at -254.4940 [cm/s2], 6.2s, Effective duration [s] is 7.69s, Approximately from 3.5s to 11s.

PGA 2 **0.215g**



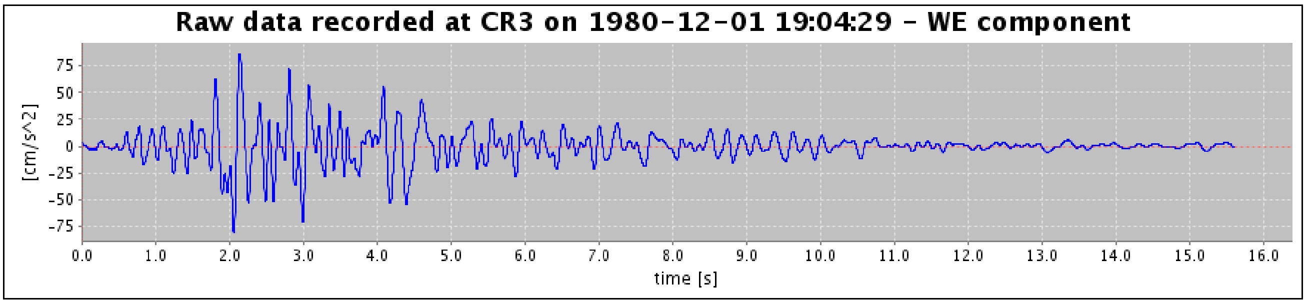
Peak at 210.726 [cm/s2], 4.37s, Effective duration [s] is 4.56s, Approximately from 3.5s to 8s.

PGA 3 **0.15g**



Peak at 147.181[cm/s2], 3.28s, Effective duration [s] is 6.245s, Approximately from 0.2s to 6.2s.

PGA 4 **0.089g**



Peak at 87.44 [cm/s2], 2.118s, Effective duration [s] is 5.85s, Approximately from 0.5s to 6.5s.

APPENDIX D

INPUT FILE OF DIANA FINITE ELEMENT MODEL (NON-LINEAR)

CASE 1)

```
FEMGEN MODEL      : NONLIN
ANALYSIS TYPE     : Structural 2D
MODEL DESCRIPTION : non_linear_1
'UNITS'
LENGTH   M
TIME     SEC
TEMPER   CELSIU
FORCE    N
'COORDINATES' DI=2
    1    0.000000E+00    0.000000E+00
    2    0.000000E+00    2.000000E-01
    3    0.000000E+00    4.000000E-01
    4    0.000000E+00    6.000000E-01
    5    0.000000E+00    8.000001E-01
    6    0.000000E+00    1.000000E+00
    7    0.000000E+00    1.200000E+00
    8    0.000000E+00    1.400000E+00
    9    0.000000E+00    1.600000E+00
   10    0.000000E+00    1.800000E+00
   11    0.000000E+00    2.000000E+00
   12    0.000000E+00    2.200000E+00
   13    0.000000E+00    2.400000E+00
   14    0.000000E+00    2.600000E+00
   15    0.000000E+00    2.800000E+00
   16    0.000000E+00    3.000000E+00
   17    0.000000E+00    3.200000E+00
   18    0.000000E+00    3.400000E+00
   19    0.000000E+00    3.600000E+00
   20    0.000000E+00    3.800000E+00
   21    0.000000E+00    4.000000E+00
   22    0.000000E+00    4.200000E+00
   23    0.000000E+00    4.400000E+00
   24    0.000000E+00    4.600000E+00
   25    0.000000E+00    4.800000E+00
   26    0.000000E+00    5.000000E+00
   27    0.000000E+00    5.200000E+00
   28    0.000000E+00    5.400000E+00
   29    0.000000E+00    5.600000E+00
   30    0.000000E+00    5.800000E+00
   31    0.000000E+00    6.000000E+00
   32    0.000000E+00    6.200000E+00
   33    0.000000E+00    6.400000E+00
   34    0.000000E+00    6.600000E+00
   35    0.000000E+00    6.800000E+00
   36    0.000000E+00    7.000000E+00
   37    0.000000E+00    7.200000E+00
   38    1.200000E+01    7.200000E+00
   39    1.200000E+01    7.000000E+00
   40    1.200000E+01    6.800000E+00
   41    1.200000E+01    6.600000E+00
   42    1.200000E+01    6.400000E+00
   43    1.200000E+01    6.200000E+00
   44    1.200000E+01    6.000000E+00
```

45	1.200000E+01	5.800000E+00
46	1.200000E+01	5.600000E+00
47	1.200000E+01	5.400000E+00
48	1.200000E+01	5.200000E+00
49	1.200000E+01	5.000000E+00
50	1.200000E+01	4.800000E+00
51	1.200000E+01	4.600000E+00
52	1.200000E+01	4.400000E+00
53	1.200000E+01	4.200000E+00
54	1.200000E+01	4.000000E+00
55	1.200000E+01	3.800000E+00
56	1.200000E+01	3.600000E+00
57	1.200000E+01	3.400000E+00
58	1.200000E+01	3.200000E+00
59	1.200000E+01	3.000000E+00
60	1.200000E+01	2.800000E+00
61	1.200000E+01	2.600000E+00
62	1.200000E+01	2.400000E+00
63	1.200000E+01	2.200000E+00
64	1.200000E+01	2.000000E+00
65	1.200000E+01	1.800000E+00
66	1.200000E+01	1.600000E+00
67	1.200000E+01	1.400000E+00
68	1.200000E+01	1.200000E+00
69	1.200000E+01	1.000000E+00
70	1.200000E+01	8.000000E-01
71	1.200000E+01	6.000000E-01
72	1.200000E+01	4.000001E-01
73	1.200000E+01	2.000000E-01
74	1.200000E+01	0.000000E+00
75	0.000000E+00	7.200000E+00
76	3.000000E-01	7.200000E+00
77	6.000000E-01	7.200000E+00
78	9.000000E-01	7.200000E+00
79	1.200000E+00	7.200000E+00
80	1.500000E+00	7.200000E+00
81	1.800000E+00	7.200000E+00
82	2.100000E+00	7.200000E+00
83	2.400000E+00	7.200000E+00
84	2.700000E+00	7.200000E+00
85	3.000000E+00	7.200000E+00
86	3.300000E+00	7.200000E+00
87	3.600000E+00	7.200000E+00
88	3.900000E+00	7.200000E+00
89	4.200000E+00	7.200000E+00
90	4.500000E+00	7.200000E+00
91	4.800000E+00	7.200000E+00
92	5.100000E+00	7.200000E+00
93	5.400000E+00	7.200000E+00
94	5.700000E+00	7.200000E+00
95	6.000000E+00	7.200000E+00
96	6.300000E+00	7.200000E+00
97	6.600000E+00	7.200000E+00
98	6.900001E+00	7.200000E+00
99	7.200000E+00	7.200000E+00
100	7.500000E+00	7.200000E+00
101	7.800000E+00	7.200000E+00

102	8.099999E+00	7.200000E+00
103	8.400000E+00	7.200000E+00
104	8.700001E+00	7.200000E+00
105	9.000000E+00	7.200000E+00
106	9.299999E+00	7.200000E+00
107	9.600000E+00	7.200000E+00
108	9.900001E+00	7.200000E+00
109	1.020000E+01	7.200000E+00
110	1.050000E+01	7.200000E+00
111	1.080000E+01	7.200000E+00
112	1.110000E+01	7.200000E+00
113	1.140000E+01	7.200000E+00
114	1.170000E+01	7.200000E+00
115	1.200000E+01	7.200000E+00
116	0.000000E+00	2.400000E+00
117	0.000000E+00	2.700000E+00
118	0.000000E+00	3.000000E+00
119	0.000000E+00	3.300000E+00
120	0.000000E+00	3.600000E+00
121	0.000000E+00	3.900000E+00
122	0.000000E+00	4.200000E+00
123	0.000000E+00	4.500000E+00
124	0.000000E+00	4.800000E+00
125	1.500000E+00	2.400000E+00
126	1.500000E+00	2.700000E+00
127	1.500000E+00	3.000000E+00
128	1.500000E+00	3.300000E+00
129	1.500000E+00	3.600000E+00
130	1.500000E+00	3.900000E+00
131	1.500000E+00	4.200000E+00
132	1.500000E+00	4.500000E+00
133	1.500000E+00	4.800000E+00
134	3.000000E+00	2.400000E+00
135	3.000000E+00	2.700000E+00
136	3.000000E+00	3.000000E+00
137	3.000000E+00	3.300000E+00
138	3.000000E+00	3.600000E+00
139	3.000000E+00	3.900000E+00
140	3.000000E+00	4.200000E+00
141	3.000000E+00	4.500000E+00
142	3.000000E+00	4.800000E+00
143	4.500000E+00	2.400000E+00
144	4.500000E+00	2.700000E+00
145	4.500000E+00	3.000000E+00
146	4.500000E+00	3.300000E+00
147	4.500000E+00	3.600000E+00
148	4.500000E+00	3.900000E+00
149	4.500000E+00	4.200000E+00
150	4.500000E+00	4.500000E+00
151	4.500000E+00	4.800000E+00
152	6.000000E+00	2.400000E+00
153	6.000000E+00	2.700000E+00
154	6.000000E+00	3.000000E+00
155	6.000000E+00	3.300000E+00
156	6.000000E+00	3.600000E+00
157	6.000000E+00	3.900000E+00
158	6.000000E+00	4.200000E+00

159	6.000000E+00	4.500000E+00
160	6.000000E+00	4.800000E+00
161	7.500000E+00	2.400000E+00
162	7.500000E+00	2.700000E+00
163	7.500000E+00	3.000000E+00
164	7.500000E+00	3.300000E+00
165	7.500000E+00	3.600000E+00
166	7.500000E+00	3.900000E+00
167	7.500000E+00	4.200000E+00
168	7.500000E+00	4.500000E+00
169	7.500000E+00	4.800000E+00
170	9.000000E+00	2.400000E+00
171	9.000000E+00	2.700000E+00
172	9.000000E+00	3.000000E+00
173	9.000000E+00	3.300000E+00
174	9.000000E+00	3.600000E+00
175	9.000000E+00	3.900000E+00
176	9.000000E+00	4.200000E+00
177	9.000000E+00	4.500000E+00
178	9.000000E+00	4.800000E+00
179	1.050000E+01	2.400000E+00
180	1.050000E+01	2.700000E+00
181	1.050000E+01	3.000000E+00
182	1.050000E+01	3.300000E+00
183	1.050000E+01	3.600000E+00
184	1.050000E+01	3.900000E+00
185	1.050000E+01	4.200000E+00
186	1.050000E+01	4.500000E+00
187	1.050000E+01	4.800000E+00
188	1.200000E+01	2.400000E+00
189	1.200000E+01	2.700000E+00
190	1.200000E+01	3.000000E+00
191	1.200000E+01	3.300000E+00
192	1.200000E+01	3.600000E+00
193	1.200000E+01	3.900000E+00
194	1.200000E+01	4.200000E+00
195	1.200000E+01	4.500000E+00
196	1.200000E+01	4.800000E+00
197	0.000000E+00	2.550000E+00
198	0.000000E+00	2.850000E+00
199	0.000000E+00	3.150000E+00
200	0.000000E+00	3.450000E+00
201	0.000000E+00	3.750000E+00
202	0.000000E+00	4.050000E+00
203	0.000000E+00	4.350000E+00
204	0.000000E+00	4.650000E+00
205	7.500000E-01	2.400000E+00
206	7.500000E-01	2.700000E+00
207	7.500000E-01	3.000000E+00
208	7.500000E-01	3.300000E+00
209	7.500000E-01	3.600000E+00
210	7.500000E-01	3.900000E+00
211	7.500000E-01	4.200000E+00
212	7.500000E-01	4.500000E+00
213	7.500000E-01	4.800000E+00
214	1.500000E+00	2.550000E+00
215	1.500000E+00	2.850000E+00

216	1.500000E+00	3.150000E+00
217	1.500000E+00	3.450000E+00
218	1.500000E+00	3.750000E+00
219	1.500000E+00	4.050000E+00
220	1.500000E+00	4.350000E+00
221	1.500000E+00	4.650000E+00
222	2.250000E+00	2.400000E+00
223	2.250000E+00	2.700000E+00
224	2.250000E+00	3.000000E+00
225	2.250000E+00	3.300000E+00
226	2.250000E+00	3.600000E+00
227	2.250000E+00	3.900000E+00
228	2.250000E+00	4.200000E+00
229	2.250000E+00	4.500000E+00
230	2.250000E+00	4.800000E+00
231	3.000000E+00	2.550000E+00
232	3.000000E+00	2.850000E+00
233	3.000000E+00	3.150000E+00
234	3.000000E+00	3.450000E+00
235	3.000000E+00	3.750000E+00
236	3.000000E+00	4.050000E+00
237	3.000000E+00	4.350000E+00
238	3.000000E+00	4.650000E+00
239	3.750000E+00	2.400000E+00
240	3.750000E+00	2.700000E+00
241	3.750000E+00	3.000000E+00
242	3.750000E+00	3.300000E+00
243	3.750000E+00	3.600000E+00
244	3.750000E+00	3.900000E+00
245	3.750000E+00	4.200000E+00
246	3.750000E+00	4.500000E+00
247	3.750000E+00	4.800000E+00
248	4.500000E+00	2.550000E+00
249	4.500000E+00	2.850000E+00
250	4.500000E+00	3.150000E+00
251	4.500000E+00	3.450000E+00
252	4.500000E+00	3.750000E+00
253	4.500000E+00	4.050000E+00
254	4.500000E+00	4.350000E+00
255	4.500000E+00	4.650000E+00
256	5.250000E+00	2.400000E+00
257	5.250000E+00	2.700000E+00
258	5.250000E+00	3.000000E+00
259	5.250000E+00	3.300000E+00
260	5.250000E+00	3.600000E+00
261	5.250000E+00	3.900000E+00
262	5.250000E+00	4.200000E+00
263	5.250000E+00	4.500000E+00
264	5.250000E+00	4.800000E+00
265	6.000000E+00	2.550000E+00
266	6.000000E+00	2.850000E+00
267	6.000000E+00	3.150000E+00
268	6.000000E+00	3.450000E+00
269	6.000000E+00	3.750000E+00
270	6.000000E+00	4.050000E+00
271	6.000000E+00	4.350000E+00
272	6.000000E+00	4.650000E+00

273	6.750000E+00	2.400000E+00
274	6.750000E+00	2.700000E+00
275	6.750000E+00	3.000000E+00
276	6.750000E+00	3.300000E+00
277	6.750000E+00	3.600000E+00
278	6.750000E+00	3.900000E+00
279	6.750000E+00	4.200000E+00
280	6.750000E+00	4.500000E+00
281	6.750000E+00	4.800000E+00
282	7.500000E+00	2.550000E+00
283	7.500000E+00	2.850000E+00
284	7.500000E+00	3.150000E+00
285	7.500000E+00	3.450000E+00
286	7.500000E+00	3.750000E+00
287	7.500000E+00	4.050000E+00
288	7.500000E+00	4.350000E+00
289	7.500000E+00	4.650000E+00
290	8.250000E+00	2.400000E+00
291	8.250000E+00	2.700000E+00
292	8.250000E+00	3.000000E+00
293	8.250000E+00	3.300000E+00
294	8.250000E+00	3.600000E+00
295	8.250000E+00	3.900000E+00
296	8.250000E+00	4.200000E+00
297	8.250000E+00	4.500000E+00
298	8.250000E+00	4.800000E+00
299	9.000000E+00	2.550000E+00
300	9.000000E+00	2.850000E+00
301	9.000000E+00	3.150000E+00
302	9.000000E+00	3.450000E+00
303	9.000000E+00	3.750000E+00
304	9.000000E+00	4.050000E+00
305	9.000000E+00	4.350000E+00
306	9.000000E+00	4.650000E+00
307	9.750000E+00	2.400000E+00
308	9.750000E+00	2.700000E+00
309	9.750000E+00	3.000000E+00
310	9.750000E+00	3.300000E+00
311	9.750000E+00	3.600000E+00
312	9.750000E+00	3.900000E+00
313	9.750000E+00	4.200000E+00
314	9.750000E+00	4.500000E+00
315	9.750000E+00	4.800000E+00
316	1.050000E+01	2.550000E+00
317	1.050000E+01	2.850000E+00
318	1.050000E+01	3.150000E+00
319	1.050000E+01	3.450000E+00
320	1.050000E+01	3.750000E+00
321	1.050000E+01	4.050000E+00
322	1.050000E+01	4.350000E+00
323	1.050000E+01	4.650000E+00
324	1.125000E+01	2.400000E+00
325	1.125000E+01	2.700000E+00
326	1.125000E+01	3.000000E+00
327	1.125000E+01	3.300000E+00
328	1.125000E+01	3.600000E+00
329	1.125000E+01	3.900000E+00

```
  330     1.125000E+01    4.200000E+00
  331     1.125000E+01    4.500000E+00
  332     1.125000E+01    4.800000E+00
  333     1.200000E+01    2.550000E+00
  334     1.200000E+01    2.850000E+00
  335     1.200000E+01    3.150000E+00
  336     1.200000E+01    3.450000E+00
  337     1.200000E+01    3.750000E+00
  338     1.200000E+01    4.050000E+00
  339     1.200000E+01    4.350000E+00
  340     1.200000E+01    4.650000E+00
'ELEMENTS'
CONNECTIVITY
    1 CL9BE  1 2 3
    2 CL9BE  3 4 5
    3 CL9BE  5 6 7
    4 CL9BE  7 8 9
    5 CL9BE  9 10 11
    6 CL9BE  11 12 13
    7 CL9BE  13 14 15
    8 CL9BE  15 16 17
    9 CL9BE  17 18 19
   10 CL9BE  19 20 21
   11 CL9BE  21 22 23
   12 CL9BE  23 24 25
   13 CL9BE  25 26 27
   14 CL9BE  27 28 29
   15 CL9BE  29 30 31
   16 CL9BE  31 32 33
   17 CL9BE  33 34 35
   18 CL9BE  35 36 37
   19 CL9BE  38 39 40
   20 CL9BE  40 41 42
   21 CL9BE  42 43 44
   22 CL9BE  44 45 46
   23 CL9BE  46 47 48
   24 CL9BE  48 49 50
   25 CL9BE  50 51 52
   26 CL9BE  52 53 54
   27 CL9BE  54 55 56
   28 CL9BE  56 57 58
   29 CL9BE  58 59 60
   30 CL9BE  60 61 62
   31 CL9BE  62 63 64
   32 CL9BE  64 65 66
   33 CL9BE  66 67 68
   34 CL9BE  68 69 70
   35 CL9BE  70 71 72
   36 CL9BE  72 73 74
   37 CL9BE  75 76 77
   38 CL9BE  77 78 79
   39 CL9BE  79 80 81
   40 CL9BE  81 82 83
   41 CL9BE  83 84 85
   42 CL9BE  85 86 87
   43 CL9BE  87 88 89
   44 CL9BE  89 90 91
```

45 CL9BE 91 92 93
46 CL9BE 93 94 95
47 CL9BE 95 96 97
48 CL9BE 97 98 99
49 CL9BE 99 100 101
50 CL9BE 101 102 103
51 CL9BE 103 104 105
52 CL9BE 105 106 107
53 CL9BE 107 108 109
54 CL9BE 109 110 111
55 CL9BE 111 112 113
56 CL9BE 113 114 115
57 CQ16M 116 197 117 206 126 214 125 205
58 CQ16M 117 198 118 207 127 215 126 206
59 CQ16M 118 199 119 208 128 216 127 207
60 CQ16M 119 200 120 209 129 217 128 208
61 CQ16M 120 201 121 210 130 218 129 209
62 CQ16M 121 202 122 211 131 219 130 210
63 CQ16M 122 203 123 212 132 220 131 211
64 CQ16M 123 204 124 213 133 221 132 212
65 CQ16M 125 214 126 223 135 231 134 222
66 CQ16M 126 215 127 224 136 232 135 223
67 CQ16M 127 216 128 225 137 233 136 224
68 CQ16M 128 217 129 226 138 234 137 225
69 CQ16M 129 218 130 227 139 235 138 226
70 CQ16M 130 219 131 228 140 236 139 227
71 CQ16M 131 220 132 229 141 237 140 228
72 CQ16M 132 221 133 230 142 238 141 229
73 CQ16M 134 231 135 240 144 248 143 239
74 CQ16M 135 232 136 241 145 249 144 240
75 CQ16M 136 233 137 242 146 250 145 241
76 CQ16M 137 234 138 243 147 251 146 242
77 CQ16M 138 235 139 244 148 252 147 243
78 CQ16M 139 236 140 245 149 253 148 244
79 CQ16M 140 237 141 246 150 254 149 245
80 CQ16M 141 238 142 247 151 255 150 246
81 CQ16M 143 248 144 257 153 265 152 256
82 CQ16M 144 249 145 258 154 266 153 257
83 CQ16M 145 250 146 259 155 267 154 258
84 CQ16M 146 251 147 260 156 268 155 259
85 CQ16M 147 252 148 261 157 269 156 260
86 CQ16M 148 253 149 262 158 270 157 261
87 CQ16M 149 254 150 263 159 271 158 262
88 CQ16M 150 255 151 264 160 272 159 263
89 CQ16M 152 265 153 274 162 282 161 273
90 CQ16M 153 266 154 275 163 283 162 274
91 CQ16M 154 267 155 276 164 284 163 275
92 CQ16M 155 268 156 277 165 285 164 276
93 CQ16M 156 269 157 278 166 286 165 277
94 CQ16M 157 270 158 279 167 287 166 278
95 CQ16M 158 271 159 280 168 288 167 279
96 CQ16M 159 272 160 281 169 289 168 280
97 CQ16M 161 282 162 291 171 299 170 290
98 CQ16M 162 283 163 292 172 300 171 291
99 CQ16M 163 284 164 293 173 301 172 292
100 CQ16M 164 285 165 294 174 302 173 293
101 CQ16M 165 286 166 295 175 303 174 294

```
  102 CQ16M  166 287 167 296 176 304 175 295
  103 CQ16M  167 288 168 297 177 305 176 296
  104 CQ16M  168 289 169 298 178 306 177 297
  105 CQ16M  170 299 171 308 180 316 179 307
  106 CQ16M  171 300 172 309 181 317 180 308
  107 CQ16M  172 301 173 310 182 318 181 309
  108 CQ16M  173 302 174 311 183 319 182 310
  109 CQ16M  174 303 175 312 184 320 183 311
  110 CQ16M  175 304 176 313 185 321 184 312
  111 CQ16M  176 305 177 314 186 322 185 313
  112 CQ16M  177 306 178 315 187 323 186 314
  113 CQ16M  179 316 180 325 189 333 188 324
  114 CQ16M  180 317 181 326 190 334 189 325
  115 CQ16M  181 318 182 327 191 335 190 326
  116 CQ16M  182 319 183 328 192 336 191 327
  117 CQ16M  183 320 184 329 193 337 192 328
  118 CQ16M  184 321 185 330 194 338 193 329
  119 CQ16M  185 322 186 331 195 339 194 330
  120 CQ16M  186 323 187 332 196 340 195 331
MATERIALS
/ 57-120 /  1
/ 1-36 /  2
/ 37-56 /  3
GEOMETRY
/ 57-120 /  1
/ 37-56 /  2
/ 1-36 /  3
'REINFORCEMENTS'
 LOCATI
   15 BAR
     LINE    -0.215000E+00   0.000000E+00   0.000000E+00
            -0.215000E+00   0.240000E+01   0.000000E+00
     ELEMEN    1-6 /
     LINE    -0.215000E+00   0.240000E+01   0.000000E+00
            -0.215000E+00   0.480000E+01   0.000000E+00
     ELEMEN    7-12 /
     LINE    -0.215000E+00   0.480000E+01   0.000000E+00
            -0.215000E+00   0.720000E+01   0.000000E+00
     ELEMEN    13-18 /
   16 BAR
     LINE     0.215000E+00   0.000000E+00   0.000000E+00
             0.215000E+00   0.240000E+01   0.000000E+00
     ELEMEN    1-6 /
     LINE     0.215000E+00   0.240000E+01   0.000000E+00
             0.215000E+00   0.480000E+01   0.000000E+00
     ELEMEN    7-12 /
     LINE     0.215000E+00   0.480000E+01   0.000000E+00
             0.215000E+00   0.720000E+01   0.000000E+00
     ELEMEN    13-18 /
   17 BAR
     LINE     0.122150E+02   0.720000E+01   0.000000E+00
             0.122150E+02   0.480000E+01   0.000000E+00
     ELEMEN    19-24 /
     LINE     0.122150E+02   0.480000E+01   0.000000E+00
             0.122150E+02   0.240000E+01   0.000000E+00
     ELEMEN    25-30 /
     LINE     0.122150E+02   0.240000E+01   0.000000E+00
```

```
          0.122150E+02   0.000000E+00   0.000000E+00
     ELEMEN    31-36 /
   18 BAR
     LINE      0.117850E+02   0.720000E+01   0.000000E+00
          0.117850E+02   0.480000E+01   0.000000E+00
     ELEMEN    19-24 /
     LINE      0.117850E+02   0.480000E+01   0.000000E+00
          0.117850E+02   0.240000E+01   0.000000E+00
     ELEMEN    25-30 /
     LINE      0.117850E+02   0.240000E+01   0.000000E+00
          0.117850E+02   0.000000E+00   0.000000E+00
     ELEMEN    31-36 /
   19 BAR
     LINE      0.000000E+00   0.678700E+01   0.000000E+00
          0.120000E+02   0.678700E+01   0.000000E+00
     ELEMEN    37-56 /
   20 BAR
     LINE      0.000000E+00   0.741700E+01   0.000000E+00
          0.120000E+02   0.741700E+01   0.000000E+00
     ELEMEN    37-56 /
MATERIALS
/ 15-20 /  4
GEOMETRY
/ 19 /  4
/ 20 /  5
/ 15-18 /  6
'MATERIALS'
   1 YOUNG     3.820000E+10
     POISON    1.500000E-01
     DENSIT    1.631000E+03
   2 YOUNG     3.820000E+10
     POISON    1.500000E-01
     DENSIT    2.548400E+03
     TOTCRK ROTATE
     TENCRV LINEAR
     TENSTR 2.660000E+06
     EPSULT 1.000000E-03
     SHRCRV CONSTA
     BETA 1.000000E-02
     COMCRV THOREN
     COMSTR 4.500000E+07
     CNFCRV VECCHI
     REDCRV VC1993
     RAYLEI    1.810000E+00    1.060000E-03
   3 YOUNG     3.820000E+10
     POISON    1.500000E-01
     DENSIT    3.376120E+03
     TOTCRK ROTATE
     TENCRV LINEAR
     TENSTR 2.660000E+06
     EPSULT 1.000000E-03
     SHRCRV CONSTA
     BETA 1.000000E-02
     COMCRV THOREN
     COMSTR 4.500000E+07
     CNFCRV VECCHI
     REDCRV VC1993
```

```
    RAYLEI    1.810000E+00    1.060000E-03
  4 YOUNG     2.000000E+11
    POISON    3.000000E-01
    YIELD VMISES
    HARDEN STRAIN
    YLDVAL 4.000000E+08
'GEOMETRY'
  1 THICK     2.500000E-01
  2 TSHAPE    7.000000E-01    2.000000E+00    6.023000E-02
          3.338500E-01
  3 RECTAN    5.000000E-01    5.000000E-01
  4 CROSSE    0.615752000E-03
  5 CROSSE    0.923628000E-03
  6 CROSSE    1.017876000E-03
'GROUPS'
ELEMEN
  1 SET1 / 57-120 /
NODES
  2 SET1_N / 116-340 /
ELEMEN
  3 SET2 / 1-36 /
NODES
  4 SET2_N / 1-74 /
ELEMEN
  5 SET3 / 37-56 /
NODES
  6 SET3_N / 75-115 /
  7 FEET / 1 74 /
'SUPPORTS'
 / 1 74 /   TR     1
 / 1 74 /   RO     1
 / 1 74 /   TR     2
 / 1 74 /   RO     2
 / 1 74 /   TR     3
 / 1 74 /   RO     3
'TYINGS'
EQUAL TR 1
  / 75-115 / 37
FIX TR 2
  75    37 TR 2 1.0
FIX TR 1
  38   37 TR 1 1.0
FIX TR 2
  115   38 TR 2 1.0
EQUAL TR 1
  / 116 125 134 143 152 161 170 179 188 205 222 239 256 273 290 307 324 62 / 13
FIX TR 2
  116   13 TR 2 1.0
FIX TR 2
  124   25 TR 2 1.0
FIX TR 2
  196   50 TR 2 1.0
FIX TR 2
  188   62 TR 2 1.0
'LOADS'
CASE 1
BASE
```

```
1    0.100E-01
CASE 2
WEIGHT
2    -9.81
'TIMELO'
LOAD 1
TIMES 0.000 … t /
FACTOR a1… at/
LOAD 2
TIMES 0 0.15 t /
FACTOR 0 1 1 /
'DIRECTIONS'
    1   1.000000E+00   0.000000E+00   0.000000E+00
    2   0.000000E+00   1.000000E+00   0.000000E+00
    3   0.000000E+00   0.000000E+00   1.000000E+00
'END'
```

CASE 2)

The same as case 1) only differs with 'TYINGS' modulus.

```
….
'TYINGS'
EQUAL TR 1
  / 75-115 / 37
FIX TR 2
  75    37 TR 2 1.0
FIX TR 1
  38   37 TR 1 1.0
FIX TR 2
  115   38 TR 2 1.0
EQUAL TR 1
   116  13
FIX TR 2
  116   13 TR 2 1.0
FIX TR 2
  124   25 TR 2 1.0
FIX TR 2
  196   50 TR 2 1.0
FIX TR 2
  188   62 TR 2 1.0
….
```

CASE 3)

The same as case 1) only differs with 'TYINGS' modulus.

```
….
'TYINGS'
EQUAL TR 1
  75    37
FIX TR 2
  75    37 TR 2 1.0
EQUAL RO 3
  75    37
FIX TR 1
  115   38 TR 1 1.0
FIX TR 2
```

```
  115   38 TR 2 1.0
EQUAL RO 3
  115   38
EQUAL TR 1
  116   13
FIX TR 2
  116   13 TR 2 1.0
FIX TR 2
  124   25 TR 2 1.0
FIX TR 2
  196   50 TR 2 1.0
EQUAL TR 1
  188   62
FIX TR 2
  188   62 TR 2 1.0
….
```

CASE 4)

The same as case 1) only differs with 'TYINGS' modulus.

```
….
'TYINGS'
EQUAL TR 1
  75    37
FIX TR 2
  75    37 TR 2 1.0
EQUAL RO 3
  75    37
FIX TR 1
  115   38 TR 1 1.0
FIX TR 2
  115   38 TR 2 1.0
EQUAL RO 3
  115   38
EQUAL TR 1
  116   13
FIX TR 2
  116   13 TR 2 1.0
FIX TR 2
  124   25 TR 2 1.0
FIX TR 2
  196   50 TR 2 1.0
FIX TR 2
  188   62 TR 2 1.0
….
```

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