DESIGN GUIDE FOR
TIMBER-CONCRETE COMPOSITE FLOORS IN CANADA

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FPInnovations

PRESENTATION OUTLINE

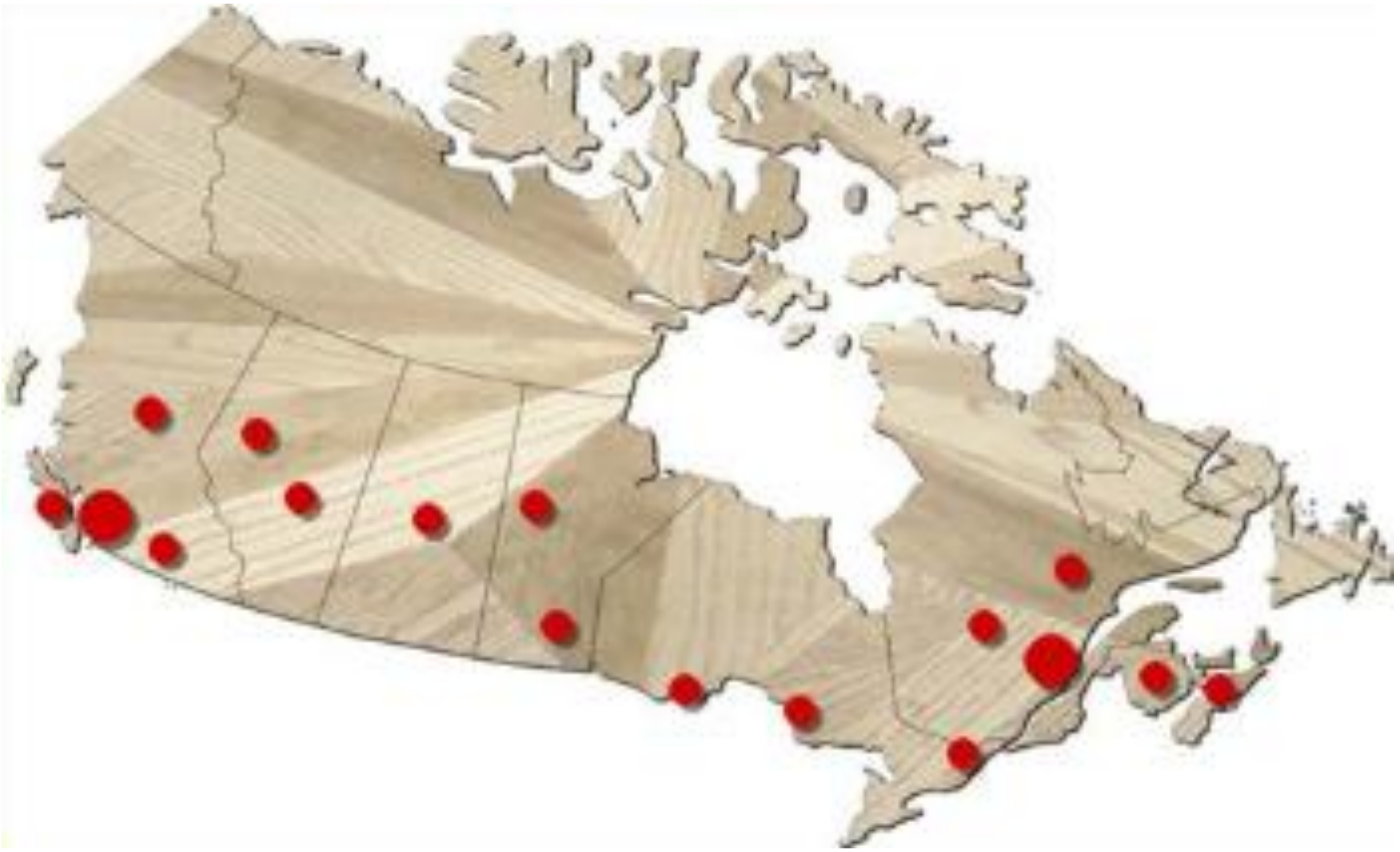
- FPInnovations
- What is a timber-concrete composite floor
- Fundamental theory
- Shear connectors
- Serviceability limit states
- Ultimate limit states
- Fire resistance

FPINNOVATIONS – WHO WE ARE

- FPInnovations is a private not-for-profit research center that specializes in the creation of solutions in support of the Canadian forest sector's global competitiveness

- +/-400 employees

- Forest Operations
- Pulp and Paper
- Wood Products
- Bio-sourced Products



1

What is Timber-Concrete Composite Floor

WHAT IS A TIMBER-CONCRETE COMPOSITE FLOOR?

- Two distinct layers of material
  - Concrete on the top (standard, lightweight, ultra-high performance, etc.)
  - Wood (Glulam, CLT, NLT, SCL, etc.)
- Materials are connected together with shear connectors
- Wood acts as reinforcement to the concrete
- When well-connected together, the capacity of the floor can be tripled and the stiffness can be six times higher compared to a traditional timber floor

WHAT IS A TIMBER-CONCRETE COMPOSITE FLOOR?

- LSL-Concrete floor tested at FPInnovations in support to the ESB building on UBC Campus

Test performed at FPInnovations laboratory - Vancouver

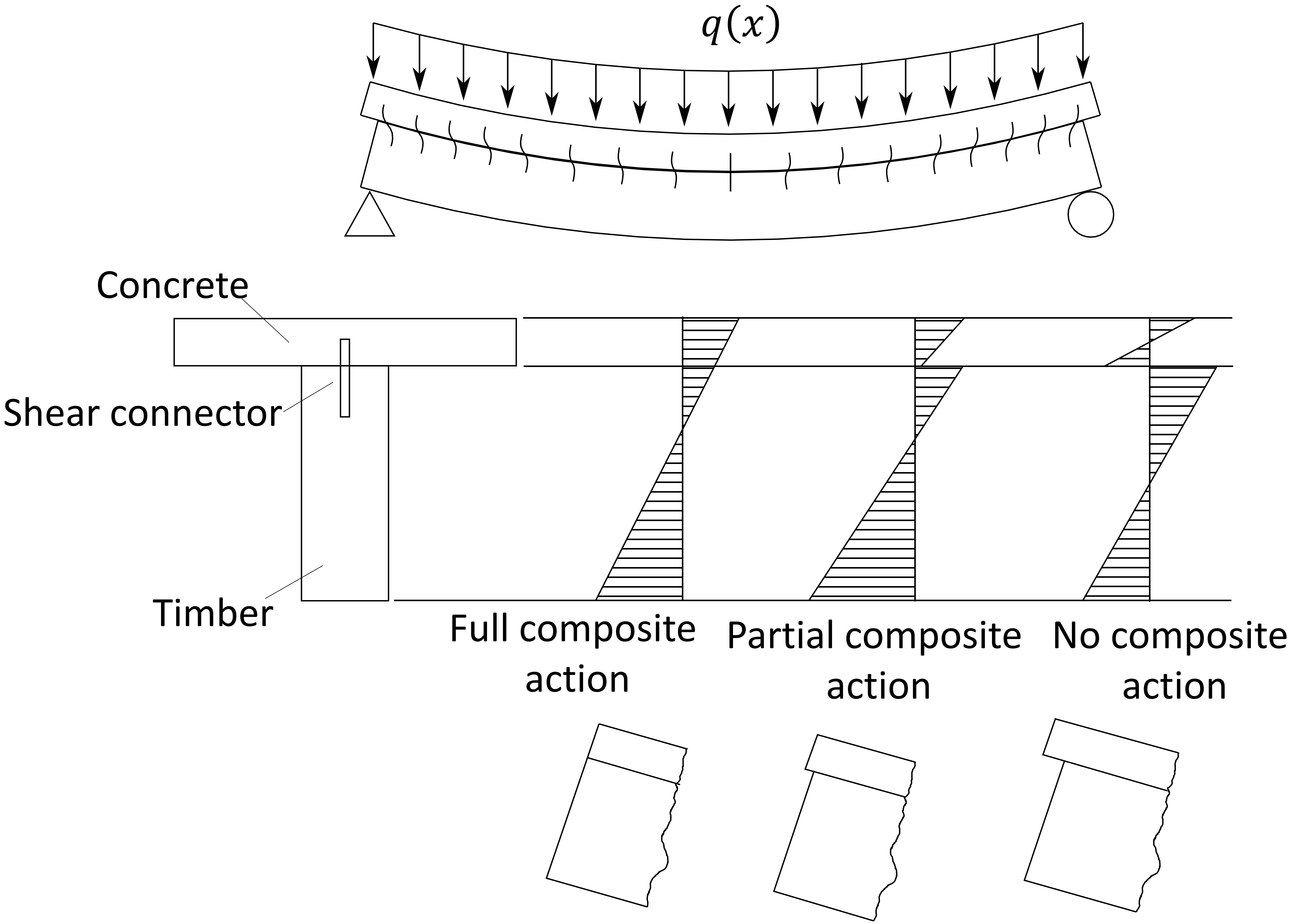
2

Fundamental Theory of Composite Beam

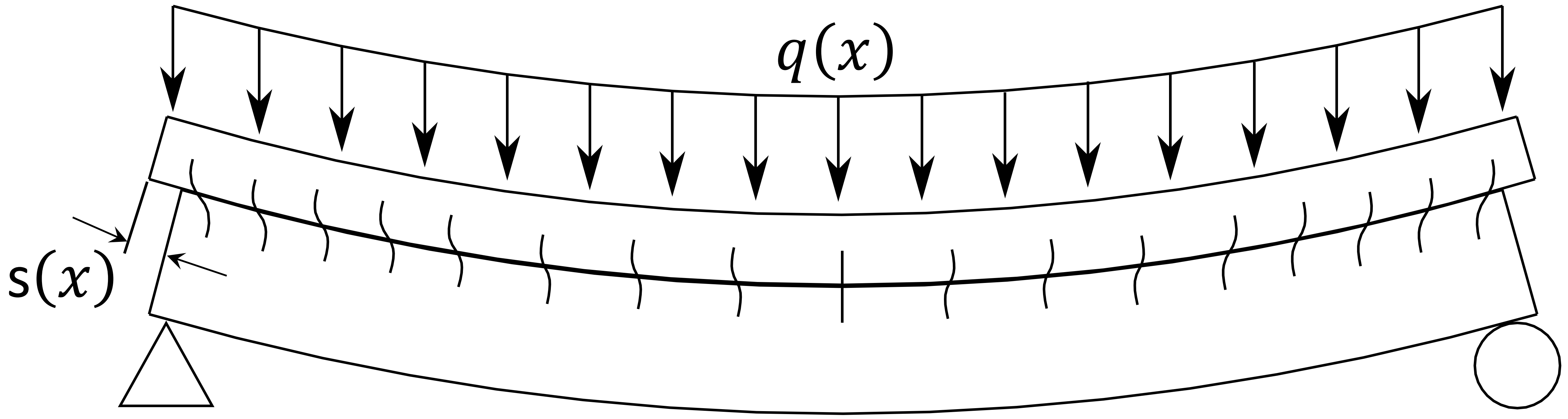
FUNDAMENTAL THEORY OF COMPOSITE BEAM

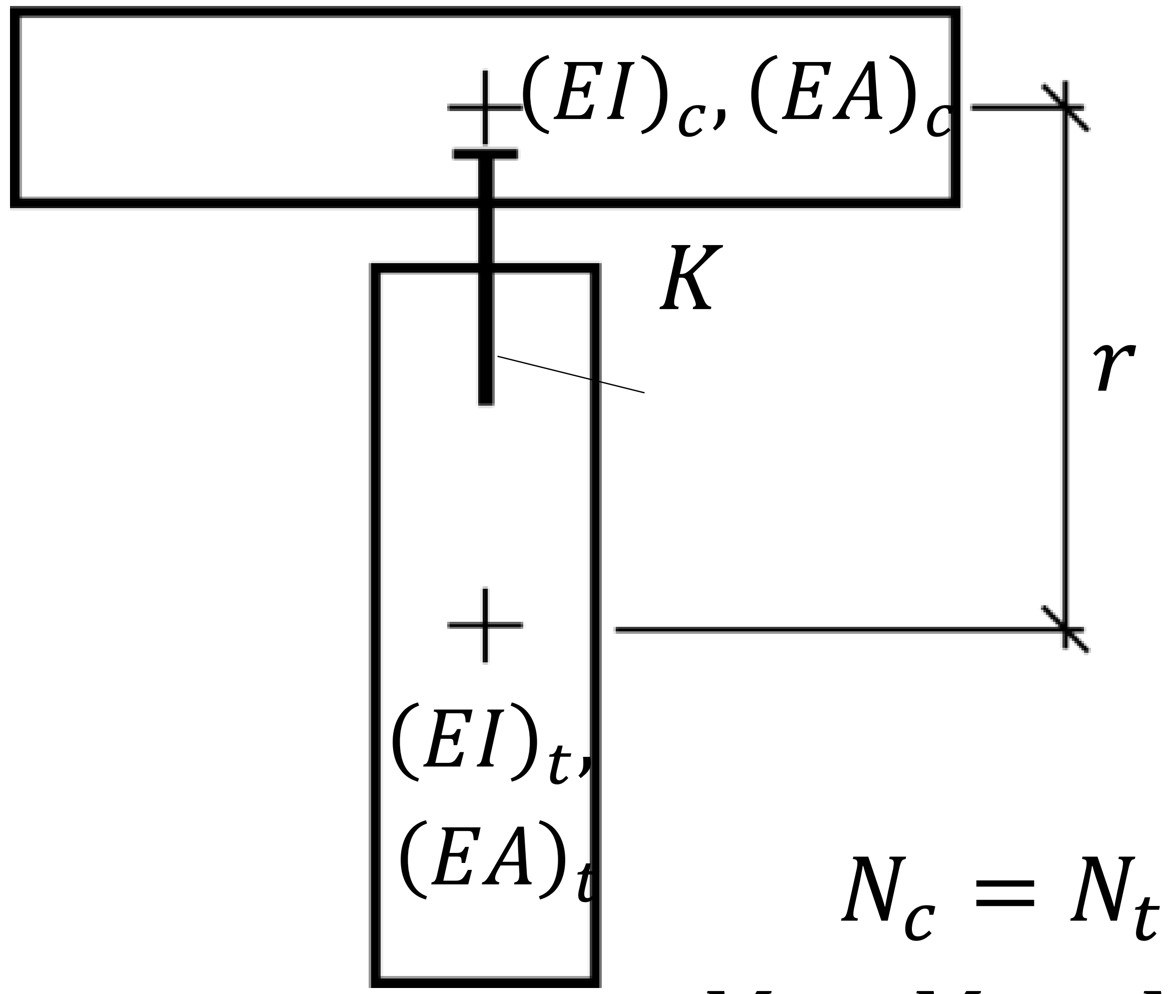
- Developed in the 1950's by several researchers around the world
- Fundamental theory is based on three main hypothesis:
  - Each layer is based on the Euler-Bernoulli beam theory (i.e. shear deformation of timber and concrete is neglected, but not from the shear connector)
  - Each layer have the same deflection, rotation and curvature (i.e. no vertical separation)
  - Distributed shear stiffness is assumed between each layer

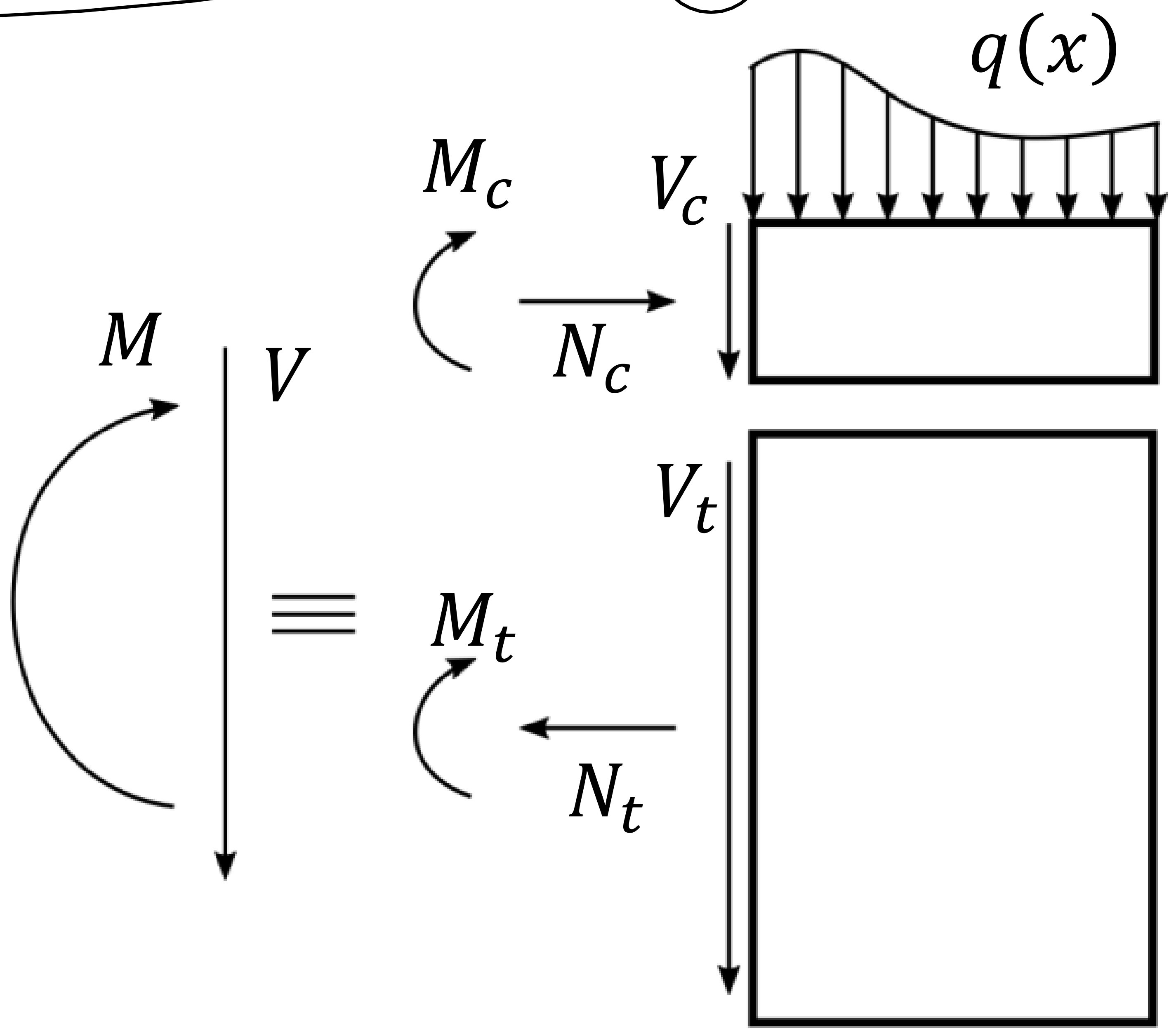
FUNDAMENTAL THEORY OF COMPOSITE BEAM



FUNDAMENTAL THEORY OF COMPOSITE BEAM







$$N_c = N_t = N$$

$$M = M_t + M_c + N \cdot r$$

$$\frac{M_t}{(EI)_t} = \frac{M_c}{(EI)_c}$$

$$N = \int_0^x K \cdot s(x)\, dx$$

FUNDAMENTAL THEORY OF COMPOSITE BEAM

- From the three main hypothesis and equilibrium of forces, the governing differential equations can be developed:

$$\frac{d^2}{dx^2}\left((EI)_0\frac{d^2w(x)}{dx^2}+rN(x)\right)=q(x)$$

$$\frac{d}{dx}\left(\frac{1}{K}\frac{dN(x)}{dx}\right)-\frac{1}{(EA)^*}N(x)+r\frac{d^2w(x)}{dx^2}=0$$

Where,

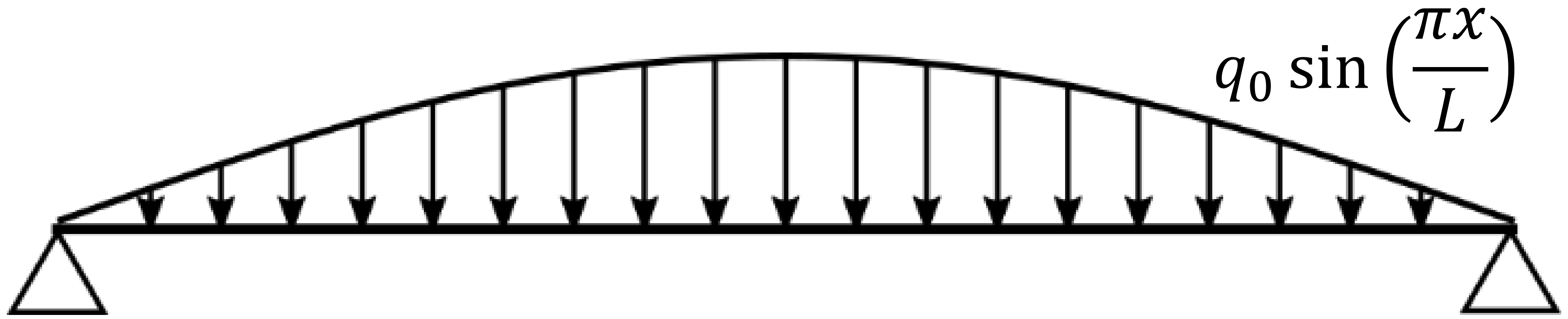
$$(EI)_0=(EI)_t+(EI)_c$$

$$(EA)^*=\left(\frac{1}{(EA)_t}+\frac{1}{(EA)_c}\right)^{-1}$$

$(EA)^*$: Axial stiffness parameter
$(EI)_0$: Bending stiffness without composite action
K: Distributed shear stiffness
N: Axial force on each layer
q: Applied load on the composite beam
w: Deflection
x: Position along the span

FUNDAMENTAL THEORY OF COMPOSITE BEAM

- Few years later, Möhler (1956) developed a simple equation to estimate the effective bending stiffness known as the **gamma (γ) method**



$$q(x) = q_0 \sin\left(\frac{\pi x}{L}\right) \qquad w(x) = C_1 \sin\left(\frac{\pi x}{L}\right) \qquad N(x) = C_2 \sin\left(\frac{\pi x}{L}\right)$$

- By substituting q, w and N in the governing equations:

$$(EI)_{eff} = (EI)_0 + \frac{1}{1 + \frac{\pi^2 (EA)^*}{KL^2}} (EA)^* r^2$$

FUNDAMENTAL THEORY OF COMPOSITE BEAM

- Equation developed by Möhler is now adopted in the Eurocode 5 (EN1995-1-1) and it is known as the γ-method

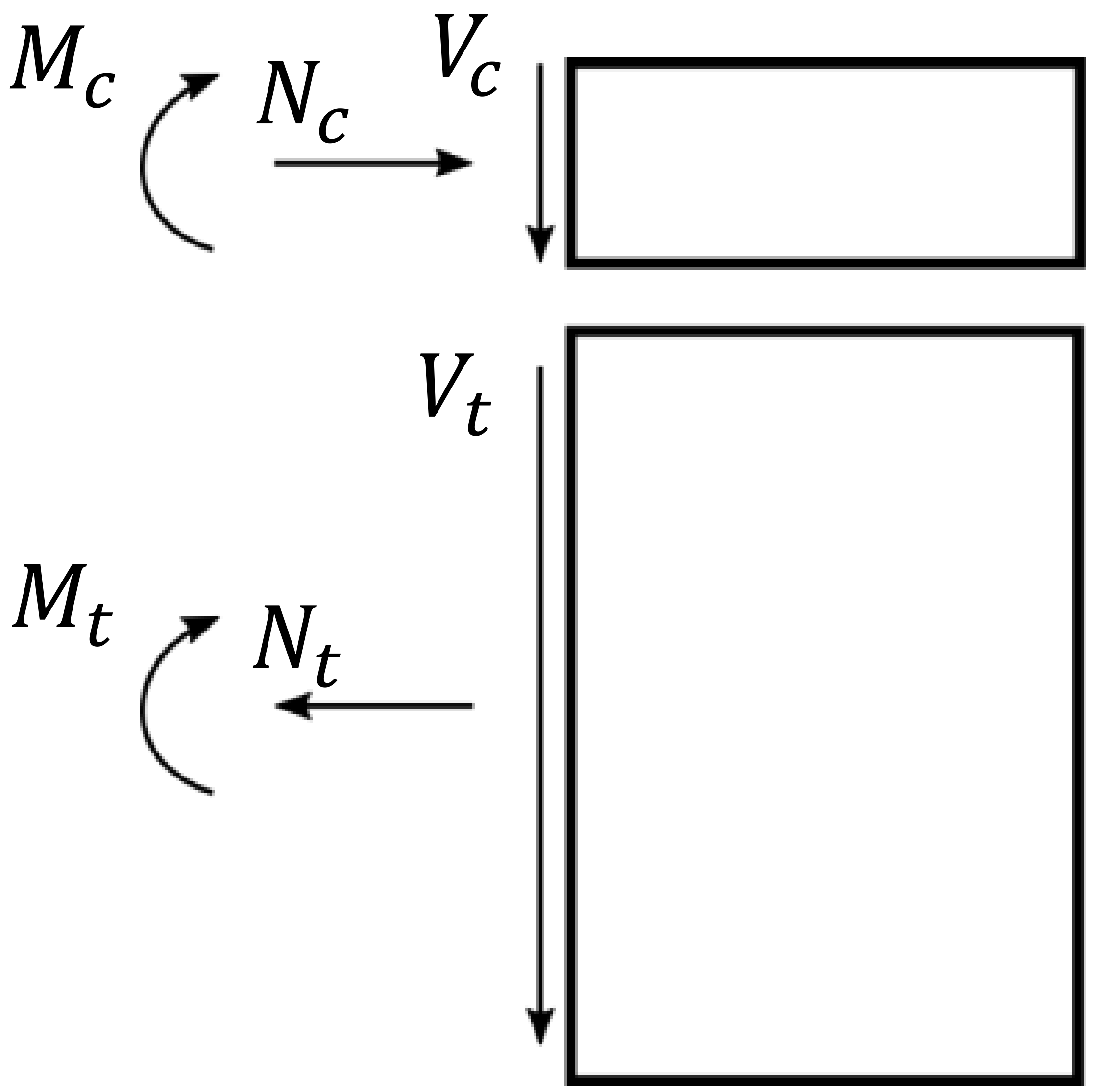
$$(EI)_{eff} = (EI)_c + (EI)_t + \gamma_c (EA)_c a_c^2 + \gamma_t (EA)_t a_t^2$$

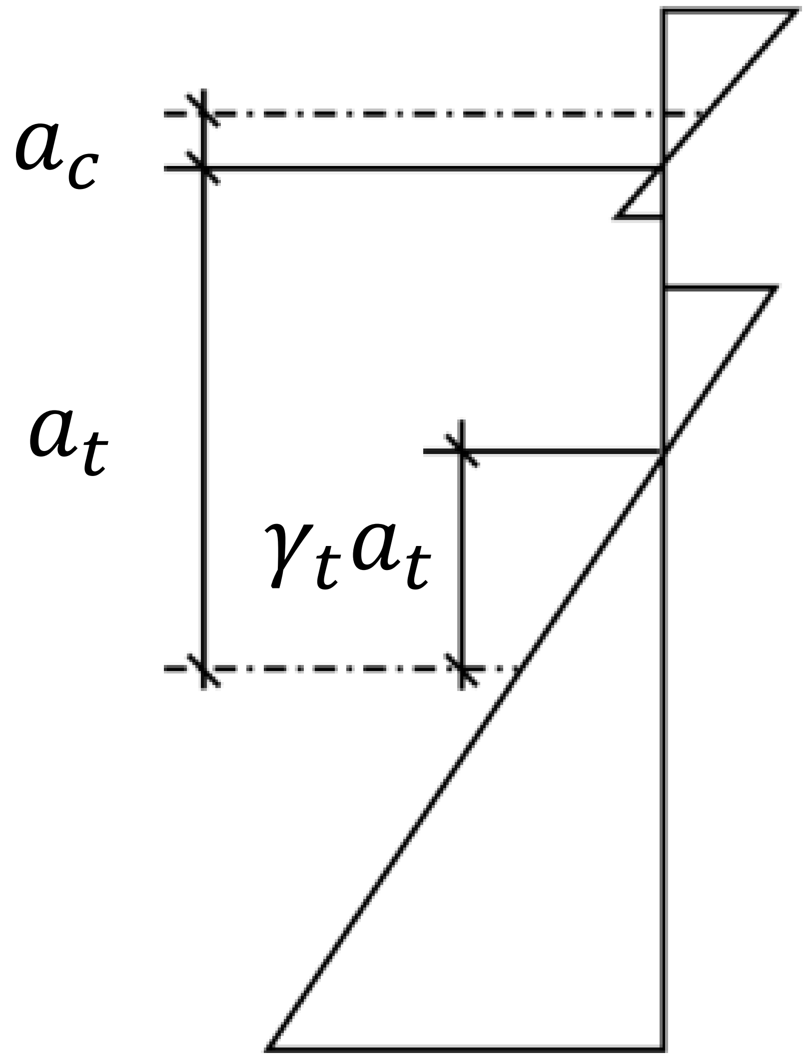
$$\gamma_c = 1$$

$$\gamma_t = \frac{1}{1 + \frac{\pi^2 (EA)_t}{K \cdot L^2}}$$

$$a_c = \frac{\gamma_t (EA)_t r}{\gamma_c (EA)_c + \gamma_t (EA)_t}$$

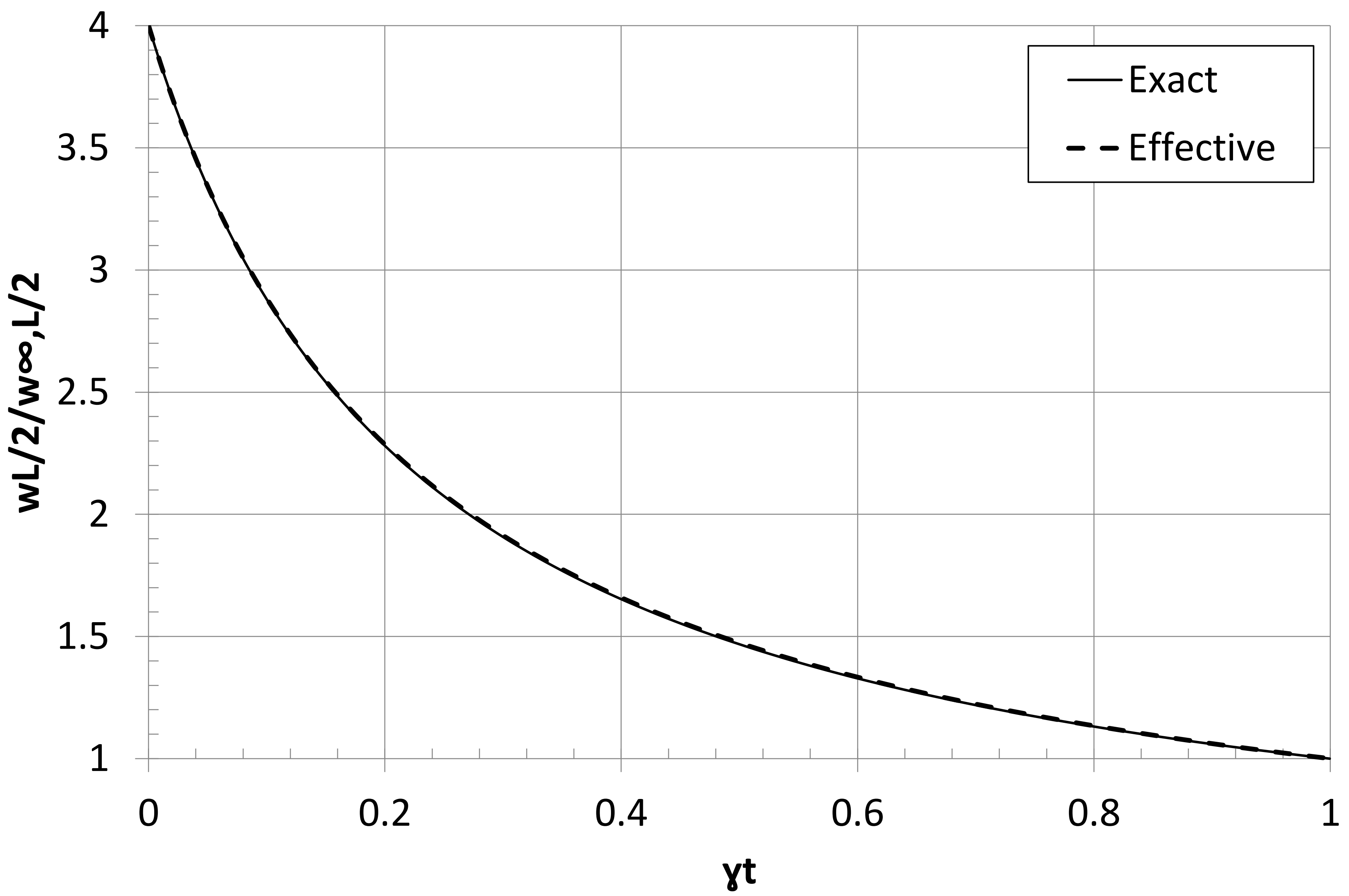
$$a_t = \frac{\gamma_c (EA)_c r}{\gamma_c (EA)_c + \gamma_t (EA)_t} = r - a_c$$



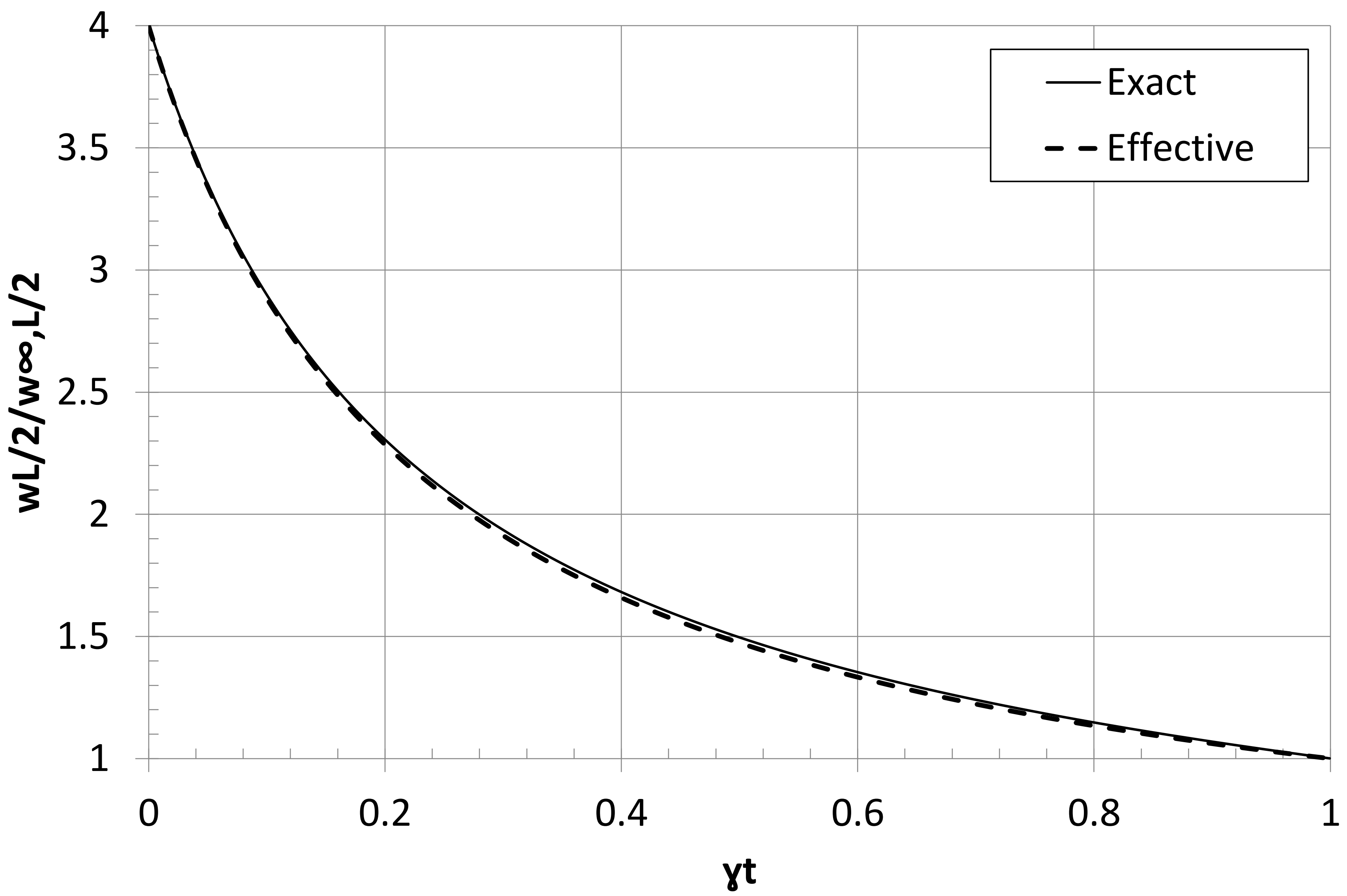


EFFECT OF LOAD CONFIGURATION

- γ-method is only exact when the load is sinusoidal and the spacing of the connector is constant and not too large (lesser than 1 m).
- However, the effect of the load has a negligible effect on the effective bending stiffness



Uniformed loading



Center point loading

EFFECT OF THE SPACING OF THE CONNECTOR

- Spacing of the connectors is considered for the calculation of γ_t

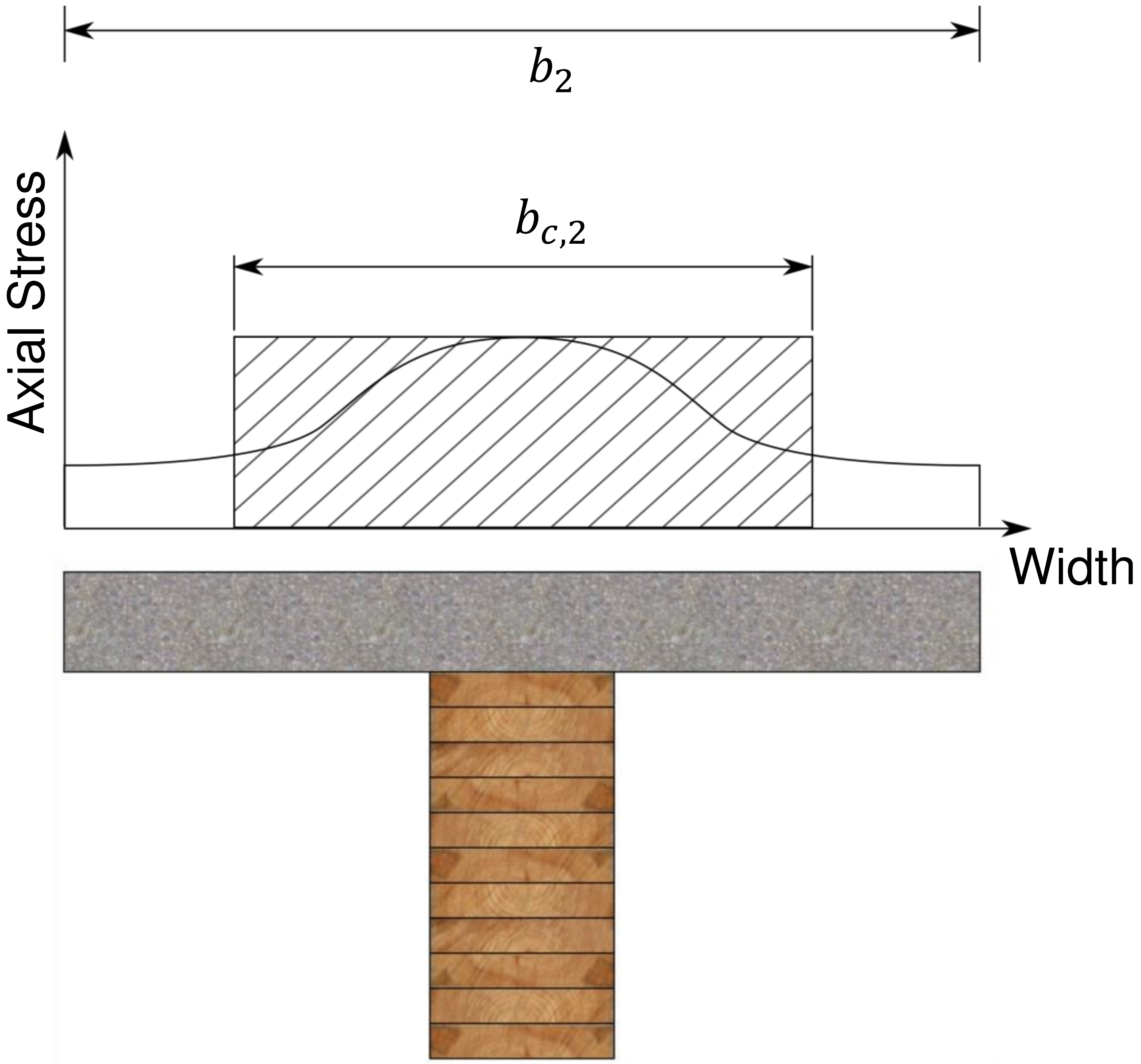
$$\gamma_t = \frac{1}{1 + \frac{\pi^2 (EA)_t}{K \cdot L^2}}$$

Where $K = \frac{n \cdot k}{s_{eff}}$

k is the stiffness of a single connector, n is the number of row of connectors and s_{eff} is the effective spacing of the connectors.

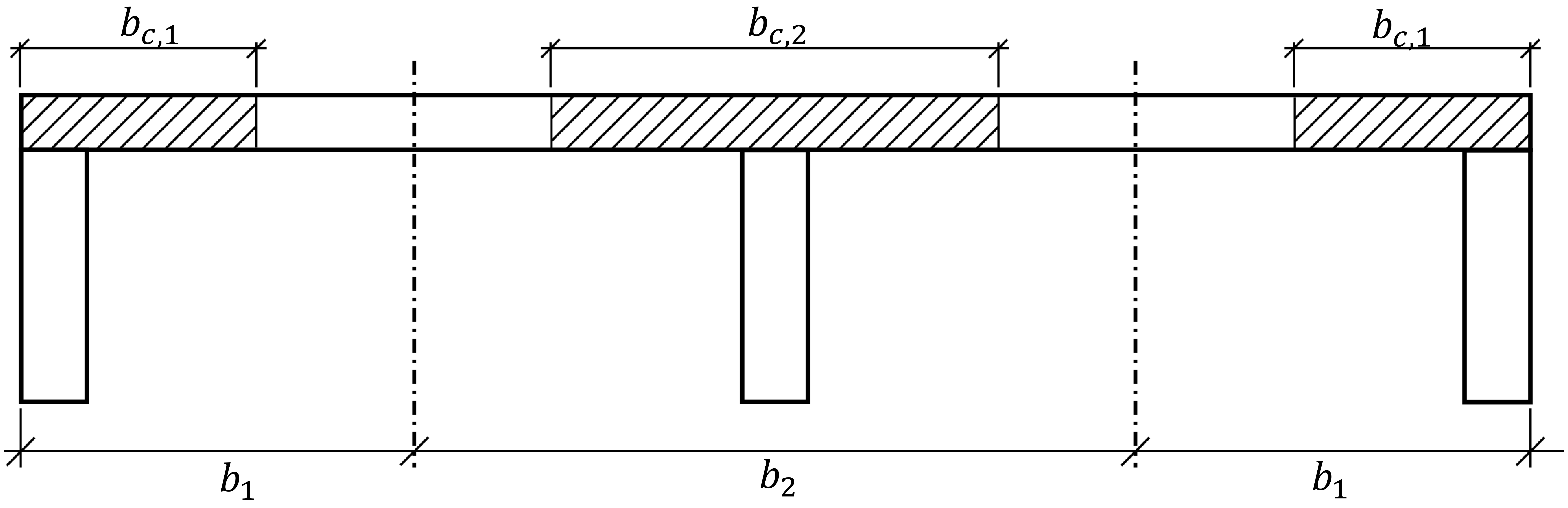
EFFECTIVE WIDTH OF THE CONCRETE SLAB

- Part of the slab that is remote from the shear connector lags behind the part that is closer
- Higher the composite action, higher is the shear lag



EFFECTIVE WIDTH OF THE CONCRETE SLAB

- Conservatively, the effective width from the CSA-S16 and CSA-A23.3 is used



$$b_{c,1} = \min(0.1L; 12h_c; b_1)$$
$$b_{c,2} = \min(0.25L; 24h_c, b_2)$$

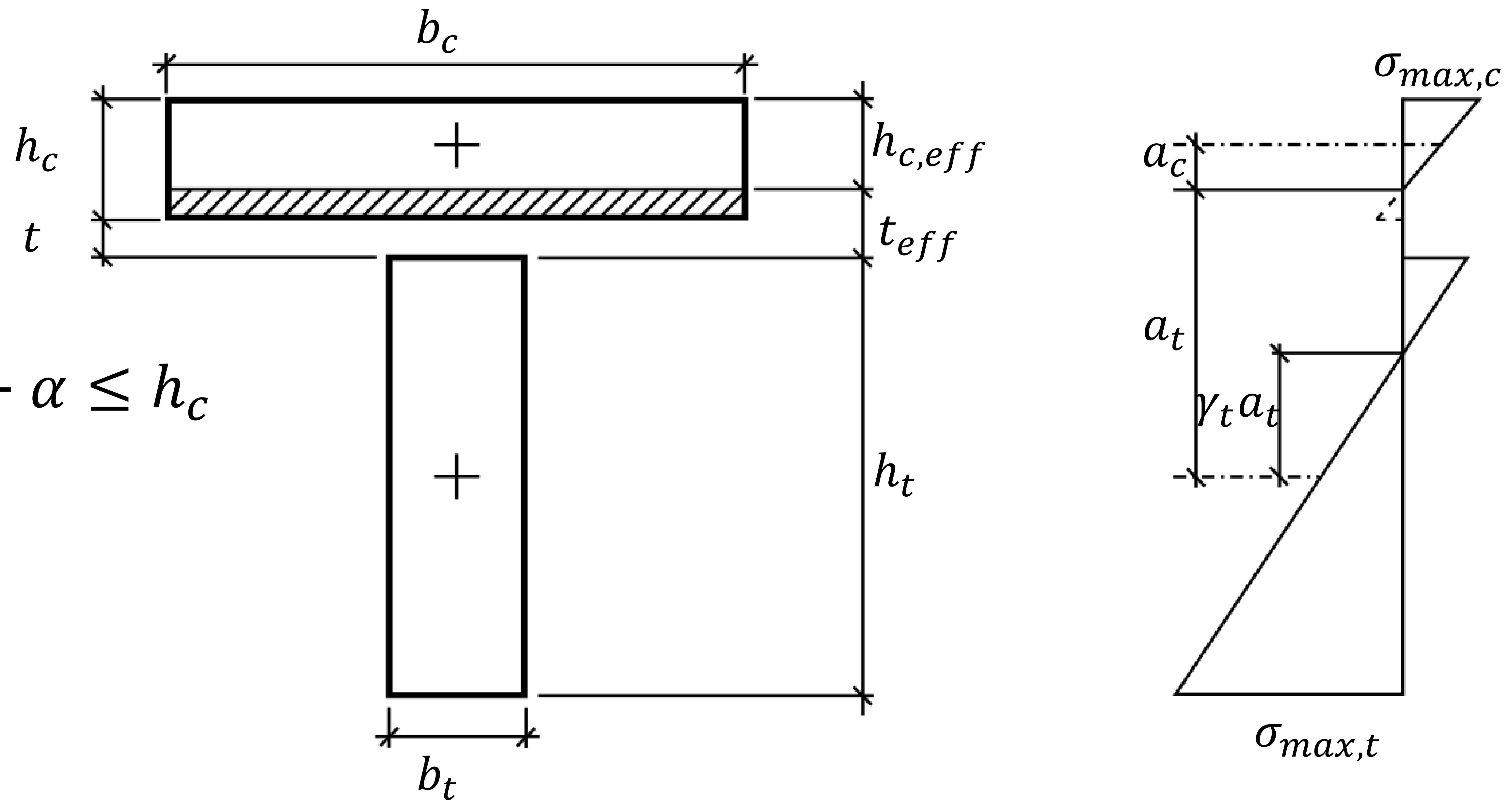
NEGLECTING THE CONCRETE IN TENSION

- With the γ-method it is possible to neglect the concrete that is in tension.
- The part of the concrete in tension is assumed as a gap

$$h_{c,eff} = \sqrt{\alpha^2 + \alpha(h_t + 2h_c + 2t)} - \alpha \leq h_c$$

$$\alpha = \frac{\gamma_t (EA)_t}{\gamma_c E_c b_c}$$

$$t_{eff} = t + h_c - h_{c,eff}$$



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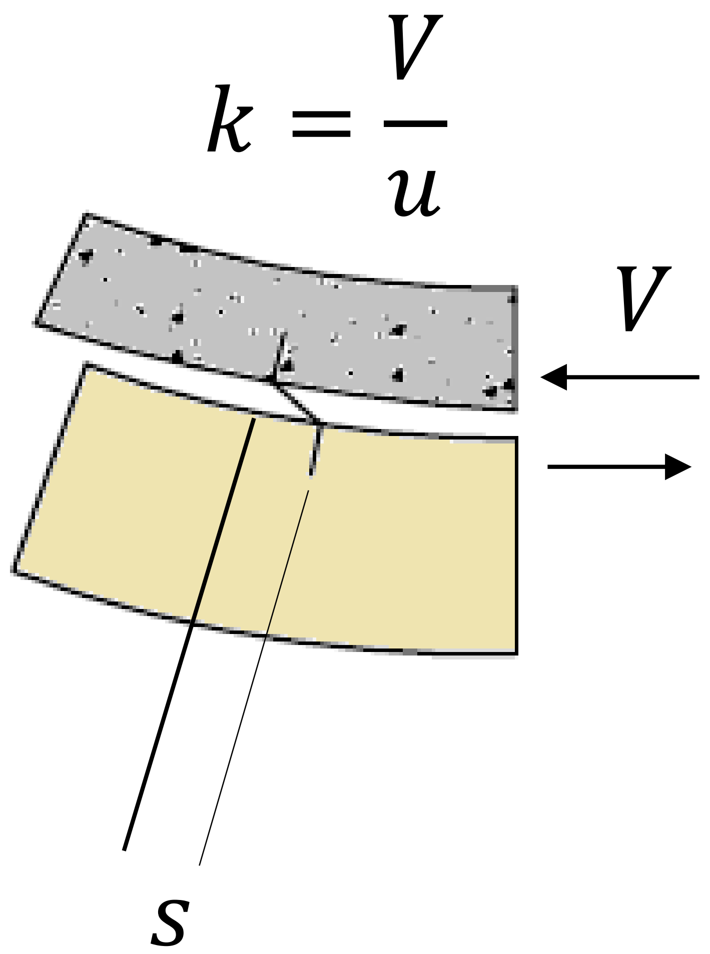
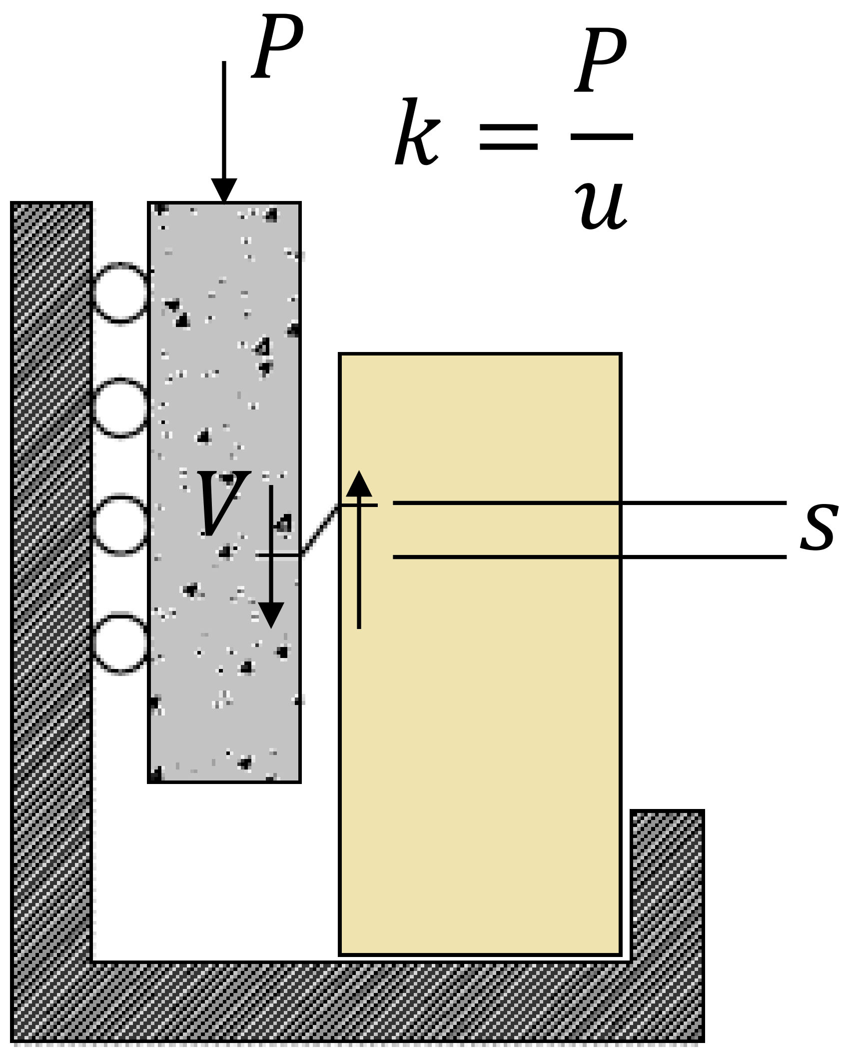
Shear Connectors

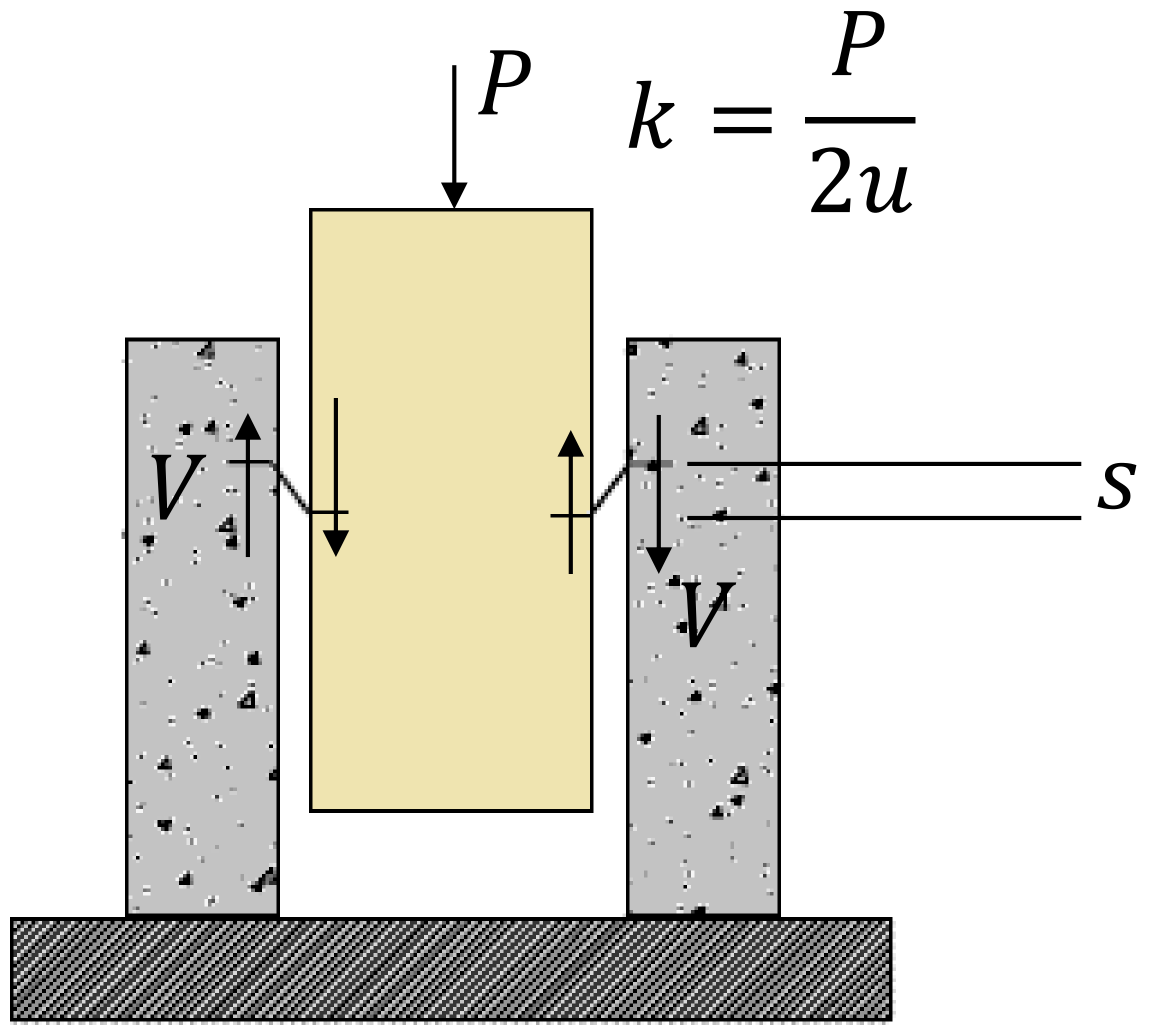
Design Guide for Timber-Concrete Composite Floor in Canada

SHEAR CONNECTORS

- Shear connectors are critical components for the timber-concrete composite floor
- It is important to well know its behaviour (stiffness, strength and ductility) during the design phase

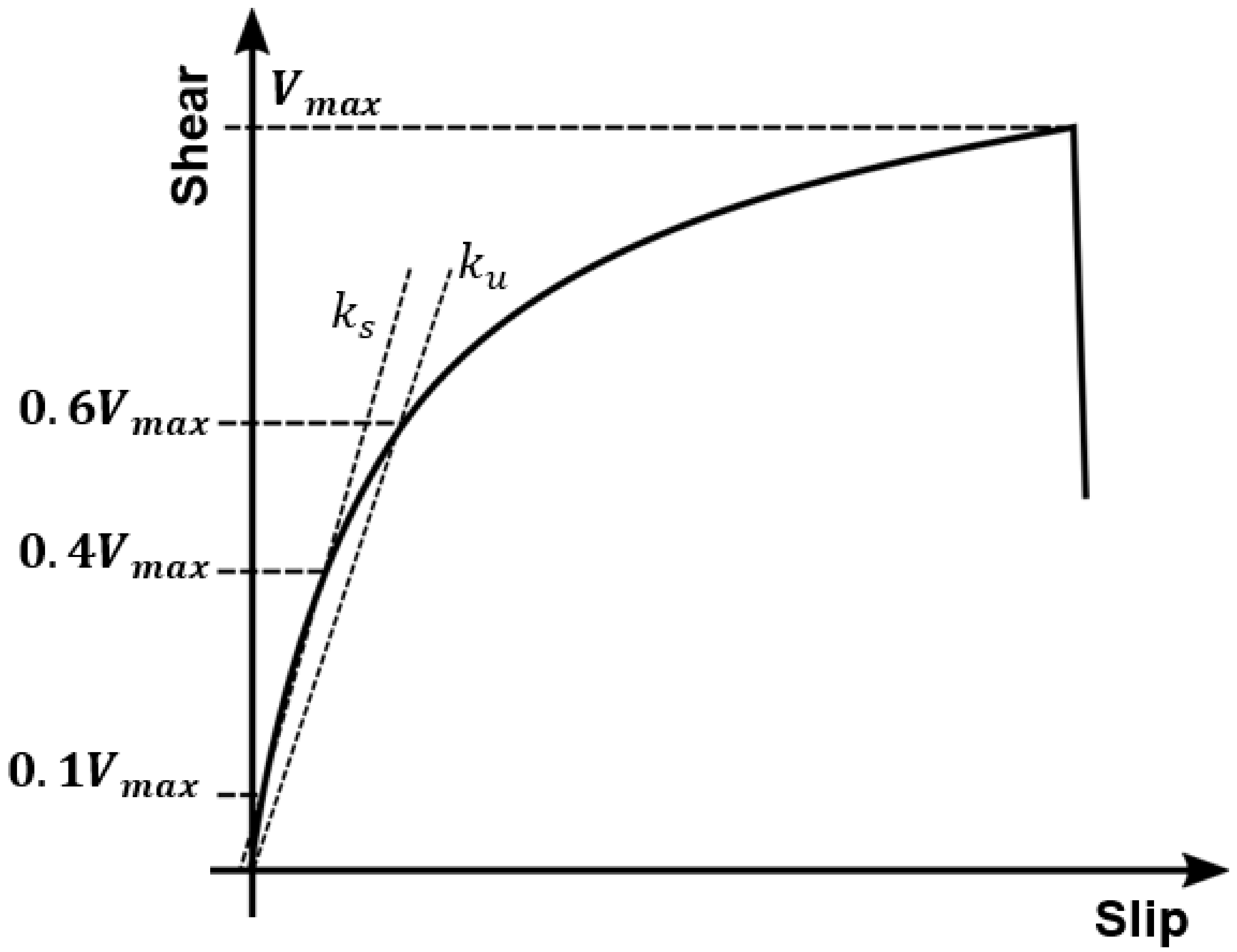


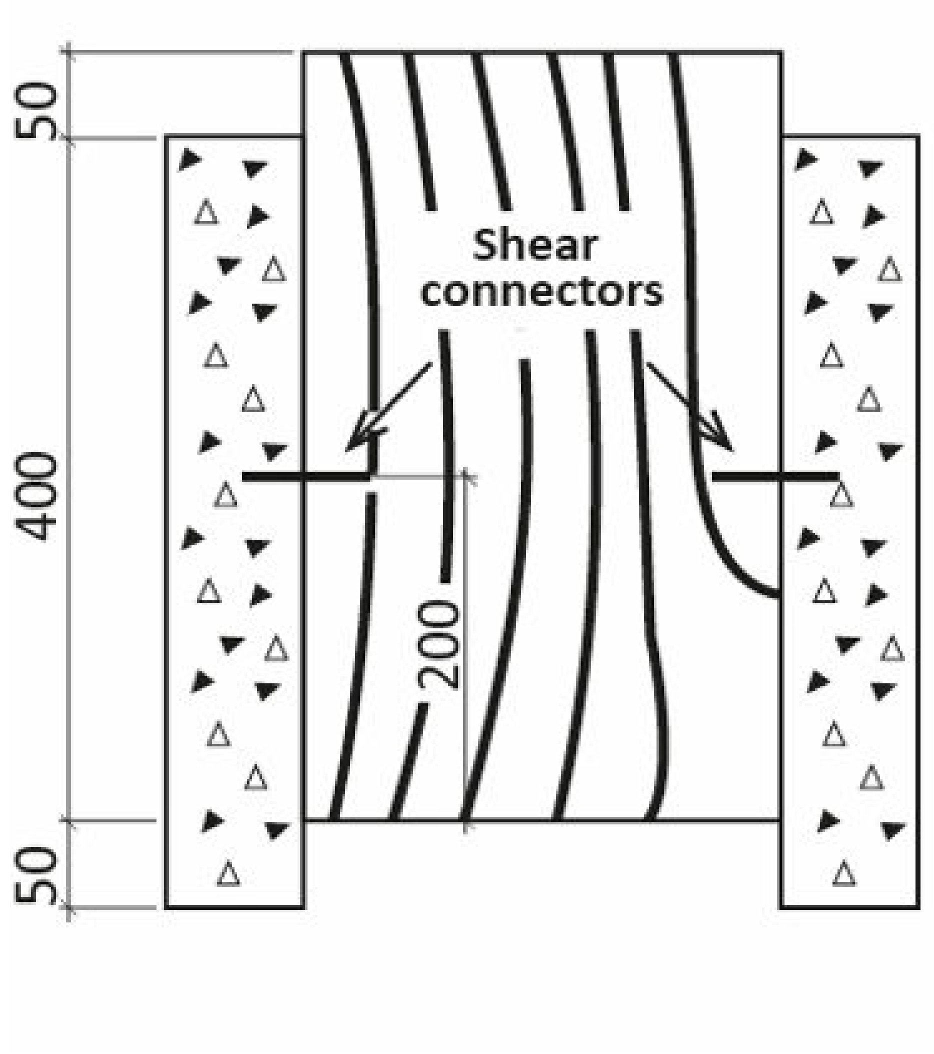


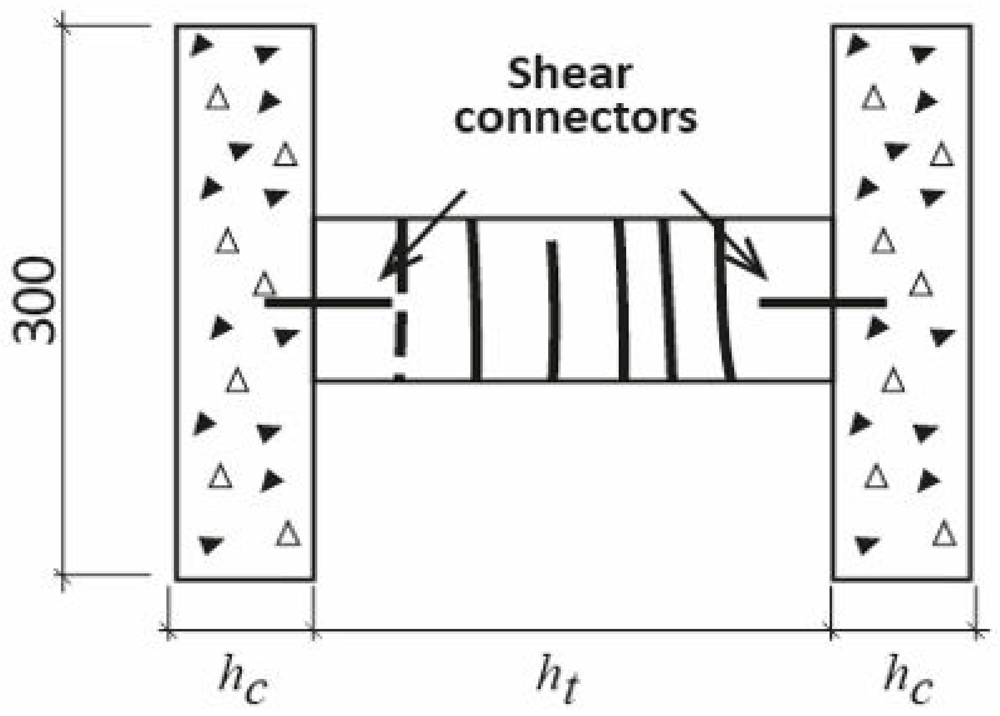


SHEAR CONNECTORS

- Shear connection test is to evaluate:
  - The stiffness
  - The strength (If possible, could be calculated with the CSA-O86)
  - Ductility







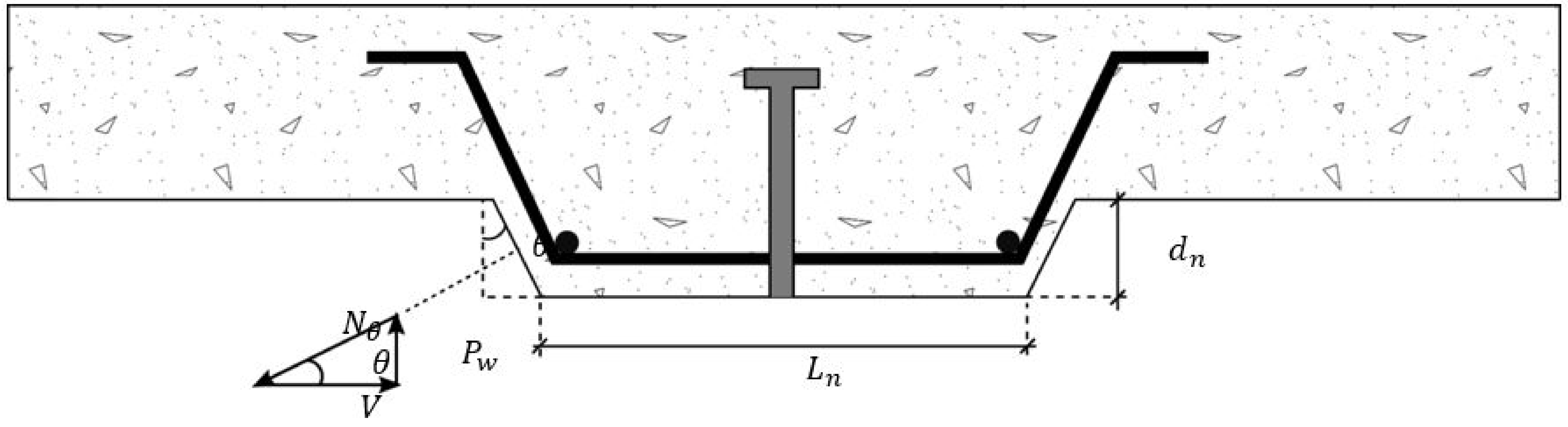
SHEAR CONNECTORS

- It exists a large variety of shear connectors on the market
- Generally, the properties are given by the manufacturer

  ⚠ Caution needs to be applied when using the value from the manufacturer, since it may not apply to your specific design
- Shear connectors can be separated in 5 families:
  1. Dowel-type fasteners
  2. Notched connectors
  3. Longitudinal connectors
  4. Glued connections
  5. Other mechanical connectors

SHEAR CONNECTORS

Truss plates



Notched connector

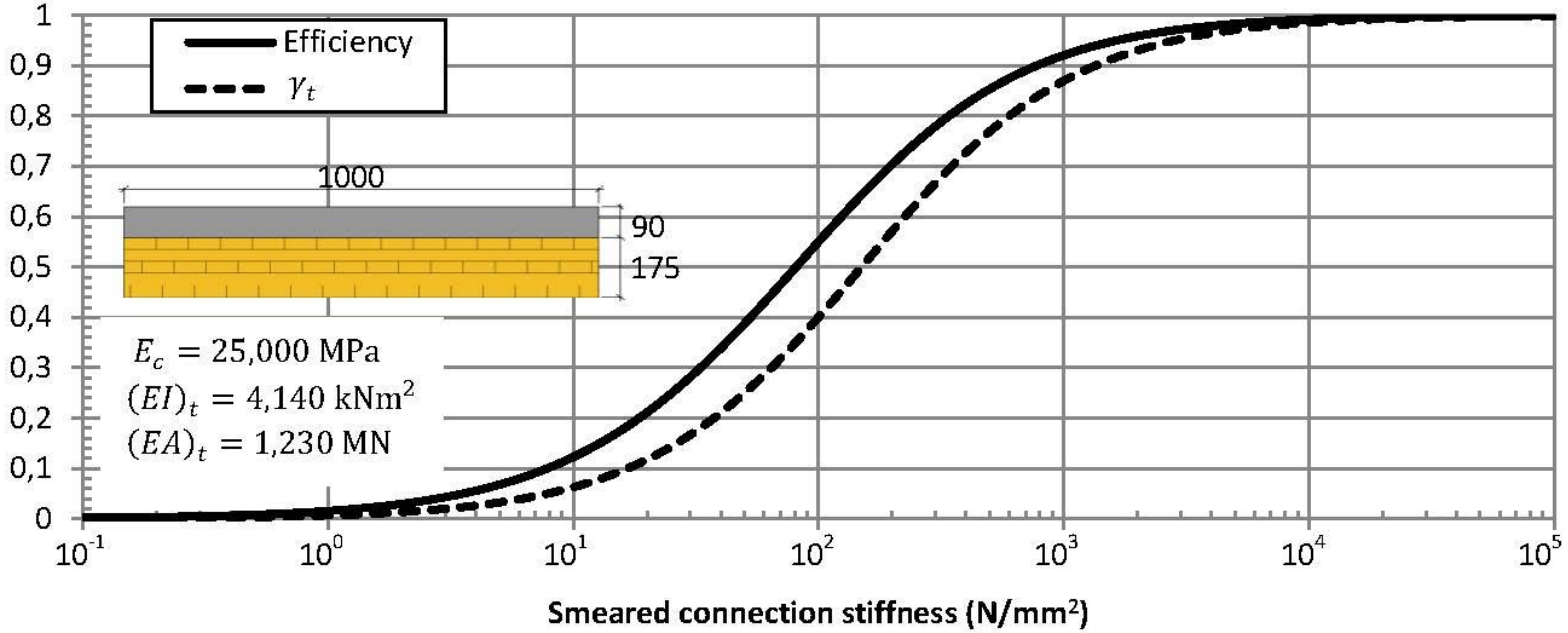
Self-tapping screws installed at 45°

Steel meshed connector (HBV)

“Benoit Gendron, Mémoire, Pont composites bois-béton en portée simple: théorie, essais et conception, Université Laval"

SHEAR CONNECTORS

- Good composite action can be achieved with several flexible connectors (e.g. self-tapping screws) of with few stiff connectors (e.g. notched connectors)
- High composite action is not always the most efficient design



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Serviceability Limit States

SERVICEABILITY LIMIT STATES – INSTANTANEOUS DEFLECTION

- Effective bending stiffness is calculated with the short-term stiffness for each material
- Maximum deflection must meet the criteria given in the National Building Code of Canada

SERVICEABILITY LIMIT STATES – LONG TERM DEFLECTION

- Few studies have evaluated the long-term deflection of timber-concrete composite floors
- Phenomenon is complex
  - Timber creeps
  - Concrete creeps
  - Shear connectors creep
  - All of them at different rate which change the strain and stress distribution through time
- Simple method is needed to promote used of TCC floor

SERVICEABILITY LIMIT STATES – LONG TERM DEFLECTION

A. M. P. G. Dias, Mechnical behaviour of timber-concrete joints, Doctoral thesis, Universidade de Coimbra, 2005

D. Yeoh, Behaviour and design of timber-concrete composite floor system, Ph.D. Thesis, Christchurch, New Zeland: University of Canterbury, 2010.

SERVICEABILITY LIMIT STATES – LONG TERM DEFLECTION

- Effective modulus method is employed to estimate the long-term deflection
  - Young modulus of each component are reduced according to their creep factor
  - For concrete without steel in the compression zone, $E_{c,LT} = \frac{E_c}{3}$
  - According to CSA-O86, creep factor for CLT is 2, the same can be assumed conservatively for other mass timber components (lower value is prescribed in the NDS)
- Creep factor for connectors vary strongly between the connection, if no values are specified by the manufacturer, a value of 4 can be conservatively assumed.

SERVICEABILITY LIMIT STATES – LONG TERM DEFLECTION

- Once all the stiffness of each material are determined, the $(EI)_{eff,LT}$ can be calculated
- Deflection is estimated with the $(EI)_{eff,LT}$ and the long term load.
- According to the Eurocode 0 (EN 1990), 30% of the live load could be assumed for residential and office area for the long term load.

SERVICEABILITY LIMIT STATES – VIBRATION

- Three different types of timber-concrete composite floors (CLT-Concrete, NLT-Concrete and GLT-Concrete) were evaluated at the FPInnovations laboratory to define the vibration criteria caused by human walking
- There are two types of dynamic responses caused by a dynamic load:
  - Resonance
  - Transitory
- When a floor has a frequency higher than 8 Hz, the predominant response is generally transitory
  - Timber-concrete composite floors have typically a frequency below 8 Hz.

SERVICEABILITY LIMIT STATES – VIBRATION

SERVICEABILITY LIMIT STATES – VIBRATION

Subjective evaluation

SERVICEABILITY LIMIT STATES – VIBRATION

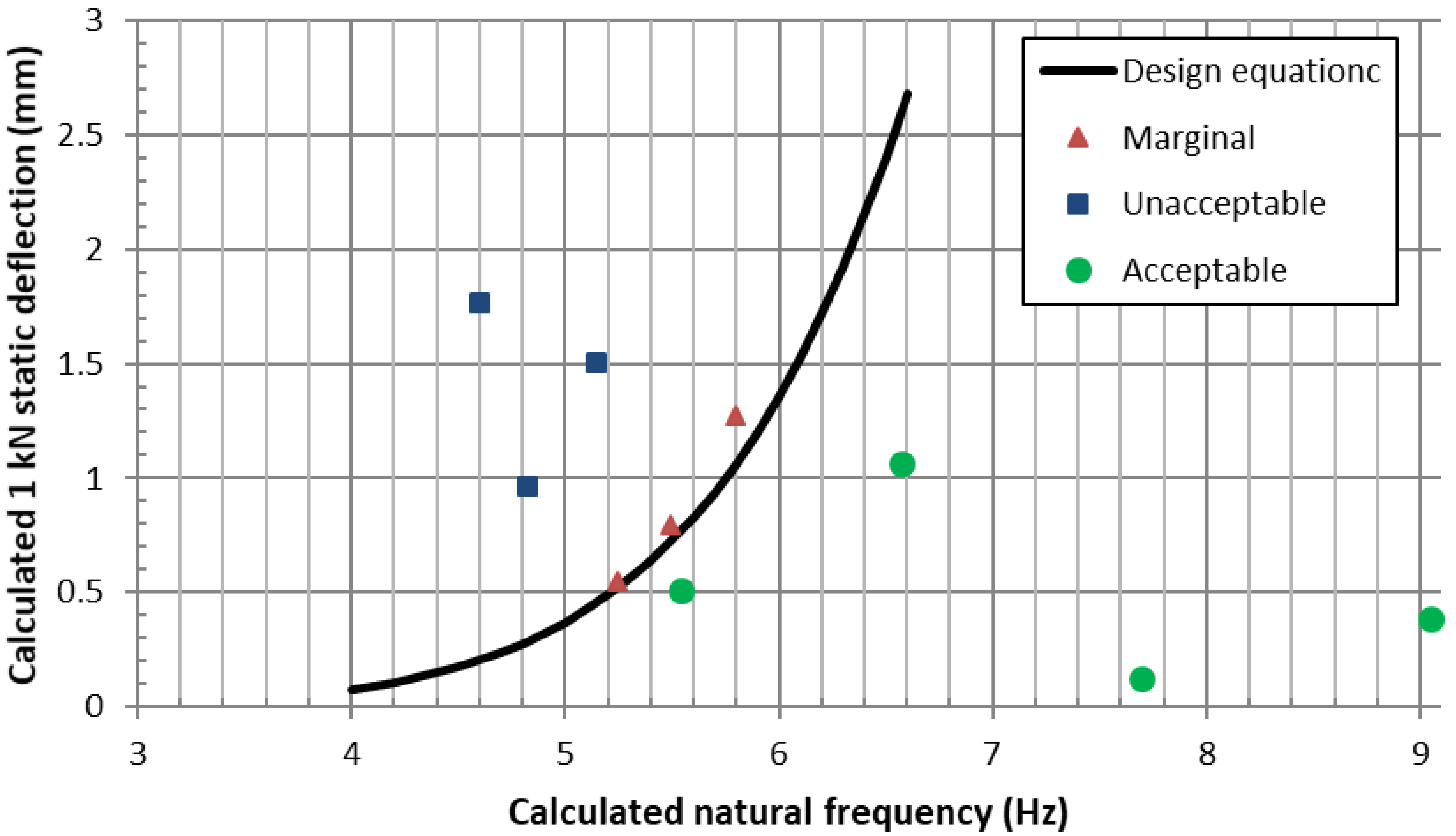
- From test results, an equation to limit the span related to the vibration criterion has been developed:
  - Method used to develop that criterion is now implemented in ISO 21136 standard.

$$L \leq 0.329 \frac{\left((EI)_{eff}^{1m}\right)^{0.264}}{m_L^{0.207}}$$

Where

m_L: is the linear mass (kg/m)

$(EI)_{eff}^{1m}$: is the effective short term bending stiffness of 1-meter floor (N-m²)



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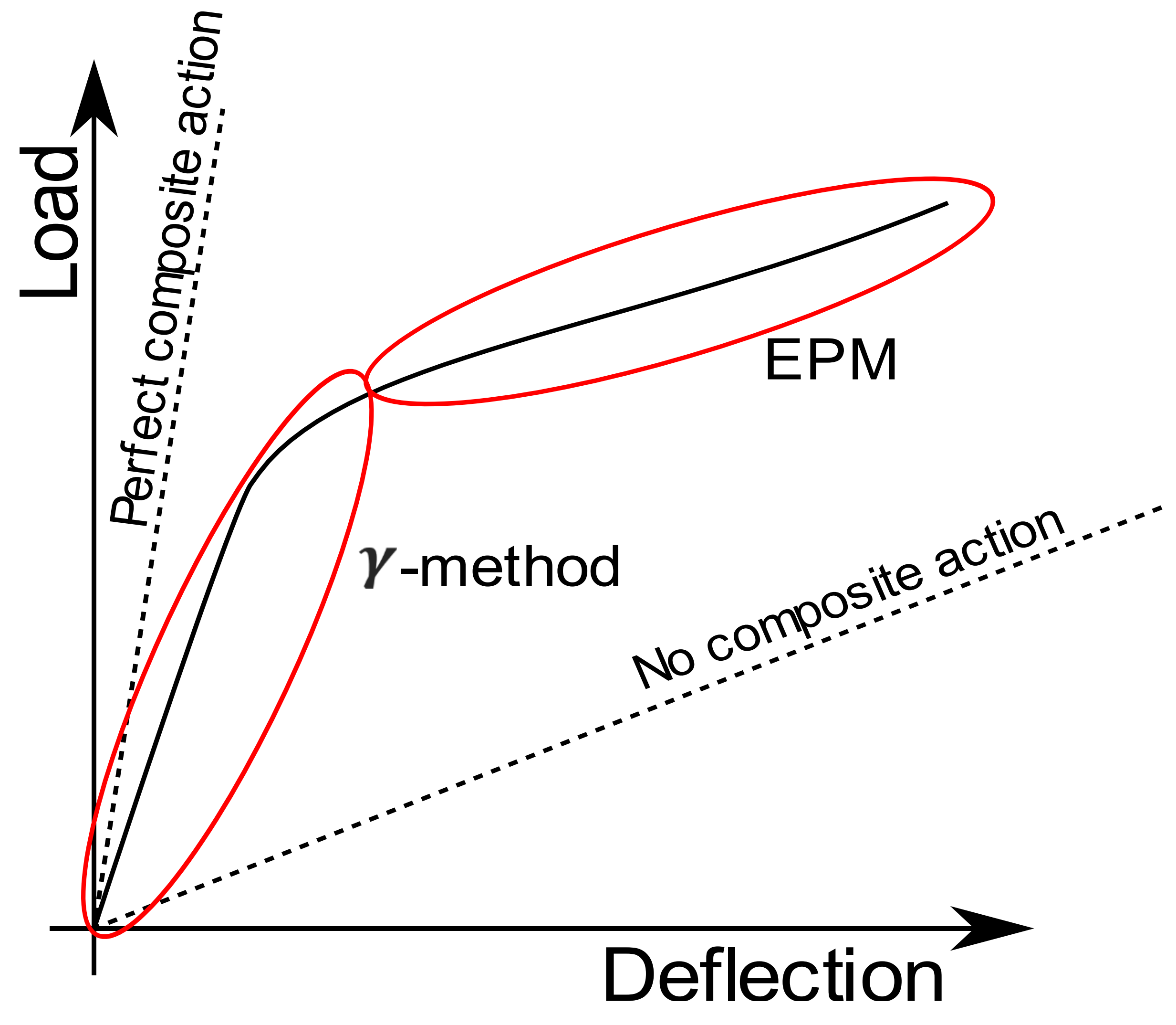
Ultimate Limit States

Design Guide for Timber-Concrete Composite Floor in Canada

ULTIMATE LIMIT STATES

- Strength of the floor must be determined as a function of the component that achieved its full strength capacity first (timber, concrete or the shear connector)
- In the design guide two scenarios are possible
  1. Shear connector is fragile or not allowed to yield (γ-method)
  2. Shear connector is ductile and allowed to yield (Elasto-plastic method)



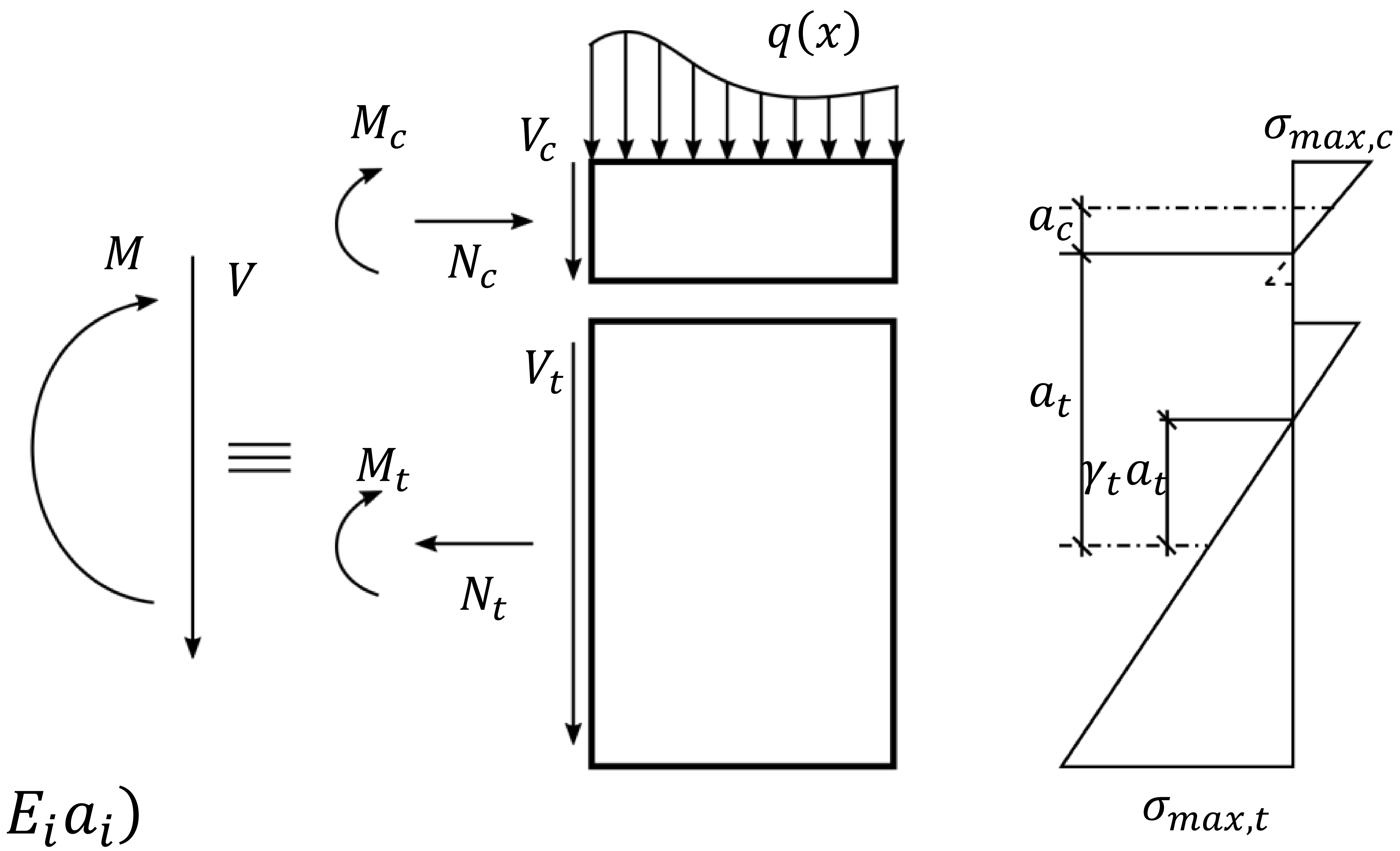
ULTIMATE LIMIT STATES – MOMENT (γ-METHOD)

- For a positive bending moment: failure either cause failure of the wood in tension or of the concrete in compression

$$N_i = \frac{\gamma_i (EA)_i a_i}{(EI)_{eff}} M \qquad M_i = \frac{(EI)_i}{(EI)_{eff}} M$$

$$\sigma_{N,i} = \frac{\gamma_i E_i a_i}{(EI)_{eff}} M \qquad \sigma_{b,i} = \frac{E_i h_i}{2(EI)_{eff}} M$$

$$\sigma_{max,i} = \sigma_{N,i} + \sigma_{b,i} = \frac{M}{(EI)_{eff}} (0.5 E_i h_i + \gamma_i E_i a_i)$$



ULTIMATE LIMIT STATES – MOMENT (γ-METHOD)

- For the concrete, $\sigma_{max,c} \leq 0.9\phi_c f_c'$ according to the clause 10.1.6 of CSA A23.3 when linear elastic behaviour is assumed.

$$M_{r,\gamma,c} = 0.9\phi_c f_c' S_c$$

$$S_c = \frac{(EI)_{eff}}{E_c\left(0.5h_{c,eff} + \gamma_c a_c\right)}$$

- For the wood, the clause regarding the interaction between the tension and the moment must be respected:

$$\frac{T_{f,t}}{T_{r,t}} + \frac{M_{f,t}}{M_{r,t}} \leq 1.0 \qquad M_{r,\gamma,t} = \frac{(EI)_{eff} T_{r,t} M_{r,t}}{\gamma_t (EA)_t a_t M_{r,t} + (EI)_t T_{r,t}}$$

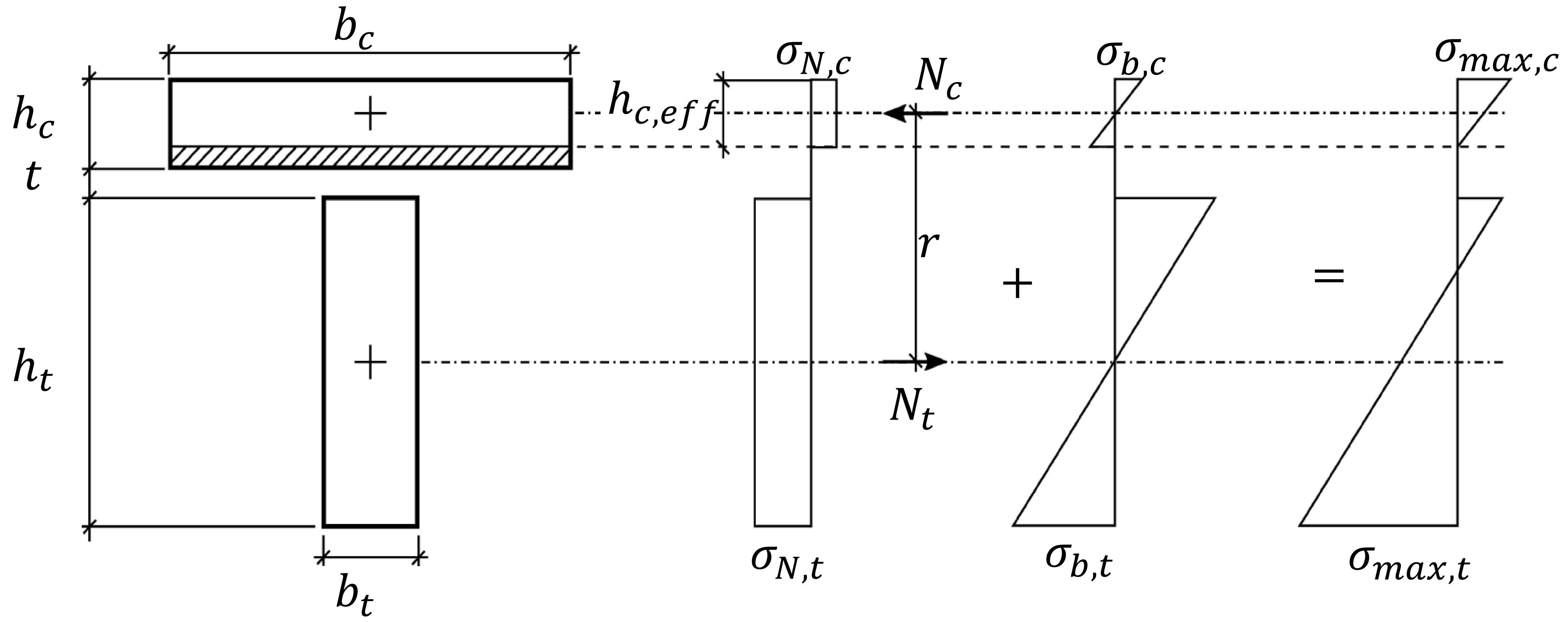
ULTIMATE LIMIT STATES - MOMENT (EPM)

- If the connector yields:

$$N_i \neq \frac{\gamma_i (EA)_i a_i}{(EI)_{eff}} M$$

$$N_i = m \cdot V_{r,conn} \leq \min\left(T_{r,t}; 0.9\phi_c b_c h_c f_c'\right)$$

- m is the number of connectors between the inflexion point and the position of the maximum moment



$$M_{r,EP} = N\left(\frac{h_t}{2} + t + h_c - \frac{h_{c,eff}}{2}\right) + \sigma_{b,c}\frac{b_c h_{c,eff}^2}{6} + \sigma_{b,t}\frac{b_t h_t^2}{6}$$

ULTIMATE LIMIT STATES - MOMENT (EPM)

- In the Elasto-Plastic Model (EPM), 4 equilibrium cinematics are possible
  - Strength limited by the timber
    - Neutral axis in the concrete
    - No neutral axis in the concrete
  - Strength limited by the concrete
    - Neutral axis in the concrete
    - No neutral axis in the concrete
- Each of the equilibrium cinematic have their own set of equations to estimate $h_{c,eff}$, $\sigma_{b,t}$ and $\sigma_{b,c}$.

ULTIMATE LIMIT STATES - MOMENT

- Bending moment resistance is:
  - $M_r = \min(M_{r,\gamma,t}; M_{r,\gamma,c})$ when the connectors are fragile or not allowed to yield
  - $M_r = \min(M_{r,\gamma,t}; M_{r,\gamma,c}; M_{r,EP})$ when the connectors are ductile and allowed to yield

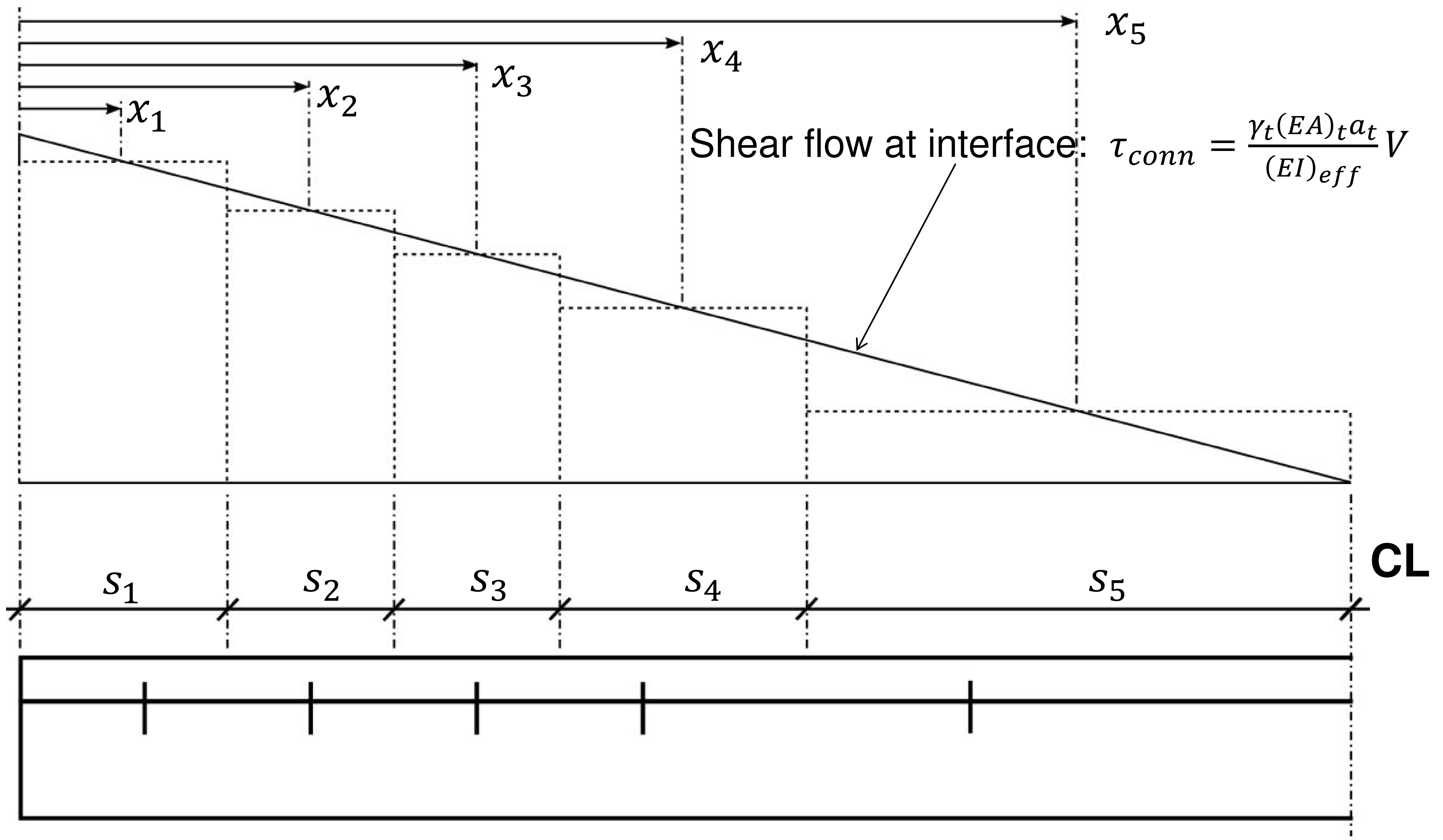
ULTIMATE LIMIT STATES – SHEAR (γ-METHOD)

- Shear capacity can be limited either by the concrete, the timber or the shear connector
  - When limited by the connector

$$\tau_{conn} = \frac{\gamma_t (EA)_t a_t}{(EI)_{eff}} V$$

$$V_{conn} = \frac{\tau_{conn} s}{n} = \frac{\gamma_t (EA)_t a_t s}{n (EI)_{eff}} V$$

$$V_{r,\gamma,conn} = \frac{n (EI)_{eff}}{\gamma_t (EA)_t a_t s} V_{r,conn}$$



ULTIMATE LIMIT STATES - SHEAR (γ-METHOD)

- When the shear capacity is limited by the wood or the concrete:

$$V_{f,i} = \frac{(EI)_i + 0.5\gamma_i (EA)_i (h_i + t) a_i}{(EI)_{eff}} V_f$$

$$V_{r,\gamma,t} = \frac{(EI)_{eff}}{(EI)_t + 0.5\gamma_t (EA)_t (h_t + t) a_t} V_{r,t}$$

$$V_{r,\gamma,c} = \frac{(EI)_{eff}}{(EI)_c + 0.5\gamma_c (EA)_c (2h_c - h_{c,eff} + t) a_c} V_{r,c}$$

ULTIMATE LIMIT STATES - SHEAR (EPM)

- If the connector is ductile and yields:

$$V_{f,conn} \neq \frac{\gamma_t (EA)_t a_t s}{n(EI)_{eff}} V_f$$

$$V_{f,i} \neq \frac{(EI)_i + 0.5\gamma_i (EA)_i (h_i + t) a_i}{(EI)_{eff}} V_f$$

- If the connector is ductile, the shear resistance is evaluated with the following equations:

$$V_{r,EP,t} = \left(V_{r,t} - \frac{mV_{r,conn}}{L_m} \frac{h_t + t}{2} \right) \frac{(EI)_0}{(EI)_t} + \frac{mV_{r,conn}}{L_m} r$$

$$V_{r,EP,c} = \left(V_{r,c} - \frac{mV_{r,conn}}{L_m} \frac{2h_c - h_{c,eff} + t}{2} \right) \frac{(EI)_0}{(EI)_c} + \frac{mV_{r,conn}}{L_m} r$$

Where L_m is the distance between the inflexion point and the critical cross-section

ULTIMATE LIMIT STATES - SHEAR

- The shear resistance is:
  - $V_r = \min(V_{r,\gamma,t}; V_{r,\gamma,c}; V_{r,\gamma,conn})$ when the connectors are fragile or not allowed to yield
  - $V_r = \min(V_{r,\gamma,t}; V_{r,\gamma,c}; V_{r,\gamma,EP}; V_{r,\gamma,EP})$ when the connectors are ductile and allowed to yield
  - When the connectors are ductile and allowed to yield, $V_{r,\gamma,conn} > V_{serv}$

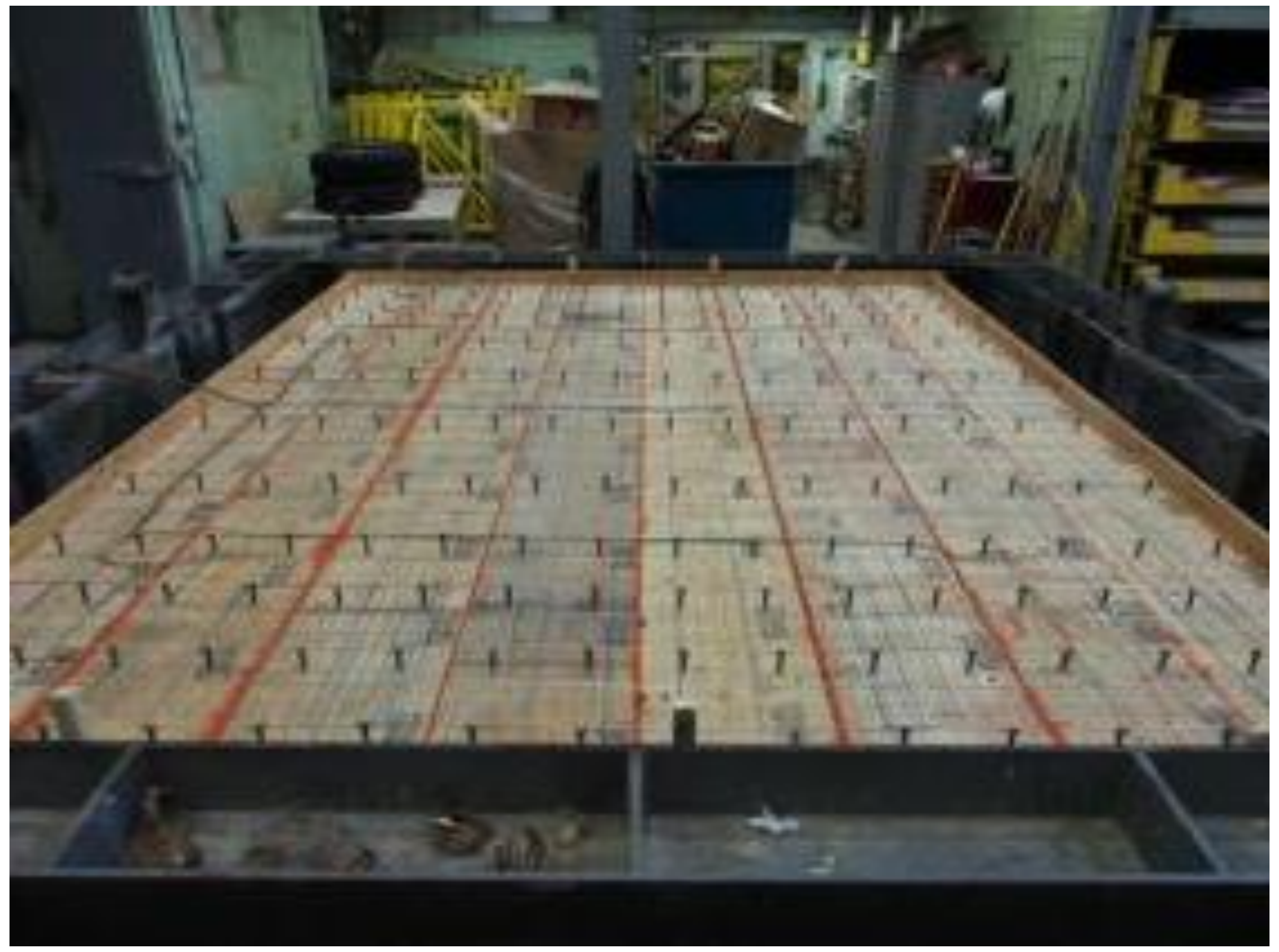
6

Fire Resistance

FIRE RESISTANCE

- 3 floors were tested at a span of 4.8 m in a full-scale furnace exposed to the CAN/ULC S101 standard fire
  - CLT-Concrete floor with self-tapping screws as shear connectors
  - NLT-Concrete floor with nail plate as shear connectors
  - LVL-Concrete floor with lag screws as shear connectors



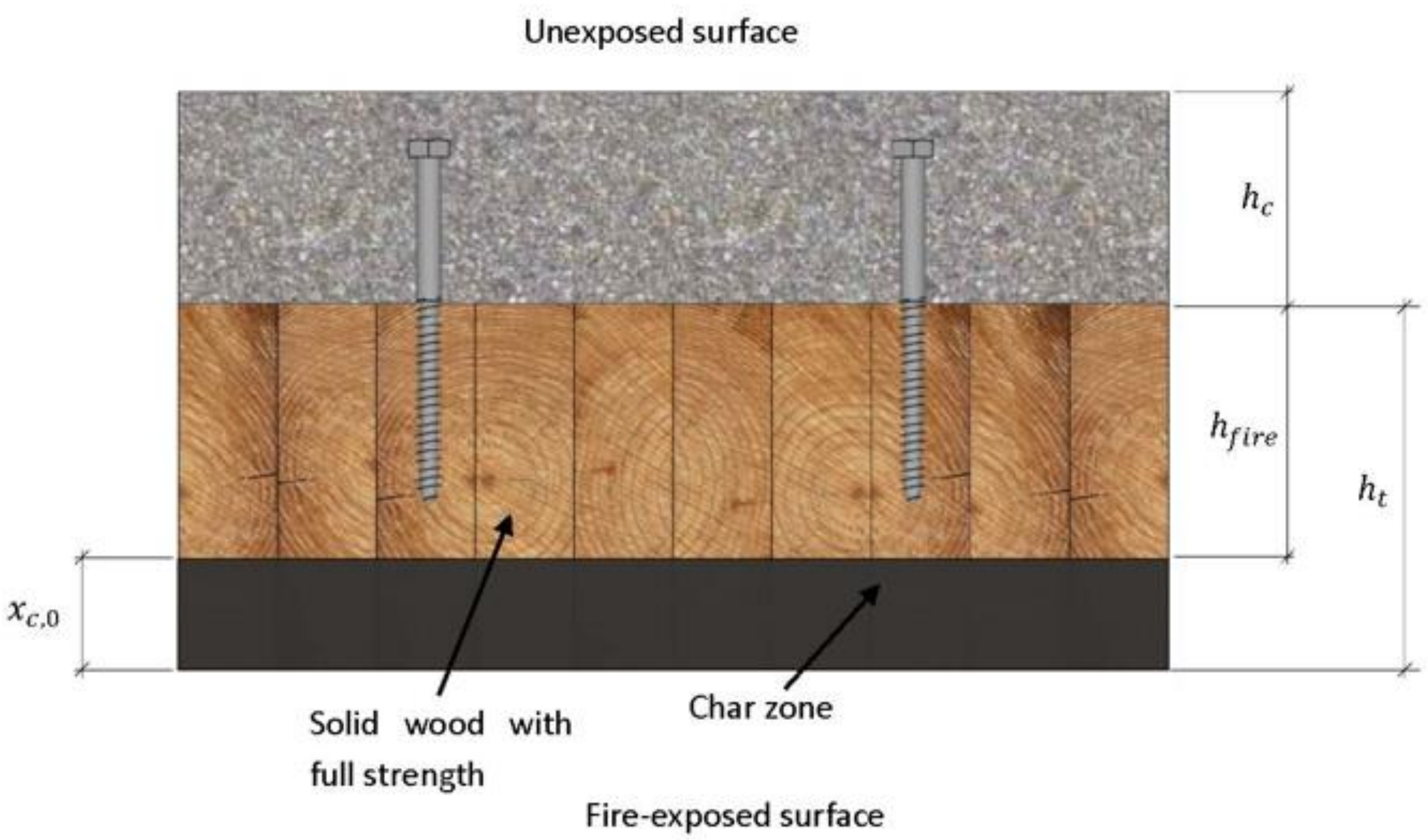


FIRE RESISTANCE

FIRE RESISTANCE

- Floors were exposed to fire from underneath

$$x_{c,0} = \beta_0 + x_t$$



	NLT-Concrete	CLT-Concrete	LVL-Concrete
Shear connector	Truss plates	Self-tapping screws	Lag screws
Test failure time (min)	>214	214	191
Predicted failure time (min)	247	198	165

FIRE RESISTANCE

- Results from the fire testing shows:
  - Remaining strength of the floor can be estimated by reducing the section of the wood according to the charring rate as long as the shear connector is not exposed (follow Annex B of CSA-O86)
  - Once exposed, it remains to the engineer judgment of record to adapt the properties of the connector
  - For dowel type and nail plate, it is conservative to reduce their properties proportionally to the remaining depth.

7

Conclusion

Design Guide for Timber-Concrete Composite Floor in Canada

CONCLUSION

- All the methods presented in this presentation are well-explained in the “*Design Guide for Timber-Concrete Composite Floors in Canada*”
- It is the most-complete multi-criteria design guide currently available for timber-concrete composite floors
- The design guide is to be used among other documents to implement timber-concrete composite floors in the next CSA-O86
- Writing the design guide was possible thanks to the financial support of Canadian Forest Services (NRCan), the BC Forest Innovation Investment (FII) and the industry members of FPInnovations

FPInnovations

KEEP IN TOUCH

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