# Canopy bridge in the Amazon rainforest

## Extreme events, design considerations and load combinations



*Canopy bridge at Tiputini Biodiversity Station, Ecuador (Haskell, 2014)*


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# Preface

This bachelor thesis is written for course CTB3000-16 bachelor Eindwerk. This course is a course at faculty of civil engineering of the Technical University of Delft. The research is done in collaboration with Universidad San Francisco de Quito (USFQ).

I chose this subject because I thought it was the most interesting on the list. A bridge suspended between a tower and a tree 28 meters high, just looked spectacular. It was quite a new experience for me, as it was the first time doing a project where there is not really a clear outline of what to do. I struggled a lot with that, and I have the bad habit of rushing things that need to go slow, but now I really experienced why they need to go slow. If you do it right the first it saves a lot of work at the end. That is one of the main lessons I learned. I also learned that it is really necessary to really get an idea of what you need to do before doing something. Another challenge was working with SCIA Engineer for the first time. But I do think I got better at that now.

I would really like to thank Dr. Eva Lantsoght from USFQ, she was my main supervisor and guided my through this project every week from Ecuador. I would also like to thank Dr. Pierre Hoogenboom from TU Delft, he was my second supervisor and gave very helpful feedback as well. Lastly I would like to thank Estefanía Cervantes  and Paul Paredes, both from USFQ, who provided me with very helpful information on the Tiputini bridge.

Delft, October 2025

Loek Schaffels

# Summary

The goal of this research was to determine critical design loads and the response of a canopy bridge under extreme events in the Amazon. So the question 'What are the effects of extreme wind and a seismic event on the usability of a canopy bridge in the Amazon rainforest?' can be answered.

The structural response is been determined for load combinations consisting of the load of two pedestrians, the wind load, the self-weight and a seismic event. Other extreme events are outside the scope of the project. See Table 1 for all relevant extreme events.

| **Extreme events** | **Impact on bridge** |
|---|---|
| Heat and cold waves | Deformations due to temperatures and material properties |
| Droughts | Material properties |
| Windthrows | Wind load |
| Floods | Hydrodynamic and hydrostatic loading, material properties |
| Widfires | Deformations and material properties |
| Seismic events | Vibrations, deformations, forces |
| El Niño | Have an impact on the other extreme events. |
| La Niña | |

*Table 1 extreme events and the loads they generate.*

For the case study the canopy bridge of Tiputini research station was studied. Only the effects of extreme wind events and seismic events were researched. The seismic event showed a deflection of 57,4 mm in the middle of bridge, when there would be 2 people on the bridge. The deflection is higher than the maximum of $L/500 = 43{,}2\ mm$. Therefore it does not meet the serviceability standard. The design and/or the model should be improved.

The most important load combination is the combination which showed the highest deflection, this was load combination 2a. This load combination consisted of the self-weight, 2 pedestrians and wind in horizontal and vertical directions. The deflection was 172,4 mm according to the model in SCIA engineer and 129 mm in vertical direction according to the cable theory. Both these deflection are higher than the maximum of 43,2 mm. Therefore the bridge would not meet the standards.

To conclude, extreme wind or a seismic event lead to a deflection of the bridge bigger then the comfort criterium of $L/500$ (AASHTO, 1997) with L being the span of the bridge. This is according the results of the rapport.

It is questionable how far the results are valid. The SCIA model has a pre-tension in the cable that is too high for the thickness of the cables. The calculations done by hand are  without any pre-tension and only for the vertical deflection.

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# Introduction

The Amazon rainforest is one of the many special places this Earth houses. The Amazon rainforest is the home of a lot of different species of animals, plants and insects. This attracts biologists and tourists. To allow them to do their research or to let them see the beauty of the canopy of the forest, canopy bridges can be build. Canopy bridges are bridges that are hung at the height of the canopy of the forest. The wet Amazonian climate can be a challenging environment for structures.

In this Bachelor thesis the main focus will be on finding out what the main design considerations are when designing for extreme events in the Amazon. After finding out the main design considerations, the canopy bridge of the Tiputini research station will be studied as a case study. Where loads of certain extreme events will be calculated. And the resulting deformations for the Service limit states will be calculated. This is done to answer the question: 'What impact do extreme events in the Amazon have on the usability of a canopy bridge?'

The objective of this study is to

- Find out what the main design considerations are when designing a canopy bridge in the amazon rain forest, in terms of governing loads and withstanding extreme events.
- Find the wind load on the canopy bridge according the Eurocode
- Find the seismic load on the canopy bridge.
- Find the deformations of load combinations on the bridge.
- Increase the robustness of the bridge by creating a second load path for when a tree fails. (as recommendation)

Boundary conditions:

- The bridge is canopy bridge at the Tiputini research station.
- The bridge needs to be for pedestrians (biologists, tourists, and maybe some wildlife).
- The bridge needs to be in the Amazon rainforest, so the bridge needs to be designed for the climate and extreme events of the Amazon rainforest.
- The Eurocode will be used to determine loads and load combinations.
- The load combinations will be made for service limit state.

The approach used for this thesis is as follows: first a literature study is done on extreme events in the Amazon rainforest. Then some of the loads that extreme events generate are computed, combined and put on the bridge. This is done in SCIA Engineer to see the amount of deformation that these loads generate. In the end a recommendation will be given to increase robustness in case one of the supporting trees fails.

# 1. Design considerations

In this chapter extreme events that occur in the Amazon are described. Followed up by a brief and general introduction on loads on a canopy bridge and the combination of these loads.

## 1.1 Extreme events in the Amazon

Extreme events are intense climatic phenomena that occur infrequently and can have adverse social and environmental consequences. All extreme events which occur in the Amazon rainforest are explained below.

### Heat and cold waves

During heatwaves more wildfires occurred and the intensity of these wildfires were higher than when wildfires happened not during a heatwave. There is a positive correlation between heat waves and wildfires. Also during drier years more and more intense heatwaves occurred, according Narcizo, L., et. al. (2022). The correlation between extreme heatwaves and extreme dryness has been researched by Costa, D. F., et. al. (2022). In this research they state that 'the most extreme heat waves in Amazonia happened under extreme dryness', as is the title of their research. The impact of droughts will be explained later on.

When a material gets warmer it starts to expand, it is good to know at what temperature a certain element of the construction starts to elongate. The opposite is true with colder temperatures, when the temperature gets lower most materials start to shorten. When designing a bridge it is good to know what extreme temperatures to expect, to prevent undesirable stresses from occurring in the material.

'A cold wave is a meteorological event generally characterized by a sharp drop of air temperature to extremely low values near the surface, a steep rise of pressure, and higher wind speed, associated with hazardous weather' (Marengo, J., 2023). Cold air intrusion can lead to heavy rainfall and floods. But according a study of Ceccherini, G., et al. (2016), no conclusion can be drawn on what the effects of a cold wave are on extreme events.

### Droughts

Droughts are often associated with heat waves, and they often co-occur. During droughts, the heat gets amplified because evaporative cooling is reduced, according to Costa, D. F., et al. (2022). The same research showed that years of extreme droughts are also the hottest years and showed that extreme droughts and warming were happening together. Looking at droughts and then temperatures, could give information about the maximum temperature. With the maximum temperature known, potential thermal stresses could be calculated and possibly prevented when designing in the amazon, or in a certain area. Unfortunately I could not find data on the website of the national meteorological agency of Ecuador, Instituto Nacional Meteorologia e Hidrologia (INAMHI), of the last couple of years.

### Windthrows

Windthrows or blowdowns are winds that blow so hard that trees break or become uprooted. These windthrows are caused by convective storms, these storms are deep moist convective cells. 'The development of these convective cells includes an updraft of moist air, which forms the cumulonimbus cloud, and a downdraft, which is associated with heavy precipitation, strong

surface winds, hail, and cloud-to-ground lightning' Feng, et al. (2023, p. 1). Horizontal Wind bursts during such an event typically range from 25 m/s to 50 m/s.

They stated specifically for the region where Ecuador is in: 'The northwestern and western Amazon, where large windthrow density is relatively high now (2.4 windthrows/10,000 km2 ), are estimated to experience a 33–50% increase in the number of windthrow events in the next 50–80 years.' Feng et al., (2023, p. 4). They only looked into large blowdowns ( > 25,000 km$^2$) and not really into small blowdowns, but small blowdowns are also very common in that area.

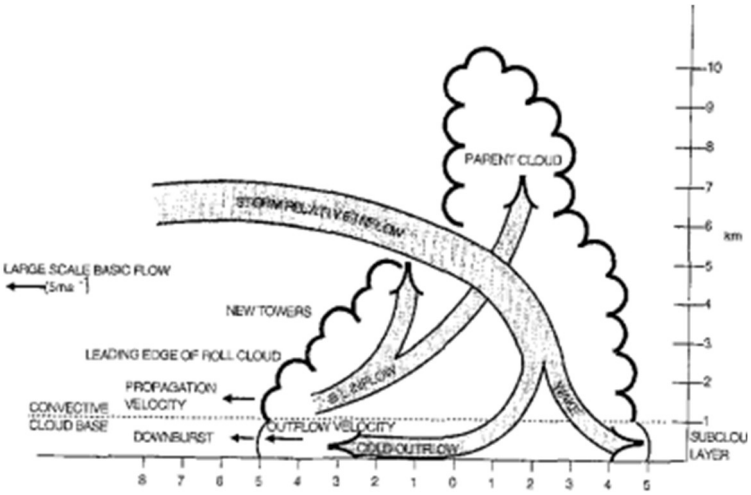



*Figure 1 Flow pattern of a convective cloud downdraft (Garstang et al. (1994, p.207)*

## Cyclones

The Amazon is not a region with a high tropical cyclone occurrence. This is mainly due to the fact that the Amazon is more land-in ward situated. Cyclones start and grow in intensity above the sea, when the cyclone comes to land the intensity of the cyclone starts to decrease. By the time it reaches the Amazon the intensity will be so low that it would not really have an impact. In the east of the Amazon it is protected by the Andes from possible cyclones.

## Floods

Floods in the amazon are caused by heavy rainfall over a period of time. Trees in the Amazon flood plain, have adapted to flooding conditions and respond to floods. Floods can cause rot, which makes trees less resistant to stress, but that depends on the type of tree.

When floods flow, water is pushing on the tree generating a horizontal load on tree, this is hydrostatic or hydrodynamic load. This will happen close to the ground and the roots. These forces will not be that high in most circumstances therefore they will not be included in this research.

## Wildfires

Wildfires occur more often during drier and warmer years, as stated under 'heat waves' and 'droughts'. A study on fires in the amazon in the 21st century showed that most fires happened close to deforested parts, like agricultural lands, of the Amazon. Temperatures typically ranged from 650 to 1250 Kelvin.

## Seismic events

Ecuador is located in a high seismic hazard region, especially the west of Ecuador. When going more to the east of Ecuador the chance of seismic event decreases. A bridge needs to be designed for possible seismic events, when it is situated in the before mentioned regions. The structure needs to be able to bear a certain force and/or be able to deform without breaking, so

it needs to have a certain strength and ductility. Based on these two terms the resilience can be computed.

The loading of an earthquake depends on the amplitude, duration and predominant period of the earthquakes ground motion. The PGA of the North-east of Ecuador is in between 0,10 g and 0,20 g for return period of 475 years. And the main source of seismic energy in Ecuador is the Nazca tectonic plate according to the

### El Niño and la Niña

El Niño increased temperatures and evapotranspiration averages and decreased monthly precipitation averages in the Amazon. La Niña showed the exact opposite according Moura M. M., *Et al.(2018)*. So El Niño causes more droughts and extremely high temperatures. And La Niña causes more wind extremes, with higher windspeeds and it cause more wet extreme events. Wet extreme events like floodings which can be caused by heavy rainfall. To conclude El Niño and La Niña both have an effect on certain extreme events and on the amount of occurrence of these extreme events.

## 1.2 Loads on a canopy bridge

This part is an introduction to the cases study in chapter 2. This serves as a general explanation of terms relevant to a canopy bridge, in terms of loading.

### Permanent loads

Permanent loads are the loads that are constant over time, these loads are also called the deadload. Examples of deadloads are the weight of the structure, the weight of elements of the structure on other elements of the structure and other elements that are at a fixed place on the structure.

### Variable loads

Variable loads are the loads on a structure that are not constant over time. These loads can appear in combination with other types of loads. They always appear in combination with the permanent loads. Extreme events cause variable loading on the structure if they happen often. Loads that happen less often are called accidental loads. Most loads caused by extreme events are accidental loads. Examples of variable and accidental loads are:

- Wind,
- live load (people, animals),
- snow load (irrelevant for the Amazon),
- thermal loads: fire load, weather temperature,
- seismic loads (earthquakes),
- hydrostatic loads,
- hydrodynamic loads (flowing water).

The permanent and variable loads are to be combined with each other to get find the highest deflection of the bridge.

# 2. Tiputini canopy Bridge

The Tiputini canopy Bridge is situated in the north-east of Ecuador, at the Tiputini biodiversity station. One of the supporting trees of the canopy has failed and the design has been updated to better standards. The design needs to be checked on serviceability limit state.

In this chapter, a brief selection of the extreme events will be summed up that are relevant to the Tiputini bridge. Followed up by the loads that are on the Tiputini Bridge. The wind load will be computed according Eurocode 1 (2019) and the impact of a seismic event will be computed with the national code of Ecuador NEC-SE-DS (2014) and Eurocode 8 (2025). The loads will be combined to compute the deformation of each load combination, in order to find the load combination with the biggest deformation. But first the properties of the bridge will be given.

Figure 2 shows the canopy bridge before the accident. Figure 3 shows the side profile of the bridge. The bridge is 28 m above ground level. It is 21,6 meters long and it is spun between two trees. The cables are 7x19 galvanized steel cables with a diameter or 18 mm. The deck of the bridge is made of wood-plastic composite planks that are 450 mm wide, 200 mm long and 40 mm thick. In Table 2 is an overview of the materials that are used.



*Figure 2 Tiputini canopy bridge (Pinterest, unknown date)*

| **Materials** | **WPC-planks** | **Steel cables** |
|---|---|---|
| Density $\rho$ $[kg/m^3]$ | $1016{,}14$ | $7850$ |
| E-modulus $E$ $[MPa]$ | $5{,}55 * 10^5$ | $1{,}00 * 10^5$ |
| Poisson | $0{,}3$ | $0{,}3$ |
| Size $[m]$ | Length $= 0{,}20$<br>Width $= 0{,}450$<br>Height $= 0{,}04$<br>54 pieces<br>Spacing of 0,4 m | Diameter $= 0{,}018$ |

*Table 2 Material properties*

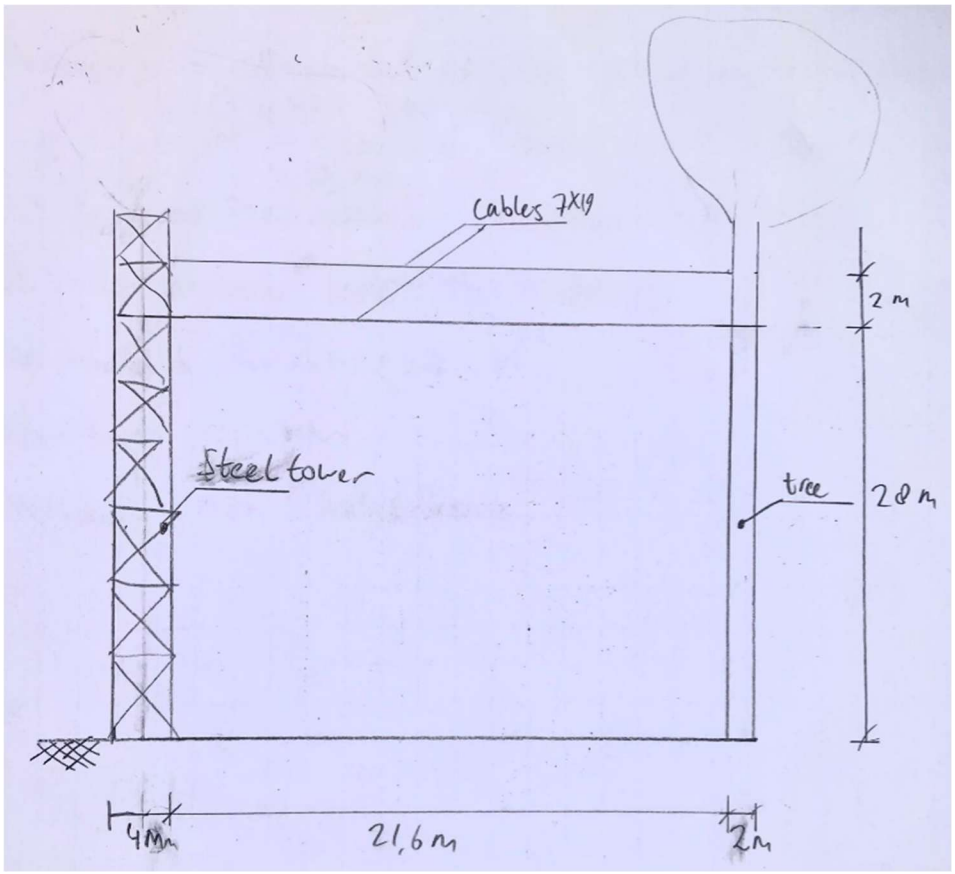



*Figure 3 Sideprofile of Tiputini bridge*

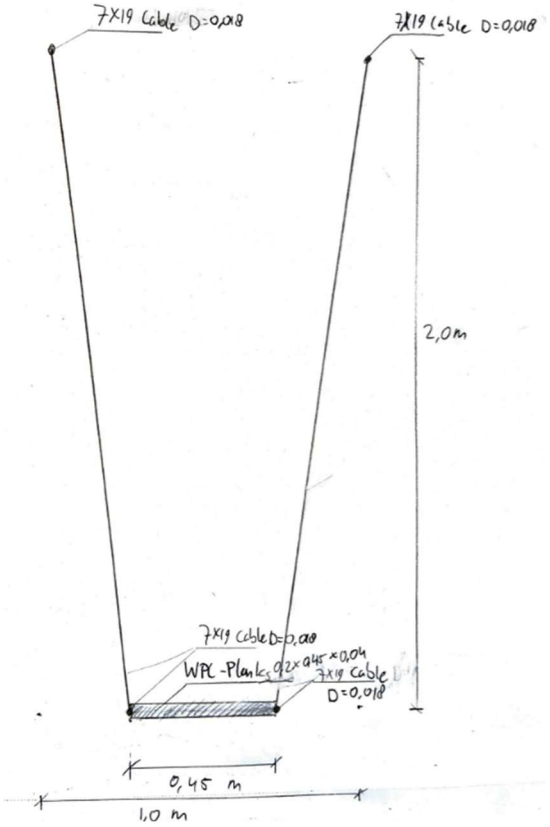



*Figure 4 side profile of bridge, at the height of the bridge*

Figure 4 shows the model of the bridge in SCIA. In between the horizontal cables, should be a plastic net. This plastic net is modelled as vertical cables with same properties as the horizontal cables.

Model in SCIA:

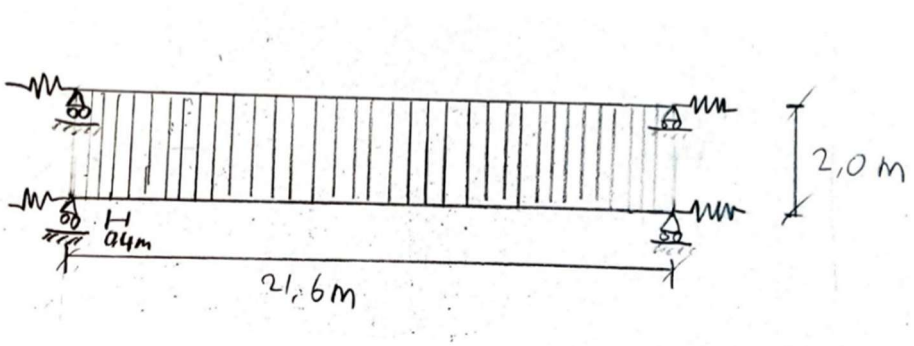



*Figure 5 model in SCIA*

## 2.1 Extreme events

The extreme events that are relevant to the Tiputini bridge are extreme wind events like windthrows. Seismic events are relevant to this bridge as it is located in the North-east of Ecuador. The north-east of Ecuador is not an area where seismic events happen more in comparison with the west of Ecuador.

Droughts do occur in this region, droughts have an effect on the supporting trees and on other material properties. It would require modeling reduction in material properties, which is outside the scope of this research.

Wildfires are a relevant extreme event as well. Especially when the bridge is located close to agricultural lands. In this case that is not the case, so wildfires will not be included in the research.

## 2.2 Loads on Tiputini bridge

First the wind load and the seismic load on the bridge are calculated according to the Eurocodes 1 (2019) and 8 (2025) . Then the permanent and variable loads are computed. The dead load are from Memoria Tecnica Puente Dosel (Parades, 2025).

### Self-weight

The self-weight of the bridge in SCIA is calculated. In the model is the netting changed for 54 vertical steel cables per side. This makes the self-weight of the cables:

$\frac{1}{4} * 0{,}018^2 * \pi * 7850 * (2 * 54 * 2 + 4 * 21{,}6) = 604{,}07\ kg$

The weight of all WPC-planks is:

$54 * 0{,}45 * 0{,}20 * 0{,}04 * 1016{,}4 = 197{,}59\ kg$

Total: $604{,}07 + 197{,}59 = 801{,}66\ kg$

### Live load

The live loads are the pedestrians on the bridge. The bridge is designed for two pedestrians. The load of a pedestrian is $1{,}5\ kN$. In the model a pedestrian will be modelled as one point load of $0{,}75\ kN$ on every lower cable at the same longitudinal direction.

### Wind load

The wind load is calculated according Eurocode 1. In appendix A the calculation can be found, the results of this calculation is.

$Q_{k,x} = 2{,}753\ kN/m^2$

$Q_{k,y} = 1{,}101\ kN/m^2$

$Q_{k,z} = \pm 2{,}477\ kN/m^2$

## Seismic action

The seismic action parameters are determined according the national norm of Ecuador: NEC-SE-DS- Peligro Sismico (2014). The calculation of these parameters can be found in appendix B – Seismic action. These parameters determine the properties of the seismic spectrum.

## Conclusion

All characteristic loads on the canopy bridge, that are used in this research are noted in Table 3.

| Type | symbol | Value | Input from Memoria Tecnica Puente Dosel |
|---|---|---|---|
| Pedestrians | $Q_{k,1}$ | $1{,}5\ kN$ per pedestrian. 2 pedestrians Total: $1{,}5 * 2 = 3\ kN$ | 2 line-loads of $93{,}375\ \frac{kg}{m}$ were modelled one on every lower cable, or 415 kg/m² |
| Wind load in x direction | $Q_{k,x}$ | $2{,}753\ kN/m^2$ | $170{,}88\ kg/m^2$ is $1{,}676\ kN/m^2$ |
| Wind load in y direction | $Q_{k,y}$ | $1{,}101\ kN/m^2$ | The same as in x-direction |
| Wind load in z direction | $Q_{k,z}$ | $\pm 2{,}477\ kN/m^2$ | Not used |
| Seismic action | $A_{Ed,SLS}$ | See 2.3 combinations of actions | Not used |
| Self-weight | $G_k$ | Automatically in SCIA | Lower cables = automatically in SAP2000. WPC-planks = $9{,}15\ \frac{kg}{m}$ line-load on every lower cable. |
| prestress | $P_k$ | $60\ kN$ per cable | |
| Maximum allowable deflection | | The same one is used: Live + dead load: $\frac{21600}{360} = 43{,}2\ mm$ | Live load: $\frac{L}{500} = 43{,}2\ mm$ Live + dead load: $\frac{L}{360} = 60\ mm$ According AASHTO (1997) |

*Table 3 Loads*

When comparing these results to the values used in the Memoria Tecnica Puente Dosel (Paredes, 2025), the following differences are important to mention:

- The pedestrians are modelled as a point load in the middle of the bridge in the SCIA model. In Memoria Tecnica Puenta Dosel, they are modelled as a line load.
- The wind load used for the SCIA model is 1,6 times higher. So bigger deflections are expected in the SCIA model, in cases where wind load is used.
- The self-weight of the bridge in SCIA is $801{,}66\ kg$.
- The maximum allowable deflection for the service limit state is taken the same as in Memoria Tecnica Puente Dosel: $43{,}2\ mm$.

## 2.3 Combinations of actions

The load combinations made are Service Limit State combinations. SCIA Engineer calculates the deformations of the service limit states, according the Eurocode. These results will be compared with the calculations of Memoria Tecnica Puente Dosel (Parades, 2025).

All values marked in yellow are the values used in the model. The colors of the color bar in the image are hard to see in the bridge. When zooming in on the bridge these colors are visible on each of the elements.

For all load combinations the pedestrians are taken as the most important load. The supports are flexible supports with a stiffness based on a supporting tree. The modelled stiffness of the supports is $2{,}905\ MN/m$, the calculation for this stiffness is found in Appendix C.

An overview of the values used for the SCIA model is provided in Table 3. Some of the properties default properties, some have been changed. Table 2 shows the material properties used. Table 4 shows the project settings that are turned on. The type of units used is metric format. Table 5 shows the settings used for the calculations

| **Project settings** | |
|---|---|
| The place in the functionality tab. | All settings that are turned on |
| General | Property modifiers, model modifiers, Parametric input, Climatic loads, Dynamics, Stability, Nonlinearity, Structural model, IFC properties, Prestressing, Bridge design |
| Dynamics | Modal & harmonical analysis, Seismic spectral analysis, Vibration analysis, Dynamic time-history analysis |
| Detailed > nonlinearity | Beam local nonlinearity, Support nonlinearity/basic soil spring, Geometrical nonlinearity, Cables, Membrame elements |
| Detailed > Steel | Steel connections, Scaffolding |
| Detailed > Bridge design | Load combinations |

*Table 4 Project settings*

| **Calculation settings** (FE analysis tab) | | |
|---|---|---|
| Mesh setup | Mesh setup | Average number of 1D mesh elements: 5<br>Average size of 1D mesh element on curved 1D members [m]: 0,200<br>Average size of 2D mesh elements [m]: 0,500<br>Connect members/nodes: on |
| | General mesh settings | Min. distance between definition point and line [m]: 0,001<br>Definition of mesh element size for panels: Automatic<br>Average size of panel element [m]: 1,00<br>Elastic mesh: on<br>Use automatic mesh refinement: off |
| | 1D elements | Min. length of beam element [m]: 0,100<br>Max. length of beam element [m]: 1000<br>Avg. size of tendons, element [m]: 1,000<br>Generation of nodes in connections: off<br>Generation of variable eccentricity: off<br>Division on haunches and arbitrary members: 5<br>Division for integration strip and 2D-1D upgrade: 50<br>Mesh refinement following the beam type: None |
| | 2D elements | Maximal out of plane angle of a quadrilateral [mrad]: 30<br>Shape of mesh elements: Quadrangles and triangles |
| Advanced solver settings | General | Use lagrange multipliers: off<br>Neglect shear force deformation (Ay, Az>>A): off<br>Neglect shear center eccentricity: off<br>Bending theory of plate/shell analysis: Mindlin<br>Type of Solver: Iterative<br>Min. number of sections on member: 10<br>Apply property modifiers: on |
| | Nonlinearity | Geometrical nonlinearity: 3rd order (large deformation)<br>Method of calculation: Newton-Raphson<br>Increments: 10<br>Iterations: 40<br>Precision and robustness ratio: 1<br>Allow compression: off |
| | Dynamics | Eigen value solver: Lanczos<br>Eigenmodes: 1<br>Nodal mass matrix: diagonal<br>Use IRS: off<br>Method of time history analysis: direct time integration<br>Mass components: translation along global x, y and z |
| | Initial stress | Initial stress: on<br>Stress from member nonlinearity data: on |

*Table 5 Overview of the calculation settings of the model*

**Nonlinear analysis**

The canopy bridge was analyzed using a geometrically nonlinear approach to calculate the deflection of the bridge. With every load on a random point on the cable, the shape of the cable changes. To calculate the deformation of the cable, the shape of the cable has to be taken into account. In SCIA Engineering the deflection of the cable can be calculated, while taking the shape of the cable into account.

In the SCIA Engineer model have the cables been assigned to be a nonlinear member. In the beam nonlinearity tab the type 'Cable' was chosen with Self weight turned on and with a normal force of 60 kN. In the same tab, the direction of the load -z and $P_n$ is 0 was taken. When taking a lower normal force, the model would not work as the stiffness matrix would become singular.

Then the load cases were made, see the section below for all specified load cases, and assigned to a non-linear combination. Every non-linear combination consists of 1 load case. In this model a load case is a load combination. In the 'Nonlinear combinations' tab the type 'Serviceability' and the stage for composite analysis model 'Final Stage, long term' were chosen. With a factor of 1,0.

When clicking on the 'Calculate' button, the FE analysis tab pops up. In the FE analysis tab, under 'Nonlinearity' the geometrical nonlinearity used is '3rd order (large deformation)'. The method of calculation used is 'Newton-Raphson'. The number of increments is 10, with the maximum iterations of 40. The precision ratio and robustness ratio are both 1. A lot more settings were used, see Table 5 and appendix C – model properties, in those sections all calculation settings can be found.

**Load case 0: Self weight + pretension**

Only the self weight and pretension are in this combination. SCIA takes these loads automatically that is the reason why no loads are visible in Figures 6 and 7.

self weight + pretension

$$\sum F_d = G_k + P_k \qquad (2.3.1)$$

The maximum deformation is in the middle of the length of the construction. The maximum deformation under these conditions will be 19,4 mm.

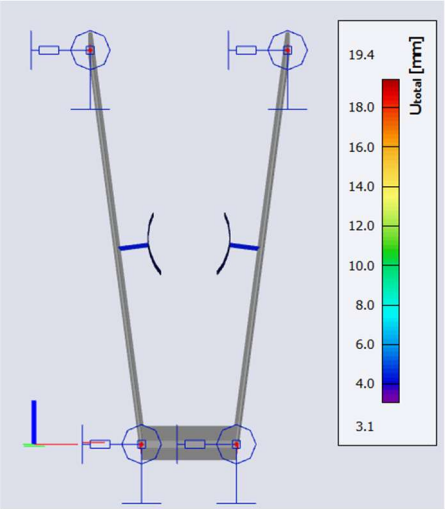



*Figure 6 Load case 0 in y-direction*

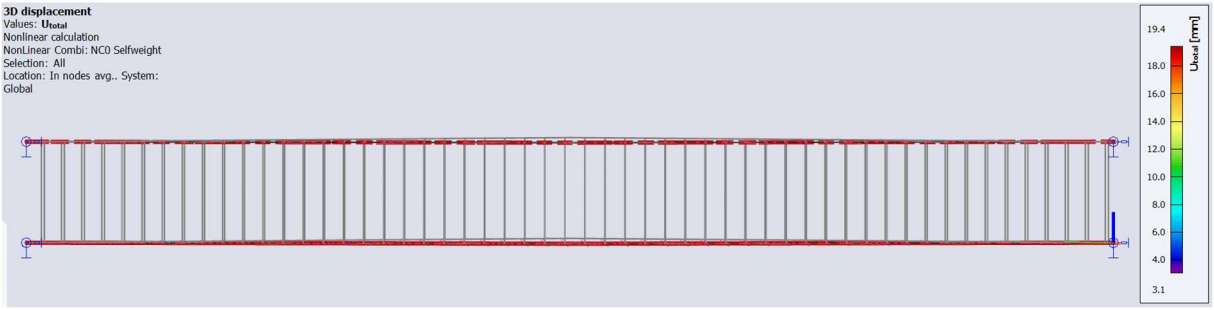



*Figure 7 Load case 0 in x-direction*

**Load case 1: Leading: Pedestrians**

Selfweight + 2 pedestrian (point loads)

This combination will not show the largest SLS deformation, as no other load is involved. This load case is used as reference, so it can later be compared. Nevertheless it is useful for comparison of the results later on. This load combination looks like formula

$$\sum F_d = G_k + Q_{k,1} + P_k \qquad (2.3.2)$$

Where:

$G_k$ = Self-weight SCIA

$Q_{k,1} = 2 * 1{,}5\ kN$ Pedestrians are the leading load in this case.

$P_k = 60\ kN$ Prestressed cables

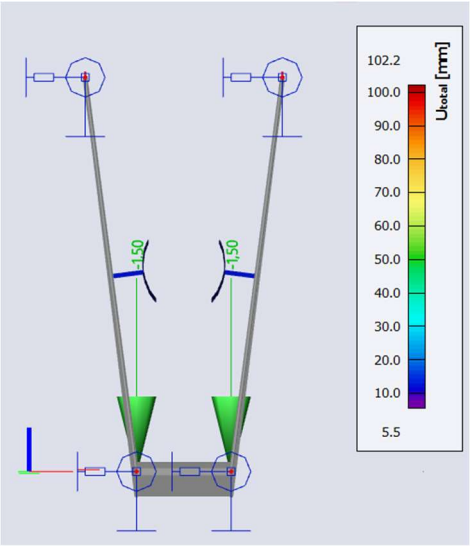



*Figure 8 Load case 1 in y-direction*

Resulting deformation: 102,2 mm

Figure 8 and 9 show the shape of the deformation and the value of the maximum deformation, 102,2 mm, in this case.




*Figure 9 Load case 1 in x-direction*

## Characteristic combinations

All characteristic load combinations are computed. Characteristic combinations are the combinations of loads that happen under normal conditions.

The general formula for the characteristic load combinations is formula 1.3.1

$$\sum F_d = \sum_i G_{k,i} + Q_{k,1} + \sum_{j>1} \psi_{0,j} Q_{k,j} + P_k \qquad (1.3.1)$$

**Load case 2a: leading: Pedestrians + downward wind**

Selfweight + 2 pedestrian + downward and horizontal wind

$$\sum F_d = G_k + Q_{k,1} + \psi_{0,2}(Q_{k,2} + Q_{k,3}) + P_k \qquad (2.3.3)$$

Where:

$G_k$ = Self-weight SCIA

$Q_{k,1} = 2 * 1{,}5\ kN$ Pedestrians are the leading load.

The wind load is a combined load.

$Q_{k,2} = Q_{wind,x} * diameter\ of\ cables$

$Q_{k,2} = 2{,}753 * 0{,}018 = 0{,}050\ kN/m$

$\psi_{0,2} Q_{k,2} = 0{,}3 * 0{,}050 = 0{,}015\ kN/m$ combination value of wind load in x direction per cable.

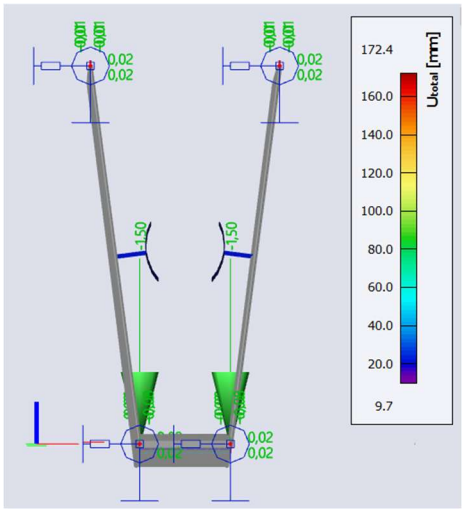



*Figure 10 Load case 2a in y-direction.*

$Q_{k,3} = Q_{wind,z} * number\ of\ WPC\ planks * width\ WPC * length\ WPC/2$

$Q_{k,3} = -2{,}477 * 54 * 0{,}450 * 0{,}20 * \frac{1}{2} * \frac{1}{21{,}6} = -0{,}275\ kN/m$

$\psi_{0,3} Q_{k,3} = 0{,}3 * -0{,}275 = -0{,}08\ kN/m$ combination value of wind load from WPC planks on lower cables in z direction per cable.

$P_k = 60\ kN$ Prestressed cables

Resulting deformation: 172,4 mm

Figure 10 and 11 show the shape of the deformation and the value of the maximum deformation, 172,4 mm, in this case.

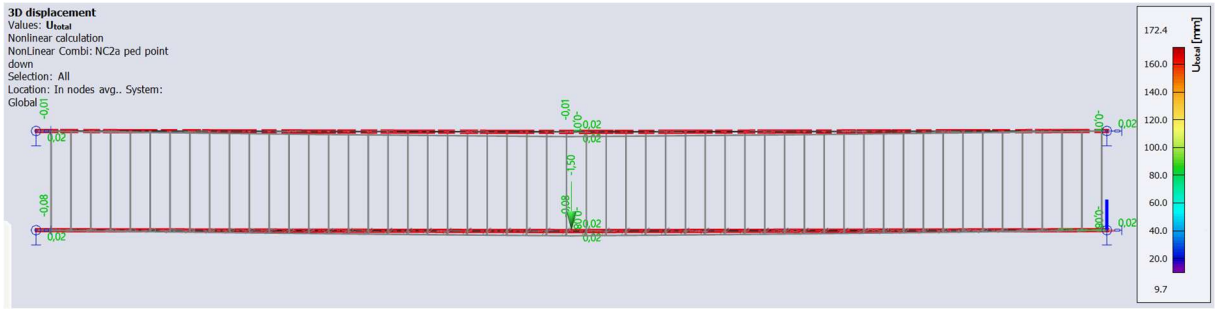



*Figure 11 Load case 2a in x-direction.*

**Load case 2b: leading: Pedestrians + upw. wind**

Selfweight + 2 pedestrian + wind upwards and horizontal

$$\sum F_d = G_k + Q_{k,1} + \psi_{0,2}(Q_{k,2} + Q_{k,3}) + P_k \qquad (2.3.4)$$

Where:

all values are the same as Load case 2a, except $Q_{k,3}$. The direction of $Q_{k,3}$ is upward. So:

$$\psi_{0,3}Q_{k,3} = 0{,}3 * 0{,}275 = +0{,}08\ kN/m$$

On the 2 lower cables.

Deformation: 49,4 mm

Figures 12 and 12 show the shape of the deformation and the value of the maximum deformation, 49,4 mm, in this case.

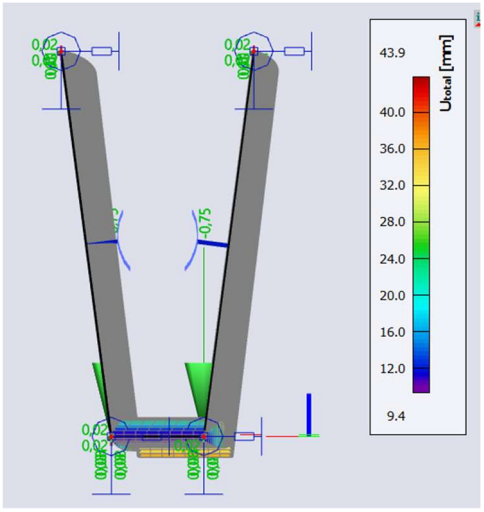



*Figure 12 Load case 2b in y-direction.*




*Figure 13 Load case 2b in x-direction.*

## Frequent combinations

Combination formula for frequent combination of actions

$$\sum F_d = \sum_i G_{k,i} + \psi_{1,1}Q_{k,1} + \sum_{j>1} \psi_{2,j}Q_{k,j} + P_k \qquad (1.3.2)$$

For almost all combinations $\psi_{2,i}$ will be equal to 0, as is visible in table A.2.8 of Eurocode 1990 except for thermal actions and construction actions. Both these actions are outside the scope of this thesis. For this reason, all frequent combinations must be lower than the characteristic combinations. Therefore all frequent combinations will not be calculated.

## Quasi-permanent combinations

$$\sum F_d = \sum_i G_{k,i} + \sum_j \psi_{2,j}Q_{k,j} + P_k \qquad (1.3.3)$$

The quasi-permanent combinations will be left out of this thesis, for the same reasons as the frequent combinations. $\psi_{2,i}$ is for almost all combinations equal to 0, and will not generate a higher deformation than the characteristic combinations.

## Seismic combinations

There is only one combination made for the seismic combinations, because $\psi_2 = 0$ for all actions for footbridges. This is because chances of an earthquake happening, while very high windspeeds are on the bridge, are very low.

Only for thermal, water and construction actions is $\psi_2$ not equal to 0, but these actions are outside the scope of this project.

**Dynamic analysis**

A Response Spectrum Analysis (RSA) has been done in SCIA to see what the effect on the deflection of the bridge is in case of a seismic event. First 2 masses are placed in the middle of the span of the bridge. These 2 masses represent two pedestrians with a mass of 152,9 kg each. So each mass is equal to 76,45 kg. This mass case was bound to load case 1 in the mass group section.

Then under 'seismic spectrum' > 'code parameters' the properties of the seismic spectrum were filled in. These code parameters were determined with NEC-SE-DS- Peligro Sismico parte 1. The values used are for a seismic event with acceleration of $0{,}15g\ m/s^2$. The g is the gravitational coefficient $9{,}81\ m/s^2$. Table 5 shows all the values. The calculation of these values can be found in appendix B, where all values are calculated. Table 6 shows an overview of the used parameters.

| **Code parameters** | |
|---|---|
| Coeff accel. ag | 0,15 |
| Ag – design acceleration $[m/s^2]$ | 1,472 |
| q-behavior factor | 1,00 |
| beta | 0,200 |
| S, Tb, Tc, Td manually | yes |
| Subsoil type | E |
| Spectrum type | Type 1 |
| Direction | Horizontal |
| S-soil factor | 1,40 |
| Tb | 0,175 |
| Tc | 0,93 |
| Td | 1,00 |

*Table 6 dynamic analysis parameters*

**Load case 3: selfweight + pedestrian mass + prestress**

The formula below is the combination of loads with a seismic event. Figures 14 and 15 show the shape of the deformation and the value of the maximum deformation, 57,8 mm, in this case.

$$F_d = G_k + A_{Ed,SLS} + P_k \qquad (2.3.5)$$

$G_k$ = Self-weight of the structure

$Q_k$ = 2 pedestrians of 90 kg each

Resulting deformation: 57,4 mm

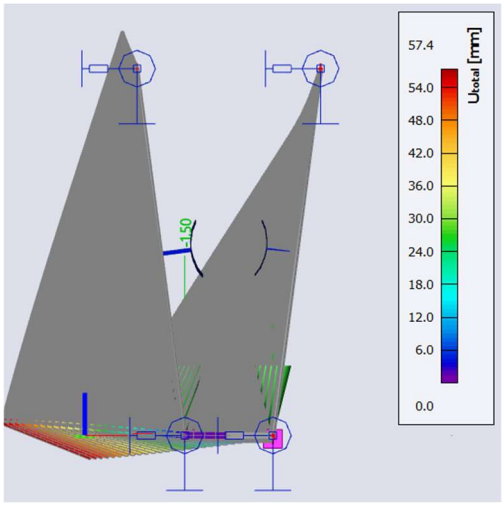



*Figure 14 Load case 3 in y-direction.*




*Figure 15 Load case 3 in x-direction.*

## Deformation check

This check is done to get a feeling for the size of the deflection. This is a very much simplified check. The deformation is calculated by hand. The horizontal line represents one of the lower cables. The self-weight of the cable and the load of the WPC-planks are modelled as point load in the middle of the bridge. There is a pre-tension of 60 kN in the cables. The supports are fixed hinges.

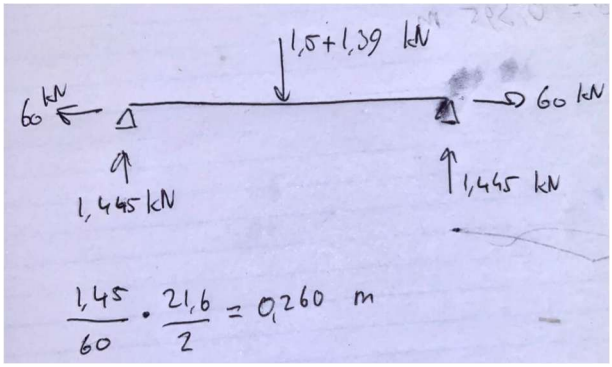



*Figure 16 calculation of deflection of point load*

According this calculation deformation should be around 260 mm. This means that deformation of the bridge should be less than 260 mm. This is because the loads are in the most unfavorable

position, this position causes the highest deformation. In this case the cable has the shape of a v.

**Cable theory**

When a line-load is put on a cable the cable will deform to the shape of a parabola. To approach the reality a bit better than the simple check, the deflection of the lower cable is calculated in the same way as in Informe final Puente dosel_Teoria de cables (Paredes, 2025), which uses the theory of 'Designing canopy walkways: Engineering calculations for building canopy access systems with cable-supported bridges' (Bouricius, 2002). The load for the calculation differs from the load in Paredes' rapport. In appendix D – Cable theory, the calculation can be found.

| Load case | Deflection [mm] |
|---|---|
| 0 | 108 |
| 1 | 118 |
| 2a | 129 |
| 2b | 106 |

*Table 7 results of deflection calculations with cable theory*

For every load combination a calculation has been done. These are only the deflections caused by the vertical loads on the cable. This means that the total deflection should be higher.

## Results

In Table 8 the results of the calculation in SCIA and of the cable theory are showed. When looking at these results it is interesting to see that combination 3 (pedestrians + seismic event) results in a smaller deformation than when there are only pedestrians on the bridge. This probably because it is a dynamic load, which works briefly on the construction. Due to inertia the bridge would not deform that much.

Load combination 2a shows the largest deformation. The load where needs to be designed for is load combination 2a. This is a combination of the two pedestrians and the wind in horizontal and downward vertical direction.

The wind load has quite a big impact on the deformation of the bridge. This is seen when comparing load combinations 1 (without wind) and 2a (with wind) with each other. The difference is $172{,}4 - 102{,}2 = 70{,}2\ mm$.

Another interesting thing to note is that, when looking at the results of the cable theory, the impact of 2 pedestrians is quite low on the deflection of the bridge. The deflection of the bridge, without any loads on it is way higher than the deflection in scia for load case 0. This is probably because the cables have a pre-tension of 60 kN on them in the SCIA model, the calculations with the cable theory do not have a pre-tension.

| Load combination | Description | Deflection [mm] | Verification Cable theory [mm] |
|---|---|---|---|
| 0 | Self-weight + Pre-tension | 19,4 | 108 |
| 1 | pedestrians | 102,2 | 118 |
| 2a | Pedestrians + horizontal & downward wind | 172,4 | 129 |
| 2b | Pedestrians + horizontal & upward wind | 49,4 | 106 |
| 3 | Pedestrians + seismic event | 57,4 | |

*Table 8 Results of the load cases.*

# 3. Discussion

The aim of this study was to find out what impact extreme events had on a canopy bridge. With de Tiputini bridge as an example to be learned of. The study on what extreme events a high occurrence had in the Amazon was done based on available literature. It was found that heat and cold waves, droughts, windthrows, floods and seismic events are all relevant extreme events for this region. It would be better to perform a data study on these events, so extreme windspeeds can be calculated. And the impact of temperatures on the deformation of the structure can be studied.

In table 9 the deformations of different calculations from different rapports are compared. There are different reasons for the difference in results. Table 10 shows differences between the models used in the rapports.

| Load case | SCIA<br>[mm] | Memoria Tecnica Puente Dosel (Paredes, 2025)<br>[mm] | Informe final puente dosel_Teoria de cables (Paredes, 2025)<br>[mm] | Anexos<br>[mm] | Verification<br>[mm] | Max. allowable deflection:<br>[mm] |
|---|---|---|---|---|---|---|
| 0 | 19,4 | - | - | - | 108 | 60 |
| 1 | 102,2 | 27 | 41 | 41,39 | 118 | 43,2 |
| 2a | 172,4 | 52 | 62 | 64,17 | 129 | 43,2 |
| 2b | 49,4 | - | - | - | 106 | 43,2 |
| 3 | 57,4 | - | - | - | | 43,2 |

*Table 9 Comparison of results of different rapports.*

Differences and similarities between the reports that possibly influenced the results.

| **Element** | **Memoria tecnica punte dosel** | **Informe final puente dosel_Teoria de cables (Paredes, 2025).** | **SCIA Model** |
|---|---|---|---|
| software | SAP2000 | SAP2000 | SCIA Engineer |
| norm | American Association of State Highway and Transportation Officials. (2020). AASHTO LRFD Bridge Design Specifications (9th ed.). AASHTO | American Association of State Highway and Transportation Officials. (2020). AASHTO LRFD Bridge Design Specifications (9th ed.). AASHTO | Eurocode |
| Pedestrian load | $415\ kg/m^2$ or $93{,}375\ kg/m$ | $415\ kg/m^2$ or $93{,}375\ kg/m$ | $1{,}5\ kN$ per pedestrian |
| Self-weight | Automatic for modelled cables. Manuel for all other materials: $0{,}039\ kg/m$ | Automatic for modelled cables. Manuel for all other materials: $0{,}039\ kg/m$ | Automatic via SCIA |
| Wind load | $170{,}88\ kg/m^2$ On lower cable: $3{,}08\ kg/m$ | $170{,}88\ kg/m^2$ On lower cable: $3{,}08\ kg/m$ | Calculated according Dutch Eurocode |
| WPC-planks | Modelled as line load on lower cable | Modelled as line load on lower cable | Automatic via SCIA |
| Profile cable | Not found | Circular steel frame cable | RD18, with the same material properties |
| Type of analysis | | nonlinear | nonlinear |
| Supports | fixed | fixed | Flexible Stiffness: 2,905 $MN/m$ |

*Table 10 Properties of the different models*

Another import thing to note is that the pre-tension of 60 kN is too high for the size of the cable. The cable would snap. Unfortunately when lowering it in the model, and calculating again. The model would show an error which said that the stiffness matrix is singular! And the construction is unstable. The same thing goes for the stiffness of the supports, lowering it more would give the same error. For that reason they are left in.

# 4. Recommendations

Some additional recommendations are given in the general part. In the robustness part a recommendation is given for a second load path. In case one of the supporting trees fails.

## 4.1 General

The wind load is not representative for an area in Ecuador. It is recommended to calculate the wind load with the national norm of Ecuador. Or even better, do a data study on wind speeds and there probability of occurrence. And then use the Ecuadorian norm or the Eurocode for wind load based on local wind speeds.

As the wind  load is probably incorrect, it is recommended to compute the load combinations again with a better estimated wind load and a lower pedestrian load. So more representative deformations can be established.

The model still gives errors when changing certain values to a more realistic one. For example, lower the stiffness of the supports and lowering the pre-tension in the cables, would give a better representation of reality. This has been tried but the model gives errors. Another thing to improve the model is to model the supporting tree and the tower so a more detailed and more realistic model is gained. And to connect this section with the other bridge sections.

Some extreme events have an impact on the material properties, therefore it is advisable to study the influence of wet, dry, hot, and cold periods, and a combination of them, on materials used in the structure.

## 4.2 Robustness

When one of the supporting trees falls down the bridge deck will also come down. To prevent the bridge deck from falling down a second load path can be created, this is also called increasing the robustness of the bridge. In this part 4 different examples of load paths will be given.

1. **Lifelines**

The first example is to hang two heavy cables above the canopy,  between two towers. These cables will function as ‘lifelines’. When one of the supporting trees fails the bridge will stay in the air, as it is vertically connected with the lifelines. It will look a bit like the canopy walkway in Nyungwe Forest National Park shown in Figure 17.

A rough sketches of the side profile is provided in respectively Figure 18, to create a better understanding.

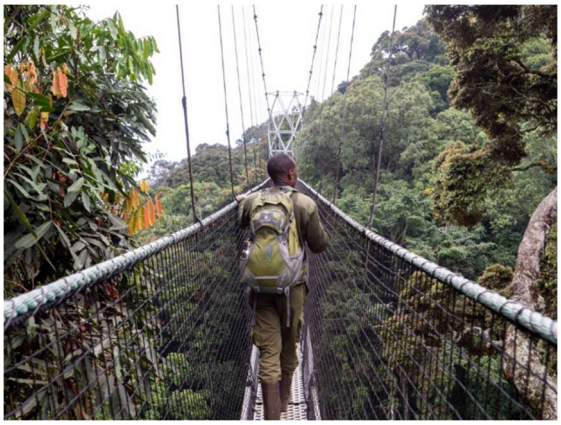


*Figure 17  Canopy walkway, Nyungwe Forest National Park (Simons, 2025)*

To realize this construction, two new towers have to be made at the beginning and the end of the construction. The existing tower probably has to be improved to accommodate or a big extra load.

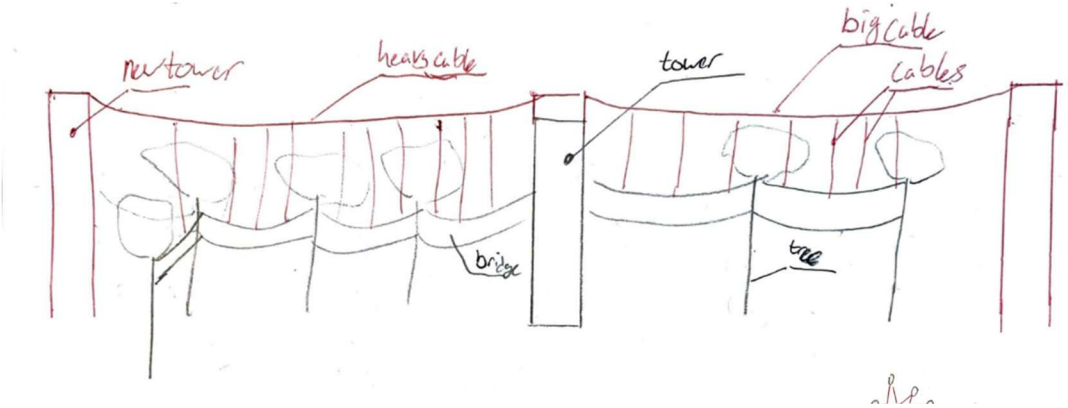



*Figure 18 Sideprofile of 'lifeline' sketch*

### 2. More tree supports

Another option is to connect each section, with a section being the bridge between to supporting trees, to more trees. In this way the load of the bridge will be divided over surrounding trees. As most of the surrounding trees are not big enough to support the whole bridge on its own.

By connecting the extra support cables in line with each other, the bridge will stay horizontal and will stay straight. Depending on the strength of an individual tree, more connections can be made with that tree. The sketch, Figure 19, gives a rough view of the idea. The supporting trees on the sides of the bridge do not actually exist.

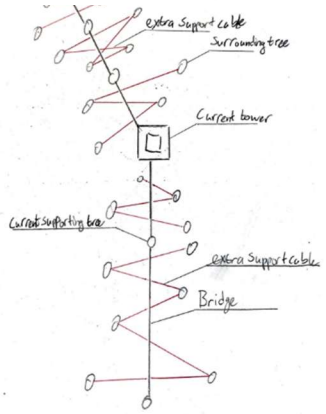



*Figure 19 Top view of 'More tree supports' sketch.*

### 3. Columns, additional construction

Columns, or a small construction could be place at the end of each section. The columns could, for example, be placed in the shape of a triangular pyramid (tetrahedron). So it creates a supporting structure. See Figure 20.

When the tree falls over and does not the small construction, the ends of the sections could be connected to each other.

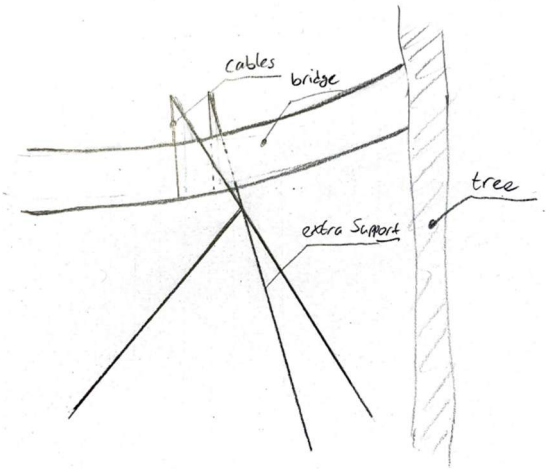



*Figure 20 sketch of an additional construction.*

### 4. Support the supporting trees

Another option would be to put three cables around the supporting tree, like seen Figure 21. These cables would be connected a bit above the cables of the bridge and with the ground. At the ground they would be connected with anchors. This way the tree would not be able to fall over, and in case there would be an extreme event where there's a lot of would. The tree would snap above the connections with the tree.

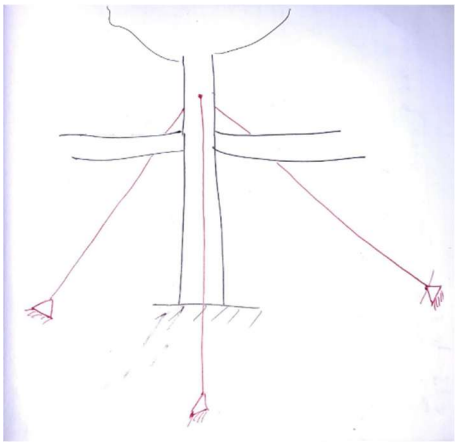


*Figure 21 sketch of support the supporting trees.*

A big disadvantage is that the broken tree will start to rot, and over time it will not reliable enough to use it as a support for the bridge. But it does hold the bridge in place.

These anchor cables will not be spun to tight, so the tree can still deform. But when the tree deforms so much it can fall over, than cables would be tight, so the tree breaks.

# 5. Conclusion

The aim of this study was to evaluate the deformation of a canopy bridge under an extreme events like wind or a seismic event. To come up with design considerations.

One of the design challenges for designing a canopy bridge, in terms of governing loads and withstanding extreme events, is designing for throwdowns. They have a very local character and cause very high windspeeds resulting in high loads on the construction.

The effect of a seismic event with a probability of 10% of in 50 years, causes a deformation of 57,8 mm. This is more than the maximum allowable deformation of 43,2 mm. The bridge would not meet the requirements.

The load combination with the highest deformation was load combination 2a. This combination was a combination of horizontal and downward wind load, 2 pedestrians and the self-weight of the bridge. The deformation in the SCIA model was 172,4 mm in the middle of the bridge. And the vertical deformation calculated according the cable theory is 129 mm, this is without pre-tension. These are both higher than the maximum allowable deformation of $L/500 = 43{,}2$ mm, from Memoria Tecnica Puente Dosel (Paredes, 2025). This means that the bridge does not meet the standard for the serviceability criterium.

The model in SCIA is not an accurate representation of the bridge. There is a pre-tension of 60 kN in the cables, which could not be lowered otherwise no results would be obtained. The same goes for the stiffness of the supports. Errors would be obtained for the load cases with the least amount of loading. Therefore it is recommended to improve the model, by someone with more knowledge on cable bridges in SCIA Engineer, or make a new one. And check if the same results are observed.

# Appendix A – Wind load

The wind load is computed with the Dutch Eurocode: NEN-EN 1991-1-4:2011+A1+C2:2011/NB:2019+C2:2023 nl, which is also written in Dutch, references to the Eurocode are translated from Dutch to English to make the rapport more readable.

All the choices that needed to be made are noted below in the same order as you would go through the Eurocode.

The consequence class is CC1b.

The wind area (windgebied) chosen is area III, because these areas come closest to a land climate. This gives $v_{b,0} = 24{,}5\ m/s$ from table 1. From table 2: $K = 0{,}281$ and $n = 0{,}5$.

**Tabel NB.1 — $v_{b,0}$ voor toepassing in Nederland**

| Windgebied | I | II | III |
|---|---|---|---|
| $v_{b,0}$ [m/s] | 29,5 | 27,0 | 24,5 |

*Table 11: windspeed vb,0 from Eurocode 1.*

**Tabel NB.2 — De factoren *K* en *n* voor toepassing in Nederland**

| Windgebied | I | II | III |
|---|---|---|---|
| *K* | 0,2 | 0,234 | 0,281 |
| *N* | 0,5 | 0,5 | 0,5 |

*Table 12: factors K and N from Eurocode 1*

The terrain category determines the terrain parameters in table NB.3. The terrain category is III developed Area (Bebouwd) $z_0 = 0{,}5\ m$, $z_{min} = 7\ m$, as trees rise above the structure.

**Tabel NB.3 – 4.1 — Terreincategorieën en terreinparameters**

| Terreincategorie | | $z_0$ m | $z_{min}$ m |
|---|---|---|---|
| 0 | Zee of kustgebied aan zee | 0,005 | 1 |
| II | Onbebouwd gebied | 0,2 | 4 |
| III | Bebouwd gebied | 0,5 | 7 |

*Table 13: Terraincategory and terrainparameters from Eurocode 1*

The extreme thrust (stuwdruk) $q_p = 0{,}80\ kN/m^2$ is from table 4, with $C_0 = 1$

| Hoogte | Gebied I | | | Gebied II | | | Gebied III | |
|---|---|---|---|---|---|---|---|---|
| m | Kust | Onbebouwd | Bebouwd | Kust | Onbebouwd | Bebouwd | Onbebouwd | Bebouwd |
| 25 | 1,88 | 1,36 | 1,16 | 1,57 | 1,14 | 0,97 | 0,94 | 0,80 |

*Table 14: extreme thrust from Eurocode 1.*

Computing $C_sC_d$ from Appendix B from Eurocode 1

b = 2 m this is the width of the construction.

$C_s$ = 0,85 from table NB.22 – C2 in Eurocode 1.

$C_d$ = 1,05

$C_sC_d = 0{,}85 * 1{,}05 = 0{,}893$, which is bigger than the required minimum of 0,85

**Force coefficients in the x-direction – General method (from 8.3.1 of Eurocode)**

$d_{tot}$ = hight of the bridge deck + cable diameter

$d_{tot} = 0{,}04 + 0{,}018 = 0{,}058\ m$

**Force coefficients in the x-direction – Simplified method (from 8.3.2 of Eurocode)**

$c_0 = 1{,}0, \qquad k_l = 1{,}0$

The values for $c_{f0}$ and $c_e$ are computed from table 5

| $b/d_{tot}$ | $(c_{f,0})$ | $z_e \leq 20$ m | | $z_e = 50$ m | |
|---|---|---|---|---|---|
| | | Kust of open zee ($c_e = 3{,}32$) | Onbebouwd gebied ($c_e = 2{,}34$) | Kust of open zee ($c_e = 3{,}90$) | Onbebouwd gebied ($c_e = 3{,}03$) |
| ≤ 0,5 | (2,4) | 8,0 | 5,6 | 9,4 | 7,3 |
| ≥ 4,0 | (1,3) | 4,3 | 3,0 | 5,1 | 4,0 |

*Table 15: Wind load factor voor bridges from Eurocode 1*

The factors $C_{f,0}$, $C_e$ and Care interpolated values from table 5. The interpolation is done in python, see the screenshots below:

```
%matplotlib inline
import numpy as np
import matplotlib.pyplot as plt
import pandas as pd
from scipy.interpolate import interp1d
```

# windload ¶

```
b = 1 # width of construction [m]
Cs = 0.85
Cd = 1.05
dd = 0.04 # height of bridge deck [m]
r = 0.018 # railing width [m]
ze = 28 # height construction [m]

CsCd = max(0.85,Cs * Cd)
dtot = dd + r #[m]

#extreme stuwdruk  (trust)
qb = 0.94 # [kN/m2]
c0 = 1.0
kl = 1.0


def windloadC(b, d, ze):
    bd = b/d
    xbd = [0.5,4] #b/dtot
    ycf0 = [2.4,1.3] #cf,0
    yC20 = [5.6,3.0] #C ze<=20
    yC50 = [7.3, 4.0] #C ze = 50
    z = [20, 50]
    yce = [2.34, 3.03]

# ze <= 20
    if (bd <= 0.5) & (ze <= 20) :
        cf0, C, Ce = 2.4, 5.6, 2.34

    elif (bd > 0.5) & (bd < 4.0) & (ze <= 20):
        f_cf0 = interp1d(xbd, ycf0, kind='linear')
        cf0 = f_cf0(bd)
        f_C = interp1d(xbd, yC20, kind='linear')
        C = f_C(bd)
        Ce = 2.34
    elif (bd >= 4.0) & (ze <= 20):
        cf0, C, Ce = 1.3, 3.0, 2.34

# 20 < Ze < 50
    elif (bd <= 0.5) & (ze > 20) & (ze <50):
        cf0 = 2.4
        f_Ce = interp1d(z, yce, kind='linear')
        Ce = f_Ce(ze)
        y_C = [5.6, 7.3]
        f_C = interp1d(z, y_C, kind='linear')
        C = f_C(ze)
    elif (bd > 0.5) & (bd < 4.0) & (ze > 20) & (ze <50):
        f_cf0 = interp1d(xbd, ycf0, kind='linear')
        cf0 = f_cf0(bd)
        f_Ce = interp1d(z, yce, kind='linear')
        Ce = f_Ce(ze)
        f_C20 = interp1d(xbd, yC20, kind='linear')
        f_C50 = interp1d(xbd, yC50, kind='linear')
        y_C = [f_C20(bd), f_C50(bd)]
        f_C = interp1d(z, y_C, kind='linear')
        C = f_C(ze)
    elif  (bd >= 4.0) & (ze > 20) & (ze <50):
        cf0 = 1.3
        f_Ce = interp1d(z, yce, kind='linear')
        Ce = f_Ce(ze)
        y_C = [3.0, 4.0]
        f_C = interp1d(z, y_C, kind='linear')
        C = f_C(ze)
```

```
# ze = 50 m
    elif (bd <= 0.5) & (ze == 50):
        f_cf0 = interp1d(xbd, ycf0, kind='linear')
        cf0 = f_cf0(bd)
        f_C = interp1d(xbd, yC50, kind='linear')
        C = f_C(bd)
        Ce = 3.03
    elif (bd > 0.5) & (bd < 4.0) & (ze == 50):
        f_cf0 = interp1d(xbd, ycf0, kind='linear')
        cf0 = f_cf0(bd)
        f_C = interp1d(xbd, yC50, kind='linear')
        C = f_C(bd)
        Ce = 3.03
    elif (bd >= 4.0) & (ze == 50):
        cf0, C, Ce = 1.3, 4.0, 3.03
    return cf0, C, Ce

cf0, C, Ce = windloadC(2,0.058,28)
# C = 1
print(f'cf0 = {cf0:.4f}, C = {C:.4f}, Ce = {Ce:.4f}, Cs = {Cs:.4f}, Cd = {Cd:.4f}, CsCd = {CsCd:.4f}')
```

```
cf0 = 1.3000, C = 3.2667, Ce = 2.5240, Cs = 0.8500, Cd = 1.0500, CsCd = 0.8925
```

```
Qkx = CsCd*c0*cf0*Ce*qb
Qky = 0.4 * Qkx
Qkz = 0.9 * Qkx

print(f'Windload Qkx = {Qkx:.3f}, Qky = {Qky:.3f}, Qkz = {Qkz:.3f}')
```

```
Windload Qkx = 2.753, Qky = 1.101, Qkz = 2.477
```

*Figure 22 calculation in python of wind factors*

$C_{f,0}$ = 1,30

$C_e$ = 2,4090

$Q_{k,x} = C_s * C_d * C_0 * C_{f,0} * C_e * q_p$

$Q_{k,x} = 2{,}753\ kN/m^2$

This is the wind load per square meters of area that is perpendicular to the wind on the bridge.

**Wind forces on bridge decks in the y-direction (8.3.4 from the Eurocode)**

40% of the wind forces are in the x-direction for plate girder bridges and truss bridges. Wind forces in the y-direction occur simultaneously with an equal wind force in the x-direction..

$Q_{k,y} = 0{,}40 * Q_{k,x}$

$Q_{k,y} = 1{,}101\ kN/m^2$

$Q_{k,y}$ will work in a direction that it will only generate loads on the supporting trees and not really much on the bridge, therefore it will be left out later on.

**Wind forces on bridge decks in the z-direction (8.3.3 from the Eurocode)**

$C_{f,z} = \pm 0{,}9$ and needs to be calculated with the wind in x-direction.

$Q_{k,z} = C_{f,z} * Q_{k,x}$

$Q_{k,z} = \pm 0{,}9 * 2{,}627 = \pm 2{,}477\ kN/m^2$

**Conclusion:**

$Q_{k,x} = 2{,}753\ kN/m^2$

$Q_{k,y} = 1{,}101\ kN/m^2$

$Q_{k,z} = \pm 2{,}477\ kN/m^2$

# Appendix B – Seismic Action

From NEC-SE-DS- Peligro Sismico parte 1

Uses data from 2011, so it is a bit old.

The seismic factor y for a 10% chance of exceedance in 50 years (475-years return period) is determined first. Tiputini is situated in Seismic zone I, which has seismic hazard characterization: Intermediate.

The maximum rock acceleration expected for the design earthquake is the z-factor

$$Z - factor = 0{,}15 * g \quad m/s^2$$

Where $g$ is the gravitational constant $9{,}81\ m/s^2$.

Poblacion: Tiputini

Parroquia: Tiputini

Canton: Aguarico

Provinicia: Orellana

The ground type for the region is: Eastern low hills (colinas bajas orientales), River landscapes (Paisajes fluviales)

Soil type is Alluvial deposits (sediments like silt, sand, clay and gravel). Fm. Curaray.

Therefore the soil is a soft soil with properties:

- $V_s < 180\ m/s$
- Thickness H of soft clay is greater than 3m.

Soil type (Tipo de perfil): **E**

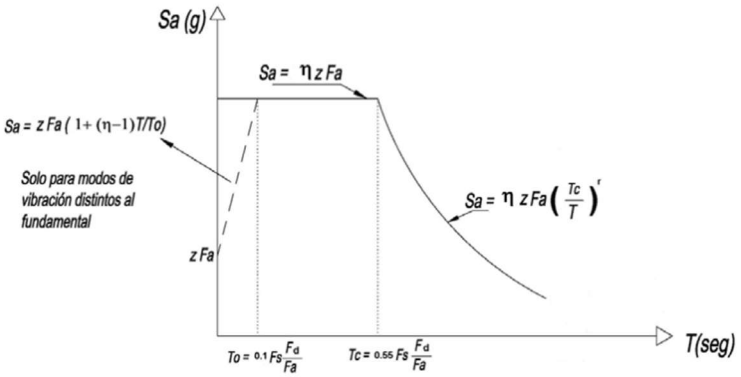
Then the soil profile coefficients $F_a, F_d, y, F_s$ are determined in section 3.2.2. of the Ecuadorian norm.

The soil amplification coefficient in short period zone, $F_a = 1{,}8$

The amplification of the ordinates of elastic displacement response spectrum for rock design, $F_d = 2{,}1$

Nonlinear soil behavior, $F_s = 1{,}5$

Design horizontal elastic spectrum under accelerations is determined according section 3.3.1 of the norm.




$$S_a = \begin{cases} z * F_a * (1 + (\eta - 1) * \frac{T}{T_0}) & 0 \le T \le T_0 \\ \eta * z * F_a & 0 \le T \le T_C \\ \eta * z * F_a \left(\frac{T_C}{T}\right)^r & T > T_C \end{cases}$$

Where:

$\eta = 2{,}60$ :provincias del Oriente

$r = 1{,}5$ for soil type E

$T_C = 0{,}55 * F_s * \frac{F_d}{F_a}$ $\quad T_C = 0{,}55 * 1{,}5 * \frac{2{,}1}{1{,}8} = 0{,}93\ s$

$T_0 = 0{,}1 * F_s * \frac{F_d}{F_a}$ $\quad T_0 = 0{,}1 * 1{,}5 * \frac{2{,}1}{1{,}8} = 0{,}175\ s$

$T_L = \min\{2{,}4 * F_d;\ 4\}$ $\quad T_L = \min\{2{,}4 * 2{,}1 = 5{,}04; 4\} = 4\ s$

$$S_a = \begin{cases} 0{,}15 * 1{,}8 * \left(1 + (2{,}6 - 1) * \frac{T}{0{,}175}\right) = 0{,}27 + 9{,}14 * T & 0 \le T \le T_0 (= 0{,}175\ s) \\ 2{,}60 * 0{,}15 * 1{,}8 = 0{,}702 & 0 \le T \le T_C (= 0{,}93\ s) \\ 2{,}60 * 0{,}15 * 1{,}8 \left(\frac{0{,}93}{T}\right)^{1{,}5} = 0{,}702 \left(\frac{0{,}93}{T}\right)^{1{,}5} & T > T_C (= 0{,}93\ s) \end{cases}$$

Fundamental period of vibration of structure, $T$

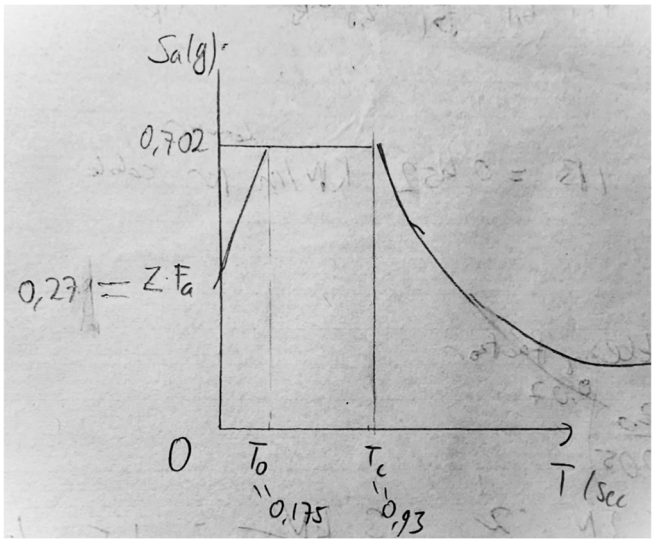



*Figure 23 seismic spectrum*

The seismic input for the model in SCIA are:

The seismic spectrum is the result of the code parameters.

- $T_0$ From appendix B is the same as $T_b$ in the model.
- $T_C$ from appendix B is the same as $T_c$ in the model.
- $T_d = 1{,}00$

Seismic spectrum:

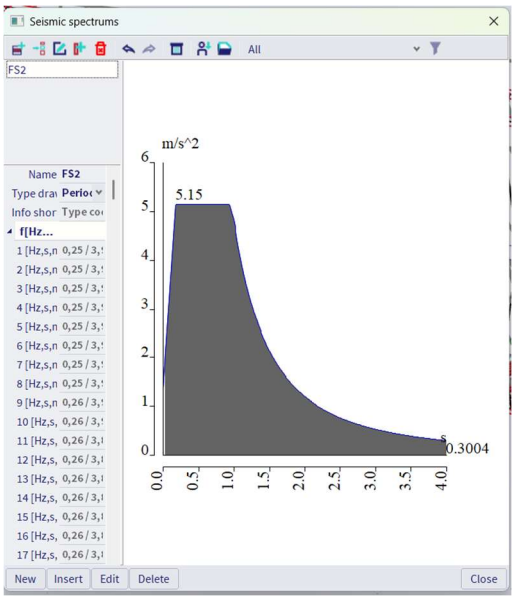



*Figure 23 seismic spectrum in SCIA*

Code parameters:

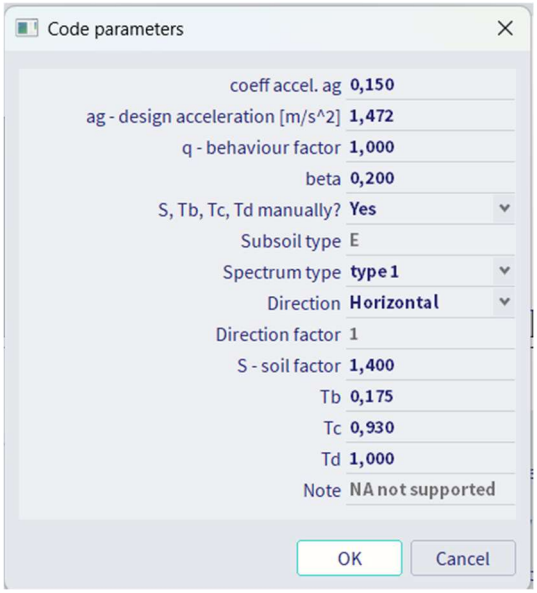



*Figure 24 code parameters in SCIA*

# Appendix C – Model properties SCIA Engineer

**Project settings:**

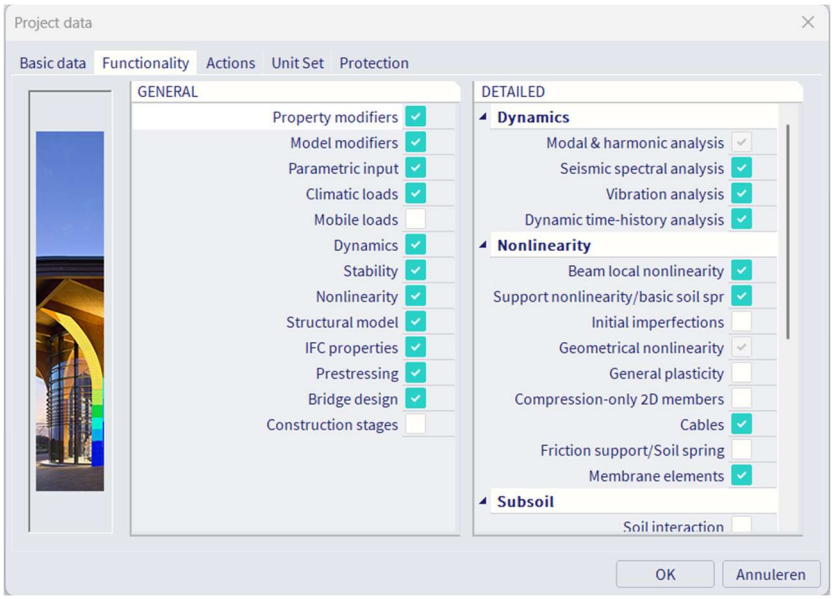



*Figure 25 project settings in SCIA*




*Figure 26 more project settings in SCIA*

## 7x19 galvanized steel cables:

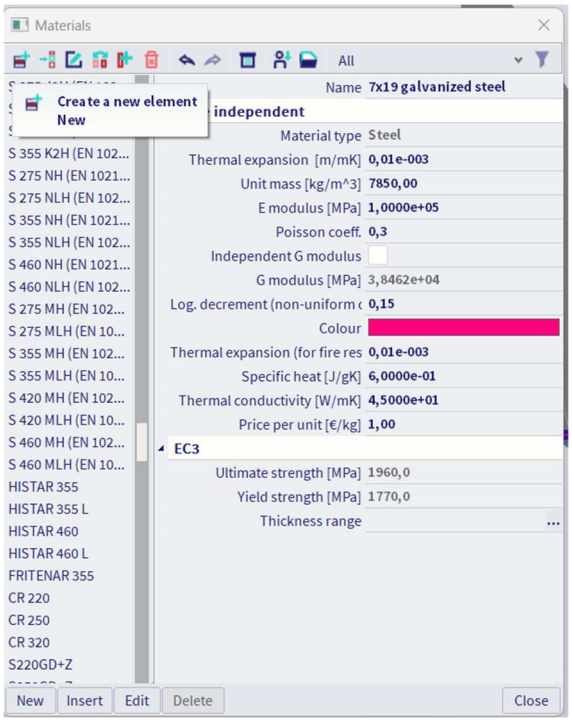



*Figure 27 material properties of galvanized steel in SCIA*

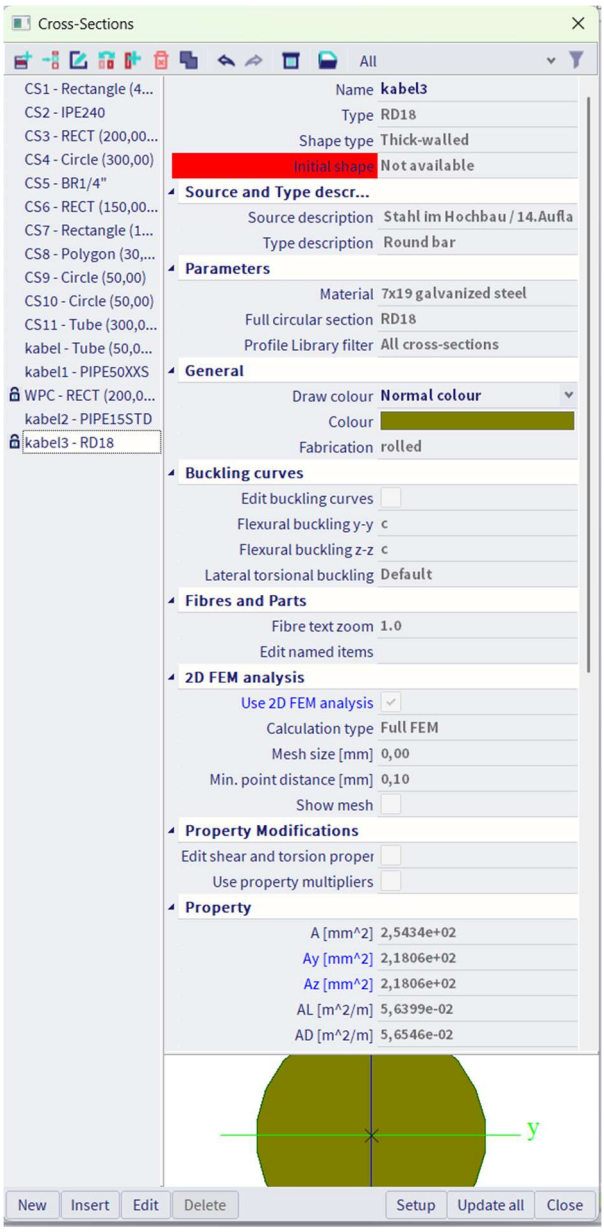



*Figure 28 shape of the 7x19 galvanized steel cable*

### WPC-planks

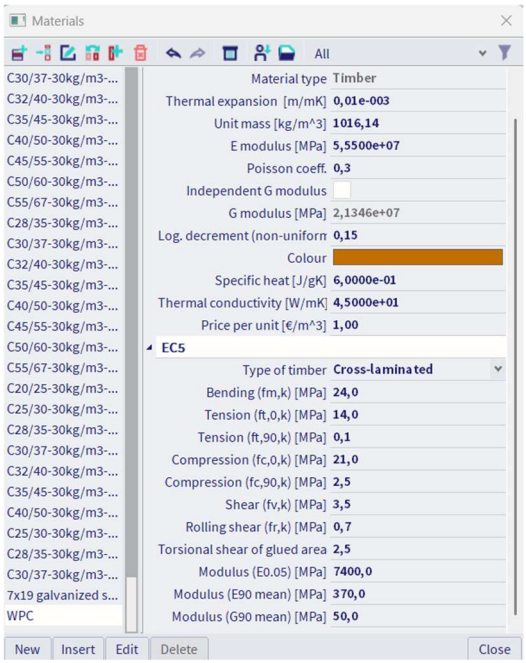



*Figure 29 material properties of WPC-planks in SCIA*

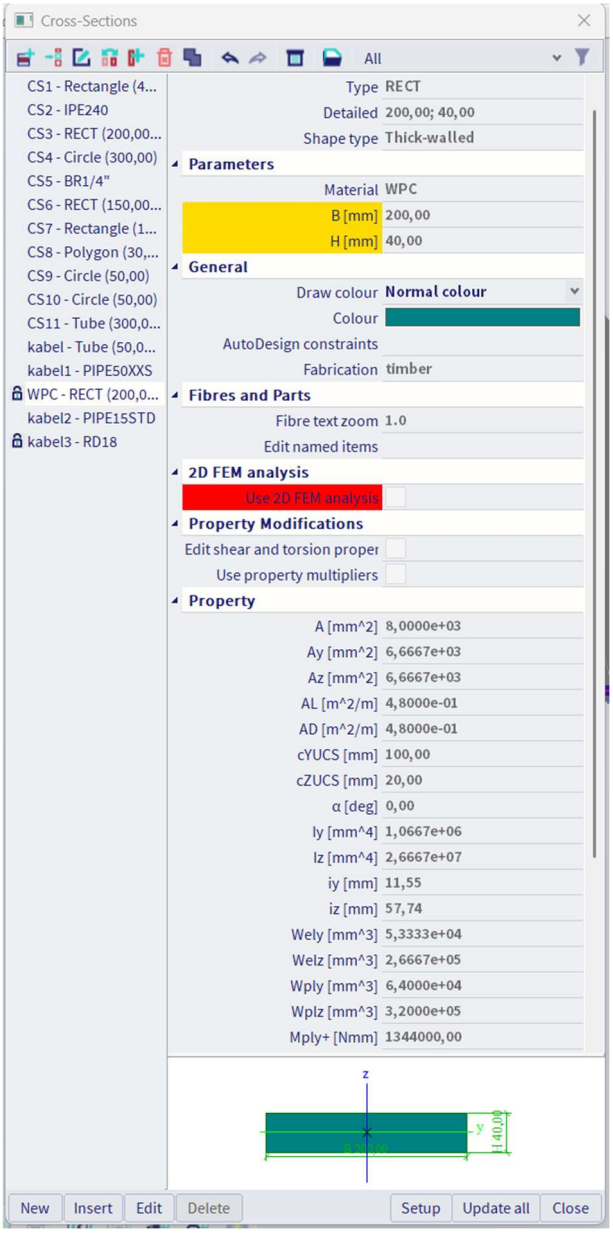



*Figure 30 shape properties of WPC-planks in SCIA*

### Supports:

The supports are flexible supports, they represent a supporting tree, by having a stiffness computed with from figure C. and formulas C. , C. and C. :

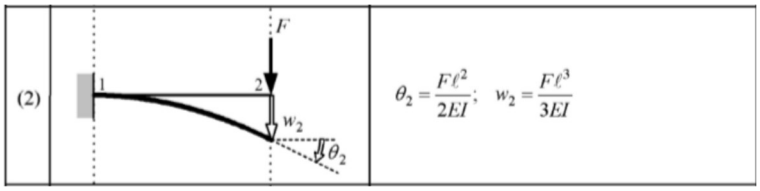



*Figure 31 forget-me-not to determine stiffness of tree*

$$w = \frac{F*l^3}{3*E*I} \qquad \text{(C.)}$$

$$I = \frac{1}{4}\pi * r^4 \qquad \text{(C.)}$$

Combining formulas C. and C. , and rewriting the formula gives the stiffness of the tree at a height of 23 m:

$$\frac{F}{w} = \frac{\frac{3}{4}E*\pi*r^4}{l^3} \qquad \text{(C.)}$$

Where:

| | |
|---|---|
| $E = 15\ GPa$ | is the E-modulus of the tree from 'INFORME Y MEMORIA TÉCNICA JUSTIFICATIVA PREVIA LA CONSTRUCCIÓN DE LOS PUENTES DOSEL' (Enrique Jacome, ). |
| $r = 1\ m$ | radius of the tree. |
| $l = 23\ m$ | Height of the tree. |

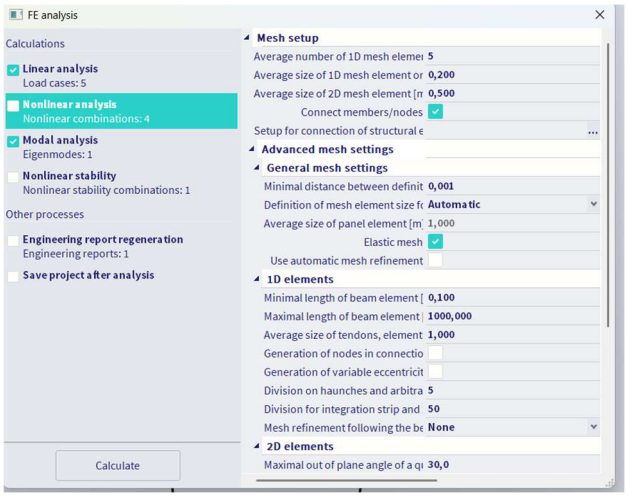
$$\frac{F}{w} = \frac{\frac{3}{4}*15000*\pi*1^4}{23^3} = 2{,}905\ MN/m$$

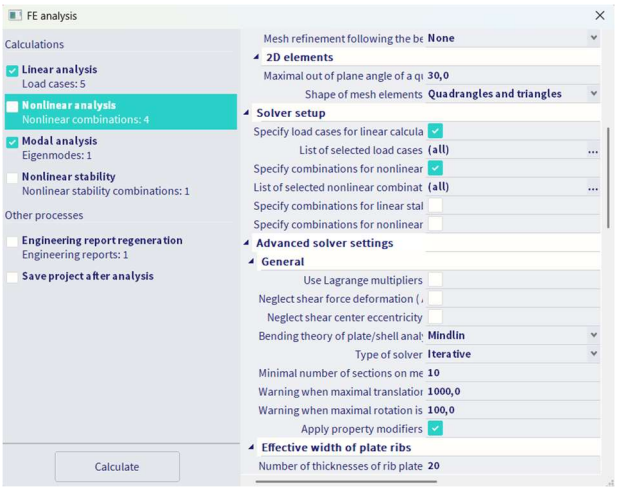
Input in model:

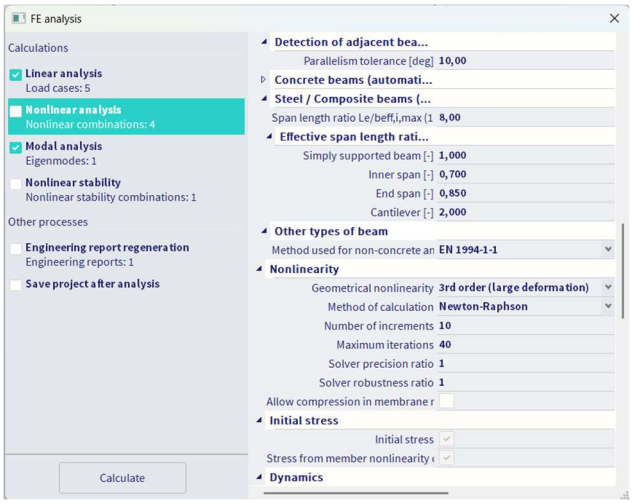
| | |
|---|---|
| Type | Standard |
| Angle [deg] | |
| Constraint | Custom |
| X | Flexible |
| Stiffness X [MN/m] | 2,9050 |
| Y | Flexible |
| Stiffness Y [MN/m] | 2,9050 |
| Z | Rigid |
| Rx | Free |
| Ry | Free |
| Rz | Free |
| Default size [m] | 0,200 |
| Node | MIXED |

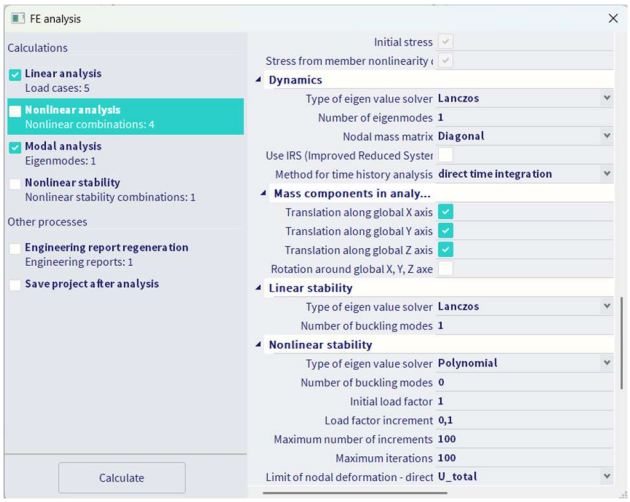
*Figure 32 stiffness properties of supports*

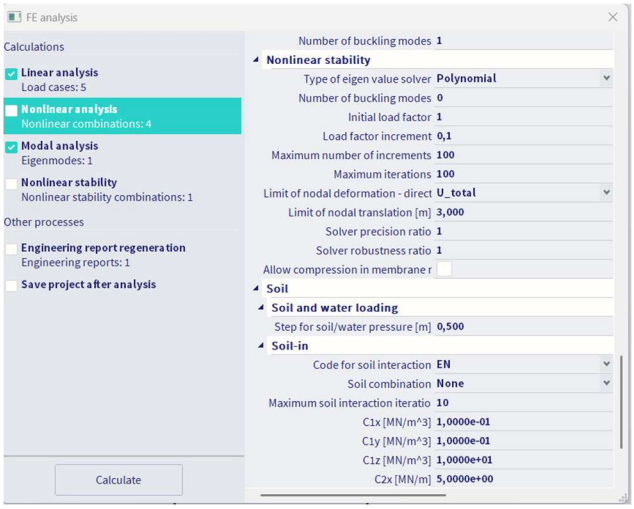
## Calculation properties:




*Figures 33, 34, 35, 36 and 37 calculation settings*

# Appendix D – Cable theory

To calculate the deformation of the bridge, the theory for a symmetrical loaded treadway has been used from Designing Canopy Walkways: Engineering Calculations for Building Canopy Access Systems with Cable-Supported Bridges (Bouricius, et al, 2002).

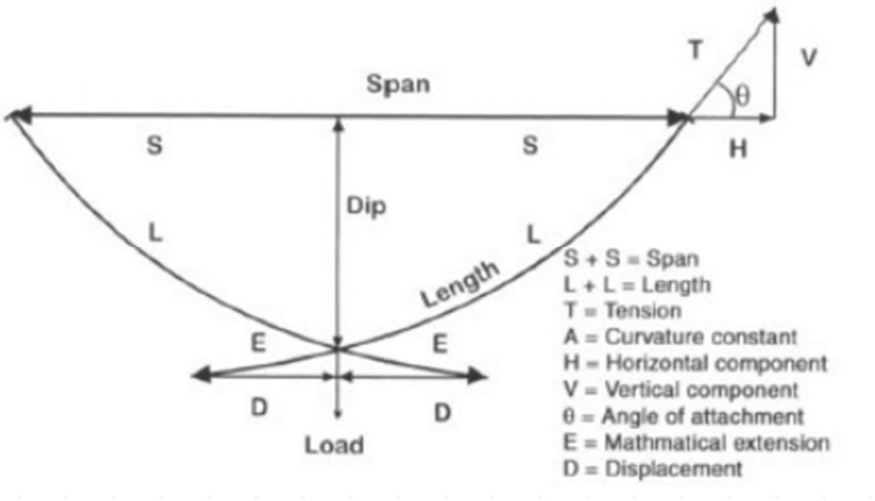



*Figure 38 scheme of model used for the deformation calculation*

Where:

$A = 590{,}9\ m$ from Informe final Puente Dosel_teoria de cables

$S = \frac{21{,}6}{2} = 10{,}8\ m$

$L = 22\ m$

$weight\ of\ plank = 0{,}20 * 0{,}45 * 0{,}04 * 1016{,}4 = 3{,}66\ kg$

$amount\ of\ planks = 54$

$weight\ of\ cable = \frac{1}{4} * \pi * 0{,}018^2 * 7850 * 21{,}6 = 43{,}15\ kg$

$Dead = \frac{1}{2} * amount\ of\ planks * weight\ of\ plank + weight\ of\ cable = 141{,}94\ kg$

$live = \frac{1}{2} * \frac{3000}{9{,}81} = 152{,}91\ kg$

$Load = Dead + Live = 294{,}85\ kg$

$E = \frac{load}{2 * Dead} = 1{,}039$

$D = A * \ln\left(\frac{E}{A} + \sqrt{1 + \left(\frac{E}{A}\right)^2}\right) = 1{,}039\ m$

$Dip = Acosh\left(\frac{S+D}{A}\right) - Acosh\left(\frac{D}{A}\right) = 0{,}118\ m$

The dip is deformation in the middle it shows a deformation of $118\ mm$.

So when there are only pedestrians on the bridge the deformation in middle of the bridge should be 118 mm downwards.

This is also calculated for load cases 0, 1, 2a and 2b in python, see Figure 8 which is a screenshot of the calculations. The dip is the deformation in the middle of the bridge. Table shows the results of the calculation.

| Load case | Deformation [mm] |
|---|---|
| 0 | 108 |
| 1 | 118 |
| 2a | 129 |
| 2b | 106 |

*Table 16 deformations calculated from designing canopy walkways.*

```
[1]: %matplotlib inline
import numpy as np
import matplotlib.pyplot as plt
import pandas as pd
import matplotlib.image as mpimg
import math
```

```
[2]: A = 590.9 # m
S = 21.6 / 2 #m
L = 22 # m
mp = 0.20 *0.45 * 0.04 * 1016.4 # weight of WPC planks [kg]
n = 54 # amount of planks
mc = 1/4 * (np.pi) * 0.018**2 * 7850 *21.6 # weight of cable [kg]
W = 0.01 # wind load [kN/m]
live = 1/2 * 3000 / 9.81 # load of two pedestrians on one cable [kg]

def cable(A, S, L, mp, n, mc, live, W):
    Fw = W * 1000 / 9.81 * L # wind load in kg
    dead = 1/2 *n* mp + mc
    load = dead + live + Fw
    E = load/ (2 * dead)
    D = A * math.log(E/A + np.sqrt(1 + (E/A)**2))
    dip = A * math.cosh((S + D)/A) - A * math.cosh(D/A)
    results = []
    results.append([A, S, L, mp, n, mc, dead, live, load, E, D, dip, W])
    df = pd.DataFrame(results, columns=['A', 'S', 'L', 'mp', 'n', 'mc', 'dead', 'live', 'load', 'E', 'D', 'dip', 'W'])
    return df


cable(A, S, L, mp, n, mc, live=0, W=0) # load case 0
```

[2]:

| | A | S | L | mp | n | mc | dead | live | load | E | D | dip | W |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 590.9 | 10.8 | 22 | 3.65904 | 54 | 43.147764 | 141.941844 | 0 | 141.941844 | 0.5 | 0.5 | 0.107839 | 0 |

```
[3]: cable(A, S, L, mp, n, mc, live, W=0) # load case 1
```

[3]:

| | A | S | L | mp | n | mc | dead | live | load | E | D | dip | W |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 590.9 | 10.8 | 22 | 3.65904 | 54 | 43.147764 | 141.941844 | 152.905199 | 294.847043 | 1.038619 | 1.038619 | 0.117684 | 0 |

```
[4]: cable(A, S, L, mp, n, mc, live, W=0.08) # load case 2a vertical wind load
```

[4]:

| | A | S | L | mp | n | mc | dead | live | load | E | D | dip | W |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 590.9 | 10.8 | 22 | 3.65904 | 54 | 43.147764 | 141.941844 | 152.905199 | 474.25581 | 1.670599 | 1.670597 | 0.129236 | 0.08 |

```
[5]: cable(A, S, L, mp, n, mc, live, W=-0.08) # load case 2b only vertical
```

[5]:

| | A | S | L | mp | n | mc | dead | live | load | E | D | dip | W |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 590.9 | 10.8 | 22 | 3.65904 | 54 | 43.147764 | 141.941844 | 152.905199 | 115.438277 | 0.406639 | 0.406639 | 0.106132 | -0.08 |

*Figure 39 screenshot of calculations of the deflection with cable theory*

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