TECTONUS
RESILIENT SEISMIC SOLUTIONS

STRUCTURAL MODELLING & DESIGN GUIDE

CUTTING EDGE SEISMIC PROTECTION TECHNOLOGY

DESIGN WITH TECTONUS

DESIGN FREEDOM FOR THE FUTURE

With the Tectonus RSFJ seismic systems, Engineers have the freedom to choose which design approach to determine the seismic demands on the RSFJ devices.

The principal design approaches are:

Force-Based Design (FBD),

Displacement-Based Design (DBD) and

Non-Linear Time History Analysis (NLTHA).

In this guide, the first 2 are described and explained with a simple example for reference.

Engineers are encouraged to discuss their modelling and findings with the engineering team at Tectonus to make sure that the optimum solution is provided for the project. Whatever design method is used (FBD or DBD), a cyclic non-linear push-over structural analysis is to be performed. This is easily performed when using advanced non-linear analysis capable software.

PROJECTS BEYOND THE USUAL

Tectonus offers a standard range for the RSFJ connections however, it is easy to adopt the connections in parallel or in custom specifications to the project - well above 500kN.

DISPLACEMENT BASED DESIGN

This method is based on the principle that displacements are controlled and the member force demands are determined to meet those displacement limits. An equivalent single-degree of freedom model is determined with an equivalent stiffness, representing the actual structure.

From this simple model, a general structural response is determined based on the SLS and ULS seismic demands. From this structural response, the characteristics of the RSFJs in the structure are determined and a non-linear push-over analysis is conducted to ensure that the structural model is in accordance with the DBD model response. More details and an example using the DBD approach are provided in this catalogue.

FORCE-BASED DESIGN

This is the most common approach but also the most conservative one. In this approach, an equivalent ductility is assumed at the start.

Structures with the RSFJ technology will easily reach equivalent ductility values in the order of 3. Then, using the procedure in NZS 1170.5, seismic forces are determined and a numerical model analysed for seismic demands. The results of this first linear analysis are then used to determine the characteristics of the RSFJs within the Lateral Load Resisting System. From the numerical model incorporating the RSFJs, a non-linear static push-over analysis is performed. The results of this analysis are used to verify the member forces and structure drifts against the criteria. More details and an example using the FBD approach are provided in this catalogue.

TECHNICAL SUPPORT

The Tectonus engineering team offers modelling support for any method chosen for designing with the RSFJ. The team offers workshops, technical sessions and training to help facilitate project and modelling development. More information and resources can be found on the tectonus.com website.

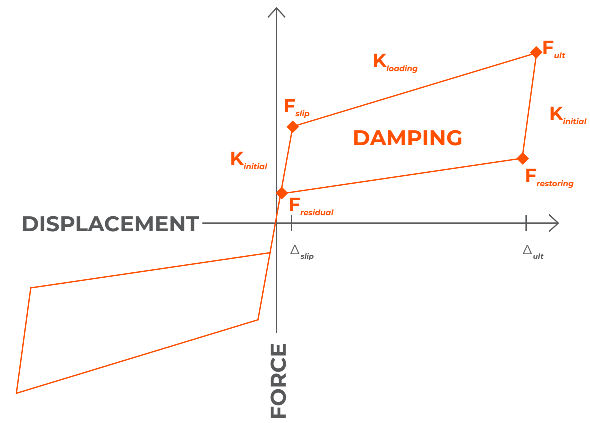
STRUCTURAL MODELLING IN ETABS/SAP2000

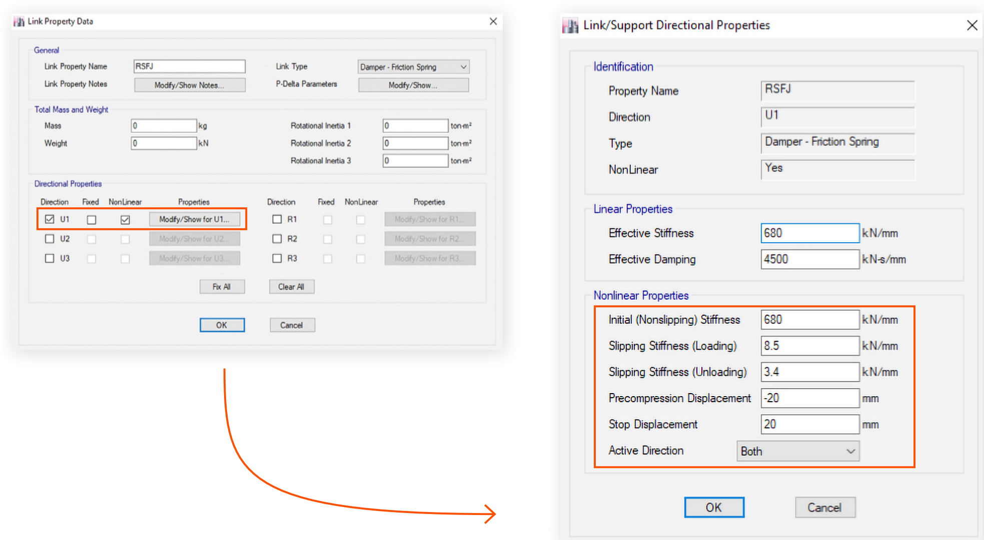
The RSFJ can be easily integrated in the structural analysis and design software ETABS and SAP2000. It allows the designer to accurately calibrate the parameters according to the requirements of the project.

In ETABS/SAP2000, the RSFJ can be modelled using the "Damper – Friction Spring" link Element. This function accurately represents the flag-shaped hysteresis of a RSFJ provided its parameters are properly calibrated in accordance with the design parameters of the joint. The parameters can be defined for any of the six translational or rotational degrees of freedom.

The design parameters of the RSFJ are:

- / F_{slip} (slip force of the RSFJ)
- / F_{ult} (ultimate force in the RSFJ at the end of loading)
- / $F_{restoring}$ (restoring force of the RSFJ)
- / $F_{residual}$ (residual force in the RSFJ at end of unloading)
- / Δ_{slip} (initial elastic deflection of the RSFJ before slip)
- / Δ_{ult} (ultimate displacement of the RSFJ)
- / $K_{initial}$ (initial stiffness of the RSFJ before slip)
- / $K_{loading}$ (loading stiffness of the RSFJ)
- / $K_{unloading}$ (unloading stiffness of the RSFJ)





Damper – Friction Spring design parameters:

- / Initial (Nonslipping)Stiffness = $K_{initial}$
- / Slipping Stiffness (Loading) = $K_{loading}$
- / Slipping Stiffness (Unloading) = $K_{unloading}$
- / Pre-compression displacement = Δ_{slip} - (F_{slip} /$K_{loading}$)
- / Stop displacement = Δ_{ult}
- / Active direction (Tension/Compression/Both): should be specified based on the application requirement.

By defining these parameters, the rest of the RSFJ parameters (Δ_{slip}, F_{ult}, $F_{restoring}$ and $F_{residual}$) will be automatically adjusted.

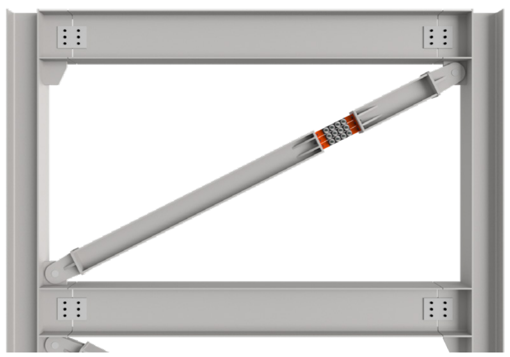
LATERAL LOAD RESISTING SYSTEMS

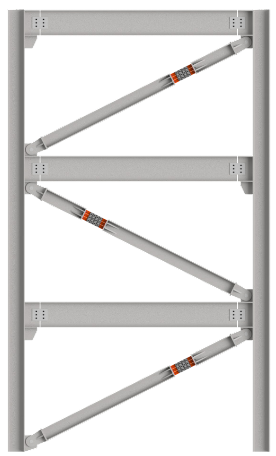
TENSION & COMPRESSION DIAGONAL BRACES

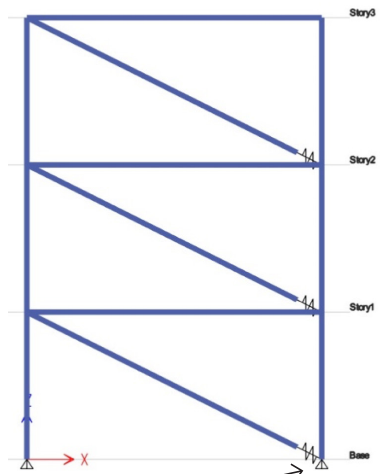
RSFJ-BRACE

The RSFJ brace can be easily modelled by attaching the link element to the end of the diagonal brace. For this case, the parameters should be defined for the axial translational degree of freedom (U1). The active direction should be defined as "both" given the brace works in tension/compression.

*For a simplified modelling method, a single link element may be used to model the whole RSFJ brace. In this case, the input stiffness values (K) need to be modified to account for the elastic stiffness of the whole brace. In this case, the RSFJ and the brace body are in series so the initial stiffness of the link is $1/K_{total} = 1/K_{RSFJ} + 1/K_{body}$.







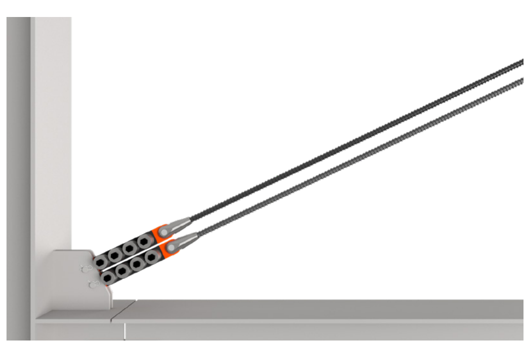
Translational link element representing the RSFJ

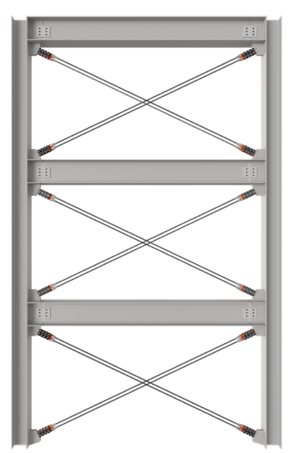
TENSION-ONLY DIAGONAL BRACES

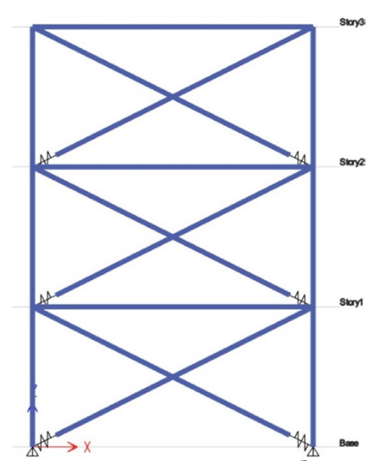
RSFJ-TBRACE

The RSFJ-Tbrace can be easily modelled by attaching the link element to the end of the diagonal braces. For this case, similar to the RSFJ brace, the parameters should be defined for the axial translational degree of freedom (U1). The active direction should be defined as "Tension" given the brace only works in tension.

*For a simplified modelling method, similar to the RSFJ brace application, a single link element may be used to model the whole RSFJ Tbrace. In this case, the input stiffness values (K) need to be modified to account for the elastic stiffness of the whole bar. In this case, the RSFJ and the brace body are in series so the initial stiffness of the link is $1/K_{total} = 1/K_{RSFJ} + 1/K_{body}$.



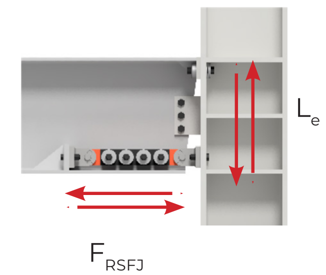




Translational link element representing the RSFJ

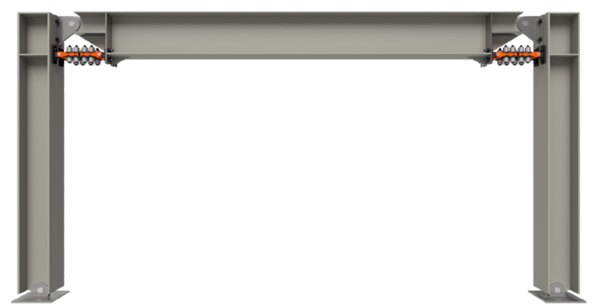
MOMENT RESISTING FRAMES

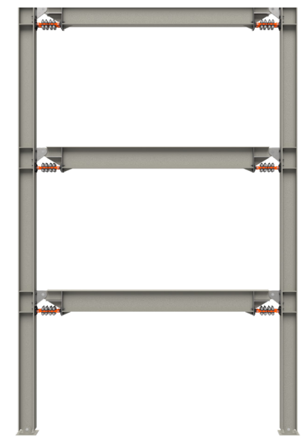
RSFJ-MRF

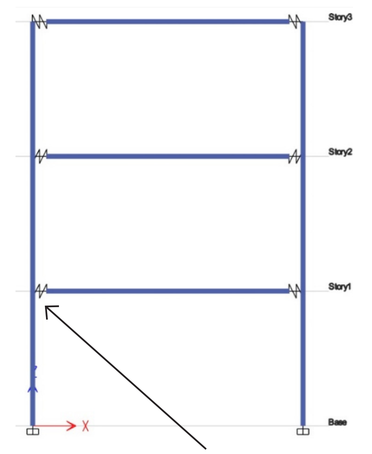


The recommended approach to model the MRFs with RSFJs is to model the moment-rotation behaviour of the beam-column connection using a single link element.

The link element can be attached to the beam and column in their contact interface. The moment-rotation behaviour of the MRF connection can be specified respecting the force in the RSFJ (F_{RSFJ}) and the lever arm (L_e) which is the vertical distance between the centre of the RSFJ and the rotating pivot close to the top beam flange. For this case, the parameters should be defined for the related rotational degree of freedom (for example, R3 when in the global "xz" plane). The other five degrees of freedom should be "fixed". The active direction should be defined as "Both" given the link element works in both directions.







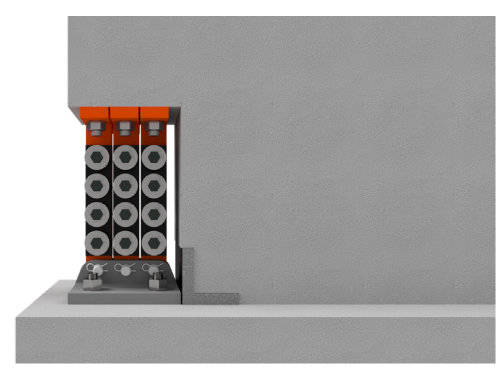
Rotational link element representing the beam-column behavior of the MRF connection with the RSFJ

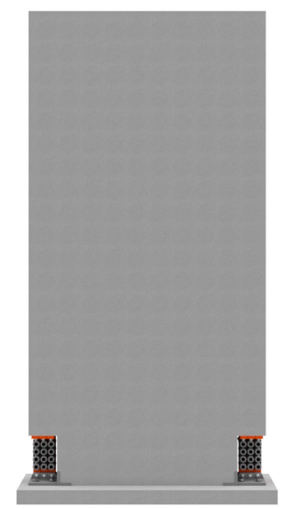
SHEARWALLS & COLUMNS

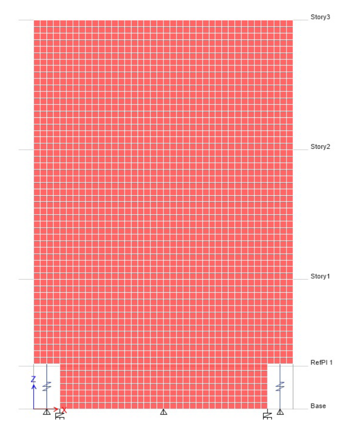
RSFJ-SHEARWALL

The RSFJ hold-downs for shear walls can be easily modelled by attaching the link elements to the pre-defined locations close to the edge of the walls.

For this case, the parameters should be defined for the axial translational degree of freedom (U1). The active direction should be defined as "Both" given the link element works in both directions although the displacement demand in compression is much lower.







Translational link element representing the RSFJ hold-down

START

Assume an equivalent ductility factor of μ = 2 ~ 3 for start

1 Determine the Ultimate Limit State (ULS) seismic forces applied to the lateral load resisting system ($F_{ult,sys}$) using the Equivalent Static Method (ESM).
For the initial estimate, use clause C4.1.2.2 (NZS 1170.5) to calculate the period of the structure (T_1).

2 Model the structure in ETABS/SAP2000.

The lateral load resisting members can be modelled using linear elastic members (there is no need to model the flag-shaped hysteresis of the RSFJs at this stage).

3 In the structural model, distribute the ULS seismic loads (obtained from ESM) in the structure to find out the forces in the members (and the corresponding RSFJs attached to those members ($F_{ult,RSFJ}$)). and determine the member sizes.

Is the period of the structure (T_1) from the modal analysis different from what is assumed in step 1?

YES → 3b. Adjust the period of the structure

NO

4 Modify the structural model from step 3 by modelling the RSFJs in the lateral load resisting members (using the "Damper – Friction Spring" link elements) based on the required capacity ($F_{ult,RSFJ}$) determined in the previous step.

Other hysteretic parameters of the RSFJ ($F_{slip,RSFJ}$, $Fr_{estoring,RSFJ}$, $F_{residual,RSFJ}$, $K_{initial,RSFJ}$ and $\Delta_{ult,RSFJ}$) can be defined for each RSFJ according to the Tectonus product tables.

5 Run the non-linear static push-over analysis to obtain the force (given the second stiffness of the RSFJs may change the load distribution in the structure) and the displacement (the criteria for ULS Is usually 2% to 2.5% of lateral drift when the base shear Is equal to the base shear obtained from the ESM in step 1) in the RSFJs.

In most cases, lateral drifts are limited within the 1% to 1.5 % range to minimise damage to non-structural elements.

Are the force and displacement demands in the structure satisfied?

NO → 5b. Adjust the hysteretic parameters of the RSFJs

YES

6 (Optional) Run non-linear time-history simulations to obtain the base shear and consequently the equivalent ductility factor (μ)

Is the assumed ductility factor accurate?

NO → 6b. Using the μ obtained at step 6, go to step 1.

YES

FINISH

STRUCTURE Three-story steel frame with RSFJ braces

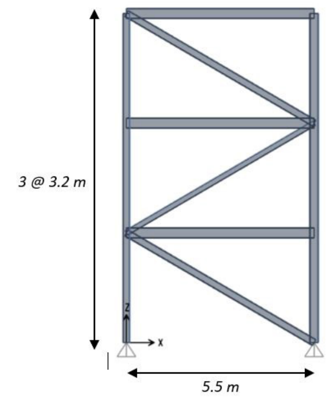
BUILDING TYPE Office or similar

SEISMIC WEIGHTS 335 kN is assumed for all stories

LOCATION Wellington, New Zealand with soil type D (deep soil).

The target Ultimate Limit State (ULS) lateral drift is 2.5% and the target Serviceability Limit State (SLS) drift limit 0.33%. The columns are continuous and beams and diagonal braces are pinned. Note that in real cases the target drift is in the range of 1.0% to 1.5% to protect the secondary and non - structural elements. The right procedure in the provided step-by-step design flowchart is used where the assumed structural ductility factor (μ) is verified by non-linear dynamic time-history simulations:

Assume an equivalent ductility factor of μ = 3.



1. **Determine the Ultimate Limit State (ULS) seismic forces applied to the lateral load resisting system ($F_{ult,sys}$) using the Equivalent Static Method (ESM). For the initial estimate, use clause C4.1.2.2 (NZS 1170.5) to calculate the period of the structure (T_1).**
Following this step, the base-shear of the structure is determined as 427 kN. Accordingly, the seismic story shears were determined as 230 kN, 131 kN and 65 kN for the roof, second story and the first story, respectively. The period of the structure (T_1) is determined as 0.34 seconds using NZS 1170.5 C4.1.2.2.

2. **Model the structure in ETABS/SAP2000. The lateral load resisting members can be modelled using linear elastic members (there is no need to model the flag-shaped hysteresis of the RSFJs at this stage):**
The structure is modelled in SAP2000 version 19.0 and the braces were modelled as linear elastic members with standard UC sections.

3. **In the structural model, distribute the ULS seismic loads (obtained from ESM) in the structure to find out the forces in the members (and the corresponding RSFJs attached to those members ($F_{ult,RSFJ}$)):**
The ULS seismic loads were applied to the structure. The force demand in the braces and the required UC sections were:

$F_{ult,RSFJ1}$ = 503 kN (200 UC 52.2)	$F_{ult,RSFJ2}$ = 402 kN (200 UC 46.2)	$F_{ult,RSFJ3}$ = 268 kN (200 UC 46.2)

The elastic lateral deformation of the structure is determined as 0.15% based on the numerical model.

CHECK: *Is the period of the structure (T_1) from the modal analysis different than what is assumed in Step 1?*

The period of the structure (T_1) from the model analysis is 0.33 seconds which is consistent with what is assumed in Step 1.

4. **Modify the structural model from step 3 by modelling the RSFJs in the lateral load resisting members (using the "Damper – Friction Spring" link elements) based on the required capacity ($F_{ult,RSFJ}$) determined in the previous step. Other hysteretic parameters of the RSFJ ($F_{slip,RSFJ}$, $F_{restoring,RSFJ}$, $F_{residual,RSFJ}$, $K_{initial,RSFJ}$ and $\Delta_{ult,RSFJ}$) can be defined for each RSFJ according to the Tectonus product tables.**

Based on the target ULS drift (2.5%) and the specified elastic drift determined in the previous step (0.15%), the displacement demand of the RSFJ braces was estimated as 85 mm. Based on the force demands specified in the previous step, the following RSFJs were adopted for the braces:

Roof: Adopt 2*RSFJ-TH2-250 (80 mm)	2nd level: Adopt RSFJ-TH4-400 (80 mm)	1st level: Adopt RSFJ-TH2-250 (80 mm)

The target Ultimate Limit State (ULS) lateral drift is 2.5% and the target Serviceability Limit State (SLS) drift limit 0.33%. The columns are continuous and beams and diagonal braces are pinned. Not that in real cases the target drift is in the range of 1.0% to 1.5% to protect the secondary and non - structural elements.

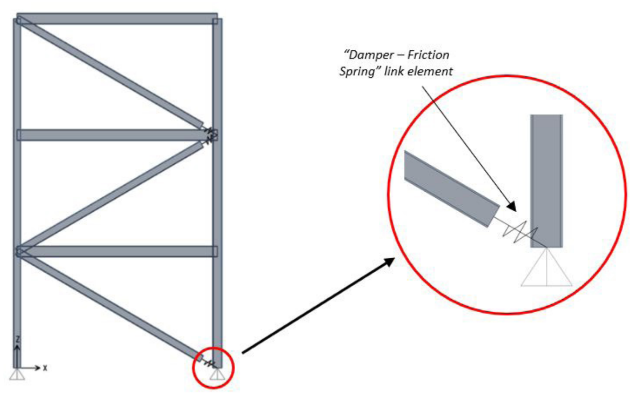
The right procedure in the provided step-by-step design flowchart is used where the assumed structural ductility factor (μ) is verified by non-linear dynamic time-history simulations:

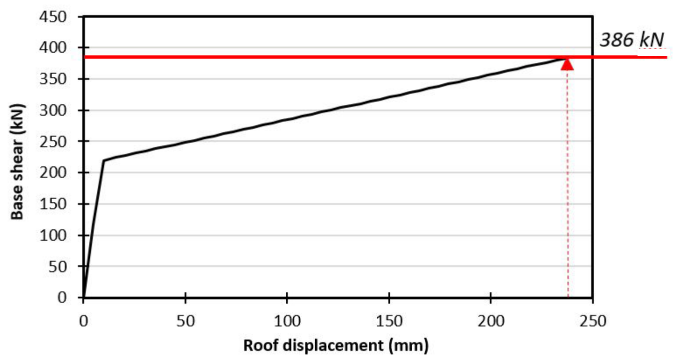
The RSFJ braces were modelled using "Damper-Friction Spring" link elements and the parameters were determined based on the described method in this hand-out. The table below summarises the numerical characteristics of the RSFJ braces. Please note that for this design example, the initial stiffness of the RSFJs ($K_{initial}$) is considered equal to the elastic stiffness of the adopted UC section. This can be different for different cases and should be determined based on the target SLS drift and the elastic stiffness of the RSFJ, brace body, pins, brackets and other attachments.

Level	Adopted RSFJ product	Initial (Non-slipping) Stiffness (kN/mm)	Slipping Stiffness (Loading) (kN/mm)	Slipping Stiffness (Unloading) (kN/mm)	Precompression displacement (mm)	Stop Displacement (mm)
Roof	RSFJ-TH2-250 (80 mm)	209	2.94	1.34	-70	70
2nd level	RSFJ-TH4-400 (80 mm)	185	2.35	0.93	-70	70
1st level	2*RSFJ-TH2-250 (80 mm)	185	1.47	0.67	-70	70

5. **Run the non-linear static push-over analysis to obtain the force (given the second stiffness of the RSFJs may change the load distribution in the structure) and the displacement (the criteria for ULS Is usually 2% to 2.5% of lateral drift when the base shear Is equal to the base shear obtained from the ESM in step 1) in the RSFJs.**

The result of the non-linear static push-over analysis is shown below. The structure is pushed to 2.5% of lateral drift corresponding to 240 mm of deflection at the roof. It can be seen that the maximum force in the system is 386 kN which is less than the demand specified by the ESM method (427 kN). This most likely means that the RSFJs are not fully expanded and did not reach their maximum capacity at the given ULS drift.





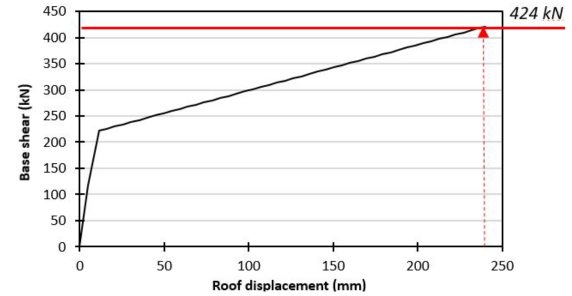
CHECK: *Are the force and displacement demands in the structure satisfied?*

The displacement capacity of the RSFJs need to be adjusted at this stage to achieve the force demand (427 kN).

Adjust the hysteretic parameters of the RSFJs. After two iterations with Δ_{ult} at 80 mm and 70mm, the force demand is reached at the given drift. The maximum displacement of the RSFJs (Δ_{ult}) is changed to 70 mm.
The adjusted specifications of the RSFJs are:

Level	Adopted RSFJ product	Initial (Non-slipping) Stiffness (kN/mm)	Slipping Stiffness (Loading) (kN/mm)	Slipping Stiffness (Unloading) (kN/mm)	Precompression displacement (mm)	Stop Displacement (mm)
Roof	RSFJ-TH2-250 (80 mm)	209	3.57	1.63	-70	70
2nd level	RSFJ-TH4-400 (80 mm)	185	2.86	1.13	-70	70
1st level	2*RSFJ-TH2-250 (80 mm)	185	1.78	0.81	-70	70

Repeating Step 4 and Step 5, the result of the non-linear static pushover analysis with the adjusted RSFJs is:



It can be seen that the new base-shear is 424 kN which matches the ESM base-shear demand. The roof drift at the slip threshold is approximately 0.15% which satisfies the SLS drift limit

6. **Run non-linear time-history simulations to obtain the base shear and consequently the equivalent ductility factor μ.**
Non-linear dynamic time-history simulations are carried out to investigate the behaviour of the structure. The seismic events were considered for a 500 years return period and soil type D (deep soil) located in Wellington, New Zealand. The table to the right shows the considered events for the simulations:

CHECK: *Is the assumed ductility factor accurate?*

From the results of the time-history simulations (the average base-shear), the equivalent ductility factor is μ = 3.47. Given that this ductility factor is relatively higher than the first assumption in Step 1 (μ = 3), the procedure needs to be repeated from the start with the new ductility factor of μ = 3.47. Also, the average displacement demand of the structure is 2.19% which is less than the ULS limit (2.5%).
Following the procedure with the new ductility factor, the base-shear from the ESM is reduced to 385 kN (Step 1).
Following Steps 2 and 3, the force demand and the adopted UC sections for the RSFJ braces are:

Event	Date
El Centro, Imperial Valley, USA	May 1940
Northridge, USA	January 1994
Landers, USA	June 1992
Christchurch, New Zealand	February 2011
Kobe, Japan	January 1995
Chi Chi, Taiwan	September 1999
Chihuahua, Mexico	November 1928
Loma Prieta, USA	October 1989
San Fernando, USA	February 1971
Duzce, Turkey	November 1999
Hokkaido, Japan	September 2003
Yarimka, Turkey	August 1999
Caleta de Campos, Mexico	September 1985

$F_{ult,RSFJ1}$= 450 kN (200 UC 52.2)
$F_{ult,RSFJ2}$ = 374 kN (200 UC 46.2)
$F_{ult,RSFJ3}$ = 245 kN (200 UC 46.2)

Accordingly, the numerical specifications of the new RSFJs are:

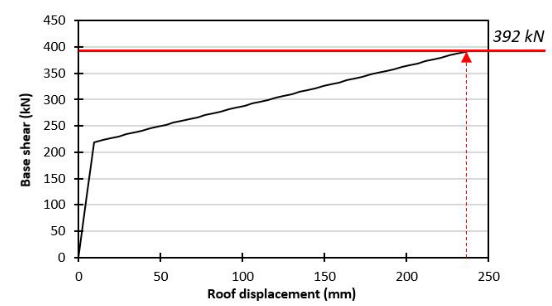
Level	Adopted RSFJ product	Initial (Non-slipping) Stiffness (kN/mm)	Slipping Stiffness (Loading) (kN/mm)	Slipping Stiffness (Unloading) (kN/mm)	Precompression displacement (mm)	Stop Displacement (mm)
Roof	RSFJ-TH2-250 (80 mm)	209	3.12	1.42	-70	70
2nd level	RSFJ-TH4-400 (80 mm)	185	2.50	0.99	-70	70
1st level	2*RSFJ-TH2-250 (80 mm)	185	1.56	0.71	-70	70

Following Step 5, the result of the new non-linear static push-over analysis is shown below.

As can be seen, the achieved base-shear (392 kN) matches well with the ESM base-shear demand (385 kN).

From the results of the time-history simulations with the new RSFJ configurations, the new average ductility factor is μ = 3.41 which is very close (2% difference) to the assumed ductility factor at Step 1.

This means that an equivalent ductility factor of μ = 3.4 can confidently be adopted for the given structure.



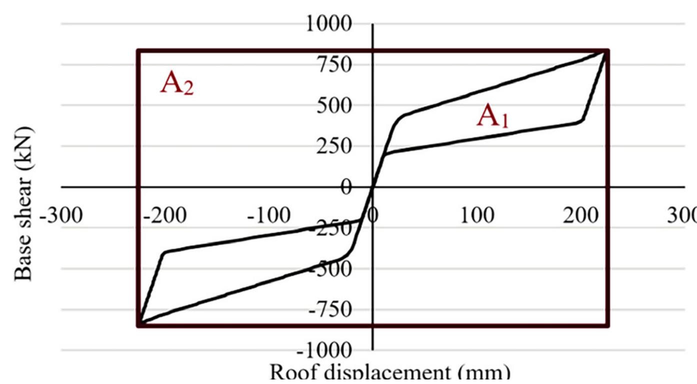
DISPLACEMENT-BASED DESIGN APPROACH

DESIGN IMPLICATIONS

DAMPING RATIO

Generally, a displacement-based approach is recommended to design the systems with RSFJ technology. The main reason being that in this method, the damping ratio of the system is directly incorporated into the calculation to scale the demand spectra.

Based on the product chosen and from the results of the cyclic pushover analysis on the structure, the damping ratio of the system can be determined using the equation below following the area-based approach:



$$\xi_{hyst} = \frac{2\,A_1}{\pi\,A_2}$$

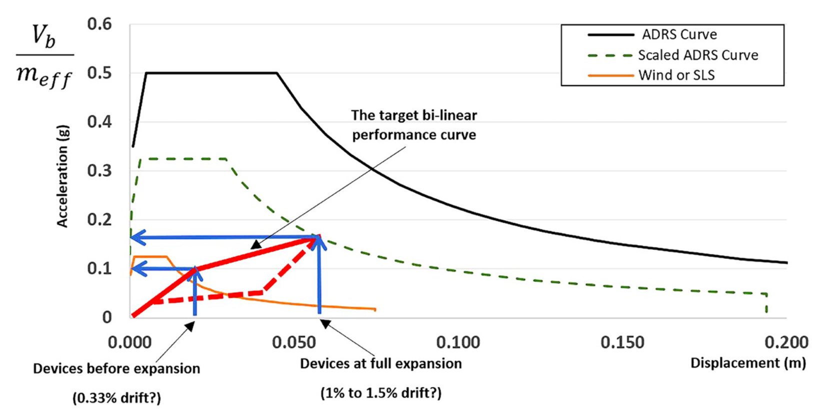
ACCELERATION-DISPLACEMENT RESPONSE SPECTRA (ADRS) CURVES

At the concept and detailed design stages, an efficient way to design the systems with RSFJs is the use of the Acceleration-Displacement Response Spectra (ADRS) curve to specify the F_{slip} and F_{ult} of the system based on the scaled and unscaled demand curves. In this approach, the SLS1 or SLS2 (depending on the importance level of the structure) would determine the F_{slip} of the system or in other words, the threshold in which the first RSFJ in the system starts to open. On the other hand, the ULS demand curve which is scaled based on the damping ratio determines the F_{ult} of the system before the secondary fuse starts to activate.

Firstly, the designer decides about the drift limit of the structure before the devices start to open. This is usually in the range of 0.33% depending on the details used for the non-structural elements. The intersection between this drift limit and the SLS spectra (or wind in rare cases) determines the base shear in which the devices start to open. All the remaining parts of the RSFJ remain elastic up to $1.5{*}F_{ult}$.

Secondly, the designer decides on the lateral drift that the structure is limited to at the design level earthquake. This is usually in the range of 1.5% to protect the secondary and non-structural components.

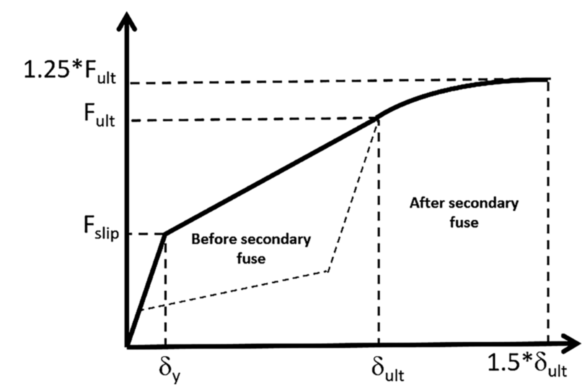
The intersection between this drift limit and the scaled ULS spectra (based on the amount of damping provided) determines the base shear in which the devices are at full expansion. The figure shows the design philosophy described.



THE COLLAPSE-PREVENTION SECONDARY FUSE

The RSFJ is designed in a way that all components remain elastic up to the design load (F_{ult} of the device). However, with the aim of collapse prevention in cases that the applied loads are higher than the design earthquake loads, a collapse-prevention secondary fuse in the body of the RSFJ is considered. When the load on the RSFJ increases beyond its maximum capacity (F_{ult}), the clamping bolts (or rods) start to yield. The inelastic elongation of the bolts provides additional travel distance for the joint allowing it to maintain a ductile behaviour (without the device locking at any stage) up to 1.5 times of the design displacement.

The devices are designed in a way that the maximum load in the joint after the full activation of the secondary fuse is 1.25 times higher than the design F_{ult}. In other words, the over-strength factor applicable for the RSFJ is 1.25 and the other parts of the structure should be designed with a minimum over-strength factor of 1.25 to maintain the hierarchy of strengths following the capacity design principles. Accordingly, an over-strength factor of 1.5 is usually considered for the attachments and the main structural members to take the material variability and dynamic effects into account.



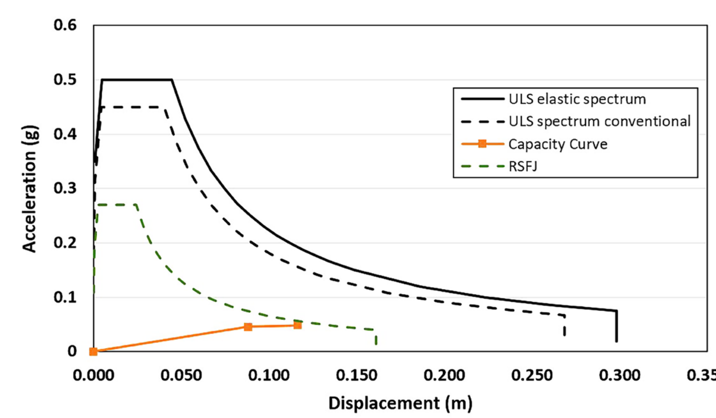
SIMPLE LATERAL MECHANISM ANALYSIS (SLAMA)

The New Zealand seismic assessment of existing structures guidelines recommend in all cases that a simplified nonlinear pushover analysis be conducted using a Simple Lateral Mechanism Analysis (SLaMA).

The SLaMA approach offers a simplified means of assessing the probable inelastic deformation mechanisms and lateral strength of a structure by running a pushover analysis on the identified lateral mechanisms. In many cases, the capacity curve cannot satisfy the demand curve and this would decrease the seismic score of the system (%NBS).

In order to solve this problem and increase the seismic score of the structure, either the displacement capacity of the system needs to be increased or the demand spectrum should be further scaled down. Increasing the displacement capacity is not always possible given the limited deformation capacity of the gravity resisting members.

For seismic retrofitting cases, RSFJs can be added to the existing lateral load resisting system. The demand spectra can then be further scaled based on the damping provided by the RSFJs so the capacity curve can intersect with the demand.



DISPLACEMENT-BASED DESIGN APPROACH

EXAMPLE

STRUCTURE Three-story rocking reinforced concrete wall with RSFJ hold-downs

BUILDING TYPE Office with an importance level of 2.
The return period considered for the design level earthquake is 1/500 years.

SEISMIC WEIGHTS 360 kN and 180 kN were assumed for floors 1 to 2 and the roof, respectively.

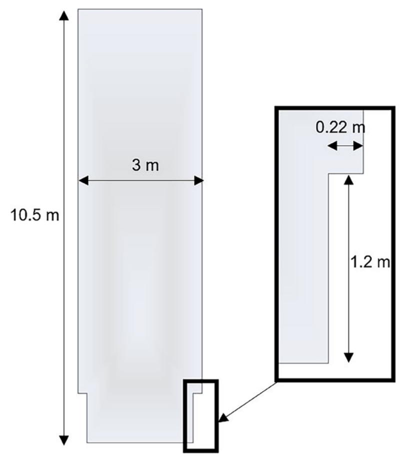
LOCATION Wellington New Zealand with a hazard factor of Z=0.4 and soil type D (deep or soft soil).

In order to protect the non- and secondary structural elements from damage, it was decided that the inter-story drifts are kept under 0.3% for the Serviceability Limit State (SLS) and 1.5% for the Ultimate Limit State (ULS).

The figure shows the general arrangement of the rocking wall.

NOTE : The layout of the RSFJs can be discussed with Tectonus Ltd. at the early stages of design. The structure is considered as regular and symmetric thus mostly dominated by the first mode of vibration.

The following steps were taken to design the RSFJ hold-downs using a displacement-based approach and ADRS curves.



1. **Determine the characteristics of the equivalent Single Degree of Freedom (SDOF) structure**

As per the principles of displacement-based design, the structure is represented by an equivalent SDOF system with the following characteristics:

Characteristics of the equivalent SDOF structure Height in meters Mass in tonnes

Level	Height (h_i)	Mass (m_i)	Δ_i	$m_i\Delta_i$	$m_i\Delta_i^2$	$m_i\Delta_i h_i$
Roof	10.5	18.3	0.16	2.89	0.46	30.34
2nd level	7	36.7	0.11	3.85	0.4	29.97
1st level	3.5	36.7	0.05	1.93	0.1	6.74
Sum				8.67	0.96	64.06

$$\Delta_d = \frac{\sum_{i=1}^{n} m_i \Delta_i^2}{\sum_{i=1}^{n} m_i \Delta_i} = 0.11\ m$$ (SDOF peak design displacement displacement)

$$m_e = \frac{\sum_{i=1}^{n} m_i \Delta_i}{\Delta_d} = 78.2\ tonnes$$ (SDOF effective mass)

$$H_e = \frac{\sum_{i=1}^{n} m_i \Delta_i h_i}{\sum_{i=1}^{n} m_i \Delta_i} = 7.4\ m$$ (SDOF effective height)

2. **Determine the hysteretic and elastic damping ratio of the system and calculate the scale factor**

For this example, an elastic damping ratio of 3% and a hysteretic damping ratio of 14% (provided by the RSFJs) are assumed. Note that RSFJ products can provide a hysteric damping ratio between 10% to 20% depending on the design. The assumed value for the hysteretic damping ratio in this step is verified at the last step of the procedure. The spectral scale factor used to scale the demand spectra is calculated using the following formula (from Eurocode 8, 1998):

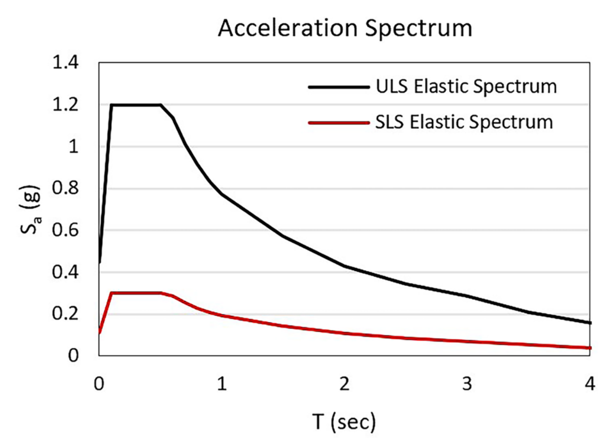
$$\eta = \sqrt{\frac{7}{2+\xi_{el}+\xi_{hyst}}} = \sqrt{\frac{7}{2+3+14}} = 0.61$$ (the scale factor)

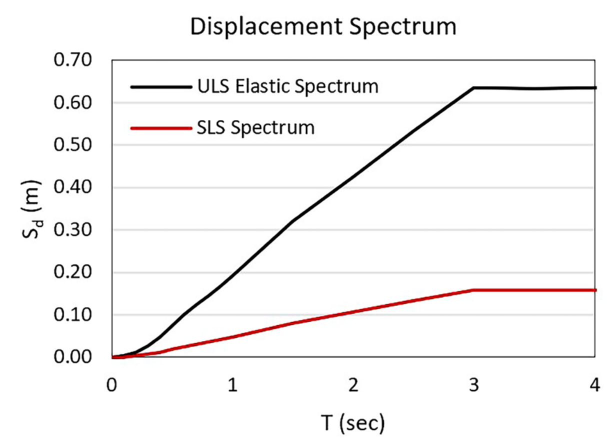
3. **Plot the Acceleration-Displacement Response-Spectra (ADRS) curves and scale the demand curve based on the calculated scale factor**

The acceleration and displacement spectrums for the given location, soil type and return period factor can be derived from the New Zealand Standard NZS 1170.5 (or any equivalent international standard). Note that the limit state at which the structure remains linear elastic is decided to be the SLS earthquake. For the international standards that do not have serviceability earthquake requirements, the wind design spectra can be used instead.

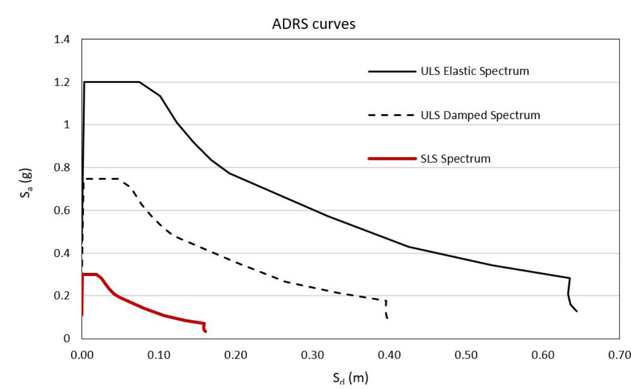
The red curve shows the SLS spectrums and the black curve shows the ULS design spectrums. The displacement spectrum is plotted from the acceleration spectrum using this equation:

$$S_d = \frac{T^2}{4\pi^2} S_a$$





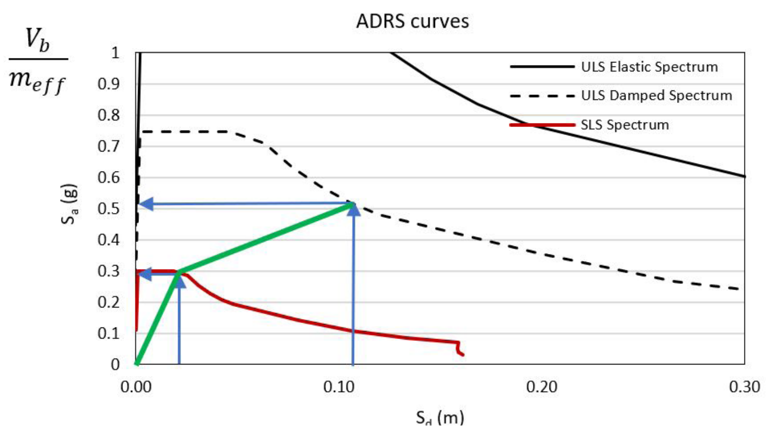
The ADRS curves can be plotted using the acceleration and displacement spectrums in which the horizontal axis is the displacement spectrum (S_d) and the vertical axis is the acceleration spectrum (S_a). The black dashed curve shows the design spectra scaled using the scale factor specified in Step 2.



4. Determine the $F_{slip,sys}$ and $F_{ult,sys}$ based on the considered drift limit states

The horizontal axis of the ADRS curves shows the displacements. As mentioned earlier, the drift limit corresponding to the SLS is 0.3%. The intersection of this value on the horizontal axis (corresponding to 0.022 m deflection in the equivalent SDOF system) and the SLS curve (the red curve in the figure below) will give the $F_{slip,sys}$ or the force in which the first RSFJ in the system start to deform. Note that the vertical axis of the ADRS curve represents (V_b/m_{eff}). The red curve could be SLS1, SLS2 or the design wind spectrum for cases where the service earthquake consideration is not a requirement.

The drift limit for the design level earthquake was considered as 1.5%. Similarly, the intersection between this value on the horizontal axis (correspond to 0.11 m deflection in the equivalent SDOF system) and the scaled ULS curve (the black dashed curve) will give $F_{ult,sys}$ or the base shear in which the RSFJs are at full expansion. For this case, $F_{slip,sys}$=230 kN (0.3*9.81*78.2) and $F_{ult,sys}$=400 kN (0.51*9.81*78.2 = 391 kN rounded up to 400 kN) were specified. The green curve below shows the target backbone performance curve of the structure.



5. Calculate the force and displacement ($F_{slip,RSFJ}$, $F_{ult,RSFJ}$ and $\Delta_{ult,RSFJ}$) demands of the RSFJs

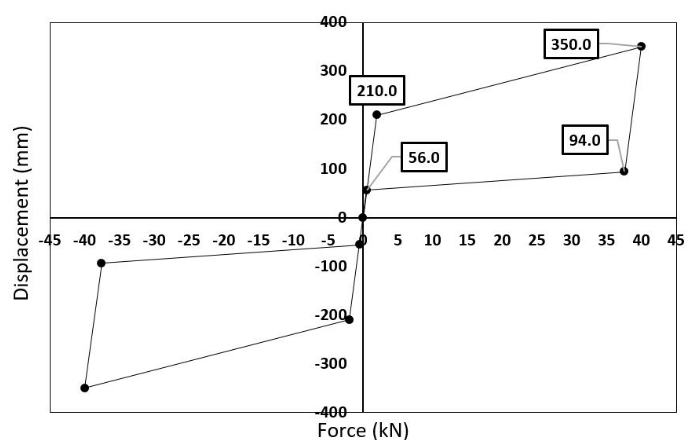
In this step, the base shears found from the last step are distributed in the structure and the force and displacement demands in the RSFJs are calculated. For this example, taking the moments around the rocking toe of the structure, the following characteristics are found for the RSFJ hold-downs. Note that for more complicated structures, a numerical model may need to be developed to distribute the lateral seismic loads and calculated force and displacement demands in the structure.

$F_{slip,RSFJ}$ = 630 kN $F_{ult,RSFJ}$ = 1065 kN $\Delta_{ult,RSFJ}$ = 40 mm

From the Tectonus product catalogue, 3* RSFJ-SH6-350 is selected for this application.

6. Determine the full flag-shape response of the RSFJs in the system

In this step, the full flag-shape response of all devices that are used should be calculated. At this stage, based on the product code selected from the Tectonus product catalogue (or in the case that a customized product is required), the designer may need to contact the Tectonus Engineering team and discuss the products required. Tectonus Ltd. will send the flag-shape response of the devices. The flag-shape response of the selected device for this example is:

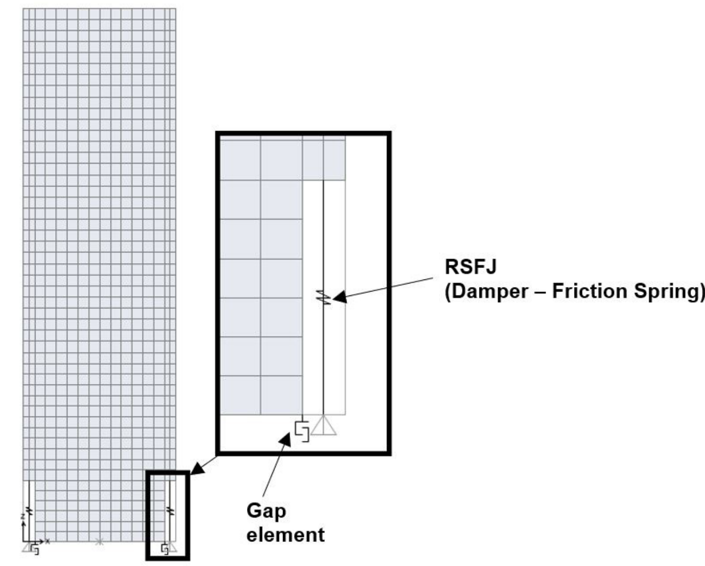


7. Develop a numerical model for the structure including the RSFJs

A numerical model in SAP2000 is developed for the shear wall with RSFJs modelled as "Damper-Friction Spring" link elements. The table below shows the numerical input used for the link element. A gap element is also used at the rocking toe to represent the foundation level.

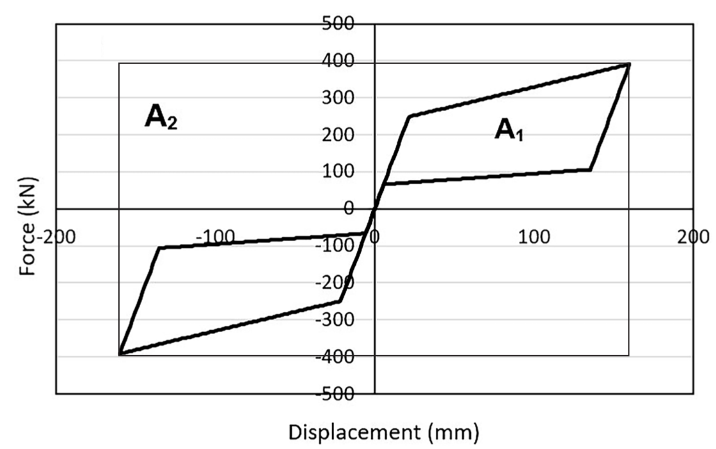
Note that one link element is used to model the three RSFJs working in parallel. The figure below shows the numerical model developed in SAP2000.

Parameter	Value
Initial stiffness (kN/mm)	320
Loading Stiffness (kN/mm)	10.96
Unloading stiffness (kN/mm)	3.02
Pre-compression displacement (mm)	-57
Stop displacement (mm)	40



8. Run cyclic pushover analysis on the structure to verify the bi-linear performance of the system and verify the hysteretic damping ratio for the system

The figure below shows the results of the non-linear static cyclic pushover analysis carried out on the model:



Following an area-based approach, the hysteretic damping ratio of the structure is calculated using the equation below.

$$\xi_{hyst} = \frac{2A_1}{\pi A_2} = 14.4\%$$

As can be seen, this number is consistent with the initial assumption at step 2 of the procedure.
Note that if this value is different from the assumption in step 2, iterations may be required to optimize the design.

CONTACT US

ENGINEERING & DESIGN SUPPORT
PROJECT ESTIMATES

Tectonus provides tailored design support for RSFJ structural modelling and analysis. Contact the expert team for more information.

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