Blast Load Time History Analysis

An Example in S-FRAME

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Objective

The objective of the following examples is to illustrate and provide guidance on the use of the features available in S-FRAME for seismic/dynamic analysis and design. While they are necessarily discussed, the intention is not to explain or advise on the application of the Seismic provisions of NBCC 2005 to building design, nor the theories underlying the Design Code and its various provisions. For those seeking such information we highly recommend the courses – many of which are offered via the internet - available as part of the Structural Engineers Association of BC ***Certificate in Structural Engineering (CSE)*** – see http://www.seabc.ca/courses.html for more information. Discussions on aspects and methods of modeling, assumptions, theories etc are kept to a minimum to aid clarity and simplicity. The intention is to outline, for competent and professionally qualified individuals, the use of S-FRAME and S-STEEL as tools in the Seismic Analysis & Design Process.

Disclaimer

While the authors of this document have tried to be as accurate as possible, they cannot be held responsible for any errors and omissions in it or in the designs of others that might be based on it. This document is intended for the use of professional personnel competent to evaluate the significance and limitations of its contents and recommendations, and who will accept the responsibility for its application. Users of information from this publication assume all liability. **The authors and S-FRAME Software Inc. disclaim any and all responsibility for the applications of the stated principles and for the accuracy of any of the material contained herein.**

Acknowledgements

With grateful acknowledgement to Luis E. García and Mete A. Sozen for their kind permission to include in this document excerpts from their Purdue University CE571 course notes (Reference [1]). **This acknowledgement does not imply any endorsement by Dr García or Professor Sozen of S-FRAME Software programs, nor any checking or validation of their operation or output.**

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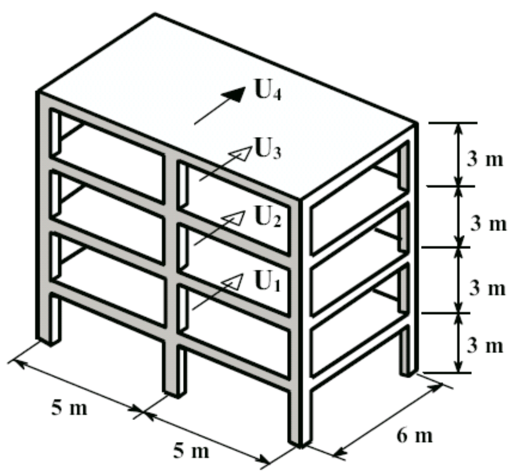
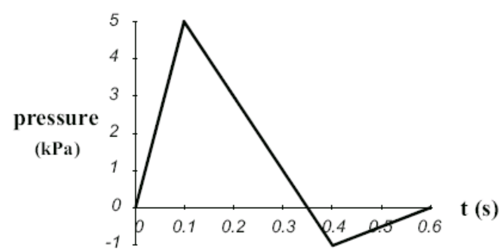
1 Example Brief

Example 4

The building shown in Fig. 5 is subjected to an explosion. The air pressure wave caused by the explosion varies in the form shown in Fig. 5(b). We are interested in obtaining the response of the structure in the short direction, as shown in the figure. Damping of the structure, for the displacement amplitude expected, is estimated to be $\xi = 2\%$ of critical. All girders of the frames have width **b = 0.40 m** and depth **h = 0.50 m**. All columns are square with a section side dimension of **h = 0.40 m**. The modulus of elasticity of the structure is **E = 25 GPa**. The building has a mass per unit area of **1000 kg/m²**.

The explosion occurred far away, therefore we can assume that the pressure applied to the building doesn't vary with height and is applied uniformly to the building façade. The tributary area for application of the pressure at the top story is $10\text{m} \cdot 1.5\text{ m} = 15\text{ m}^2$ and for the other floors $10\text{m} \cdot 3\text{ m} = 30\text{ m}^2$.





(a) *(b)*

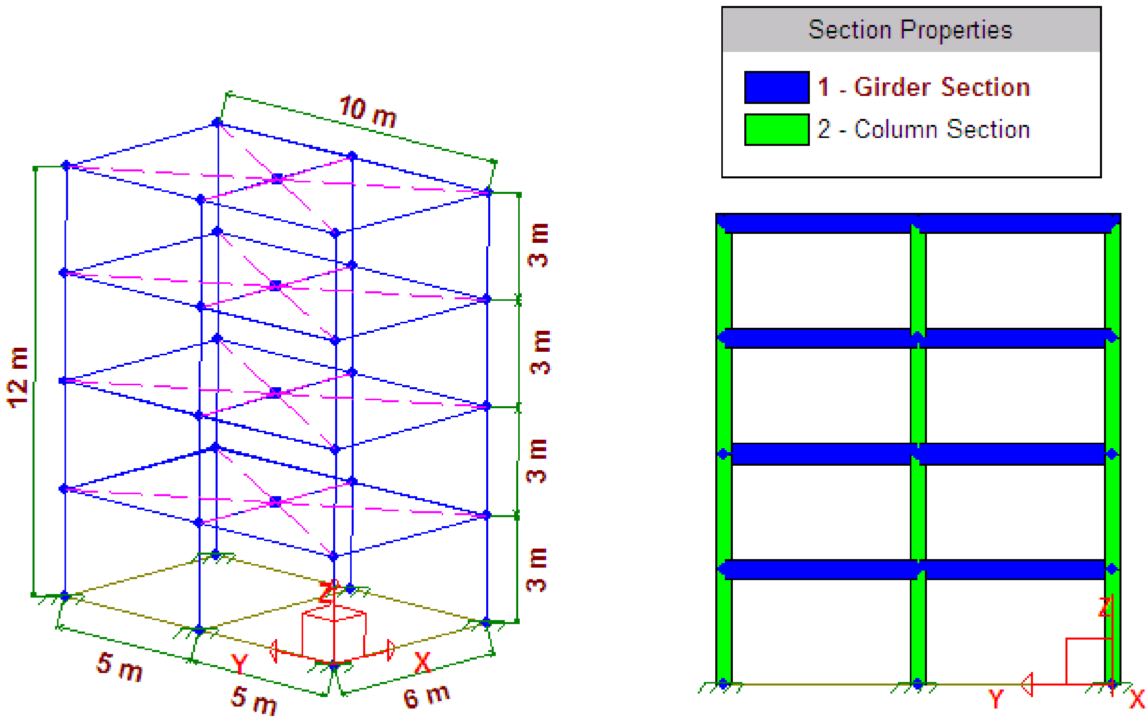
Fig. 5 - Example 4

Further aspects of the building/analysis model either discussed within the body of the example or not stated explicitly are:

- Supports are fully fixed.
- Floors act as rigid diaphragms.
- Mass distribution is idealized as concentrated at the floors only – column and girders elements are massless.
- Contribution to stiffness matrix of shear stiffness is not considered.

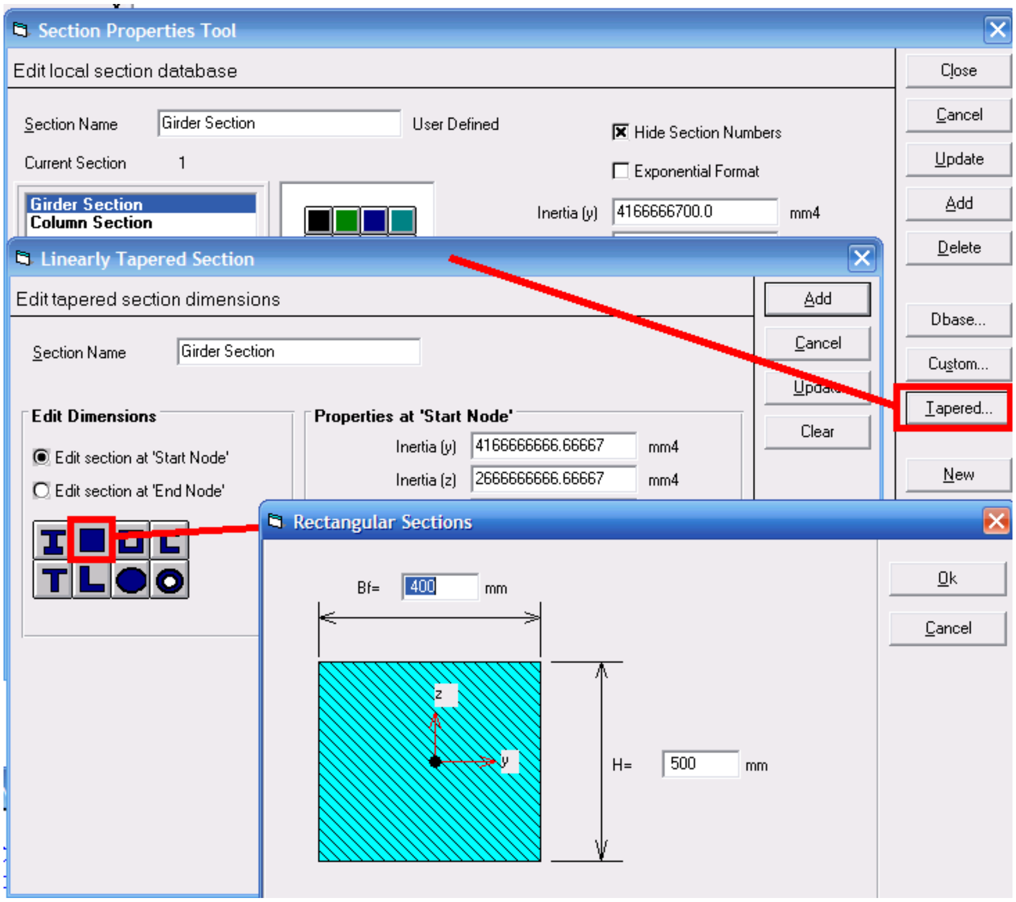
2 S-FRAME Model

The frame is input as per the Example details and dimensions. One analysis element per beam/column is used. All connections are rigid and supports are fully fixed (it was found that this is the case, though it is not explicitly stated in the Example). The X-axis is chosen as the axis of displacement in S-FRAME.

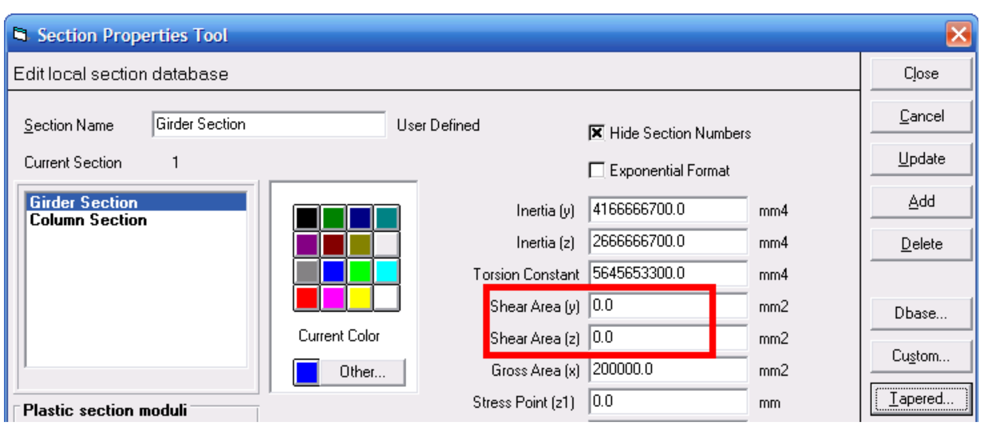


2.1 Sections

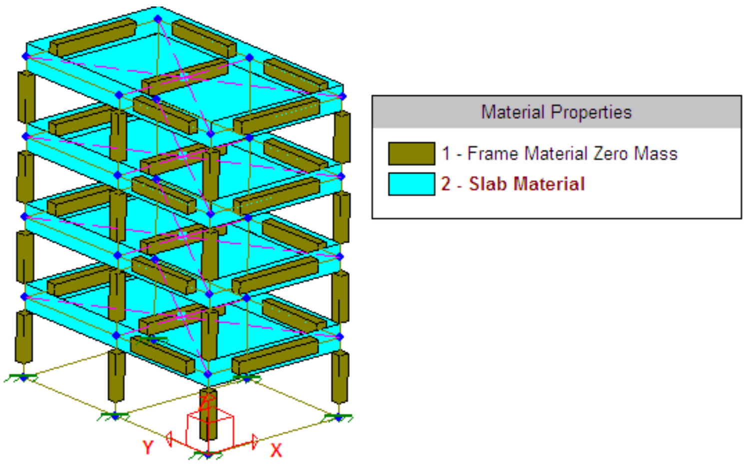
S-FRAME's **Tapered** section tool is used to create the 'Girder' and 'Column' sections (sections do not have to be tapered). Sections thus created will render correctly and can be readily edited.



To give closer agreement with the example (which does not consider shear stiffness) the Shear Area (A_v) of the sections is set = 0. There is no reason why this should be done in practice; while shear effects are often small enough to be neglected they nevertheless occur.



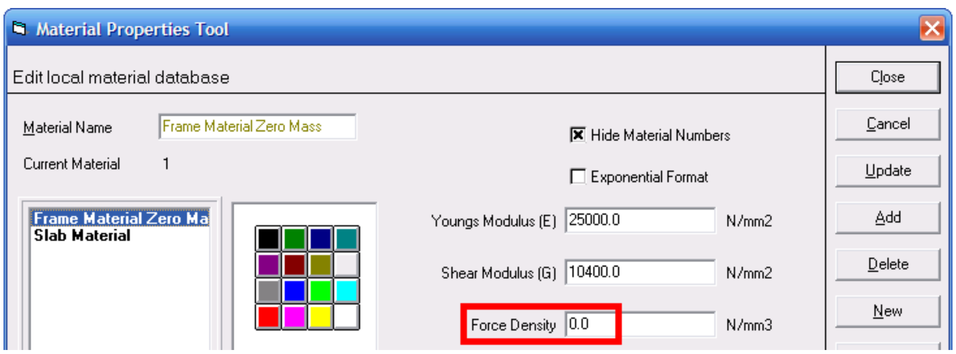
2.2 Materials



Since the building mass in the example is idealized as concentrated at the floors, again to give closer agreement, a zero density material is created for the frame members so these do not introduce distributed mass which would change the vibration characteristics to some extent.

A second material with the specified force density (per unit thickness) is created for the diaphragm panels which are used to introduce the floor mass.

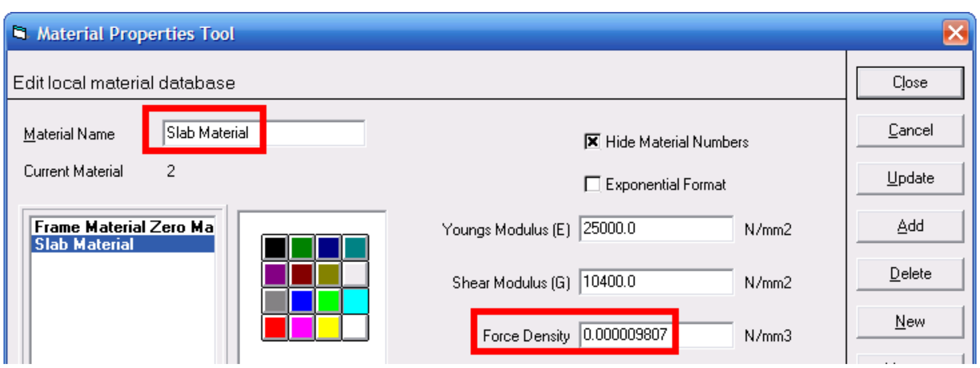
Material Modulus of Elasticity; $\mathbf{E}$ = 25 GPa = **25000** N/mm^2; (we note that this is a typical value for reinforced concrete for short term loading)



Note a sensible value of $\mathbf{G}$ (shear modulus) is also entered as this is a component of the S-FRAME Beam Element stiffness matrix for both shear and torsional stiffness.

Slab material force density; $\rho = g_{acc} \times 1000$ kg/m^3 = **0.0000098067** N/mm^3

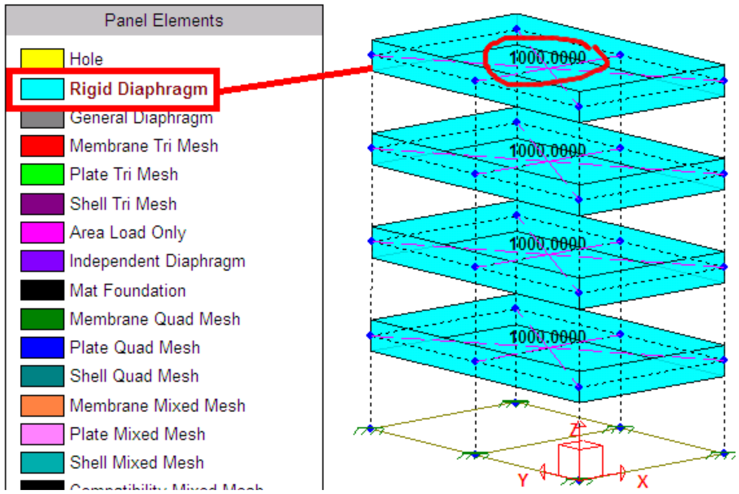
(gravitational acceleration; g_{acc} = **9.807** m/s^2;)



Since the 'Slab Material' is only applied to the Rigid Diaphragm panel for mass generation, the E and G values for this material are irrelevant.

2.3 Rigid Diaphragm Floors

Since a ‘rigid diaphragm effect’ is specified in the Example, Rigid Diaphragm **Panels** are used to model this behaviour. These panels also provide a convenient method of introducing the floor mass. The panels are assigned thickness = 1000mm and a material with appropriate force density (see above). S-FRAME automatically calculates the mass of the diaphragm and applies it as a lumped mass at the centroid of the panel area. For convenience, since engineers usually work in terms of weight rather than mass, S-FRAME describes mass in terms of force units.



Note - The E & G values of the material assigned to a Rigid Diaphragm Panel have no effect on its stiffness – the diaphragm is *rigid* in-plane and has zero stiffness out of plane

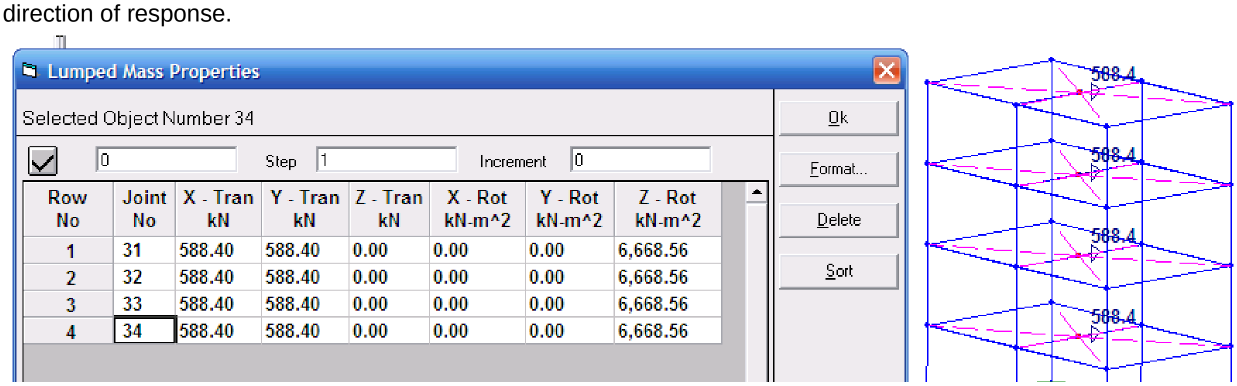
Slab material force density; $\rho = g_{acc} \times 1000$ kg/m^3 = **9.807×10^{-6}** N/mm^3

Floor Volume; V_i = 10m × 6m × 1000mm = **60** m^3

Floor Weight; $W_i = \rho \times V_i$ = **588.40** kN

Floor Mass; $M_i = \rho \times V_i / g_{acc}$ = **60.00** tonne

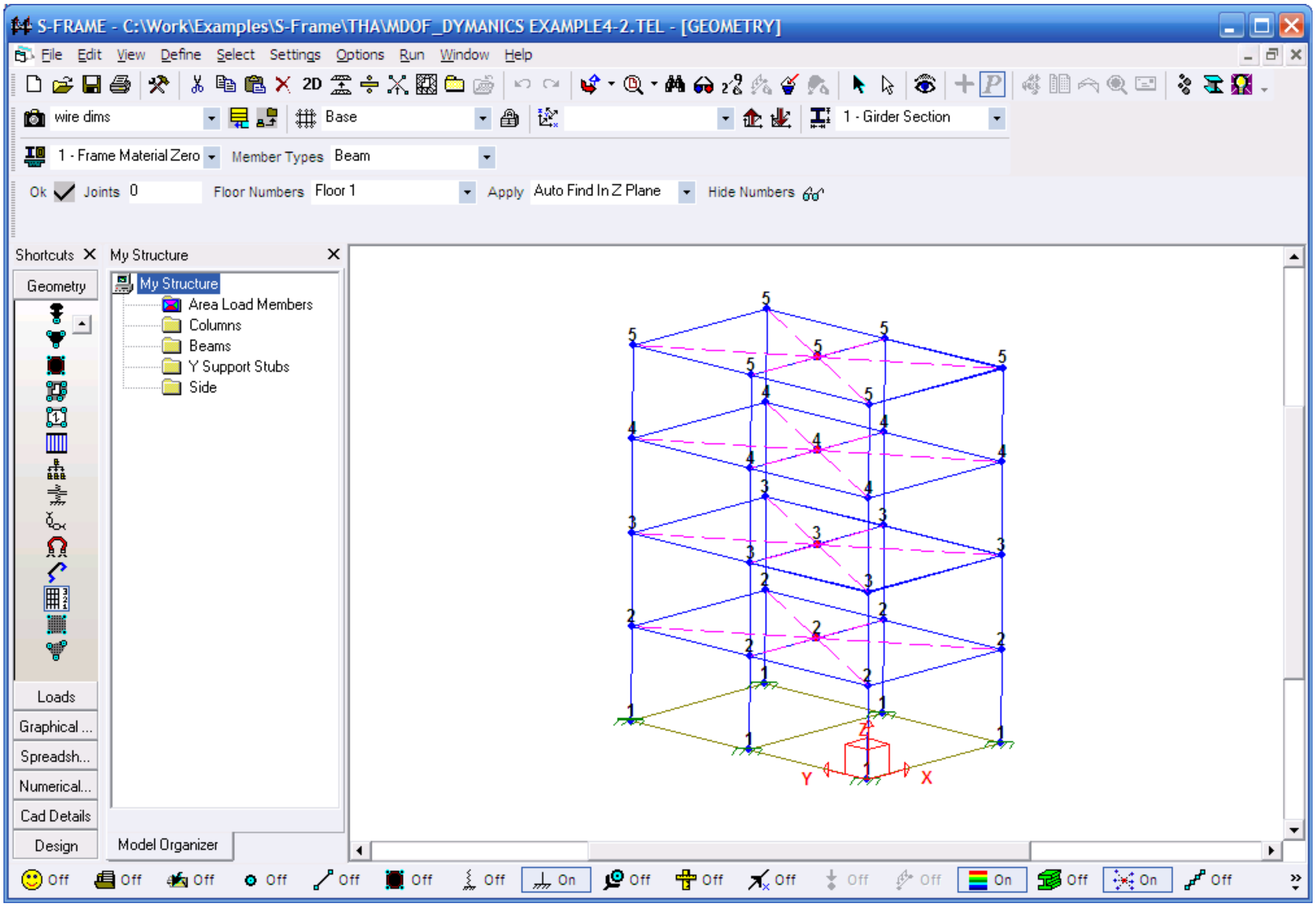
Although it is not necessary for analysis, the lumped mass of the panel can be exposed by running the S-FRAME command **EDIT/Mesh Generator/Generate Rigid Diaphragm Mass**. Note that the diaphragm produces translational mass in both lateral axes, as well as Z-rotational mass. The example only actually requires X-Translational Mass since it only considers one direction of response.



Row No	Joint No	X - Tran kN	Y - Tran kN	Z - Tran kN	X - Rot kN-m^2	Y - Rot kN-m^2	Z - Rot kN-m^2
1	31	588.40	588.40	0.00	0.00	0.00	6,668.56
2	32	588.40	588.40	0.00	0.00	0.00	6,668.56
3	33	588.40	588.40	0.00	0.00	0.00	6,668.56
4	34	588.40	588.40	0.00	0.00	0.00	6,668.56

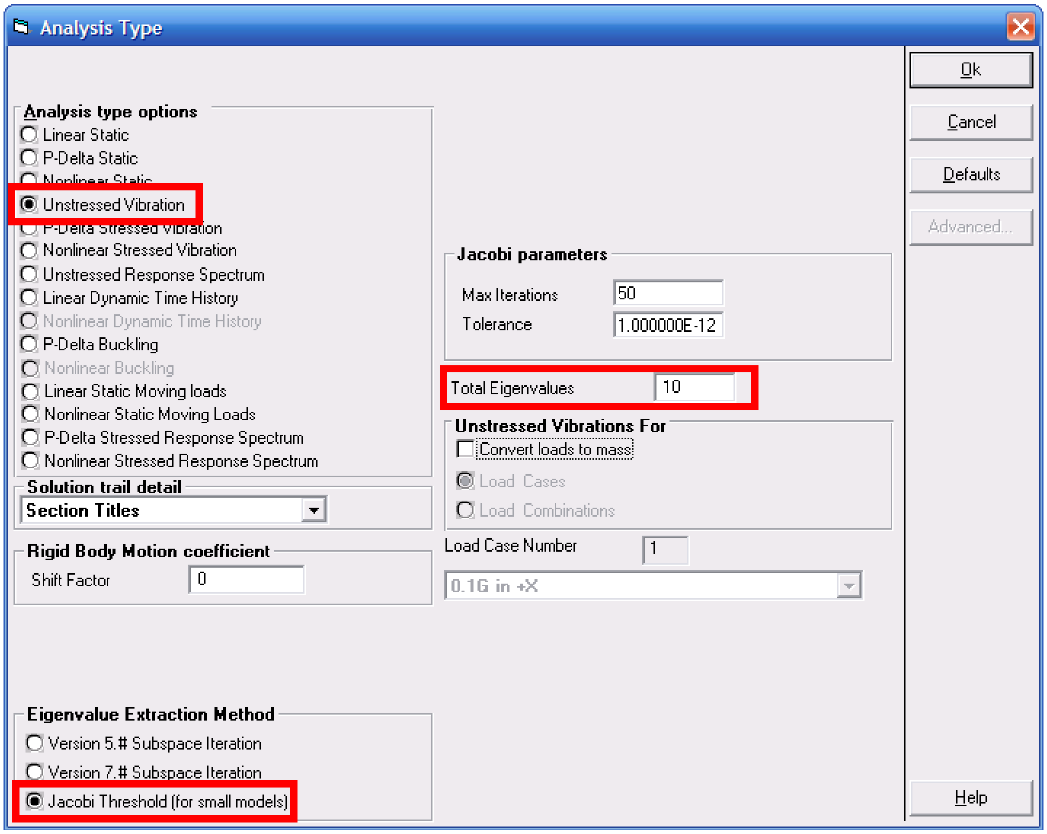
3 Floor Numbers

Finally **Floor Numbers** are applied to joints to identify floors. Note that Floor Number = 1 is applied to the base support joints. This is required for S-FRAME to calculate storey drifts, storey shears and floor forces. This completes the modeling process.

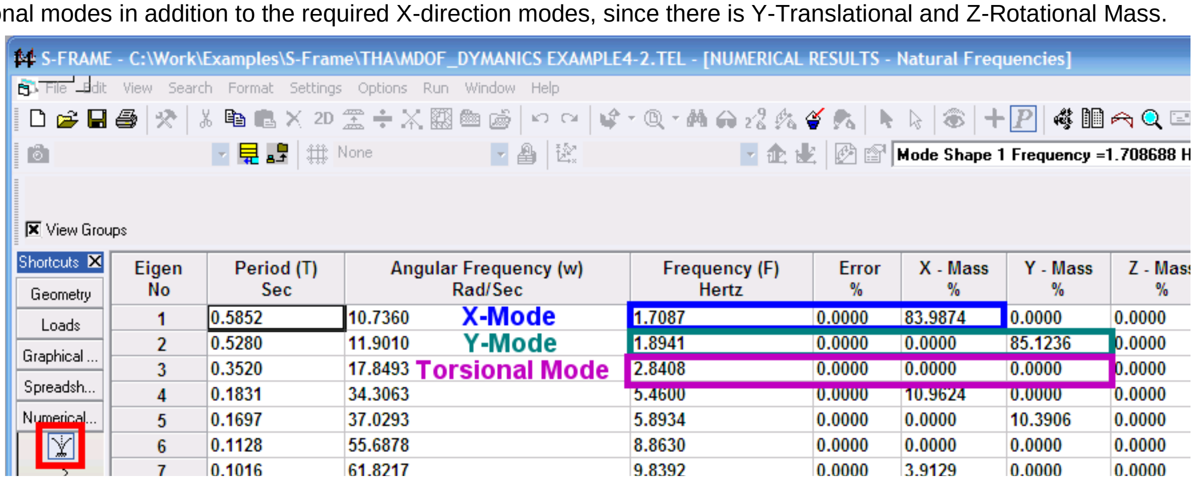


4 Vibration Analysis

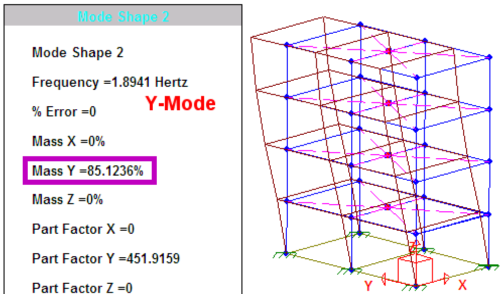
Before running Time History Analysis, the vibration characteristics of the structure are assessed and verified. The following Vibration settings are initially used; **Jacobi Threshold** Eigenvalue Extraction Method (since the model is relatively small and has few modes) and **10 Eigenvalues** (mode shapes) requested.

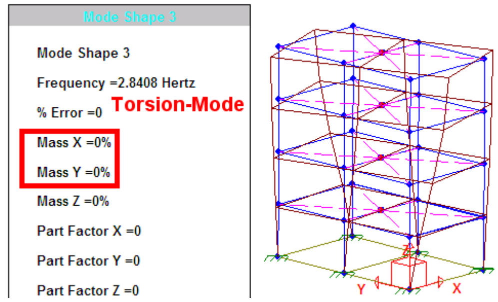


With this 3D model, some modes are found which are not given in the Example. S-FRAME finds typical Y-direction and torsional modes in addition to the required X-direction modes, since there is Y-Translational and Z-Rotational Mass.

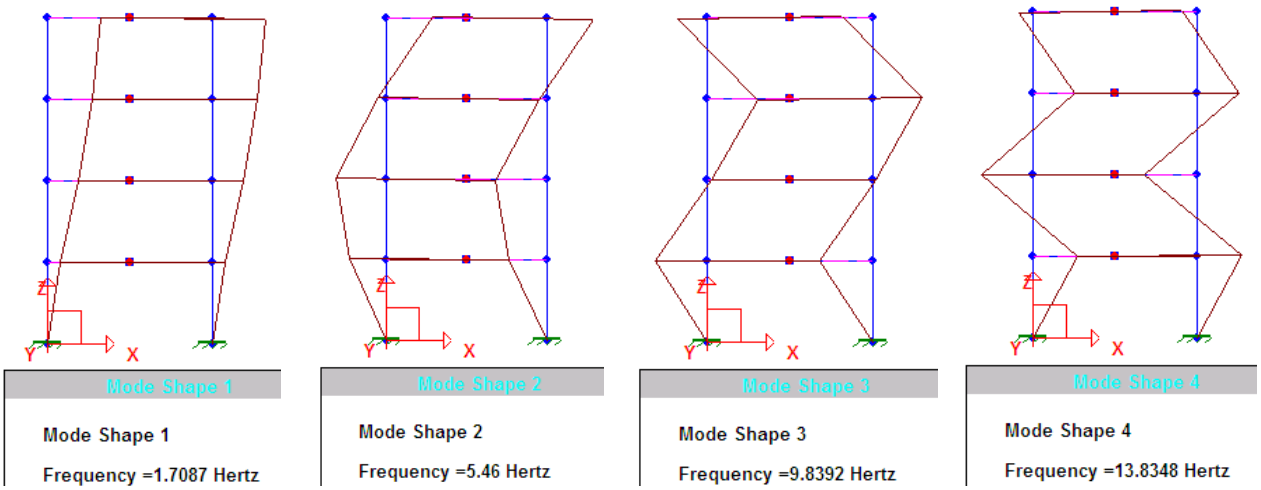


Eigen No	Period (T) Sec	Angular Frequency (w) Rad/Sec	Frequency (F) Hertz	Error %	X - Mass %	Y - Mass %	Z - Mass %
1	0.5852	10.7360	1.7087	0.0000	83.9874	0.0000	0.0000
2	0.5280	11.9010	1.8941	0.0000	0.0000	85.1236	0.0000
3	0.3520	17.8493	2.8408	0.0000	0.0000	0.0000	0.0000
4	0.1831	34.3063	5.4600	0.0000	10.9624	0.0000	0.0000
5	0.1697	37.0293	5.8934	0.0000	0.0000	10.3906	0.0000
6	0.1128	55.6878	8.8630	0.0000	0.0000	0.0000	0.0000
7	0.1016	61.8217	9.8392	0.0000	3.9129	0.0000	0.0000





Undesired modes can be removed by deleting the Y-Trans and Z-Rotational lumped masses to restrict the DOF's to the X-axis. This gives the following first 4 modes (only 4 Eigenvalues requested) which are all X-direction modes only. Removal of undesired modes could also be achieved by preventing Y-translation and rotation with appropriate supports.



4.1 *Vibration Results Comparison*

Comparing S-FRAME'S results to the example there is excellent agreement.

S-FRAME

Eigen No	Period (T) Sec	Angular Frequency (w) Rad/Sec	Frequency (F) Hertz
1	0.585	10.736	1.709
2	0.183	34.306	5.460
3	0.102	61.822	9.839
4	0.072	86.926	13.835

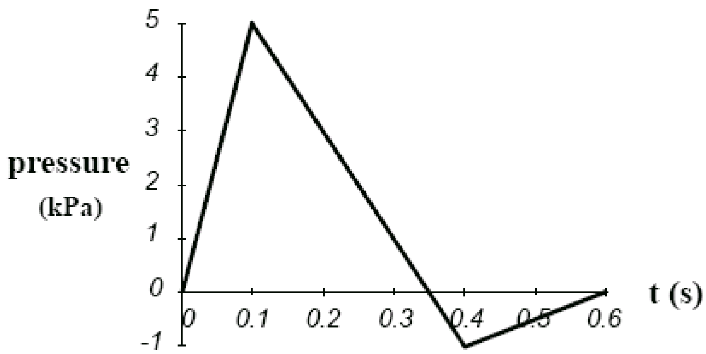
Reference

Mode	ω^2 (rad/s)²	ω (rad/s)	f (Hertz)	T (s)
1	115.22	10.73	1.708	0.59
2	1176.5	34.30	5.458	0.18
3	3820.2	61.80	9.836	0.10
4	7552.6	86.90	13.83	0.072

5 Time History Analysis

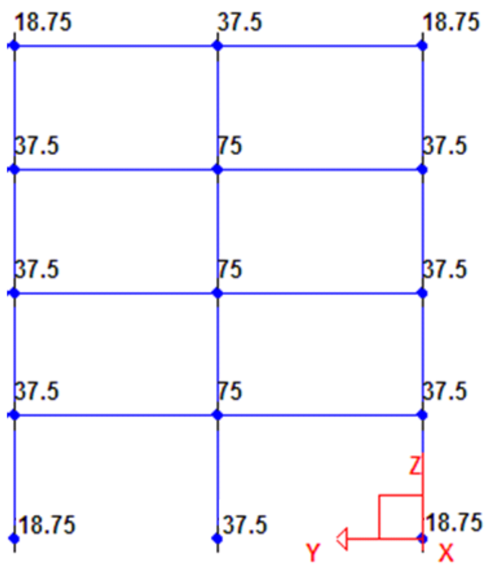
5.1 *Time History Loads*

The Example specifies the following linearly varying time dependent pressure vs time load which has; a total duration of 0.6s, a maximum of 5 kPa @ 0.1s and minimum of -1 kPa @ 0.4s. This must be ‘converted’ to force vs time functions which can be applied to discrete joints in the structure.

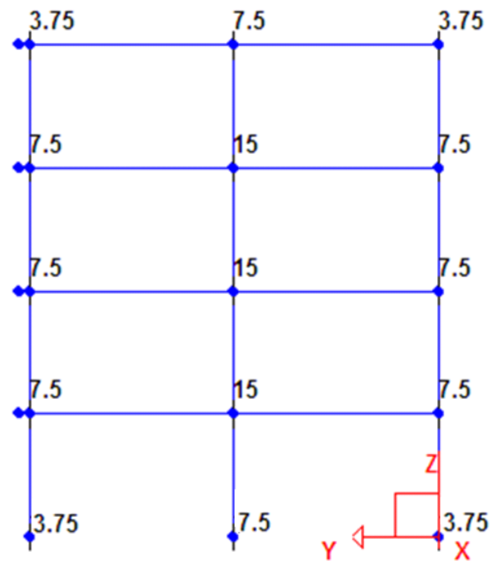


Since the example considers only the displacement of the floors, it is sensible to apply the load at the beam/column intersection joints. We calculate the tributary area of each joint and hence the maximum and minimum forces for the function at this joint.

Force for 5 kPa



Force for 1 kPa



Three functions are required;

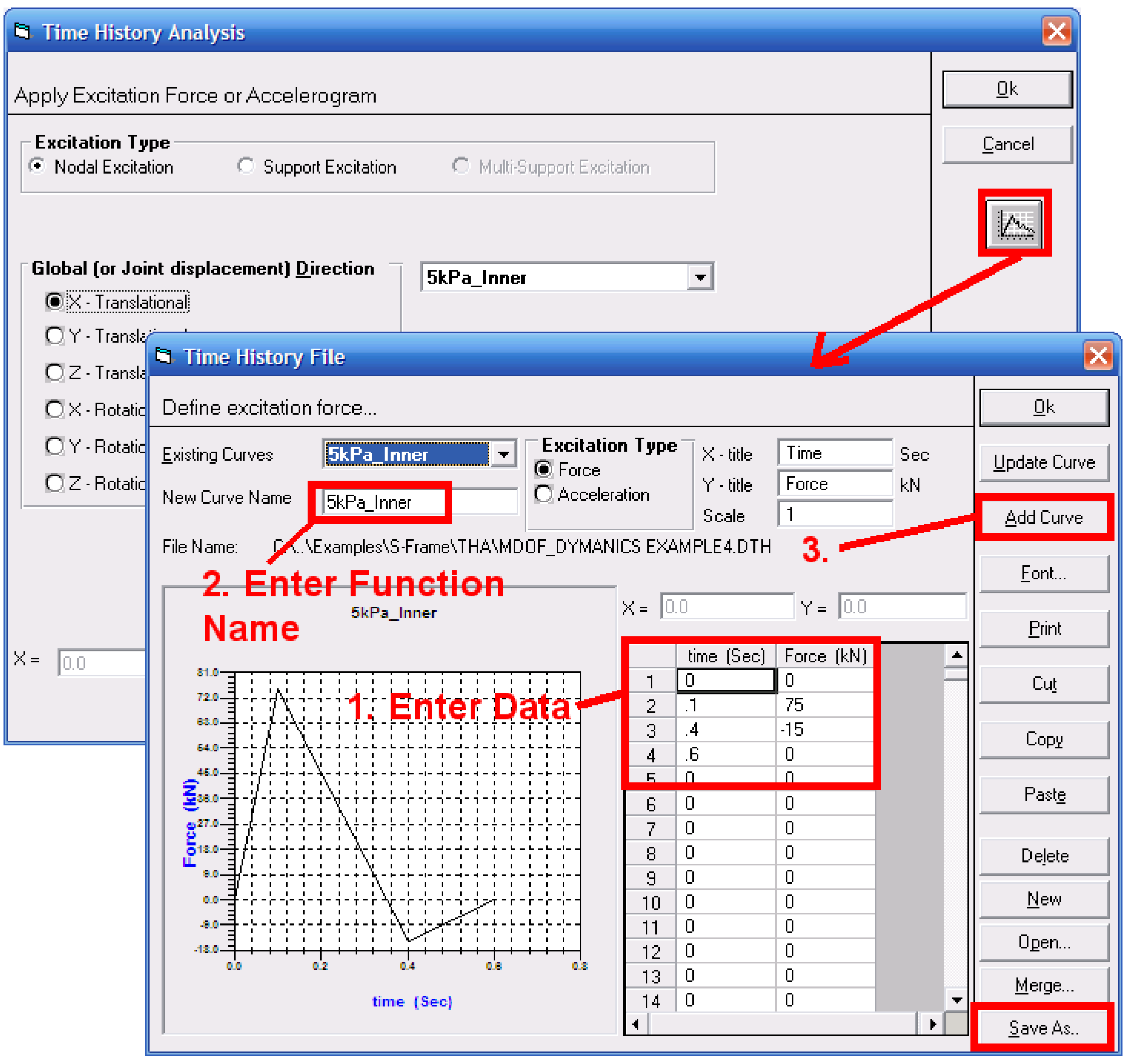
- Interior Joints; F_Max = 75 kN, F_Min = -15 kN
- Edge Joints; F_Max = 37.5 kN, F_Min = -7.5 kN
- Corner Joints; F_Max = 18.75 kN, F_Min = -3.75 kN

5.2 *Inputting Time History Loads*

First the required Force vs Time functions are created and saved to a file. Once the data is input and the curve ***Added*** the function is plotted and can be verified graphically. E.g. the following data is input for the function for 'Interior Joints':

Time (s)	Force (kN)
0	0
0.1	75
0.4	-15
0.6	0

The **Nodal Excitation** Type is set and the appropriate functions created and added to a Time History Data file (file type ***.DTH**) with the Function Tool.

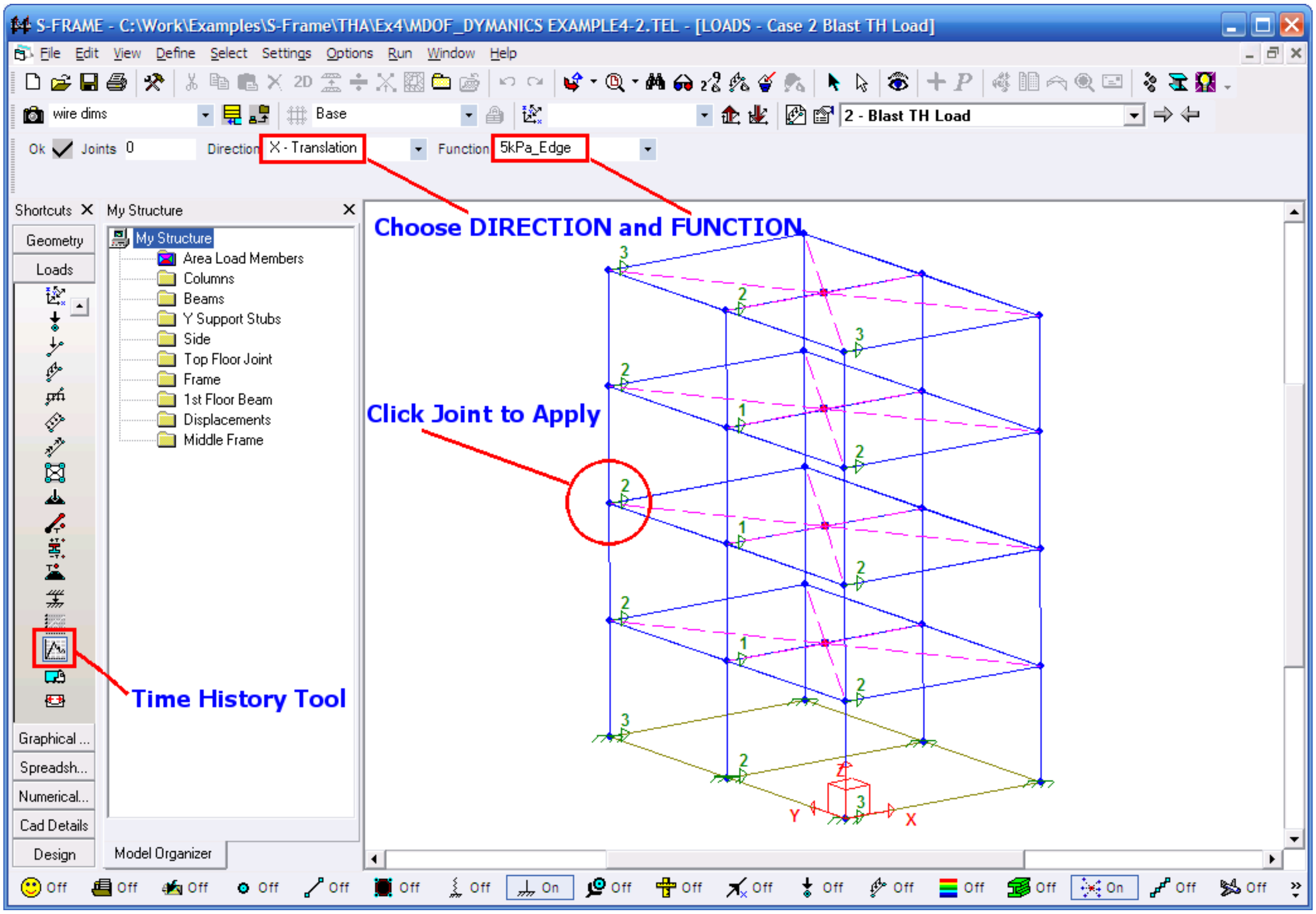


When all the required functions have been created the Time History Load file is saved. Note that this file is saved seperately from the model file – i.e. the time history load data is not held in the *.TEL model file.

5.2.1 Assigning Functions to Model

The functions are then assigned to the appropriate joints in the appropriate direction (X Translation) in the following manner. Each function is assigned a number so that correct assignment can be graphically verified.

1. Select the ***Direction*** to apply in – X-Translation
2. Select the ***Function*** to apply – e.g. function#1
3. Click on ***joint(s)*** using mouse to apply



5.3 Damping & Rayleigh Damping Coefficients

S-FRAME employs the Newmark direct time integration method for Time History Analysis, not a modal combination method. For more detailed information on theory and solutions see S-FRAME's Theory Manual, and References 1 and 2.

For the direct Integration methods damping can not be explicitly set to a single value for all modes. Damping is a function of frequency and is introduced via the ***Rayleigh Damping Coefficients*** α and β which are used to form the damping matrix **[C]**. It is important to note that these coefficients are not themselves damping values. For non-zero values of both α and β a desired value of damping ζ can be set precisely only for two frequencies, ω_r and ω_s. The damping value ζ is expressed as a percentage (of critical damping) and the frequencies ω are *angular* frequencies in terms of radians/s, not Hz. The values of α and β which produce precisely the desired damping value **<u>at two frequencies only</u>** can then calculated (from the following equation (see References 1. and 2) and input into S-FRAME for analysis.

$$\begin{Bmatrix} \alpha \\ \beta \end{Bmatrix} = \frac{2\xi}{\omega_r + \omega_s} \begin{Bmatrix} \omega_r \, \omega_s \\ 1 \end{Bmatrix}$$

Damping for other modes is a function of their frequency and hence will ≠ ζ (the desired damping). See the plots on the following pages for examples. Ideally the damping should be reasonably close to the desired value for modes which contribute significantly to the response. Hence a prior frequency analysis is generally required to determine the model's frequency characteristics and make a rationale choice of two frequencies for which to calculate α and β. If only one or two frequencies dominate the response then the choice is obvious. Otherwise two frequencies can be chosen which give a reasonable approximation to the desired damping for a range of frequencies in which the modes which contribute most to the response fall. Reference [1] states:

It is convenient to take $\boldsymbol{\omega_r}$ as the value of the fundamental frequency and $\boldsymbol{\omega_s}$ as the frequency corresponding to the last of the upper modes that significantly contribute to the response. This way the first mode and mode ***s*** will have exactly the same damping, and all modes in between will have somewhat smaller similar values and the modes with frequencies larger than $\boldsymbol{\omega_s}$ will have larger damping values thus reducing their contribution to response.

5.3.1 Damping Value ζ

The Example does not discuss the rationale for using ζ = 2%. See the table at the end of the document (page **<u>32</u>**), reproduced from Reference [2], for some recommended values. Inferring from the Example's material E value and section sizes that the building is concrete, the value could have been chosen for **"well-reinforced concrete (only slight cracking)"**.

For the Example, we could first choose Mode 1 and Mode 4 for initial values of α and β from:

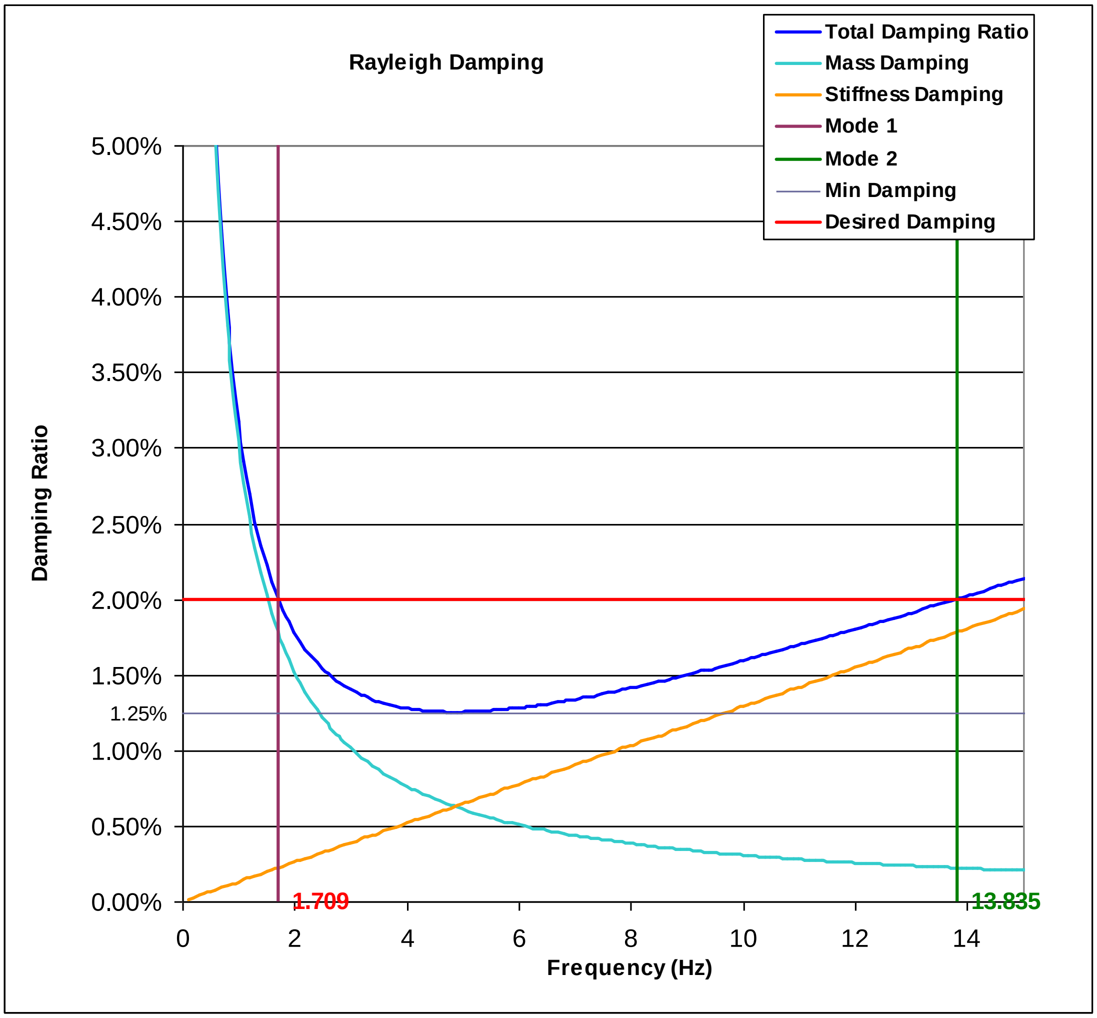
$$\alpha = \frac{2\xi\omega_r\omega_s}{\omega_r + \omega_s} \quad \text{and} \quad \beta = \frac{2\xi}{\omega_r + \omega_s}$$

Damping; ζ = 2% ; 1st mode; ω_r = 10.736; 2nd mode; ω_s = 86.926

$\alpha = 2\times\zeta\times\omega_r\times\omega_s/(\omega_r + \omega_s)$ = **0.382232**

$\beta = 2\times\zeta/(\omega_r + \omega_s)$ = **0.0004096**

These values produce the following damping/frequency relationship:



The damping for other frequencies can be determined preceisely from; $$\zeta_i = \frac{\alpha}{2\omega_i} + \frac{\beta\omega_i}{2}$$

Mode 2; f_2 = 5.46 Hz; ω_2 = 34.306; $\zeta_2 = \alpha/(2\times\omega_2) + (\beta\times\omega_2)/2$ = **1.260** %

Mode 3; f_3 = 9.839 Hz; ω_3 = 61.822; $\zeta_3 = \alpha/(2\times\omega_3) + (\beta\times\omega_3)/2$ = **1.575** %

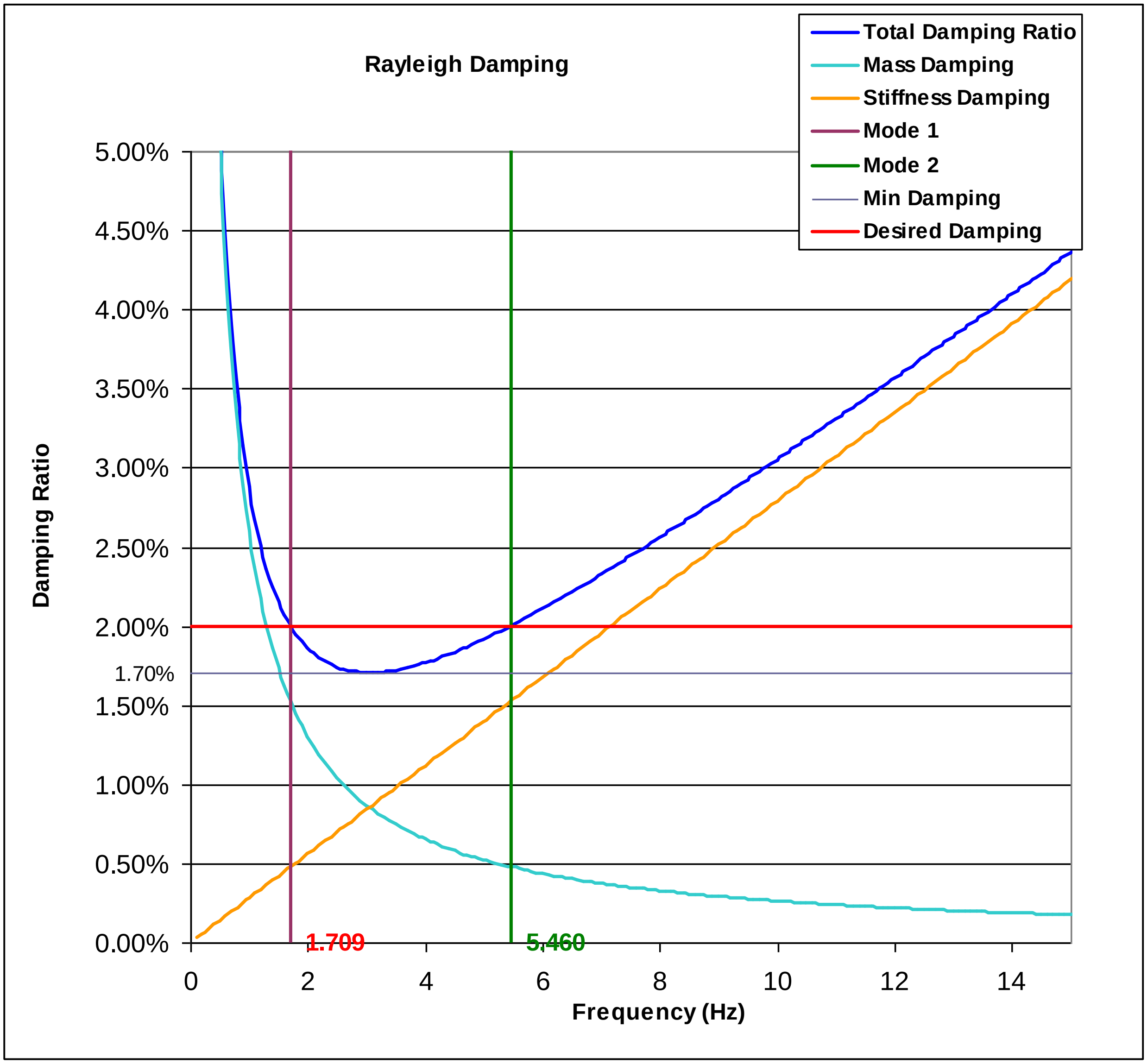
Alternatively Mode 1 and Mode 2 could be used giving:

1st mode; ω_r = 10.736; 2nd mode; ω_s = 34.306

$\alpha = 2\times\zeta\times\omega_r\times\omega_s/(\omega_r + \omega_s)$ = **0.327081**

$\beta = 2\times\zeta/(\omega_r + \omega_s)$ = **0.0008881**

These values produce the following damping/frequency relationship:



The damping for other frequencies > 2%;

Mode 3; f_3 = 9.839 Hz; ω_3 = 61.822; $\zeta_3 = \alpha/(2\times\omega_3) + (\beta\times\omega_3)/2$ = **3.010** %

Mode 4; f_4 = 13.035 Hz; ω_4 = 86.926; $\zeta_4 = \alpha/(2\times\omega_4) + (\beta\times\omega_4)/2$ = **4.048** %

5.4 Constant time step size

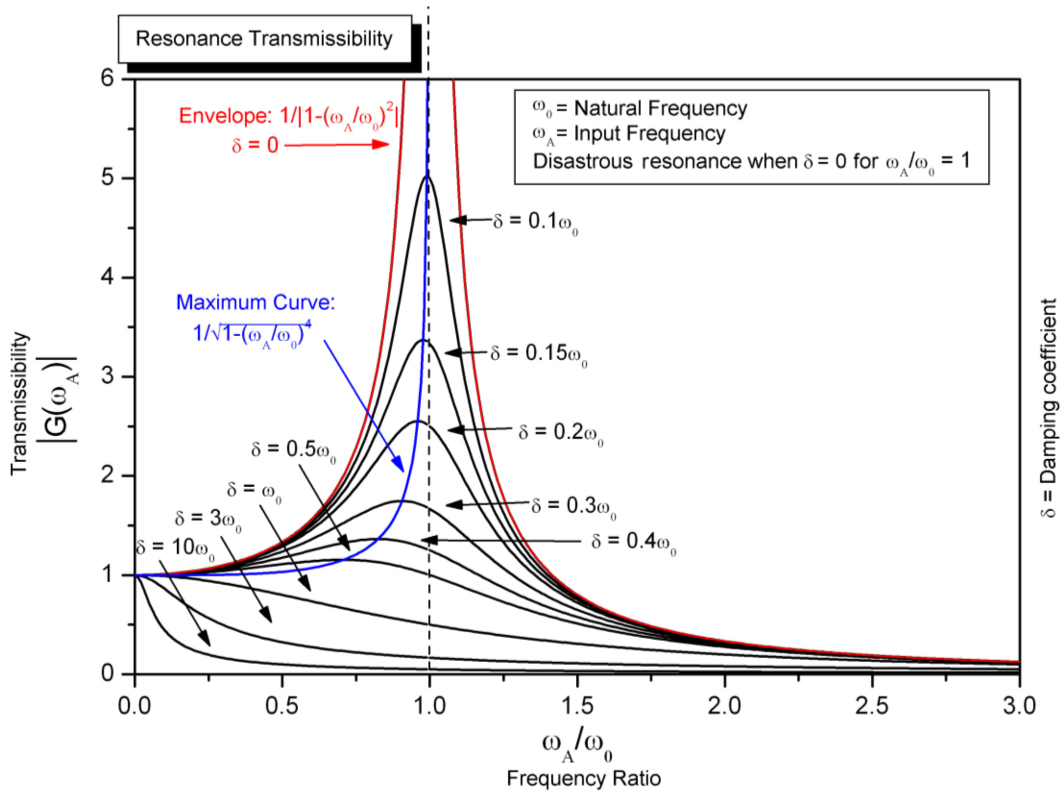
Next the analysis Time Step size Δt is considered. The time step size should be sufficiently small for accurate analysis. However, the smaller the time step size, the higher the 'cost' (in terms of computational time) of analysis for a given *duration* of analysis – e.g. the Example's first 2.5s of response. Additionally the analysis duration is limited by the maximum number of allowable time steps which is **32,767** (a limit imposed by current software architecture). So ideally the time step size should be no smaller than it need be. The S-FRAME Theory Manual pg 34 gives a rule of thumb for Δt as follows:

> frequencies up to Ω^*. As a rule of thumb we recommend that the user-selected constant time step size satisfies the following condition
>
> $$\Delta t_{cr} \leq \frac{2\pi}{20\Omega^*} \qquad (68)$$

Where Ω = the highest (angular) frequency component of the forcing function in rads/s and $\Omega^* = 4\Omega$. Since we more commonly think in terms of period the above expression can be conveniently re-formulated as follows:

$$T = \frac{1}{f} = \frac{2\pi}{\Omega} \quad \Rightarrow \Delta t_{cr} \leq \frac{T_f}{20 \times 4} = \frac{T_f}{80}$$

Where $T_f = 1/\Omega$. However, what if T_f is unknown or not applicable? The rule of thumb derives from two principles, the first of which is illustrated by the following figure



Principles:

1. Refer to the figure above; for a given forcing (or *input*) frequency Ω (ω_A in the figure) the response of the system for a given *natural* frequency ω_i (ω_O in the figure) generally reduces as the frequencies diverge. The divergence is conveniently expressed in terms of the ratio of the forcing frequency to the natural frequency Ω/ω_i. The figure illustrates this phenomenon. Furthermore it can be seen that the response in modes with a **small** ratio Ω/ω_i is essentially *static* and the response in modes with a **large** ratio Ω/ω_i is negligible. Below Ω/ω_i = 1/4 further increases in ω_i do not produce significant change in response hence frequencies (of response) higher than $4\Omega = \Omega^*$ need not be considered.
2. Around **20** equal time intervals are required to discretize both the input and response motions with sufficient accuracy.

These principles explain the origin of the values of ***4*** and ***20*** in the rule of thumb. Thus a rational choice of time step size is based on a) deciding which is the highest frequency required to be discretized considering both input ***and*** response and b) dividing the resulting (lowest) period by 20.

From the foregoing discussion we can derive a more general and practical rule of thumb. Let us call the highest frequency of the ***response*** f_r with corresponding period T_r.

- If T_f is known or guessed at to a reasonable degree then it is not necessary to know T_r (following Principle 1) and from Principle 2. we set the time step $\Delta t = (T_f /4)/20 = T_f /80$.
- If T_f is unknown or inapplicable (as in Example 4) we consider the likely highest significant ***response*** frequency f_r = 1/T_r and set $\Delta t = T_r /20$.

For the Example T_f is inapplicable and T_r is known, being the period of mode 4.

$T_r = T_4 = 0.072$ s; $\Delta t \leq$; $T_r/20$ = **0.0036** s

A value of Δt = 0.00125 was chosen since it is < the maximum calculated above and conveniently gives a time step at **t = 0.2875s** which is close to **t = 0.2873s** for which the Example gives results.

5.5 Analysis Duration

The Example's analysis duration of 2.5s is used. As discussed previously, the duration has a maximum possible value governed by the time step size: Duration Limit; $T_{A\_max} = 32{,}767\ \Delta t$

For a constant time step, the analysis duration is not explicitly entered; the user inputs the time step size Δt and total time steps N from which the duration derives.

Δt = 0.00125s; Desired analysis time; T_A = 2.5s; total time steps; $N = T_A/\Delta t$ = **2000**

The peak response may occur during the ***free-vibration*** portion of response – i.e. after the exciting function has ended – and the time to peak response (or steady state in the case of a periodic forcing function) may not be known with any great certainty. Hence in practice it is generally sensible to continue analysis beyond the end of excitation where applicable and some experimentation may be required with analysis duration having viewed initial results.

5.6 Newmark Coefficients

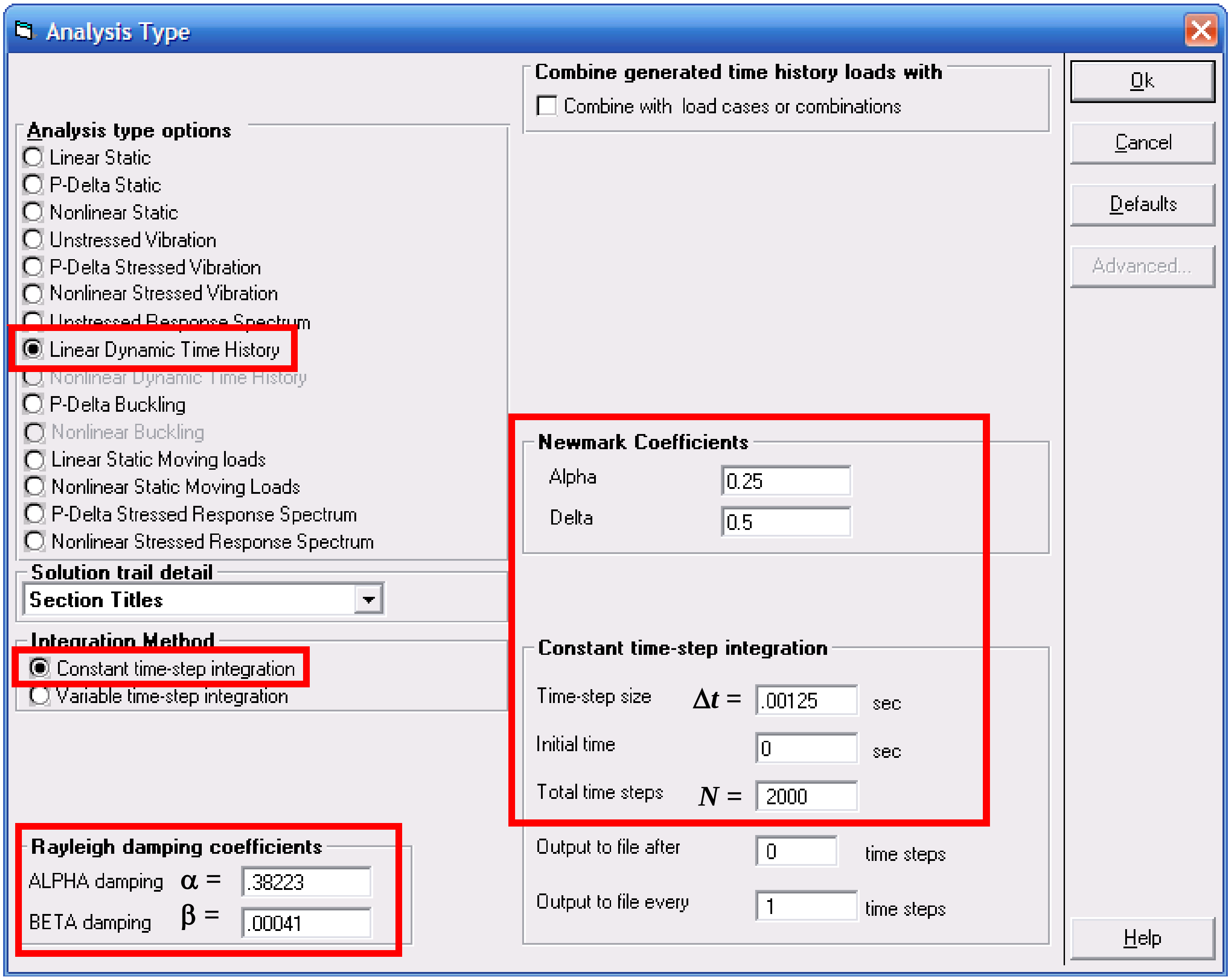
These are generally one of two sets of values as follows. See S-FRAME Theory Manual and References for more information.

	Alpha	**Beta**
Zero Damping	**0.2525**	**0.5050**
Non-zero Damping	**0.25**	**0.5**

Since the example has a specified non-zero damping the appropriate values are input in S-FRAME.

5.7 Time History Analysis Settings

The preceding consideration leads to the following initial Linear Dynamic Time History analysis settings



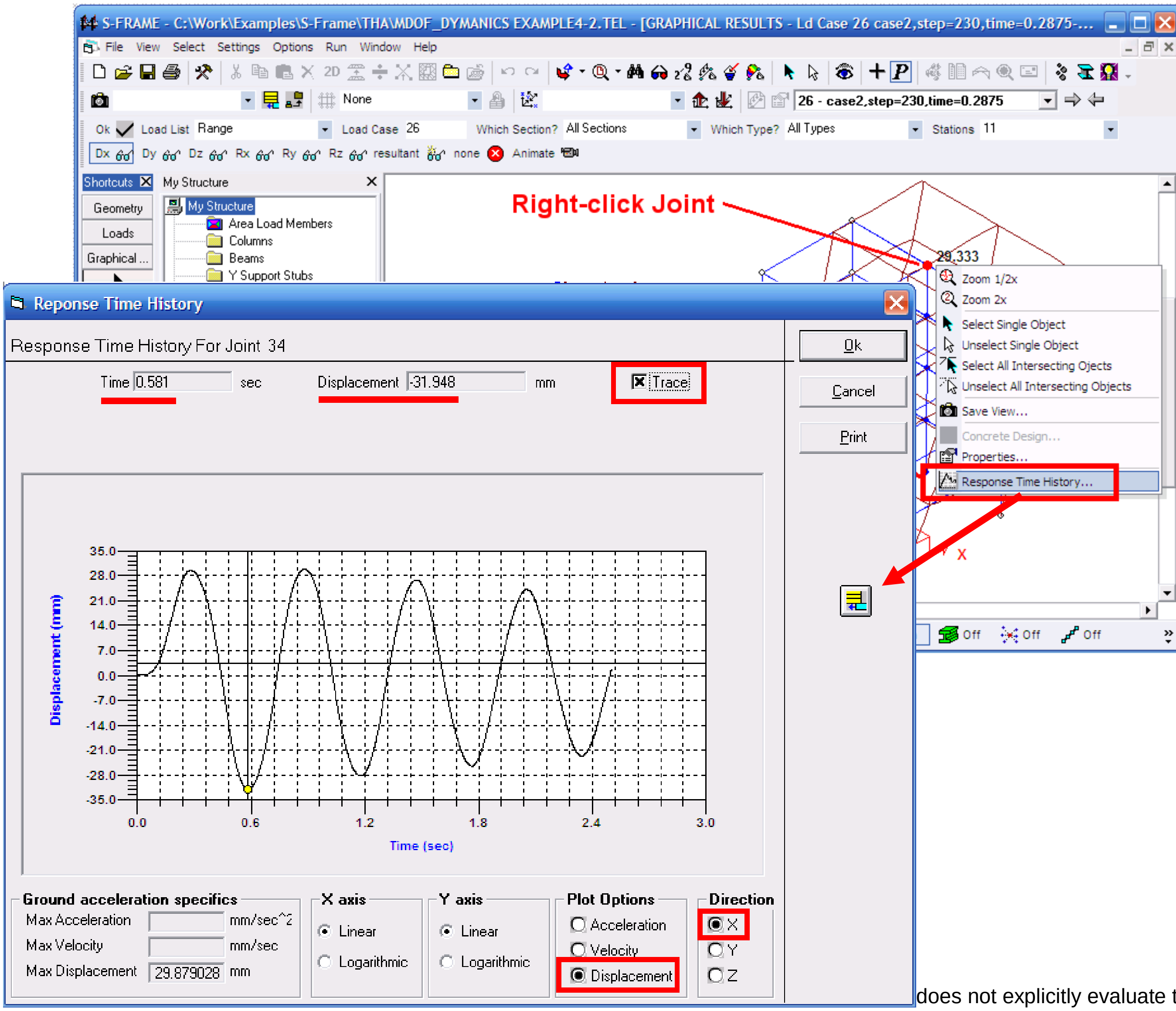
The Rayleigh damping coefficient values are those calculated on page **15 above** for Modes 1 and 4.

The other Constant time-step integration parameters are discussed later (see pg **25**) and are not considered at this stage.

6 RESULTS

6.1 *Time History Response*

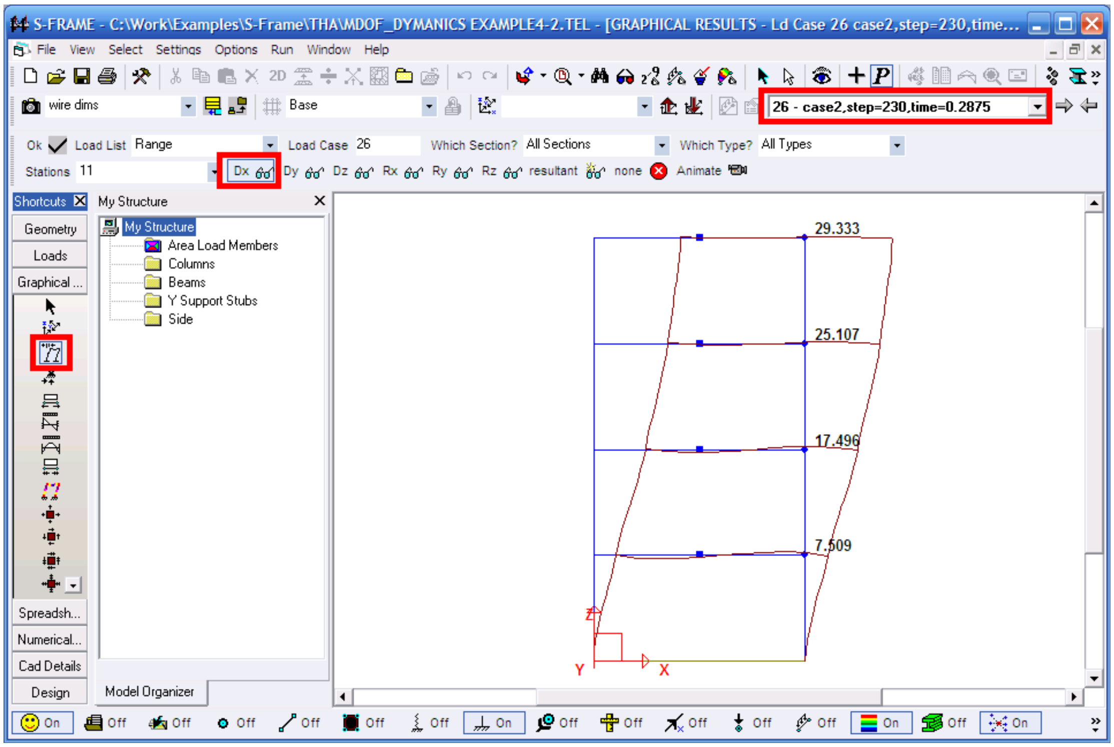
The time history response at a chosen joint can be readily assessed by right-clicking the joint and choosing '**Response Time History…**' from the context menu: S-FRAME plots the chosen result parameter – e.g. X-Displacement – vs time so the user can easily identify the maximum response and the approximate time at which this occurs. There is a **Trace** function to assist with this – using this it can be seen that the maximum -ve
X-displacement response occurs at around 0.58s for example.



does not explicitly evaluate the response of each mode as per the Example's method as this is not usually required, the total response generally being of primary interest.

6.2 Deflections

More detailed results for each time step are also available. S-FRAME presents each time step as a discrete loadcase for which all the usual results (both Graphical and Numerical) are available as for a static analysis. We wish to compare the displacement results, so the displacement diagram is chosen, viewing of ***Dx*** (X-displacement) values is enabled and time step 230 @ time = 0.2875 s is selected.



The results are given (in meters) on page 33 of the Example for **t = 0.2873s**

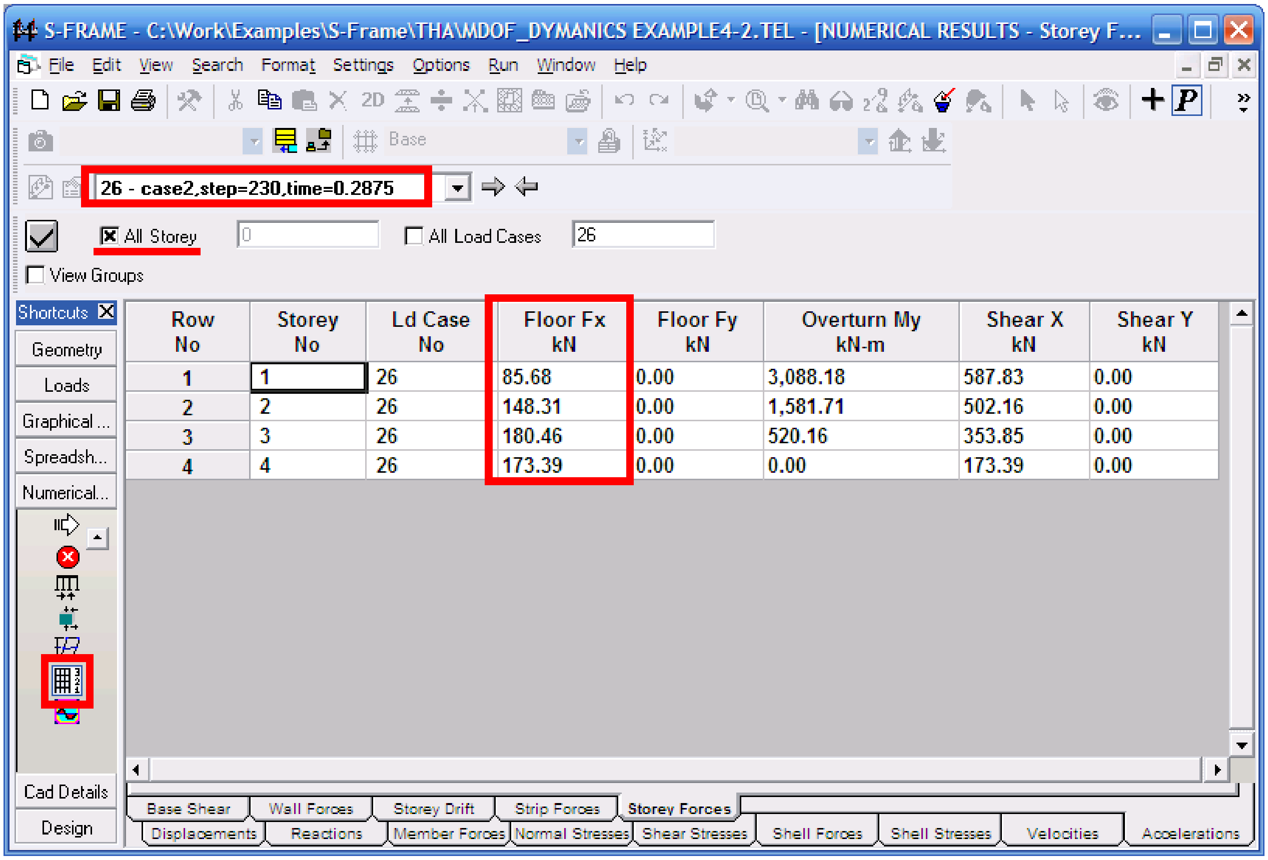
$$\begin{Bmatrix} U_4 \\ U_3 \\ U_2 \\ U_1 \end{Bmatrix} = \begin{Bmatrix} 0.029373 \\ 0.025103 \\ 0.017455 \\ 0.007479 \end{Bmatrix}$$

6.2.1 Comparison

Floor Displacement (mm)	Reference	S-FRAME
4th Floor	29.373	29.333
3rd Floor	25.103	25.107
2nd Floor	17.455	17.496
1st Floor	7.479	7.509

6.3 *Floor Forces*

Floor Forces which can be compared with those in the Example are available in Numerical Results.



The Example gives results for t = 0.2873s

$$\{F\} = [K_E]\{U\} = \begin{Bmatrix} 175.97 \\ 180.18 \\ 145.05 \\ 83.75 \end{Bmatrix}$$

6.3.1 Comparison

Floor Force (kN)	Reference	S-FRAME
4^{th} Floor	175.97	173.39
3^{rd} Floor	180.18	180.46
2^{nd} Floor	145.05	148.31
1^{st} Floor	83.75	85.68

6.4 Sensitivity to Rayleigh Damping Coefficients

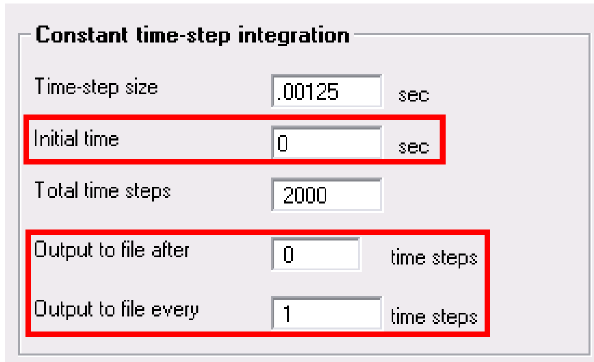
If the α and β values calculated for Mode's 1 and 2 (see page **16**) are used the following results are obtained (**Damping 2**). While these generally give marginally better agreement, this serves to illustrate that results for this example are relatively insensitive to the choice of the second frequency (ω_S) for deriving the damping coefficients, since the response is dominated by that of mode1. However, this may not always be the case. In practice some experimentation may be required with the choice of frequencies and hence damping coefficient values.

Floor Displacement (mm)	Reference	S-FRAME Damping 1	S-FRAME Damping 2	% Change
4^{th} Floor	29.373	29.333	29.352	0.06%
3^{rd} Floor	25.103	25.107	25.104	-0.01%
2^{nd} Floor	17.455	17.496	17.473	-0.13%
1^{st} Floor	7.479	7.509	7.493	-0.21%

Floor Force (kN)	Reference	S-FRAME Damping 1	S-FRAME Damping 2	% Change
4^{th} Floor	175.97	173.39	174.61	0.70%
3^{rd} Floor	180.18	180.46	180.62	0.09%
2^{nd} Floor	145.05	148.31	146.2	-1.42%
1^{st} Floor	83.75	85.68	85.00	-0.79%

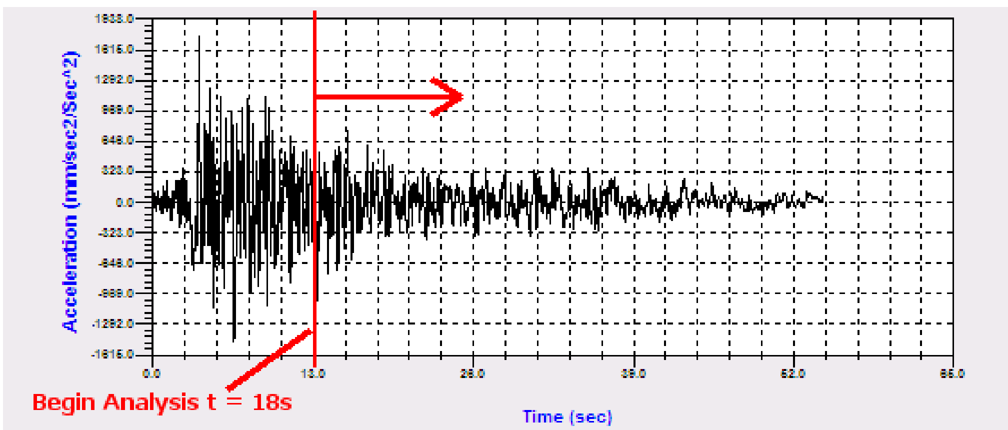
7 Further Output Parameters

Further analysis options are also available for Constant time-step integration.



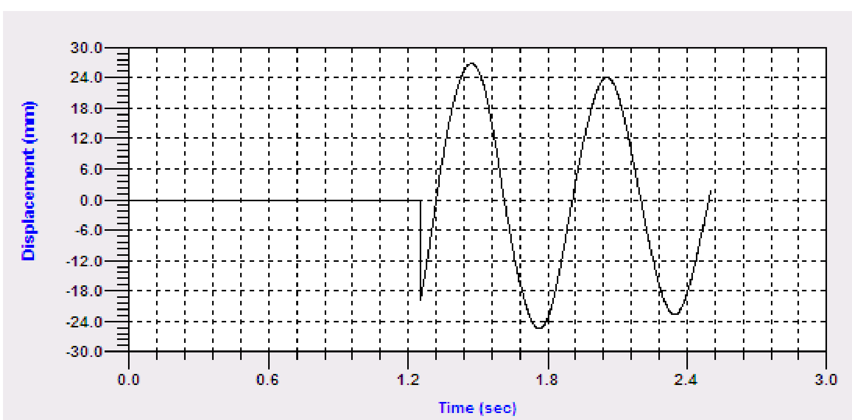
Initial time 18 sec

This parameter applies to the *input record* and is the time at which S-FRAME begins to read data from the input function for analysis. This might be employed to use only part of a long record.



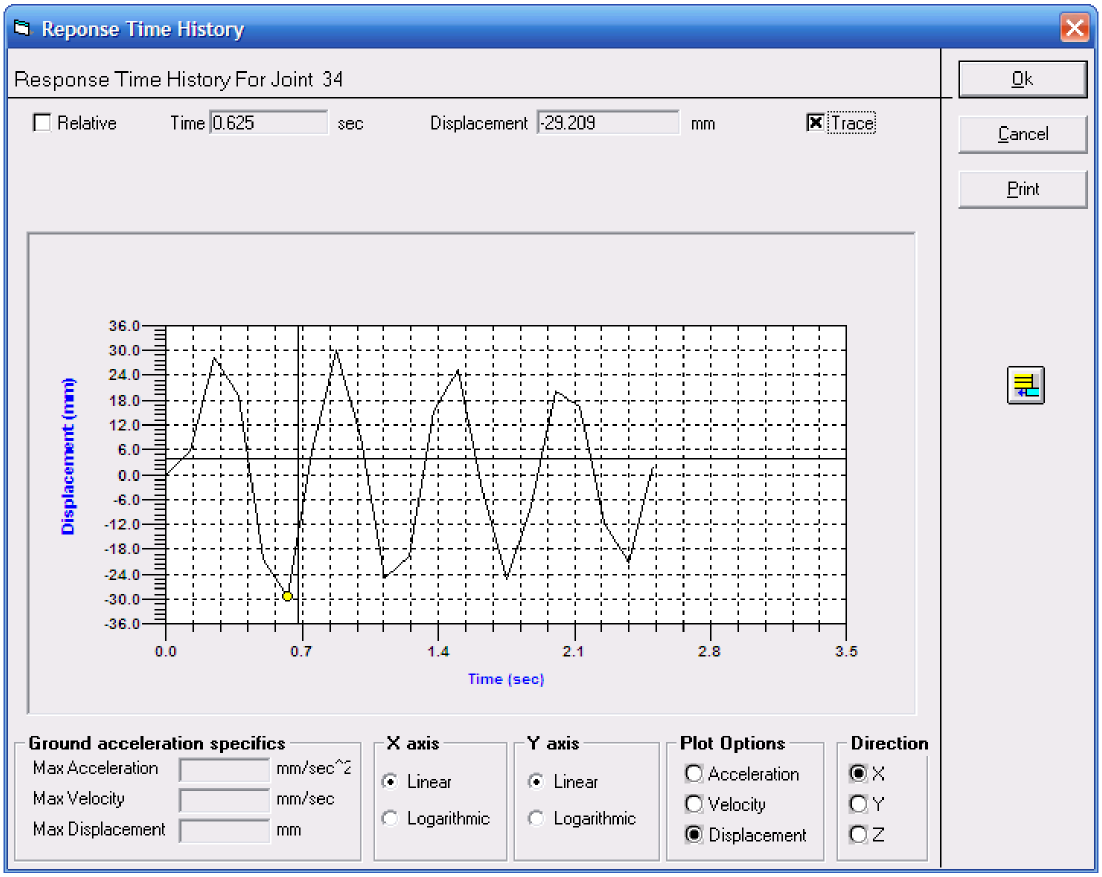
The final two parameters can be used to minimize the amount of analysis output – i.e. the number of time step result cases. They can best be understood by viewing their effect on the response result plot of this example.

Output to file after 1000 time steps



N and Δt remain as before – the analysis solution is unchanged but output (to result files) only begins at t = 1000×Δt = **1.25s**. In this example this results in the peak response, which occurs before t = 1.25s, being missed. This option might be used to limit output to a portion where steady state is achieved for a periodic input.

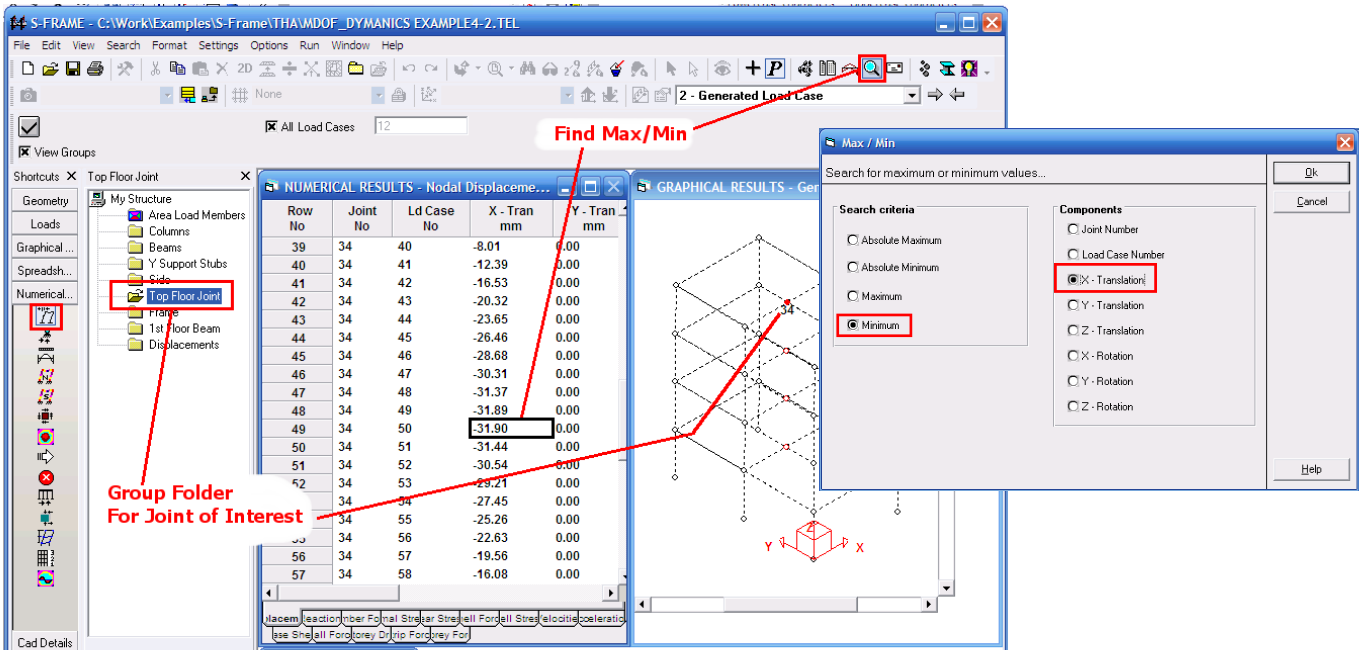
Output to file every 100 time steps



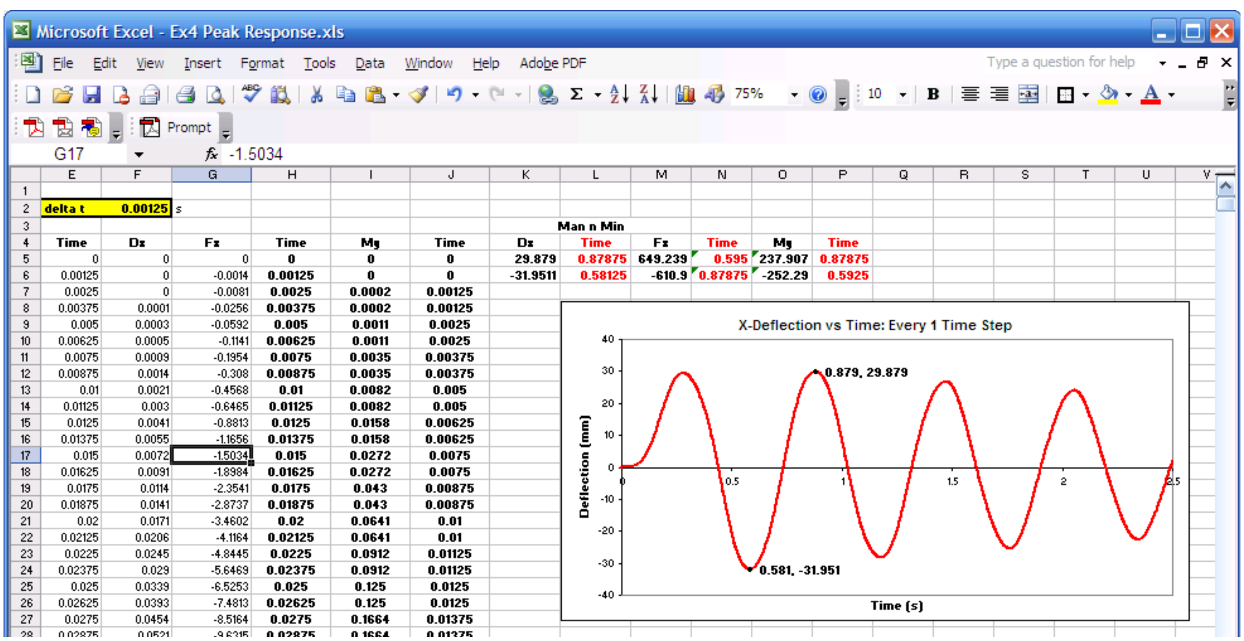
N and Δt remain as before – the analysis solution is unchanged but results are only available at time intervals = 100×Δt = **0.125s**. This produces a more crude record of the response which may not capture some peak values with sufficient accuracy.

8 Practical Issues; Peak Response and Output Reduction

The Example does not discuss Peak Response but this will usually be of primary interest. This can be estimated for certain values using the Response Plot disucussed previously. ‘Exact’ values can be found in the Numerical Results Spreadsheet using the **Find Max/Min** function when viewing results for All Load Cases (i.e. Time Steps)



Additionally, the Numerical Results Spreadsheets can be exported wholesale (via a simple copy/paste operation) to a dedicated spreadsheet application like Microsoft Excel where customized response plots can be reproduced of all results and maxima and minima easily found. This was done to produce the following plots.



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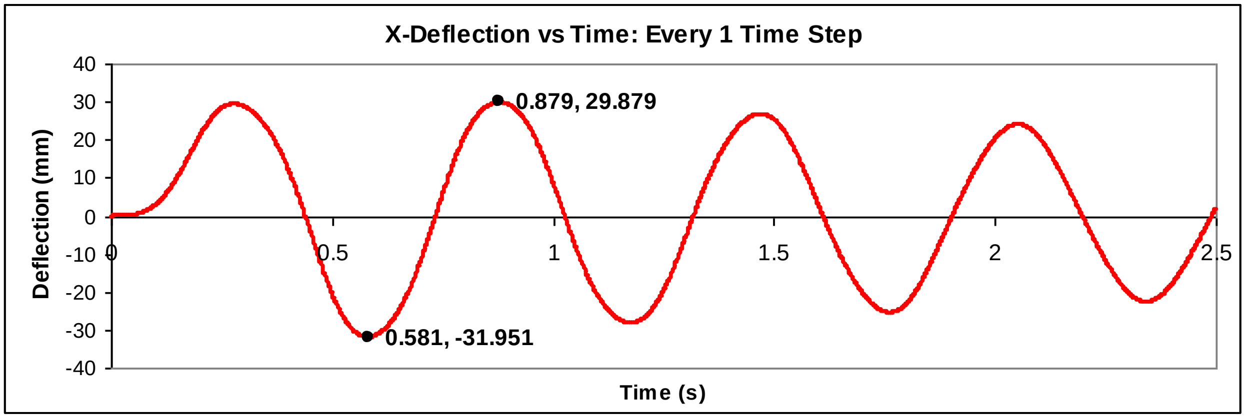
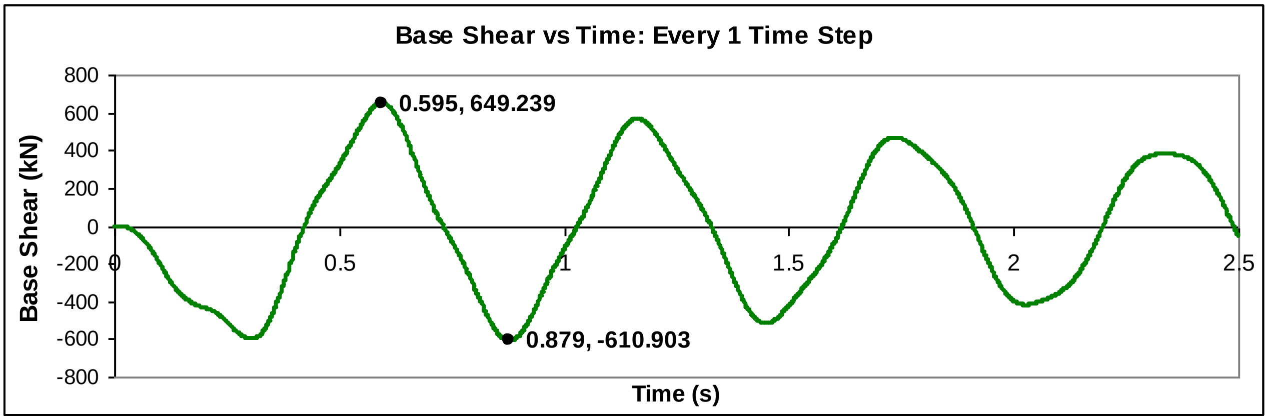
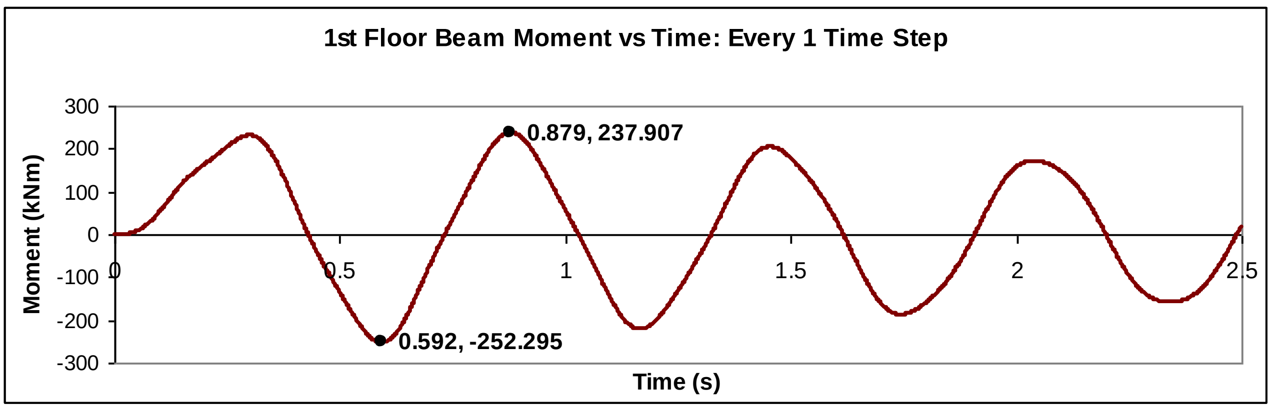
The following plots displaying the response for 2.5s and the max/minima were produced in this manner using data for every 1 time step for; **Roof Displacement** (X-direction), **Base Shear** and **Maximum (end) Moment** in a 1st floor beam (parallel to X-axis)



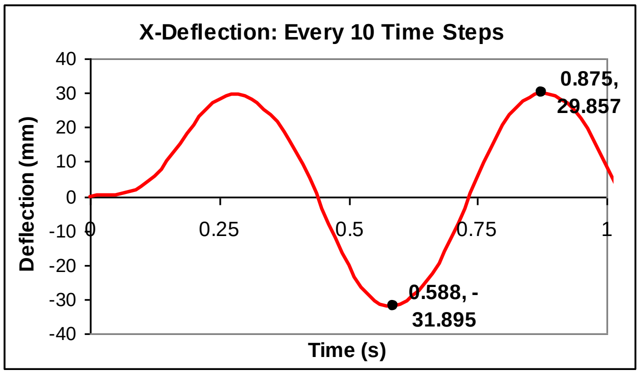


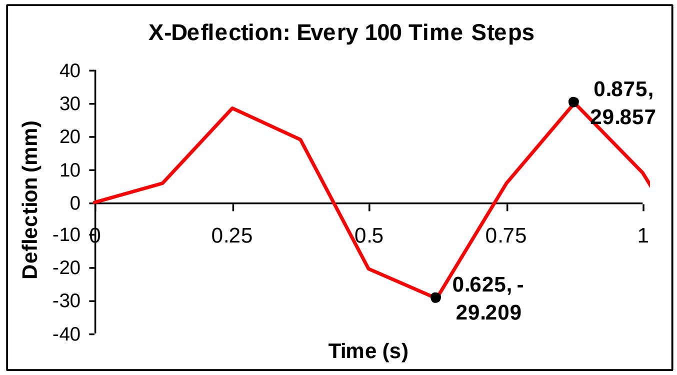


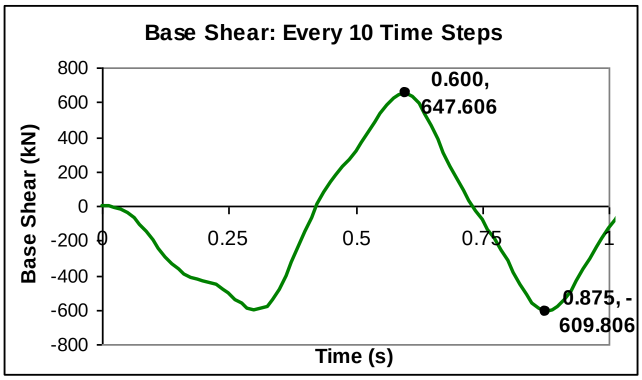
It is interesting to note that the peak positive displacement, positive moment and negative base shear occur during the free vibration portion of response – i.e. after the cessation of excitation. It can also be seen that the response in general is dominated by that of the 1st mode, the period of response peaks being around 0.6s. Finally, we see that the peak values, which would be used for maximum design values, all occur at t < 1s. Thus analysis duration could sensibly be reduced to 1s to minimize analysis cost (i.e. time for an analysis run) and result-processing demand.

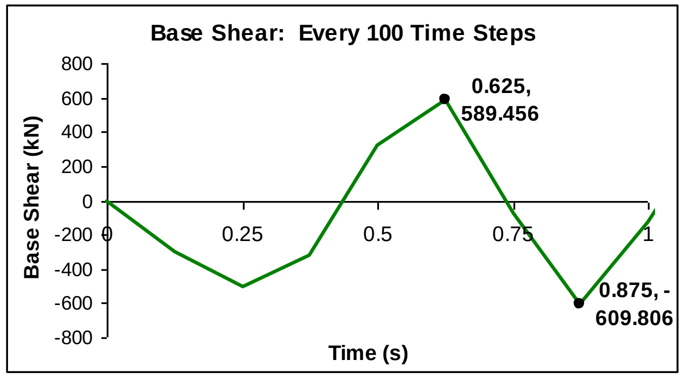
8.1.1 Output Every __ Time Steps

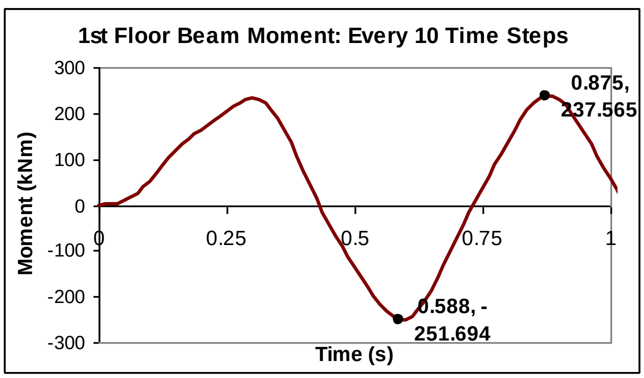
The following plots illustrate the effect of Ouput **Every** *10* and **Every** *100* time steps – clearly this gives an increasingly approximate record of the response.

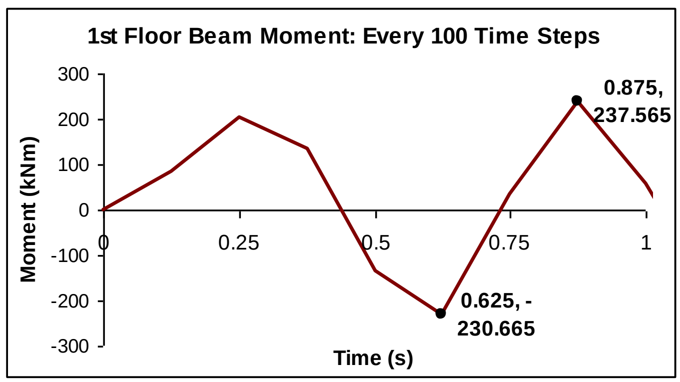












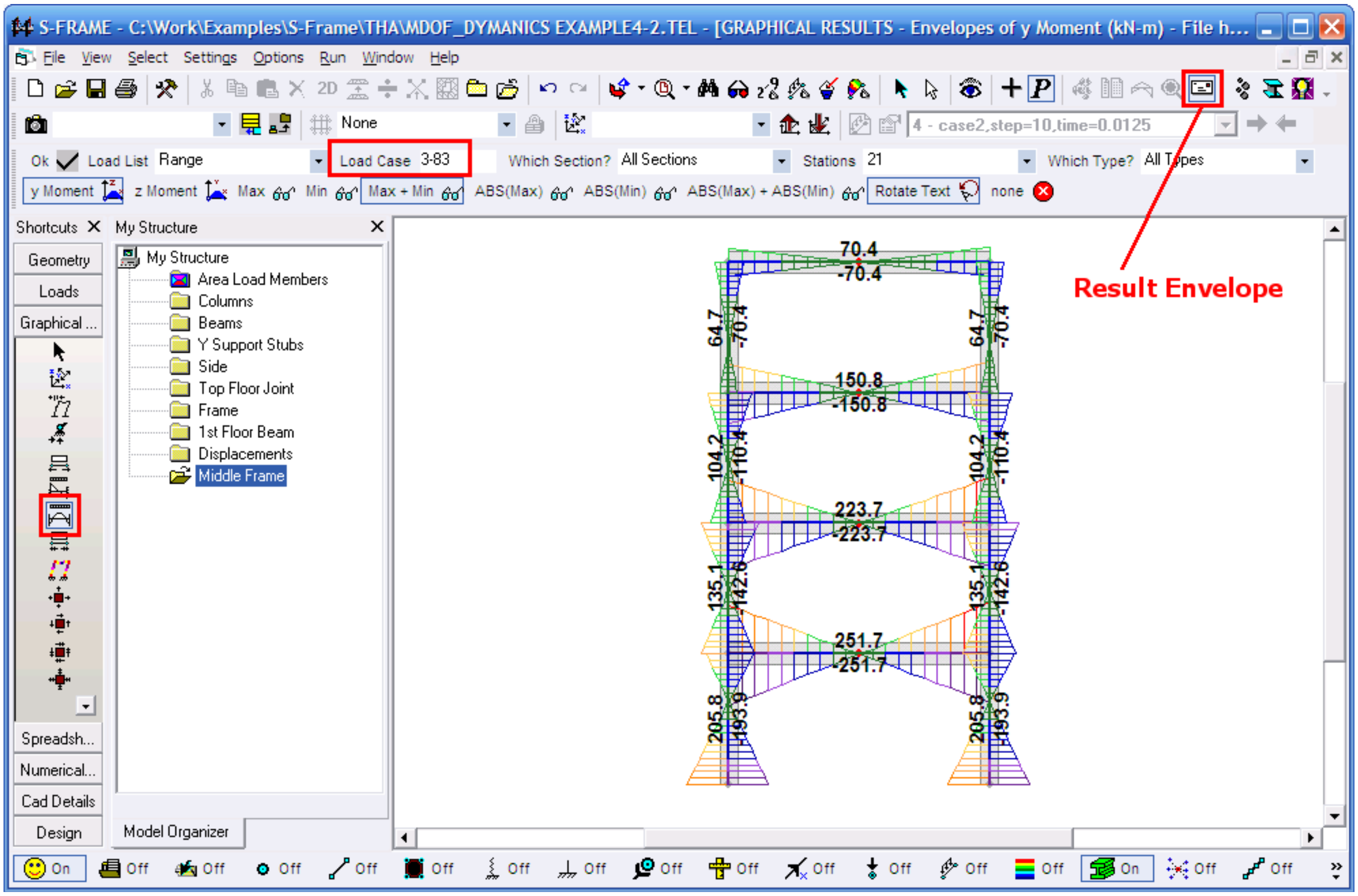
Comparing with the plots for every *1* step; for ouptut every *10* steps the change in peak values is insignificant, while every *100* steps significantly under estimates some;

	Every 1	Every 10	% diff	Every 100	% dif
+ve Base Shear (kN)	649.2	647.6	**-0.25%**	589.5	**-9.20%**
-ve Beam Moment (kNm)	-252.3	-251.7	**-0.24%**	-230.7	**-8.56%**

Output every *10* time steps could thus sensibly be used to reduce output and improve post-analysis operations.

8.2 Further Design Values

Using the options discussed above to minimize analysis cost and output, S-FRAME's Envelope function can be efficiently used to produce envelopes of design forces for members for example – e.g. Moment:



9 REFERENCES

[1] Luis E. García & Mete A. Sozen, Multiple Degrees of Freedom Structural Dynamics, Purdue University CE571 – Earthquake Engineering, 2002

[2] Anil K. Chopra, Dynamics of Structures: Theory and Applications to Earthquake Engineering (2nd Edition), Prentice Hall, 2000

10 Suggested Values for Damping Ratio

The following Table is reproduced from Reference [2].

Stress Level	Type and Condition of Structure	Damping Ratio (%)
Working stress, no more than about $\frac{1}{2}$ yield point	Welded steel, prestresses concrete, well-reinforced concrete (only slight cracking)	2-3
	Reinforced concrete with considerable cracking	3-5
	Bolted and/or riveted steel, Wood structures with nailed or bolted joints	5-7
At or just below yield point	Welded steel, prestressed concrete (without complete loss in prestress)	5-7
	Prestressed concrete with no prestress left	7-10
	Reinforced concrete	7-10
	Bolted and/or riveted steel, Wood structures with bolted joints	10-15
	Wood structures with nailed joints	15-20